The Seismic Assessment of Existing Buildings

Technical Guidelines for Engineering Assessments

July 2017

nzsee NEW ZEALAND SOCIETY FOR EARTHQUAKE ENGINEERING

Sesoc

NEW ZEALAND GEOTECHNICAL SOCIETY INC

MINISTRY OF BUSINESS, INNOVATION & EMPLOYMENT
HIKINA WHAKATUTUKI

EQC EARTHQUAKE COMMISSION
Kōmihana Rūwhenua

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Assessment Objectives and Principles

Part A

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This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

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1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

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Please go to www.EQ-Assess.org.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.EQ-Assess.org.nz

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A1. Introduction

A1.1 General

The purpose of these engineering assessment guidelines is to assist engineers, building owners, territorial authorities (TAs) and building consent authorities (BCAs) responding to the challenges involved in understanding, managing and, over time, reducing seismic risk for people using existing buildings.

These guidelines provide engineers with the means to assess the seismic behaviour of existing buildings and building parts and to report the assessment results to building owners and agencies responsible for managing these. Such assessments can be for a range of purposes, including general property risk identification, change of use, and alterations.

These guidelines are also an integral part of the EPB methodology produced by the Ministry of Business, Innovation and Employment (MBIE) under section 133AV of the Building Act 2004 to identify earthquake-prone buildings.

This version of the guidelines is the result of an extensive revision by a Project Technical Group and incorporates research, knowledge and experience obtained from the significant New Zealand earthquakes between 2010 and 2016.

These guidelines are in three parts:

- Part A (this Part) provides assessment objectives and core principles to support detailed guidance on the Initial Seismic Assessment (ISA) method in Part B and the significantly more extensive Detailed Seismic Assessment method (DSA) in Part C. Part A also aims to provide an accessible reference for building owners and managers who need to understand the seismic assessment process.
- Part B covers the qualitative ISA method (including the Initial Evaluation Procedure, or IEP), which enables a broad indication of the earthquake rating of a building. It guides an engineer who is developing a holistic view of a building's structural weaknesses and assigning a qualitative earthquake rating to the building.
- Part C describes the more extensive, quantitative DSA method, which provides a more comprehensive assessment of the likely earthquake rating of a building.

An **Initial Seismic Assessment** procedure using Parts A and B together is essentially a qualitative procedure that observes building attributes, uses these to develop a holistic understanding of how the building would respond to an earthquake and provides an initial assessment of its earthquake rating. An ISA may include quantitative assessments of some elements if an engineer considers they are relatively easily carried out to improve the overall assessment.

An ISA is generally the first part of any seismic assessment because it provides a valuable 'first look' at the likely building performance and a valuable benchmark for comparison with buildings of similar age and other characteristics.

A **Detailed Seismic Assessment** using Parts A and C is a quantitative procedure that can take several forms. These have been developed specifically for assessing existing buildings and, it is important to note, are not simply a back calculation of the design process used for new buildings.

A DSA is used to confirm an earthquake rating for a building, particularly when a higher degree of reliability than considered available from a qualitative ISA rating is required. It can also be used to identify retrofit needs and provide a benchmark for proposed upgrading strategies to be tested against.

A1.2 Scope

These guidelines are specifically for the seismic assessment of existing buildings and apply to buildings of all eras and of all construction types and materials. They are also intended to be used for assessing existing building construction that is included in an upgrade of an existing building (e.g. seismic retrofit or alterations generally), or where a change of use is intended.

The assessment methods and criteria in these guidelines are not intended for use when designing new buildings (e.g. as justification for Building Code compliance), or for the design of new elements within an existing building that is being altered (e.g. retrofitted).

Note:

These guidelines are also not intended to be specifically applied to bridges, towers, masts, retaining walls, or building contents.

Although not intended for these types of structure, many of the approaches outlined and criteria presented may be helpful for this purpose if suitably adapted.

A1.3 Objectives

The objective of these guidelines is to provide engineers with a framework for assessing an existing building and the associated technical methods to:

- lead engineers toward understanding how that building might perform across a range of ground shaking levels
- provide an earthquake rating for the building based on the minimum expectations and requirements for a new building and with a level of conservatism that is appropriate for the level of detail available
- provide a level of assessment that is appropriate for the person commissioning the assessment, including appropriate information to help TAs determine whether or not a building is earthquake prone under the Building Act if the assessment is for that purpose, and
- produce consistent assessments by different engineers when based on the same information and briefing.

These guidelines aim to provide engineers with the communication tools to:

- effectively and consistently communicate the outcomes of assessments, and
- enable building owners to understand and be able to improve the seismic safety of their buildings and, where necessary, prioritise any mitigation works.

Note:

The guidelines also support the wider objective of reducing risk for those buildings expected to perform poorly in significant earthquakes, such as work to reduce seismic risk via seismic retrofit or other improvement measures where required. A general overview and key principles associated with seismic improvement are provided in Section A10, with further guidance and examples to be provided in separate documentation.

A1.4 Regulatory Interface

As noted above, this version of the engineering assessment guidelines forms an integral part of the framework for managing earthquake-prone buildings under the Building Act 2004 (as amended by the Building (Earthquake-prone Buildings) Amendment Act 2016), and also for other aspects of the Building Act relevant for existing buildings including change of use and alterations.

The earthquake-prone building framework includes the following interdependent components:

- the Building Act, which contains earthquake-prone building provisions
- associated regulations which define ultimate capacity, establish earthquake rating categories, prescribe the form of earthquake-prone building notices, and prescribe criteria whereby requirements to carry out seismic work to earthquake-prone buildings may be either exempted or completed earlier because of substantial alterations
- the EPB methodology, which describes how TAs identify potentially earthquake-prone buildings, how these are to be assessed, and how decisions about earthquake-prone buildings and their ratings are made by TAs, and
- these guidelines, which provide the technical means of meeting the requirements for engineering assessments undertaken for potentially earthquake-prone buildings in accordance with the EPB methodology.

Section B1 of the Building Code provides the reference point for the performance standard required for seismic assessments in general. Refer to Section A3.2 for further information.

For more detail on the way in which these guidelines are intended to be used in conjunction with the Building Act refer to Section A5.

Note:

It is expected that an engineer will have available, and be familiar with, the latest versions of the Building Act, associated regulations and the EPB methodology when completing a seismic assessment in accordance with these guidelines. Although these documents relate to the assessment for potentially earthquake-prone buildings the principles captured within them are applied to seismic assessments generally. This is discussed in further detail in Section A5.1.

A1.5 Requirements for Engineers Undertaking Assessments

All seismic assessments are expected to be undertaken by experienced engineers with considerable knowledge of how buildings respond to earthquakes, as well as an ability to exercise judgement regarding key attributes and their effects on building seismic behaviour.

These engineers need to develop a holistic understanding of how the building and its elements would perform during an earthquake and ensure that specialised assessments of building elements such as facades, ceilings and building services adequately address how they interact with the building during an earthquake.

It is therefore essential that every assessment is carried out under the direction of a New Zealand Chartered Professional Engineer (CPEng), or equivalent, who:

- has sufficient relevant experience in the design and evaluation of buildings for earthquake effects to exercise the degree of judgement required, and
- has specific training in the objectives of and processes involved in the assessment procedures contained in these guidelines.

When independent review is called for, the requirements outlined above for the assessment itself should also apply for the engineer overseeing the review.

The EPB methodology specifies qualification requirements for completing engineering assessments as part of the process to determine earthquake-prone status.

Note:

The requirement for high levels of judgement when establishing an earthquake rating from the ISA process cannot be understated, and is discussed further in Part B.

A1.6 Definitions and Acronyms

(New Zealand) Building Code	Section B1 of the New Zealand Building Code (Schedule 1 to the Building Regulations 1992)
Building Element	Any structural or non-structural component and assembly incorporated into or associated with a building. Included are fixtures, services, drains, permanent mechanical installations for access, glazing, partitions, ceilings and temporary supports (from the Building Code).
CERC	Canterbury Earthquake Royal Commission (of Enquiry)
Critical Structural Weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA, all structural weaknesses are considered to be potential critical structural weaknesses.
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part A and Part C of these guidelines
Earthquake-prone Building (EPB)	Has the meaning defined in section 133AB of the Building Act 2004, and explained in Section A5.1.1 of these guidelines.
Earthquake rating	The rating given to a building as a whole to indicate the seismic standard achieved in regard to human life safety compared with the minimum seismic standard required of a similar new building on the same site. Expressed in terms of percentage of new building standard achieved (XXX*%NBS*). The earthquake rating for a building as a whole takes account of, and may be governed by, the earthquake scores for individual building elements. For earthquake-prone buildings earthquake rating has the meaning defined in section 133AC of the Building Act 2004.
Earthquake Risk Building (ERB)	A building that falls below the threshold for acceptable seismic risk, as recommended by NZSEE (i.e. <67*%NBS* or two thirds new building standard)
Earthquake score	The score given to an individual aspect of the building (member/element/non-structural element/foundation soils) to indicate the seismic standard achieved in regard to human life safety compared with the minimum seismic standard required for the same aspect in a similar new building on the same site. Expressed in terms of percentage of new building standard achieved (XXX*%NBS*). The aspect with the lowest earthquake score is the CSW and this score will represent the earthquake rating for the building.
Existing building	A building that is complete and ready for use, i.e. a code compliance certificate is able to be issued
Existing members/ elements	Members/elements that are part of an existing building and not defined as new members/elements as defined below
Importance Level (IL)	Categorisation defined in the New Zealand Loadings Standard, AS/NZS 1170.0:2002 used to define the ULS shaking for a new building based on the consequences of failure. A critical aspect in determining new building standard.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part A and Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.
Moderate earthquake (shaking demand)	Has the meaning defined in the Building (Specified Systems, Change the Use, and Earthquake-prone Buildings) Regulations 2005 (as amended).
New Building Standard (NBS)	Intended to reflect the expected seismic performance of a building relative to the minimum life safety standard required for a similar new building on the same site by Clause B1 of the New Zealand Building Code.

New building	A building is considered to be a new building until all of it is complete and ready for use (i.e. a code compliance certificate is able to be issued)
New members/ elements	Members/elements to be added to an existing building. Members/ elements are considered to be new until an alteration involving their addition is complete and a code compliance certificate has been issued.
Non-structural element	An element within the building that is not considered to be part of either the primary or secondary structure
Part (of a building)	An individual member or element or section of a building's structure (as distinct from the building structure as a whole), or a non-structural element. Refer also to the EPB methodology for the extent to which parts of buildings must be considered in assessments of potentially earthquake-prone buildings.
Primary gravity structure	Portion of the main building structural system identified as carrying the gravity loads through to the ground. Also required to carry vertical earthquake induced loads through to the ground. May also be part of the primary lateral structure.
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. It may also be part of the primary gravity structure.
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects, this is determined using probable material strengths and a strength reduction factor set at 1. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed or calculated for design.
Probable material strength	The expected or estimated mean material strength. For geotechnical issues assessed in accordance with these guidelines it is typically the soil properties that would be used to calculate the ultimate geotechnical strength/resistance assumed for design.
Secondary structural element	A structural element that is not part of the primary structure
Secondary structure	Portion of the structure that is not part of either the primary lateral or primary gravity structural systems but nevertheless is required to transfer inertial and gravity loads for which assessment/design by a structural engineer would be expected. Included are pre-cast concrete panels, curtain wall framing systems, stairs and supports for significant building services items.
Serviceability limit state (SLS)	Limit state defined in the New Zealand loadings standard NZS 1170.0:2002
Severe structural weakness (SSW)	A defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge. For an ISA, potential SSWs are expected to be noted when identified, and may extend to issues that require Detailed Seismic Assessment before they can be removed from consideration.
Significant life safety hazard	As described in Section A3.1.1.
Space Class	Classification of a particular space based on functional purpose and hazard
SSNS	Secondary structural and non-structural
Structural weakness (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100*%NBS*. An aspect of the building structure scoring less than 100*%NBS* but greater than or equal to 67*%NBS* is still considered to be a structural weakness even though it is considered to represent an acceptable risk.

Ultimate capacity (seismic)	Has the meaning defined in the Building (Specified Systems, Change the Use, and Earthquake-prone Buildings) Regulations 2005 (as amended) and explained in Section 5.1.1 of these guidelines. This definition is applicable for all assessments.
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings
Unreinforced masonry (URM)	A member or element comprising masonry units connected together with mortar and not containing any steel, timber, cane or other reinforcement
(XXX)*%NBS*	The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage. It is the rating given to a building as a whole expressed as a percent of new building standard achieved, based on an assessment of the expected performance of an existing building relative to the minimum that would apply under the Building Code (Schedule 1) to the Building Regulations 1992) to a new building on the same site with respect to life safety. A score for an individual building element is also expressed as a percent of new building standard achieved. This is expected to reflect the degree to which the individual element is expected to perform in earthquake shaking compared with the minimum performance prescribed for the element in Clause B1 of the Building Code (Schedule 1 to the Building Regulations 1992) with respect to life safety. The *%NBS* rating for the building as a whole takes account of, and may be governed by, the scores for individual elements.
(XXX)%ULS shaking demand	Percentage of the ULS shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.

A2. Background

A2.1 Introduction

This version of the guidelines provides a comprehensive revision of the 2006 New Zealand Society for Earthquake Engineering (NZSEE) document - *Assessment and Improvement of Structural Performance of Buildings in Earthquakes* (the 2015 version incorporating Corrigenda 1, 2, 3 and 4).

This section provides background on the development over time of these guidelines to address earthquake risk and earthquake-prone buildings, and the historical role of the NZSEE in producing technical guidance in support of the building legislation and related activities of the time.

A2.2 Earthquake Risk and New Zealand Buildings

The earthquake risk of existing buildings has been recognised for many years. The initial legislation in 1968 (the Municipal Corporations Act) was directed at unreinforced masonry (URM) buildings, whose potential dangers were first apparent in the 1931 Hawke's Bay earthquake. URM construction ceased with the introduction of New Zealand's first earthquake standard in 1935, but large stocks of these buildings remain throughout the country. Action was taken by some TAs via the passage of by-laws in response to the 1968 legislation, and a number of buildings (most notably in Wellington) were strengthened to its requirements and sometimes beyond.

The level of strength below which a building required strengthening or demolition was set at one-half of the 1965 New Zealand Standard (NZS 1900 Chapter 8). While this has always been considered to be a low threshold by structural engineers, it was retained when these provisions were incorporated within the earthquake-prone building provisions of the 1991 Building Act.

The NZSEE first produced guidelines to assist in the assessment and strengthening of URM buildings in the 1970s. In 1986 the guidance was fully revised and published as the 'Red Book', a label that remained attached to subsequent versions.

Valuable information and experience has been brought back from overseas earthquakes via the NZSEE reconnaissance programme, funded by the Earthquake Commission (EQC). Most notably the Northridge (1994) and Kobe (1995) earthquakes brought to light concerns about the adequacy of more recent designs, particularly those constructed in the period of early seismic codes between 1935 and 1976. Most buildings designed before the publication of the 1976 structural loadings standard NZS 4203:1976 and its associated materials codes typically do not have either the level of ductility or appropriate hierarchy of failure (i.e. the principles of capacity design) required by current design standards.

Acknowledging these concerns in 1994, the Building Industry Authority commissioned NZSEE to produce a document setting down the requirements for structural engineers to follow when evaluating and strengthening post-1935 buildings. An initial 1996 draft was

further developed into the 2006 NZSEE Guidelines - *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* to accompany the widened scope of the earthquake-prone building provisions of the Building Act 2004.

The 2006 NZSEE Guidelines, and particularly the Initial Evaluation Procedure (IEP), were widely used both by TAs and structural engineers carrying out assessments and improvement measures for existing buildings.

The Building Act 2004 extended the scope of earthquake-prone buildings from URM to include any building found to be below the required earthquake rating. The threshold was raised to include buildings that would have their ultimate capacity exceeded in a moderate earthquake defined as one-third of shaking defined for the design of a similar new building.

TAs were required to have earthquake-prone building policies stating what their approaches and timetables would be to identify and require action on "earthquake-prone" buildings. Implementation by TAs varied considerably, but each TA had to assess its local earthquake risks in the course of producing a policy. Some TAs had active policies that typically involved screening of commercial buildings to identify those that were potentially earthquake prone (typically using the IEP); others had passive policies that only addressed earthquake-prone buildings when they came to their attention via consent applications. The extent of implementation nationwide was as a result quite variable.

The initial TA policies were due for their 5-year review in June 2011. In July 2010, the then Department of Building and Housing and EQC sponsored a major workshop for TAs, owners, developers and designers to share their experiences of developing and implementing earthquake-prone building policies. The aim was to better inform each TA ahead of the policy update.

However, the Canterbury earthquake sequence, which began with the Darfield earthquake on 4 September 2010, forced other priorities on affected TAs, owners, designers and communities. As a result, the revision of earthquake-prone building policies was put on hold in many locations while the impacts and implications of these earthquake events were considered.

A2.3 Drivers for this Revision

A2.3.1 General

A change to legislation was the key driver for this extensive revision of these engineering assessment guidelines. However, this was the culmination of a demand that began with damage to URM buildings in the 4 September 2010 Darfield earthquake and major damage to buildings across all types and eras in Christchurch and surrounding areas during the 22 February 2011 Christchurch earthquake. The impetus has continued with experiences of the Cook Strait earthquakes in 2013 and the Kaikoura Earthquake in 2016.

The Canterbury earthquake sequence led to an unprecedented number of seismic assessments being undertaken, both within the greater Christchurch region, for regulatory and insurance purposes, and nationally, in response to the heightened awareness of seismic risk. This sequence also provided a significant volume of information about the behaviour of New Zealand buildings.

It is clear from the experiences of the Canterbury earthquake sequence that URM buildings remain a significant challenge. The section of these guidelines covering URM buildings (released in 2015) was revised and expanded to incorporate additional research and to take advantage of the lessons from Canterbury.

When reviewing the factors contributing to the failure of buildings where lives were lost, the Canterbury Earthquakes Royal Commission (CERC) gave extensive consideration to the failure of URM buildings and to the collapse of the two non URM buildings where most lives were lost. Their recommendations and other research into building failures, were taken into account when developing these guidelines.

The Royal Commission recommendations also led to a government review of the earthquake-prone buildings regulatory framework in 2011 and the subsequent legislative change (refer to Section A2.3.2) also contributed to the need to update the technical guidance for engineers.

The number of structural and geotechnical engineers in New Zealand who were experienced in seismic assessments was relatively limited at the time that the Canterbury earthquake sequence began, and this resource quickly became overloaded. Many of the subsequent assessments have been undertaken by engineers with limited seismic assessment experience and little or no formal training in seismic assessment.

The 2006 NZSEE Guidelines focused on pre-1976 concrete and steel multi-storey and URM buildings, and provided little guidance on low-rise buildings generally, and timber structures in particular. It also had no guidance on geotechnical matters or building elements that are not part of a primary structural system.

These gaps, which led to many conservative assessments of low-rise buildings during the period from 2011 to 2014, have been addressed in this revision of the guidelines, along with a desire for greater consistency in assessment and reporting outcomes. It is recognised that improved consistency will require extensive and ongoing education and training of structural and geotechnical engineers conducting assessments.

These new guidelines therefore include consideration of the latest available information, take advantage of the lessons learnt from the Canterbury earthquakes, address the changes to the Building Act 2004 (refer also Sections A2.3.2 and A5), and target the wider areas of need and concern amongst engineers, owners and the public.

Note:

The changes to the Building Act 2004 and associated regulations specifically address earthquake-prone buildings. However, the key principles embodied in the assessment processes for earthquake-prone buildings also apply to all assessments completed in accordance with these guidelines. These include the concepts of:

- focusing on life safety
- earthquake ratings/scores and what should be included in determining the rating
- ultimate (probable) capacity
- relating the demand to that which applies to a new building on the same site
- the inclusion of building parts
- significant life safety hazard.

A2.3.2 Changes to legislation

The Building (Earthquake-prone Buildings) Amendment Act 2016 came into effect in July 2017 and provided another key driver for the revision of these guidelines.

Significant changes to the regulatory framework for identifying and managing earthquake-prone buildings under the Building Act have resulted.

A summary of these changes is provided on the following website:
https://www.building.govt.nz/managing-buildings/managing-earthquake-prone-buildings/.

What this means for engineers undertaking assessments of existing buildings for the purpose of identifying whether or not they are earthquake prone is outlined in Section A5 of these guidelines.

A3. Underlying Principles

A3.1 Life Safety Focus

These guidelines focus the assessment on life safety issues as the primary objective. This means that the earthquake scores or rating are based primarily on life safety considerations rather than damage to the building or its contents unless this might lead to damage to adjacent property. The earthquake rating assigned is, therefore, not reflective of serviceability performance and the reporting should warn of this.

There are two general forms of life safety hazard to consider; when the ultimate capacity of the building, a section of the building or a primary structural element is exceeded to the extent that a significant life safety issue arises, or when a falling secondary structural or non-structural (SSNS) building element poses a significant life safety hazard.

Determining when a significant life safety hazard is developed is an important aspect of assessments carried out in accordance with these guidelines. This is discussed in Section A3.1.1.

For the evaluation of earthquake-prone status it is the TA that needs to make the final decision as to whether or not a significant life safety issue exists. The EPB methodology sets out the process requirements to do this. Further information on respective roles in determining a building earthquake prone are explained in Section A5 of these guidelines.

Note:

Health and Safety considerations associated with building elements in general and contents are outside the scope of these guidelines.

The onset and progression of building damage, other than to the extent that it can affect adjacent property and the ability to egress a building, is not a primary consideration in the approaches outlined in these guidelines. Assessment of behaviour at serviceability limit state levels of loading is therefore not expected in the assessment process outlined. This is a significant difference to new building design, where the application of the serviceability limit state requirements is intended to limit damage at moderate levels of earthquake shaking and therefore often determines the minimum strength of a new element or of a new building as a whole.

Often building owners and occupiers may also be interested in the damage potential to the building, including structural and non-structural building elements, and how this might affect business continuity and economic considerations. These aspects are beyond the scope of these guidelines and the guidance provided will not necessarily address them.

A3.1.1 Significant life safety hazard

A significant life safety hazard is an unavoidable danger that a number of people are exposed to.

A significant life safety hazard, as distinct from a life safety hazard, is intended to preclude from consideration building elements that are of insufficient size to constitute a life safety

hazard of reasonable extent. Such items may include relatively small volumes of falling debris (e.g. individual bricks in an URM building or spalling of concrete in a reinforced concrete building), or small light weight items or items that fall from a relatively low height.

Failure of a building or building section as a whole (leading to collapse) is considered to be a significant life safety hazard but failure of individual members/elements in the primary structure will only constitute a significant life safety hazard, when considered individually, if their failure causes them to fall.

A significant life safety hazard can also result from the loss of gravity load support of a member/element of the secondary structure, or of the supporting ground, or of non-structural items that would reasonably affect a number of people.

Elements considered to represent a significant life safety hazard if they were to fall are set out in Table A4.1.

Note:

The life safety hazard exposure is only defined qualitatively. It would be very difficult to quantify this numerically in order to provide a uniform exposure to every possible danger, number of people exposed to it, and their exposure duration. The same difficulty applies to quantifying the number of people exposed.

A danger from a significant life safety hazard can be considered to be avoidable if furniture is present that would provide protection as part of the "drop, cover, hold" mitigation plan.

A3.1.2 Importance Level 4 buildings

The focus of seismic assessments using this document is on the life safety of those occupying and immediately outside the building. Therefore earthquake assessment of an Importance Level 4 (IL4) building (i.e. one with critical post-disaster functions) is expected to consider the enhanced ultimate limit state (ULS) requirement resulting from the higher importance level, but it is not expected to consider how earthquake-induced damage would affect operational requirements.

How damage would affect the ability of an IL4 building to continue to function in the post-disaster period is, however, an important consideration for the intended functional purpose of the building – i.e. serving community needs at a time of crisis. While this may be a life safety consideration in its broadest sense, this is not intended to be part of the decision on whether or not a building is earthquake prone or of its earthquake rating.

Note:

The serviceability required to provide confidence that an existing IL4 building will be able to maintain operational continuity (i.e. SLS2) may be satisfied by simply assessing behaviour at an appropriate level and using judgement to determine what the outcomes may be for usability.

Notwithstanding the focus on life safety, it is recommended that an IL4 building should either attain a 67*%NBS* (IL4) rating as a minimum and fully satisfy SLS2 requirements, or be re-designated.

For an IL4 building not to be found to be earthquake prone it is only necessary for it to attain a 34*%NBS* (IL4) rating as a minimum. SLS1 or SLS2 requirements are not expected to be met.

It may be necessary to guide owners through the differences between IL2 and IL4 requirements as they relate to the risks to general occupants in order to put earthquake ratings based on IL4 requirements into context.

A3.2 Seismic Performance and the Building Code

A3.2.1 Relationship between these guidelines and the Building Code

The Building Code provides (minimum) structural performance objectives for buildings in clause B1.

To meet the full performance requirements of clause B1 of the Building Code requires consideration of life-safety, amenity and damage to other buildings. The performance requirements are stated in holistic terms, referencing overall performance expectations over the whole building life, and do not relate this to a specific level of earthquake shaking in the case of seismic performance.

As noted in Section A3.1 the primary focus in seismic assessment of existing buildings is on life safety, although damage to other buildings also needs to be considered.

These guidelines use clause B1 of the Building Code to provide the minimum life safety performance standard for seismic assessments and, in particular, for the earthquake scores and ratings. This relationship is depicted in Figure A3.1.

A 100*%NBS* earthquake score or rating for an existing building (refer to Section A6 for the way in which these are calculated) indicates that the minimum life safety standard for earthquakes has been reached.

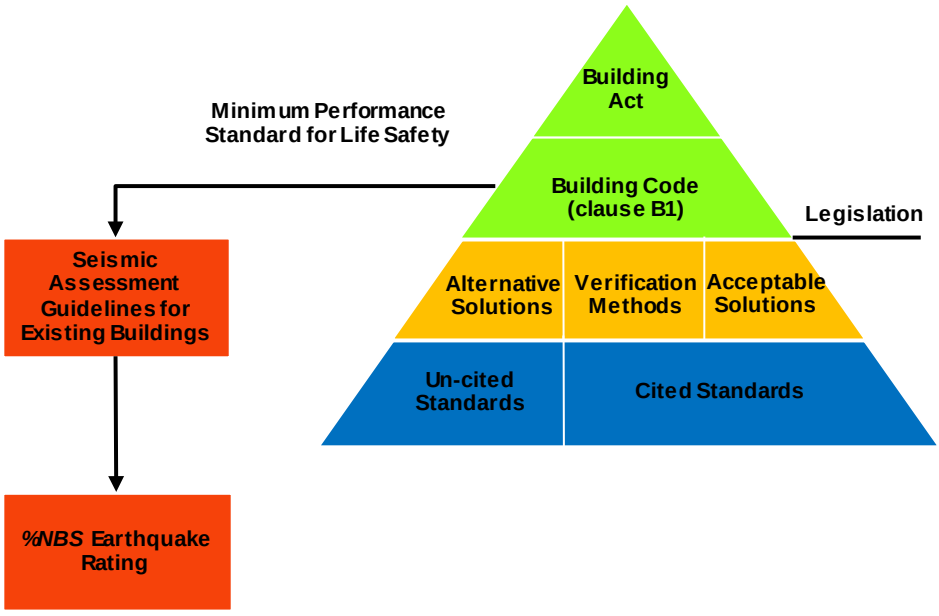


Figure A3.1: Relationship between Clause B1 of the Building Code and Earthquake Rating

An earthquake score or rating is not intended to convey any indication of performance relating to earthquake damage potential. This may be important for some assessments if business interruption or expected damage losses need to be estimated but these considerations are outside the scope of these guidelines.

For earthquake scores and ratings other than 100*%NBS*, a relative level of performance to the minimum level is implied. This is discussed in Section A3.2.4.

A3.2.2 Relationship between the Guidelines and the Building Code Verification Methods

The Building Code defines means of compliance that are intended to deliver the performance objectives set out in clause B1. There are several means of compliance paths given in the Building Code as indicated in Figure A3.1. The most commonly used path for new building design is the deemed to comply verification method B1/VM1.

The design methodologies set out in B1/VM1, including for the detailing of the structural elements, are all calibrated to provide reasonable assurance that the performance required by the Building Code will be met. However, the designer does not need to specifically consider how reliable the actual performance is, nor how much greater the resulting performance is compared with the minimum requirements.

Designers are generally only required to consider two design points or limit states (ultimate limit state (ULS), and serviceability limit state (SLS)), within the deemed to comply provisions of the design standards. The expectation is that if the building meets the prescribed provisions for these limit states the overall performance will meet or exceed the performance objectives in clause B1 of the Building Code taking into account the range and likelihood of earthquake shaking over the design life of the building.

When following this approach it is quite possible that satisfying SLS requirements may govern and improve the life-safety performance and vice versa.

There is the ability to mould a new building into a form that ensures all requirements of B1/VM1 can be met. This ability is not always achievable with an existing building. This is discussed further in Section A3.2.3.

These guidelines draw heavily on the verification methods contained within B1/VM1 (including the cited material Standards and the Standard for earthquake loadings) but recognise that it is not always necessary to meet all of the requirements of the verification methods to achieve the minimum required seismic performance standard for life safety.

Modifications to reflect that the buildings are existing, the need to better understand the seismic behaviour of the building in relation to minimum performance requirements rather than simply ensuring that every element exceeds minimum requirements, and the focus in these guidelines on life safety are therefore important aspects of the seismic assessment procedures outlined in these guidelines.

To this end it is intended that the verification methods can be used to meet the requirements of these guidelines, but the following modifications should be made:

- use of probable (expected) rather than characteristic material properties

- use of probable rather than dependable member/element capacities. This is achieved by using probable material properties and setting strength reduction factors to 1, and
- allow structural mechanisms to form to the point that a significant life safety hazard can be said to have developed.

Part C of these guidelines sets out how it is expected that these modifications will be carried out and other concessions that are considered acceptable to reflect that the building is existing and the focus is on the minimum life safety standard requirements for seismic actions.

Part C also provides guidance on how to assess the capability of historical details that are present in existing buildings but are no longer no longer considered appropriate for new buildings and therefore are not covered in B1/VM1. These details would be expected to typically rate less than 100*%NBS* although this is not always the case. Very poor detailing in existing buildings still needs to be appropriately rated even though this might be at low levels.

Note:

The degree of compliance with B1/VM1 should not be confused with the degree to which a building meets or does not meet the minimum performance requirements of B1. It is quite possible for a building not to meet the full requirements of B1/VM1 and still meet the minimum life safety performance requirements of B1. It is fundamental to the approach set out in these guidelines that a seismic assessment consider how well a building meets the minimum holistic performance requirements rather than solely the extent to which it satisfies the deemed to comply requirements of the prescribed verification method.

A3.2.3 Differences between existing and new buildings

The main difference between a new building at the end of the design phase and an existing building is that the existing building is a physical entity, whereas a new building, yet to be constructed, only exists in conceptual form. Definitions of existing and new buildings can be found in Section A1.6.

A seismic assessment is based on observations of a physical building. Aspects such as poor construction, poor design and poor integration of secondary structural and non-structural elements, when they are found by inspection to be present, can be explicitly allowed for.

While knowledge of an existing building (e.g. material strengths, hidden details, etc.) will be less than complete, the physical presence of an existing building, and what can be determined from it, is considered to provide a significant advantage from the point of view of understanding every (design and construction) issue over that of a theoretical building defined only by drawings. The drawings always convey the designers' intent rather than the constructed reality.

If a new building is observed to be well conceived, designed and constructed and then assessed in accordance with the guidance within this document after it has been constructed, its earthquake rating might be above the minimum required life safety standard associated with 100*%NBS*. This is to be expected given the underlying difference between the

objectives of design and assessment and the physical reality of an existing building that can be observed and confirmed. This is discussed further in Section A3.2.4.

Note:

Assessing a new building solely off drawings in accordance with these guidelines is not encouraged for similar reasons that comprehensive site inspections are always recommended for an existing building (refer to Section A7.4.2). Also, post-construction assessments by the original designer should be objective and include an appropriate evaluation of how well the design objectives (including any interaction with secondary structural and non-structural items) and construction objectives (including the quality of construction and materials) have been implemented in the actual building.

A3.2.4 Consistent expression of seismic performance

The life safety performance objectives for earthquake stated in clause B1 of the Building Code are not quantified nor are they restricted to consideration of only some levels of earthquake shaking. The intent is that the minimum level of performance is a holistic one, covering all levels of earthquake shaking that the building could be reasonably exposed to.

The ISA and DSA assessment processes outlined in these guidelines indicate the seismic performance/behaviour as an earthquake score or rating. The rating provides a measure of the expected performance from a life safety point of view, compared with the minimum required by the Building Code for new buildings. This is expressed as a percentage of the minimum standard required by the Building Code, or *%NBS*, the derivation of which is discussed in more detail in Section A6.

Note:

The use of *%NBS* to describe the result from all levels of assessment (ISA through to DSA) is deliberate. The rating for the building need only be based on the lowest level of assessment that is warranted for the particular circumstances. The *%NBS* assessed using a full DSA process is expected to be more reliable than one assessed using an ISA, but the latter may be sufficient to provide a result that the engineer is confident reflects the expected building behaviour.

Observations following actual earthquakes show there can be a considerable range of performance for quite similar buildings in reasonable proximity. Therefore, any prediction of performance in earthquakes has considerable uncertainty. This uncertainty arises from both a lack of current knowledge and the inability to predict the considerable variability in how earthquake waves propagate from their source to a building and how the building responds to the shaking. The building response is affected by the complex nature of its structure, involving the interaction of many elements, and the complex nature of the ground on which it sits.

Therefore, it is unreasonable to believe that seismic performance can be predicted in absolute terms, and it always needs to be communicated within a probabilistic framework. However, this is not easily done and is more difficult when a holistic view of performance across all levels of earthquake shaking is required.

Note:

The assessment processes outlined in this document are not intended to provide a means of predicting the actual performance of any particular building at any particular level of earthquake shaking or to quantify the performance across all levels of earthquake shaking. Nor is it intended that the risk be quantified beyond the approximate relative measures provided in Table A3.1.

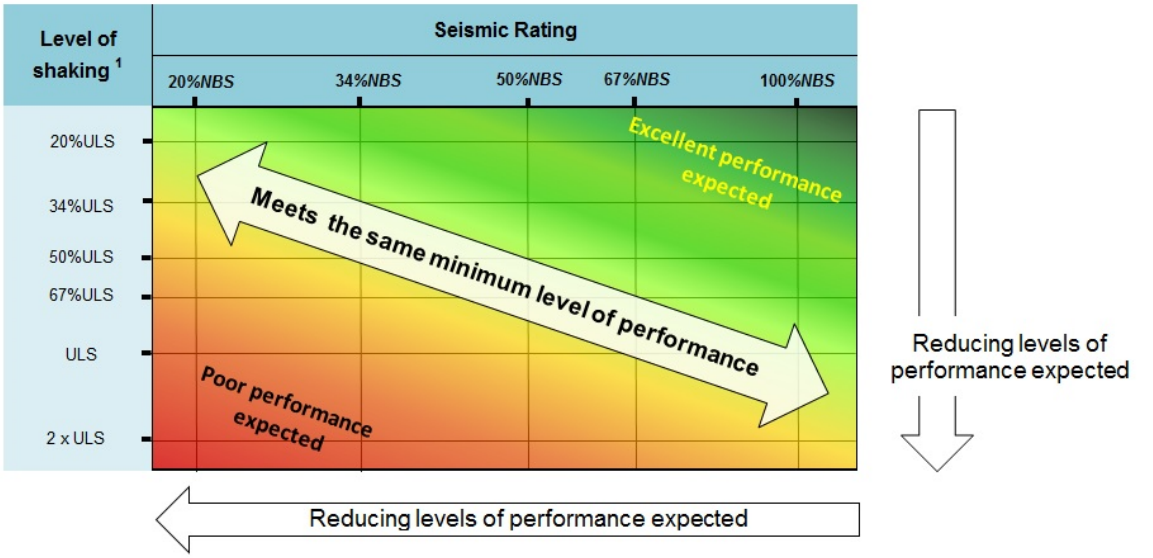
For these reasons, the approach used by these guidelines is to establish how the building is likely to behave in a structural sense and then to relate this to how the building will perform compared with a new building that just meets the minimum standard for life safety in earthquake indicated by clause B1 of the Building Code. By definition, the minimum standard is assumed to provide an acceptable level of performance when considered across all levels of earthquake shaking. Measuring building performance relative to a minimum acceptable level avoids the need to quantify the expected performance.

Therefore, these guidelines assign an earthquake rating to a building in terms of the percentage of the minimum standard that would apply to a similar new building, or XXX*%NBS*. The rating is assigned by comparing various aspects of building behaviour against the minimum standard for a similar new building (expressed in terms of the ratio of the strength and available deformation capacity of the building and the shaking demand specified for a similar new building).

Figure A3.2 illustrates how a building is expected to perform for the full range of earthquake shaking for the difference levels of *%NBS* earthquake rating. In essence, a building will perform similarly (from a life safety point of view) to the lowest standard, when subjected to a level of shaking factored by the XXX*%NBS* earthquake rating. For example, a building rated at 34%*NBS*, subjected to shaking at 34% of the design level shaking for an equivalent new building (34%ULS shaking), is expected to perform to at least the same minimum level as a new building subjected to the design level of shaking (100%ULS shaking). Similarly, when subjected to 67%ULS shaking, this building would be expected to perform to at least the same minimum level as a new building subjected to 200% ULS shaking.

If two buildings are different, e.g. in configuration, size, material type, etc., a similar level of performance may not be expected at all levels of shaking factored by the rating for each. However, when a building is compared with a new building, the performance at any level of shaking should be at least the minimum required level when the shaking is factored by *%NBS*.

Earthquake ratings are not expected to be determined for multiple levels of shaking. The assessment procedures typically focus on assessing how the building would respond to the ULS shaking factored by the determined *%NBS* (XXX%ULS shaking demand). Allowances within the assessment process are intended to provide confidence that the building will also meet the minimum life safety performance requirements at other levels of shaking, while recognising that the level of reliability of meeting the required performance will reduce as the shaking increases. Often these allowances are inherent within the general process, but sometimes specific adjustments need to be made.



Notes:

1. The diagram represents the tolerable (minimum) outcomes for buildings of any earthquake rating, at any level of shaking.
2. For any earthquake rating, the expected seismic performance represented on the vertical axis should be achieved over the full range of possible shaking levels.

Figure A3.2: Indicative relationship between seismic performance, earthquake rating and level of shaking

Discussion on how the earthquake rating is to be determined can be found in Section A6, and in Part B for the ISA and Part C for the DSA.

It is essential that the *%NBS* earthquake rating given to a building reflects its expected relative performance. Therefore, a building should not be rated as 100*%NBS* unless there is confidence that it will perform to the minimum level expected of a new building (life safety only) across all levels of shaking. The rating should be reduced until there is confidence that this will be the case.

Ratings of 120 to 150*%NBS* are possible when well configured buildings, designed to meet the current requirements of B1/VM1, are assessed in accordance with these guidelines and reflect a potentially higher life safety performance when compared with a building that just meets the minimum rating of 100*%NBS*. The actual level of performance is not expected to be significantly higher at any particular level of earthquake shaking (as can be observed in Figure A3.2, noting that the scales for *%NBS* and %ULS shaking are not linear in the figure), and well within the overall uncertainties that are contained in the assessment process generally when the likely overall performance of a particular building is considered across all levels of shaking.

Undue focus on these differences is not encouraged, and an earthquake rating presented as >100*%NBS* is recommended, rather than to present a fixed value. The exception is when change of use might be under consideration. Refer also to Section A8.2.

In addition to the *%NBS* earthquake rating, it is recommended that the corresponding seismic 'grade' and relative risk also be indicated to provide context. Table A3.1 outlines the grading system that was developed by NZSEE in 2000 and a relative risk description as it relates to life safety. Also given is an approximate indication of the risk relative to that of a new building.

Table A3.1: Assessment outcomes (potential building status)

Percentage of New Building Standard *(%NBS)*	Alpha rating	Approx. risk relative to a new building	Life-safety risk description
>100	A+	Less than or comparable to	Low risk
80-100	A	1-2 times greater	Low risk
67-79	B	2-5 times greater	Low to Medium risk
34-66	C	5-10 times greater	Medium risk
20 to <34	D	10-25 times greater	High risk
<20	E	25 times greater	Very high risk

The approximate relative risks given are the risk to occupants or to neighbouring buildings relative to a building that just meets the minimum performance standard indicated by clause B1 of the Building Code.

The risk descriptions given can be considered to be relative life safety risks if a large earthquake occurs.

A4. Assessment Process

This section covers:

- differences between the processes of seismic assessment and new building design
- engineering objectives of assessment, and
- required scope (extent) of the assessment, including considerations external to the site, parts of buildings and buildings comprising multiple interconnected structures.

A4.1 Differences between Seismic Assessment and New Building Design

A4.1.1 General

There are distinct differences between the processes used traditionally for design and for assessment. Further, a designer has the ability, within reason, to modify the behaviour of the building structure to support the design assumptions.

Most importantly, designers are generally only required to consider specific design points (e.g. ULS for life-safety), as the deemed to comply provisions of the design standards are intended to ensure that the performance objectives will be achieved across all expected levels of loading.

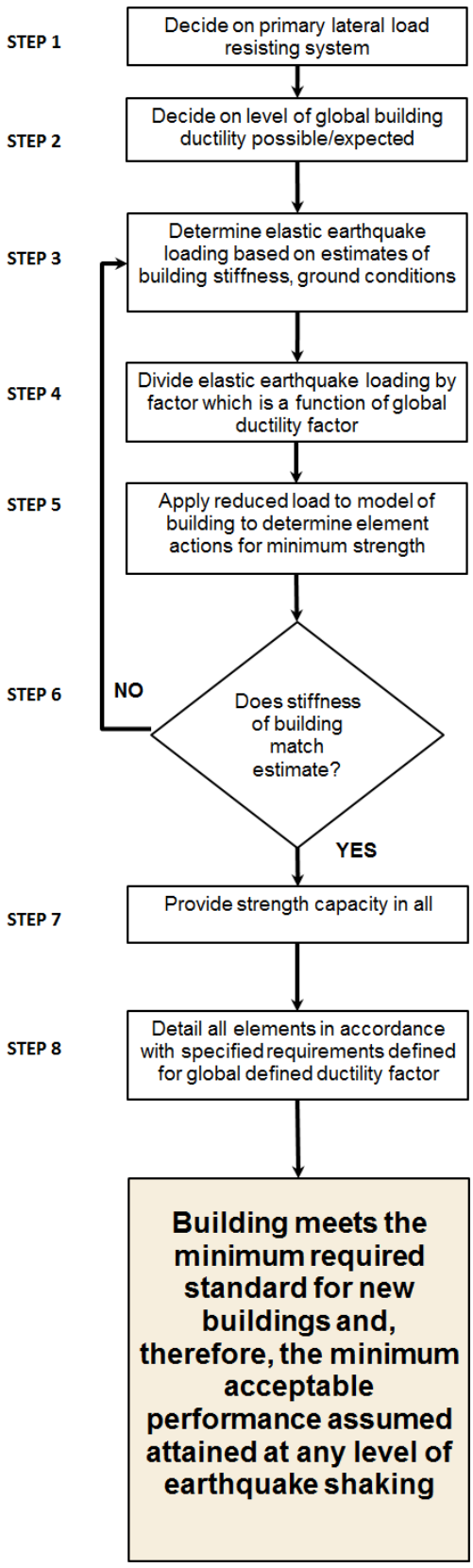
In contrast, engineers assessing existing buildings should be more aware of the range of possible building behaviour across the expected shaking levels, as the same safeguards do not necessarily exist. An existing building's potential behaviour is already determined by the form and detailing of the structure as it was originally designed and constructed, along with such alterations as it may have been subjected to since construction (including the effects of deterioration over time).

A4.1.2 Differences between traditional design and assessment processes

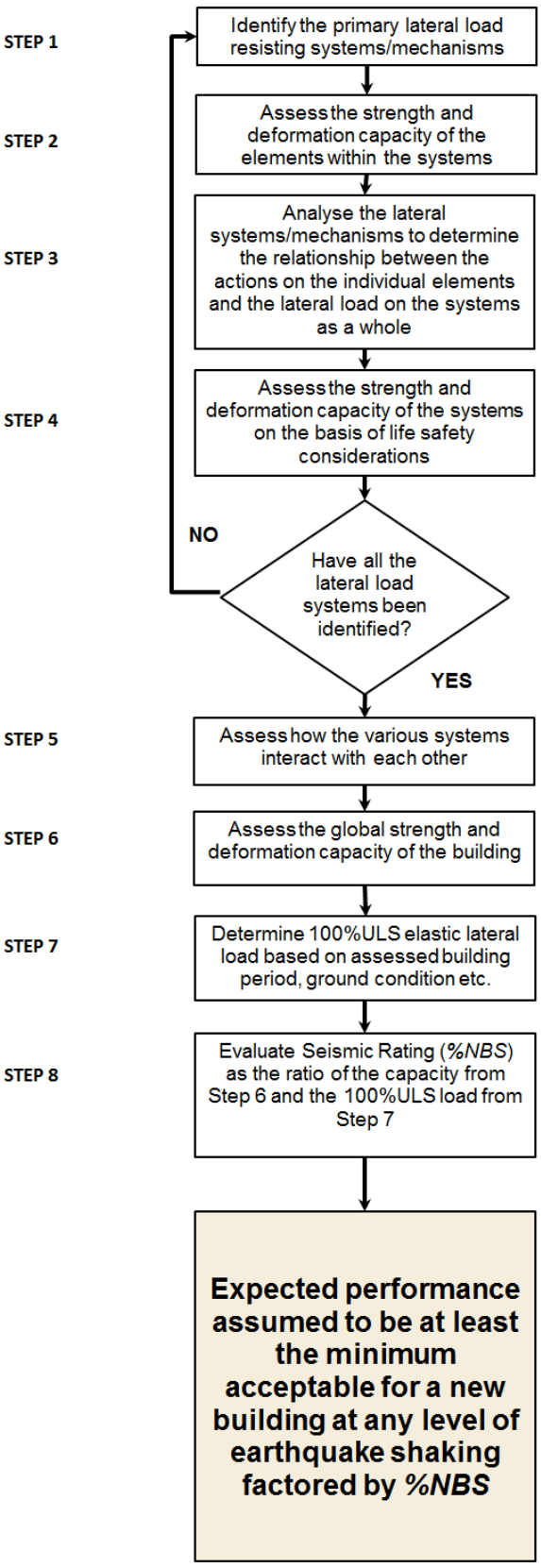
Figure A4.1 maps the key elements of the processes used for new building design and for assessment of existing buildings.

The traditional design process described above for new buildings, where design loads are determined based on a predetermined outcome (required at yield based on a level of ductility that is assumed will be available) and applied to a "model" of the building to determine design actions, and then uses these to proportion the strength capacity of the individual elements and then details the elements for the level of ductility assumed at the outset, is summarised in Figure A4.1(a).

The provisions of both the design loadings and materials standards within B1/VM1 are intended to ensure that the ductile deformation capability of the primary structural elements is sufficient to provide the defined global ductility. This is typically achieved by the process delivering a building form for which it is expected that the energy dissipation will be reasonably well distributed throughout the primary lateral system and that the vertical load carrying capacity of the primary gravity system will be assured at the expected level of lateral deformation.



(a) Traditional design approach



(b) Generic assessment approach

Figure A4.1: Design and assessment approaches compared

However, these fundamental assumptions for the traditional design process are rarely valid when assessing an existing building; unless the structure remains predominantly elastic. The approach to seismic assessment for existing buildings is therefore intended to be fundamentally different to that employed for the design of new buildings. The use of traditional design approaches for assessment can lead to an assessment result that is significantly incorrect if assumptions inherent in the design process are ignored. The result can be either excessively conservative or non-conservative.

The generic assessment process adopted in these guidelines is shown in Figure A4.1(b).

Note:

Practical considerations mean that the assessment process needs to follow a compliance type approach where the assessment process is defined and appropriate acceptance criteria are set, but the intent is to gain a much better understanding of how the building will behave under lateral loading than is necessarily required for design.

The identification of the various systems/mechanisms (Steps 1 to 4), and establishing how they work together (Step 5) are significant iterative parts of the assessment process that have seldom been used in the design of buildings, where reliance is at best placed on a particular mechanism chosen by the designer. In the case of assessment, the features that lead to the development of mechanisms are already present and need to be identified so they can be assessed.

Other differences between the design of new buildings and the assessment of existing buildings as outlined in these guidelines include:

- The focus is on life safety and not on damage prevention. The assessment process addresses the life safety focus by allowing elements or members that are not expected to lose gravity support (and, therefore, fall) once their capacity is exceeded, or if they were to fall would not be likely to lead to a significant life safety hazard, to either be removed from further consideration, or to maintain a residual capacity, with or without a deformation limit as appropriate.

  The consequence is that, in assessment, the assumed system mechanism can be fully developed until the first element that constitutes a life safety hazard reaches its deformation capacity. This is a potentially significant concession compared with new building design.

  Note:

  An assessment that considers all elements but limits the global capacity of the building to the element with the lowest score, without considering whether or not this element is critical from a life safety perspective, will not meet a key principle of these guidelines.

  Assessments carried out using these guidelines are not expected to consider performance against the SLS requirements.

- Design procedures for new buildings aim to deliver buildings that can be expected to meet or exceed the minimum seismic performance objectives set out in Clause B1 of the Building Code for overall life safety risk and acceptable loss of amenity. In contrast, the

assessment process seeks to establish how well an existing building will perform in terms of the minimum performance objectives for earthquakes defined in clause B1 of the Building Code.

- As a result of the process used, many new buildings are expected to exceed these minimum requirements if they are both designed as envisaged by the code writers and constructed as intended by the designer. However, this is not always realised in practice.
- When assessing an existing structure, realistic values for the material properties, particularly strengths, should be used to obtain the best estimate of the strengths and displacements of members, joints and connections. Seismic assessments make use of probable (expected) element capacities (as-built) to recognise that the building physically exists. The justification for using probable rather than nominal capacities for assessment is outlined in Section A4.2, and in more detail in Section C1.
- Mixed ductility structural systems may be present in new building design and, if they do occur, specific provisions are provided to ensure they are correctly incorporated. The experience suggests that designers have not always recognised the presence of mixed ductility systems and therefore these specific provisions have not always been complied with. In contrast, these systems are almost always present in older existing buildings and should be correctly evaluated if a realistic and reasonable assessment of the building's seismic behaviour is to be obtained.

A4.2 Engineering Objectives of Assessment

The main technical objective of any seismic assessment is to come to an understanding of the expected behaviour of the building in earthquakes. There are several important aspects to consider:

- A holistic assessment of seismic performance should consider a wide range of events that the building may be subjected to. Thus, when the standard that a building achieves is reported in *%NBS* terms (refer to Section A6), this implies different levels of reliability of performance across a range of shaking levels that, when considered together, imply that a minimum performance level is achieved. Although engineers may consider only one level of shaking in design, the other levels of shaking are implicitly accounted for in our general design methodologies.
- It is the intent of these guidelines that a full understanding of the behaviour of the building, including an assessment of this behaviour against ULS shaking, and the identification of severe structural weaknesses will also provide confidence that the minimum level of performance for a particular earthquake rating has been achieved overall without necessarily the need to assess at multiple levels of shaking. However, to achieve this objective, the level of experience and understanding of building behaviour in earthquakes needs to be at a significantly higher level for assessment, than required for design.
- The key assessment objective is to arrive at earthquake scores for individual elements with the potential to be significant life safety hazards and an overall earthquake rating for the building based on these.

  Any element that limits the earthquake rating to below 100*%NBS* is referred to as a structural weakness (SW).

  The SW that limits the earthquake rating of the building is referred to as the critical structural weakness (CSW). Particular care is therefore required to identify all possible

SWs. Also, the SWs identified in an ISA assessment are only considered to be potential CSWs until a DSA determines which of them is the CSW.

Note:

If the CSW is addressed by retrofit, the next SW that limits the building's rating becomes the CSW and so on. Ranking the SWs according to their effect on the rating of the building provides an important understanding the sensitivity of the scores and an invaluable basis for any retrofit strategy.

- Assessment should be undertaken to an appropriate level of detail, having due regard to the scale of the building, the potential consequence of its failure and the other work that may be undertaken in parallel with, or as an outcome of, the assessment.
- There needs to be a clear understanding that assessment is not a prediction of the way in which a particular building might perform when subjected to a particular level of earthquake shaking.
- Gaining an understanding of the expected mode of failure and physical consequences of failure is an important consideration for an assessment. These aspects are required to be reported on in the engineering assessment reporting (refer to Section A8.5.2) as part of the EPB methodology but it is considered desirable to include discussion on these for all assessments.

Although there is generally a desire on the part of building owners and occupiers to quickly arrive at an earthquake rating expressed as a single number, by far the most important part of any assessment is to form a view of the expected behaviour of the building. Behaviour encompasses both the elastic and potential inelastic deformation of a building under seismic loading; and its consequential effect on the other elements of the building. It should also include consideration of the potential effects of soil-structure interaction, which may significantly affect the overall building behaviour.

A summary of the key differences between assessment and design was outlined in Section A4.1.

The role of the engineer is to ascertain what the behaviour of the building is expected to be, with regard to these factors, and may need to explicitly address the consequences of failure of elements in more detail than a designer would. This means that an engineer should consider a number of factors, including:

- the materials that the building was constructed with, and how these may vary from what was originally intended
- the designer's intended structural form and behaviour, and how that could have been modified during the actual execution
- the detailing used in design (and as constructed), and how it may modify the intended behaviour
- the changes that may have happened over time and how they may impact on reliability and performance.

In all of the above, the role of the codes and standards of the day are significant, as they would have informed the design and construction process. However, engineers should not simply assume that anything that predates current standards will not perform adequately nor that the codes of the day would necessarily have been interpreted as intended or fully

complied with. One of the most significant issues is that, while a designer may have considered the lateral load resisting and gravity load resisting structures separately, engineers should consider the behaviour of both responding together as one structure. Decisions to exclude elements of the structure from an analysis should be made carefully.

The role of the engineer is therefore as much about investigation as it is about analysis. The engineer should be conscious of the designers' intent but open to consideration of any other factors may influence behaviour and that may not have been within the designers' knowledge, experience or ability to control during construction or subsequent alterations.

A4.3 Extent of the Assessment

A4.3.1 Considerations external to the site

The assessment of an earthquake rating for a building is not intended to extend beyond those aspects that an owner or owners of a building (including across multiple titles) could reasonably be expected to be able to address.

This means that the assessment of the earthquake rating can be restricted to those issues that relate specifically to the site boundaries. Factors that are not intended to be considered include:

- geohazards originating away from the site such as tsunami, rockfall, rolling boulders, slope instability from above (slope instability from below that could undermine the building foundations is intended to be included)
- items falling from adjacent buildings (this being included in the earthquake rating for the adjacent building).

Note:

Issues associated with pounding against adjacent buildings are intended to be included in the earthquake rating for the building. This distinction is made because pounding is an issue that occurs because the subject building is present (in conjunction with the adjacent building) and, while the owner may have no ability to influence the presence or otherwise of the adjacent building, the effects can nevertheless be mitigated by actions the owner of the subject building can take.

The site boundary is typically defined by the legal title boundaries. Determining the site boundaries for a building with multiple interconnected structures will need to include the full extent of the legal boundaries for the building sections and elements that could affect the earthquake rating for the section of interest.

Notwithstanding that these factors may not directly affect the earthquake rating for a building, they could nevertheless affect the overall seismic risk and should therefore be reported alongside the earthquake rating when they have been identified.

A4.3.2 Parts of buildings

The EPB methodology sets the scope of parts of buildings required to be considered in an assessment of a potentially earthquake-prone building. This scope is applicable to all assessments of existing buildings.

A building part requiring consideration in an assessment is an individual building element that would pose a significant life safety hazard (Section A3.1.1) if it is able to:

- lose support or fall, or
- cause another building element to lose support or fall from the building, or
- cause any section of the building to lose support or collapse.

The provision to consider if a significant life safety hazard exists provides for mitigation measures to be taken into account. While this reduces the scope of building elements that are likely to pose significant life safety hazards, it requires more information to be taken into account. For example, the most common form of mitigation will be furniture that the building occupants can reasonably expect to be able to shelter beneath as part of the national 'drop, cover and hold' Civil Defence Emergency Management advice.

Note:

Mitigation could also be provided using redundant and ductile connections (tethers) to attach the building element to the structure.

The types of building element and situations that are most likely to pose a significant life safety hazard and need to be included in engineering assessments are given in Table A4.1. This refers to five classes of floor spaces that are defined in Table A4.2.

Contents are not considered as part of the assessment of an earthquake rating.

Table A4.1: Parts that pose a significant life safety hazard

Parts that could pose a life safety hazard	Expected to be a significant life safety hazard	
	Yes	**No**
Vertical or horizontal cantilevering elements	Heavy (>25kg) elements (e.g. URM parapets, chimneys, cantilevering concrete beams, canopies)	Lightweight ($\leq$25 kg) elements
Light cladding systems including curtain walls	Support frames that could fall into space class I	Lightweight cladding elements, including individual glazing elements (moderate sized panes of glass)
External cladding elements, including connections	Heavy elements (e.g. precast concrete panels (not integral with the primary structure) above space class I	Lightweight cladding elements
Appendages and ornamentation	Heavy elements above space class I	All Other
Stairs and their associated supports	Above or supporting space classes I, II and III and Above space classes IV	All other stairs
Ceiling systems and/or ceiling grids	Heavy ceiling systems above space classes I, II, III or IV Ceiling systems above space classes I, II, and IV Unrestrained ceiling tiles >7.5 kg/tile above space class I, II, III and IV	Unrestrained ceiling tiles < 7.5 kg/tile Conventional lightweight ceiling grid systems above space class III Ceiling grids/tiles within space class V
Partitions and walls	> 25 kg/m^2 (e.g. blockwork or clay tiles) bordering space classes I, II, III or IV	$\leq$ 25 kg/m^2 (e.g. conventional timber or light gauge steel framed partitions and walls) Any partitions and walls within space class V
Signs or billboards	Large (> 25 m^2) or heavy with a fall height greater than 3 m onto space classes I, II, III and IV	All other
Plant and tanks with non-hazardous contents	Large/heavy elements where failure of restraints/supports could lead to the item falling onto building occupants Does not include the integrity of the item itself	All other
Vessels containing hazardous materials	Where spillage would pose a health hazard for building occupants or those within 3 m of its perimeter	Small robust containers that are unlikely to spill their contents
Storage racking systems	Heavy systems in generally occupied spaces	Lightweight systems Systems in space class V
In-ceiling building services	Only when failure of one building element could lead to failure of another that would pose a significant life safety hazard (e.g. a heavy ceiling over a class II area).	Lighting, heating, ventilation and air conditioning ducts and equipment

Explanatory Notes:

A heavy element has a mass >25 kg

A lightweight element has a mass $\leq$ 25 kg

A large element has an area >25 m^2

Table A4.2: Building space classes and their functional purposes

Space class	Functional purpose and hazard
I	Spaces that function as egress paths, provide emergency assembly or access, are public property or where people regularly congregate
II	Open spaces with minimal furniture
III	Furnished spaces where the furniture can reasonably be expected to provide shelter during an earthquake
IV	All spaces beneath overhead building elements that are heavier or could fall further than normal furniture can provide shelter from
V	Service or storage areas such as plant rooms or warehouses that are not expected to be occupied during an earthquake

Guidance on approaches to assessing SSNS building elements as part of an ISA and a DSA are included within Part B and Section C10 respectively.

A4.3.3 Buildings with interconnected structures

For the purposes of seismic assessment the boundaries of a building extend to include all structurally interconnected structural forms/systems. The structural interconnection may be in the form of common members/elements/structural systems providing vertical or lateral support. In some cases the building or the boundaries of the individual structural systems may extend beyond legal (title) property boundaries.

Note:

Separate structures that share a common foundation (as the only form of structural interconnection), and are not otherwise reliant on the interconnection for either lateral or gravity support, can be considered as separate buildings.

The approach taken to assessing a building comprising multiple interconnected structures crossing multiple titles is as follows:

- Ascertain the extent of the building perimeter by considering all structurally interconnected structural systems.
- Identify all separate primary lateral structural systems in the building.
- Identify any primary structural elements that cross between different structural systems.
- Identify any SSNS elements that could be considered to be a significant life safety hazard and their location relative to the property legal title boundaries.
- Determine the earthquake score for each structural system, each primary element crossing between each structural system and identified SSNS elements that are a significant life safety hazard.

For a building within one property boundary the earthquake rating is the minimum earthquake score determined in the final bullet item above.

The earthquake rating for a part of a building within particular property title boundaries, where the building extends beyond these boundaries, is the minimum of:

- the earthquake scores determined for all structural systems within the building
- any primary structural elements that cross between structural systems, and
- the scores determined for any SSNS elements that are considered to be a significant life safety hazard for the part/section of the building which is of interest.

Note:

In the case of a building extending beyond legal property boundaries a warning should be provided regarding the implications of future removal of any structural system(s) or supports of SSNS elements beyond the boundaries.

Adjacent structures that have no or minimal separation but are otherwise not structurally connected can be considered as separate buildings. For such situations the effects of pounding between the buildings will need to be considered. For separate URM buildings in a row this may require special consideration for the buildings on the ends of the row.

Note:

The approach outlined above for these often difficult and complex situations is a pragmatic one that is clear to apply.

However, application in the manner suggested will lead to situations where structurally interconnected structures (i.e. buildings) with different legal titles within a city or town block will all be provided the same earthquake rating based on the lowest score.

In such situations, an assessment of a part of the building for one of the owners may only be able to result in a *%NBS* score for that structure (considering the impact of adjacent structures), unless there is knowledge of all of the interconnected structures.

The engineer will need to be prepared to explain to clients/stakeholders what the impact of any SWs in building as a whole will have on the particular part that may be of interest and the reason why it is only possible to provide a score rather than a rating. For many situations, the difference will not be significant and will have no practical impact on the way in which that part of the building may be used.

A5. Assessments for Building Regulatory Purposes

These guidelines support seismic assessments undertaken for building regulatory requirements set by the Building Act 2004, including providing information relevant to determining the earthquake-prone status, change of use and also when evaluating alterations.

The following sections set out specific requirements for seismic assessments completed to meet legislative requirements.

A5.1 Seismic Assessments for Earthquake-prone Building Purposes

The Building Act 2004 sets out the framework for identifying and managing earthquake-prone buildings including that:

- TAs must identify potentially earthquake-prone buildings
- building owners of potentially earthquake-prone buildings must commission an engineering assessment
- Tas must use this information to determine whether or not a building or part is earthquake prone.

The EPB methodology (see Section A5.1.2 below) supports the Building Act by establishing process requirements to undertake these roles.

Engineers undertaking a seismic assessment of a potentially earthquake-prone building must use these guidelines to meet the requirements of the EPB methodology. The resulting information produced by a seismic assessment for this purpose assists Tas in determining whether a building or part of a building is earthquake prone or not.

Engineers should familiarise themselves with the entire EPB methodology to understand how the TA will use their reports to make this decision and to assign the earthquake rating category on the earthquake-prone building notice.

A5.1.1 Meaning of earthquake-prone building

Section 133AB of the Building Act 2004 sets out the meaning of earthquake-prone building. The definition was revised in the Building (Earthquake-prone Buildings) Amendment Act 2016.

The definition now:

- makes it clear that a building can be earthquake prone by virtue of its parts
- makes it clear that a building must be assessed for its expected performance and possible consequence
- ties the meaning to a moderate earthquake, i.e. the earthquake shaking used to design a new building at that site if it were designed on the commencement date.

The definition of an earthquake-prone building contained within the Building Act is:

> **133AB Meaning of earthquake-prone building**
>
> (1) A building or a part of a building is **earthquake prone** if, having regard to the condition of the building or part and to the ground on which the building is built, and because of the construction of the building or part, -
>
> (a) the building or part will have its ultimate capacity exceeded in a moderate earthquake, and
>
> (b) if the building or part were to collapse, the collapse would be likely to cause –
>
> (i) injury or death to persons in or near the building or on any other property, or
>
> (ii) damage to any other property.
>
> (2) Whether a building or part of a building is earthquake prone is determined by the territorial authority in whose district the building is situated: *see* section 133AK.
>
> (3) For the purpose of subsection (1)(a), **ultimate capacity** and **moderate earthquake** have the meanings given to them by regulations

As covered in 133AB(3), to assist with application of this definition, both ultimate capacity and moderate earthquake are terms defined in the Building (Specified Systems, Change the Use, and Earthquake-prone Buildings) Regulations 2005 (as amended).

These regulations define **ultimate capacity** as:

> The probable capacity to withstand earthquake actions and maintain gravity load support assessed by reference to the building as a whole and its individual elements or parts.

While defined for the purposes of undertaking engineering assessments of potentially earthquake-prone buildings, the definition for ultimate capacity is used for all types of seismic assessment carried out using these guidelines as described in Section A6.3.

These regulations define **moderate earthquake** as:

> In relation to a building, an earthquake that would generate shaking at the site of the building that is of the same duration as, but that is one-third as strong as, the earthquake shaking (determined by normal measures of acceleration, velocity, and displacement) that would be used to design a new building at that site if it were designed on 1 July 2017.

The important change from the previous definition is the fixing of the date and therefore the version of the earthquake design actions standard that should be used for building assessments.

Note:

This change leads to an important potential difference between seismic assessments carried out to establish earthquake-prone status and assessments for general purposes. The earthquake shaking demand to establish the earthquake rating for earthquake-prone purposes is that which applied on the date that Subpart 6A of Part 2 of the Building Act,

2004, and associated amendments came into force. For assessments for other purposes the demand is that which applies at the time the assessment is completed.

This difference means that the *%NBS* scores determined in accordance with the EPB methodology may differ from those calculated for general assessments if the earthquake loads defined for the ULS in NZS 1170.5:2004 have been amended after the Building Act amendments come into force.

In such cases, it may be necessary to quote both ratings depending on the scope of the assessment.

A5.1.2 EPB methodology

The EPB methodology is set by the Chief Executive of MBIE in accordance with section 133AV of the Building Act 2004. It provides the operational basis for identifying potentially earthquake-prone buildings, assessing them and making decisions about whether or not they are earthquake prone.

Specifically, engineers undertaking engineering assessments of potentially earthquake-prone buildings must ensure these are completed in accordance with the requirements set out in the EPB methodology. There are requirements for:

- the qualifications of the engineers carrying out the assessment
- determining whether an ISA or DSA is the appropriate form of assessment
- the technical considerations that the engineering assessment must include and consider
- the contents of the report and summary report that must be supplied to the TA.

Note:

The EPB methodology gives the requirements for an 'engineering assessment' and refers to these guidelines for details of how they are to be carried out.

The technical requirements themselves are given in the relevant sections of this guidance to avoid unnecessarily extending the length of this overview.

It is important to note that the TA determines whether or not the building is earthquake prone in accordance with section 133AB of the Building Act 2004 and is required to assign its earthquake rating. The information contained in an engineering assessment will inform this.

Note:

Engineers should familiarise themselves with the entire EPB methodology to understand how the TA will use their reports to make this decision and to assign the rating. It also includes conditions under which the TA must accept and can reject an engineering assessment.

The technical considerations in the EPB methodology include the requirement to consider parts of buildings as set out in Section A4.3.2.

A5.2 Seismic Assessments for Other Building Regulatory Purposes

A5.2.1 Change of Use

A TA can only approve a change of use under section 115 of the Building Act 2004 when it is satisfied, on reasonable grounds that, in its new use, the building will comply, as nearly as is reasonably practicable, with the structural performance requirements of the Building Code.

The TA may require documentation to be submitted to accompany the owner's application for a change of use, including a seismic assessment. The nature and extent of this assessment, if required, will depend on the nature and implications of the change of use and the particular circumstances.

It is considered that a seismic assessment carried out in accordance with these guidelines, and in particular the earthquake rating determined, should be sufficient to establish the extent to which the building structure meets the life-safety performance requirements of the Building Code.

It may also be necessary to confirm other requirements of the Building Code have been met to the required degree, e.g. for amenity, to fully comply with the requirements of the Building Act.

Note:

The implication of using these guidelines for the assessment of the structural (seismic) status of the building from a life-safety point of view for "change of use" purposes is that it is acceptable to base the structural (seismic) capacity of the building based on probable capacities and other relaxations of B1/VM1 requirements contained within these guidelines.

A5.2.2 Alterations

The basic requirement of section 112 of the Building Act 2004 in terms of structure is that alterations cannot result in the building complying with the Building Code to a lesser extent than before the work (s112(1)(b)).

From a seismic point of view this requires that either the building's seismic capability is not diminished, or it can be shown that the building meets the minimum performance requirements of the Building Code.

It is considered that a seismic assessment carried out in accordance with these guidelines will provide an evaluative tool to help establish that the test under section 112(1)(b) is met. It may also be necessary to confirm other requirements of the Building Code have been met to the required degree, e.g. for amenity, to fully comply with the requirements of the Building Act.

Note:

The implication of using these guidelines for the assessment of the structural status of the building from a life-safety point of view for "alteration" purposes is that it is acceptable to confirm the structural (seismic) capacity of existing building elements based on probable capacities and other relaxations of B1/VM1 requirements contained within these guidelines.

Any new elements to be incorporated into the structure will need to be detailed to the full requirements of the Building Code for the actions resulting from application of the targeted XXX%ULS shaking demand. The intention for the retrofit of an existing building, where new building elements are being added to improve the overall earthquake rating of the building, is indicated in Figure A5.1. Refer also to Sections A10.2.4 and A10.2.5 for general considerations when seismic performance improvement works are being designed.

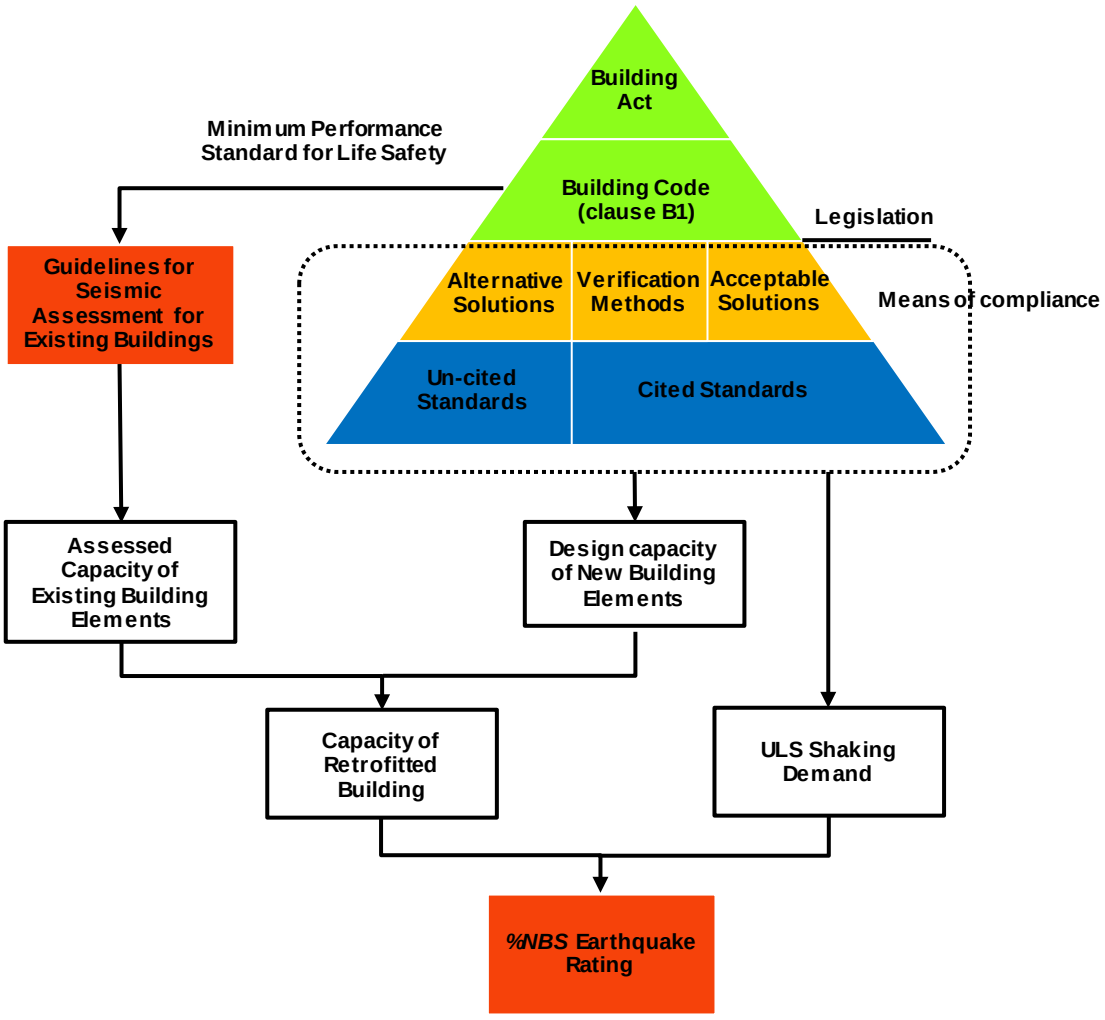


Figure A5.1: Use of Guidelines and Building Code to determine *%NBS* earthquake rating for alterations involving new building elements

Whether a building may be acceptable for alteration, with or without firstly, an engineering assessment and secondly, strengthening, will depend on the particular circumstances.

A6. Earthquake Scores and Rating

A6.1 Introduction

The earthquake rating or score (as appropriate) is intended to provide a measure of the seismic standard for life safety achieved by the building relative to the minimum that would meet the performance objectives set out in clause B1 of the Building Code. The earthquake rating and earthquake scores are expressed as the ratio of the ultimate capacity and the ULS seismic demand, or *%NBS*.

Note:

The intent is that an earthquake score is assigned to individual aspects of the building (these may include sections of the building, individual building elements or specific aspects such as slope stability in geotechnically dominated structures). The earthquake score of the lowest scoring element is the earthquake rating for the building. Therefore a building may have multiple earthquake scores but will have only one earthquake rating.

When establishing the earthquake rating or score, the procedures require consideration of the following in the context of the consequence to life safety:

- ultimate (seismic) capacity of the building as a whole (both strength and deformation)
- expected behaviour of the ground the building is founded on and how this might affect the response of the building
- influence of adjacent buildings (pounding)
- behaviour of elements in the primary gravity structure
- behaviour of secondary structure and non-structural parts.

The other input into the calculation of the earthquake rating or score is the seismic demand or ULS shaking demand.

A6.2 Calculation of *%NBS*

%NBS is obtained by dividing the calculated ultimate capacity (seismic) of the building or part by the ULS seismic demand as shown in the following equation:

$$\%NBS = \frac{\text{Ultimate capacity (seismic)}}{\text{ULS seismic demand}} \times 100$$

where:

Ultimate capacity (seismic) is taken as:

- probable capacity of the primary lateral structure of the building including the impact of geotechnical issues (refer to Section A6.6), or
- probable capacity of structural elements, the failure of which could lead to a significant life safety hazard (refer to Section A6.3), or
- capacity assessed for any identified SSWs (refer to Section A6.6), or

- probable capacity of SSNS elements which would pose a significant life safety hazard (refer to Section A4.3.2)

ULS seismic demand as described in Section C3, including the appropriate value of S_p (the structural performance factor) for the particular aspect under consideration. Refer to Section A6.4 for further discussion.

The earthquake rating will be the minimum value of *%NBS* calculated as above.

Note:

This is essentially the same for both the ISA (typically via the IEP) and the DSA. For the ISA (IEP), *%NBS* for the primary structure is assessed qualitatively against the design requirements that would have applied at the time the building was designed (adjusted for presence of SWs and the presence of secondary and critical non-structural items), whereas for the DSA it is determined quantitatively.

The earthquake rating should always be quoted together with the Importance Level that was assumed to determine the ULS seismic demand. The recommended presentation format, showing the percentage as XXX, the Importance Level as Y, and with "*%NBS"* always italicised, is:

XXX*%NBS* (ILY)

The Importance Level assumed when setting the demand, and therefore the basis for the earthquake rating, is critical to establishing the standard to which the building has been assessed.

A6.3 Ultimate Capacity (Seismic)

Ultimate capacity is a term defined in the Building (Specified Systems, Change the Use, and Earthquake-prone Buildings) Regulations 2005 (as amended), and set out in Section A5.1.1 of these guidelines.

While this definition relates to the use of the term ultimate capacity within the meaning of earthquake-prone building set out in the Building Act, it is applicable for all assessments of existing buildings carried out in accordance with these guidelines.

It is intended that ultimate capacity be calculated for the building as a whole and also for any building elements that are determined to be a significant life safety hazard.

The ultimate strength and deformation capacities are based on probable or expected values.

A6.4 ULS Seismic Demand

For the purposes of a seismic assessment, the ULS seismic demand is the 100%ULS shaking demand determined from the appropriate version of NZS 1170.5.

The Building (Specified Systems, Change the Use, and Earthquake-prone Buildings) Regulations 2005 (as amended) define the term moderate earthquake, which is used to

evaluate potentially earthquake-prone buildings under the Building Act. The definition fixes the date and version of the earthquake design actions standard, and therefore the ULS seismic demand, that must be used for assessments of potentially earthquake-prone buildings under the Building Act.

For assessments undertaken for other purposes, the demand is determined using the current version of the loadings standard applicable at the time of the assessment (rather than commencement of the earthquake-prone building provisions in the Building Act). This means that over time the demand used may differ between assessments to inform whether or not a building is earthquake prone and assessments for other purposes.

The quantification of the seismic demand is required for the DSA and is discussed further in Sections C1 and C3.

A6.5 *%NBS* Threshold for Earthquake-prone Buildings

One of the criteria that the TA use to determine if a building is earthquake prone is that its ultimate capacity will be exceeded in a moderate earthquake (refer to Section A5.1.1).

The moderate earthquake is defined as generating shaking at the site that is of the same duration and one third as strong as that used to design a similar new building. This is equivalent to 34%ULS shaking if the focus is to be on life safety.

Therefore, it follows from the equation given for *%NBS* in Section A6.2, that for the criterion above to be met, the *%NBS* for the building must be less than or equal to 33.3% or less than 34% if rounded.

The threshold can, therefore, be considered to be 34*%NBS*.

The same approach can be taken for a part of a building, i.e. the ratio of its ultimate capacity to the ULS shaking demand that would be used to design a similar part for a new building will be less than 34*%NBS*.

A6.6 Structural Resilience

The *%NBS* earthquake rating must reflect the ability of the building to continue to perform in earthquake shaking beyond the XXX%ULS shaking demand levels (where XXX*%NBS* is the determined earthquake rating). This ability is defined in these guidelines as the available structural resilience.

Structural resilience is necessary to allow a building to meet the overall performance objectives set in the Building Code. These objectives would not be met if the building had a high probability of failure once the XXX%ULS shaking demand levels are exceeded. Structural resilience is inherent in most building systems as observed from actual building performance in earthquakes that exceed XXX%ULS levels of shaking demand.

However, experience indicates there are some systems that have little structural resilience, are susceptible to a sudden reduction in their ability to continue to carry gravity load as the earthquake shaking increases beyond a particular value, and are difficult to quantify based

on current knowledge or inability to analyse. These are referred to in these guidelines as severe structural weaknesses (SSWs). If SSWs are present they require careful assessment and a process that ensures that there is sufficient margin against them causing system failure.

The general criteria for a SSW feature is that it must satisfy **all** of the following conditions:

- **the system has a demonstrated lack of structural resilience** so that there is very little margin between the point of onset of nonlinear behaviour (e.g. cracking of structure or large deformation of soil) and a step-change brittle behaviour of the building that could result in catastrophic collapse, and
- **there would be a severe consequence** if catastrophic collapse occurs. A severe consequence is intended to only be associated with building typologies with potentially large numbers of occupants and where the mode of failure could lead to full collapse, and
- **there are recognised limitations in the analysis and assessment of the behaviour** so that the reliability of the assessment of probable capacity of the expected aspect is low. This could be simply because there is currently considered to be insufficient experimental data or experience to confirm the behaviour to generally accepted levels of reliability.

The currently identified potential SSWs (ISA) and actual SSWs (DSA) are listed in Part B and Section C1 respectively, and cover aspects such as columns and walls in multi-storey buildings with high levels of axial load under dead and live loads, significantly inadequate connections between floor diaphragms and lateral load resisting elements and complex slope failure situations.

The manner in which the effect of the SSWs is to be accounted for is covered in Part B and Part C for an ISA and DSA as appropriate.

A6.7 Geotechnical Considerations

Geotechnical issues are covered in a similar manner to structural weaknesses. To affect the calculation of *%NBS,* the ground behaviour must lead directly to a significant life safety consequence for the building.

Ground conditions influence the behaviour of buildings in several ways, depending on the nature of the ground, the expected building behaviour and the nature of the earthquake. Some of these are discussed below.

The first direct influence is on the seismic actions in the building or its parts, as the soil class is a critical input to the spectral shape factor in NZS 1170.5:2004. For ISAs and relatively simple DSAs, it will generally be suitable to infer the soil classification from local knowledge, surrounding buildings and desktop study if required. For a more complex DSA, where the soil classification could significantly impact on the outcomes, more detailed investigation may be required.

Soil-foundation-structure interaction effects may significantly influence the assessment where there is significant non-linearity, either through the behaviour of the soils, for example in cases involving liquefaction, or through the behaviour of the building itself, for example where foundation rocking occurs.

Nonlinear behaviour in the soil requires careful consideration, but a key question to consider in all cases is whether the non-linearity will impact life safety or just amenity and serviceability. Only life safety concerns that relate to the behaviour of the building need to be addressed in assessing *%NBS*, although in some cases, the brief may include a request to consider serviceability. That is beyond the scope of this document, although some of the guidance provided may be relevant.

The most obvious form of soil non-linearity is liquefaction, but it is important that the impact of liquefaction on building behaviour is considered before embarking on exhaustive geotechnical analysis. The significant settlement that results from widespread liquefaction may not have any significant impact on life safety, especially if the foundations are well connected and when there is an element of toughness in the building superstructure. Conversely, even relatively nominal differential effects may have a significant life safety impact on unreinforced masonry buildings with isolated footings.

Foundation rocking (often by those that were originally designed as fixed base foundations) has often been regarded as the saviour of buildings that may otherwise have been significantly overstressed by larger earthquakes. Rocking has the effect of lengthening the building period and consequently increasing the displacements of the system. In many cases, this will not be critical, but the consequences of the additional displacement should be considered, particularly on the primary gravity structure, which must 'go along for the ride'.

A7. Planning a Seismic Assessment

A7.1 Introduction

This section outlines the steps involved in planning a seismic assessment, which involves working through the steps of briefing, gathering information, carrying out physical inspections and investigation, undertaking initial qualitative assessments followed by quantitative assessments to the extent considered appropriate.

Emphasis is placed on developing a strategy and approach that reflects both the assessment objectives and the nature of the building, taking into account the level of available information.

A7.2 Assessment Procedure

A generalised assessment process is illustrated in Figure A7.1. The steps in the process are summarised in the sections that follow.

A7.3 Briefing – Clarifying Scope and Objectives

Before commencing an assessment, the brief should be clearly understood.

It is important to verify the brief carefully to ensure the client receives everything they require from the assessment process. Accepting a brief from a client is an opportunity to develop an understanding of their needs. Think about:

- What is driving the need for the study? In particular, consider whether potential alterations or change of use requirements may force the evaluation at a higher level than the earthquake prone assessment.
- Does the client wish the study to be limited only to those aspects of the building that require assessment under the earthquake-prone building framework, or do they require the scope expanded to address a broader range of building elements?
- Is the assessment in response to another assessment (e.g. by a TA). If so, does the scope of the proposed assessment address all of the issues that have been raised?
- Are upgrading options to be considered, and if so, what is the performance objective (noting that this is partly about the target loading, partly about the tolerable damage that will be acceptable)? Are there multiple performance objectives?
- Do future insurance requirements have a bearing on the decisions that may need to be taken for the building?
- Does the building have a heritage rating, and/or what are the major heritage features of the building that should be retained?

It is recommended that a reflected brief be prepared and returned to the client for approval before finalising an assessment contract. This reduces the potential for miscommunicating expectations.

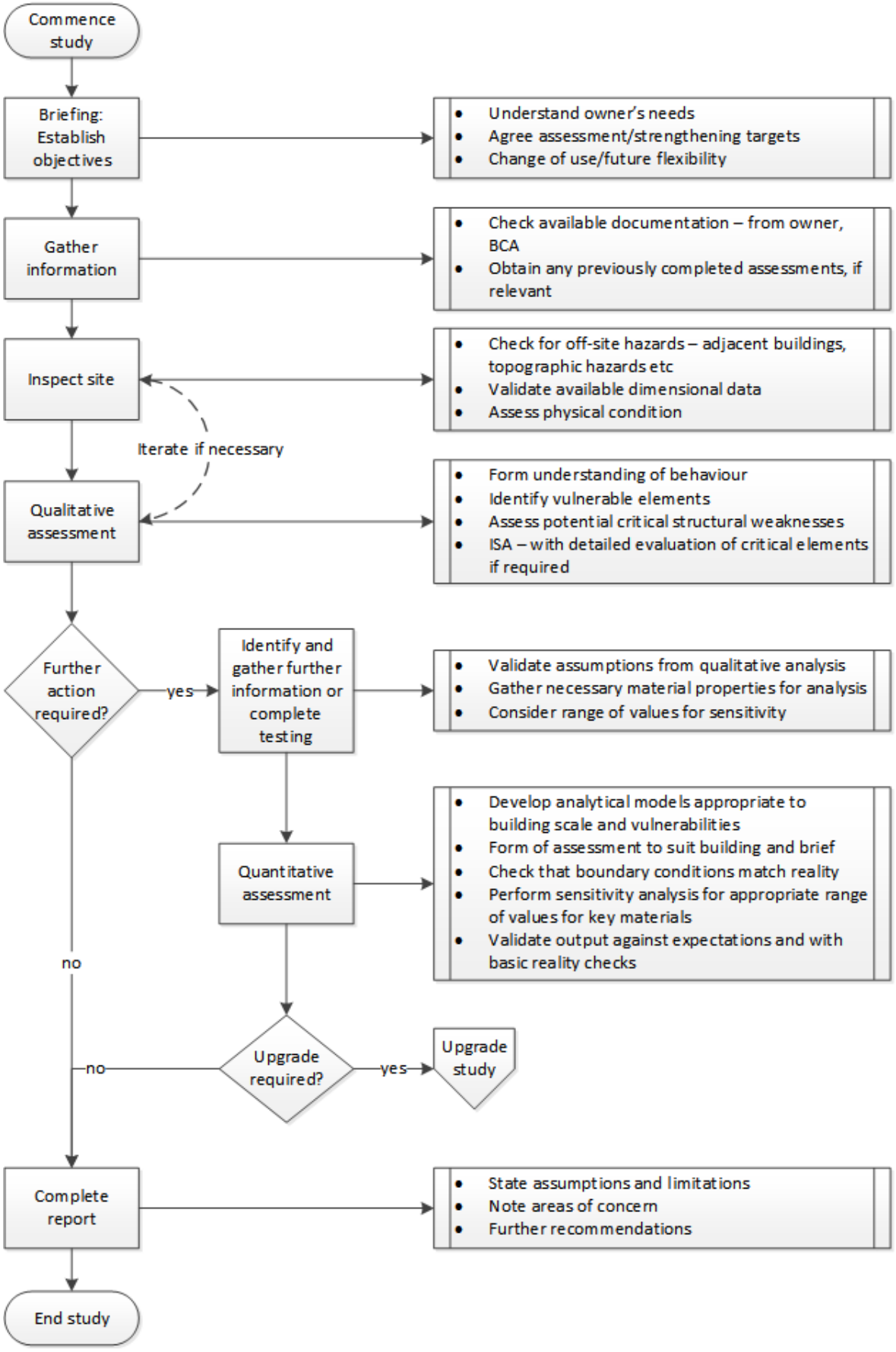


Figure A7.1: Generalised assessment process flow

A7.4 Gathering Information

Assessment of existing buildings requires careful information gathering, the level of which may vary considerably according to the building type and the purpose of the assessment. In general, the more complex the building and the more detailed the study, the more care should be taken to assemble the information required.

Equally, it may be possible to complete an ISA with limited information on key aspects of the building only, to a level that may be sufficient for the purposes of determining whether or not a building is potentially earthquake prone. There may be limited value in obtaining further information if this is the sole purpose of the assessment.

Information gathering is generally iterative. It may be more time efficient in many cases to perform preliminary analysis using relatively approximate data, in order to develop an initial understanding of a building; this may then inform the subsequent detailed information gathering. A targeted information gathering process may then be developed that places more emphasis on the most critical elements.

Equally, it is often found that a study may be limited by the information available. In such cases, the underlying assumptions should be clearly stated and recommendations made on further information that is required to give a more comprehensive assessment. In such cases, a reasonably conservative set of assumptions may be appropriate and should be based on knowledge of the generic details of the age and form of the construction.

Note:

Information gathering should include obtaining access to any prior assessments. All previous views should be taken into account when reviewing a building, although care should be taken to verify any differences in the briefing requirement, particularly when these may lead to differences in the assessment outcomes.

A7.4.1 Accessing documentation

Building documentation may be held by a number of sources, including:

- TAs
- building designers (from both original design and for subsequent alterations)
- builders
- owners, either original or subsequent
- facility maintenance contractors.

It is important to note that the documentation provided may not always be the most current. It is common for construction documentation to vary considerably from the consent (or permit) documentation, and old records often contain a mix of structures that were built and others that were not. Documentation for subsequent alterations may not always be archived or stored with the original documentation. Engineers should satisfy themselves thoroughly that the documentation is representative of the building being studied before relying on it.

Documentation may not be available for all buildings, in which case more reliance needs to be placed on inspections and testing to provide the information required to complete the assessment.

A7.4.2 Inspections

A visit to the building is a key part of the assessment process and should be completed as part of both an ISA and a DSA.

It is possible for an ISA to be completed using only external inspection. Where this is the case, it should be noted in the report so that suitable allowance can be made for this when the assessment is being used by others.

Note:

For assessments of potentially earthquake-prone buildings being undertaken in accordance with the EPB methodology, an external inspection of the building is required, and an internal assessment is also required where it is appropriate to do so.

An initial visit (before any analysis) is essential to develop a broad understanding of the building and to verify that the documentation obtained is truly representative of the building. The engineer would generally have made a qualitative evaluation of the building first, in order to identify key elements or details for review. Matters to be considered include:

- verification that the general arrangement of the building matches the drawings or assumptions
- checking of key dimensions for overall accuracy
- consideration of neighbouring buildings – assess the potential for pounding
- consider the extent of the building so that the affected building owners can be involved where the building extends over more than one title
- consideration of the expected geotechnical conditions and how these may vary with shaking intensity (including accounting for variability)
- consideration of off-site hazards, such as landslide
- general condition assessment – can key elements develop their calculated probable capacity?
- identification of key configurational issues, such as irregularity, diaphragm openings, etc.
- identifying the other building elements that need to be assessed and any of those that will require specialised assistance.

Note:

Fundamental differences between available drawings and what has actually been built can be observed, even from a relatively brief exterior inspection at the time an ISA is completed. A full inspection to confirm details and potential interaction of primary structure with other building elements is considered an essential part of a DSA process.

Subsequent visits will be required to investigate key elements and details more closely. This will normally follow sufficient analysis to have a preliminary opinion of the building behaviour, allowing investigation on site to verify that the most critical elements are as analysed.

Note:

Inspections could identify building elements that meet the Building Code performance requirements for earthquake but may not meet its requirements for other physical situations such as wind or for clauses other than B1 - Structure. These guidelines are not intended to be used to assess building element performance in those situations.

A7.4.3 Geotechnical investigation

All building assessments require some consideration of the geotechnical conditions, in order to assess the ULS shaking demand, any soil-structure interaction and structural response.

The level of geotechnical investigation required may vary from a desk-top study for relatively small structures on 'good' ground (i.e. not expected to be subject to significant differential settlement or liquefaction) for the purposes of determining whether or not a building could be earthquake prone, to comprehensive studies for large, complex structures on ground with the potential for significant differential settlement.

In general, it is recommended that the level of investigation required is determined in conjunction with a suitably experienced geotechnical engineer who has a level of familiarity with the expected site conditions.

Soil conditions may be assumed, based on knowledge and experience, for qualitative analysis. However, such assumptions should be clearly described and should be verified on site in the event that further quantitative analysis is required.

A7.4.4 Intrusive investigations

Intrusive inspection may be required for the verification of key details and for material testing.

In the case of verification of key details, engineers should be aware of the potential for variation within the building and choose enough locations throughout the building to develop an appropriate degree of confidence in the assumptions that are being made. This may vary depending on the criticality of the details being investigated, the stage of the assessment, and the convenience of exposing the details.

For example, in URM buildings, the floor-wall connections are critical. At the preliminary stages of assessment, it may be sufficient to expose only one or two locations to verify whether there are any connections at all, i.e. is there a load path. In later stages, the precise detail and spacings may be critical, in which case further investigation may be required.

Where investigation requires a level of destructive testing or exposure of concealed elements, locations should be selected carefully to provide all of the information that may potentially be required. For example, if exposing reinforcement in concrete buildings, locations should be selected to verify not only the size and location of the reinforcement, but also key detailing and conditions that may affect underlying assumptions. These include:

- Are the bars plain or deformed?
- Where are the laps located relative to potential plastic hinges?
- Where is the transverse (confining) steel and how is it anchored?
- What is the condition of the reinforcement in key locations?

A7.5 Assessment

A7.5.1 Different levels of seismic assessment

All assessments need to have a clearly defined set of objectives, without which the outcomes may be unclear and inconsistent. This is often a significant factor when assessments of the same building by different engineers have had very different outcomes. Regardless of the purpose of the assessment, a clearly identified set of objectives should be defined at the outset and the outcomes of the assessment should be validated against these objectives on completion.

The objectives of the ISA and DSA were introduced in Section A1. This section provides a more detailed contrast between these two forms of assessment and also the continuum in assessment available across and within each.

When undertaking ISA and DSA assessments, it is important that the quality and quantity of data discovered on the form and condition of the existing building is appropriate to the level of reliability required for the assessment and is recorded as part of the assessment.

The EPB methodology gives the requirements for assessments under the earthquake-prone building framework set by the Building Act.

A7.5.2 Assessment continuum

The ISA and DSA processes presented in these guidelines provide two slightly overlapping bands within a continuum of possible seismic assessments. This is represented in Figure A7.2.

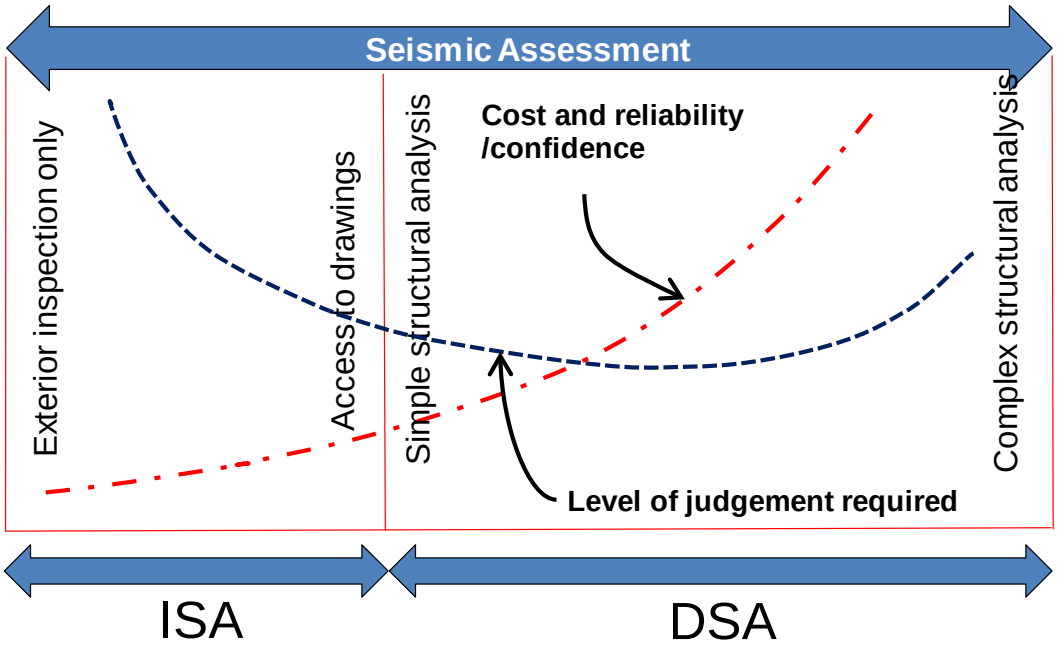


Figure A7.2: Assessment continuum

Each of the ISA or DSA processes can be carried out with a varying degree of knowledge and detail. At the extremes, a well-executed ISA may yield a result that is at least as reliable as a DSA carried out using very simplistic analyses.

Generally, however, the further the assessment processes moves to the right in Figure A7.2, the more reliable should be the result, albeit at generally greater cost for the assessment.

At all levels of assessment, the judgement of the engineer is an important input. As shown indicatively, the level of judgement required is highest during an ISA when there is little data on which to base the assessment. The level of judgement reduces as the assessment proceeds from ISA to DSA as the understanding of the attributes of the building become clearer. However, the need for judgement/experience rises if more sophisticated analysis techniques are employed in a DSA because the results can become very dependent on the inputs, and experience will be necessary to judge if the results are reasonable and valid.

A7.6 Distribution of Assessment Outcomes

All assessment involves assumptions such as:

- materials used in the original construction
- structural mechanisms that will form as the level of shaking increases
- founding conditions for the building
- alterations to the building over time.

As the assessment proceeds, assumptions are validated or changed to suit what is learned. The more assumptions that are validated, the greater the knowledge of the building's expected behaviour. Hence the assessment may be considered more reliable.

Of necessity, the more unverified assumptions that are involved, the more conservative the assessment of capacity should be, relative to the actual capacity of the building.

It is a matter of judgement as to how much effort should be expended to refine the assessment, either by completing more extensive (and possibly destructive) investigation of the building itself, or by using more elaborate methods of assessment. In some cases, it may be more appropriate to use the time (and cost) to instead provide improvement, especially in cases where the building is clearly earthquake prone.

Note:

Additional information and new findings over the course of an assessment may reduce (or increase) the assessed earthquake rating.

A7.7 Level of Detail of Assessment

Assessment of existing buildings requires considerable judgement to be exercised, not least in determining what elements of the building require assessment and how detailed that assessment should be.

There are three generally useful principles[1][2] that engineers should be mindful of:

- The Principle of Requisite Detail, which states that there is a minimum level of detail necessary in a (system) model for adequately emulating the reality that is intended to be modelled. In other words, it is important that engineers do not over-simplify the assessment to the extent that poor behaviour of a building is not identified or captured.
- The Principle of Decision Invariance, which states that the system should be sufficiently detailed that the addition of further refinement will not affect the decision. The point here is that there is no value in making models ever more complicated or comprehensive in the name of accuracy, if the additional detail makes no difference to the outcome; in fact, it may serve to obscure the outcome and simply add time and cost to the assessment.
- The Principle of consistent crudeness, which states that the quality of the output of a model cannot be greater than the quality of the crudest input or of the model itself, modified according to the sensitivity of the output to that input.

It may often be the case that until a model has been run and the hypothesis tested, a suspected outcome cannot be discounted. However, there is little point in modelling elements to a high level of detail, if there are other aspects of the building that have much more significance for the overall performance, within the broad range of interest of the assessment.

Note:

Boundary conditions assumed in modelling often play a critical role in the assessment and should be carefully considered. An example of a critical boundary condition is whether to assume a fixed base condition under a shear wall. If a fixed base is assumed, the building model may be artificially stiffened, shortening the period and increasing demand, which may at first look conservative. However, this may also have the effect of decreasing the displacement of the system, which may artificially reduce deformation demand on the primary gravity and secondary systems. Conversely, if the wall is modelled with too soft a foundation support, the base may rotate more than it should, reducing load demand but possibly over-estimating drifts.

A realistic assessment of the geotechnical conditions is one of the most important boundary condition assumptions for building modelling. It is advised that a range of soil stiffness options should be considered when modelling building systems for which this may be critical. Typically, this will occur when there are soft soil conditions and/or where assessing building types that are vulnerable to significant ground deformations. This includes in particular URM, which has relatively little tolerance to ground deformation. At the other extreme, most moment frame structures should be able to tolerate significant differential settlements.

[1] Brown CB & Elms DG (2015), Engineering decisions: Information, knowledge and understanding, Structural Safety 52 (2015) 66-77, Elsevier.
[2] Elms DG (1985), The principle of consistent crudeness,, Purdue University, IN, 1985 Proc Workshop on Civil Engineering Application of Fuzzy Sets, Purdue University, IN, 1985.

A7.7.1 Qualitative assessment

The first step in any building evaluation should be a qualitative assessment. The ISA provided in Part B is essentially a qualitative assessment. Qualitative assessment is a vital predecessor to quantitative analysis. It informs the engineer of the key elements of the building and assists in focusing the subsequent detailed evaluation.

This requires the engineer to consider not only the mechanisms that may have been envisaged by the original designer, but also the combined effect of unrecognised load paths, structural incompatibilities (that may be better understood now than at the time of design) and the impact of alterations over time. The last may include the effects of time itself. That is, aging of the building and maintenance (or lack of it).

The ISA should include assessment of available plans and specifications; but this should always be approached with caution. Often the plans that are available are not those that were built from and may not include subsequent alterations. Moreover, then as now, buildings were not always built according to the plans. Part of the role of the engineer is to consider the possible impact of these variables and make reasonable allowance for them in the assessment.

Qualitative assessment should include (but is not limited to) an IEP. This is at least a useful benchmarking exercise that enables engineers to consider at a high level those attributes of the building that may have significant impact on the behaviour of the building. By approaching this in a qualitative sense before detailed assessment, it gives a sound basis for self-checking of the outcomes of future detailed analysis.

Qualitative assessment may include some ‘back of envelope’ calculations of key element capacities and demands, in order to test how critical mechanisms or details are and to verify the findings or judgement calls of an IEP.

A7.7.2 Quantitative assessment

Quantitative assessment generally consists of a DSA in the form outlined in Part C. It is informed by the findings of the qualitative assessment, which should assist in identifying likely failure mechanisms that should be investigated in more detail.

Prior to commencing quantitative assessment, the outcomes of the qualitative assessment should be reviewed, with emphasis on what matters may need to be included in a detailed assessment, including consideration of:

- Is further investigation required to confirm assumptions made in the qualitative assessment?
- What boundary conditions have been or will be assumed and how do these relate to reality?
- What foundation conditions have been assumed?

Geotechnical conditions are a key consideration for quantitative analysis, requiring a suitable degree of investigation in order to validate assumptions and to provide the inputs required for detailed evaluation.

Note:

Particular emphasis should be given to the impact of significant differential settlements, with close attention being paid to the range of possible outcomes. For example, in sites that are expected to have significant liquefaction at a given level of shaking, consideration should also be given to the possibility that liquefaction does not happen. In such cases, liquefaction may be considered to be a limiting factor for the building's rating, but a premature failure of a brittle element under higher levels of shaking may pose a greater risk for occupant life safety.

A7.8 Establishing the Assessment and Analysis Strategy and Approach

The assessment procedure followed will be determined according to a number of factors, including:

- The objectives of the study. If the primary purpose is simply to provide information to assist TAs in establishing whether a building is earthquake prone, it may be enough to complete an ISA, based on relatively generic information, so long as this meets the requirements of the EPB methodology. However, if a client requires a more comprehensive assessment of risks for a building for other purposes, that may determine the need for a detailed assessment.
- The complexity of the building. Although scale may determine the risk (as it impacts occupant numbers), the complexity of the structural form is a more significant factor in determining the assessment methodology.
- For example, simple, regular, low-rise structures may be assessed using a combination of an ISA with specific analysis of identified critical elements to establish an overall earthquake rating. The scale of such a building may not be relevant, provided that the load paths are simple and the building may be relied upon to respond in a regular fashion. Conversely, a mid-to-high rise building with significant irregularity (for example a corner building with walls on the internal boundaries) is expected to behave poorly, and is expected to require a full higher order analysis.
- The degree of influence of soil conditions. This can be a significant influence, particularly when there is potential for significant differential settlement, with or without liquefaction. The analysis of buildings should include appropriate allowance for soil non-linearity, foundation flexibility and possible variations (through sensitivity analysis).

In all cases, engineers should consider the limits of applicability of the assessment processes being considered. This is particularly important when assessing buildings with mixed systems and/or unknown ductility demand, or irregular buildings with diaphragms of sufficient rigidity to redistribute actions between lines of support (i.e. the potential for torsional response).

A8. Reporting Seismic Assessment Results

A8.1 Introduction

It is important to report assessment results in an appropriate context, at both the ISA and DSA levels of assessment.

This includes *%NBS* earthquake scores and rating at an appropriate levels of precision, and a seismic grade and qualitative risk classification.

For assessments of potentially earthquake-prone buildings, the EPB methodology sets out reporting requirements, which includes the completion of a report summary to be submitted with the full assessment report. It is recommended that this summary is prepared for all assessments, whether or not they are being completed for earthquake-prone building purposes. The template summary report is discussed further in Section A8.5.

Note:

Adherence to these recommendations is considered essential. It is very important that the report correctly describes the result of the assessment in terms that define the scope of the assessment and the basis for it. Just providing a *%NBS* earthquake rating without these could suggest that the building meets the new building standard generally (i.e. including gravity and wind, etc.) and earthquake provisions in particular without inclusion of the existing building concessions that are included in these guidelines.

It is important to include a discussion of the grading and level of risk to put the earthquake rating in context. Without this, there is no reference point for the rating and the recipients could perceive it requires an unintended immediate action (e.g. vacating a building).

A8.2 Level of Precision in Reported *%NBS* Earthquake Scores and Rating

The *%NBS* earthquake rating given needs to reflect the reliability/accuracy implied. For this reason, earthquake ratings should only be quoted as a whole number. Except for 18, 19, 33, 34, and 67*%NBS* earthquake ratings that are close to the two earthquake-prone and the earthquake risk thresholds respectively, it is further recommended that the whole number scores be rounded to the nearest 5*%NBS* (up or down).

Numerical scores above 100*%NBS* may provide an erroneous indication of expected performance. It is recommended that these are simply stated as >100*%NBS* unless, for example, there is a need to reflect a score or rating relative to a different importance level standard in which case a rating of 130*%NBS* (IL2) may be relevant as representing 100*%NBS* (IL3) in change of use discussions.

Likewise scores below 15*%NBS* have no practical meaning unless the building's expected seismic performance is extremely tenuous which will rarely be the case. Therefore, it is recommended that a score or rating is not quoted as less than 15*%NBS*.

Table A8.1indicates the intent of the recommended rounding.

Table A8.1: Rounding of *%NBS* earthquake ratings

Raw (Assessed) score	Assigned rating for reporting purposes
0% - 17%	15*%NBS*
18%	18*%NBS*
19%	19*%NBS*
20% - 22%	20*%NBS*
23% - 27%	25*%NBS*
28% - 32%	30*%NBS*
33%	33*%NBS*
34%	34*%NBS*
35% - 37%	35*%NBS*
38% - 42%	40*%NBS*
…	
63% - 66%	65*%NBS*
67%	67*%NBS*
68% - 72%	70*%NBS*
…	
98% - 102%	100*%NBS*
> 102%	>100*%NBS*

Engineers should consider carefully before rating a building between 18*%NBS* and 20*%NBS*, 30*%NBS* and 37*%NBS* or between 65*%NBS* and 70*%NBS*. The ramifications of not exceeding each threshold level can be significant in terms of implications for additional assessment and action required and should be carefully considered. More detailed consideration of the CSW in a DSA may move the rating away from these critical ranges. Refer to Part B for further discussion on the reliability available from the ISA and how to deal with this.

A8.3 Grading Scheme

The NZSEE developed a grading system to complement the *%NBS* earthquake rating. This bands the assessment results to reduce the emphasis of the percentage value within an earthquake rating.

The wider objective of the grading scheme is to raise industry awareness and allow market forces to work in reducing earthquake risk.

The NZSEE grading scheme and the linkage with the *%NBS* earthquake rating is summarised in Table A3.1.

The NZSEE grading system is not a requirement of the Building Act. Instead, earthquake rating categories (which are consistent with this scale) are established in the Building

(Specified Systems, Change the Use, and Earthquake-prone Buildings) Regulations 2005 (as amended) and assigned by TAs to earthquake-prone buildings.

Note:

Other grading schemes are also currently under development, e.g. Quakestar. These can consider a broader range of seismic issues than just life safety and structural issues.

A8.4 Qualitative Risk Classification

It is useful to provide a qualitative risk classification to provide context for reporting the assessment results.

The intended risk classifications are shown in Table A3.1.

Buildings that are classified as earthquake prone in accordance with the Building Act are regarded as high risk buildings. Those with ≥67*%NBS* are regarded as being low risk. This leaves a group in between that meet the requirements of the Act but cannot be regarded as low risk. These have been termed low to medium risk.

Note:

For many years NZSEE has referred to buildings <67*%NBS* as being earthquake risk. Broadly speaking, these can be assumed to be all buildings not classified as low risk.

A8.5 Assessment Summary Reports

A8.5.1 General

Part B provides a reporting framework and covering letter for ISAs and a corresponding framework is provided in Part C (Section C1) for DSAs.

A suggested template table, referred to as an Assessment Summary Report, is also provided on www.EQ-Assess.org.nz as a separate file. This table summarises the key points from seismic assessments, and should accompany both ISA and DSA reports. For seismic assessments for earthquake-prone building purposes, the EPB methodology requires the engineer to provide an assessment summary report.

The purpose of the summary is to provide better consistency in both the information provided and the way it is provided, and hence provide clearer communication between all parties. All judgements made need to be justified/substantiated, and preferably recorded, as part of the assessment process.

Assessment Summary Reports should provide the following information:

Building information

- Address etc., No. of storeys, year of design, structural system, previous retrofit

Assessment information

- Person responsible for the assessment, when inspected, what information reviewed, geotech info, previous reports referred to

Summary of engineering methodology and key parameters

- Assessment methodology used, and how these Guidelines were applied

Assessment outcomes

- *%NBS* rating, seismic grade and qualitative risk classification, governing CSW; mode of failure and physical consequences statements (refer Section A8.5.2).

A8.5.2 Understanding and Determining Building Failure Modes

One of the reporting requirements for engineering assessments from the EPB methodology is the need to report on modes of failure where the building is found to have an earthquake rating of less than 34*%NBS.* This includes parts of buildings as well as the building as a whole where the scores are less than 34*%NBS*.

Building failure, in common language, implies the complete failure of a structure, resulting in widespread physical harm to the occupants. However, failure requires a more comprehensive definition for the purposes of building assessment. In a life safety context (as discussed in Section A3.1.1), building failure implies a form of failure that will lead to a significant life safety hazard (noting that the Building Act 2004 is also concerned with damage to adjacent buildings).

Once again, the differences between design and assessment (refer to Section A4.1) are critical:

- When designing buildings, it is relatively straightforward to ensure that building elements will meet or exceed their target capacity (either strength or displacement). It does not preclude the failure of elements, but the design approach (including, for example, the principles of capacity design), generally provides confidence in the overall building performance. By detailing elements for the assumed ductility demand and keeping redistribution within code limits, designers are assured that elements are not pushed beyond their expected deformation capacity. This is especially critical for elements that carry significant axial gravity load. Confidence is also provided that the building will continue to perform satisfactorily at levels of shaking well beyond design levels.
- In assessment, it is important to address the implications of element failure more comprehensively. Failure of individual elements within a building does not necessarily lead to failure of the building as a whole or the need to consider the element as a significant life safety hazard.

  For example, a beam that has its shear capacity exceeded may ‘fail’, but is expected to hang in catenary action and so would not be regarded a significant life safety hazard. However, a column that has its shear capacity exceeded also loses the capacity to resist simultaneous axial load and hence may cause a localised or more widespread collapse condition. The former does not limit the building’s capacity, but the latter does.

Note:

While exceeding the shear capacity of individual beams may not in itself constitute a significant life safety hazard, successive shear failures in multiple beams in a concrete frame, for example, is expected to lead to deterioration in lateral capacity and a significant life safety issue for the building as a whole.

The Mode of Failure and Physical Consequence statement is a description by the engineer of the manner and extent to which the lowest building element scoring less than 34*%NBS* could collapse or fall and give rise to a significant life safety hazard.

Table A8.2 provides sample Mode of Failure and Physical Consequence statements for some building elements relating to the building overall and parts. The entries are illustrative of both the information required and the brevity expected. The entries in the Exposure of People column are indicative only, to illustrate how it is anticipated that TA officers will consider the numbers of people or other people that could be exposed to the hazard described in the mode of failure statement as they apply s133AB(1)(b) as the final step in making an earthquake prone decision.

Table A8.2: Sample summary statements of the mode of failure and significant life safety hazard (building or parts) for selected situations

Engineering statement of structural weaknesses <34*%NBS* and location	Mode of failure and physical consequence statement (including the nature and extent of impact giving rise to a significant life safety hazard)	Exposure of people or damage to other property
ENGINEER TO DETERMINE		**TA TO DETERMINE**
Building overall		
Building column shear failure	A sudden loss of gravity support for a section of the floor, leading to collapse of part or all of the building within or beyond its footprint	People inside or outside the building People in adjacent building(s)
Failure of wall bracing	Building may displace excessively, leading to overall collapse	People inside or outside the building
Parts of buildings		
Unrestrained URM parapet	Parapet sections may fall from the building onto areas outside the building and/or inwards onto roof	Public access in street in front of building People in or around vehicles in the accessway alongside the building People in main entry lobby inside the building People in the neighbouring building(s) Damage to the roof structure of the neighbouring building(s)
Precast panel connection unable to accommodate lateral movement of the building	Panel(s) may fall on the areas outside the building	
Roof diaphragm connection to perimeter wall elements	Wall panels may fall outside the footprint of the building	

Inadequate restraint for air handling units of mass greater than 50 kg within suspended ceiling system	Air handling units could fall more than 3 m to floor (and/or impact the suspended ceiling system and cause that to fall also)	People in the building

A8.6 Acknowledging Other Assessments

The report should list any other known seismic assessments of the building, and acknowledge any differences in the assessment results (e.g. earthquake rating) and the reasons for them.

Note:

The explanation need not present technical detail other than to note if the various assessments were DSAs or ISAs, that there were additional inspections, that it included material test results, etc.

When noting the differences on an Assessment Summary Report it is advisable for all comparisons to be with the current assessment and to note on the form "The earthquake rating increased/decreased to XXX*%NBS* (ILY) because…" rather than giving the superseded rating(s) to avoid confusion with the current earthquake rating being reported.

A9. Reconciling Differences in Assessment Results

Due to the nature of the seismic assessment process, it should not come as a surprise that, in some circumstances, assessments of the same building by two or more experienced engineers may differ – sometimes significantly. This is to be expected, especially if a different level of information was available to each engineer. This will particularly be the case for earthquake ratings determined using the ISA process, but can also happen should multiple DSAs be completed for the same building.

However, it is expected that experienced engineers will be able to identify the critical issues that are expected to affect seismic behaviour and that, through discussion, a consensus position should be able to be agreed. If the assessments are at the ISA level and consensus cannot be reached, a DSA is recommended (refer to Part B). If the disagreements occur at the DSA level and cannot be readily resolved, the differences in opinion should be acknowledged and recorded.

Note:

Any assessment that has been independently reviewed is expected to provide a more robust earthquake rating than one based solely on the judgement of one engineer. Therefore, independent review is encouraged.

A10. Improving Seismic Performance

A10.1 Introduction

There are many buildings in New Zealand constructed prior to the introduction of the modern earthquake design approach in 1976. The cost to the community that would be requiring for full compliance with current standards (i.e. all buildings brought to 100*%NBS*) would be considerable, and arguably disproportionate to the risk reduction.

Note:

It is considered that the community would accept a higher level of risk in an existing building than for a new building, if only for the reason that it will, in general, be economically more feasible to provide higher levels of dependable strength and reliable ductility in a new building than in an existing one. As a result, existing buildings that can be shown to have an earthquake rating $\geq$67*%NBS* can be considered to represent a low to medium risk in regards to life safety.

Accepting an earthquake rating of 67*%NBS* as a minimum for existing buildings to be categorised as low/medium risk corresponds to an increase in risk for an existing building of up to approximately five times that of an equivalent new building. This is judged reasonable and compares well to equivalent levels set for the evaluation of existing buildings in the United States. For example, the approach taken in ASCE 41-13 (2014) leads to approximately 75% of the new building standard for the defined performance objective BPOE (basic performance objective for existing buildings).

While this increase in risk could appear high on a building-by-building basis, it appears a reasonable minimum target overall and needs to be considered in the context of the low level of risk implied in absolute terms.

Upgrading to as nearly as is reasonably practicable to new building standard is recommended. However, it is considered more important and realistic to identify the high risk buildings, and reduce the risk they pose to a more acceptable level, rather than to attempt to ensure that all existing buildings comply with the latest standards. The elimination of non-ductile failure mechanisms and CSWs is in itself of greater importance than the actual assessment and strengthening level. Buildings rarely fail during earthquakes solely because the design forces have been underestimated. More often than not, poor performance is the result of some obvious configurational or detailing deficiency.

A10.2 Overview of Improvement Processes

A generalised assessment process is as illustrated below in Figure A10.1. The steps in the process are summarised in the sections that follow.

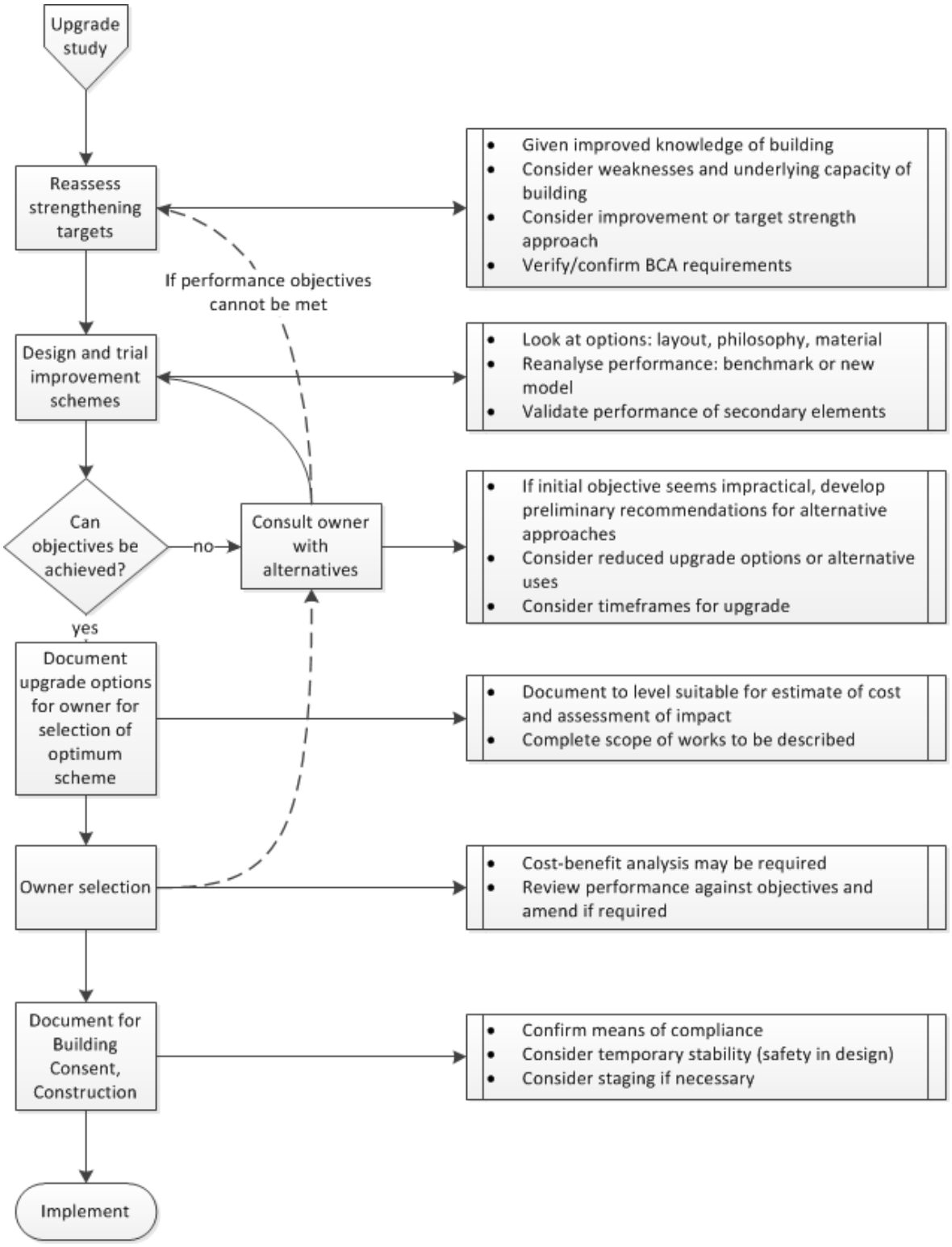


Figure A10.1: Generalised seismic upgrade process flow

A10.2.1 Establishing performance objectives

It is important that a detailed understanding of the owner's future performance requirements/expectations is achieved. Although this will often be expressed simply as a strengthening target in terms of *%NBS*, this may only provide a part of the picture.

With improving the performance of buildings essentially about risk reduction, it is important that the owner's risk appetite and main concerns over likely outcomes are both well understood. Factors that may be considered include:

- compliance with Building Act 2004 requirements
- usability following earthquake
- reparability
- cost of repairs
- non-structural performance
- future flexibility.

Determining the owner's performance objectives and requirements will inform the repair strategies that may be worthy of investigation.

A10.2.2 Improvement philosophy

There are many different methods for improving buildings. Some of the most common may be broadly categorised as follows:

- **Replacement**: Inserting a new lateral system that will resist the majority of the seismic load. This may be used where a building's rating is very low and would be difficult to improve, or where a building is being extended.
- **Enhancement**: Improving the existing lateral systems without substantially changing the mode of behaviour. May be used where a building requires only a relatively minor increase in rating.
- **Protection**: Increasing the capacity of the structure (principally the gravity system) to tolerate the imposed displacements. May be used where the primary lateral load resisting structure has sufficient capacity but the primary gravity system and/or other building elements have insufficient displacement capacity to tolerate the ultimate drift.
- **Reduction in demand**: Reducing the demand on the building by isolation or by increasing the damping in the system. May be used where there is a need to reduce damage to contents or where the primary systems cannot tolerate the imposed displacements.

Several alternative approaches and different levels of intervention should be investigated before adopting any one particular approach. Although direct cost is often one of the primary criteria for assessing improvement concepts, there are other important considerations, including those mentioned in Section A10.2.1 above.

It is generally recommended that improvement is not approached dogmatically with a specific rating target in mind - that is to say, compromise in the desired performance objectives or outcomes should be considered.

Note:

There is generally not a linear relationship between upgrading cost and rating improvement. For example, introduction of new elements into a building may create a step change in cost, but there may only be a minor incremental cost increase to improve the building's rating to the limits of what the new element could contribute.

A10.2.3 Other considerations

A10.2.3.1 Improving buildings with higher importance levels

Buildings of higher importance levels (IL3 or IL4) may require improvement to satisfy functional requirements including post-disaster use, or for reduced levels of damage.

Where reduced levels of damage are an essential outcome of the improvement process, consideration should be given to displacement limits based on the most displacement sensitive elements that need protecting.

For buildings that require improvement in order to become an IL4 facility, it is recommended that full compliance with SLS2 requirements is targeted, along with a minimum of 67*%NBS* for ULS requirements. All parts that are required to be operational following the SLS2 event, or the failure of which might limit the building's use for its intended post-disaster purpose require consideration.

A10.2.3.2 Improving heritage buildings

Many heritage buildings are either earthquake prone or earthquake risk buildings. While the assessment of these buildings will generally follow the same principles as other buildings, their improvement requires more careful consideration in order to determine an acceptable upgrading strategy. In practice this often requires a significant degree of compromise between heritage impact and structural upgrading objectives.

This is outside the scope of this document, but the principles outlined herein will be generally applicable. Reference should be made in particular to the ICOMOS New Zealand charter[3].

A10.2.3.3 Improving buildings with more than one structure

Improving the performance of an individual structure that is either structurally connected to or expected to be affected by its close proximity to one of more other structures requires careful consideration.

Note:

Improving the performance of these structures requires similar considerations to their assessment, with the general condition that improvement of one structure cannot adversely affect the others it is connected to or in close proximity to.

Close attention needs to be paid to how load is expected to be transferred between the structures – either intentionally or unintentionally (e.g. through pounding). Particularly where there are stiffness incompatibilities between adjacent structures.

[3] ICOMOS New Zealand Charter, for the Conservation of Places of Cultural Heritage Value, 2010.

Some additional considerations apply with regard to party walls.

There is an obligation on the owners of the property on either side to consider the implications of the work they propose on both existing and future configurations. There may be some legal constraints if there are specific agreements in place (for example a party wall agreement or right of support). However, the support of the wall and what it supports should be considered both with and without the adjacent building in place.

When separating the structure under consideration from the neighbouring property, the adverse effect of the separation to the adjacent structure(s) should be considered. In such cases, the party doing the work is responsible for ensuring that the adjacent property is made no worse by the alteration. This may require new wall anchors to be provided for the roof and/or floor(s) in the other property in order to provide the separation.

A10.2.4 Use of analysis methods from this document in conjunction with the design of new strengthening elements

The design of new elements of buildings must comply with section 17 of the Building Act 2004, which requires that all new work must comply with the New Zealand Building Code, to the extent required by the Act. Buildings that are not being upgraded to 100*%NBS* require careful consideration. Depending on the improvement philosophy being followed, new elements are required to interact with the existing structure in different ways.

In general, the following approach is recommended:

- New building elements should be designed using current building code methods and detailing, so that their dependable capacity meets or exceeds the demand calculated to the seismic provisions of the relevant standards, factored by the target *%NBS* for the overall building.
- Where the new systems are augmenting the existing building capacity, the new elements may be designed to resist a greater proportion of the overall seismic demand, provided that due allowance is made for the ability of the structure to redistribute loads (through diaphragms or collectors). This may require additional collector elements or diaphragm enhancement, which may be designed for the lesser of the overstrength capacities of the elements being loaded by them, or by rational analysis at the target *%NBS*. Where a capacity approach has been used, probable capacities may be used for design, in accordance with the appropriate material standards.
- Regardless of the ductility capacity of the new elements, the displacement compatibility of the new and existing elements should be carefully considered. The distribution of demand to new elements may be limited by the displacement capacity of the existing building (refer to Section C2).
- To provide validation of the proposed improvements (if necessary), the building should be re-assessed with the new elements, assuming probable (expected) strength properties in accordance with Part C, as if the new elements were already in place. This approach may be of most value when using nonlinear techniques to provide a potential rating for the improved building, possibly in comparing alternatives improvement strategies. This step would not generally be required when simply adding elements to meet or exceed a target rating (for example, improving a building so that it is no longer earthquake prone or earthquake risk) and using linear analysis to determine design actions.

Note:

When the proposed improvement measures essentially replace or substantially replace the existing lateral systems, this step would generally be omitted, providing that the stiffness compatibility of the new system has been assessed to ensure that a premature failure of the gravity system is not expected to occur assuming the worst combination of stiffnesses of existing and new structures. Such analysis should include consideration of the potential impact of foundation strength and stiffness.

For buildings that require improvement solely for the purpose of ensuring that the building is no longer earthquake prone, the building and its parts need only be improved to 34*%NBS* and only those building elements that require consideration as noted in Section A3 need be upgraded.

A10.2.5 Regulatory requirements for improvements

Where buildings are being altered with no change of use, section 112 of the Building Act 2004 must be complied with. This requires that the building complies with the Building Code (for provisions relating to structure) to at least the extent that it complied before the alterations. If the building is earthquake prone, the TA may request that the building is upgraded to no longer be earthquake prone at the same time.

Where buildings are undergoing a change of use, section 115 of the Building Act 2004 must be complied with. This requires that the building comply as nearly as is reasonably practicable with the Building Code as if it were an equivalent new building.

Note:

The determination of "as nearly as is reasonably practicable" may vary between TAs according to local regulation and practice.

In the past, a level of 67*%NBS* was regarded as sufficient to comply with section 115 for most uses. However it is recommended that consideration be given to what additional work may be required to bring the building to full compliance, especially for IL4 buildings. A simple cost-benefit study often enables a suitable target load level to be established.

The Building Act 2004 promotes progressive upgrades of earthquake-prone buildings (in section 133AT) by only allowing a building consent to be granted for a substantial alteration when the alteration includes the seismic work necessary to ensure the building will no longer be earthquake prone. The trigger for a substantial alteration is set out in the Building (Specified Systems, Change the Use, and Earthquake-prone Buildings) Regulations 2005 (as amended), and is applied by TAs.

A10.2.6 Temporary stability of buildings during construction

Unlike new buildings, which generally increase in capacity as the building work progresses, existing buildings may have their capacity reduced during the construction process, prior to the upgrading work being completed. This may occur, for example, through activities such as:

- undermining of foundations to install underpinning

- partial removal of URM walls in order to replace them with reinforced elements
- separation of diaphragms from primarily lateral elements.

It may not be practically possible to ensure no reduction in rating during construction, and the provisions of the Building Act 2004 regarding change of use and alterations generally consider the building in its completed condition. It is recommended that designers collaborate with the owner and contractor(s) responsible for the work and consider safety on site and to the public, where appropriate, to develop a solution that satisfies health and safety requirements and good risk management practice.

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The Seismic Assessment of Existing Buildings

Technical Guidelines for Engineering Assessments

July 2017

Initial Seismic Assessment

Part B

nzsee
NEW ZEALAND SOCIETY FOR EARTHQUAKE ENGINEERING

Sesoc

NEW ZEALAND GEOTECHNICAL SOCIETY INC

MINISTRY OF BUSINESS, INNOVATION & EMPLOYMENT
HIKINA WHAKATUTUKI

EQC
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3 Part C – Detailed Seismic Assessment

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B1. Introduction

B1.1 General

This section describes the Initial Seismic Assessment (ISA), which is the recommended first step in the overall assessment process. It is intended to be a coarse evaluation involving as few resources as reasonably possible.

Figure B.1 summarises the main elements of the ISA. It also highlights that the continuum ranges from a "basic" ISA which involves collecting basic building data, an exterior inspection and completing an Initial Evaluation Procedure (IEP) (explained below) to a "comprehensive" ISA which adds the collection of all readily available building data, an interior inspection, drawing review, and supplementary calculations as required. The use of original drawings will also allow a reasonable review of internal details such as foundations, stairs, column ductility and floor type; and this is recommended if the building's earthquake rating is around the threshold levels of 34*%NBS* and 67*%NBS*.

If important decisions need to be made that rely on a building's seismic status, it is expected that an ISA would be followed by a Detailed Seismic Assessment (DSA). Such decisions could include those relating to pre-purchase due diligence, arranging insurance, or before designing seismic retrofit works. A comprehensive ISA with reference to drawings and interior and exterior inspections and supplemented with calculations (if required) may be used to confirm the status of an earthquake-prone building, provided that the engineer is confident that the result reflects the expected behaviour of the building.

Note:

It is likely that this option would only be viable in cases where the assessment clearly indicates that either the building is earthquake prone or it is not. The situations when a comprehensive ISA would be considered appropriate are covered in the EPB methodology. Refer also to Part A.

The process adopted for a particular assessment will depend to a large extent on its specific objectives and the number of buildings involved. For example, the ISA process for a portfolio of buildings may have a different focus than that for a single building. If multiple buildings are involved, the engineer may need to prioritise, as it will probably be impractical to assess all buildings simultaneously and immediately.

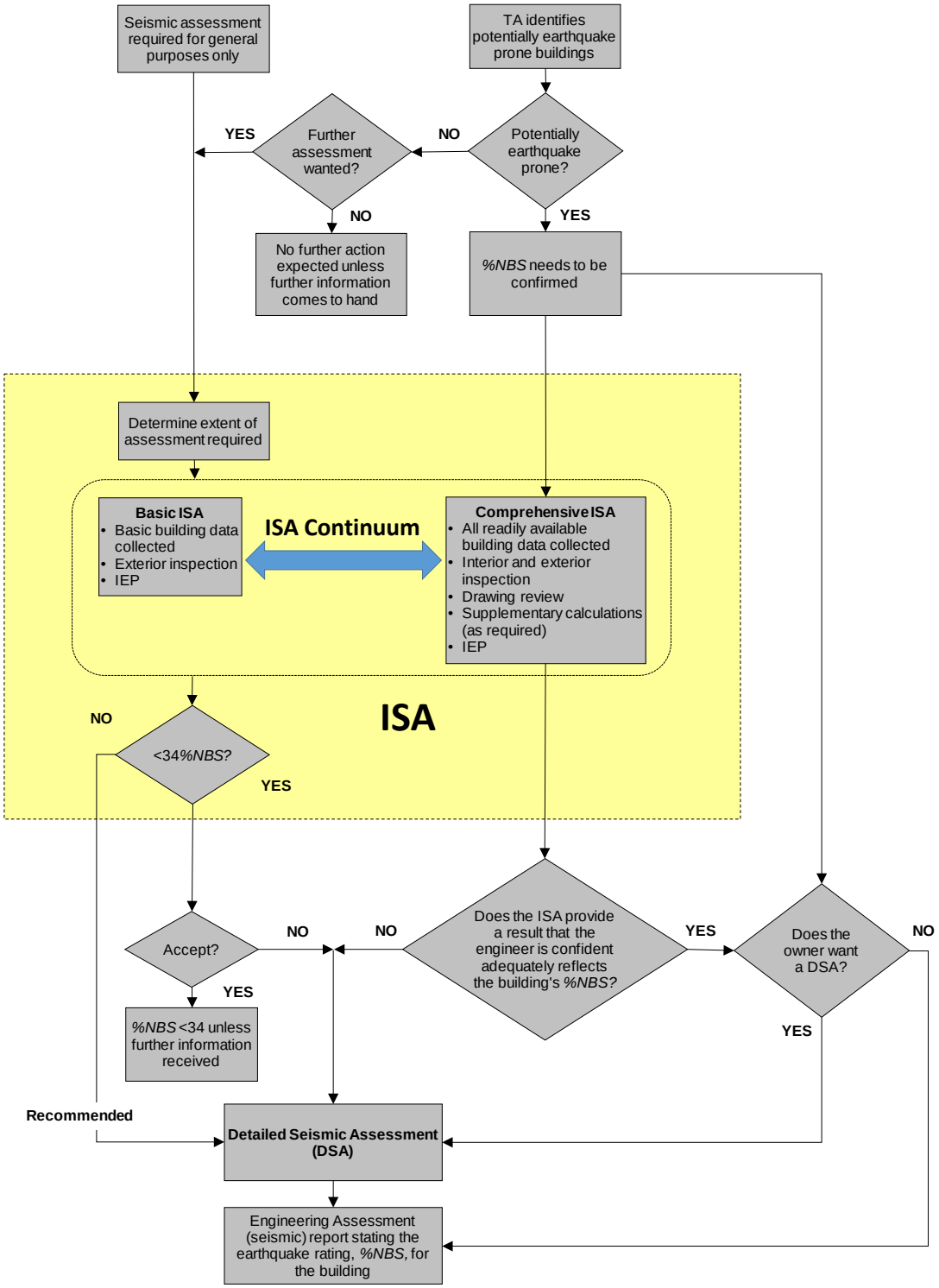

Figure B.1: Diagrammatic representation of the Initial Seismic Assessment process

When undertaking an ISA for post-1976 buildings (those designed and constructed using seismic design codes from 1976 onwards) the engineer will need to approach the assessment from a slightly different perspective. While these buildings are unlikely to be earthquake prone, they can contain structural weaknesses that could lead to a sudden, non-ductile mode of failure at levels of seismic shaking less than current design levels for the ultimate limit state (ULS) shaking. It is also important that buildings that may be earthquake risk but that are not earthquake prone (i.e. that lie between 34*%NBS* and 67*%NBS*) and that have unacceptable failure modes are identified. How this might be done is discussed further in Section B4.3. Post-1976 buildings can also feature potential CSWs that relate to detailing issues rather than configurational SWs relating to regularity. It is therefore important that ISAs of post-1976 buildings involve both a full interior inspection and a review of available structural documentation.

Note:

In buildings of the post-1976 era, the greater use and availability of computer programs for structural analysis and architectural developments has led to the adoption of sometimes quite complex structural configurations and lateral load paths. Whereas for earlier buildings it might have been possible to identify a generic structural form from an exterior inspection, it is often difficult to pick this for post-1976 buildings.

This is particularly the case for mixed-use buildings involving the competing structural layouts of accommodation, office and car parking. These structures typically feature offset columns or other transfer structures which cause irregular steps in the load path that may or may not have been taken into account appropriately in the original design.

The main tool provided by these guidelines for carrying out an ISA is the Initial Evaluation Procedure, or IEP. While other procedures can be substituted for the IEP in the ISA, it is important for consistency that the IEP's essence is maintained and the result is reflective of the building as a whole.

Section B3 discusses the IEP process and level of experience required. It also discusses the limitations of this process and how to deal with differing assessment results. Section B4 covers issues specific to building type and era, and Appendix BA details the steps involved in the IEP and includes the required worksheets (Tables IEP-1 to IEP-5).

Note:

The IEP was introduced in these guidelines in 2006 and refined in 2014 (NZSEE Guidelines, 2006 including corrigenda 1, 2, 3 and 4). The version in these guidelines is essentially unchanged from 2014.

A fundamental aspect of the IEP is the identification, and qualitative assessment, of the effects of any aspects of the structure and/or its parts that would be expected to reduce the performance of the building in earthquakes, and thereby increase the life safety risks to occupants and/or have an adverse effect on neighbouring buildings. These deficiencies in the building are referred to as potential critical structural weaknesses (pCSWs). Section B2 discusses these further and also lists the potential severe structural weaknesses (pSSWs) that must be noted if identified.

These guidelines recognise that the IEP can be meaningfully enhanced for certain building types such as unreinforced masonry (URM) by considering specific attributes. Appendix BB provides an attribute scoring method for URM buildings which can be used in conjunction with the IEP. However, this method generally requires a greater level of knowledge of a building than is typically expected or intended for an IEP carried out as part of a basic ISA.

Calculations to support judgement decisions on particular aspects of the ISA are encouraged. This would be expected to lead to a more reliable earthquake rating from the ISA without the full cost of a DSA. However, care should be taken to avoid over-assessment in one area at the expense of another without a more holistic assessment of the building. The potential rating for a building as a whole from an ISA must reflect the best judgement of the engineer, taking into account all aspects known to that engineer

The result from the ISA process is reported in terms of a *%NBS* (percentage of new building standard) earthquake rating the same way as the result from a DSA. For the reasons outlined above, the results from an ISA are generally reported as a potential earthquake rating for the building, and all potential SWs are given the status of potential CSWs. More detailed assessment, or consideration of further information, could potentially raise or lower the ISA rating and this should be expected. The exception to this is when an ISA is considered by the engineer to provide sufficient justification to establish the earthquake rating for earthquake prone assessments in accordance with the requirements set out in the EPB methodology. In such cases the SWs remains as potential CSWs but the result of the ISA is reported as the earthquake rating.

The reporting of the results of the ISA should be appropriate for the particular circumstances. These guidelines recommend that ISA reports sent to building owners and/or tenants include explanatory information such as:

- a description of the building structure
- the results of the ISA
- the level of knowledge available, and
- the limitations of the process.

Section B5 covers expectations for reporting the ISA and providing an accompanying technical summary, which is required if the ISA is to be submitted as an engineering assessment to a Territorial Authority (TA) for the purposes of determining whether or not a building is earthquake prone. These guidelines also include recommended templates for the covering letter from the engineer to building owners or tenants who have commissioned an ISA (Appendix BC).

B1.2 Regulatory Considerations

Before mid-2017, ISAs and IEPs as outlined in the NZSEE's 2006 guidelines were used extensively by a number of TAs to help them establish which buildings in their cities or districts were potentially earthquake prone. This was typically undertaken as part of TAs' active earthquake-prone building policies established under the Building Act 2004.

On 1 July 2017 significant changes to the Building Act's earthquake-prone building provisions took effect (via the Building (Earthquake-prone Buildings) Amendment Act 2016). As a result TAs no longer have individual earthquake-prone building policies. However, they are still responsible for determining whether or not individual buildings in their district are earthquake prone or potentially earthquake prone.

As well as following the provisions in the Building Act and supporting regulations, TAs must now follow the Earthquake-prone Building (EPB) methodology set by the chief executive of the Ministry of Business, Innovation and Employment. This methodology has similar status to a regulation and references these guidelines.

The EPB methodology contains profiles of potentially earthquake-prone buildings; i.e. categories of buildings with known seismic vulnerabilities and that can be considered potentially earthquake prone. TAs must consider which buildings in their district fall within these profile categories within set time frames and then write to the owners to request an engineering assessment. The methodology contains criteria for when an ISA (or other form of seismic assessment) may be used as a suitable "engineering assessment" to meet the legislative requirements – either for buildings within the profile categories or for those the TA wishes to consider at any other time.

TAs may also continue to use the ISA in addition to the profiling as a more specific screening tool or as an additional engineering input to the profiling process for certain types of buildings.

Note:

The EPB methodology also contains criteria for accepting IEPs and ISAs previously submitted to TAs in relation to their earthquake-prone buildings policies or for other reasons. These criteria take into account factors such as the level of detail of the assessment and the degree of review or moderation that has been applied.

This does not include situations where earthquake-prone building notices were issued before the 2017 changes to the Building Act (i.e. notices issued under the previous section 124 of this Act) based on an ISA and/or IEP. In these cases, it is expected that buildings already identified as earthquake prone and issued with a notice requiring remediation work will have this notice replaced under the new provisions, so long as the building remains within scope of the Building Act. Obligations on owners to undertake remediation work and further engineering assessment to move out of earthquake-prone status will remain.

B1.3 Definitions and Acronyms

Critical structural weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be potential critical structural weaknesses.
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines
Earthquake-prone Building (EPB)	A legally defined category which describes a building that has been assessed as likely to have its ultimate capacity (which is defined in Regulations) exceeded in moderate earthquake shaking. In the context of these guidelines it is a building with an earthquake rating of less than 34*%NBS* (1/3 new building standard).
Earthquake rating	The rating given to a building as a whole to indicate the seismic standard achieved in regard to human life safety compared with the minimum seismic standard required of a similar new building on the same site. Expressed in terms of percentage of new building standard achieved (*%NBS*).
Earthquake Risk Building (ERB)	A building that falls below the threshold for acceptable seismic risk, as recommended by NZSEE (i.e. $\leq$67*%NBS* or two thirds new building standard)
(Earthquake) score	The score given to part of a building to indicate the seismic standard achieved in regard to human life safety compared with the minimum seismic standard required of a similar new building on the same site. Expressed in terms of percentage of new building standard achieved (*%NBS*).
IEP	Initial Evaluation Procedure
Importance Level (IL)	Categorisation defined in the loadings standard, AS/NZS 1170.0:2002 used to define the ULS shaking for a new building based on the consequences of failure. A critical aspect in determining new building standard.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.
NBS	New building standard – i.e. the standard that would apply to a new building at the site. This includes loading to the full requirements of the Standard.
NZS	New Zealand Standard
NZSEE	New Zealand Society for Earthquake Engineering
PAR	Performance Achievement Ratio
Potential critical structural weakness (pCSW)	Any structural weakness identified at the time of an ISA is a pCSW
pSSW	Potential severe structural weakness
Severe structural weakness (SSW)	A defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge. For an ISA, all severe structural weaknesses are considered to be potential severe structural weaknesses and are only expected to be noted when identified.
SLS	Serviceability limit state as defined in AS/NZS 1170.0:2002 (or NZS 4203:1992) being the point at which the structure can no longer be used as originally intended without repair
SSNS	Secondary structural and non-structural

Structural weakness. (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100*%NBS*. Note that an aspect of the building structure scoring less than 100*%NBS* but greater than or equal to 67*%NBS* is still considered to be a structural weakness even though it is considered to represent an acceptable risk.
T(L)A	Territorial (Local) Authority. Use of TA in this document is intended to describe a Council administering the requirements of the Building Act. A Council's role as a building owner is intended to be no different from any other building owner.
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings.
Unreinforced masonry (URM)	A member or element comprising masonry units connected together with mortar and containing no steel, timber, cane or other reinforcement

B1.4 Notation, Symbols and Abbreviations

Symbol	Meaning
$\%NBS$	Percentage of new building standard. Refer to Section BA.2.2
A_{p}	Plan area of building above storey of interest
A_{w}	Cross sectional area of all URM walls extending over full height of storey
b	Span of diaphragm perpendicular to direction of loading
D	Depth of diaphragm parallel to direction of loading
e_{d}	Distance between the storey centre of rigidity and the centre of mass for all levels above that storey
H	Height to the level being considered or height of the lower building as appropriate
h_{w}	Height of wall between lines of horizontal lateral restraint
I	Importance factor defined by NZS 4203:1992 used for the design of the building
k_{μ}	Structural ductility scaling factor defined in NZS 1170.5:2004
l_{w}	Length of wall between lines of positive lateral restraint
M	Material factor defined by NZS 4203:1992
$N(T, D)$	Near fault factor defined by NZS 1170.5:2004
R	Return period factor defined by NZS 1170.5:2004 based on the importance level appropriate for the building in accordance with AS/NZS 1170.0:2002
R_0	Risk factor used for the design of the building
S	Structural type factor defined in NZS 4203:1992
S_{p}	Structural performance factor defined in NZS 1170.5:2004
T	Fundamental period of a structure
t	Thickness of wall
Z	Seismic hazard factor defined by NZS 1170.5:2004
Z_{1992}	Zone factor from NZS 4203:1992 (for 1992-2004 buildings only)
Z_{2004}	Seismic hazard factor from NZS 1170.5:2004 (for post August 2011 buildings only)
$(\%NBS)$	Percentage of new building standard achieved. Refer to Section BA.2.2
$(\%NBS)_{\mathrm{b}}$	Baseline Percentage of new building standard. Refer to Section BA.2.2
$(\%NBS)_{\mathrm{nom}}$	Nominal Percentage of new building standard. Refer to Section BA.2.2
μ	Structural ductility factor defined by NZS 1170.5:2004

B2. Structural Weaknesses

B2.1 Potential Critical Structural Weaknesses (CSWs)

A structural weakness is an aspect of the building structure and/or the foundation soils that scores less than 100*%NBS*. Note that this includes aspects that score at least 67*%NBS*, even though these are considered to represent an acceptable risk.

For a DSA, the critical structural weakness (CSW) is the lowest scoring structural weakness. However, for an ISA all potential structural weaknesses are considered to be potential CSWs and are defined as either insignificant, significant or severe as follows:

Insignificant The potential CSW is not evident in the building and/or its parts, or it is of such an extent or nature that it is considered very unlikely to lead to loss of life and/or have an impact on neighbouring property and/or impede egress from the building when the building is subjected to severe earthquake shaking.

Significant The potential CSW is evident in the building and/or its parts, and it is of such an effect or nature that it is considered likely to lead to moderate loss of life and/or have a significant impact on neighbouring property and/or impede egress from the building when the building is subjected to severe earthquake shaking. For the potential CSW to be categorised as having a 'significant' consequence under this level of shaking it would need to be likely to result in deformations in the structure and/or its parts such that localised collapse should be considered a possibility.

Severe The potential CSW is evident in the building and/or its parts, and it is of such an extent or nature that it is considered likely to lead to significant loss of life and/or have a severe impact on neighbouring property and/or severely impede egress from the building when the building is subjected to severe earthquake shaking. For the potential CSW to be categorised as having a 'severe consequence' it would need to result in partial or complete collapse of the building and/or its parts.

B2.2 Potential Severe Structural Weaknesses

There are some severe structural weaknesses (SSWs) that experience from previous earthquakes shows are often associated with catastrophic pancake collapse or significant loss of egress. At the ISA level, these are referred to as potential SSWs that could result in significant risk to a significant number of occupants.

Note:

If <u>any</u> potential SSWs have been identified then careful consideration should be given before rating the building above 34*%NBS*.

It is important that the potential existence of these is noted as part of an ISA assessment even if the ISA earthquake rating is greater than the required target level (e.g. 34*%NBS*). Having said that, these potential SSWs are only expected to be noted when identified.

Note:

Potential SSWs should not be confused with the 'severe' performance category for the more general, potential CSWs scored within the IEP and described above.

It is considered reasonable to limit consideration of potential SSWs to buildings of three or more storeys, as it is unlikely that buildings with fewer storeys would contain sufficient occupants to be considered a significant risk in this context. Similarly it is unlikely that buildings with lightweight (e.g. timber) floors, with the possible exception of URM buildings, are of the type that would be particularly susceptible to pancake failure.

The potential SSWs considered to be indicative of possible significant loss of resilience and rapid deterioration of performance in severe earthquake shaking are:

1. **A weak or soft storey, except for the top storey**
   This SSW would have the potential to concentrate inelastic displacements in a single storey. It may be difficult to identify without calculation unless that storey height is much larger than for the other storeys and the element sizes have not been obviously increased to compensate.

2. **Brittle columns and/or brittle beam/column joints, the deformations of which are not constrained by other structural elements**
   Older multi-storey framed buildings with little or no binding reinforcement (beam/column joints), small columns and deep beams are particularly vulnerable to severe earthquake shaking. Once the capacity of such columns has been exceeded, failure can be expected to be rapid. When associated with a soft storey, the effect can be even greater.

3. **Flat slab buildings with lateral capacity reliant on low ductility slab-to-column connections**
   Although not common in New Zealand, this building type has a poor record in severe earthquakes overseas. The failure can be sudden, resulting in pancaking of floor slabs as the slab regions adjacent to the columns fail in shear. This SSW may be mitigated by special slab shear reinforcement and, to some extent, by the presence of slab capitals.

4. **No effective connection between primary seismic structural elements and diaphragms**
   Buildings with no obvious interconnection between primary seismic structural members, such as lateral load resisting elements and diaphragms, have little chance of developing the full seismic capacity of the structure in severe earthquakes, especially when the building has irregularities and/or the need to distribute actions between lateral load resisting elements.

5. **Seismically separated stairs with ledge and gap supports**
   This only needs to be an identifiable issue here for buildings with more than six storeys. It is considered that evacuation of lower height buildings will be relatively easily achieved through other means.

It is acknowledged that these SSWs and/or any mitigating factors that are present may only be recognisable from construction drawings. Therefore, an ISA based on a visual inspection only will not necessarily identify their presence. As stated earlier, the intent of these guidelines is that the items listed above should only be noted if they are observed: i.e. the engineer does not need to confirm that they are not present as part of an ISA.

Both the IEP (Table IEP-5) and the template letter provided in Appendix BC have provision for recording the presence of these SSWs as potential SSWs if they have been observed.

Note:

The SSWs highlighted above vary slightly from those which require specific consideration in a DSA. This is because some of the above (e.g. weak or soft storeys) can be assessed adequately in a DSA, while others listed for special consideration in a DSA may be difficult to identify from an ISA.

B3. Initial Evaluation Procedure (IEP)

B3.1 Background

The IEP is an integral part of the ISA process outlined in these guidelines. It has been designed to accommodate a varying level of knowledge of the structural characteristics of a building and its parts. It also recognises that knowledge of the building may increase with time.

This section provides guidance to engineers on what the IEP process should achieve, the level of experience required, the IEP's limitations, and how to address specific issues with the objective of achieving greater consistency in assessments. However, it should not be assumed that the higher level of guidance given will address all aspects and compensate for a lack of engineer experience and/or judgement.

Note:

Many buildings have now been assessed using the IEP. The changes made to this section of the guidelines are not expected, or intended, to significantly alter previous ratings for buildings if the judgement of experienced seismic engineers has been exercised.

The expectation is that the IEP will be able to identify, to an acceptable level of confidence and with as few resources as possible, most of those buildings that fall below the earthquake-prone building threshold without catching an unacceptable number of buildings that will be found to pass the test after a DSA. Therefore, an IEP earthquake rating higher than this threshold determined as part of a comprehensive ISA may be sufficient justification under the EPB methodology to confirm the building is not earthquake prone. Of course the IEP cannot take into account aspects of the building that remain unknown to the engineer at the time the IEP is completed. Therefore, it cannot be considered as reliable as a DSA.

Note:

The requirements for an ISA to enable it to be used as justification for raising the earthquake rating for a building identified as potentially earthquake prone using the profiling process are covered in the EPB methodology.

The IEP was developed in 2000 and first presented in June 2006. Since then thousands of buildings throughout New Zealand have been assessed using this procedure and a number of issues have become apparent. These include:

- the wide range of ratings achieved for the same buildings by different engineers
- undue reliance being placed on the results of the IEP, notwithstanding the stated preliminary/first-stage nature of this assessment
- an inappropriate level of accuracy being implied in some assessments
- lack of application of judgement in many assessments that is often evidenced by an unreasonably low score that even the engineer does not support
- varying skill level of engineers, many of whom lack the experience to apply the judgements required
- the incorrect view of some engineers that assessments are solely restricted to the issues specifically raised in the IEP and also do not include the secondary structural and critical non-structural components

- further confirmation from the Canterbury earthquake sequence of 2010/11 regarding the performance of buildings over a range of earthquake shaking levels, and
- a need to recognise that the importance level classification of a building may have changed since the design was completed.

B3.2 Outline of the IEP Procedure

An outline of the IEP procedure is shown in Figure B.2 (refer to Appendix BA for more details of the expected process, including Tables IEP-1 to IEP-5). It involves making an initial assessment of the standard achieved for an existing building against the minimum life safety standard required for a new building (the percentage of new building standard, or *%NBS*).

As knowledge of a particular building may increase with time, an IEP may be carried out several times for the same building and that the assessed rating may change as more information becomes available. Therefore, the level of information that a particular IEP has been based on is a very important aspect of the assessment and must be recorded so it can be referred to by anyone considering or reviewing the results.

For a typical multi-storey building, the process is envisaged as requiring limited effort and cost. It would be largely a visual assessment but supplemented by information from previous assessments, readily available documentation and general knowledge of the building.

The IEP should be repeated if more information comes to hand. It should also be adjusted until the engineer believes the result is a fair reflection of the standard achieved by the building.

The first step in the process is to survey the subject building to gather relevant data on its characteristics sufficient for use in the IEP.

The next steps involve applying the IEP to the building to determine the percentage of new building standard (*%NBS*) for that building.

The *%NBS* is essentially the assessed structural standard achieved in the building (taking into consideration all reasonably available information) compared with requirements for a new building and expressed as a percentage.
A *%NBS* of less than 34 (the limit in the legislation is actually one third) fulfils one of the requirements for the building to be assessed as earthquake prone in terms of the Building Act.
A *%NBS* of 34 or greater means that the building should be regarded as being outside the requirements of the earthquake-prone building provisions of the Building Act, although the TA will need to be satisfied that the assessment is valid. It is likely that the IEP will need to be at the more comprehensive end of the continuum, with review of drawings and interior inspections for the TA to be satisfied (refer to Part A and Figure B.1).
A *%NBS* of 67 or greater means that the assessment is indicating that the building should not be a significant earthquake risk, based on NZSEE recommendations.

Note:

It is important to realise that the reliability of the *%NBS* rating determined at the IEP level will depend on the level of information available during the assessment process. A rating determined by a DSA should generally be assumed to be more reliable than one from an ISA. As noted above an IEP prepared as part of a comprehensive ISA may be sufficient to confirm earthquake-prone building status. Refer to the EPB methodology for specific requirements.

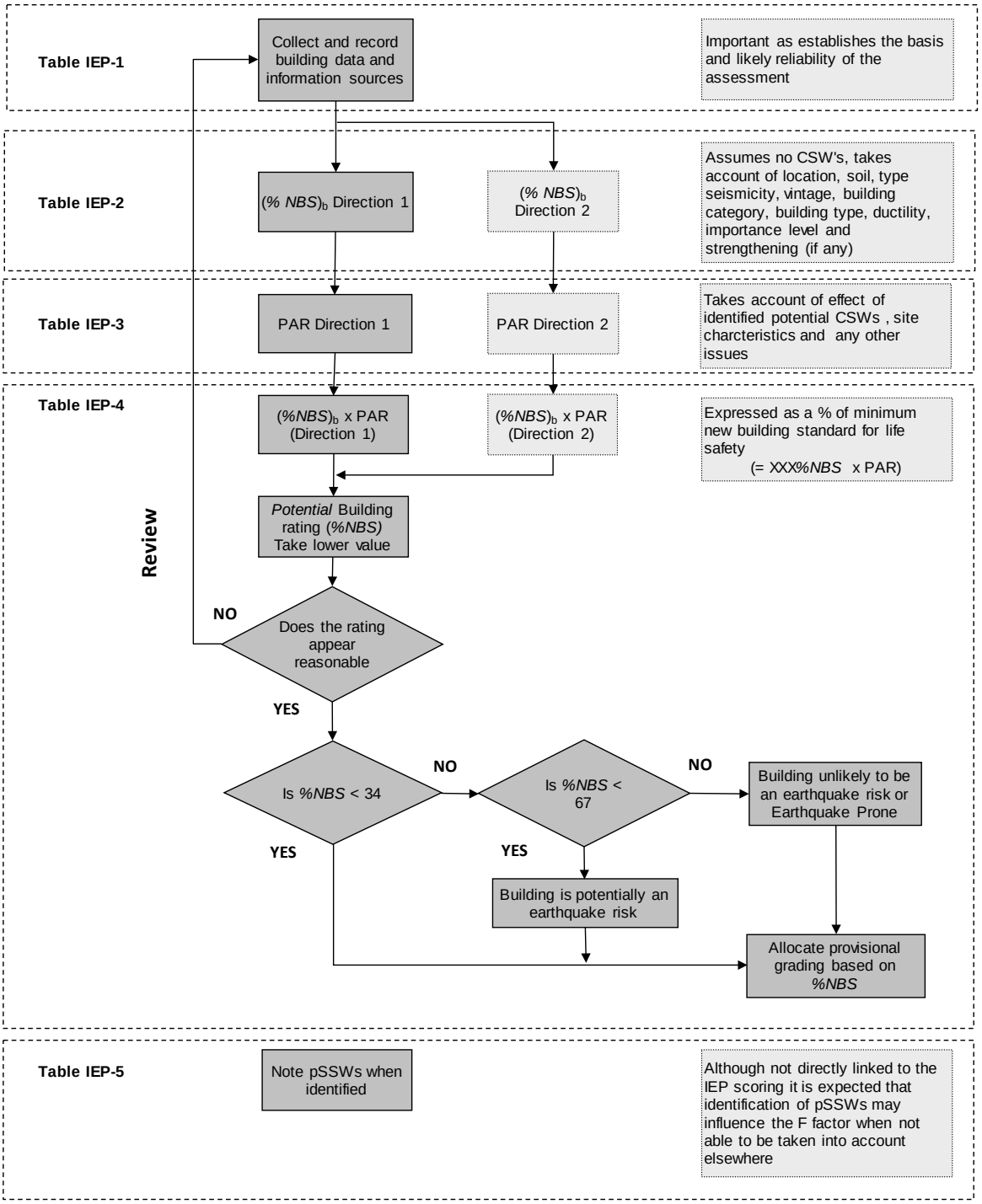

Figure B.2: Initial Evaluation Procedure

For URM buildings, the IEP can be used as presented but it may be difficult to apply in some circumstances. An attribute scoring method (refer to Appendix BB) is suggested as an alternative to Steps 2 and 3 of the IEP for these buildings. However, this method generally requires a greater knowledge of the building than typically expected or intended for an IEP.

B3.3 Level of Experience Required

The IEP is an attribute based and largely qualitative process which is expected to be undertaken by experienced engineers. It requires considerable knowledge of the earthquake behaviour of buildings, and judgement as to key attributes and their effect on building performance.

Therefore, it is essential that IEPs are carried out, or reviewed by, New Zealand Chartered Professional Engineers (CPEng) or their equivalent who:

- have sufficient relevant experience in the design and evaluation of buildings for earthquake effects to exercise the degree of judgement required, and
- have received specific training in the objectives of, and processes involved in, the IEP.

Note:

The IEP is based on the current standard for earthquake loadings for new buildings in New Zealand, NZS 1170.5:2004, as modified by the New Zealand Building Code. It is assumed that the person carrying out the IEP has a good knowledge of the requirements of this standard.

The IEP is not a tool that can be applied by inexperienced personnel without adequate supervision. Less experienced 'inspectors' can be used to collect data on the buildings provided that they have an engineering background so that the information collected is appropriate. However, the lower the experience of the inspectors, the greater the need for adequate briefing and review by experienced engineers before the IEP building rating is finalised.

B3.4 Implied Accuracy and Limitations

The IEP is a largely qualitative, score based assessment. It is based on generic building characteristics and is dependent on knowledge available at the time of the assessment.

Accordingly, *%NBS* ratings determined by an IEP should reflect the accuracy achievable and should not be quoted other than as a whole number. Except for the ranges 34 to 37% and 67 to 69% it is further recommended that the ratings are rounded to the nearest 5*%NBS*.

Engineers should also consider carefully before rating a building between 30 and 34*%NBS* or between 65 and 67*%NBS*. The ramifications of these ratings are potentially significant in terms of additional assessment required, perhaps for arguable benefit. Providing specific ratings above 100*%NBS* is also to be discouraged as these may provide an erroneous indication of actual performance. It is recommended that such ratings are simply stated as >100*%NBS*.

The score based nature of the IEP can also lead to very low ratings for some buildings. While these low ratings may correctly reflect the number of the potential CSWs present they may not truly reflect the expected performance of the building, particularly when considering against earthquake-prone building criteria. In such cases the engineer should be careful to advise his/her client of the limitations of the IEP and the recommendation that a DSA should be completed before any significant decisions are made.

In general terms, these guidelines recommend that engineers make sure building owners and other recipients of IEP assessment reports are fully aware of the limitations of the IEP process when discussing the results. These include the following:

- The IEP assumes a building has been designed and built in accordance with the building standard and good practice current at the time. In some instances, a building may include design features ahead of its time, leading to better than predicted performance and therefore warranting a higher rating. Conversely, some unidentified design or construction issues not picked up by the IEP process may result in the building performing not as well as predicted.
- An IEP can be undertaken with variable levels of information; e.g. exterior only inspection, structural drawings available or not, interior inspection, and so on. The more information available, the more reliable the IEP result is likely to be. Therefore, it is essential that the information sources available for the assessment are recorded and that the likely effect of including additional information, such as inspection of drawings, is reported.
- The IEP is intended to be somewhat conservative, identifying some buildings as having a lower *%NBS* rating than might be shown by subsequent detailed investigation to be the case. However, there will be exceptions, particularly when potential CSWs cannot be recognised from what can be largely a visual assessment of the building exterior for a less than comprehensive ISA.
- The IEP cannot take into account aspects of the building that are unknown to the engineer at the time the IEP is completed. (While this is also the case with a DSA, it is perhaps less likely given the greater level of information required.)
- An IEP is designed to assess the building against the ULS only. It does not assess against the serviceability limit state (SLS) as defined in AS/NZS 1170.0:2002. While this is consistent with the general seismic assessment approach in these guidelines of focusing only on aspects that could impact on life safety, it is important to bring this to the attention of the building owner or end user of the assessment results.
- For buildings designed after 1976, drawings and/or design calculations should be reviewed for an IEP unless it is a very preliminary screening. This is because of the increased complexities due to a significant change in construction materials and technology, structural systems, assumed ductility, sophistication of analysis and design procedures post the mid-1970s. Drawings should also be reviewed if the structural system is not clear or if the building has been strengthened, irrespective of the building's age.
- The IEP is an attribute based procedure where identified potential CSWs are penalised and the penalties are accumulated. For buildings with several potential CSWs, unrealistically low ratings may result, even after the full available adjustment for judgement. In such cases, the end users receiving the rating should be cautioned that the rating may not be truly representative of the seismic performance of the building (particularly around 34*%NBS*) and that a DSA is recommended.

- TAs are required to consider any information that might be available for a building. This means that they reserve the right to react to any additional information and adjust the seismic status of a building at any time, even though they may have carried out the process (that may have included an IEP) that conferred the original status. Therefore, reliance on an IEP for important decisions carries risks.
- The IEP process is only intended to focus on the building under consideration. It does not consider aspects such as the possible detrimental effects of neighbouring buildings (as current legislation assumes that these are the responsibility of the neighbour) or the hazards resulting from items that could be classified as building contents. However, these items may be important considerations for building owners and tenants, and should be brought to their attention if this is appropriate for the level of assessment being undertaken.

B3.5 Reconciling Differences in Assessment Results

Due to the qualitative nature of the ISA it should not come as a surprise that, in some circumstances, assessments of the same building by two or more experienced engineers may differ – sometimes significantly. This is to be expected, especially if the level of information available was different for each assessment.

In situations where assessment outcomes are significantly different, engineers should enter into a dialogue to understand the points of difference. These guidelines recommend that any differences in opinion regarding the IEP that cannot be resolved through discussion and sharing of information are resolved by the completion of a DSA. This should either be for the aspect under contention if it is appropriate to consider this in isolation, or for the building as a whole.

All judgements made need to be justified/substantiated if this is requested (e.g. by TAs). Judgements should preferably be recorded on the IEP sheets and included as part of the ISA.

B4. Issues Specific to Building Type and Era

B4.1 General

This section provides guidance on how to address some commonly encountered issues when carrying out IEPs. It is recognised that some of these issues will not be identifiable without access to drawings or an internal inspection of the building. However, this level of knowledge is consistent with the objectives that underpin the IEP.

Buildings should not be penalised in the IEP unless there is some evidence that the issue is present. The IEP can be amended at any time if further information comes to hand. Note also the recommendation in Section B4.3 to review drawings for post-1976 buildings and the requirements for engineering assessments for earthquake prone status given in the EPB methodology.

Judgement decisions on particular aspects of the IEP can be supported by calculations. This would be expected to lead to a more reliable (but still potential) rating from the IEP without the full cost of a DSA. However, engineers should be careful to avoid over-assessment in one area at the expense of another. The potential rating for a building as a whole from an IEP must reflect the best judgement of the engineer, taking into account all aspects known to the engineer.

B4.2 Site Characteristics

Identified site characteristics (including geohazards and potentially at-risk neighbouring buildings, etc.) that could have a direct impact on the building and, as a result, could lead to the building presenting an enhanced risk to building occupants, those in the immediate vicinity of the building, or to adjacent property must be recorded on the IEP forms and in the covering letter. Therefore, the engineer needs to be cognisant of the site's terrain setting and have an awareness of the possible geohazards and other hazards that could impact on the building.

In the IEP, penalties are applied based on the potential effects on the building in a severe earthquake. Therefore the penalty should not be reduced simply because the hazard is not expected to initiate at levels of shaking implied by the *%NBS* rating.

Penalties are generally not applied for hazard sources located outside the site. This includes geohazards such as rockfall from above, rolling boulders, landslide from above and tsunami and hazards resulting from neighbouring buildings (e.g. adjacent URM walls and parapets).

Note:

This is consistent with the philosophies underlying the concept of earthquake-prone buildings within the Building Act, where the focus is on the building and its effect on its neighbours rather than the risk presented by neighbouring property.

Site characteristics that are to be considered, and that will potentially attract a penalty, include:

- excessive ground settlement
- liquefaction
- lateral spreading, and
- landslide from below.

However, penalties should only be applied when these characteristics would lead to building damage to an extent that would result in the potential enhanced risks outlined above and when there is some evidence that the particular hazard exists. For example, a building should not be penalised solely because it is located on a slope. For such a building to attract a penalty there must be evidence of prior slope instability or knowledge of instability, and the potential loss of support of the building must be such that it would be likely to lead to the enhanced risks outlined.

Regarding liquefaction, the Canterbury earthquakes have provided evidence that this on its own is unlikely to lead to a risk to life in light timber buildings or other low rise (less than three storeys) buildings that are well tied together and are therefore likely to maintain their integrity after significant settlement occurs. However, unstrengthened URM buildings are considered to be particularly vulnerable to ground settlement of the extent expected if liquefaction occurs.

Issues relating to ground amplification are assumed to be dealt with when setting the subsoil conditions in the determination of $(\%NBS)_{\mathrm{nom}}$. However, as with any other issue, the engineer is required to make a judgement call regarding any additional impact on the score that may be appropriate, over and above any allowance in the procedure.

Engineers are referred to geohazard assessments that have been carried out for TAs and regional councils to identify the potential hazards that are likely to be appropriate for the site in question. These are typically in the form of hazard maps. Engineers are also referred to Table BA.4 and to Section C4 of these guidelines for further discussion on geotechnical matters.

B4.3 Post-1976 Buildings

B4.3.1 General

Note the following for buildings designed after 1976:

- From the mid-1970s, perhaps coinciding with the introduction of the modern earthquake design philosophies into Standards and the greater availability and use of computer programs for structural analysis, quite complex structural configurations and lateral load paths were often adopted. Whereas for buildings built earlier than this it might have been possible to identify a generic structural form from an exterior inspection, it is often difficult to pick this for post-1976 buildings.

  For this reason it is recommended that the engineer reviews drawings and/or design calculations of post-1976 buildings for an IEP, unless it is only a preliminary screening or drawings cannot be located. In such cases it might be best to err on the side of caution if it is suspected that there might be issues with the structural system.

- Consideration should also be given to the following:
  - location and clearance to non-structural infill walls (refer to Section B4.7)
  - poorly configured diaphragms (Section B4.7.7)
  - gap and ledge stairs, particularly if these are in a scissor configuration (Section B4.7.8)
  - non-ductile columns (Section B4.3.2)
  - unrestrained/untied columns (Section B4.3.3), and
  - detailing and configuration of shear walls (Section B4.3.4).

It is not expected that the issues outlined above will result in an earthquake-prone designation, although this cannot be completely discounted.

Also note that post-1976 buildings can feature potential CSWs that relate to detailing issues rather than configurational SWs relating to regularity. Examples of these can include:

- heavily penetrated floor diaphragms (typically reinforced with welded wire mesh) which may lack adequate collector elements back to the lateral load resisting structure
- exterior columns without sufficient connection back into the supporting diaphragm
- non-structural infill walls with some movement allowance but an insufficient allowance to meet current code requirements
- egress/access stairs which may not have sufficient displacement capacity for the expected inter-storey drifts
- steel tension braces which may be vulnerable to fracture at threaded ends, where there may be insufficient threaded length to allow the required inelastic drift to develop, and
- detailing no longer considered to provide the level of ductility assumed at the time of design or previous strengthening.

B4.3.2 Non-ductile columns

Investigation into the collapse of the CTV building during the 22 February 2011 Christchurch earthquake highlighted the potential for incorrect interpretation of requirements for secondary columns in buildings designed using NZS 3101:1982. These requirements were clarified in NZS 3101:1995, but there is potential for non-ductile secondary columns in buildings designed during the period broadly from 1982 to 1992.

Such detailing is unlikely to make the building rate less than 34*%NBS*, unless the columns are already highly stressed under gravity loads. However, the presence of non-ductile columns should result in the building being rated less than 67*%NBS*.

B4.3.3 Unrestrained/untied columns

The evidence would suggest that there are a number of multi-storey buildings constructed in the 1980s that have perimeter frames where the columns are not adequately tied back into the floor diaphragm. In some cases, as noted in Section B4.7.7, the floor mesh taken over the beam reinforcement provides the sole means of restraint. The lack of column ties is likely to lead to a rapid reduction in capacity of the columns once beam elongation and/or fracture of the slab mesh has occurred.

The lack of column ties back to the floors is unlikely to make the building rate less than 34*%NBS* but should result in a rating less than 67*%NBS*.

B4.3.4 Concrete shear wall detailing and configuration

The performance of concrete shear wall buildings in the Canterbury earthquakes has indicated that current detailing for ductility (spacing and positioning of wall ties) may not be sufficient when the wall is subjected to significant nonlinear behaviour. Asymmetric walls (i.e. C and L shaped walls) were also shown to be problematic when capacity design procedures were not applied. New provisions for wall detailing are being developed. When they are finalised the *%NBS* for existing buildings will need to be compared against these requirements.

This issue is unlikely to cause post-1976 buildings to be rated less than 34*%NBS*, but could potentially reduce the rating below 67*%NBS*.

B4.4 Timber Framed Buildings

The Christchurch earthquake sequence of 2010/11 confirmed what has been long known that timber framed residential and small commercial buildings generally perform extremely well in earthquakes and that, even when significantly distorted due to ground movements, the risk of a significant life safety hazard as a result is low.

Buildings of this type have been shown to have significant inherent capacity and resilience (beyond the ULS as might be determined by consideration of NZS 3604:2011 requirements) which means that they should rarely be found to be less than 34*%NBS* unless they are located on a slope and have substructures that are poorly braced and/or poorly attached to the superstructure.

Buildings located on flat sites and poorly attached to their foundations may come off their foundations. However, although this may lead to significant damage, this is unlikely, on its own, to result in fatalities, particularly if the floor is less than 600 mm above the ground. These buildings are rarely completely reliant on their roof diaphragms unless the spacing of parallel walls is large.

Whether or not these building are potentially earthquake risk, i.e. less than 67%, will depend on issues such as:

- site characteristics
- age (i.e. is the building likely to have been engineered? Correct application of non-specific design requirements such as NZS 3604:2011 may be considered as achieving this.)
- adequacy of connection between subfloor and super structure
- poorly braced basement structures
- walls lined with materials of little reliable capacity
- excessive spacing between walls
- condition (decayed timber, etc.)
- excessive stud height
- roof weight.

Larger timber framed buildings such as churches, school and church halls and commercial buildings have also been shown to have inherent capacity and resilience and to perform in earthquakes well above what their ULS capacity as assessed in comparison to new building requirements might suggest. These buildings are typically characterised by larger spans,

greater stud heights, greater spacing between walls, and fewer connection points between building elements than for the smaller, more cellular buildings discussed above. Nevertheless, these buildings should also rarely be classified as less than 34*%NBS* unless the following are evident, and then judgement will be necessary to determine the likely effect:

- missing load paths (e.g. open frontages, particularly at ground floor level of multi-storey buildings)
- obvious poor connections between elements (e.g. between roof trusses and walls)
- lack of connection between subfloor and super structure and poorly braced basement structures for building on slopes
- walls lined with materials of little reliable capacity
- heavy roofs
- likely effect on non-structural elements of a particularly hazardous nature (e.g. effect of building racking on large areas of glazing or of brick veneers adjacent to egress paths).

At the earthquake risk level the other aspects given above for the smaller buildings will also be relevant.

To reflect these observations the following parameters may be assumed for timber framed buildings in the IEP:

- The structural performance factor, S_p, may be taken as 0.5.
- For most buildings of this type, plan irregularity may be assumed to be insignificant.
- Unbraced subfloors for buildings on flat ground may be assumed insignificant if the height above the ground is less than 600 mm.
- No penalty should typically be applied for site characteristics; e.g. liquefaction (refer also to Section B4.2).
- Ductility, μ, is equal to 2 and 3 for pre and post 1978 buildings respectively.

The judgement factor, or F Factor, should be chosen to reflect the overall expected performance of the building based on the observations set out above. For timber framed structures of a cellular configuration, F Factor values approaching the upper limit should be used.

B4.5 Single Storey Steel Industrial Structures

Single storey industrial structures with profiled steel roofing and wall cladding typically perform well in earthquakes. These buildings typically have steel portals carrying the seismic loads in one direction and steel bracing (roof and wall) in the other.

Such structures should rarely be found to be less than 34*%NBS*. Although the cladding cannot be relied on in a design sense, it is nevertheless likely to provide reasonable capacity if bracing is missing.

Weaknesses that could potentially affect the capacity of these structures include:

- missing wall and/or roof bracing
- lack of lateral flange bracing to portals
- open walls with little obvious bracing
- non-capacity designed bracing connections.

B4.6 Tilt-up Industrial and Commercial Structures

Concrete tilt-up panels inherently provide significant lateral capacity to a building. However, the capacity that can be utilised is very dependent on the connections from the panels to the structure (typically the roof structure) and the capacity of the roof diaphragm.

If complete load paths can be seen (including the roof diaphragm), with no obvious problems with the connections (e.g. missing or obviously undersized bolts, poor welds to weld plates), such buildings are unlikely to be less than 34*%NBS*.

Non-ductile mesh as the sole means of panel reinforcement could lead to an issue for panels under face loading.

Any identified issues should be subjected to further investigation. The heavy nature of these buildings and possible lack of redundancy means that they are unlikely to perform well when the earthquake shaking is greater than moderate if:

- any failures occur in connections
- the diaphragms have insufficient capacity to transfer loads (e.g. such as might be necessary when large wall openings are present), or
- there are reinforcement fractures in the panels.

It is recommended that an inspection of the interior of such buildings be included when completing an IEP.

B4.7 Other Building Elements

The behaviour of secondary structural and non-structural (SSNS) elements, where the failure of these could present a significant life safety hazard or damage to neighbouring property, must be considered in the overall assessment of the building.

The rationale for what elements are expected to lead to a significant life safety hazard and therefore need to be included in a seismic assessment is provided in Part A.

Parts of buildings that should be included in an assessment include, but are not limited to:

- URM parapets and walls (Section B4.7.1), and chimneys (Section B4.7.2)
- masonry veneers above a public thoroughfare, neighbouring buildings or egress routes (Section B4.7.3)
- masonry infill panels (Section B4.7.4)
- heavy non-loadbearing partition walls (Section B4.7.5)
- precast panels located over egress routes, public areas or neighbouring buildings (Section B4.7.6)
- diaphragms (Section B4.7.7)
- stairs (Section B4.7.8)
- support frames for cladding systems including curtain walls (Section B4.7.9), and
- heavy and large items of building services plant, tanks, etc. (Section B4.7.10).

Note:

These elements also fall within the scope of "parts" under the Building Act for the purposes of determining whether or not a building is earthquake prone (refer to Part A).

The behaviour of general building services is not intended to limit the earthquake rating of a building.

B4.7.1 Unreinforced masonry parapets and walls

The presence of URM walls (irrespective of whether or not these are bearing walls) or cantilevering parapets should be sufficient grounds to rate a building as less than 34*%NBS*, at least until the stability of the wall or the effectiveness of the restraint of the masonry can be confirmed.

Appendix BB contains specific provisions for URM buildings – the attribute scoring method – which is intended for use in conjunction with the IEP. However, as noted earlier, this method generally requires a greater level of knowledge of a building than is typically expected or intended for an IEP.

B4.7.2 Chimneys

Experience indicates that chimneys can be vulnerable in earthquake shaking and can score less than 34*%NBS*; particularly if they are unreinforced or poorly restrained back to the building. Failure of such chimneys has led to fatalities in past earthquakes in New Zealand and this should be reflected in the IEP. The following approach is recommended for assessing chimneys and determining their effect on the building rating.

If a building has a chimney that is not restrained by the roof structure or other fixing at the roofline, the building should be assigned an earthquake rating of less than 34*%NBS* and the F Factor in Table IEP-3 set accordingly.

A building with a chimney should also be assigned an earthquake rating of less than 34*%NBS* (and the Factor F in Table IEP-3 set accordingly) if the chimney meets **ALL** of the following criteria:

- it is constructed of URM or unreinforced concrete, **AND**
- the ratio of the height of the chimney (measured vertically from the chimney intersect with the lowest point on the roofline to the top of the chimney structure, and excluding any protruding flues or chimney pots) and its plan dimension in the direction being considered is more than:
  - 1.5 when $ZR \geq 0.3$, **or**
  - 2 when $0.2 < ZR < 0.3$, **or**
  - 3 when $ZR \leq 0.2$

  where Z and R are as defined in NZS 1170.5:2004, **AND**
- if either or both of the following apply:
  - there is any possibility that the chimney could topple onto an egress route, entrance way, over a boundary (including over a street frontage), over any public/private access way, or more than 2 m down onto an adjoining part of the building, **and/or**

- the roofing material comprises concrete masonry, clay tiles or other brittle material, unless suitable sheathing (extending horizontally at least the height of the chimney away from the chimney) has been laid within the ceiling space to prevent the roofing material and collapsed chimney from falling through the ceiling.

If the engineer determines a building with a chimney is less than 34*%NBS*, he or she should record in the IEP the particular issues (from the options listed above) that led to this rating.

B4.7.3 Masonry veneers

If a masonry veneer above an egress route or public space became separated from the supporting structure, this would likely create a significant life safety hazard.

Heavy veneers in these locations consisting of stone or brick and that are thicker than the standard 110 mm units require specific investigation.

Veneers that have ties (from any code era) are considered to rate ≥34*%NBS*. Accordingly, if the presence of ties to veneers above egress routes and public thoroughfares is indicated from scanning, then a rating ≥34*%NBS* can generally be taken. For buildings of three or more storeys, the engineer will need to verify the condition and effectiveness of the veneer ties by intrusive investigation. This can be undertaken as part of an ISA or recommended for inclusion within a subsequent DSA.

A building with masonry veneer should not be rated ≥67*%NBS* without first verifying the condition and effectiveness of ties by intrusive investigation.

B4.7.4 Masonry infill panels

Infill masonry panels are typically used to form boundary walls within concrete and encased steel frames.

Prior to the early 1970s, infill masonry walls typically comprised unreinforced brick and concrete masonry blocks, mortared up against the framing elements with no seismic separation.

From the early 1970s, infill walls (typically in reinforced blockwork) were separated from the primary structure to prevent the walls from carrying in-plane shear and therefore participating in the lateral load resisting system. Prior to 1992, the separation requirements were much less than subsequently required. Gaps of 10 mm to 20 mm were common and in many instances filled with sealants or fillers that were only partially compressible.

However, once these gaps have been taken up, the walls will act as shear walls to the limit of their capacity. Problems arise because of the irregular layout of the secondary structural wall panels, both in plan and over the height of the structure. The eccentricities that result can be severe. If gaps have been provided it is unlikely that the building will score less than 34*%NBS* but the expected performance at higher levels of shaking will be dependent on the wall layouts and the type of primary structure present. The effects will be greater for more flexible primary structures such as moment resisting frames.

Infill walls not separated from the primary structure should be considered as shear walls of uncertain capacity and scored accordingly. Their impact on the regularity of the structure

(horizontal and vertical) should be carefully considered. In many cases it may be difficult to determine the effect, and a DSA is recommended.

As noted in the previous section, the potential for masonry infill panels to fail out of plane when there are gaps between the panel and the perimeter framing (particularly adjacent to egressways, public spaces and thoroughfares) should be assessed and appropriate scoring and recommendations made.

B4.7.5 Heavy non-loadbearing partition walls

Heavy non-loadbearing partition walls typically comprise:

- unreinforced clay brick masonry
- hollow clay brick masonry (which can be filled or unfilled, reinforced or unreinforced), or
- concrete block masonry (which can be solid or hollow, unfilled, partially filled or fully filled, and either reinforced or unreinforced).

Common issues that affect the seismic behaviour of heavy non-loadbearing partition walls include:

- insufficient or absent restraint at the tops of the walls to prevent out-of-plane movement, and
- insufficient gaps between ends of walls and the main structure to allow for inter-storey drift.

Heavy non-loadbearing partition walls will generally be < 34*%NBS* without a detailed investigation and supporting calculations.

B4.7.6 Precast panels

Issues relating to precast panels include:

- whether they are primary structure or secondary structure
- their size
- the effect they may have on the building structure regularity (horizontal and/or vertical) if they are not adequately separated from the expected structural deformations of the structure, and/or
- the hazard they may present if they were to fall from the building. For this situation the focus will be on panels located over egress routes, public areas or neighbouring buildings.
- the detailing of the connections to the structure, and
- the condition of the fixings.

When precast panels have clearance to the building structure or are obviously built into the structural system, they may be assumed to score ≥34*%NBS*.

In order to score above 67*%NBS*, panels should be clearly recognisable as either primary or secondary structure. If it is determined that they are primary structure, the connections to the structure would be expected to be robust reflecting the actions that would need to be transferred. If secondary structure, then separations to reflect the need to accommodate the flexibility of the structure and, at the same time, provide restraint to the panels under face

loading would be expected. Slotted holes should be inspected to ensure that bolts are appropriately positioned in the slots and that the connection is free to slide.

Note:

When it is apparent that precast panels do not have separation from the primary lateral structure, their effect on the primary structure (e.g. consideration of plan irregularity) should be considered. From an ISA point of view this has the same effect as assuming the panels are part of the primary structure.

B4.7.7 Diaphragms

The role of diaphragms in a building may be complex. All diaphragms act as load collectors distributing lateral load to the lateral load resisting elements. Where the lateral load resisting system changes (e.g. at basements or transfer levels) the diaphragms may also act as load distributors between the lateral load resisting elements. In the post elastic range, progressive inelastic deformations in lateral load resisting elements may impose significant internal forces detrimental to both the diaphragms and the behaviour of the lateral load resisting elements.

In addition to the configuration (plan irregularity) issues noted in Figure BA.5 and Table BA.4 there are also issues relating to diaphragm detailing that could affect the seismic performance of the building as a whole. These include:

- poor placement of penetrations interrupting potential load/stress paths
- inadequate load paths (e.g. no chords which lead to little in-plan diaphragm moment strength) or lack of means to transfer loads into the lateral load resisting system (e.g. lack of “drag” steel to concrete walls)
- incomplete or inexistent means of load transfer, e.g. missing roof bracing elements
- inadequate capacity in the diaphragm and its connections, and
- poor connections from non-structural elements to the diaphragms (e.g. connections from the tops of brick walls to the diaphragms).

The potential behaviour of precast floor diaphragms (and in particular hollowcore floors) has received much attention over the last decade and evidence of diaphragms under stress was seen after the 2010/2011 Christchurch earthquake sequence and in Wellington buildings following the 2016 Kaikoura earthquake. This included:

- cracking in floor toppings and fracture of floor mesh (a particular issue if mesh is the sole reinforcement in the topping), and
- separation of the perimeter concrete frames from the diaphragm, e.g. after elongation of the concrete beams, fracture of the topping reinforcement or lack of ties to the perimeter columns.

Diaphragm capacity issues are unlikely to become an issue until the earthquake shaking becomes severe so are unlikely, on their own, to cause the building to be rated less than 34*%NBS*.

The engineer will need to use his or her judgement to assess the effect of missing elements and will need to check for the existence of other, less direct or less desirable load paths for transferring loads before determining that the building’s rating is <34*%NBS*.

B4.7.8 Stairs

Stairs required for egress or above occupied areas should be considered as part of an IEP and where appropriate noted as pSSWs.

The experience of the 2010/2011 Canterbury earthquake sequence showed that some types of stairs may be vulnerable in earthquakes. The arrangement that was shown to be particularly vulnerable was the "gap and ledge" stair where a heavy stair flight (typically precast concrete) is vertically supported on a corbel, typically with a seating less than 100 mm, and with or without a definite gap. Monolithic concrete stairs in multi-storey reinforced concrete or steel frame buildings could be similarly vulnerable.

Such details, on their own, are very unlikely to make a building rate <34*%NBS* unless the stair flights are precariously supported, but their presence should result in a rating less than 67*%NBS*.

The ability of the connections of steel stair framing to withstand the inter-storey drifts needs consideration. Generally this would be very unlikely to make a building rate less than 34*%NBS.*

Where concrete stair flights are cast integrally with the floors of a building, they may influence the response of the primary structure in an earthquake, and in turn be susceptible to failure. This needs to be considered as a potential structural weakness (refer to Table BA.4).

B4.7.9 Lightweight cladding systems including curtain walls

The failure of individual panes of glass or individual lightweight cladding panels is not typically considered to be of sufficient severity to meet the criteria associated with a significant life safety hazard or damage to adjacent buildings, as defined in these guidelines. What would meet these criteria are the failure of:

- a glazing or cladding support system where large sections could fall, or
- large glass panels adjacent to egress routes.

The more likely cause for system failure of these elements (i.e. that would lead to them falling from the building) is failure of the connections to the structure due to undersizing and inadequate allowance for building movements. At the time an ISA is completed the connections are not generally available to view. It is considered that the connections are unlikely to score <34*%NBS* but that a score greater than 67*%NBS* should not be assumed unless the connections have been viewed and confirmed as reasonable.

Therefore it is considered reasonable to assume that these elements are unlikely to make the building rate <34*%NBS* but should result in a rating less than 67*%NBS* unless the connections are able to be assessed.

B4.7.10 Building services plant, tanks, etc.

In-ceiling building services and lightweight services in general are not intended to influence a building's rating. The exceptions include heavy items such as large tanks and large items of plant which if they were to lose support could fall and create a significant life safety hazard.

If heavy items are precariously supported or have no restraint, and their failure could lead to a significant life safety hazard, they should be scored less than 34*%NBS*. If restraints have been provided it is considered reasonable to assume that the score is greater than 34*%NBS*. Robust connections and/or supports would need to be present before scoring these items over 67*%NBS*.

Tanks with hazardous contents will require special consideration.

B5. Reporting

B5.1 Covering Letter

The way the results of an ISA are reported is extremely important to make sure these are appropriately interpreted and their reliability is correctly conveyed.

Recipients of an ISA must be warned of its limitations and the need to proceed to a DSA if any decisions reliant on the seismic status of the building are contemplated.

To avoid any misinterpretation by building owners and/or tenants of an ISA result it is recommended that the ISA (typically expected to be in the form of an IEP) is accompanied by a covering letter. This letter should describe the:

- building
- scope of the assessment and information available for this
- rationale for the various decisions made
- limitations of the process, and
- implications of the result.

Refer to Appendix BC for a template covering letter showing how these aspects might be addressed.

When the results of a TA-initiated ISA are being reported, building owners must be advised of the limitations of the process. If the building has been found to be not earthquake prone and the IEP report is provided, it should be made clear that the primary objective was to determine the earthquake-prone status and not necessarily the rating for the building.

B5.2 Technical Summary

A stand-alone technical summary should also be provided as part of reporting an ISA. This should follow the template that can be found in Part A to achieve consistency in the reporting of assessment outputs and to allow comparisons between assessments of multiple buildings.

Note:

Providing a technical summary in a consistent form is considered an important part of an ISA and is a requirement of an engineering assessment completed in accordance with the EPB methodology to identify earthquake-prone buildings. This summary is considered to be essential for TAs managing the requirements of the earthquake-prone building legislation, and potentially very useful for owners of multiple buildings, and for future engineers looking at the same building.

The same template should be used to record the results of a DSA.

References

AS/NZS 1170.0:2002. *Structural design actions – Part 0: General principles*, Standards Australia/Standards New Zealand.

EPB methodology. The methodology to identify earthquake-prone buildings, Ministry of Business, Innovation and Employment, July 2017

New Zealand Building Act (2004) as amended by the Building (Earthquake-prone Buildings) Amendment Act 2016 and including Building (Specified Systems, Change of Use, and Earthquake-prone Buildings Regulations, 2017, New Zealand Legislation.

New Zealand Building Code Clause B1- Structure, First Schedule of the Building Regulations 1992, New Zealand Legislation

NZS 1170.5:2004. *Structural design actions, Part 5: Earthquake actions – New Zealand,* Standards New Zealand, Wellington, NZ.

NZSS 1900.8:1965 (1965). *Model building bylaw: Basic design loads*, Standards Association of New Zealand, Wellington, NZ.

NZS 3101:1982 (1982). *Code of practice for the design of concrete structures*, Standards Association of New Zealand, Wellington, NZ.

NZS 3101:1995 (1995). *Concrete structures standard*, Volume 1 Code of Practice and Volume 2 Commentary. Standards New Zealand, Wellington, NZ.

NZS 3604:2011 (2011). *Timber-framed buildings,* Standards New Zealand, Wellington, NZ.

NZS 4203:1976 (1976). *Code of practice for general structural design and design loading for buildings*, Standards Association of New Zealand, Wellington, NZ.

NZS 4203:1984 (1984). *General structural design and design loadings for buildings*, Standards Association of New Zealand, Wellington, NZ.

NZS 4203:1992 (1992). *General structural design and design loadings for buildings*, Standards New Zealand, Wellington, NZ.

NZSEE (2006). *Assessment and improvement of the structural performance of buildings in earthquakes, Incl. Corrigenda 1, 2, 3 and 4*, New Zealand Society for Earthquake Engineering (NZSEE), Wellington, NZ.

NZSS 95:1939 (1939). *New Zealand Standard code of buildings by-laws*, New Zealand Standards Institute, Wellington, NZ.

Appendix BA: Initial Evaluation Procedure – IEP

This appendix summarises the eight steps involved in the IEP and includes the necessary worksheets (Tables IEP-1 to IEP-5). It also includes information tables and figures required for this assessment.

Section BA.2 provides further guidance and commentary to support the IEP process.

Note:

Working spreadsheet versions of Tables IEP-1 to IEP-5 are available from the EQ-assess website at www.EQ-Assess.org.nz.

BA.1 Summary of Step-by-Step Procedures

Step 1 Collect general information

Use Table IEP-1.

1.1 Add photos of the building exterior for all visible exterior faces, showing features.

1.2 Draw a rough sketch of the building plan that can be ascertained from the exterior of the building, noting relevant features.

1.3 List any particular features that would be relevant to the seismic performance of the building.

Note if the building has been strengthened in the past and the level of strengthening targeted at that time.

Record the characteristics of any adjacent buildings if the separation is not sufficient to prevent pounding.

1.4 Note any information sources used to complete the assessment.

Note:

As noted in Section B3.2, Steps 2 and 3 may not be appropriate for URM buildings. Appendix BB provides an attribute scoring method that can be used for these buildings.

Step 2 Determine baseline percentage of new building standard $(\%NBS)_b$

Use Table IEP-2. **An assessment is required for each orthogonal direction.**

2.1 Determine $(\%NBS)_{nom}$ = A x B x C x D as shown (making any adjustments to account for reaching minimum lateral coefficients in either the design or current Standards), unless the building is post-2004; in which case set this equal to 100% and go to Step 2.7.

2.2 a) Refer to NZS 1170.5:2004 for near fault factor.

b) Calculate near fault scaling factor.

2.3 a) Refer to NZS 1170.5:2004 for hazard factor.

b) Calculate hazard scaling factor (Factor F).

2.4 a) Refer to original design for design importance level if date of design is post-1984; otherwise set to 1.0. For buildings designed prior to 1976 and known to have been designed as a public building I may be taken as 1.25.

b) Refer to original design for design risk factor or set to 1.0.

c) Determine current return period factor from Table BA.1.

d) Calculate return period scaling factor (Factor G).

2.5 a) Assess available ductility for building (refer to Table BA.2 for maximum allowable).

b) Obtain ductility scaling factor (Factor H) from Table BA.3. Set to 1.0 for post 1976 buildings.

2.6 a) Obtain structural performance factor from Figure BA.2, or from the appropriate materials Standard, whichever requires the greater value.

b) Calculate structural performance scaling factor (Factor I).

2.7 $(\%NBS)_{\mathrm{b}}$ = $(\%NBS)_{\mathrm{nom}}$ x E x F x G x H x I as shown.

Step 3 Determine performance achievement ratio (PAR)

Use Table IEP-3. An assessment is required for each orthogonal direction.

3.1 to 3.5
Assess effect on structure of each potential CSW.
(Choose from the factors given – do not interpolate. Note that a 'severe' categorisation for the general factors considered in the IEP should not be confused with a potential SSW determined in Step 8 and as described in Section B2.)

3.6 Note that the effect of any known potential CSWs not listed is intended to be via Factor F.

Refer to Section BA.2.3 and B4 for further guidance as required.

3.7 PAR = A x B x C x D x E x F as shown.

Step 4 Determine the percentage of new building standard, *%NBS*

Use Table IEP-4.

4.1 Compare product of PAR x $(\%NBS)_{\mathrm{b}}$ for each direction.

4.2 *%NBS* = the lowest value of PAR x $(\%NBS)_{\mathrm{b}}$.

4.3 Review and adjust as necessary.

Step 5 Is the earthquake rating less than 34*%NBS*?

Use Table IEP-4. Assess on basis of *%NBS* in Step 4.

Step 6 Is the building potentially earthquake risk?

Use Table IEP-4. Assess on basis of *%NBS* in Step 4.

6.1 If *%NBS* $\geq$ 67 then it is not considered to be a significant earthquake risk.

6.2 If *%NBS* < 67 then a DSA is recommended before confirming the building as earthquake risk.

Step 7 Provide provisional grading based on IEP

Use Table IEP-4. Assess on basis of *%NBS* in Step 4.

7.1 Grade building based on *%NBS* earthquake rating using the relative risk table provided in Part A. Use the lowest result from consideration of both orthogonal directions.

Step 8 Note any identified potential SSWs that could result in significant risk to a significant number of occupants

Use Table IEP-5.

8.1 If the number of storeys is less than or equal to three it is assumed that the number of occupants is not significant and no further consideration of this issue is required for the ISA.

8.2 If the floors and/or the roof are not of heavy (concrete) construction it is assumed that the risk is not significant and no further consideration of this issue is required for the ISA.

If the number of storeys is greater than three and the floors and/or roof are of heavy construction then the presence of the listed potential SSWs should be noted. Note that the potential stair issue is only activated in the list if the number of storeys is greater than or equal to six.

Table IEP-1: Initial Evaluation Procedure – Step 1

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} **Page 1**

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out in the "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

Street Number & Name:		**Job No.:**	
AKA:		**By:**	
Name of building:		**Date:**	
City:		**Revision No.:**	

Table IEP-1 Initial Evaluation Procedure Step 1

Step 1 - General Information

1.1 Photos (attach sufficient to describe building)

NOTE: THERE ARE MORE PHOTOS ON PAGE 1a ATTACHED

1.2 Sketches (plans etc, show items of interest)

NOTE: THERE ARE MORE SKETCHES ON PAGE 1a ATTACHED

1.3 List relevant features (Note: only 10 lines of text will print in this box. If further text required use Page 1a)

1.4 Note information sources Tick as appropriate

Visual Inspection of Exterior	☐	Specifications	☐
Visual Inspection of Interior	☐	Geotechnical Reports	☐
Drawings (note type)	☐	Other (list)	☐

Table IEP-1a: Carry-over page, if required

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} **Page 1a**

Street Number & Name:		**Job No.:**	
AKA:		**By:**	
Name of building:		**Date:**	
City:		**Revision No.:**	

Table IEP-1a Additional Photos and Sketches

Add any additional photographs, notes or sketches required below:

Note: print this page separately

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

Table IEP-2: Initial Evaluation Procedure – Step 2

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} **Page 2**

Street Number & Name:		**Job No.:**	
AKA:		**By:**	
Name of building:		**Date:**	
City:		**Revision No.:**	

Table IEP-2 Initial Evaluation Procedure Step 2

Step 2 - Determination of *(%NBS)*$_b$

(Baseline *(%NBS)* for particular building - refer Section B5)

2.1 Determine nominal *(%NBS)* = *(%NBS)*$_{nom}$

	Longitudinal	Transverse
a) Building Strengthening Data		
Tick if building is known to have been strengthened in this direction	☐	☐
If strengthened, enter percentage of code the building has been strengthened to	N/A	N/A
b) Year of Design/Strengthening, Building Type and Seismic Zone		
Pre 1935	◉	◉
1935-1965	○	○
1965-1976	○	○
1976-1984	○	○
1984-1992	○	○
1992-2004	○	○
2004-2011	○	○
Post Aug 2011	○	○
Building Type:		
Seismic Zone:	Not applicable	Not applicable
c) Soil Type		
From NZS1170.5:2004, Cl 3.1.3 :		
From NZS4203:1992, Cl 4.6.2.2 : (for 1992 to 2004 and only if known)	Not applicable	Not applicable
d) Estimate Period, *T*		
Comment:		
h_n =	25	25 m
A_c =	1.00	1.00 m^2
Moment Resisting Concrete Frames: $T = \max\{0.09h_n^{0.75}, 0.4\}$	◉	◉
Moment Resisting Steel Frames: $T = \max\{0.14h_n^{0.75}, 0.4\}$	○	○
Eccentrically Braced Steel Frames: $T = \max\{0.08h_n^{0.75}, 0.4\}$	○	○
All Other Frame Structures: $T = \max\{0.06h_n^{0.75}, 0.4\}$	○	○
Concrete Shear Walls $T = \max\{0.09h_n^{0.75}/A_c^{0.5}, 0.4\}$	○	○
Masonry Shear Walls: $T \leq 0.4$sec	○	○
User Defined (input Period):	○	○
Where h_n = height in metres from the base of the structure to the uppermost seismic weight or mass.	T: 1.01	1.01
e) Factor A: Strengthening factor determined using result from (a) above (set to 1.0 if not strengthened)	Factor A: 1.00	1.00
f) Factor B: Determined from NZSEE Guidelines Figure 3A.1 using results (a) to (e) above	Factor B: 0.03	0.03
g) Factor C: For reinforced concrete buildings designed between 1976-84 Factor C = 1.2, otherwise take as 1.0.	Factor C: 1.00	1.00
h) Factor D: For buildings designed prior to 1935 Factor D = 0.8 except for Wellington and Napier (1931-1935) where Factor D may be taken as 1.0, otherwise take as 1.0.	Factor D: 1.00	1.00
***(%NBS)*$_{nom}$ = AxBxCxD**	***(%NBS)*$_{nom}$ 3%**	3%

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Table IEP-2: Initial Evaluation Procedure – Step 2 continued

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} **Page 3**

Street Number & Name:		**Job No.:**	
AKA:		**By:**	
Name of building:		**Date:**	
City:		**Revision No.:**	

Table IEP-2 Initial Evaluation Procedure Step 2 continued

2.2 Near Fault Scaling Factor, Factor E

If $T \leq 1.5$sec, Factor E = 1

		Longitudinal	Transverse
a) Near Fault Factor, *N(T,D)* (from NZS1170.5:2004, Cl 3.1.6)	N(T,D):	1	1
b) Factor E = 1/N(T,D)	**Factor E:**	1.00	1.00

2.3 Hazard Scaling Factor, Factor F

a) Hazard Factor, *Z*, for site

Location: Akaroa — Refer right for user-defined locations

Z =	0.3	(from NZS1170.5:2004, Table 3.3)
Z_{1992} =	0.6	(NZS4203:1992 Zone Factor from accompanying Figure 3.5(b))
Z_{2004} =	0.22	(from NZS1170.5:2004, Table 3.3)

b) Factor F

For pre 1992	=	1/Z
For 1992-2011	=	Z_{1992}/Z
For post 2011	=	Z_{2004}/Z

	Factor F:	3.33	3.33

2.4 Return Period Scaling Factor, Factor G

a) Design Importance Level, I

(Set to 1 if not known. For buildings designed prior to 1965 and known to be designed as a public building set to 1.25. For buildings designed 1965-1976 and known to be designed as a public building set to 1.33 for Zone A or 1.2 for Zone B. For 1976-1984 set I value.)

	I =	1	1

b) Design Risk Factor, R_o (set to 1.0 if other than 1976-2004, or not known)

	R_o =	1	1

c) Return Period Factor, R (from NZS1170.0:2004 Building Importance Level)

Choose Importance Level — ◉ 1 ○ 2 ○ 3 ○ 4 | ◉ 1 ○ 2 ○ 3 ○ 4

	R =	0.5	0.5

d) Factor G = IR_o/R

	Factor G:	2.00	2.00

2.5 Ductility Scaling Factor, Factor H

a) Available Displacement Ductility Within Existing Structure

Comment:

	μ =	2.00	2.00

b) Factor H

		k_μ	k_μ
For pre 1976 (maximum of 2)	=	2.00	2.00
For 1976 onwards	=	1	1
	Factor H:	2.00	2.00

(where k_μ is NZS1170.5:2004 Inelastic Spectrum Scaling Factor, from accompanying Table 3.3)

2.6 Structural Performance Scaling Factor, Factor I

a) Structural Performance Factor, S_p (from accompanying Figure 3.4)

Tick if light timber-framed construction in this direction

	S_p =	0.70	0.70

b) Structural Performance Scaling Factor = $1/S_p$

	Factor I:	1.43	1.43

Note Factor B values for 1992 to 2004 have been multiplied by 0.67 to account for Sp in this period

2.7 Baseline *%NBS* for Building, $(\%NBS)_b$ (equals $(\%NBS)_{nom}$ x E x F x G x H x I)	55%	55%

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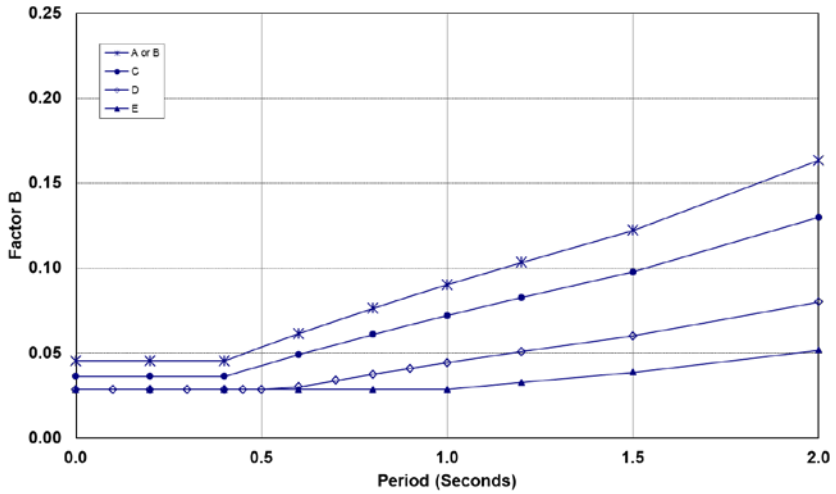


Figure BA.1(a): Factor B Pre-1965, All Zones

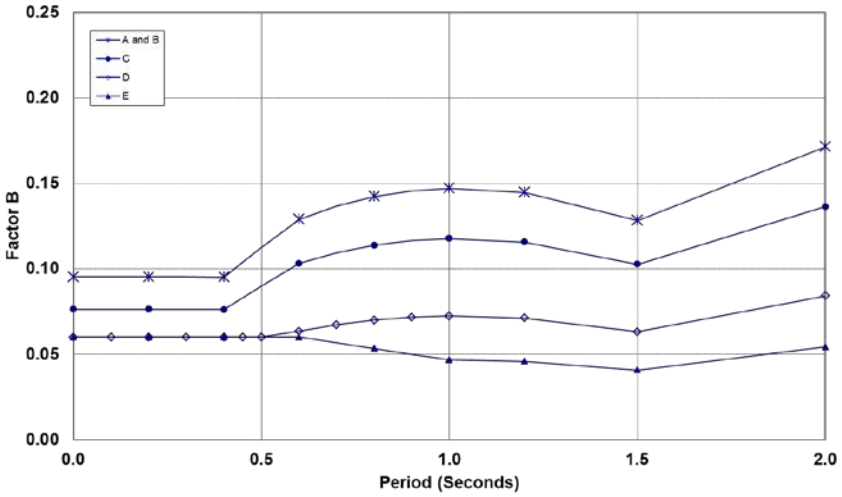


Figure BA.1(b): Factor B 1965-76, Zone A

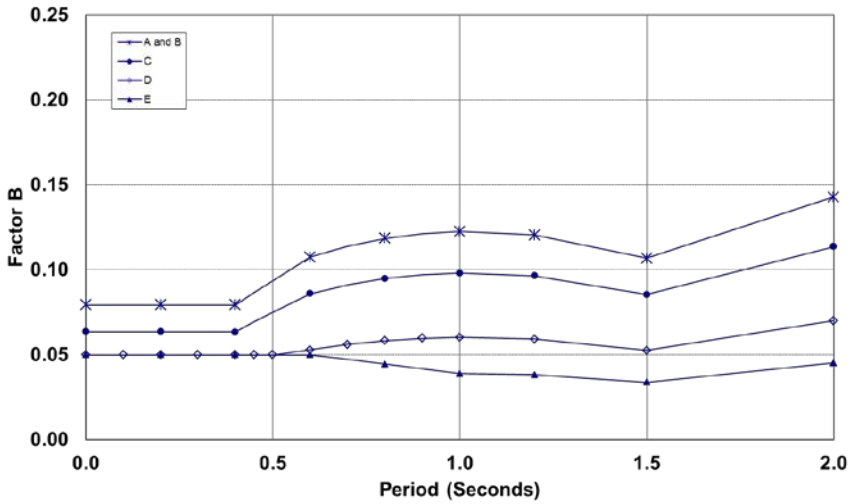


Figure BA.1(c): Factor B 1965-76, Zone B

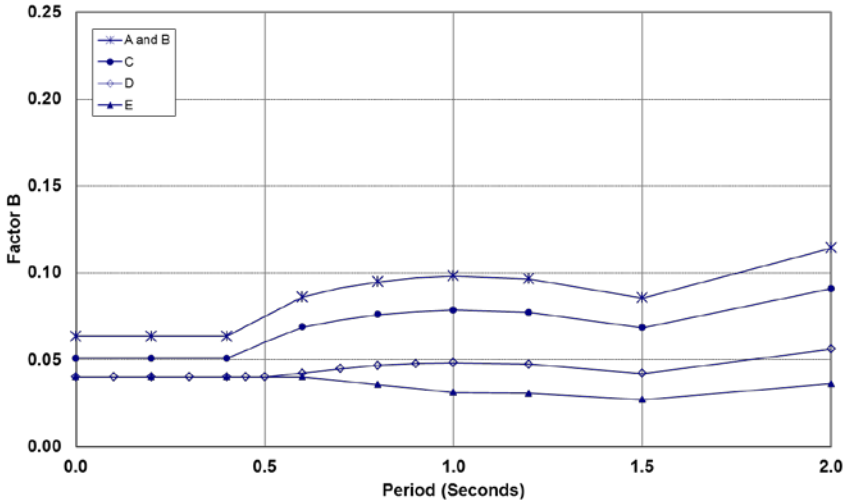


Figure BA.1(d): Factor B 1965-76, Zone C

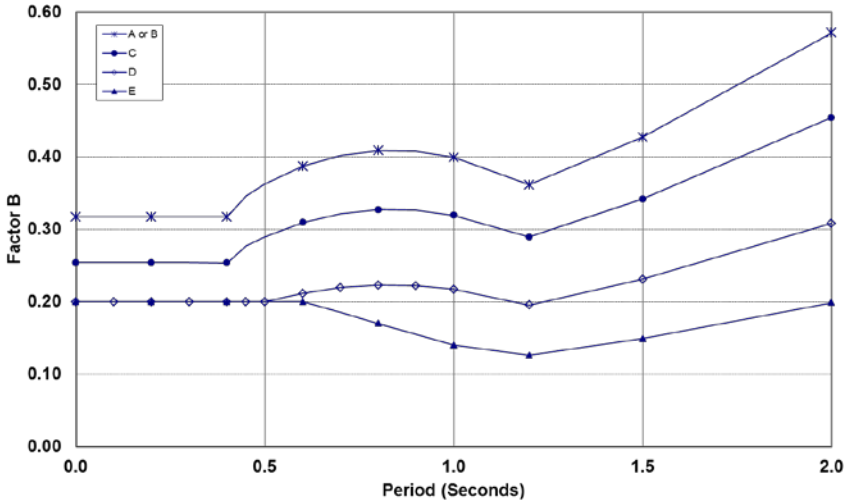


Figure BA.1(e): Factor B 1976-92, Zone A

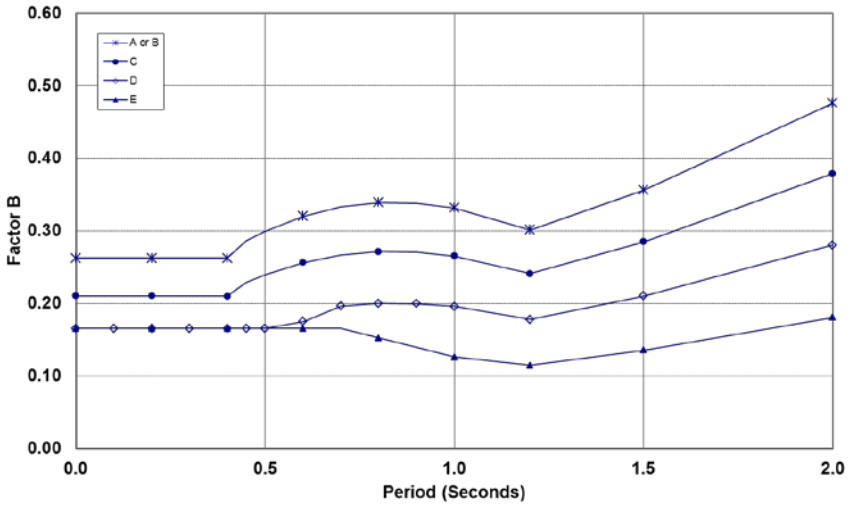


Figure BA.1(f): Factor B 1976-92, Zone B

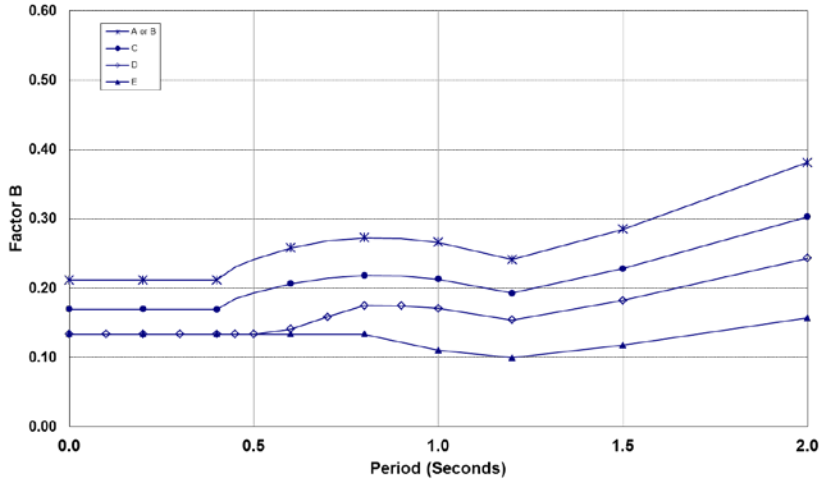


Figure BA.1(g): Factor B 1976-92, Zone C

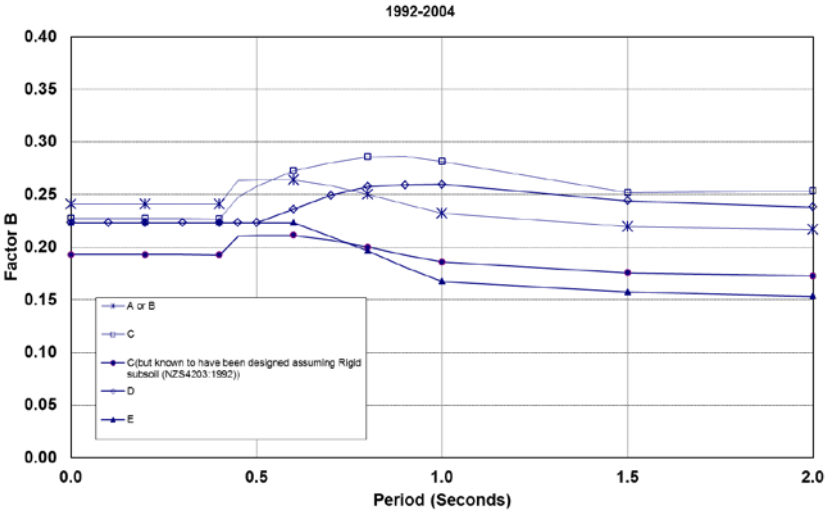


Figure BA.1(h): Factor B 1992-2004

Figure BA.1: Factor B for different building design vintages

Table BA.1: Current return period factor, *R*

Importance level	Comments	*R*
1	Structures presenting a low degree of hazard to life and other property	0.5
2, or if otherwise unknown	Normal structures and structures not in other importance levels	1.0
3	Structures that as a whole may contain people in crowds or contents of high value to the community or pose risks to people in crowds	1.3
4	Structures with special post-disaster functions	1.8
5	Special structures (outside the scope of this Standard — acceptable probability of failure to be determined by special study)	

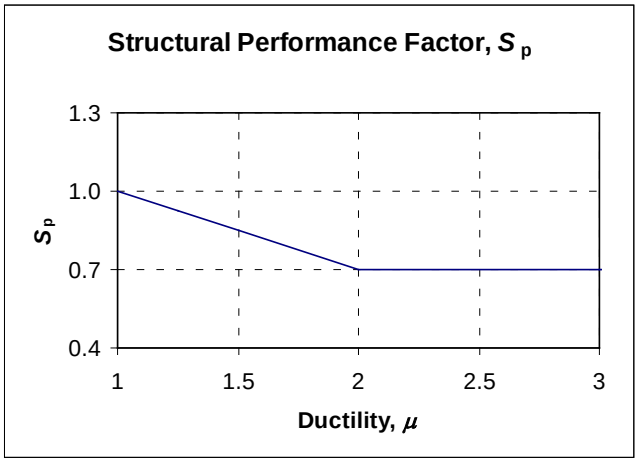
Table BA.2: Maximum ductility factors to be used in the IEP

Structure type	Maximum ductility factor			
	Pre-1935	1935-65	1965-76	>1976
Unstrengthened URM buildings	1.5	1.5	N/A	N/A
All other buildings	2	2	2	6

Table BA.3: Ductility scaling factor, Factor H

`	Structural ductility factor, μ							
	1.0		1.25		1.50		2	
Soil Type	A,B,C & D	E	A,B,C & D	E	A,B,C & D	E	A,B,C & D	E
Period, *T*								
≤ 0.40s	1	1	1.14	1.25	1.29	1.50	1.57	1.70
0.50s	1	1	1.18	1.25	1.36	1.50	1.71	1.75
0.60s	1	1	1.21	1.25	1.43	1.50	1.86	1.80
0.70s	1	1	1.25	1.25	1.50	1.50	2.00	1.85
0.80s	1	1	1.25	1.25	1.50	1.50	2.00	1.90
≥1.00s	1	1	1.25	1.25	1.50	1.50	2.00	2.00

Note:
For buildings designed post-1976, Factor H shall be taken as 1.0.



Where S_p is the structural performance factor from NZS 1170.5:2004, Clause 4.4.2
for light framed timber structures S_p = 0.5

Figure BA.2: Structural performance factor, S_p

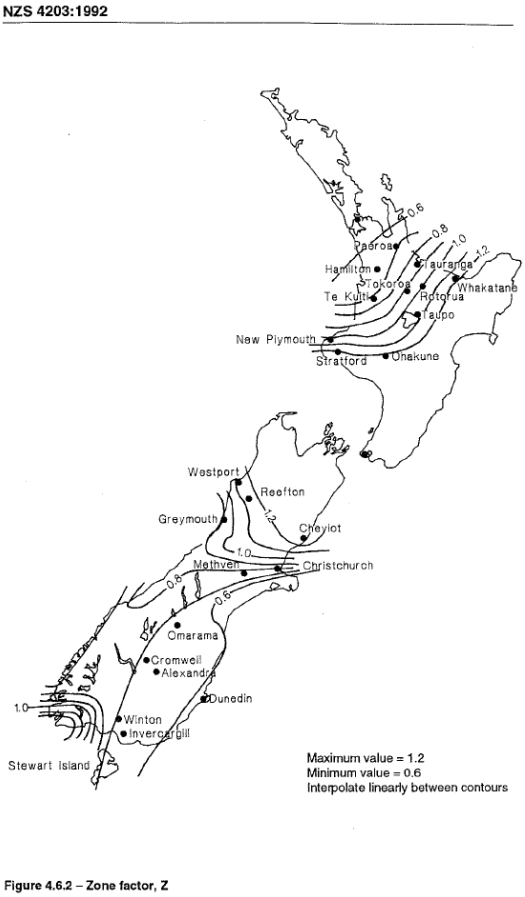


Figure BA.3(a): NZS 4203:1984, NZS 4203:1976 and NZSS 1900:1965 Chapter 8

Figure BA.3(b): NZS 4203:1992

Figure BA.3: Extracts from previous Standards showing seismic zoning schemes

Table IEP-3: Initial Evaluation Procedure – Step 3

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} **Page 4**

Street Number & Name:		**Job No.:**	
AKA:		**By:**	
Name of building:		**Date:**	
City:		**Revision No.:**	

Table IEP-3 Initial Evaluation Procedure Step 3

Step 3 - Assessment of Performance Achievement Ratio (PAR)
(Refer Appendix B - Section B3.2)

a) Longitudinal Direction

potential CSWs	Effect on Structural Performance (Choose a value - Do not interpolate)			Factors
3.1 Plan Irregularity				
Effect on Structural Performance	○ *Severe*	○ *Significant*	◉ *Insignificant*	**Factor A** 1.0
Comment				
3.2 Vertical Irregularity				
Effect on Structural Performance	○ *Severe*	○ *Significant*	◉ *Insignificant*	**Factor B** 1.0
Comment				
3.3 Short Columns				
Effect on Structural Performance	○ *Severe*	○ *Significant*	◉ *Insignificant*	**Factor C** 1.0
Comment				

3.4 Pounding Potential
(Estimate D1 and D2 and set D = the lower of the two, or 1.0 if no potential for pounding, or consequences are considered to be minimal)

a) Factor D1: - Pounding Effect

Note:
Values given assume the building has a frame structure. For stiff buildings (eg shear walls), the effect of pounding may be reduced by taking the coefficient to the right of the value applicable to frame buildings.

Factor D1 For Longitudinal Direction: 1.0

Table for Selection of Factor D1	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	○ 1	◉ 1	○ 1
Alignment of Floors not within 20% of Storey Height	○ 0.4	○ 0.7	○ 0.8

Comment

b) Factor D2: - Height Difference Effect

Factor D2 For Longitudinal Direction: 1.0

Table for Selection of Factor D2	Severe	Significant	Insignificant
	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Height Difference > 4 Storeys	○ 0.4	○ 0.7	◉ 1
Height Difference 2 to 4 Storeys	○ 0.7	○ 0.9	○ 1
Height Difference < 2 Storeys	○ 1	○ 1	○ 1

Comment

Factor D 1.0

3.5 Site Characteristics - *Stability, landslide threat, liquefaction etc as it affects the structural performance from a life-safety perspective*

Effect on Structural Performance	◉ *Severe*	○ *Significant*	○ *Insignificant*	**Factor E** 0.5
Comment				

3.6 Other Factors - *for allowance of all other relevant characterstics of the building*
Record rationale for choice of Factor F:
For ≤ 3 storeys - Maximum value 2.5
otherwise - Maximum value 1.5.
No minimum.
Factor F 1.0

Comment

3.7 Performance Achievement Ratio (PAR)
(equals **A x B x C x D x E x F**)

PAR
Longitudinal **0.50**

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out in "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

Table IEP-3: Initial Evaluation Procedure – Step 3 continued

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} **Page 5**

Street Number & Name:		**Job No.:**	
AKA:		**By:**	
Name of building:		**Date:**	
City:		**Revision No.:**	

Table IEP-3 Initial Evaluation Procedure Step 3

Step 3 - Assessment of Performance Achievement Ratio (PAR)
(Refer Appendix B - Section B3.2)

b) Transverse Direction

Factors

potential CSWs — **Effect on Structural Performance** (Choose a value - Do not interpolate)

3.1 Plan Irregularity
Effect on Structural Performance ○ Severe ● Significant ○ Insignificant **Factor A** 0.7
Comment

3.2 Vertical Irregularity
Effect on Structural Performance ○ Severe ● Significant ○ Insignificant **Factor B** 0.7
Comment

3.3 Short Columns
Effect on Structural Performance ○ Severe ● Significant ○ Insignificant **Factor C** 0.7
Comment

3.4 Pounding Potential
(Estimate D1 and D2 and set D = the lower of the two, or 1.0 if no potential for pounding, or consequences are considered to be minimal)

a) Factor D1: - Pounding Effect

Note:
Values given assume the building has a frame structure. For stiff buildings (eg shear walls), the effect of pounding may be reduced by taking the coefficient to the right of the value applicable to frame buildings.

Factor D1 For Transverse Direction: 0.7

Table for Selection of Factor D1	Severe	Significant	Insignificant
Separation	0<Sep<.005H	.005<Sep<.01H	Sep>.01H
Alignment of Floors within 20% of Storey Height	○ 1	○ 1	○ 1
Alignment of Floors not within 20% of Storey Height	○ 0.4	● 0.7	○ 0.8

Comment

b) Factor D2: - Height Difference Effect

Factor D2 For Transverse Direction: 0.4

Table for Selection of Factor D2	Severe	Significant	Insignificant
	0<Sep<.005H	005<Sep<.01H	Sep>.01H
Height Difference > 4 Storeys	● 0.4	○ 0.7	○ 1
Height Difference 2 to 4 Storeys	○ 0.7	○ 0.9	○ 1
Height Difference < 2 Storeys	○ 1	○ 1	○ 1

Comment

Factor D 0.4

3.5 Site Characteristics - *Stability, landslide threat, liquefaction etc as it affects the structural performance from a life-safety perspective*

Effect on Structural Performance ○ Severe ○ Significant ● Insignificant **Factor E** 1.0
Comment

3.6 Other Factors - *for allowance of all other relevant characterstics of the building*
Record rationale for choice of Factor F:
For ≤ 3 storeys - Maximum value 2.5
otherwise - Maximum value 1.5.
No minimum.
Factor F 1.00
Comment

PAR

3.7 Performance Achievement Ratio (PAR)
(equals **A x B x C x D x E x F**)
Transverse 0.14

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Table IEP-4: Initial Evaluation Procedure – Steps 4, 5, 6 and 7

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} **Page 6**

Street Number & Name:		**Job No.:**	
AKA:		**By:**	
Name of building:		**Date:**	
City:		**Revision No.:**	

Table IEP-4 Initial Evaluation Procedure Steps 4, 5, 6 and 7

Step 4 - Percentage of New Building Standard *(%NBS)*

		Longitudinal	Transverse
4.1	**Assessed Baseline *%NBS* $(\%NBS)_b$** (from Table IEP - 1)	55%	55%
4.2	**Performance Achievement Ratio (PAR)** (from Table IEP - 2)	0.50	0.14
4.3	**PAR x Baseline $(\%NBS)_b$**	25%	15%
4.4	**Percentage New Building Standard *(%NBS)* - Seismic Rating** (Use lower of two values from Step 4.3)		15%

Step 5 - Is *%NBS* < 34? YES

Step 6 - Potentially Earthquake Risk (is *%NBS* < 67)? YES

Step 7 - Provisional Grading for Seismic Risk based on IEP

Seismic Grade E

Additional Comments (items of note affecting IEP based seismic rating)

Relationship between Grade and *%NBS*:

Grade:	A+	A	B	C	D	E
%NBS:	> 100	100 to 80	79 to 67	66 to 34	< 34 to 20	< 20

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out in "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

Table IEP-5: Initial Evaluation Procedure – Step 8

Initial Evaluation Procedure (IEP) Assessment - Completed for {Client/TA} **Page 7**

Street Number & Name:		Job No.:	
AKA:		By:	
Name of building:		Date:	
City:		Revision No.:	

Table IEP-5 Initial Evaluation Procedure Step 8

Step 8 - Identification of potential Severe Structural Weaknesses (SSWs) that could result in significant risk to a significant number of occupants

8.1	Number of storeys above ground level	6
8.2	Presence of heavy concrete floors and/or concrete roof? (Y/N)	Y

Potential Severe Structural Weaknesses (SSWs):

Note: Options that are greyed out are not applicable and need not be considered.

Occupancy not considered to be significant - no further consideration required

Risk not considered to be significant - no further consideration required

The following potential Severe Structural Weaknesses (SSWs) have been identified in the building that could result in significant risk to a significant number of occupants:

1. None identified ☐
2. Weak or soft storey (except top storey) ☐
3. Brittle columns and/or beam-column joints the deformations of which are not constrained by other structural elements ☐
4. Flat slab buildings with lateral capacity reliant on low ductility slab-to-column connections ☐
5. No identifiable connection between primary structure and diaphragms ☐
6. Ledge and gap stairs ☐

IEP Assessment Confirmed by _____ **Signature**

_____ **Name**

_____ **CPEng. No**

WARNING!! *This initial evaluation has been carried out solely as an initial seismic assessment of the building following the procedure set out in "The Seismic Assessment of Existing Buildings" Technical Guidelines for Engineering Assessments, July 2017. This spreadsheet must be read in conjunction with the limitations set out in the accompanying report, and should not be relied on by any party for any other purpose. Detailed inspections and engineering calculations, or engineering judgements based on them, have not been undertaken, and these may lead to a different result or seismic grade.*

BA.2 Guidance and Commentary

BA.2.1 Step 1 – Collect general information (Table IEP-1)

The first step in the IEP should be to collect relevant information necessary to carry out the assessment and to record this as the basis of the assessment. It is a fundamental premise of the IEP that limited definitive information is likely to be available and the assessment will necessarily be made on the basis of a visual inspection of only the exterior of the building.

Photographs of the building should be taken as part of the IEP and should form part of the permanent record. Likewise, a record of the features observed and the extent of information that was available at the time of the assessment will be important considerations if the assessment is questioned in the future. Table IEP-1 provides a means of recording this information.

BA.2.2 Step 2 – Determine baseline percentage of new building standard $(\%NBS)_b$ (Table IEP-2)

Introduction

One of the first questions typically asked regarding existing buildings is how their overall expected seismic resistance compares to a building designed to the standard required for new buildings as specified in NZS 1170.5:2004. The comparison available through the IEP provides a simple and convenient measure of relative performance in earthquakes; provided that the limitations of the IEP are recognised (refer to Section B3.4).

It must be emphasised that the percentage figure derived, $(\%NBS)_b$, is a first step in any evaluation. It gives only an indication of the likely situation. It does not take full account of the particular characteristics of a building, which may be beneficial (as in the case when extra walls are added for architectural reasons but are nevertheless significant structural elements). It also does not take into account the effect of potential CSWs that can greatly reduce the overall seismic resistance predicted by the $(\%NBS)_b$ calculation.

Approach

There are a number of variables that feed into the calculation of a baseline percentage current code ratio $(\%NBS)_b$. These include:

- the natural period of vibration of the building
- its location in relation to seismic hazard
- the site subsoil characteristics
- the vintage or code to which the building was designed or strengthened. If the building has been strengthened, the level of strengthening is required.
- the available ductility in the building, and
- the design and current importance level designation of the building.

Different codes have had different requirements for design over the years. In broad terms these amount to:

- pre-1935: no seismic design (except for buildings in Wellington)
- pre-1965: typically design for 0.1 g lateral force
- 1965-1976: design to NZSS 1900:1965, Chapter 8
- 1976-1992: design to NZS 4203:1976, with some changes to risk and importance factors and for reinforced concrete structures in 1984
- 1992-2004: design to NZS 4203:1992
- post-2004: design to NZS 1170.5:2004.

Note:

Each orthogonal direction should be assessed separately unless it is clear from the start which governs.

Definitions

$(\%NBS)_{\text{nom}} \Rightarrow$ The assessed nominal performance compared to NZS 1170.5:2004, assuming ductility of 1.0, hazard factor of 1.0, near fault factor of 1.0, return period factor of 1.0, and structural performance factor of 1.0 (refer to Table IEP-1).

$(\%NBS)_{\text{b}} \Rightarrow$ The baseline $(\%NBS)$ modifies $(\%NBS)_{\text{nom}}$ to account for assessed ductility, location (hazard factor and near fault factor from NZS 1170.5:2004) and occupancy category (i.e. return period factor) but assuming a good structure complying with the relevant code provisions at the time it was built.

The resulting value of $(\%NBS)_{\text{b}}$ may be regarded as a measure of the seismic capacity of a well designed and constructed regular building of its type and vintage on the site in question. It is a "yardstick" against which to measure the effect of critical structural weaknesses that may exist in a particular building of the same type.

Note that an assessment of the likely ductility is required but the choice of ductility for post 1976 buildings will have little effect on the IEP score. In formulating the process it has been assumed that what constitutes available ductility has not changed significantly since 1976. If this is not correct the adjustment should be via Factor F in Step 3.2.

PAR $\Rightarrow$ The performance achievement ratio (PAR) may be regarded as the ratio of the performance of the particular building, as inspected, in relation to a well designed and constructed regular building of its type and vintage on the site in question that just meets the requirements of the code of the day. Therefore, such a building would have a PAR of 1.0 (refer to Table IEP-3).

It is expected that all known issues (including for those items that would be considered as parts of the building but nevertheless would present a life safety risk should they fail) will be included in the assessment of PAR.

$\%NBS \Rightarrow$ Percentage of new building standard, $\%NBS$. This adjusts $(\%NBS)_{\mathrm{b}}$ to account for particular characteristics of the building, especially SWs (pCSWs), (refer to Table IEP-4).

Note:

$\%NBS$ = $(\%NBS)_{\mathrm{b}}$ x Performance Achievement Ratio (PAR)
= a relative measure, in percentage terms, of the earthquake performance of the building under consideration with respect to NZS 1170.5:2004, taking into account SWs and other relevant features.

Step 2.1: Determine nominal percentage of new building standard $(\%\boldsymbol{NBS})_{\mathbf{nom}}$

Use the steps (a) to (g) to calculate $(\%NBS)_{\mathrm{nom}}$ using the following equation:

$$(\%NBS)_{\mathrm{nom}} = A^{*}B^{*}C^{*}D \qquad \ldots\text{BA.1}$$

a) Determine code used in design of building:

Note:

If the building is known to have been strengthened, adjust $(\%NBS)_{\mathrm{nom}}$ for an appropriate level of strengthening.

- pre-1935: refer to discussion in (f) below.
- pre-1965 (0.08 g uniform load or 0.06 g applied as a triangular load)
- 1965-1976 (NZSS1900, Chapter 8):
  - Zone A
  - Zone B
  - Zone C
- 1976-1992 (NZS 4203:1976 or NZS 4203:1984)
  - Zone A
  - Zone B
  - Zone C

  For concrete structures designed to NZS 4203:1976 (refer also to (e) below).
- 1992-2004 (NZS 4203:1992), and
- Post-2004 (NZS 1170.5:2004).

b) Determine soil type at the site:

- Use NZS 1170.5:2004 classifications:
  - Class A – Strong rock
  - Class B – Rock
  - Class C – Shallow soil sites
  - Class D – Deep or soft soil sites
  - Class E – Very soft soil sites.

c) Assess period of building:

- Use any recognised method.
- Note that accurate analysis is not warranted in many cases since results are not highly sensitive to changes in period. Refer to Figure BA.1 for an indication of the variation.
- Simplified period calculations given in Table IEP-2 come from the commentary of NZS 1170.5:2004 with an additional limit on A_c.

d) Use the appropriate part of Figure BA.1 to determine Factor B.

e) Concrete buildings designed to NZS 4203 up to 1984 were required to be designed using a structural material factor, M = 1.0. This was amended in NZS 4203:1984 to M = 0.8; hence the adjustment required by Factor C. Take Factor C as 1.0 for buildings outside the date range 1976 to 1984.

f) Prior to 1935, no earthquake provisions were in place in New Zealand except for Wellington and in Napier (post the Hawkes Bay 1931 Earthquake). While it would be possible to discount completely the seismic performance of buildings designed prior to 1935 this is clearly too severe. The approach taken in the IEP is to assume that buildings designed in Wellington prior to 1935 and in Napier in the period 1931 to 1935 will perform at least as well as those designed to NZSS 95:1939 as they are likely to have been subjected to some design for earthquakes. Elsewhere a 20% penalty has been included (Factor D) to reflect that these buildings would not have been required to be designed for earthquakes. It is expected that major deficiencies, if any, will be picked up in the assessment of PAR. For post-1935 buildings take Factor D as 1.0.

Step 2.2: Determine near fault scaling factor (Factor E)

a) Use NZS 1170.5:2004 to determine the $N(T, D)$ value applicable for a new building at the site of the existing one under consideration.

Step 2.3: Determine hazard scaling factor (Factor F)

a) Use NZS 1170.5:2004 to determine the hazard factor, Z, for the site.

b) For 1992-2004 also determine the zone factor, Z, for the site from NZS 4203:1992 (refer to accompanying Figure BA.3(b)).

Step 2.4: Determine return period scaling factor (Factor G)

a) and b) Enter design values for I and R_0, if known. Otherwise enter 1.0. For buildings designed pre-1976 and known to have been designed as public buildings, I may be taken as 1.25.

c) Use NZS 1170.0:2002 (accompanying Table BA.1) to determine the building's current importance level and enter the appropriate return period factor from Table BA.1.

d) Calculate the return period scaling factor.

Step 2.5: Determine ductility scaling factor (Factor H)

a) Assess overall ductility available in the building in question (refer to Table BA.2 for maximum values).

b) Read the ductility scaling factor from Table BA.3.

For 1976 onwards the ductility is effectively included in the appropriate part of Figure BA.2; therefore set this as 1.0.

For buildings designed before 1976 take the value from within the table. This value varies with period and soil type and is effectively k_μ from NZS 1170.5:2004.

Step 2.6: Determine structural performance scaling factor (Factor I)

Use NZS 1170.5:2004 or the appropriate materials Standard (whichever provides the higher value) to determine the structural performance factor (refer to Figure BA.2).

Step 2.7: Determine baseline percentage of new building standard for building $(\%NBS)_{\mathbf{b}}$

a) Use values from Steps 2.1 to 2.7 to calculate $(\%NBS)_{\mathrm{b}}$ using the following equation:

$$(\%NBS)_{\mathrm{b}} = (\%NBS)_{\mathrm{nom}} =^{*} E^{*}F^{*}G^{*}H^{*}I \qquad \text{…BA.2}$$

$$(\%NBS)_{\mathrm{b}} = (\%NBS)_{\mathrm{nom}} * \frac{1}{N(T,D)} * \left(\frac{1}{Z} \text{ or } \frac{Z_{1992}}{Z}\right) * \frac{IR_0}{R} * (\mu \text{ or } 1) * \frac{1}{S_{\mathrm{p}}} \qquad \text{…BA.3}$$

where:

$(\%NBS)_{\mathrm{b}}$ = is the baseline percentage capacity of the building assuming regular, complying construction

$(\%NBS)_{\mathrm{nom}}$ = the nominal value of $(\%NBS)$ which assumes $N(T,D) = 1.0,\ Z = 1.0$

$R = 1.0,\ \mu = 1.0$, and $S_p = 1.0$

$N(T, D)$	=	the near fault factor from NZS 1170.5:2004
Z	=	the hazard factor from NZS 1170.5:2004
Z_{1992}	=	the zone factor from NZS 4203:1992 (for 1992-2004 buildings only)
R	=	the return period factor from the accompanying Table BA.1
R_0	=	the risk factor used for the design. If it is not known with certainty, R_0=1
I	=	the importance factor used for the design of the building, applicable for 1965-1984 buildings only and only if known with certainty; otherwise take as = 1.0. For buildings designed 1965-1976 as public buildings take $I = 1.25$; otherwise take as $= 1.0$
k_μ	=	the structural ductility scaling factor from accompanying Table BA.3. Note that μ cannot be greater than the values given in Table BA.2.
S_p	=	the structural performance factor applicable to the type of building under consideration (refer to Figure BA.2).

Factor B

The above procedure allows calculation of $(\%NBS)_b$ for a particular type of building providing its location and original design code are known, and an assessment of the available ductility is made.

The values for Factor B shown in Figures BA.1(a) to (h) are based on:

- near fault factor of 1.0
- hazard factor of 1.0
- return period factor of 1.0
- ductility of 1.0
- structural performance factor of 1.0.

The values for Factor B shown are the ratios of the NZS 1170.5:2004 coefficient on the above basis and the coefficient that comes from the Standard used in design (that depends on date of design). Refer to Figure BA.4.

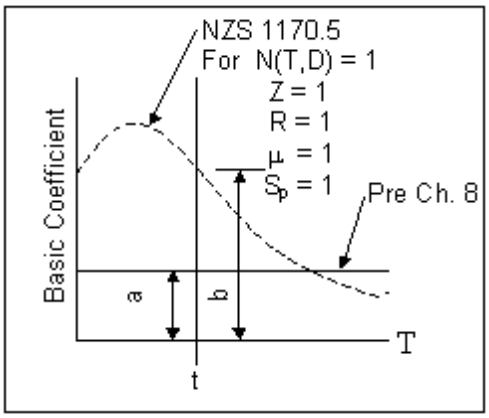


Figure BA.4(a): Pre-NZSS 1900:1965

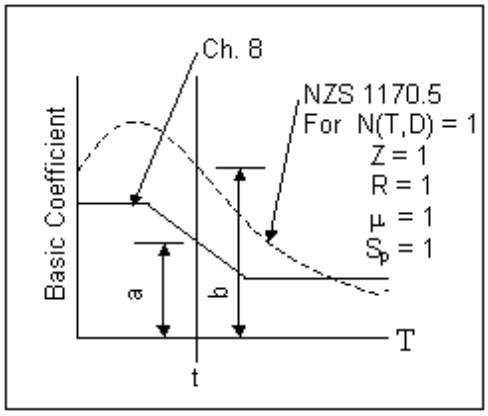


Figure BA.4(b): NZSS 1900: Chapter 8: 1965

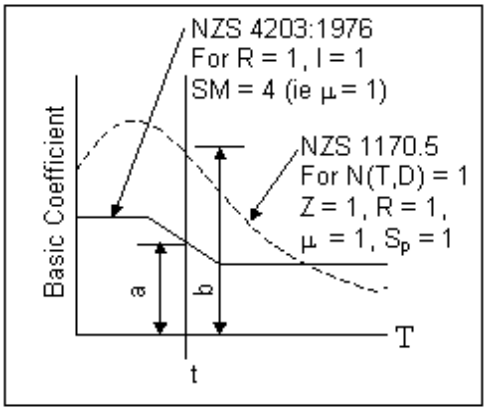


Figure BA.4(c): NZS 4203:1976

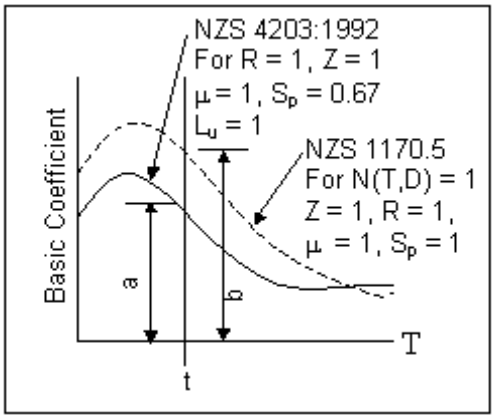


Figure BA.4(d): NZS 4203:1992

Figure BA.4: Concepts behind scaling Factor *B*

For a particular T, $(\%NBS)_{\text{nom}}$ = a/b

a) to adjust for near fault factor — multiply by Factor E = $\frac{1}{N(T,D)}$

b) to adjust for hazard factor — multiply by Factor F = $\frac{1}{Z}$ for pre 1992, or

= $\frac{Z_{1992}}{Z}$ for 1992-2003

= $\frac{Z_{2004}}{Z}$ for August 2011 onwards. This allows for the change in Z in the Canterbury region following the 2010/2011 Canterbury earthquakes.

c) to adjust for return period factor — multiply by Factor G = $\frac{IR_0}{R}$

d) to adjust for ductility — multiply by Factor H = k_μ for pre 1976, or

= 1 for 1976 onwards

e) to adjust for structural performance — multiply by Factor I = $\frac{1}{S_p}$

The values for Factor B are approximate and based on the simplifying assumptions listed below. Engineers can substitute their own code comparisons if they wish.

Assumptions inherent in the assessment of $(\%NBS)_b$

There are a number of assumptions inherent in the assessment of $(\%NBS)_b$. These include the following:

- The building has been designed and built in accordance with the building standard current at the time, and with good practice.
- The building has been designed for the correct subsoil category. (Make pro rata adjustments according to NZS 1170.5:2004 spectra, if this is not the case). Note that the rigid subsoil category in NZS 4203:1992 has been split into two categories in NZS 1170.5:2004. The IEP assumes that buildings on site subsoil type C (NZS 1170.5:2004) designed to NZS 4203:1992 would have been designed assuming intermediate subsoil. The procedure allows an adjustment if it is known that rigid subsoil was originally assumed.
- Buildings designed prior to 1965 have had their assessed capacity increased by a factor of 1.5 to convert from allowable stress to ULS design, and divided by 1.4 to convert from a rectangular shear distribution over the height of the building to a triangular distribution with 10% of the base shear applied at the roof. (The basis for this is the ratio of overturning moments derived by the two methods.)
- Buildings designed to the 1965 code have had their period shifted by a factor of 1.25 to take account of greater flexibility resulting from the allowance for cracking assumed in later Standards.
- Buildings designed to the 1976 code are assumed to use the same elastic spectral values as given in the 1984 code. Therefore, for a μ of 1, the 1976 values are increased by a factor of 4 (i.e. SM = 4).
- Buildings designed to the 1992 code are assumed to have been designed for an S_p of 0.67. If this is not the case adjust accordingly.

BA.2.3 Step 3 - Determine performance achievement ratio (PAR) (Table IEP-3)

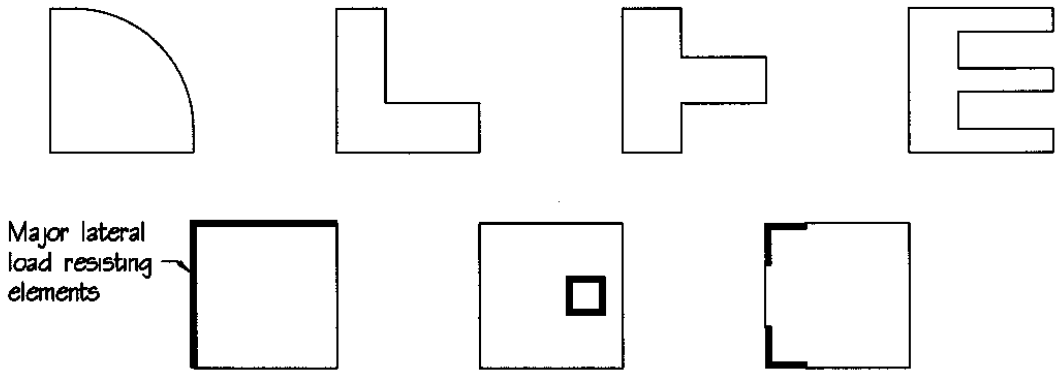
Assessment of effects of potential critical structural weaknesses (Steps 3.1 to 3.4)

Note:

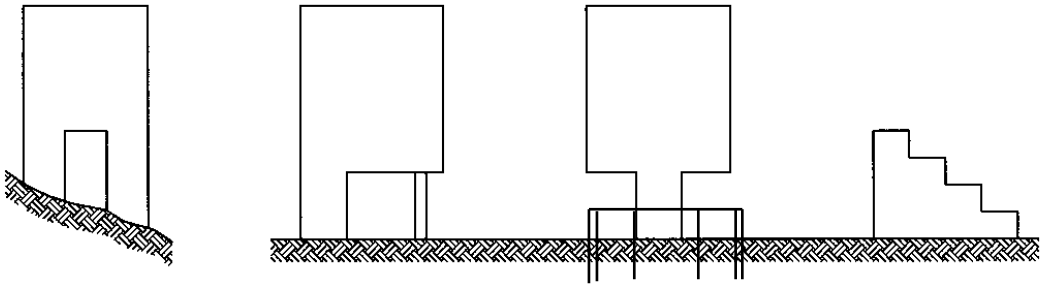
Consider each orthogonal direction separately unless it is clear from the start which governs.

A potential critical structural weakness (CSW) is <u>any</u> potential structural weakness (SW) that could potentially influence the building's performance/capacity in severe earthquake shaking. Some examples are shown in Figure BA.5.

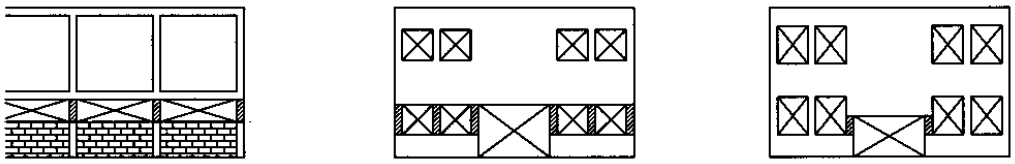
Potential CSWs, therefore, are not restricted to the features shown in Table IEP-3. Any potential CSW that is identified but is not specifically included in Table IEP-3 should be accounted for by setting an appropriate value for Factor F.



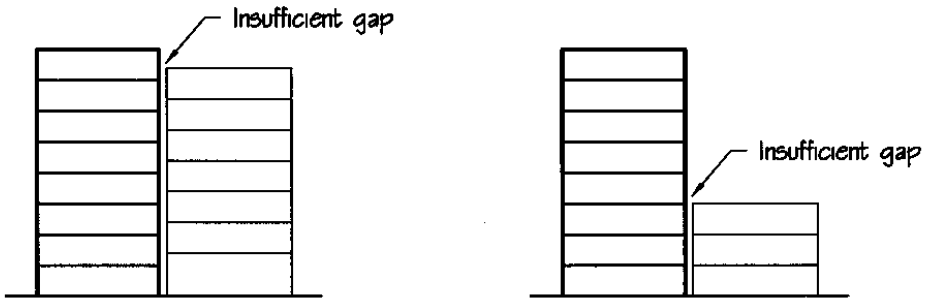
a) Examples of Plan Irregularity



b) Examples of Vertical Irregularity



c) Examples of Short Columns



(i) Floor Mis-alignment (ii) Height Difference

d) Examples of Pounding Potential

Figure BA.5: Examples of potential critical structural weaknesses

Note:

Figure BA.5 does not describe all potential CSWs that need to be considered.

Note:

The 2017 changes to earthquake-prone building legislation confirm that parts of buildings are not able to be excluded from consideration.

The effect of a potential CSW on the structural capacity is assessed on the basis of its severity – i.e. whether this is insignificant, significant or severe – in each case (refer to Section B2 for definitions). The engineer should consider the objectives contained in these definitions and refer to Table BA.4 for more specific guidance on determining the severity for common potential CSWs.

Note:

The 'severe' rating for potential CSWs should not be confused with potential severe structural weaknesses (SSWs).

Table BA.4: Guide to severity of potential critical structural weaknesses

	Effect on structural performance		
potential Critical Structural Weakness	**Severe**	**Significant**	**Insignificant**
Plan irregularity L-shape, T-shape, E-shape	Two or more wings length/width > 3.0, or one wing length/width > 4	One wing length/width > 3.0	All wings length/width ≤ 3.0
Long narrow building where spacing of lateral load resisting elements is …	> 4 times building width	> 2 times building width	≤ 2.0 times building width
Torsion (corner building)	Mass to centre of rigidity offset > 0.5 width	Mass to centre of rigidity offset > 0.3 width	Mass to centre of rigidity offset ≤ 0.3 width or effective torsional resistance available from elements orientated perpendicularly
Ramps, stairs, walls, stiff partitions	Clearly grouped, clearly an influence	Apparent collective influence	No or slight influence
Vertical irregularity Soft storey	Lateral stiffness of any storey < 0.7 of lateral stiffness of any adjoining storeys	Lateral stiffness of any storey < 0.9 of lateral stiffness of the adjoining storeys	Lateral stiffness of any storey ≥ 0.9 of lateral stiffness of the adjoining storeys
Mass variation	Mass of any storey < 0.7 of mass of adjoining storey	Mass of any storey < 0.9 of mass of adjoining storey	Mass of any storey ≥ 0.9 of mass of adjoining storey
Vertical discontinuity	Any element contributing > 0.3 of the stiffness/ strength of the lateral force resisting system discontinues vertically	Any element contributing > 0.1 of the stiffness/ strength of the lateral force resisting system discontinues vertically	Only elements contributing ≤ 0.1 of the stiffness/strength of the lateral force resisting systems discontinue vertically
Short columns Columns < 70% storey height between floors clear of confining infill, beams or spandrels	Either > 80% short columns in any one side Or > 80% short columns in any storey	> 60% short columns in any one side > 60% columns in any one storey	No, or only isolated, short columns, or Columns with width > 1.2 m, or Free column height/column width ≥ 2.5.
Pounding effect Vertical differences between floors ≤ 20% storey height of building under consideration			No penalty
Vertical differences between floors > 20% storey height of building under consideration	0 < separation < 0.005 H	0.005 H < separation < 0.01 H	Separation > 0.01 H
	where H = height to the level of the floor being considered		

	Effect on structural performance		
potential Critical Structural Weakness	**Severe**	**Significant**	**Insignificant**
Height difference effect No adjacent building, or height difference < 2 storeys			No penalty
Height difference 2-4 storeys	0 < separation < 0.005 H	0.005 H < separation < 0.01 H	Separation > 0.01 H
Height difference > 4 storeys	0 < separation < 0.005 H	0.005 H < separation < 0.01 H	Separation > 0.01 H, or Floors aligning and height difference < 2 storeys, **or** At least one building is lightweight construction
	where H = height of the lower building and separation is measured at H		
Site characteristics (refer to TA or regional council hazards maps, where available)	Unstable site. Structure prone to underslip and very susceptible to excessive loss of foundation support.	Signs of past site instability. Underslip may threaten structure and structure not capable of sustaining loss of foundation support.	Geohazards are not a significant threat to life in or immediately outside the building
	Significant liquefaction potential and building very susceptible to excessive settlement	Liquefaction potential and structure not capable of sustaining soil deformation	Liquefaction potential but structure capable of sustaining soil deformations

Compensating provisions (Step 3.6)

Factor F (or the compensating factor) has been introduced in the IEP assessment process. It reflects the engineers' confidence in the final building rating. In general, this factor has been devised to account for:

- any parameter including other CSWs that might not have been accounted for in the evaluation process discussed above, but that the engineer believes should be accounted for
- apparent CSWs that might have been compensated for in design
- hidden strengths and weaknesses
- compensation for over penalty
- higher levels of ductility than might have been assumed in the design of the building
- potential hazards to life (including from parts of buildings), and
- any other parameters.

Note:

Factor F is entirely based on the judgement of the engineer and therefore it is a requirement of the IEP that the factors that have led to the decision for Factor F are appropriately recorded.

In general, 1.0 is considered as the base number for Factor F. The factor should be less than 1.0 to reflect deficiencies not accounted in the process or to highlight that a detailed assessment of the building as a whole or of some specific parts is recommended. Similarly the factor could be more than 1.0 to reflect that the building has higher capacity than evaluated above. The limits on this compensating factor are as follows:

- No limit on factor less than 1.0
- Up to 2.5 for buildings up to three storeys high
- Up to 1.5 for buildings more than three storeys high.

Reasons for adopting a compensating factor higher than 1.0 include, but are not limited to:

- greater than minimum lengths of shear wall
- design for significantly higher gravity loading than current use requires
- need to compensate for otherwise severe effect of combinations of potential CSWs that are not mutually exclusive (e.g. when a single issue results in both a plan and a vertical irregularity – although in such cases it is acceptable to penalise assuming only one potential CSW)
- ductile detailing in pre-1976 buildings
- in timber houses, even older ones, higher ductility could be available
- compensating for inappropriate assignment of penalties (e.g. a soft-storey mechanism is unlikely if reasonable walls are present, in the direction under evaluation)
- presence of details that are known to improve performance (e.g. existence of bond beams in URM buildings)
- frame buildings with strong column-weak beam are unlikely to develop soft-storey mechanism despite having a stiffness discontinuity in the vertical direction
- buildings with long walls are unlikely to develop soft-storey mechanisms
- pounding against walls rather than columns, or wall and frame rather than between frame and frame structure
- pounding between lightweight and stiff heavy buildings is unlikely to be a serious issue
- the known resilience of timber buildings to earthquake shaking, and
- any other known factor.

It may be apparent that potential CSWs have been compensated for in design. This should be established by viewing building design/construction documentation as part of the assessment. Note that even where compensating design has been carried out, a building with discontinuities, such as those nominated as potential CSWs, will likely suffer more damage than a geometrically regular building.

There may be negative factors that are known but have not been included in the IEP. In such cases it is up to the judgement of the engineer to evaluate the potential life-safety risk and adjust the *%NBS* down accordingly. If a reasonable hazard exists due to structural or non-structural items it is recommended to set *%NBS* < 34 with a note that the earthquake rating is due to these items.

Possible negative factors include, but are not limited to, the:

- quality of previous retrofit, if any
- hierarchy of failure, and consequences, and
- hazard arising from parts of buildings such as face loaded infill panels, parapets, chimneys and stairs where this might be known.

These and other issues are discussed in more detail in Section B1, together with guidance on how to make allowance for them in an IEP.

The maximum value of Factor F has been set at 1.5 (no minimum) unless the building has no more than three storeys; in which case the maximum value has been set at 2.5 (also no minimum). The reason for the distinction based on height is that it is felt there is more scope for judgement for low rise structures and as any positive compensating factors are likely to have a more dramatic effect on earthquake performance.

It is expected that the engineer may need to revisit Factor F after a *%NBS* earthquake rating has been determined if this rating appears unreasonable or is not reflective of actual observed building behaviour. Such review is a part of the overall process as indicated in Section B3 and below.

Calculation of performance assessment ratio (PAR) (Step 3.4)

The calculation of PAR is simply the product of the factors identified and shown on Table IEP-3. The focus of the review is on the capacity to resist lateral load.

The earthquake rating for the building shall be taken as the lowest result for the two directions considered.

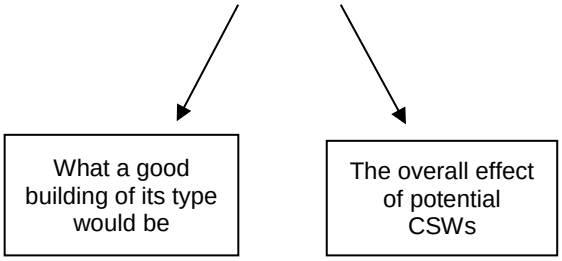
As noted above the engineer should now stand back and reflect on the appropriateness of the *%NBS* that has been determined. If the result is considered unrealistic or inappropriate, the engineer should review all steps including the available information on the building and whether this is sufficient, and also the basis for the Factor F. Several iterations may be required.

If the IEP cannot provide a result that the engineer is satisfied with by virtue of the limits on Factor F, the engineer shall note this in the assessment.

BA.2.4 Step 4 - Determine the percentage of new building standard, *%NBS* (Table IEP-4)

This is a simple calculation:

$$\%NBS = (\%NBS)_{\mathrm{b}}\, x\, PAR$$



BA.2.5 Step 5 – Is the earthquake rating less than 34*%NBS*? (Table IEP-4)

%NBS less than 34 ⟶ Building meets one of the criteria for an earthquake-prone building in terms of the Building Act. Further action is required; e.g. TA consideration, DSA, review of drawings or further inspections.

%NBS greater than or equal to 34[1] ⟶ If the assessment meets the requirements of the engineering basement in the EPB methodology and is accepted by the TA, the building does not require further action in terms of the Building Act unless further knowledge becomes available that suggests otherwise.

BA.2.6 Step 6 – Is the building an earthquake risk?(Table IEP-4)

%NBS less than 67 ⟶ Building is potentially an earthquake risk. Further action is recommended; e.g. DSA, review of drawings or further inspections.

%NBS greater than or equal to 67 ⟶ Building is unlikely to be an earthquake risk unless further knowledge becomes available that suggests otherwise.

BA.2.7 Step 7 – Provide provisional grading based on IEP (Table IEP-4)

The grading scheme shown in Part A is being promoted by the New Zealand Society for Earthquake Engineering to improve public awareness of earthquake risk and the relative risk between buildings.

It is not a requirement of the Building Act to provide a seismic grade but it is strongly recommended that this is recorded in order to promote this concept.

Seismic grading determined from the results of the IEP should be considered provisional and subject to confirmation by detailed assessment.

The NZSEE grading scheme is only intended to grade the building under consideration. Aspects such as the possible detrimental effects of neighbouring buildings or the hazards resulting from items that could be classified as building contents are not considered but may nevertheless be important considerations for building owners and tenants. These should be brought to their attention if this is appropriate for the level of assessment being undertaken.

[1] The target for an earthquake-prone building is defined in legislation as one third of the requirements for a new building. In these guidelines this target has been rounded up to be less than 34*%NBS*.

BA.2.8 Step 8 – Note identified potential SSWs from list provided (Table IEP-5)

Table IEP-5 has six tick boxes which only need to be considered if the building has more than three storeys and has heavy floors and/or a heavy roof.

The engineer should tick the first box if none of the listed potential SSWs have been identified as being present. Otherwise, he or she should tick as many of the other boxes as appropriate. These represent the five potential SSWs that represent particular weaknesses (vulnerabilities) that it is believed have significant potential to lead to catastrophic collapse and/or loss of egress that would result in a significant risk to occupants. (Refer to Section B2.2 for more details of these.)

The six tick boxes are:

1. None identified. This should not be construed as advice that none are present.
2. A weak or soft storey, except for the top storey.
3. Brittle columns and/or brittle beam/column joints the deformations of which are not constrained by other structural elements.
4. Flat slab buildings with lateral capacity reliant on low ductility slab to column connections.
5. No effective connection between primary seismic structural elements and diaphragms.
6. Seismically separated stairs with ledge and gap supports.

If <u>any</u> potential SSWs (items 2 to 6) have been ticked then careful consideration should be given before rating the building $\geq$34*%NBS*. The Factor F would then be set as considered appropriate, noting that a DSA is recommended to confirm the rating.

It is acknowledged that these structural weaknesses may only be recognisable from construction drawings and therefore an ISA based on a visual inspection only will not necessarily identify their presence.

The presence of any of these potential SSWs should also be noted in the covering letter (refer to Appendix BC).

Appendix BB: Initial Seismic Assessment of Unreinforced Masonry Buildings using an Attribute Scoring Methodology

BB.1 General

For URM buildings built prior to 1935, Steps 2 to 4 of the ISA can be carried out using the attribute scoring method outlined in this appendix. The *%NBS* is then determined directly from the total attribute score as described below.

The derivation of *%NBS* using the attribute scoring method assumes that all appendages likely to present a significant life safety hazard have been adequately secured or measures taken to remove the risk to life: e.g. provision of appropriately designed canopies or designated "no go" zones adjacent to the building.

If appendages have not been restrained the *%NBS* shall not be taken $\geq$34*%NBS*.

BB.2 Procedure

The recommended procedure is:

- complete the attribute scoring table, Table BB.1, using the guidance provided in Table BB.2
- then, from the total attribute score determine the *%NBS* from Table BB.3.

Interpolation may be used for intermediate attribute scores. While attributes may differ for each principal direction, it is the intention that the attribute score applies to the building as a whole. Therefore, it will be necessary to choose an attribute score that is representative of the building. This will not necessarily be the lowest score for either direction but can conservatively be taken as such.

Given that local collapse is viewed as having the same implications as total collapse, attributes should correspond to the weakest section of a building where relevant.

Table BB.1: Assessment of attribute score

Item		Attribute ranking				Assessed score	
		0	1	2	3	Long	Trans
1	**Structure continuity**	Excellent	Good	Fair	Poor or none		
2	**Configuration**						
2a	Horizontal regularity	Excellent	Good	Fair	Poor		
2b	Vertical regularity	Excellent	Good	Fair	Poor		
2c	Plan regularity	Excellent	Good	Fair	Poor		
3	**Condition of structure**						
3a	Materials	Sound	Good	Fair	Poor		
3b	Cracking or movement	Not evident	Minor	Moderate	Severe		
4	**Wall (URM) proportions**						
4a	Out-of-plane	Good			Poor		
4b	In-plane	Excellent	Good	Fair	Poor		
5	**Diaphragms**						
5a	Coverage	Excellent	Good	Fair	Poor		
5b	Shape	Excellent	Good	Fair	Poor		
5c	Openings	None			Significant		
6	**Engineered connections between floor/roof diaphragms and walls, and walls and diaphragms** capable of spanning between	Yes			No		
7	**Foundations**	Excellent	Good	Fair	Poor		
8	**Separation from neighbouring buildings**	Adequate			Inadequate		
		Total attribute score:		**For each direction**			
				For building as a whole			

Note:
For definition of grading under each attribute refer to Table BB.2.

Table BB.2: Definition of attributes and scores

	Attributed score[1]
Attribute Item (1): Structure continuity	
Totally unreinforced masonry	3
Some continuity, e.g. unreinforced masonry with a reinforced concrete band at roof *or* floor level	2
Good continuity, e.g. unreinforced masonry with reinforced bands at *both* roof and floor levels	1
Full continuity (i.e. vertical stability not reliant on URM), e.g. reinforced concrete or steel columns and beams with URM walls/infill or separate means of vertical support provided to floors and roof	0
Attribute Item (2): Configuration	
(a) Plan regularity	
Severe eccentricity, i.e. distance between storey centre of rigidity and the centre of mass for all levels above that storey, $e_{\mathrm{d}} < 0.3\ b$ (b = longest plan dimension of building perpendicular to direction of loading)	3
$e_{\mathrm{d}} < 0.3\ b$	2
$e_{\mathrm{d}} < 0.2\ b$	1
Building symmetrical in both directions	0
(b) Vertical regularity	
Vertical stiffness discontinuities or discontinuities in load paths present	3
All walls continuous to foundations	2
and no soft storeys *and* minimal vertical stiffness changes	1
and no weak storeys *and* no significant mass irregularities	0
where: • *soft storey* is a storey where the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average stiffness of the three storeys above • *weak storey* is a storey where the storey strength is less than 80% of the strength of the storey above • *mass irregularity* exists if the mass varies by more than 50% from one level to another (excluding light roofs which should be considered as a part of the building).	
(c) Diaphragm shape	
Sharp re-entrant corners present where the projection of the *wing* beyond the corner > 0.15 b	3
Regular in plan	0
Attribute Item (3): Condition of structure	
(a) Materials	
Poor, i.e. considerable deterioration, fretting or spalling, etc., or lime or other non-competent mortar or rubble wall construction	3
Fair, i.e. deterioration leading to reduced strength	2
Good, i.e. minor evidence of deterioration of materials	1
Sound	0

		Attributed score[1]
(b)	Cracking or movement	
	Severe, i.e. a considerable number of cracks or substantial movement leading to reduced strength *or* isolated large cracks	3
	Moderate	2
	Minor	1
	Non-evident	0
Attribute Item (4): Wall (URM) proportions		
(a)	Out-of-plane performance	
	Poor:	3
	- for one storey buildings $h_w/t > 14$ and $l_w/t > 7$	
	- for multi-storey buildings:	
	top storey $h_w/t > 9$ and $l_w/t > 5$	
	other storeys $h_w/t > 20$ and $l_w/t > 10$	
	Good (not poor)	0
	Where:	
	h_w = height of wall between lines of positive lateral restraint, and	
	l_w = length of wall between lines of positive lateral restraint	

(b) In-plane performance[2]

	A_p/A_w			
	One storey building	**2 and 3 storey buildings**		
		Top storey	**Other storeys**	
Poor	≥25	≥20	≥17	3
Fair	>20	>15	>12	2
Good	>15	>10	>7	1
Excellent	≤15	≤10	≤7	0

where:

A_w = cross sectional area of all URM walls/wall sections extending over full height of storey

A_p = plan area of building above storey of interest.

For buildings of greater than 3 storey take attribute score = 3

Attribute Item (5): Diaphragms (refer to Figure BB.1)		
(a)	Coverage	
	No diaphragm	3
	Full diaphragm	0
	To achieve an attribute ranking of 0 requires a diaphragm to be present at each level, including roof level, covering at least 90% of the building plan area at each level. Interpolation for attribute rankings of 1 and 2 may be made using judgement on the extent of coverage. Note that unless the diaphragm is continuous between walls, its effectiveness may be minimal.	

<table>
<tr><th colspan="6"></th><th>Attributed score[1]</th></tr>
<tr><td>(b)</td><td colspan="2">Shape</td><td colspan="4">Limiting span to depth ratios for diaphragms of different construction material</td><td></td></tr>
<tr><td></td><td colspan="2"></td><td>Concrete</td><td>Sheet materials</td><td>T&G timber</td><td>Steel roof bracing</td><td></td></tr>
<tr><td></td><td colspan="2">Poor</td><td>> 4</td><td>> 4</td><td>> 3</td><td>> 5</td><td>3</td></tr>
<tr><td></td><td colspan="2">Fair</td><td>< 4</td><td>< 4</td><td>< 3</td><td>< 4</td><td>2</td></tr>
<tr><td></td><td colspan="2">Good</td><td>$\leq$ 3</td><td>$\leq$ 3</td><td>$\leq$ 2</td><td>$\leq$ 3.5</td><td>1</td></tr>
<tr><td></td><td colspan="2">Excellent</td><td colspan="4">As for good, but in addition the projection of “wings” beyond sharp re-entrant corners $< 0.5\,b$.</td><td>0</td></tr>
<tr><td>(c)</td><td colspan="6">Openings</td><td></td></tr>
<tr><td></td><td colspan="6">Significant openings</td><td>3</td></tr>
<tr><td></td><td colspan="6">No significant openings</td><td>0</td></tr>
<tr><td></td><td colspan="6">Interpolation for attribute rankings of 1 and 2 may be made using judgement.</td><td></td></tr>
<tr><td></td><td colspan="6">Significant openings are those which exceed the limiting values given below.</td><td></td></tr>
<tr><td></td><td colspan="2">Diaphragm construction material</td><td colspan="4">Limiting values of</td><td></td></tr>
<tr><td></td><td colspan="2"></td><td colspan="2">X/b</td><td colspan="2">Y/D</td><td></td></tr>
<tr><td></td><td colspan="2">Concrete</td><td colspan="2">0.6</td><td colspan="2">0.5</td><td></td></tr>
<tr><td></td><td colspan="2">Sheet material</td><td colspan="2">0.5</td><td colspan="2">0.4</td><td></td></tr>
<tr><td></td><td colspan="2">T&G timber</td><td colspan="2">0.4</td><td colspan="2">0.3</td><td></td></tr>
<tr><td></td><td colspan="6">Refer to Figure BB.1 for definition of terms.</td><td></td></tr>
<tr><td colspan="6">Attribute Item (7): Foundations</td><td></td></tr>
<tr><td colspan="6">Separate foundations with no interconnection or unreinforced piles (unless ramification of pile failure is assessed to be minor).</td><td>3</td></tr>
<tr><td colspan="6">Pads, strips or piles with some interconnection. Concrete piles to be reinforced unless ramification of pile failure is assessed to be minor.</td><td>2</td></tr>
<tr><td colspan="6">Pads, strips or piles with good interconnection in both directions.</td><td>1</td></tr>
<tr><td colspan="6">Concrete raft with sound connections to walls.</td><td>0</td></tr>
<tr><td colspan="6">Attribute Item (8): Separation</td><td></td></tr>
<tr><td colspan="6">Inadequate – no separation provided or obviously inadequate provisions for separation</td><td>3</td></tr>
<tr><td colspan="6">Adequate – separation provided</td><td>0</td></tr>
<tr><td colspan="7">Note:
1. Individual attribute scores may be interpolated.
2. This is an index describing the extent of brick walls within the building. The numbers given are only loosely related to lateral load capacity.</td></tr>
</table>

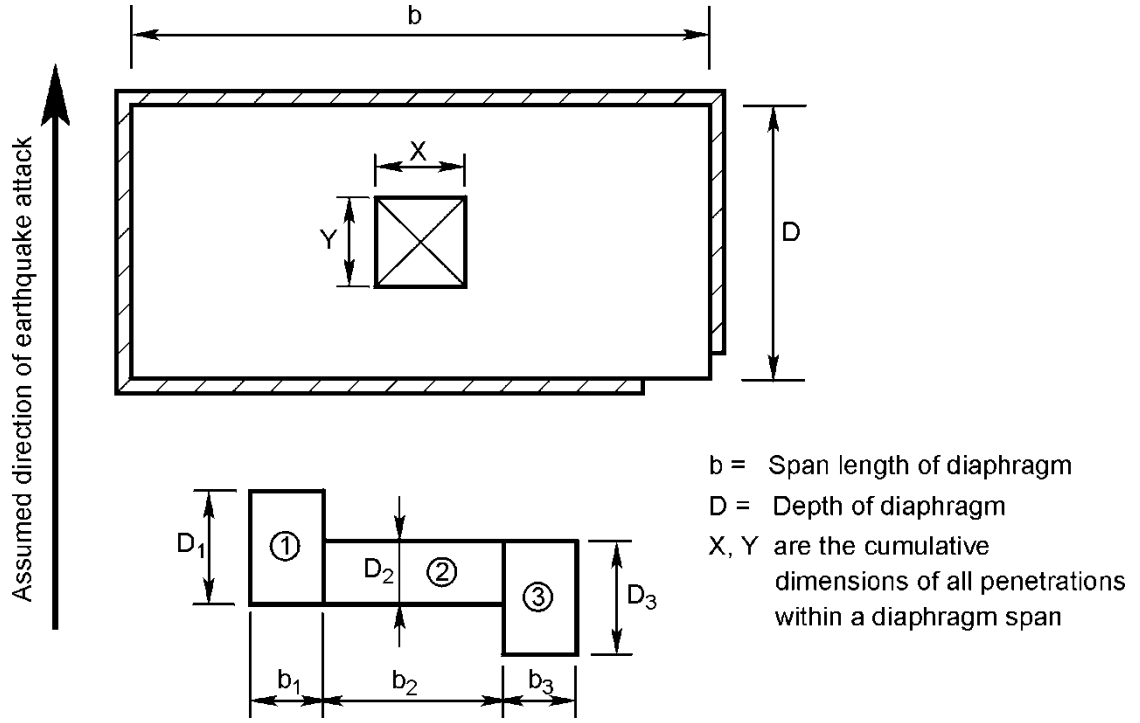


Figure BB.1: Diaphragm parameters

Table BB.3: Assessment of *%NBS* from attribute score

Item	Attribute score	*%NBS*
1	A score of 0 for all attribute scoring items	$67\ x\ 1/Rx\ 0.4/Z$[1]
2	Less than or equal to 1 for all of attribute scoring items 1 to 6 inclusive, and less than 2 for each of attribute scoring items 7 and 8	$34\ x\ 1/Rx\ 0.4/Z$
3	As for 2 but a score of 0 for attribute scoring item 1	$40\ x\ 1/Rx\ 0.4/Z$
4	5 < total attribute score ≤ 10	$20\ x\ 1/Rx\ 0.4/Z$
5	10 < total attribute score ≤ 15	$15\ x\ 1/Rx\ 0.4/Z$
6	15 < total attribute score ≤ 25	$10\ x\ 1/Rx\ 0.4/Z$
7	Total attribute score > 25	$5\ x\ 1/Rx\ 0.4/Z$

Note:

1. R and Z are defined in NZS 1170.5:2004.

Appendix BC: Template Covering Letter – Building Owner or Tenant Commissioned IEP

Building Owner
PO Box XYZ
SHAKESVILLE

Date

Dear Sir

Initial Seismic Assessment of Building at XX Tremor Grove

We have now completed an Initial Seismic Assessment (ISA) of the building at XX Tremor Grove, Shakesville using the Initial Evaluation Procedure (IEP) as described in Part B of the guideline document, *The Seismic Assessment of Existing Buildings-Technical Guidelines for Engineering Assessments*, dated **[XXX]**. The assessment was carried out after completing a site visit [*add anything else that formed the basis of the assessment: e.g.* reviewing the original structural drawings *etc.*].

Executive Summary

*Provide the final **potential** earthquake rating and building grade. Note the Importance Level (in accordance with NZS 1170.5:2004) that was assumed to apply as this will define the new building standard that the building is rated against. Use the following form:* XX*%NBS* (ILY).

Give the potential status of the building in relation to 34*%NBS and the earthquake risk (*67*%NBS) criteria.*

Note if any of the potential SSWs in IEP Table IEP-5 have been identified. If they have, also note that these could potentially present an enhanced risk in severe earthquake shaking.

The ISA is considered to provide a relatively quick, high-level and qualitative measure of the building's performance. A more reliable result will be obtained from a Detailed Seismic Assessment (DSA) and is recommended for this building. A DSA could find structural weaknesses not identified from the IEP, or it could find that identified potential CSWs have been addressed in the design of the building.

Introduction

Provide the background and basis of the assessment, i.e. that it has been based on the IEP as defined by the Technical Guidelines for Engineering Assessments referenced above, and if it also meets the requirements of an engineering assessment as prescribed in the EPB methodology

Background to the IEP and its Limitations

The IEP procedure was developed in 2006 by the New Zealand Society for Earthquake Engineering (NZSEE) and updated in 2017 to reflect experience with its application and also as a result of experience from the Canterbury earthquakes of 2010/11. It is a tool to assign a percentage of New Building Standard (*%NBS*) rating and associated grade to a building as part of an Initial Seismic Assessment of existing buildings.

The IEP enables building owners and managers to review their building stock as part of an overall risk management process.

Characteristics and limitations of the IEP include:

- An IEP assessment is primarily concerned with life safety. It does not consider the susceptibility of the building to damage, and therefore to economic losses.
- It tends to be somewhat conservative, identifying some buildings as earthquake prone, or having a lower *%NBS* score, which subsequent detailed investigation may indicate is less than actual performance. However, there will be exceptions, particularly when potential critical structural weaknesses (CSWs) are present that have not been recognised from the level of investigation employed.
- An IEP can be undertaken with variable levels of available information: e.g. exterior only inspection, structural drawings available or not, interior inspection, etc. The more information available, the more representative the IEP result is likely to be. The IEP records the information that has formed the basis of the assessment and consideration of this is important when determining the likely reliability of the result.
- It is an initial, first-stage review. Buildings or specific issues which the IEP process flags as being problematic or as potentially critical structural weaknesses need further detailed investigation and evaluation. A Detailed Seismic Assessment is recommended if the seismic status of a building is critical to any decision making.
- The IEP assumes that buildings have been designed and built in accordance with the building standard and good practice current at the time. In some instances, a building may include design features ahead of its time, leading to better than predicted performance. Conversely, some unidentified design or construction issues not picked up by the IEP process may result in the building performing not as well as predicted.
- It is a largely qualitative process, and should be undertaken or overseen by an experienced engineer. It involves considerable knowledge of the earthquake behaviour of buildings, and judgement as to key attributes and their effect on building performance. Consequently, it is possible that the *%NBS* derived for a building by independent experienced engineers may differ.
- An IEP may over-penalise some apparently critical features which could have been satisfactorily taken into account in the design.
- An IEP does not take into account the seismic performance of non-structural items such as ceilings, plant, services or general glazing that are not considered to present a significant life safety hazard.

Experience to date is that the IEP is a useful tool to identify potential issues and expected overall performance of a building in an earthquake. However, the process and the associated *%NBS* rating and grade should be considered as only providing an indicative indication of the building's compliance with current code requirements. A detailed investigation and analysis of the building will typically be required to provide a definitive assessment.

Include if appropriate and if comfortable that the rating reflects the building's expected behaviour.

The IEP has been based on a review of drawings and an inspection of both the interior and exterior of the building and can be considered to be a comprehensive assessment at the ISA level. The rating determined is greater than or equal to 34*%NBS* and therefore, if ratified by the TA, the building should not be considered as earthquake prone.

Basis for the Assessment

The information we have used for our IEP assessment includes:

List the information that has been available to carry out the assessment, e.g. structural drawings, whether the site visit included an interior inspection, and basis for the assessment of geotechnical conditions.

Building Description

The building located at [WW Tremor Grove, Shakesville] is a [X] storey structure designed in [YYYY]. It is currently used as [ZZZZZZ].

Provide a brief description of the building including relevant features such as age, structural configuration, lateral seismic resisting system in each direction, relationship to neighbouring buildings and foundation/soil conditions.

IEP Assessment Results

Our IEP assessment of this building indicates the building can achieve XXX%*NBS* (ILY) in the longitudinal direction and YYY%*NBS* (ILY) in the transverse direction. The IEP assessment of this building therefore indicates an overall earthquake rating of ZZZ%*NBS* (ILY), corresponding to a 'Grade X' building as defined by the NZSEE building grading scheme. This is [*above/below*] 34%*NBS* (one of the tests the TA will apply to determine the buildings earthquake-prone building status), but [*above/below*] the threshold for earthquake risk buildings (67%*NBS*) as recommended by the NZSEE.

The key assumptions made during our assessment are shown in Table 1 below. Refer also to the attached IEP assessment and engineering assessment technical summary

Table 1: IEP Assessment Results

Fill in the scoring assumptions from the IEP.

Briefly outline the justification for the scoring decision. Justification is an important aspect of the IEP as it records the judgement decisions of the engineer.

IEP Item	Assumption	Justification
Date of Building Design		
Subsoil Type		

IEP Item	Assumption	Justification
Building Importance Level		
Ductility of Structure		
Plan Irregularity Factor, A		
Vertical Irregularity Factor, B		
Short Columns Factor, C		
Pounding Factor, D		
Site Characteristics		
Factor F		

IEP Grades and Relative Risk

Table 2 taken from the Technical Guidelines referred to earlier provides the basis for a proposed grading system for existing buildings, as one way of interpreting the *%NBS* earthquake rating.

Table 2: Relative Earthquake Risk

Building Grade	Percentage of New Building Standard (*%NBS*)	Approx. Risk Relative to a New Building	Life-safety Risk Description
A+	>100	<1	low risk
A	80 to 100	1 to 2 times	low risk
B	67 to 79	2 to 5 times	low or medium risk
C	34 to 66	5 to 10 times	medium risk
D	20 to 33	10 to 25 times	high risk
E	<20	more than 25 times	very high risk

This building has been classified by the IEP as a Grade [X] building and is therefore considered to be a [YYYYYYYY] life-safety risk.

The NZSEE (which provides authoritative advice to the legislation makers, and should be considered to represent the consensus view of New Zealand structural engineers) classifies a building achieving greater than 67*%NBS* as “low or medium risk”, and having “acceptable (improvement may be desirable)” building structural performance.

Seismic Restraint of Non-Structural Items

During an earthquake, the safety of people can be put at risk due to non-structural items falling on them. These items should be adequately seismically restrained, where possible, as specified by NZS 4219:2009 "The Seismic Performance of Engineering Systems in Buildings".

An assessment has not been made of the bracing of the ceilings, in-ceiling ducting, services and plant or contents. We have also not checked whether or not tall or heavy furniture has been seismically restrained. These issues are outside the scope of this initial assessment but could be the subject of another investigation.

If particular potential hazards that could lead to an enhanced risk have been identified during the assessment but they do not influence the %NBS rating (e.g. geohazards outside the site boundaries, tsunami, adjacent URM walls, etc.) they should be noted here using the following wording.

Other Issues

Although their identification is beyond the scope of this assessment and they do not influence the *%NBS* score, the following hazard(s) have been identified as potential issue(s) for this building:

List any hazards.

Conclusion

Our ISA assessment for this building, carried out using the IEP, indicates an overall score of XXX*%NBS* (ILY) which corresponds to a Grade [X] building, as defined by the NZSEE building grading scheme. This is [*above/below*] the threshold for earthquake-prone buildings (34*%NBS*), and [*above/below*] the threshold for earthquake -risk buildings (67*%NBS*) as defined by the NZSEE.

The ISA is considered to provide a relatively quick, high-level and qualitative measure of the building's performance. In order to confirm the seismic performance of this building with more reliability you may wish to request a DSA. A DSA is likely to focus on the following issues:

List the issues identified, including all CSWs (including those identified when establishing the F Factor).

A DSA would also investigate other potential structural weaknesses that may not have been considered in the Initial Seismic Assessment.

If any potential Severe Structural Weaknesses (SSWs) have been noted in Table IEP-5 of the IEP include the following:

While completing this ISA we identified the following potential Severe Structural Weakness(es) in the building:

List any potential Severe Structural Weaknesses noted in Table IEP-5 of the IEP.

If confirmed as structural weaknesses these could have the potential to significantly reduce the resilience of the building and adversely affect its performance in severe earthquakes.

We trust this letter and Initial Seismic Assessment meets your current requirements. We would be pleased to discuss further with you any issues raised in this report.

Please do not hesitate to contact me if you would like clarification of any aspect of this letter.

Experienced, Competent and Appropriately Trained Structural Engineer

CPEng

Encl: IEP Assessment
Engineering Technical Summary

The Seismic Assessment of Existing Buildings

Technical Guidelines for Engineering Assessments

July 2017

Detailed Seismic Assessment

Part C

nzsee
NEW ZEALAND SOCIETY FOR EARTHQUAKE ENGINEERING

Sesoc

NEW ZEALAND GEOTECHNICAL SOCIETY INC

MINISTRY OF BUSINESS, INNOVATION & EMPLOYMENT
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PART C

C1 General Issues

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This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

This document may be downloaded from www.EQ-Assess.org.nz in parts:

1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

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Please go to www.EQ-Assess.org.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.EQ-Assess.org.nz

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C1. General Issues

C1.1 Introduction

C1.1.1 Overview of Detailed Seismic Assessments

Part C sets out a methodology for engineers to conduct a Detailed Seismic Assessment (DSA) to assess the structural load paths of the building, the capacities of the structural elements, the likely inelastic mechanisms, the global building response to earthquake shaking and the earthquake rating for the building.

Note:

Please refer to Part A for the basis behind the approaches adopted for the DSA and the seismic assessment process in general.

Familiarity with the underlying principles in Part A is considered essential for those completing seismic assessments in accordance with these guidelines. These principles are applicable to all DSAs irrespective of analysis methods or material type.

The detailed procedures for assessment given in Part C are intended to provide a more reliable and consistent outcome than is available from the Initial Seismic Assessment (ISA) (described in Part B).

Note:

It is highly recommended that engineers develop a qualitative view of the building behaviour as a first step to a DSA. While achieving this by carrying out an ISA before the DSA is not a prerequisite, it is strongly encouraged.

The value of conducting an ISA in this context is not necessarily the earthquake rating that it delivers but the opportunity to gain a holistic view of the building's potential structural weaknesses.

The focus of the DSA is to achieve an understanding of the likely behaviour of the building in earthquakes by quantifying the strength and deformation capacities of the various structural elements, by checking the building's structural integrity against the loads/ deformations (demands) that would be used for the design of a similar building on the same site.

Note:

The overall assessment process requires a significant departure in mindset when compared to the process used for a conventional new building design, in which a prescriptive "deemed-to-comply" approach is generally adopted. Although the procedures presented for the DSA are focused on quantifying the building capacity, the whole approach is necessarily still reliant on the judgement that can be applied by the engineer.

As is the case for the ISA, the DSA can be completed to various levels of detail depending on the circumstances and the level of reliability required. The onus is on the assessing engineer to understand the level of reliability available from the chosen assessment approach and to be able to articulate this to the end user of the DSA.

Note:

Situations will vary from small simple buildings to large complex ones. The approach to determine demand and capacity will be up to the assessing engineer. The intention of these guidelines is to help the engineer to adopt the simplest available approach consistent with the circumstances and still achieve a consistent assessment outcome.

Existing buildings involve a wide range of structural types, materials and details. As the procedures presented in these guidelines focus on the most common situations and elements they will not cover every aspect the assessing engineer is likely to encounter. The intention is that an experienced earthquake engineer will be able to adapt and extend the guidance to best match the particular situation.

Communicating the results of a DSA to end users in an appropriate and consistent manner is considered a crucial aspect of the assessment process and one that warrants particular attention by the engineer.

C1.1.2 Definitions and acronyms

ADRS	Acceleration-displacement response spectrum (spectra)
BPON	Basic Performance Objective Equivalent to New Building Standards defined in ASCE 4-13 (2014) for use with that document
Catastrophic collapse	Complete collapse of one or more storeys in a building
Collapse prevention (CP) performance level	A performance level defined in ASCE 41-13 (2014) for use with that document
Critical structural weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be potential critical structural weaknesses.
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines
Earthquake Prone Building (EPB)	A legally defined category which describes a building that has been assessed as likely to have its ultimate capacity (which is defined in Regulations) exceeded in moderate earthquake shaking. In the context of these guidelines it is a building with an earthquake rating of less than 34*%NBS* (less than one third of new building standard).
Earthquake rating	The rating given to a building as a whole to indicate the seismic standard achieved in regard to human life safety compared with the minimum seismic standard required of a similar new building on the same site. Expressed in terms of percentage of new building standard achieved (*%NBS*). Also see earthquake score.
Earthquake Risk Building (ERB)	A building that falls below the threshold for acceptable seismic risk, as recommended by NZSEE (i.e. <67*%NBS* or two thirds new building standard)
Earthquake score	The score given to an individual aspect of the building (member/element/non-structural element/foundation soils) to indicate the seismic standard achieved in regard to human life safety compared with the minimum seismic standard required for the same aspect in a similar new building on the same site. Expressed in terms of percentage of new building standard achieved (*%NBS*). The aspect with the lowest earthquake score is the CSW and this score will represent the earthquake rating for the building.
Importance Level (IL)	Categorisation defined in the New Zealand Loadings Standard, NZS 1170.0:2002, used to define the ULS shaking for a new building based on the consequences of failure. A critical aspect in determining new building standard.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.
Life safety (LS) performance level	A performance level defined in ASCE 41-13 (2014) for use with that document
Maximum Considered Earthquake (MCE)	Sometimes referred to as the Maximum Credible Earthquake. Often used to define the maximum level of shaking that needs to be considered in a design or assessment process.
Moderate earthquake (shaking)	The level of earthquake shaking (defined in Regulations) used in the process to determine whether or not a building is earthquake prone
Nonlinear time history analysis (NLTHA)	An analysis of the building using strong motion earthquake records and modelling the nonlinear behaviour of the structure (also referred to as nonlinear response history analysis)
Non-structural element	An element within the building that is not considered to be part of either the primary or secondary structure

(Performance) step change	A significant change in seismic performance with increasing earthquake shaking levels typically associated with collapse of a building rather than just an increase in damage to its members/elements. Can be in the structure and/or the foundations/foundation soils.
Primary gravity structure	Portion of the main building structural system identified as carrying the gravity loads through to the ground. Also required to carry vertical earthquake induced accelerations through to the ground. May also be part of the primary lateral structure.
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be part of the primary gravity structure.
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects this is determined using probable material strengths. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Probable material strength	The expected or estimated mean material strength. For geotechnical issues assessed in accordance with these guidelines it is typically the ultimate geotechnical strength/resistance that would be assumed for design.
Secondary structure	Portion of the structure that is not part of either the primary lateral or primary gravity structure but, nevertheless, is required to transfer inertial and vertical loads for which assessment/design by a structural engineer would be expected. Includes precast panels, curtain wall framing systems, stairs and supports to significant building services items.
Serviceability limit state (SLS)	Limit state defined in the New Zealand loadings standard NZS 1170.5:2004
Severe structural weakness (SSW)	A defined structural weakness that is potentially associated with catastrophic collapse and for which the probable capacity may not be reliably assessed based on current knowledge
Significant life safety hazard	A hazard resulting from the loss of gravity load support of a member/element of the primary or secondary structure, or of the supporting ground, or of non-structural elements that would reasonably affect a number of people. When shelter under normally expected furniture is available and suitable, mitigation of the hazard below a significant status is assumed.
Simple Lateral Mechanism Analysis (SLaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (push-over) relationship for the building as a whole
SSI	Soil-structure interaction
SSNS element	Secondary structural or non-structural building element
(Structural) element	Combinations of (structural) members that can be considered to work together; e.g. the piers and spandrels in a penetrated wall, or beams and columns in a moment resisting frame
(Structural) member	Individual items of a building structure; e.g. beams, columns, beam/column joints, walls, spandrels, piers
(Structural) resilience	Ability of the building as a whole to perform acceptably from a structural and geotechnical point of view at levels of earthquake shaking greater than XXX%ULS shaking. Includes potential impact of the supporting soils on the performance of the building structure.
Structural system	Combinations of structural elements or structural sub systems that form a recognisable means of lateral or gravity load support; e.g. moment resisting frame, frame/wall. Also used to describe the way in which support/restraint is provided by the foundation soils.

Structural weakness (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100*%NBS.* Note that an aspect of the building structure scoring less than 100*%NBS* but greater than or equal to 67*%NBS* is still considered to be a structural weakness even though it is considered to represent an acceptable risk.
Ultimate capacity (seismic)	A term defined in regulations that describes the limiting capacity of a building for it to be determined to be an earthquake-prone building. This is typically taken as the probable capacity but with the additional requirement that exceeding the probable capacity must be associated with the loss of gravity support (i.e. it creates a significant life safety hazard).
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings
Unreinforced masonry (URM)	A member or element comprising masonry units connected together with mortar and not containing any steel, timber, cane or other reinforcement
(XXX)*%NBS*	The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage. It is the rating given to a building as a whole expressed as a percent of new building standard achieved, based on an assessment of the expected performance of an existing building relative to the minimum that would apply under the Building Code (Schedule 1) to the Building Regulations 1992) to a new building on the same site with respect to life safety. A score for an individual building element is also expressed as a percent of new building standard achieved. This is expected to reflect the degree to which the individual element is expected to perform in earthquake shaking compared with the minimum performance prescribed for the element in Clause B1 of the Building Code (Schedule 1 to the Building Regulations 1992) with respect to life safety. The *%NBS* rating for the building as a whole takes account of, and may be governed by, the scores for individual elements.
(XXX)%ULS shaking (demand)	Percentage of the ULS shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.

C1.1.3 Notation, symbols and abbreviations

Symbol	Meaning
%NBS	Percentage of new building standard as assessed by application of these guidelines
A_g	Gross section area of the column
f'_c	Probable concrete compressive strength
S_p	Structural performance factor in accordance with NZS 1170.5:2004
v_c	Probable shear stress carried by concrete mechanism in a shear wall
v_s	Probable shear stress carried by shear reinforcement in a shear wall

C1.2 Outline of Part C

Section C1 – General Issues

Section C1 (this section) provides an overview to the DSA process. It explains the objectives and sets out key steps for an assessment at this level, including specific guidance on the calculation of *%NBS* in the context of a DSA.

This section also covers the use of alternative assessment procedures, categorisation of assessments based on the influence of geotechnical aspects, the approach to building inspection and investigation, and reporting of DSA results. A recommended report template is included as Appendix C1A.

Section C2 – Assessment Procedures and Analysis Techniques

Section C2 sets out the DSA procedures proposed by these guidelines.

The section specifies general analysis requirements including basic assumptions, selection of seismic analysis procedures, requirements and limitations for the types of analysis procedure, and specific considerations of CSWs. The procedures presented include a first principles, mechanism-based method based on either a force based or displacement based approach.

Note:

It is expected that the engineer will use Section C2 in conjunction with the relevant specific provision chapters (Section C5 to C10) as the detailed assessment progresses.

Section C3 – Earthquake Demands

Section C3 explains how to determine the earthquake hazard and loading requirements used to assess the ultimate limit state (ULS) demands that are then used to determine the earthquake rating.

This section relates the guidelines to the appropriate version of the New Zealand loadings standard NZS 1170.5:2004 depending on the purpose of the assessment, and it provides guidance on the derivation of spectral displacement demand and the acceleration-displacement response spectra (ADRS). It also covers the determination of earthquake demand (e.g. peak ground acceleration, number of cycles, representative (effective) magnitude) for geotechnical considerations.

It is intended that all provisions of NZS 1170.5 that relate to defining the seismic demands for a new building will also apply for the assessment. This includes aspects such as allowance for irregularity, accidental eccentricity, direction of actions, structural performance factor, requirements for parts and components and P-delta.

Section C4 – Geotechnical Considerations

Section C4 provides guidance for considering geotechnical behaviour and its impact on the seismic behaviour and earthquake rating of existing buildings. This section includes

guidance on the recommended interactions between structural engineers and geotechnical engineers and their particular roles and responsibilities. It also explains the identification and assessment of geohazards, selection of geotechnical parameters, and the consideration of soil-structure interaction (SSI) and foundation systems.

Note:

As all structural assessments are expected to include consideration of the influence the ground can have on structural performance, engineers assessing buildings for a DSA are expected to be familiar with the information in this section.

Sections C5 to C9 – Recommendations for specific materials

These sections include provisions for various common construction materials (concrete, structural steel, moment resisting frames with infill masonry panels, unreinforced masonry (URM) and timber). They include specific recommendations for buildings generically constructed of the specific material as well as guidance on establishing capacities for structural elements and components made from that material.

It is recognised that existing buildings often comprise elements and components of different construction materials that work together to provide resistance to seismic shaking. Each material section aims to take a consistent approach to establishing element and component capacities to allow the capacities to be integrated, as appropriate, after making due allowance for deformation compatibility issues.

Where specific guidance is not provided for a particular material of situation it is intended that the relevant current material design standard will be used but substituting probable for characteristic material strengths and using nominal capacities based on these.

Note:

The material sections also include information on the observed performance and historical construction practices relating to the particular material. It is recommended that all engineers become familiar with these sections before attempting an assessment. This is because without an holistic view of the expected performance of the particular materials and configurations it will be impossible to apply the considerable judgements that are intended and required by these guidelines.

Section C10 – Secondary Structural Elements and Critical Non-structural Components

Section C10 provides guidance on the assessment of secondary structural and non-structural (SSNS) elements that would be expected to pose a significant life safety hazard (refer to Part A). How these influence the overall earthquake rating for the building is indicated in Section C1.5.

C1.3 Objectives of a Detailed Seismic Assessment

The objectives of the DSA procedure set out in these guidelines are to:

- provide a procedure that allows different engineers carrying out assessments to consistently assess the level of earthquake shaking at which the ultimate capacity (seismic) is reached for an existing building and to identify the point at which the conditions are met for a significant life safety hazard
- determine an earthquake rating for that building in terms of *%NBS* that is more reliable than the rating available from an ISA
- provide details on the expected mode of failure to assist territorial authorities to make decisions on life safety issues to determine earthquake prone building status
- determine whether or not a building is an earthquake risk building (ERB) as defined in these guidelines (i.e. <67*%NBS*)
- provide guidance on the likely needs for retrofit.

Note:

Some building owners/occupiers may also be interested in serviceability aspects such as post-earthquake functionality and the ability to occupy the building after an earthquake. While these aspects are not covered specifically by these guidelines they may need to be considered and commented on as part of providing holistic advice and meeting a wider brief than assumed for these guidelines.

The approach taken to performing a DSA may vary considerably depending on the circumstances and stated objectives. Many buildings will not require, or justify, the use of lengthy and detailed analyses. For some buildings, effort may even be better spent in completing an appropriate retrofit rather than necessarily understanding fully how the existing building configuration might perform.

Note:

An example of this is a simple building that is obviously earthquake prone and that the building owner has already decided to strengthen it seismically. In this case, establishing the *%NBS* of the un-retrofitted building quantitatively may be unnecessary. Instead, the effort may be better spent analysing the building assuming that the intended retrofit has been completed.

For more complicated buildings, proceeding immediately to a strengthening scheme without completing a DSA may be counterproductive unless there is significant prior knowledge of the building. A DSA is likely to be essential in such cases to gain a suitable understanding of the building and to identify any significant issues that need to be addressed.

C1.4 Key Steps

The key steps in the DSA procedure covered by these guidelines are as follows (also refer to Figure C1.1).

Note:

It should not be assumed that the assessment will always proceed in a linear, step by step process. Iteration should be expected and will generally be required, as indicated in Figure C1.1.

The possibility of using alternative assessment procedures is acknowledged but is not the focus of the guidelines. The exception is use of a nonlinear time history analysis (NLTHA) approach following the approach set out in ASCE 41-13 (2014) (refer to Section C1.6 for specific guidance).

Step 1 Establish the scope of work and assessment objectives

It is important that the scope of work and objectives of the DSA are well understood from the outset by all relevant parties.

In particular, engineers need to be aware of the complexity of the building structure, the likely assessment procedure and analysis techniques to be employed, the level of documentation/drawings available, and the inputs likely to be required from other disciplines (e.g. geotechnical engineers, heritage architects, etc.).

Step 2 Information collection, initial review, appraisal of building complexity and input required from a geotechnical engineer

Collect relevant information and documents about the building including drawings, design feature reports, geotechnical data, calculations and specifications, and any historical material such as test results, previous assessments and inspection reports (if available). Obtaining this information often involves detailed research of the property files in the relevant Territorial Authority's records, the building designer's archives and building owners' records.

If the building has been previously altered or strengthened collect all available drawings, calculations and specifications for this work.

Review the available information and structural drawings thoroughly to determine the lateral and gravity load resisting structural systems, potential SWs, SSWs, fall hazards and "hot spots" for on-site inspection.

Note:

Secondary structural and non-structural elements will not always be shown on the structural drawings and may need to be identified from site inspections (refer to Step 3).

If there is no existing ISA conduct an initial appraisal of the building structure's overall complexity following the drawing review. Completing an ISA is considered an excellent way of gaining a high level view of the issues that might exist in the building.

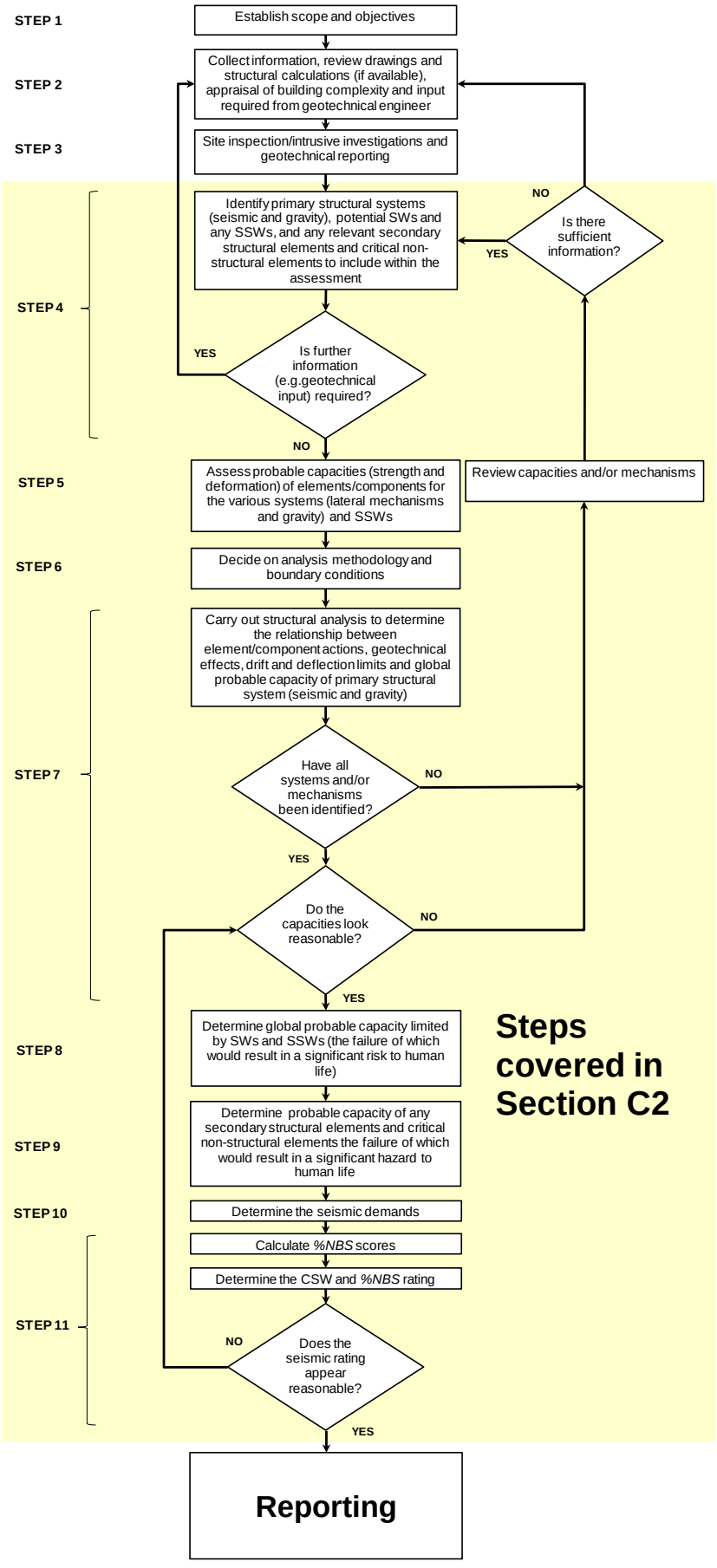

Figure C1.1: Detailed Seismic Assessment process

This initial stage of drawing review and appraisal will inform the assessment strategy and on-site inspection, allowing the engineers to concentrate effort on the key elements.

Form an initial view of the extent of geotechnical input required by categorising the assessment approach as one of the following:

- structurally dominated (geotechnical parameters required for structural analysis)
- interactive (specialist geotechnical input required to evaluate the structural behaviour), or
- geotechnically dominated (significant specialist geotechnical input required to determine the behaviour of the ground and/or evaluate the structure behaviour).

Refer to Section C1.8 for further guidance on these categories.

Step 3 Site inspection/intrusive investigation and geotechnical considerations

Undertake visual site inspections to confirm the as-constructed structure is as documented in the available structural drawings. Note any deterioration in the condition of the structure that could potentially affect its seismic behaviour; e.g. settlement, cracking, corrosion or decay. Identify any site conditions that could potentially affect the building behaviour.

The lack of structural drawings is often a source of uncertainty and conservatism in a DSA. If specific detailing or material properties are deemed critical to the seismic performance of the building, and are not available from other sources, carry out intrusive investigation in sufficient detail to confirm these.

Note:

While a minimum level of knowledge of the building is required, the cost of undertaking extensive intrusive investigations often cannot be justified by the benefits that the additional knowledge of the building will provide. Refer to Section C1.7 for more guidance on building inspection and intrusive investigations.

Decide on the extent of geotechnical investigation and reporting required. Discussion between the geotechnical and structural engineer is likely to be necessary, especially if the interactive or geotechnically dominated categories in Step 2 apply. Refer to Section C4 for guidance on how to approach the geotechnical issues that might be present.

Step 4 Identify primary structural systems, potential structural weaknesses (SWs), severe structural weaknesses (SSWs), and relevant secondary structural and critical non-structural elements and review potential effect of geotechnical aspects

Identify the primary structural systems in the building (both lateral and gravity (vertical)) and identify potential SWs and SSWs. For example, a reinforced concrete wall system may have a number of potential mechanisms or SWs such as foundation uplift, in-plane shear mechanisms, flexural mechanism, etc. The identification of potential SWs requires a good understanding of the issues that might be present in buildings of the same generic type. SSWs are discussed further in Section C1.1.1.

Note:

Typically, this will essentially be a visual process that starts with the determination of likely load paths for both lateral and gravity loads. Note that, particularly for lateral load paths, there may be alternatives that were not considered or relied on by the original designer but are still viable.

Early recognition of SWs and SSWs and their relative criticality and interdependence is likely to reduce assessment costs and focus the assessment effort. Engineering judgement is considered essential when identifying the SWs in complex buildings.

The relative vulnerability of various members/elements and mechanisms needs to be established in terms of strength and in terms of deformation demands. Separate the various members/elements into those that are part of the primary lateral system (primary seismic resisting system), those that are part of the primary gravity system (primary gravity load resisting system), and those that would be considered secondary structural elements. Some components may be categorised as having both a primary seismic resisting function and a primary gravity load resisting function (e.g. moment resisting frames).

Identify any relevant secondary structural and critical non-structural elements that should be assessed in calculating the earthquake rating of the building. (Refer Part A for the rationale for choosing the building elements that need to be considered).

The potential impact of geotechnical issues on the various members/elements/structure as a whole should be reviewed at this stage.

Decide if further information is required to carry out the assessment and what this is. If necessary, return to Step 2.

Step 5 Assess structural component probable capacities for various mechanisms (e.g. flexural, shear)

Identify the potentially critical structural members/elements within the overall system.

Note:

Identification of the critical areas is likely to be an iterative process which is continually reviewed as the assessment proceeds.

For the potential critical element, calculate the probable strength and deformation capacities for the various members and failure mechanisms. For example, for reinforced concrete moment resisting frames it would be necessary to calculate the flexural and shear capacities for the beams and columns, joint shear capacity and anchorage/lap-splice capacity if applicable.

Consideration of the various probable capacities within the critical sub-assembly/load path will indicate the probable inelastic mechanism for the element under lateral load. Some simplifying assumptions (e.g. first mode response dominant, contraflexure points, etc.) will generally be required.

Calculate the probable seismic capacities (strength and deformation) from the most to least vulnerable members/elements in both the primary lateral and gravity systems in turn. There may be little point in expending effort on refining existing capacities only to find that the capacity is significantly influenced by a more vulnerable item that will require addressing to meet earthquake-prone requirements or the target earthquake rating.

Note:

For members/elements within the primary gravity structure (that is not also part of the primary lateral structure) this will be the capacity relevant to sustaining the lateral deformations imposed by the primary lateral structure.

An element may consist of a number of individual members. For example, the capacity of a penetrated wall (an element) loaded in plane will need to consider the likely behaviour of each of the piers and the spandrel regions (the members) between, above and below the openings respectively. For some elements the capacity will be a function of the capacity of individual members and the way in which these members act together. Therefore, establishing the capacity of an element may require structural analysis of that element to determine the manner in which actions in its members are distributed and how they interrelate.

For each member/element assess whether or not exceeding its capacity (which may be more easily conceptualised as failure for these purposes) would lead to a significant life safety hazard. If it is determined that it will not, that member/element can be neglected in the assessment of the expected seismic performance of the structure.

Note:

When assessing the capacity of an element/member in relation to a significant life safety hazard it may be necessary to consider several aspects at increasing levels of loading before the capacity of that element/member can be confirmed.

The requirements in NZS 1170.5:2004 and the materials standards for particular items (e.g. design ledge lengths in Section 8 of NZS 1170.5:2004) also need to be reflected in the capacity values determined.

Step 6 Decide assessment procedure, analysis methodology and boundary conditions

Decide on an appropriate analysis methodology (e.g. simplified hand calculation, elastic computer model, nonlinear hand calculation, nonlinear static procedure) and establish the boundary conditions (e.g. restraints at foundation level) with reference to Section C2.

Note:

The completion of the analysis is intended to support the seismic assessment and is not the “goal” in itself.

The engineer must have a good understanding of the building structure and its supporting ground and likely behaviour before undertaking any analysis. As part of the assessment, the engineer can then select the analysis methodology that most appropriately investigates the issues that matter. While simpler analysis may typically be adequate, it follows that

the degree of complexity of analysis should be determined by the problem. Overly complex methods have the potential to confuse and provide a false sense of precision: these should be used with care.

For example:

- For simple systems with easily identifiable inelastic mechanisms (e.g. a timber frame structure or URM building in a region of low seismicity) a simple elastic based calculation of the capacity to demand using force based assessment procedures may yield a conservative, but adequate, result. The term "adequate" here refers to the level of refinement and confidence in the results with respect to the objectives of the DSA itself.
- The presence of a non-ductile, gravity only load resisting system that may be susceptible to failure under significant lateral displacement will require specific consideration.
- When parallel systems of varying ductile capability are present, a displacement based seismic assessment is likely to be more appropriate to address the deformation compatibility issues.
- For complex structural systems a 3D dynamic analysis may be necessary to supplement the simplified nonlinear Simple Lateral Mechanism Analysis (SLaMA) described in Section C2.

The more sophisticated the structural modelling and analysis, the greater the need to verify the model and validate the input data and modelling assumptions. It is highly recommended that advanced structural analysis is cross-checked with simpler first principles based analysis as part of the verification process.

In all situations a SLaMA should be completed as part of the assessment strategy. This is part of the general requirements for the assessment process described in Section C2.

Step 7 Carry out the structural analyses

Structural analyses will be required to establish the relationship between the member/ element actions and the global capacity of the building.

This will generally be done by applying a lateral load distribution to the structural model. However, it should be recognised that the aim is to find the maximum ground motion intensity (or lateral load) that can be sustained by the building and not the actions that result from a certain level of lateral load.

Start with simple analyses, progressing to greater sophistication only as necessary. In general, the complexity and extent of the analysis should reflect the complexity of the building.

The analysis will need to recognise that the capacity of members/elements will not be limited to consideration of elastic behaviour. Elastic linear analysis is likely to be the easiest to carry out, but the engineer must recognise that restricting to elastic behaviour will likely lead to a conservatively low assessment score and/or potentially erroneous distributions of lateral loads between different structural systems.

The analysis will need to consider the likely impacts of plan eccentricities (mass, stiffness and/or strength) and the ability of the diaphragms to resist and distribute the resulting actions.

Cross check any inelastic displacement response results using simple elastic based analyses based on equal energy or equal displacement concepts as appropriate.

Review the validity of any assumed boundary conditions.

Consider the potential impacts of the results of variability in input parameters; particularly with respect to assumptions relating to the supporting soils.

Critically review the analysis against the actual building to confirm that all key systems and/or mechanisms have been identified and also to confirm whether the capacities that have been determined appear reasonable. If not, review the assessment to date obtaining further information as required.

Note:

It is recommended that any elastic analysis (e.g. modal response spectrum analysis) is completed using the full unscaled elastic derived response spectrum demand. The loading can then be scaled back as required to match the available strength and the nonlinear behaviour can be considered separately.

Reducing the loading (demand) by a globally applied ductility factor, as is done for force based design, is not generally recommended for assessment. This is because the criteria that often make this valid for design (e.g. ductility well spread in the structural system and reliably achieved through a strength hierarchy) are not typically present in older buildings.

Step 8 Assess global capacity based on the governing mechanism and capacity of any SSWs

From the structural analyses determine the global seismic capacity of the building. This will be the lateral strength and deformation capacity of the building as a whole determined at the point at which the most critical member/element of the primary structure (lateral and/or gravity) reaches its determined probable capacity, or the building as a whole reaches its defined drift/deflection limits.

Note:

Drift limits specified in NZS 1170.5:2004 are intended to provide assurance that the nonlinear demands in new buildings are limited to levels that can be relied on.

For relatively lightweight existing buildings (e.g. light timber buildings and light steel industrial buildings) the drift limit of 2.5% may be too severe a constraint when there is confidence that additional drift can be accommodated without compromising either the lateral or vertical load carrying capacity of the building. To recognise this, the limits prescribed in NZS 1170.5:2004 for this type of building may be exceeded in an assessment provided that it can be shown that the capacity of the building is not compromised.

Notwithstanding that the life safety hazard of the various members/elements was reviewed in Step 5, reflect again on whether or not the identified global capacity is likely to represent a significant life safety hazard. If it is determined that it does not, refine the residual seismic capacity (or adjust to zero where appropriate) of the lowest scoring member/element and repeat to find the new global capacity. Identify the impact of any identified SSWs.

Continue the assessment process until the appropriate capacity is identified. This will provide the probable global capacity score for the building.

It may also be useful to determine the global capacity assuming that successive SWs and/or SSWs are addressed (retrofitted). This will provide information on the extent of retrofit that would be required to achieve a target rating for the building as a whole.

Review if the assumed mechanisms require amendment: if so, reassess.

Review the impact of any global geotechnical issues.

Step 9 Assess the probable capacity of relevant secondary structural and non-structural (SSNS) elements

Determine the probable capacity of any SSNS building elements that would constitute a significant life safety hazard if they were to fail. Refer to Part A for further guidance on the types of element that are to be considered. Refer to Section C10 for guidance on how to determine their capacity. Section C1.5 shows the impact of SSNS elements on the building's earthquake rating.

Step 10 Determine the ULS seismic demands

Determine the ULS seismic demands from Section C3.

The demand values are obtained from NZS 1170.5:2004 as this is fundamental to the calculation of *%NBS*, but their form (e.g. response spectra, peak ground acceleration, member actions, displacements) will depend on the aspect for which the *%NBS* is being calculated.

For the calculation of *%NBS* the reference level demand might be expressed in terms of the ULS base shear, design response spectrum (acceleration and displacement), or scaled strong motions obtained from Sections 5 and 6 of NZS 1170.5:2004.

At the member/element level the demand might be the actions (stress and strain) obtained from a structural analysis using the global demands as inputs.

For some aspects of primary structure (e.g. floor restrained face-loaded walls) and for secondary structural items, the design actions for parts obtained from Section 8 of NZS 1170.5:2004 will be more appropriate. This will include, where appropriate, imposed deformations from the primary lateral structure.

Consideration of geotechnical effects may require the reference demands to be in the form of design ground shaking (e.g. peak ground accelerations and/or displacements at the appropriate return period).

Note:

It is intended that the version of NZS 1170.5 that is cited in the Building Code at the time of the assessment is used. Engineering assessments for the purposes of evidence of earthquake proneness are required by the EPB methodology to be referenced to the version of NZS 1170.5:2004 cited in the Building Code as at 1 July 2017. Refer Part A for further discussion on this point.

Step 11 Calculate *%NBS*

Refer to Section C1.5 for a detailed discussion of the calculation of *%NBS*.

The *%NBS* earthquake rating for the building is the minimum of the earthquake scores assessed for the global performance of the primary structure for each principal direction and the earthquake score for each individual secondary and non-structural element that also meets the significant life safety criterion (refer to Part A).

Before considering the assessment as being complete, reflect on the earthquake rating that has been determined and whether or not it appears reasonable. If not, investigate whether the identified critical members/elements have been adequately assessed or whether more reliable data should be obtained.

Step 12 Reporting

This is perhaps the most critical part of the assessment process as the technical output needs to be communicated to the client, stakeholders and building users in a way that puts the results in context.

The report of findings should cover a clear description of the structural system and key structural vulnerabilities, the *%NBS* and the expected structural behaviour in earthquakes, the assessed seismic risk, and the assessment methodology and analysis undertaken.

An important aspect of the assessment process is reporting all individual *%NBS* scores that have been evaluated and the rationale for decisions on life safety potential and modes of failure. This will provide a complete picture of the issues associated with the building's seismic performance, will aid in the development of a retrofit program if this is to be considered, and will also assist the relevant Territorial Authority's consideration of earthquake-prone building status.

Refer to Section C1.10 for further guidance and Appendix C1A for a recommended report template.

There are specific requirements set out in the EPB methodology for reporting on results of engineering assessments for earthquake prone status. Refer to Part A.

C1.5 Calculation of *%NBS* for a DSA

C1.5.1 General

Note:

The earthquake rating for the building, expressed as *%NBS*, is discussed in Part A. The material is largely repeated here for convenience.

%NBS earthquake rating from a DSA is obtained by dividing the calculated ultimate capacity (seismic) of the building or part by the ULS seismic demand as shown in the following equation:

$$\%NBS = \frac{\text{Ultimate capacity (seismic)}}{\text{ULS seismic demand}} \times 100 \quad \ldots C1.1$$

where:

Ultimate capacity (seismic) is taken as:

- probable capacity of the primary lateral structure of the building including the impact of geotechnical issues (refer to Section C1.5.2), or
- probable capacity of structural elements, the failure of which could lead to a significant life safety hazard (refer to Section C1.5.2), or
- capacity assessed for any identified SSWs (refer to Section C1.5.3), or
- probable capacity of SSNS elements which would pose a significant life safety hazard (refer to Section C1.5.4)

ULS seismic demand as described in Section C3, including the appropriate value of S_p (the structural performance factor) for the particular aspect under consideration. Refer to Section C1.5.6 for further discussion.

The earthquake rating will be the minimum value of *%NBS* calculated as above.

The earthquake rating should always be quoted together with the Importance Level that was assumed to determine the ULS seismic demand. The recommended presentation format, showing the percentage as XXX, the Importance Level as Y, and with "*%NBS"* always italicised, is:

XXX*%NBS* (ILY)

The Importance Level assumed when setting the demand, and therefore the basis for the earthquake rating, is critical to establishing the standard to which the building has been assessed.

Rather than just evaluating the ultimate seismic capacity for the building as indicated above and then dividing by the global demand on the building, it may be more appropriate to evaluate the *%NBS* score for each aspect of the building and then take the lowest value. This is because the demand may need to be represented in different ways depending on the particular aspect under consideration. This is discussed further in Section C1.5.6.

It is also considered important that the *%NBS* scores for the individual aspects are included in the DSA report, as this helps to put the earthquake rating in context and assists when retrofit to a particular target level is being considered.

C1.5.2 Probable capacity of primary structure

C1.5.2.1 Assessing the probable structural capacity

The probable structural capacity of the primary structure is assessed using probable material strengths and nominal member/element capacities calculated using these. Guidance is provided in Sections C5 to C9 and the methods of analysis and assumptions set out in Section C2. The term "primary structure" is intended to describe both the seismic lateral structural and the gravity (vertical) structural support systems in a building. Identification will be particularly important when these systems are separate.

Note:

It is the intent that the probable capacity is typically the capacity determined from application of the Building Code design Verification Method, B1/VM1, substituting probable material properties for characteristic design values and using strength/capacity reduction factors equal to 1.0. When further concessions from the design values, modified as indicated above, are possible these are outlined in Sections C4 to C9.

It is also intended that member/element stiffnesses defined in B1/VM1 are taken for assessment.

The response of the ground under the building and its effect on the building needs to be factored in, as discussed in Section C4. This will include the effect on the dynamic characteristics of the building (SSI considerations and the subsoil amplification) and also structure support conditions (e.g. degree of restraint to foundations).

As discussed throughout these guidelines, the structural capacity is described both in terms of the strength capacity and the deformation capacity. Inclusion of the nonlinear deformations in the assessment can significantly influence the calculated earthquake rating for the building but it is recognised that not all of the assessment procedures (e.g. force based approaches) will fully account for this.

The primary structure includes such items as URM parapets, face-loaded walls, building penthouses, and stiff panels that are required to participate because of inadequate separation from the intended primary lateral structure. These will typically be considered both as building elements not part of the primary lateral structure and as part of the building lateral load resisting system for assessment of seismic loading (the demand).

Note:

Items such as parapets have been considered as secondary structure or even non-structural elements in the past. However, in the context of these guidelines, and to avoid confusion, it is considered that they are part of the primary structure as they are required to provide both gravity and lateral load support to their own, often not inconsiderable, weight/mass, and they often contribute to the in-plane capacity of URM walls. Whether or not they are considered as primary or secondary structure (they are not non-structural as defined in these guidelines), the assessment of their capacity and effect on the *%NBS* rating does not change.

Once analysis of the complete primary structure is completed and the behaviour of the various members/elements in the primary structure has been assessed, the global probable

seismic capacity of the primary structure will be limited by the member/element with the lowest probable capacity; **provided the failure of that element would result in a significant life safety hazard**. The capacity of elements that do not fulfil this requirement should be set to a residual value (or zero if appropriate) and the analysis repeated until the lowest scoring member/element is identified that would result in a significant life safety hazard.

C1.5.2.2 What constitutes a significant life safety hazard?

What is considered to constitute a significant life safety hazard for primary structure is discussed in Part A. In summary, it is a hazard resulting from the failure of a member/element of the primary structure or of the supporting ground that would lead directly to collapse of the building as a whole or to a part of it and that could reasonably affect a number of people.

What does and what does not constitute such a hazard will rely to some extent on the judgement of the engineer. Therefore, it is important that the decision making process for this is recorded in the DSA report.

C1.5.2.3 Other considerations

The analysis of the primary structure often focuses only on the primary lateral structure under horizontal earthquake shaking.

The assessment of the primary structure must also consider all of the systems that support gravity loads. Often, these will not be relied on to resist lateral loads. However, experience in numerous earthquakes has highlighted that it is the performance of assumed non-lateral load resisting gravity load supporting systems that is often critical when determining the collapse potential of the building, and the extent to which collapse might occur (e.g. pancaking of floors for SSWs).

The assessment of assumed non-lateral load resisting systems (e.g. frames assigned to be gravity only) requires consideration of the effects of imposed deformations from the primary lateral system. It is intended that the stiffness and strength of members/elements in such systems be based on probable values. However, care must be taken to ensure that actions within these systems are cautiously appraised. This will require the stiffness of joints, for example, to be reasonably modelled and an understanding of the sensitivity of any assumptions on the conclusions reached. The support stiffness at foundation level may also be important in this regard.

Vertical earthquake motions may also be important in the same situations as identified in design standards for new buildings; e.g. for horizontal cantilevers, etc. If present, vertical demands should also be included in the assessment.

Note:

Although the assessment of SWs in these guidelines is focused on the demands at XXX%ULS levels of shaking, the expectation is that a building will be able to continue to perform to a satisfactory level in shaking much higher than this to meet the overall performance objectives. The provisions of these guidelines (acceptance criteria, treatment of SSWs and the like) have been set at a level to provide confidence that this will be achieved. Therefore, it is not the intention that higher levels of shaking are specifically addressed in the assessment process.

However, some buildings are complex and will not always be fully covered by the guideline provisions. The engineer should always be mindful that the intensity of earthquake shaking is not limited, even when MCE levels are defined, and consider the implications of this when scoring aspects that are not fully covered within the scope of these guidelines.

Refer also to Section C1.5.3 for further discussion relating to SSWs.

C1.5.3 Capacity of SSWs

C1.5.3.1 General

While the assessment process outlined in these guidelines is primarily focused on determining the probable seismic capacity and relating this to the XXX%ULS loading demands, as discussed in Part A and Section C1.5.1, the intention is also that this process needs to deliver a reasonable expectation of satisfactory performance at higher levels of shaking to meet the overall performance objective. This is referred to in Part A as the structural resilience available and is a necessary aspect of a building's behaviour if it is to deliver the overall expected seismic performance.

In many cases there will be inherent structural resilience (i.e. sufficient margin will have been provided when setting the acceptance criteria, etc. given in Sections C4 to C10) and it is not necessary to specifically account for this other than by adhering to the assessment procedures outlined in Sections C4 to C10.

However, it should be recognised that there are potentially some aspects of a building's seismic behaviour which may not be adequately captured within general assessment procedures but are likely to have a step change response resulting in sudden (brittle) and/or progressive, but complete (i.e. pancaking), collapse of the building's gravity load support system in shaking greater than that represented by XXX%ULS shaking.

These guidelines define complete collapse of this type as catastrophic collapse. Collapse of this type has the potential to result in high fatality rates for occupants and little or no chance of escape after the earthquake. Experience indicates that while the public may accept some fatalities as being a consequence of living in an earthquake hazardous region they are intolerant of such collapses, particularly for more modern buildings. Therefore, it is essential that the overall assessment procedure satisfactorily accounts for such behaviour when it is identified.

Experience from past earthquakes also indicates that such behaviour is typically restricted to a relatively small number of mechanisms. In these guidelines these mechanisms are referred to as SSWs (refer to Part A for the criteria used to determine these).

For the purposes of these guidelines the aspects that need to be assessed as SSWs in a DSA have been predetermined and may be assumed to be restricted to the particular SSWs listed below:

- Lightly reinforced concrete columns and/or beam-column joints (refer to Section C5 for definition) with axial loads greater than 0.5 $A_g f'_c$ which are part of the primary structural system of buildings. To be a SSW the failure of a column and/or beam/column joint would need to lead to a progressive collapse scenario for the entire storey.

- Interconnected concrete shear walls (shear core) with axial loads greater than 0.15 $A_g f'_c$ which are shear dominated and which are required to carry, in total, more than 60% of the seismic lateral demand.
- Flat slab configurations in cast insitu concrete buildings with more than two storeys where the lateral capacity is reliant on low ductility slab to column connections and the gravity support is susceptible to imposed lateral drift and punching shear failure. This is only intended to apply to systems with gravity-only shear demand exceeding 40% of the probable shear capacity ($v_c + v_s$) at the critical shear interface.
- Lack of positive connection between diaphragm(s) and primary lateral structure comprising a single element (e.g. shear core) of buildings.
- Complex slope failure resulting in significant ground mass movement and the loss of support over more than 50% of the building platform (i.e. where the building is on a slope or cliff edge).
- Liquefiable ground supporting poorly tied together URM buildings (refer to Section C8 for definition) with more than two storeys.

Note:

This list of SSWs is similar to the list of SSWs that must be highlighted in an ISA (if they have been identified). However, it excludes those building aspects which should be adequately addressed in the DSA using the quantitative assessment procedures set out elsewhere in these guidelines; e.g. ledge and gap stairs and weak or soft storeys.

C1.5.3.2 Determining the capacity of SSWs

Additional factors (typically 0.5) are applied to reduce the available probable capacity of SSWs to reflect their potential catastrophic nature. How this factor is intended to be applied is shown in Section C2 (Appendix C2G).

Note:

It is important to recognise that the above procedures for dealing with SSWs are not replaced by subjecting the model of the building to the traditional concept of a Maximum Credible/Considered Earthquake (MCE). The SSWs are not mechanisms where it is considered that the step change behaviour can currently be adequately assessed from calculation/analysis; nor should there be any expectation that the shaking the building could be subjected to is capped at a specific level.

The use of the additional factor of 0.5 on the probable capacity for these critical mechanisms (which can also be considered as a factor of 2 on the ULS demand, although reduction in capacity is preferred) is considered sufficient to provide a reasonable expectation of satisfactory performance overall.

It may be assumed that, for SWs covered by these guidelines and not listed as SSWs, adherence to the recommendations of Sections C4 to C10 will provide the required level of resilience without the need to apply any additional factors/margins.

C1.5.4 Probable capacity of secondary structure and non-structural (SSNS) elements

For SSNS elements to influence the building rating they must be of sufficient size and located such that their failure would lead to a significant life safety hazard. This will typically relate to their ability to continue to support gravity loads (including their own weight). Refer to Part A for a description of elements that should be considered when determining the earthquake rating.

Guidance on assessing the probable capacity of SSNS elements is provided in Section C10. SSNS elements spanning between levels in a building or between adjacent buildings and supported on ledges is considered particularly vulnerable.

The probable capacity of SSNS elements should be assessed in the same manner as for members/elements in the primary structure as outlined in Section C1.5.2.

Note:

Restraints (tethers) can be used to mitigate the life safety hazard of SSNS elements if the restraint is sufficient to arrest their fall above a height that would constitute a significant life safety hazard. Refer to Section C10 for further discussion.

C1.5.5 Critical structural weakness (CSW)

The CSW is the issue that finally limits the *%NBS* earthquake rating for the building after consideration of the capacity of the primary structure (seismic and gravity) and the SSNS elements. The process should deliver a CSW, the failure of which would result in a significant life safety hazard.

It is recommended that a final check is made that this is the case before accepting this as the CSW.

C1.5.6 ULS seismic demand

Guidance for assessing the ULS demand is provided in Section C3.

The appropriate value of S_p needs to be used when assessing the demand if it is not otherwise accounted for in the assessment of the capacities. This may require different values for S_p depending on the level of nonlinear deformation possible from the aspect under consideration, in accordance with NZS 1170.5:2004 and Section C3. S_p for the primary structural system is set based on the displacement ductility that is assessed to be available from the global capacity. This may be limited by any primary system element/member's ductile capability. Refer to Section C2.

C1.6 Use of Alternative Verification Methodologies

C1.6.1 General

The adoption of internationally used assessment methodologies (e.g. ASCE 41 and Eurocode-8 Part 3) is not precluded by these guidelines. However, engineers must be confident that compliance with these methodologies will lead to the expected level of performance that is outlined in Part A and, where not otherwise noted, use of international methodologies should be considered to be outside the scope of the guidelines.

Irrespective of the method of assessment, it is essential that the result is delivered in terms of a *%NBS* earthquake rating to avoid market and community confusion.

There will be instances where the component performance (deformation) acceptance criteria prescribed in ASCE 41) will provide valuable information where, otherwise, none exists.

Use of linear (elastic) verification methods other than those set out in Section C2 is not recommended nor supported by these guidelines. However, it is recognised that alternative nonlinear methods may provide additional insight into building performance and, therefore, assessment using nonlinear time history analyses (NLTHAs) and the Tier 3 approach in ASCE 41 is considered an acceptable alternative provided that some minimum requirements as outlined in Section C1.6.2 are satisfied.

Note:

Sufficient verification to "test" the ASCE 41 linear elastic or the Eurocode 8 Part 3 methods against the objectives of these guidelines has not been completed to provide the necessary confidence that these objectives will be met.

C1.6.2 NLTHA using ASCE 41-13 (2014) Tier 3 assessment

The ASCE 41 Non-linear Dynamic Procedure (NDP) Tier 3 assessment prescribes what is referred to as a performance based method of assessment. Using this approach the engineer decides, in consultation with the client, on the level of performance required. This defines a hazard level and acceptance criteria that must be met at that level of hazard. A range of structural and non-structural performance levels corresponding to various performance objectives can be selected.

Note:

ASCE 41 NDP Tier 3 does not represent a full performance based method of assessment as it does not predict the *actual likelihood* of achieving a given level of performance. It is still a deterministic assessment based on pass/fail acceptance criteria completed on an element by element basis. In that respect it is still checking the degree of compliance against defined minimum criteria for the particular performance level being assessed.

The general approach outlined in the ASCE 41 NDP Tier 3 systematic assessment is considered to be an acceptable alternative to the DSA as outlined in these guidelines and may be assumed to comply with these guidelines provided that the following recommendations are followed:

- The assessment outcome should be reported in terms of a *%NBS* earthquake rating. This may require a modification to the ASCE 41 process as outlined below.
- The target building performance objective for the reference 100*%NBS* should be the Basic Performance Objective Equivalent to New Building Standards (BPON) for the appropriate return periods defined in Table C1.1.

Note:

Only target performance levels associated with Life Safety (LS) and Collapse Prevention (CP) need to be considered to meet the recommendations of these guidelines. However, in the case of Importance Level 4 buildings it may also be appropriate to consider the calculated performance at a level consistent with the functionality limit state SLS2 in NZS 1170.5:2004. Refer to Part A.

- The acceptance criteria and recommendations given in ACSE 41, unless specifically modified for use with NLTHA in these guidelines, should be used in their entirety this includes the process of selection and scaling of strong motion records.
- The *%NBS* for each target performance level is the scale factor that needs to be applied to the input strong motion record so that the acceptance criteria for that performance level are just met. Duration of shaking is not adjusted.
- The *%NBS* global earthquake score for the building is the lowest of the global *%NBS* values determined for both performance levels.
- The behaviour of SWs not otherwise included in the analysis model (e.g. support of precast floor units and frame elongation effects) must be considered separately as required elsewhere in these guidelines and the *%NBS* earthquake rating adjusted accordingly.
- The behaviour of SSNS elements (as defined by these guidelines), if not otherwise included in the analysis model, must be considered separately as required elsewhere in these guidelines and the *%NBS* earthquake rating adjusted accordingly.
- If SSWs listed in Section C1.1.1 are identified as being present the *%NBS* rating should be adjusted down to achieve the required margin of two on the calculated score for the SSW at the life safety performance level. The earthquake rating may also require adjustment if the SSW is not specifically addressed in the analysis; e.g. the impact of slope failure or liquefaction.

Note:

The intention is that the score for SSWs is determined as follows:

- Identify the members/elements that participate in the mechanism that leads to the SSW.
- Determine the earthquake score for these members/elements at the Life Safety (LS) performance level.
- Halve the Life Safety score to obtain the *%NBS* score for the SSW.

Reliance is not placed on the results from the Collapse Prevention level for SSWs because the method is not considered to provide the necessary margins against catastrophic collapse; nor does it recognise the uncertainties involved in predicting such behaviour.

Table C1.1: Reference return periods and ground motion scaling factors for 100*%NBS* for BPON[1]

Importance level (IL)	Building performance level	
	Life safety (return period)	Collapse prevention (scale factor)*
IL1	100	1.8
IL2	500	1.8
IL3	1000	1.8
IL4	2500	1.5

Note:

* These scale factors are intended to be applied to the ground motion that is used for the life safety analyses. The factor has been set for IL2 to give a return period of 2500 years for the collapse prevention analyses. This factor is representative of the range of shaking that has been found to be important when considering the prevention of collapse for all ILs and therefore the same value has been used for all ILs. A margin against actual collapse at the collapse prevention level is still expected to meet overall performance objectives.

The use of a 1.5 factor for IL4 buildings is a recognition that the principle objective of assessment under these guidelines is life safety of occupants. It recognises that a higher level of reliability of performance of these facilities is required but not to the extent that a 1.8 factor would imply when compared with an IL2 building. This is considered to be a pragmatic decision pending detailed evaluation of these issues.

Note:

NLTHA carried out in accordance with the general requirements of ASCE 41NDP Tier 3 methodology provides a way of applying the principles of these guidelines provided the additional considerations as outlined are made. The typical approach is that the structural components are modelled (elastically and inelastically) in accordance with rules specified. The demand is established for the required performance level, strong motion records are chosen to match the required hazard level (demand), the model of the building is "shaken", and the acceptance criteria for the performance level are checked. If the analysis shows the criteria are met it is assumed that the performance level has been met.

Further to the ASCE 41methodology, if the criteria are not met the input motions can be dialled back (by scaling), and the analyses rerun until the criteria are met. The degree to which the building meets the 100%ULS shaking demands is then effectively the scale factor applied to achieve compliance with the acceptance criteria.

The performance levels typically assessed using ASCE 41are "life safety" and "collapse prevention". Thus, a check is made at two levels of demand (500 and 2500 year return period shaking for IL2 buildings) rather than the single level check made at the ULS level of shaking using the standard guideline approach. This approach provides an assessment of the structural resilience of the structural system beyond ULS shaking levels (500 year return period demands for IL2 buildings) and is often presented as a major benefit of this methodology over the guideline methods.

The engineer needs to be satisfied that checking for collapse (collapse prevention) in accordance with ASCE 41still achieves the level of performance required at a severe level of shaking, beyond those defined for the collapse prevention level. These are the checks required for SSWs as outlined in the specific requirements above.

In addition to consideration of the two performance levels outlined above, the following adjustments are considered necessary to provide consistency with the *%NBS* earthquake rating used in these guidelines:

When running the analyses, care should be taken to ensure that the element that is limiting the level of shaking that can be sustained does represent a significant life safety hazard. If it is determined that the critical element does not then the level of shaking should be increased until the critical member/element is considered to represent a significant life safety hazard. URM spandrels supported on lintels, for example, may be damaged to the extent that they can no longer participate in the lateral load resistance of the building but may have a low risk of collapse until the actions are fully redistributed in the building and the deformations become high enough that collapse is expected. This is because the structure reconfigures the way in which it is resisting the shaking until the spandrels sustain a level of deformation beyond which they can no longer be assumed to remain reliably in place. It is only at this point that a significant life safety hazard develops. Ignoring this ability to redistribute could result in a *%NBS* rating significantly less than intended by the methods set out elsewhere in these guidelines.

NZS 1170.5:2004 specifically requires shear deformations to be included in the modelling for time history analyses but also has provisions for new structures that are intended to reliably prevent the shear capacity of the primary structure from being exceeded. These same prevention methods are not always available in existing buildings, especially when traditional capacity design methods were not used in their original design. Accordingly, for shear dominated systems the modelling of shear deformations when assessing an existing building needs to be approached with care, especially when these effects are beneficial (i.e. limit the shear actions). These concerns are not present when the system is flexurally dominated. Therefore, when any shear critical primary seismic structural elements are present in significant buildings (i.e. with more than six storeys) it is recommended that nonlinear shear behaviour is not relied on to limit the actions in the building for the purposes of the assessment, unless a deformation margin (to that required to reach the maximum predicted shear (strength) capacity of these elements) of at least two can be shown to be present. This behaviour can be treated similarly to an SSW. An alternative may be to model the shear as linear elastic in such situations or treat it as force controlled in applying ASCE 41.

The required margin of two at the life safety performance level needs to be achieved for any of the SSWs listed.

NLTHAs should never be approached on the basis that they provide a “black box” assessment procedure. These are highly sophisticated analyses requiring particular skills and experience. While the results of such analyses may not be replicated by more simple methods it is not considered appropriate to complete such analyses in the absence of at least some confirmatory analyses at a more basic level. Simply relying on the NLTHA to deliver a *%NBS* without careful consideration of the results and the behaviour they imply is considered inappropriate. Peer review, including the modelling, the analysis and the calculation of *%NBS*, is considered essential when such methods are used. Refer also to Section C2.

C1.6.3 Eurocode-8 Part III

It is recognised that EN 1998-3: Eurocode 8 – Design of structures for earthquake resistance – Part 3: Assessment and retrofitting of buildings is available and will be familiar to some engineers in New Zealand. Use of this document is not covered by these guidelines. If this Eurocode is to be used as a basis for assessment in New Zealand it is considered essential that the assessment results are expressed in terms of a *%NBS* earthquake rating and the entire process is subjected to appropriate holistic peer review that includes specific consideration of how the assessment meets the objectives outlined in Part A. The engineer will also need to consider the issues/adjustments discussed for NLTHA using the ASCE 41-13 (2014) Tier 3 approach discussed in Section C1.6.2.

C1.7 Building Inspection and Investigations

Detailed building inspections should be made as part of the assessment of an existing building's earthquake rating and before the preparation of any strengthening proposals.

The following sections list the main items to be inspected during the detailed inspection and what information to record.

The site will also need to be inspected for potential geohazards, refer to Section C4.

C1.7.1 Structural configuration

Most of the details of the structural configuration required for an analysis should be available on design or construction drawings. As-built checks should also be made. Where detailed plans are unavailable, field measurements and invasive exploration and testing may be necessary.

It is recommended that an initial field measure is carried out to confirm the general adequacy of the available documentation. This can be followed by a more detailed inspection to confirm detailed dimensions and detailing as required for the assessment.

Note:

Experience indicates that it is unlikely that tender, consent, or even construction drawings will always fully represent the as-built condition of a building. Therefore, a site inspection to confirm as-drawn details is recommended in all cases.

The structural configuration information gathered should include the following:

- plans, elevations and dimensions of frames and walls on each level
- location and size of openings in walls and floors
- identification of load-bearing/non load-bearing walls
- identification of any discontinuities in the structural system
- arrangement of roof and floor trusses, beams and lintels
- identification and location of reinforcing bands, columns and bracing
- dimensions of non-structural components to allow storey masses to be reliably assessed
- location and configuration of precast elements
- lift and stairwell construction and dimensions
- foundation dimensions, type and identification of connections between foundations and between superstructure and foundations
- clearances to adjacent buildings
- evidence of structural modifications, and
- seismic status of adjacent buildings, if this can be obtained.

Note:

Identifying the structural configuration will enable both the intended load-resisting elements and the effective load-resisting elements to be recognised. Effective load-resisting elements may include both structural and secondary structural elements that

participate in resisting lateral loads, whether or not they were intended to do so by the original designers. Potential discrepancies in intended and effective load-resisting elements may include discontinuities in the load path, weak links, irregular layouts, and inadequate strength and deformation capacities.

While the seismic status of adjacent buildings will not affect the earthquake rating it is nevertheless an important consideration when providing holistic advice on a building.

C1.7.2 Member/element properties

The following member/element properties should be obtained:

- cross-sectional shape and physical dimensions of the key members/elements and overall configuration of the structure
- configuration of connections, size and thickness of connected materials, and continuity of load path, particularly for precast elements
- modifications to individual members/elements or overall configuration of the structure
- location and dimensions of braced frames and shear walls
- current physical condition of members/elements and extent of any deterioration present
- reinforcing details in reinforced concrete structures, and
- connection details for primary and secondary structure.

Behaviour of the components – including shear walls, beams, diaphragms, columns, and braces – is dictated by physical properties such as area; material; thickness, depth, and slenderness ratios; lateral torsional buckling resistance; and connection details. The actual physical dimensions should be measured; e.g. 50 x 100 mm timber dimensions are generally slightly less due to choice of cutting dimensions and later shrinkage. Modifications to members need to be noted including notching, alterations, tack welds, etc. that may modify geometric and material properties. The presence of corrosion, decay or deformation should be noted.

These key element/member properties are needed to properly characterise capacities in the seismic assessment. The starting point for establishing member/element properties should be the available construction documents. Preliminary review of these documents shall be performed to identify key gravity and lateral load carrying members and elements and key connections. Site inspections should be conducted to verify conditions and to ensure that remodelling has not changed the original design concept. In the absence of a complete set of building drawings, the engineer must thoroughly inspect the building to identify these members and elements. Where reliable record drawings do not exist, an as-built set of plans for the building could be created (even as sketches) as part of the assessment.

The adequacy of interconnection between the various members and elements of the structural system will be critical to its behaviour. The type and character of the connections should be determined by a review of the plans and a field verification of the conditions. The connection between a diaphragm and the supporting structure is likely to be of prime importance in determining whether or not the separate parts of the structure can act together or if gravity-only members and elements are likely to be sufficiently protected by the lateral load resisting system (e.g. concrete shear walls).

If drawings of reinforced concrete buildings are not available it may be necessary to carry out on-site investigations to obtain details of size and spacing of reinforcing bars or to determine whether plain or deformed reinforcing bars were used. Investigations could include the removal of concrete cover in chosen locations to expose the reinforcing but may be possible using non-destructive scanning techniques (refer to Section C5). Default values given in Sections C5 to C9, based on age of construction, may also be used. Refer to Section C1.7.3 below.

Note:

In the case of steel structures some useful information is contained in Section C6 on the mechanical properties of the steel members and components (fixings such as bolts and rivets) used in these.

C1.7.3 Material properties and testing

Realistic (expected/mean) values for the material properties must be used to obtain the estimates of the probable strengths and deformation capabilities of members, joints and connections. Refer also to the discussion in Part A on the use of probable material strengths in member capacity assessment.

Recognising the significant cost associated with an extensive material testing programme and the difficulty in sampling all materials, these guidelines recommend a more pragmatic approach to setting the probable material strength for seismic assessment based on the use of default values (where not otherwise recorded in the construction documentation) and consideration of the likely sensitivity of the choice of material properties on the final assessment result. The testing effort is then concentrated in the areas of the structure that are likely to yield the greatest benefit in terms of defining the earthquake rating.

In other jurisdictions, a penalty is sometimes applied to the assessment results (e.g. a *lack of knowledge* factor). However, this approach has not been adopted in these guidelines for the reasons outlined above. It is recognised that material variability will always exist and will be difficult to quantify even when extensive testing has been carried out.

The material sections (Sections C5 to C9) provide specific guidance on the default probable material strengths to be used in absence of any documentation of the original prescribed materials or test results. These sections also provide recommendations on the type and frequency of testing if this is considered.

C1.7.4 Condition, maintenance and alterations

The condition of all structural components should be recorded with particular attention given to deterioration such as cracking, spalling, corrosion and decay. Locations and extent of any significant deterioration should be recorded. Any lack of watertightness in the roof and wall openings should be noted.

The foundation soil type should be determined and a careful inspection made to identify any settlement or indications of foundation distress.

Damage from previous earthquakes or other overloads should be carefully inspected and recorded.

The impact of any building alterations on the behaviour of the main structural elements should be considered carefully.

C1.7.5 Previous seismic retrofit

If the building has already been seismically retrofitted it will be necessary to ascertain the extent of this retrofit and the philosophy adopted. For example, in the past, buildings may have only been secured rather than fully strengthened or the strengthening scheme would now be considered incomplete or inadequate.

A design features report, if available, could provide valuable insight into what the design retrofit engineer was intending when completing only previous retrofit works. However, relying on the stated intent of any previous retrofit is not recommended when establishing its capacity as a number of issues have been identified with previous retrofit works that could reduce their effectiveness. If possible, discussions with the engineers who carried out the retrofit should be considered.

Common issues with previous retrofits include:

- Incomplete documentation – details were often varied to suit conditions found on site when the structure was opened up, so what is shown on the retrofit drawings may not always be representative of what was finally installed. Site or contract instructions, if available, can be a valuable source of information in this regard.
- Incomplete retrofit – only the most vulnerable issues may have been addressed. This would have typically been the case where interim securing had been adopted.
- Poor deformation compatibility between the original structure and the retrofit works; e.g. steel bracing used to strengthen in-plane URM walls or infilled concrete/steel frames.

C1.7.6 Intrusive inspections

Intrusive inspections while the building is occupied will typically be costly and disruptive but may not be able to be avoided. Non-destructive techniques such as electronic scanning of concrete members can often provide an attractive alternative to intrusive investigations.

C1.8 Geotechnical Investigations

The extent of geotechnical investigation required will depend on the likely influence on the earthquake rating and/or the target rating required. The categorisation outlined in Section C1.9 may assist. Further guidance is provided in Section C4.

C1.9 Geotechnical Influence on Detailed Seismic Assessments

C1.9.1 General

These guidelines categorise DSAs in terms of the influence that geotechnical issues might have as:

- structurally dominated
- interactive, or
- geotechnically dominated.

Which category a particular assessment falls into should be considered in Step 2 of the DSA process outlined above and reviewed as the assessment proceeds. The category has an impact on the way in which an assessment might proceed, including the respective roles of the geotechnical and structural engineers and the level of geotechnical reporting and investigation required.

A description of each project category and some common examples of situations where each might be considered to apply are given below. This is followed by a matrix of these examples classified by foundation type.

C1.9.2 Structurally dominated

These are assessments where the structural response is unlikely to be significantly influenced by geohazards, foundation soil nonlinearity or SSI up to the capacity of the structure. Such assessments are only likely to require geotechnical input into soil parameters for analysis purposes or in assessing the appropriate site soil class.

Examples include:

- a building on shallow foundations on competent gravels or rock
- a piled building where the ground does not liquefy at ULS levels of shaking
- an URM building on alluvial soils where the limiting structural capacity is lower than the liquefaction triggering level, and
- a building that is structurally well tied together on potentially liquefiable ground or ground prone to shaking-induced settlement, when the deformations expected are unlikely to lead to a significant life safety hazard.

C1.9.3 Interactive

These are projects where geohazards, soil nonlinearity and SSI may have an influence on the critical structural mechanism(s). Discussions with a specialist geotechnical engineer would be expected to evaluate the extent to which the geotechnical issues might affect the rating and how this should be allowed for in the assessment.

Examples include:

- a reinforced concrete frame building on well-tied shallow foundations, where the behaviour of the structure is affected by soil deformations
- a piled structure where liquefaction-induced lateral spreading imposes potentially intolerable lateral displacements

- a piled structure where liquefaction above the founding level of the piles results in significant horizontal ground lurch
- a reinforced concrete frame building on shallow foundations where column restraint may be reduced by foundation rotations/displacements due to soil flexibility
- rocking of shear walls in a combined moment frame building, and
- liquefaction-induced down-drag on a piled structure.

C1.9.4 Geotechnically dominated

For these projects the structure response is likely to be governed by geohazards, ground behaviour and SSI. Geotechnical step change (described in Section C4) is often a characteristic of ground and foundation performance in these scenarios. In addition to discussions as indicated for the interactive category above, the geotechnical engineer may need to carry out assessments of the ground behaviour which may then become a separate SW (noting that it is only issues that have a direct influence on the structure behaviour that can be considered in the earthquake rating, and then only when a significant life safety hazard can result).

Examples include:

- a building on shallow foundations where the pads supporting the bracing elements punch through into liquefiable soils
- a building on shallow foundations on liquefiable soil incorporating tension-only ground anchors holding down its bracing elements
- a piled structure located behind a seawall subject to collapse upon backfill liquefaction, and
- a piled structure on sloping ground subjected to downslope deformations.

C1.10 Reporting

C1.10.1 Communicating seismic risk

Communication of the seismic risk and the assessed seismic behaviour of the building is a very important part of the DSA process. The written report should be carefully written to suit its intended audience.

The level of ground shaking should be expressed in terms of the return period. This can be expressed as the proportion (e.g. one third, one half, two thirds, full) of the 500 year return period shaking for the site or %ULS loading for IL2 and adjusted appropriately for other ILs.

The term "*%NBS* shaking" should not be used as this does not correctly convey the contribution that the building ultimate capacity (strength and deformation) makes to the standard achieved and the expected behaviour of the building. *%NBS* is not solely about the loading. It therefore confuses "seismic demand" with "earthquake rating".

C1.10.2 Suggested report content

The following information is recommended as minimum content for a DSA report:

- the scope and objective of the DSA
- assessment methodology, limitations and list of assumptions. This includes documentation/drawings available and sources of information reviewed, including any previous assessments
- background on the regulatory requirement and assessment process
- general building description including number of storeys, occupancy/use and general dimensions, heritage categorisation
- structural systems (gravity and lateral load resisting systems, foundations, etc.)
- ground condition, site seismicity and geohazards identified and impact on the seismic behaviour of the building
- assessment results
- SWs (including any SSWs) identified and assessed (in primary seismic and gravity structure, secondary structure and critical non-structural components)
- *%NBS* and assumed Importance Level, the CSW that defines the *%NBS*, and reconciliation with any previous assessments
- modes of failure for any SWs scoring less than 34*%NBS* to allow Territorial Authorities to confirm earthquake-prone status
- secondary risks (adjacent building, non-structural element, stairs) if applicable, and
- any recommendations.

An executive summary that summarises the assessment, the key aspects of the building and the key outputs is considered desirable.

Appendix C1A provides a recommended report template.

A separate geotechnical report should be appended to the DSA (the level of geotechnical reporting will depend on the geotechnical contribution to the building's performance) – refer to Section C4 for more details.

C1.10.3 Technical Summary

To achieve consistency in assessment outputs being reported and to allow comparisons between assessments of multiple buildings, a stand-alone technical summary should be provided as part of the reporting using the template that can be found in Part A.

Note:

Providing a technical summary in a consistent form is considered to be an essential part of any DSA. It will be very useful for Territorial Authorities managing the requirements of the earthquake-prone building legislation, to owners of multiple buildings, and also to any future engineers of the same building.

The same template is presented in Part B to record the results of an ISA when this is used as the justification for the earthquake rating for earthquake-prone building purposes.

References

AS/NZS 1170.0:2002. *Structural design actions – Part 0: General principles*, Standards Australia/Standards New Zealand.

ASCE 41-13 (2014). *Seismic Evaluation of Existing Buildings,* American Society of Civil Engineers and Structural Engineering Institute, Reston, Virginia, USA.

EN 1998-3:2005. *Eurocode 8 – Design of structures for earthquake resistance – Part 3: Assessment and retrofitting of buildings*, European Committee for Standardization (CEN), Updated in 2005.

Module 1 of the Geotechnical Earthquake Engineering Practice Series dated March 2016.

NZBC B1 (2014), Acceptable Solutions and Verification Methods for New Zealand Building Code Clause B1 Structure, Ministry of Business Innovation & Employment, Wellington, NZ.

NZS 1170.5:2004. *Structural Design Actions, Part 5: Earthquake actions – New Zealand*, Standards New Zealand, Wellington, New Zealand.

Appendix C1A: Recommended Report Template for a Detailed Seismic Assessment

Detailed Seismic Assessment - XX Tremor Grove, Shakesville

Prepared by ……. **CPEng Nº……..**

Dated …….

Executive Summary

Background

Provide a summary of the background to the Detailed Seismic Assessment (DSA) including:

- *who the report has been carried out for and the intended scope*
- *any previous assessment(s) and the resulting earthquake rating result(s), including assumed IL*
- *The name and/or address for the building.*

Building Description

Provide a brief description of the building including relevant features such as:

- *age*
- *structural configuration*
- *current usage*
- *primary structural system (lateral and gravity)in each direction*
- *relationship to neighbouring buildings*
- *any previous retrofit*
- *heritage status*
- *original design standard, if known*
- *foundation type*
- *subsoil description*
- *any identified geohazards*
- *impact of geotechnical aspects on building behaviour*

Assessed Earthquake Rating

The results of the DSA indicate the building's earthquake rating to be [**XXX**]*%NBS* (IL**Y**) assessed in accordance with the guideline document *The Seismic Assessment of Existing Buildings-Technical Guidelines for Engineering Assessments*, dated [**ZZZ**]. The earthquake rating assumes that Importance Level [**Y**] (IL[**Y**]), in accordance with the Joint Australian/ New Zealand Standard – Structural Design Actions Part 0, AS/NZS 1170.0:2002, is appropriate.

Therefore this is a Grade [W] building following the NZSEE grading scheme. Grade [**A+/A/B/C/D**] buildings represent a risk to occupants [**less than/equivalent to/5-10 times/10-20 times/greater than 25 times**] that expected for a new building, indicating a [**low and acceptable/low and acceptable/acceptable/high/very high**] risk exposure**.**

A building with an earthquake rating less than 34*%NBS* fulfils one of the requirements for the Territorial Authority to consider it to be an Earthquake-Prone Building (EPB) in terms of the Building Act 2004. A building rating less than 67*%NBS* is considered as an Earthquake Risk Building (ERB) by the New Zealand Society for Earthquake Engineering. [**XX Tremor Grove, Shakesville**] is [**not**] therefore categorised as an Earthquake Risk Building and also it **[does not fall below/meets one of]** the criteria that could categorise it as an Earthquake Prone Building.

In accordance with the provisions of the Earthquake Prone Building requirements of the Building Act 2004 the determined earthquake rating requires the following actions for this building:

Summarise the requirements for TA confirmation of status, and upgrading and timeframes for addressing if the building is confirmed as earthquake prone.

The assessment identified the following SWs in the building:

List the SWs (including any identified SSWs) and associated earthquake scores in ascending order and note the modes of failure for any SWs scoring less than 34%NBS.

The following secondary structural and non-structural aspects were considered in the assessment of the earthquake rating:

List the non-structural aspects considered

The CSW was found to be **[complete]**.

As part of the assessment we also noted the following:

List other noteworthy items reviewed during the assessment process that would be expected to be of interest to the report recipient. This includes aspects that may or may not have influenced the earthquake rating; e.g. stairs, geohazards, neighbouring buildings.

Also discuss the next lowest scores where the CSW is a relatively easily addressed issue as this will provide context for the earthquake rating.

Acknowledge previous assessments and provide an explanation for any discrepancies.

Basis for the Assessment

This assessment has been based on the following information:

List the information that has been available for this assessment; e.g.

- *original construction (structural/architectural) drawings dated….*
- *original construction specifications dated…*
- *on-site inspection completed on ….*
- *intrusive investigations comprising…*
- *materials testing of….*
- *ISA report dated…*
- *previous assessments….*
- *geotechnical report dated…*

Seismic Retrofit Options (if required)

Depending on the brief, the engineer may wish to include this heading and some options for retrofitting. This may involve addressing the CSWs in turn and the effect this would have on the earthquake rating.

Recommended Next Steps (if required)

Depending on the brief, the engineer may wish to include this heading and some recommended next steps.

Technical Summary (present as pages that can be separable from the report- refer to Part A)

Main Report

The structure of the main report will depend on the assessment brief and personal preference. The following is a suggested contents list.

Introduction

Building Description

Results of the Detailed Seismic Assessment

Commentary on Seismic Risks

Commentary on Associated Seismic Risks/Hazards

Seismic Retrofit Options (if required)

Recommended Next Steps (if required)

Appendices

PART C

Assessment Procedures and Analysis Techniques C2

Document Status

Version	Date	Purpose/ Amendment Description
1	July 2017	Initial release

This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

This document may be downloaded from www.EQ-Assess.org.nz in parts:

1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

This document is managed jointly by the Ministry of Business, Innovation and Employment, the Earthquake Commission, the New Zealand Society for Earthquake Engineering, the Structural Engineering Society and the New Zealand Geotechnical Society.

Please go to www.EQ-Assess.org.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.EQ-Assess.org.nz

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C2. Assessment Procedures and Analysis Techniques

C2.1 General

C2.1.1 Scope and outline of this section

This section sets out the elastic and nonlinear assessment procedures that can be used in the Detailed Seismic Assessment (DSA) Steps 4 to 11 (outlined in Section C1). It includes recommendations for selecting the most appropriate procedures and associated analysis techniques, details of these, and guidance on specific issues.

The procedures presented include a first principles, mechanism-based method based on either a force or displacement-based approach. A significant change from the previous edition of these guidelines (NZSEE, 2006) is the emphasis on understanding the nonlinear behaviour of the structural systems present, even when elastic-based procedures are being used.

For this reason, these guidelines recommend using the Simple Lateral Mechanism Analysis (SLaMA) procedure as a first step in any assessment. While SLaMA is essentially an analysis technique, it enables engineers to investigate (and present in a simple form) the potential contribution and interaction of a number of structural elements and their likely effect on the building's global capacity.

In some cases, the results of a SLaMA will only be indicative. However, it is expected that its use should help engineers achieve a more reliable outcome than if they only carried out a detailed analysis, especially if that analysis is limited to the elastic range. The objective is not to rely on sophisticated techniques without first developing an understanding of how the building resists seismic loads and identifying the various critical load paths and how the various systems might interact.

Note:

The previous edition (NZSEE, 2006) presented both force-based and displacement-based assessment approaches.

The force-based assessment was based on a first principles approach developed by Priestley and Calvi, 1991 and by Park, 1996 for reinforced concrete frames. This was developed further by Priestley in 1996 but in the form of a displacement-based approach (Priestley, 1996) and has now been significantly expanded to clarify the procedures and consider some practical implementation issues (Kam et al. 2013).

It has been observed that the traditional force-based assessment approach is often misused by engineers who attempt to link it too closely to the design process adopted for a new building. This has been neither straightforward nor successful, as current capacity design procedures are deterministic in nature and rarely achievable in older buildings.

While these guidelines emphasise the understanding of the nonlinear behaviour of structures and the governing inelastic/collapse mechanism, it is acknowledged that most designers are currently more familiar with force-based procedures. Therefore, these procedures have been retained but with additional requirements.

These guidelines shift the assessment focus away from an overreliance on detailed analysis and, instead, encourage this as just one way to better understand the way in which the building resists seismic loads.

These guidelines also stress that structural analysis is just one part of the assessment procedure and that identifying potential modes of behaviour (mechanisms), potential structural weaknesses (SWs) and severe structural weaknesses (SSWs) is crucial. To do this satisfactorily, even for relatively simple buildings, requires judgement and experience so that the analysis techniques chosen allow the engineer to appropriately investigate and quantify the relevant issues.

Note:

It is apparent that many previous DSAs have been approached using a detailed analysis as the principal – and often only – part of the assessment procedure. This approach has largely been driven by the adherence to conventional structural design processes, which rely on an elastic analysis as the principal means of deriving internal actions for member sizing. This is rarely appropriate for seismic assessment.

This section provides guidance on the:

- assessment procedures supported by these guidelines and issues that might arise in following them (Sections C2.2 to C2.5), and
- analysis techniques that can be adopted (Sections C2.6 to C2.9).

Note:

A significant amount of material on specific subjects has been moved to the appendices to improve flow and readability. Much of this material, when relevant, is essential to these guidelines and forms an integral part of them.

C2.1.2 Definitions and acronyms

ADRS	Acceleration-displacement response spectrum (spectra)
Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable strength capacity has been reached. A brittle structure has little tendency to deform before it fractures.
Critical structural weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be potential critical structural weaknesses.
Damping	The value of equivalent viscous damping corresponding to the energy dissipated by the structure, or its systems and elements, during the earthquake. It is generally used in nonlinear assessment procedures. For elastic procedures, a constant 5% damping as per NZS 1170.5:2004 is used.
DBA	Displacement-based assessment
DDBD	Direct displacement-based design
Design level/ULS earthquake (shaking)	Design level earthquake shaking or loading is taken to be the seismic load level corresponding to the ULS seismic load for the building at the site as defined by NZS 1170.5:2004 (refer to Section C3)
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines
Diaphragm	A horizontal structural element (usually a suspended floor or ceiling or a braced roof structure) that is connected to the vertical elements around it and that distributes earthquake lateral forces to vertical elements, such as walls, of the primary lateral system. Diaphragms can be classified as flexible or rigid.
Ductile/ductility	Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
Elastic analysis	Structural analysis technique that relies on linear-elastic assumptions and maintains the use of linear stress-strain and force-displacement relationships. Implicit material nonlinearity (e.g. cracked section) and geometric nonlinearity may be included. Includes equivalent static analysis and modal response spectrum dynamic analysis.
Equivalent static analysis (ESA)	Equivalent static analysis as prescribed in NZS 1170.5
Flexible diaphragm	A diaphragm which for practical purposes is considered so flexible that it is unable to transfer the earthquake loads to shear walls even if the floors/roof are well connected to the walls. Floors and roofs constructed of timber, and/or steel bracing in a URM building, or precast concrete without reinforced concrete topping fall in this category. A diaphragm with a maximum horizontal deformation along its length that is greater than or equal to twice the average inter-storey drift. In a URM building a diaphragm constructed of timber and/or steel bracing.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.
Irregular building	A building that has an irregularity that could potentially affect the way in which it responds to earthquake shaking. A building that has a sudden change in its plan shape is considered to have a horizontal irregularity. A building that changes shape up its height (such as one with setbacks or overhangs) or that is missing significant load-bearing elements is considered to have a vertical irregularity. Structural irregularity is as defined in NZS 1170.5:2004.

MRSA	Modal response spectrum analysis
NLSPA	Nonlinear static pushover analysis
Nonlinear analysis	Structural analysis technique that incorporates the material nonlinearity (strength, stiffness and hysteretic behaviour) as part of the analysis. Includes nonlinear static (pushover) analysis and nonlinear time history dynamic analysis.
Nonlinear time history analysis (NLTHA)	An analysis of the building using strong motion earthquake records and modelling the nonlinear behaviour of the structure (also referred to as nonlinear response history analysis)
Non-structural element	An element within the building that is not considered to be part of either the primary or secondary structure
PGA	Peak ground acceleration
Primary gravity structure	Portion of the main building structural system identified as carrying the gravity loads through to the ground. Also required to carry vertical earthquake induced accelerations through to the ground. May also be part of the primary lateral structure.
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be part of the primary gravity structure
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects this is determined using probable material strengths. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
pseudo-Equivalent static analysis (pESA)	Loading for rigid diaphragm assessment. Refer to Section C2.9.3 and the broader definition in NZS 1170.5:2004.
Rigid diaphragm	A diaphragm that is not a flexible diaphragm
Secondary structure	Portion of the structure that is not part of either the primary lateral or primary gravity structure but, nevertheless, is required to transfer inertial and vertical loads for which assessment/design by a structural engineer would be expected. Includes precast panels, curtain wall framing systems, stairs and supports to significant building services items
Serviceability limit state (SLS)	Limit state as defined in AS/NZS 1170.0:2002 (or NZS 4203:1992) being the point at which the structure can no longer be used as originally intended without repair
Severe structural weakness (SSW)	A defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge
SFSI	Soil-structure-foundation-interaction (refer to Section C2.9.2)
Simple Lateral Mechanism Analysis (SLaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (push-over) relationship for the building as a whole
Single-degree-of-freedom (SDOF)	A simple inverted pendulum system with a single mass
Soil-structure-foundation-interaction (SFSI)	Interaction between the structure of the building and the foundation and soil surrounding it. Synonymous with SSI
SRSS	Square root sums of squares method of combining variables
SSI	Interaction between the structure (including foundations) and the soil surrounding the foundation

Structural element	Combinations of structural members that can be considered to work together; e.g. the piers and spandrels in a penetrated wall, or beams and columns in a moment resisting frame
Structural member	Individual items of a building structure, e.g. beams, columns, beam/column joints, walls, spandrels, piers
Structural sub-system	Combination of structural elements that form a recognisable means of lateral or gravity load support for a portion of the building: e.g. moment resisting frame, frame/wall. The combination of all of the sub-systems creates the structural system.
Structural system	Combinations of structural elements that form a recognisable means of lateral or gravity load support; e.g. moment resisting frame, frame/wall. Also used to describe the way in which support/restraint is provided by the foundation soils.
Structural weakness (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100*%NBS.* Note that an aspect of the building structure scoring less than 100*%NBS* but greater than or equal to 67*%NBS* is still considered to be a structural weakness even though it is considered to represent an acceptable risk.
Ultimate capacity (seismic)	A term defined in regulations that describes the limiting capacity of a building for it to be determined to be an earthquake-prone building. This is typically taken as the probable capacity but with the additional requirement that exceeding the probable capacity must be associated with the loss of gravity support (i.e. creates a significant life safety hazard).
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings
Unreinforced masonry (URM)	A member or element comprising masonry units connected together with mortar and not containing any steel, timber, cane or other reinforcement
(XXX)*%NBS*	The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage. Intended to reflect the expected seismic performance of a building relative to the minimum life safety standard required for a similar new building on the same site by Clause B1 of the New Zealand Building Code.
(XXX)%ULS shaking (demand)	Percentage of the ULS shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.

C2.1.3 Notation, symbols and abbreviations

Symbol	Meaning
%NBS	Percentage of new building standard as assessed by application of these guidelines
$ADRS_{\mathrm{ULS}}$	Refer to Figure C2.7
$A_{\mathrm{c}}/A_{\mathrm{g}}$	Low core-to-gross concrete area - refer to Section C2G.2
$A_{\mathrm{g}}f'_{\mathrm{c}}$	Refer to Section C2G.2 & C2G.3
A_{χ}	Torsional amplification multiplier
C	Refer to Table C2A.1
$C(T_1)$	Ordinate of elastic site hazard spectrum for first mode period T_1 and relevant site subsoil type as defined in NZS 1170.5:2004
C_{vx}	Vertical load distribution as defined in Equation C2.18
El	Refer to Figure C2A.1
$e_{\mathrm{stiffness}}$	Distance between centre of mass and centre of rigidity/stiffness
e_{strength}	Distance between centre of mass and centre of strength
e_{vx}	Strength eccentricity - refer to Figure C2F.2 and Section C2F.4
e_{vy}	Refer to Figure C2F.2
F	Generalised term for lateral force
f'_{c}	Refer to Figure C2A.1
F_{y}	Generalised term for yield force
g	Acceleration of gravity (9.81m/s^2)
G_{j}	Joint rotational springs – refer to Figure C2C.2
H	Refer to Table C2A.1
h_{eff}	Effective height of the equivalent SDOF system
$h_{\mathrm{eff,beam\ sidesway}}$	Refer to Table C2A.1
$h_{\mathrm{eff},col\ sidesway}$	Refer to Table C2A.1
$h_{\mathrm{effmixed\ sidesway}}$	Refer to Table C2A.1
h_{i}	Height (in m) from the base to floor level i
h_{n}	Height from the base of the building to the top of the primary lateral structure
h_{sx}	Refer to Table C2A.1
h_{x}	Height (in m) from base to floor level x
h_{w}	Height of the walls – refer to Table C2A.1
k	Refer to Equation C2.16
k_{eff}	Effective stiffness of the equivalent SDOF system
k_{s}	Refer to Figure C2A.3

Symbol	Meaning
k_μ	Inelastic spectrum scaling factor based on the achievable ductility of the building, μ_{sys} ,in accordance with NZS 1170.5:2004
L	Distance between the centroids of the lateral load resisting lines in the orthogonal direction
L_w	Length of wall
l_w	Refer to Figure C2F.2
l_{wt}	Refer to Figure C2F.2
Mb	Beam moment – refer to Figure C2A.1
M_{bl}, M_{br}	Beam expected maximum flexural strengths at the left and right of the joint, respectively, at the joint centroid
M_c	Column moment – refer to Figure C2A.1
M_{ca}, M_{cb}	Minimum expected column flexural strengths above and below the joint, respectively, at the centroid of the joint
m_{eff}	Effective mass of the equivalent SDOF system
m_i	Lumped mass at level i
m_n	Refer to Figure C2.5 a)
M_{wp}	Refer to Table C2A.1
n	Number of levels above the base of the primary lateral structure
N^*_g	Refer to Section C2G.3
N_f	Refer to Table C2A.1
n_t	Number of storeys
OTM	Refer to Table C2A.1
p'_t	Refer to Figure C2A.1
$P-delta$	The destabilising effects from a compression force on a column or building acting through the lateral end displacement of a column or building drift
$P/A_g f'_c$	High axial load demand - refer to Section C2G.2
Q_u	Dead and reduced live load – refer to Section C2G.3
r	Generalised post-yield stiffness
r_b	Beam plastic hinge – refer to Figure C2C.2
r_c	Column plastic hinge – refer to Figure C2C.2
r_j	Joint rotational springs – refer to Figure C2C.2
R_v	Refer to Section C2E.4
S	Separation between adjacent buildings at any level (for pounding assessment). Refer to Appendix C2B.
s	Spacing – refer to Section C2G.2
S_a	Pseudo spectral acceleration
S_d	Pseudo spectral displacement

Symbol	Meaning
S_{i}	Sway index
S_{p}	Structural performance factor associated with the detailing and assessed ductility of the system. Determined in accordance with NZS 1170.5:2004. Refer to Section C3.
$S_{\mathrm{p,nltha}}$	Refer to Section C2C.6
T	Refer to Table C2A.1
T_1	Fundamental period of the building which has the greatest mass participation
T_{eff}	Effective period of the equivalent SDOF structure
V	Total lateral seismic force. Refer to Figure C2.11 in C2.7.3, Figure C2A.4 and Table C2A.1
V_{b}	Refer to Table C2A.1
V_{base}	Horizontal seismic base shear
$(V_{\mathrm{base}})_{\mathrm{i}}$	Horizontal seismic base shear associated with sub-system i
v_{c}	Refer to Section C2G.4
V_{E}	Refer to Section C2E.5
V_{e}	Plastic base shear corresponding to M_{e}
V_{f}	Refer to Table C2A.1
$V_{\mathrm{o/s}}$	Horizontal seismic shear consistent with the overstrength capacity of the structure
V_{prob}	Probable horizontal seismic capacity of the global structural system
$\left(V_{\mathrm{prob}}\right)_{\mathrm{i}}$	Probable horizontal seismic base shear associated with sub-system i
v_{s}	Refer to Section C2G.4
V_{shear}	Refer to Table C2A.1
V_{u}	Refer to Figure C2.11 in C2.7.3 and Table C2A.1
V_{wp}	Refer to Figure C2F.2
V_{ℓ}	Refer to Figure C2.11 in C2.7.3
W_{t}	Total seismic weight of structure
w_{i}	Portion of total building weight W on floor level i
w_{x}	Portion of total building weight W on floor level x
α	Refer to Section C2.9.4 and Figure C2C.2
β	Refer to Figure C2C.2
Δ	Lateral displacement at centre of action of lateral seismic forces. Refer to Figure C2.11 in C2.7.3
Δ_{bX}	Refer to Table C2A.1
Δ_{cap}	Global system displacement capacity – refer to Section C2.5.11
$(\Delta_{\mathrm{elastic}})_{\mathrm{i}}$	Elastic displacement at level i $\leq \left(\Delta_{\mathrm{y}}\right)_{\mathrm{i}}$

Symbol	Meaning
Δ_{fy}	Refer to Figure C2A.3
Δ_i	Refer to Figure C2.5 a), Step D3 and Equation C2A.2
$\Delta_{plastic}$	Plastic displacement (or rotation)
$(\Delta_{plastic})_i$	Plastic lateral displacement at level i
Δ_{prob}	Probable lateral displacement capacity of the global structural system
$(\Delta_{prob})_i$	Probable lateral displacement capacity of the global structural system at level i
$(\Delta_{prob})_{top}$	Probable lateral displacement capacity of the global structural system measured at the top of the primary lateral structure
Δ_s	Refer to Table C2A.1
Δ_u	Global roof displacement – refer to Figure C2.9 in C2.5.11 and Figure C2A.3
$\Delta_{u,sys}$	Refer to Figure C2.9 in C2.5.11
Δ_{ULS}	100%ULS demand deflection on the equivalent inelastic SDOF system (i.e. for T_{eff} , ξ_{sys})
ΔV_t	Shear force increase in the lateral load resisting members in the orthogonal direction
Δ_{wy}	Refer to Figure C2A.3
Δ_X	Refer to Table C2A.1
Δ_y	Yield displacement (or rotation)
$(\Delta_y)_i$	Lateral yield displacement at level i
$\Delta_{y,sys}$	Refer to Figure C2.9 in C2.5.11
$(\Delta_y)_{top}$	Refer to Section C2.3.1 Step F1 and Section C2.A.2 Step 5 & 6
δ_1	Estimated lateral deflection of Building 1 relative to ground under the loads used for the assessment
δ_2	Estimated lateral deflection of Building 2 relative to ground under two-thirds of the loads used in the assessment
$\delta_{average}$	Refer to Equation C2.14 in C2.5.8
δ_i	Displacement profile for the primary structural system normalised to the top level displacement
δ_{max}	Refer to Equation C2.14 in C2.5.8
ε_c	Refer to Figure C2A.1
ε_y	Yield strain of steel reinforcement
θ	Refer to Table C2A.1
θ_b	Beam hinge rotation – refer to Figure C2A.2
θ_c	Column hinge rotation – refer to Figure C2A.2
θ_j	Joint rotation – refer to Figure C2A.2
θ_p	Plastic rotation at the base of a wall

Symbol	Meaning
μ	Structural ductility in accordance with NZS 1170.5:2004
μ_{f}	Ductility available from the frame. Refer to Figure C2A.3
μ_{hy}	Ductility of part in accordance with NZS 1170.5:2004
μ_{p}	Ductility of part in accordance with NZS 1170.5:2004
μ_{sys}	Achievable global structural ductility for the system
μ_{w}	Ductility available from the wall. Refer to Figure C2A.3
ξ_{hy}	Available hysteretic damping for the structural system
ξ_{0}	Available Inherent equivalent viscous damping
ξ_{d}	Added damping due to supplemental viscous dampers – refer to Equation C2D.1
ξ_{i}	Refer to Equation C2.11 in C2.4.2 Step D3 and Equation C2A.3 in C2A.2 Step 6
ξ_{sys}	Achievable equivalent viscous damping for the global structural system
$\sum_{\mathrm{i}} M_{\mathrm{coli}}$	Refer to Table C2A.1
$\sum_{\mathrm{n}} V_{\mathrm{end\ beam.n}}$	Refer to Table C2A.1
$\sum_{\mathrm{x}} V_{\mathrm{end\ beam.x}}$	Refer to Table C2A.1
$\sum V_{\mathrm{wpi}}$	Refer to Figure C2F.2
ϕ	Strength reduction factor
ϕ_{ob}	Overall building overstrength factor (for diaphragm assessment)
χ	Ratio of the maximum displacement at any point on the level x diaphragm to the average displacement $\chi = \delta_{\mathrm{max}}/\delta_{\mathrm{average}}$
ω_{v}	Dynamic magnification factor for shear demand

C2.2 Choosing an Assessment Procedure

C2.2.1 Key objectives of the DSA

The fundamental objective of a DSA should be to understand the structural load paths and likely behaviour of the building in earthquake shaking in sufficient detail to allow quantification of the behaviour and the earthquake rating. A good understanding of the governing nonlinear load path generally leads to a consistent assessment outcome.

The most suitable procedure will depend on the circumstances, and many buildings will not require – or justify – the use of lengthy and detailed analysis. However, the focus in all cases should be on determining the displacement of the structure and the governing inelastic lateral and loss-of-gravity support mechanisms during "severe" earthquakes. Internal actions generated, such as shear, moment and axial load, should be considered as consequences of this deformation, not the cause of it.

Note:

This is the essence of the displacement-based procedures covered in these guidelines. As noted earlier, force-based procedures are still included but engineers are expected to adopt displacement-based thinking when using them, especially when mixed mode systems are present.

These guidelines also recommend that the capacity of a building should be considered independently from the demands (imposed inertial loads and displacements) placed on it, bringing both together in the final step of the assessment.

The extent to which the structure is modelled and the length to which other analysis needs to be carried out requires careful thought. A qualitative overview of the structure (such as might be obtained by applying the ISA procedures) will help to identify structural weaknesses and/or particularly vulnerable elements.

Note:

Engineers should remind themselves that the objective of the earthquake-prone building legislation is to reduce seismic risk. It may be better for a relatively crude but effective strengthening measure to be carried out than to postpone strengthening work while the owner saves up to pay for an unnecessarily expensive analysis.

Sophisticated analyses should be complemented with various preliminary "throw-away" studies to gain insights into the complex structural response and its sensitivity to various input parameters.

More focus on the assessment of the loss of gravity load support and "brittle" inelastic mechanisms is also recommended, noting that both of these are challenging to model explicitly using commercially available analysis packages. It may be necessary to carry out a series of analyses using various techniques, in conjunction with post-processing assessment of the gravity load path, to confirm the capacity of the building.

Note:

Any analysis requires a level of judgement, experience and expertise, but this is especially so when using more sophisticated procedures. The dangers of relying on "black box" analysis, where the impact of the assumptions made or included within the analysis are not always obvious to the engineer, should be apparent.

The uncertainties, precision and reliability remain a function of the level of checking and rigour of the analysis (e.g. the number of runs, sensitivity analysis and well-defined analysis parameters). Even the most advanced analysis should always be "tested" using simpler models and rational methods.

C2.2.2 Determining a suitable approach

This section divides the assessment procedures into two categories, each with a corresponding selection of analysis techniques. These are:

- **Elastic-based assessment procedures** (force-based), which employ analysis techniques such as:
  - simplified nonlinear pushover analysis using SLaMA (an essential step)
  - equivalent static analysis (linear static)
  - modal response spectrum analysis (linear dynamic)
  - pseudo-nonlinear static pushover analysis

  Note:

  A SLaMA should be carried out as part of the elastic-based procedure because of the insights this is expected to bring into the expected seismic behaviour of the building, particularly as it goes into the nonlinear range. This is discussed below.

- **Nonlinear assessment procedures** (force-based and displacement-based), which employ analysis techniques such as:
  - simplified nonlinear pushover analysis using SLaMA
  - nonlinear static pushover analysis
  - nonlinear time history analysis (nonlinear dynamic).

These assessment procedures are considered applicable for all lateral force-resisting elements and materials. However, they may require some modification depending on the circumstances. The necessary adaptation for particular materials and structural forms is indicated in the specific material sections (Sections C5 to C9).

Note:

These guidelines distinguish between assessment procedures and the structural analysis techniques used within these procedures. While these concepts have previously been interchangeable, it has been recognised that too much focus on the analysis – particularly on computer-based modelling – can affect the validity of the DSA outcome. The emphasis is therefore on the assessment procedure as a whole, which uses detailed structural analysis as one of the techniques to gain a better understanding of the building's seismic behaviour.

SLaMA is recommended as a starting point whether the engineer selects an elastic or nonlinear based assessment procedure. It is also recommended that engineers use a range of analysis techniques to support their assessment as this is often necessary to address all of the issues. For example, the use of modal response spectrum analysis by itself may be of little use for mixed-ductility systems, but the linear dynamic analysis result can give some information about the higher mode behaviour and potential to affect the response.

Note:

While there are no specific restrictions on which assessment procedure an engineer can use, Section C2.6 outlines some limitations with the associated analysis techniques which may inform this choice.

Engineers should start simply and gradually increase the sophistication of their assessment and associated analyses as appropriate. In practice, engineers may adopt an iterative process of selecting the appropriate techniques as the assessment progresses, as implied by the DSA flowchart in Section C1.

Figure C2.1 summarises the common assessment procedures and serves as a reminder that they all involve simplifications and assumptions regarding earthquake shaking, building characteristics, analysis and the likely performance of structural elements.

This figure also illustrates that force-based and displacement-based assessments are two ways of looking at the same issue:

- In the linear/force-based approach, the behaviour of the various components is assessed by examining the forces in critical elements and using rules to assess the limits of integrity of the structural members.
- In the nonlinear/displacement-based approach, the response of the building structure is considered from the outset on the basis of the structural displacements resulting from the ground shaking. These are then used to examine the effect on the structural elements, again using rules to measure the limits of integrity and performance.

In relation to Figure C2.1:

- **Modelling of the earthquake shaking** – this will vary according to the assessment procedure and analysis techniques used. For example, the NZS 1170.5:2004 design hazard spectra may be used for force-based equivalent static or modal analysis but a site specific set of earthquake records (scaled as required by NZS 1170.5:2004) will be needed for nonlinear time history analyses. For displacement-based methods, displacement spectra can be generated using the NZS 1170.5:2004 pseudo acceleration spectra. Refer to Section C3 for further information.
- **Modelling of the structure –** numerous assumptions are necessary in relation to member properties and boundary conditions, and to the nonlinearity that should be considered and/or modelled.
- **Considering of geotechnical effects and soil-structure-interaction –** soil-structure interaction (SSI) is a key boundary condition for any structure analysis. Refer to Section C2.9.2 on SSI and to Section C4 for detailed guidance on consideration of geotechnical effects.

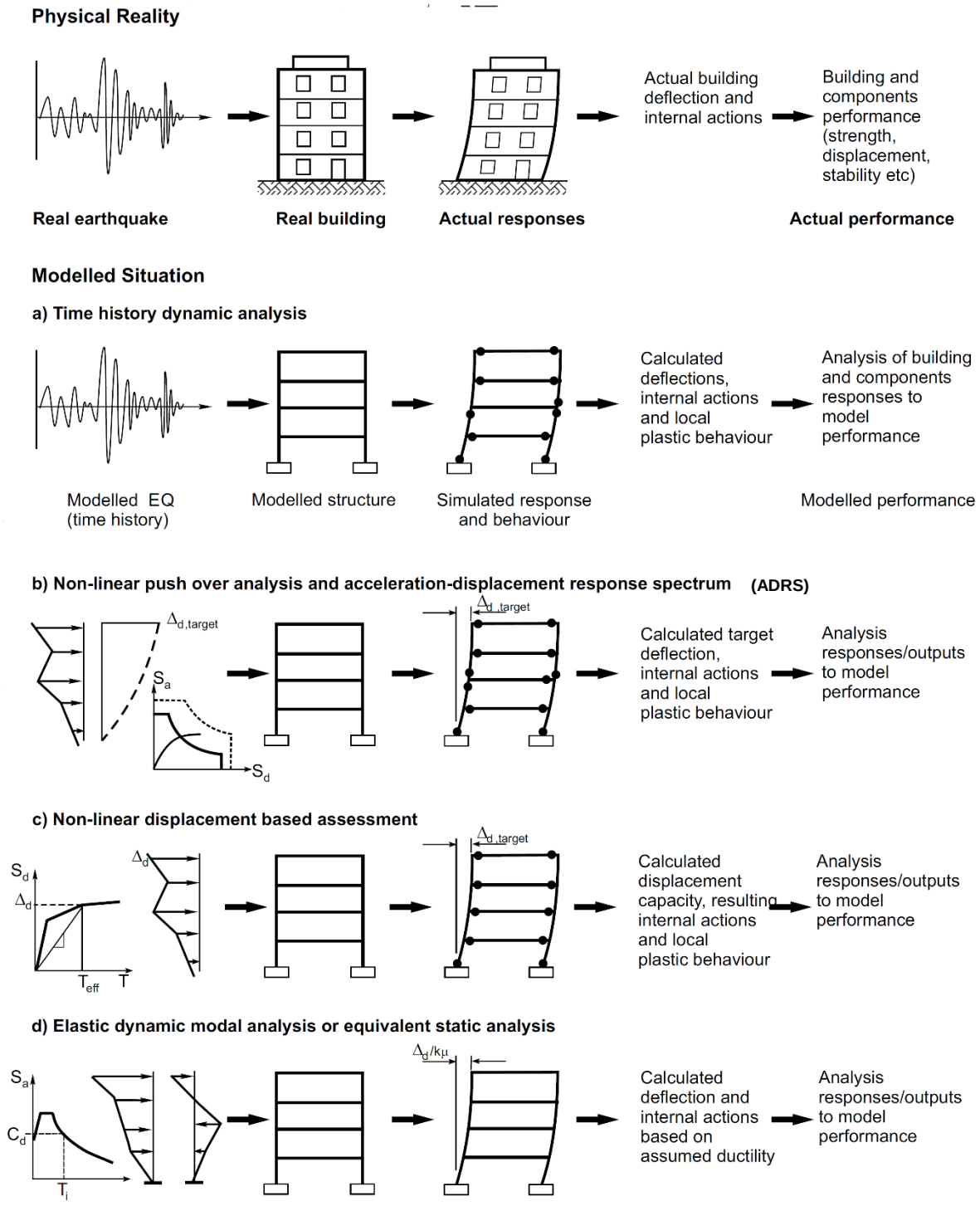


Figure C2.1: Real and modelled responses of buildings to earthquakes

- **Choice of analysis techniques –** this may depend on the building's structural configuration and level of complexity of the problem to be analysed. This choice will also drive the inputs and assumptions required, as well as the details of response derived (i.e. internal actions or deformations).
- **Modelling of the capacity of structural elements –** this process is significantly different from that used in the design of new buildings. For new buildings there are prescribed details (e.g. stirrup spacing) which will achieve the global ductility assumed. For existing buildings the ability of elements to deform plastically will depend on their detailing. A first principles approach to assess member ductility is likely to be required.

- **Comparison of demand and ultimate capacity –** irrespective of the analysis technique, a consistent measure of the probable capacity of the building should be derived. It is intended that this is always expressed in terms of the earthquake rating of the building (i.e. *%NBS*).

Note:

Figure C2.1 illustrates a range of options to characterise the ground shaking for the different assessment procedures and analysis techniques. Regardless of how the seismic demand is characterised, this should be correlated with the ultimate limit state (ULS) uniform risk design hazard spectra as given in NZS 1170.5:2004. The ULS hazard provides a benchmark for comparison of assessments using different procedures and analysis techniques.

Engineers should always be aware that all approaches – including the design hazard spectra, site-specific probabilistic uniform risk hazard spectra or deterministic scenario spectra – are predictors of long recurrence period events based on highly incomplete information.

These guidelines emphasise the need for the engineer to carry out appropriate quantitative analyses to form an understanding of the structural response and behaviour across a range of ground shaking levels rather than specifically for a single level of hazard.

C2.2.3 Treatment of uncertainties

Research in the past decade has improved the ability to identify and quantify the sources of uncertainties within the seismic assessment process. These uncertainties include:

- source seismicity and hazard – e.g. what earthquake magnitudes contribute to the hazard?
- ground motion attenuation/site effects – given an earthquake for a given tectonic mechanism at a specific distance and magnitude, what is the peak ground acceleration for a given site?
- structural responses analysis/modelling – given the ground shaking, what is the structural response in the linear and nonlinear range for a particular building?
- material properties and as-built conditions – what is the actual reinforcing detail or mortar strength?
- structural element nonlinear behaviour and capacities – given the structural response (drift or internal actions), what are the local mechanisms for the columns and beams?

While targeted in-depth or more advanced assessment (e.g. material testing, intrusive inspection, site-specific hazard analysis and nonlinear dynamic analysis) may reduce some of these uncertainties, the underlying uncertainties and the limitation of current analysis techniques remain and it is practically impossible to predict performance accurately. Conceptually, one can carry out multiple analyses with varying parameters and ground motions – i.e. the "Monte Carlo simulation" (FEMA P-58, 2012) – to get an idea of the average trends and behaviour, and this is assuming all mechanisms and failure modes can be modelled.

However, while this type of analysis may provide some insights into general trends relating to a building's seismic performance to inform code development, it is not necessarily very meaningful for the assessment of individual buildings at this stage.

These guidelines promote the use of nonlinear assessment procedures with simple analysis techniques (e.g. SLaMA or nonlinear pushover) as the preferred assessment method, as this provides a balance between the uncertainties in the input parameters and the inherent uncertainties in these simpler forms of analysis.

In practice, uncertainties can be treated in a more pragmatic way in which reasonable upper and lower bounds of key variables are assessed as part of a sensitivity analysis to form a view of the range of expected seismic performance. For example, these could be the effects of:

- variation in critical material properties such as concrete compressive strength for unreinforced beam-column joint shear strength calculation
- achievable structural ductility (μ and S_p)
- seismic loading and the associated input parameters such as site subsoil class.

The upper and lower bounds can give a range to the seismic assessment result (*%NBS*) which reflects the uncertainties in the inputs and assumptions. This also allows the engineer to determine which assumptions can be refined via further analysis, intrusive inspection, material testing, etc. in order to reduce uncertainties in the assessment result in the best manner. However, care and engineering judgement are needed so the engineer does not report a range of results that is no longer relevant to the client or end user.

Note:

The engineer should always bear in mind that the objective of assessment is not to predict the response of the building in a particular level of earthquake shaking. Rather it is to gain a better understanding of how the building might respond in earthquakes in general. Therefore, it is not considered critical to fully understand the degree of variance in material strengths (for example) other than when this might change the way in which the building will behave, i.e. affect the hierarchy of yield if this is being relied on for the assessment outcome.

C2.2.4 Role of SLaMA

As noted earlier, these guidelines recommend SLaMA as a starting point for any DSA. This is a simplified technique for determining the probable inelastic deformation mechanisms and their lateral strength and displacement capacity by examining load paths, the hierarchy of strength along critical load paths, the available ductility/displacement capacity of the identified mechanisms and the manner in which various mechanisms might work together.

Put another way, SLaMA is a method to determine the global nonlinear pushover capacity curve by summation of simple representations of the capacities of the individual members/elements/subsystems. It is expected that these summations are completed by hand.

SLaMA involves a degree of simplification and some assumptions of the structural response and capacity. For example:

- First mode response is dominant and higher mode amplification is negligible. For low-rise structures this will almost always be the case.

- The hierarchy of strength of interconnecting elements can be established by comparison of comparable "internal actions"; e.g. relative probable capacity in flexure and shear for beams.
- The governing mechanism at local element level can be extrapolated to global building behaviour by assuming either that load redistribution is possible (ductile response) or it is not (brittle global response). For example:
  - Beam flexural mechanism at different levels of a multi-storey frame building is assumed to be "ductile" locally such that sufficient load redistribution is possible to mobilise all beam flexural hinges to achieve a beam-sway global mechanism. Localised limitations on member behaviour (e.g. shear in a member) can then be superimposed based on deformation capacities if required.
  - Conversely, a weld failure in a diagonal bracing connection is a relatively brittle local mechanism. The analysis would either proceed assuming the diagonal bracing does not contribute to the lateral load resisting system beyond the deformation consistent with the connection failure, or the analysis would terminate with the connection failure being the governing global mechanism.
- A complex structural configuration can be simplified to stick models of individual bracing lines and ultimately to an equivalent single-degree-of-freedom (SDOF) oscillator.

Note:

Even for SLaMA it should not be forgotten that the focus is on assessing the capacity of the mechanism at the point a significant life safety hazard is created. This may not be at the point the capacity of the first member or element is reached.

In some ways, SLaMA is a kind of a reverse form of capacity design used for modern seismic design. By comparing the probable capacities, the hierarchy of strength and governing mechanism at each level can be determined for:

- individual members (e.g. beam, column or joint)
- interconnecting members forming structural elements (beam-column joints sub-assembly or perforated walls)
- interconnecting elements forming a system per bracing line (full height moment frame and foundation), and
- multi-systems together providing lateral resistance to the global building (summation of different systems).

This method is well developed for reinforced concrete structures (Priestley, 1996; Park, 1996; Priestley and Calvi, 1991; NZSEE, 2006 Appendix 4E.10; and Kam et al., 2013).

SLaMA's main weakness is that the sequence of development of inelastic action between different members of the structure may not be identified. For structures with low member ductility capacity there may be a tendency to overestimate the load distribution and thus also the global strength and displacement capacity.

Figure C2.2 illustrates the SLaMA assessment pathway and gives an example for a moment resisting system. Appendix C2A sets out key steps for carrying out a SLaMA.

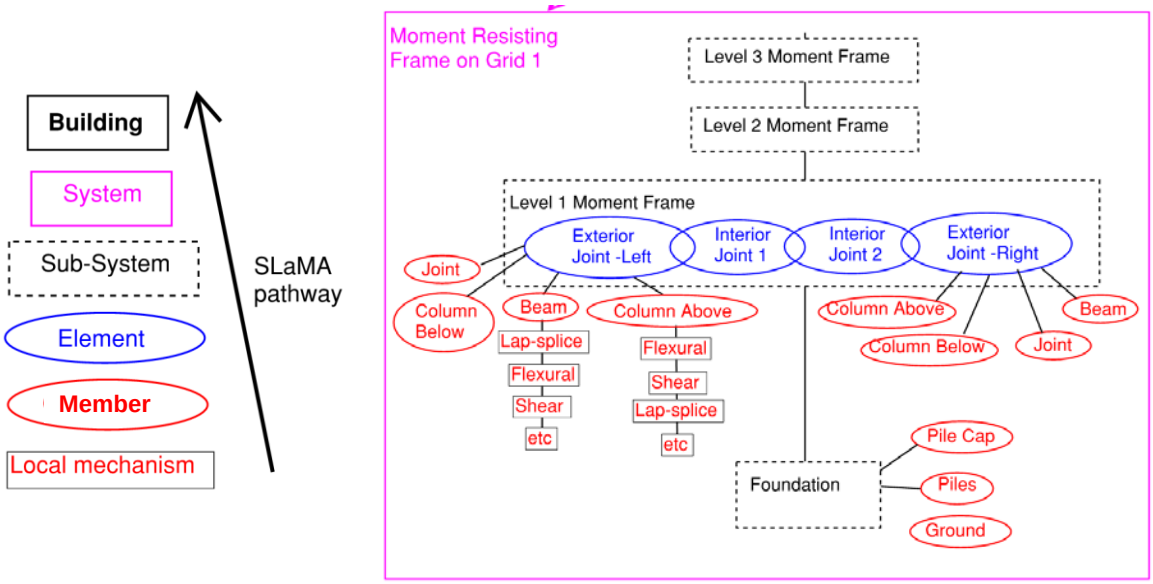


Figure C2.2: SLaMA assessment pathway and example for a moment resisting system

Note:

For many typical low to mid-rise buildings, the assessment can proceed solely on the basis of the SLaMA results as computer modelling is unlikely to provide any additional insights on the nonlinear behaviour of the building.

For more complex structures, SLaMA can inform the next level of analysis. For structures with brittle or no ductile capability, a force-based elastic analysis may suffice. Similarly, a force-based elastic analysis would suffice for structures in low seismicity regions with limited ductility demand on structural elements. SLaMA has identified the critical structural weakness and the inelastic mechanism of the critical elements/systems: therefore, a more appropriate structural ductility factor can be used within the force-based assessment.

Even if sophisticated nonlinear analyses are to be used, SLaMA provides information on the members/elements and nonlinearity that need to be modelled in detail and which mechanisms can likely be ignored in the nonlinear analysis.

C2.3 Assessment Procedure – Elastic

C2.3.1 Force-based procedure and key steps

The force-based assessment procedure outlined in these guidelines relies on the engineer having a good understanding of the probable inelastic mechanism of the structure and an indirect assessment of the local and global ductility capacities. Nonlinear behaviour is considered in the first step (SLaMA) and also indirectly in the analysis (e.g. reducing the stiffness of damaged/hinged elements).

The key steps of this procedure are modified from the recommendation of Park, 1996 and previous versions of these guidelines (NZSEE, 2006). The sequencing of, and interaction between, these steps is shown as a flowchart in Figure C2.3. More detail of each step follows.

Note:

The conventional force-based assessment procedure is based on the design for new building methodology in which the design engineer selects a structural system and a structural ductility and then calculates the required seismic base shear and internal actions for the lateral load resisting system. An elastic analysis with seismic loads (reduced for the assumed ductility) is then used to distribute the forces.

The problem with this approach is that often the structural ductility factor that is assumed will not be appropriate as the underlying assumptions will not be met. As a result, there can be an inadequate assessment of mixed ductility systems or of systems where a concentration of ductility demand occurs, e.g. presence of soft or weak storeys or potential for torsional twist, especially once the lateral system experiences nonlinear behaviour.

These guidelines recommend carrying out a SLaMA as part of the force-based assessment procedure to encourage direct determination of the ductility available. This can then be compared against the original assumption of what might have been considered possible.

The use of force-based procedures for the assessment of mixed ductility systems is problematical, just as it is for design. In such cases the engineer will not be able to avoid explicit consideration of deformation compatibility, particularly post yield, if undue conservatism is to be avoided.

Note:

Determination of global ductility from local ductilities needs to be undertaken with care. For a mixed ductility system, the global structural ductility capacity can be estimated by considering the probable yield displacement and probable deformation capacity of the various systems and limiting the assumed ductile capability to the system with the lowest deformation capacity (i.e. a displacement-based consideration is required even if a full displacement-based procedure is not followed). Refer also to Section C2.5.11.

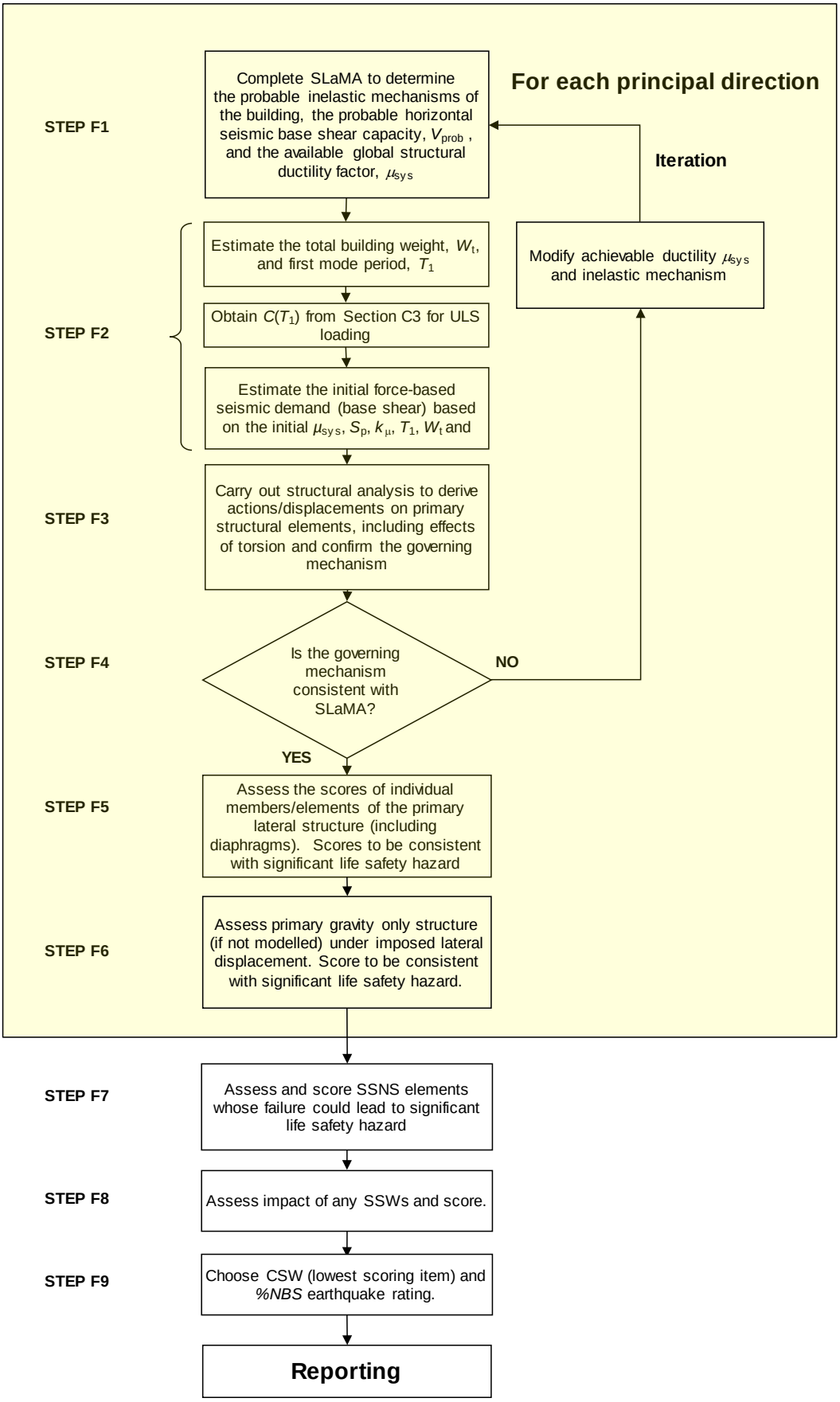


Figure C2.3: Summary of force-based assessment procedure

Step F1

Determine the inelastic sub-system mechanisms within the building that are likely to occur during seismic loading and from these calculate the probable horizontal seismic base shear capacity of the structure, V_{prob} , $(\Delta_{\text{prob}})_{\text{top}}$, and $(\Delta_{\text{y}})_{\text{top}}$ using the SLaMA technique outlined in Section C2.2.4 and Appendix C2A.

Note:

It is recommended that the SLaMA technique is extended through to calculating the global earthquake score, recognising that this may make the following steps redundant for a simple building.

This step will require the assessment of the member/element/system probable capacities and also the interaction between modes of behaviour, e.g. degrading of the ductile capacity of concrete members in the presence of shear. It provides a simplistic representation of the probable strength capacity of the primary lateral structure and also the probable lateral displacement capacity measured at the top of the primary lateral structure.

Estimate the global structural ductility factor, μ_{sys} as the ratio $(\Delta_{\text{prob}})_{\text{top}}/(\Delta_{\text{y}})_{\text{top}}$.

For multiple sub-systems, $(\Delta_{\text{y}})_{\text{top}}$ can be determined graphically as illustrated in Figure C2.7.

The global structural ductility capacity is generally governed by the dominant inelastic mechanism that contributes most to the plastic deformation of the system.

For structural systems with well distributed and defined plastic hinges of similar ductility capacity, the local structural ductility can be taken as the global ductility capacity. For example, if the local ductilities for a 1980s designed moment frame system with a ductile beam-sway mechanism is assessed as adequate for a displacement ductility of 4, the global ductility can also be taken as 4 by assuming sufficient moment redistribution can occur across the various levels.

Step F2

Estimate the fundamental period of vibration, T_1, and calculate the total seismic weight, W_{t}, of the structure.

Obtain the seismic coefficient $C(T_1)$ for ULS loading from Section C3.

Calculate the initial force-based ULS seismic demand (V_{base}) based on the estimated, μ_{sys}, and values for S_{p}, k_{μ}, T_1, W_{t} and $C(T_1)$.

$$V_{\text{base}} = \left(\frac{C(T_1)S_{\text{p}}W_{\text{t}}}{k_{\mu}}\right) \qquad \text{…C2.1}$$

where:

$C(T_1)$	=	ordinate of the elastic site hazard spectrum for T_1 and for the site subsoil type, calculated in accordance with Section C3
W_t	=	total seismic weight of the structure
S_p	=	structural performance factor (a function of μ_{sys})
k_μ	=	initial inelastic spectrum scaling factor based on the achievable structural ductility of the building, μ_{sys}.

Step F3

Carry out appropriate structural analysis to determine the internal actions (demands) for the members/elements of the primary structural system (lateral and/or gravity) and the relationship between displacement/drift and lateral load. Either an equivalent static analysis or modal response spectrum analysis can be carried out, depending on the criteria set out in Section C2.6.

If appropriate, the elastic analysis model can be modified to reflect the actual nonlinear response of the building to avoid unnecessarily penalising the assessment. For example:

- The stiffness of members identified to respond nonlinearly can be modified to reflect the secant stiffness under the ULS loading.
- Members/elements considered not to present a significant life safety hazard can either be removed from the model or made to be "fully hinged".

Where appropriate, dynamic magnification factors due to higher mode and nonlinear behaviours should be included (refer to Section C2.5.10). Refer to Section C2.7.3 for guidance on the use of a pseudo-nonlinear analysis.

The effects of torsion should be included as outlined in Appendix C2F.

Calculate the lateral displacement and inter-storey drift demand of the structure under ULS loading. The displacement and drift output from the elastic analysis should be scaled appropriately in accordance with Section 6 of NZS 1170.5:2004.

Step F4

Review the structural behaviour based on the analysis results, and if the governing mechanism is not consistent with the results from SLaMA, review the SLaMA and/or the analysis and adjust the value of μ_{sys} as appropriate and repeat from Step F2.

Step F5

Assess the scores of the members/elements of the primary lateral system by dividing their probable strength capacity by the demands determined in Step F3. Determine the lowest scoring member/element that is consistent with the development of a significant life safety hazard.

If the failure of a member/element is unlikely to present a significant life safety hazard the assessment should be reiterated removing this member/element or reducing its capacity to its residual value until the appropriate lowest scoring member/element is identified.

Note:

It is important to realise that the failure or exceedance of the capacity-to-demand ratio of one structural member does not necessarily constitute global capacity exceedance. In order for the exceedance to be relevant in this context the failure of the element or member must create a significant life safety hazard as outlined in Part A.

Also score the calculated displacements/drifts against the design limits given for the ULS in NZS 1170.5:2004. Consider carefully if the design displacement/drift limits are appropriate for the particular building and, if not, adjust the score.

Note:

For relatively lightweight existing buildings (e.g. light timber buildings and light steel industrial buildings) the drift limit of 2.5% may be too severe a constraint when there is confidence that additional drift can be accommodated without compromising either the lateral or vertical load carrying capacity of the building. To recognise this, the limits prescribed in NZS 1170.5:2004 for this type of building may be exceeded in an assessment provided that it can be shown that the capacity of the building is not compromised.

The lowest score from consideration of the strength capacity of the primary structure and the deformation limits will provide the score for the global capacity of the primary lateral structure.

Step F6

Assess the primary gravity structure, if this has not been included in the assessment of the primary lateral structure, under the imposed lateral displacement and inter-storey drift demands determined in Step F4. The primary gravity structure must be able to accommodate the imposed deformations without exceeding its probable gravity load carrying capacity; i.e. it must be able to "go along for the ride".

Score the primary gravity structure.

If the score for the primary gravity structure is less than that for the primary lateral structure, then it will be the limiting score for the primary structure of the building.

Note:

Steps F5 and F6 are both required.

Step F7

Score any secondary structural systems or critical non-structural items in the building whose failure could lead to a significant life safety hazard. The demands on these elements/items should be typically determined in accordance with Section 8 of NZS 1170.5:2004.

Refer also to Part A and Section C10 for further guidance.

Step F8

Assess the impact of any severe structural weaknesses (SSWs) identified in the building and score these. Refer to Section C1.

Note:

SSWs are specific inelastic mechanisms that have the potential to result in disproportionate loss of gravity support and catastrophic collapse. Refer to Appendix C2G for guidance on assessing and scoring these.

Step F9

The lowest scoring item will be the critical structural weakness (CSW) and its score will become the *%NBS* earthquake rating for the building, in accordance with Section C1.

C2.4 Assessment Procedures – Nonlinear

C2.4.1 Available approaches

There are a number of ways to complete a nonlinear assessment procedure. The main ones outlined in these guidelines are:

- **Nonlinear static pushover procedures**
  - displacement-based assessment, extending the SLaMA (Priestley, 1996; Kam et al., 2013)
  - pseudo-nonlinear using iterative elastic analysis (also commonly known as elastic-plastic analysis for steel plastic design)
  - explicit nonlinear modelling
- **Nonlinear time history procedures.**

Alternative rational assessment procedures based on fundamental principles of engineering, mechanics and dynamics (e.g. using finite element analysis) may be appropriate but are not covered specifically by these guidelines and therefore cannot be used to determine whether or not a building is earthquake prone. If other procedures are to be used for other purposes it is recommended that they be shown to meet the objectives described in Part A and Section C1 and deliver an earthquake rating in the same form as outlined in Part A. Alternative procedures should be reviewed by a suitably qualified and independent engineer experienced in the procedure used, with particular emphasis on how well these aspects have been met.

C2.4.2 Displacement-based procedure and key steps

Displacement-based assessment (DBA) procedures focus on establishing the probable displacement capacity of the primary lateral system (Priestley, 1996; Priestley et al., 2007; Sullivan and Calvi, 2011; Kam et al., 2013; Grant, 2016). DBA utilises displacement spectra which can more readily and directly represent the response of a building in earthquake shaking.

Displacement-based methods use the same methods as force-based assessment to determine the force-displacement response of the structure. However, the expected displacement

demand is based on the structural characteristics (effective stiffness and equivalent viscous damping) at the assessed displacements rather than on initial elastic characteristics. Displacement spectra set for different levels of elastic damping or ductility are used rather than the acceleration spectra reduced for ductility that are used for force-based design.

The displacement-based approach enables degrading strength and the influence of poor hysteretic response characteristics to be incorporated in the analysis. Similarly, the concepts can be extended to seismic retrofit design (Priestley et al., 2007; Kam and Pampanin, 2011).

The key steps of a DBA procedure are explained below. The sequencing of, and interaction between, these steps is described in the flowchart in Figure C2.4. The changes from the force-based procedure (described in Section C2.3.1) are shown in blue.

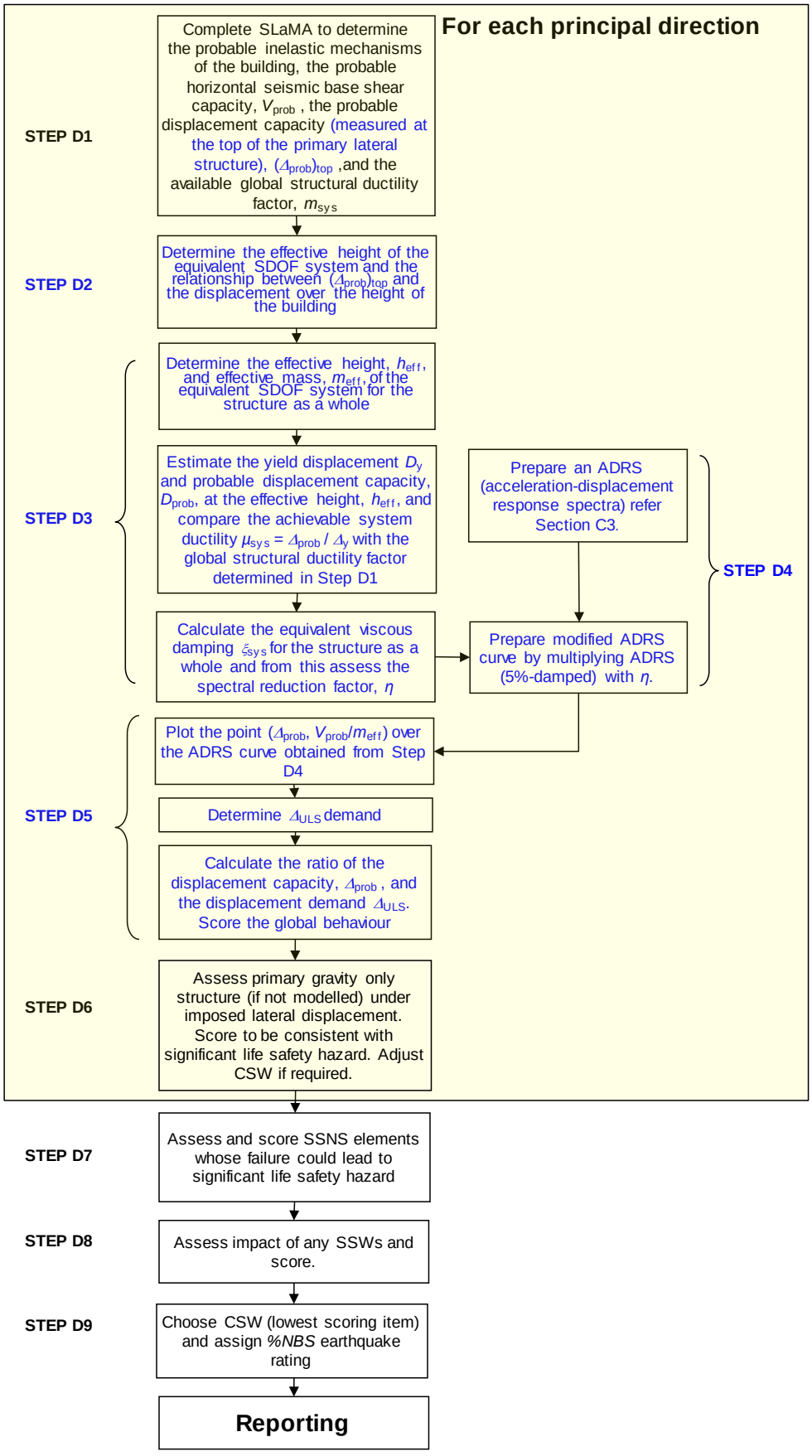


Figure C2.4: Summary of displacement-based assessment procedure (changes from the force-based procedure shown in blue)

Step D1 (similar to Step F1 for the force-based procedure)

Determine the inelastic sub-system mechanisms within the building that are likely to occur during seismic loading. From these calculate the probable horizontal seismic base shear capacity of the structure, V_{prob}, $(\Delta_{\text{prob}})_{\text{top}}$, and $(\Delta_{\text{y}})_{\text{top}}$ using the SLaMA technique outlined in Section C2.2.4 and Appendix C2A.

This step will require the assessment of the member/element/system probable capacities and also the interaction between modes of behaviour, e.g. degrading of the ductile capacity of concrete members in the presence of shear. It provides a simplistic representation of the probable strength capacity of the primary lateral structure and also the probable lateral displacement capacity measured at the top of the primary lateral structure.

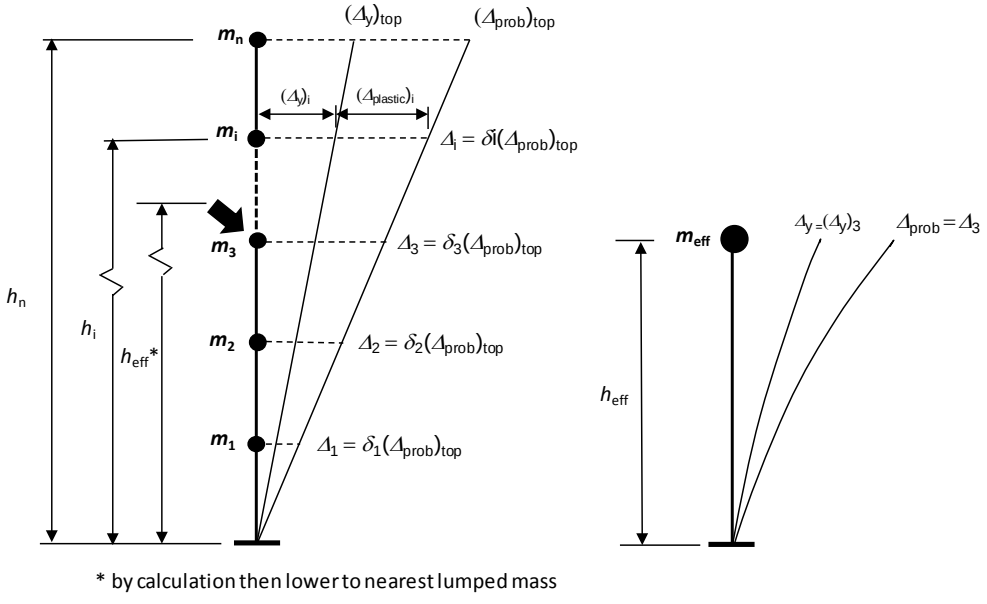
Estimate the global structural ductility factor, μ_{sys} as the ratio $(\Delta_{\text{prob}})_{\text{top}}/(\Delta_{\text{y}})_{\text{top}}$. For multiple sub-systems, $(\Delta_{\text{y}})_{\text{top}}$ can be determined graphically as illustrated in Figure C2.7.

Step D2

Identify the governing inelastic mechanism in the building to determine an appropriate deflection profile over the height of the building.

The global structural ductility capacity is generally governed by the dominant inelastic mechanism that contributes most to the plastic deformation of the system. The choice may not be immediately obvious and several options may need to be trialled.

A generic deflection profile is shown in Figure C2.5(a).



a) Deflection profile **b) Equivalent SDOF system**

Figure C2.5: Deflection profile and equivalent SDOF system for a generic multi-degree of freedom system

The deflection at the top of the building is taken as $(\Delta_{prob})_{top}$ and the deflections at the assumed positions of the lumped masses are taken as $\delta_i(\Delta_{prob})_{top}$ where δ_i represents the normalised deflection profile at level i.

Deflection profiles are indicated below for various well developed inelastic mechanisms. These profiles include both the elastic and plastic proportion of the deflection.

- For a frame structure with beam-sway mechanism:

$$\delta_i = \frac{h_i}{h_n} \text{ for } n \leq 4 \quad \text{…C2.2}$$

$$\delta_i = \frac{4}{3}\frac{h_i}{h_n}.\left(1 - \frac{h_i}{4h_n}\right) \text{ for } n > 4 \quad \text{…C2.3}$$

- For a (moment or braced) frame structure with a potential ground floor column-sway soft-storey mechanism:

$$\delta_i = 0.8 \text{ for } n = 1 \quad \text{…C2.4}$$

$$\delta_i = 0.8 + \frac{h_i}{h_n}(0.2) \text{ for } n > 1 \quad \text{…C2.5}$$

- For cantilevered walls with a flexural mechanism governed by curvature ductility:

$$\delta_i = (\Delta_y + \Delta_{plastic})_i = \frac{\varepsilon_y h_i^2}{L_w}\left(1 - \frac{h_i}{3h_n}\right) + \theta_p h_i \quad \text{…C2.6}$$

  Equation C2.6 above can be used as a lower bound envelope deformation profile, as wall systems generally have a linear deformation profile.

- For a steel braced frame with distributed plasticity mechanism:

$$\delta_i = \frac{h_i}{h_n} \text{ (linear profile)} \quad \text{…C2.7}$$

If a steel braced frame is susceptible to a soft-storey mechanism because the storey sway mechanism is not suppressed by capacity design considerations, then Equations C2.4 and C2.5 above can be used.

Guidance on various deformed shape profiles can also be found in literature (Priestley et al., 2007; Sullivan and Calvi, 2011) and the material sections (Sections C5 to C9) of these guidelines. Further research is required to extend the range of inelastic displacement profiles for other inelastic mechanisms and construction forms but, in the interim, it is considered reasonable to derive a representative deflection profile by reference to those that are available.

Note:

The dominant mechanism may not involve a well-developed ductile mechanism as outlined above and Δ_{prob} achievable for the equivalent SDOF system may not be above the flexural yield displacement. Notwithstanding these issues, it is expected that it will be possible to estimate appropriate values for deflections up the height of the structure based on the estimates of deflection capability at the top of the primary lateral structure determined from SLaMA.

Step D3

Determine the effective height, h_{eff}, and effective mass, m_{eff}, for the equivalent SDOF representation for the primary lateral system as follows:

$$h_{\text{eff}} = \frac{\sum m_{\text{i}} \Delta_{\text{i}} h_{\text{i}}}{\sum m_{\text{i}} \Delta_{\text{i}}} \quad \text{…C2.8}$$

$$m_{\text{eff}} = \frac{(\sum m_{\text{i}} \Delta_{\text{i}})^2}{\sum m_{\text{i}} \Delta_{\text{i}}^2} \quad \text{…C2.9}$$

where:

Δ_{i} = $\delta_{\text{i}}(\Delta_{\text{prob}})_{\text{top}}$ determined from Step D2
m_{i} = lumped mass at level i
δ_{i} = generalised displacement at level i of the deformed shape profile.

These are shown diagrammatically in Figure C2.5.

Estimate the yield displacement, Δ_{y}, and the probable displacement capacity, Δ_{prob}, at the effective height, h_{eff}, from the deflection profile determined in Step D2. In practical terms, this will be the deflection determined for the level immediately below h_{eff}. Refer to Figure C2.5(b) which indicates what is intended.

Note:

For buildings less than or equal to five storeys (i.e. $n \leq 4$), h_{eff} can be assumed (to sufficient accuracy) to be the height to the level below $0.67h_{\text{n}}$ and m_{eff} should be taken as the total mass of the building.

m_{eff} is associated with the effective mass in the principal inelastic deformed shape, which is typically first modal shape. Therefore m_{eff} should be calculated as per Equation C2.9 above and the engineer should not use the modal mass from elastic modal analysis.

Calculate the achievable system ductility, μ_{sys}, as follows:

$$\mu_{\text{sys}} = \frac{\Delta_{\text{prob}}}{\Delta_{\text{y}}} \quad \text{…C2.10}$$

Compare with the estimate obtained from SLaMA in Step D1, rationalise any significant differences and adjust if necessary.

Estimate the equivalent viscous damping, ξ_{sys}, available from the structure as a whole as set out in Appendix C2D.

The equivalent viscous damping of the system may be taken (Priestley et al., 2007) as the weighted average of the effective viscous damping values for each sub-system (identified in Step D1) based on the probable base shear capacity of each, i.e.:

$$\xi_{\text{sys}} = \frac{\sum (V_{\text{base}})_{\text{i}} \xi_{\text{i}}}{\sum (V_{\text{base}})_{\text{i}}} \quad \text{…C2.11}$$

where: $(V_{\text{base}})_{\text{i}}$ and ξ_{i} are respectively the lateral base shear capacity and effective viscous damping for each sub-system i.

Step D4

Prepare the 100%ULS acceleration-displacement response spectrum (ADRS) for 5% damping and modify for the estimated effective viscous damping, ξ_{sys}, as described in Section C3.

Step D5

Plot the point $(\Delta_{\text{prob}}, V_{\text{prob}}/m_{\text{eff}})$ over the modified inelastic ADRS curve.

Determine the Δ_{ULS} demand by extending a line radiating out from the origin of this plot through the point $(\Delta_{\text{prob}}, V_{\text{prob}}/m_{\text{eff}})$ to intersect with the ADRS curve.

The *%NBS* earthquake score for global behaviour is the ratio of Δ_{ULS}, and Δ_{prob}.

Step D5 can be completed numerically using the principle of substitute structure (Priestley et al, 2007) as follows:

- Calculate the effective stiffness and effective period of the equivalent SDOF system

$$k_{\text{eff}} = \frac{V_{\text{prob}}}{\Delta_{\text{cap}}} \qquad \text{C2.12}$$

$$T_{\text{eff}} = 2\pi\sqrt{\frac{m_{\text{eff}}}{g.k_{\text{eff}}}} \qquad \text{C2.13}$$

- Plot T_{eff} line on the ADRS plot and read off the displacement demand Δ_{ULS} at the intersection of the T_{eff} and the modified inelastic ADRS demand curve.

Figure C2.6 shows a system involving several sub-systems and indicates the intent of Step D5.

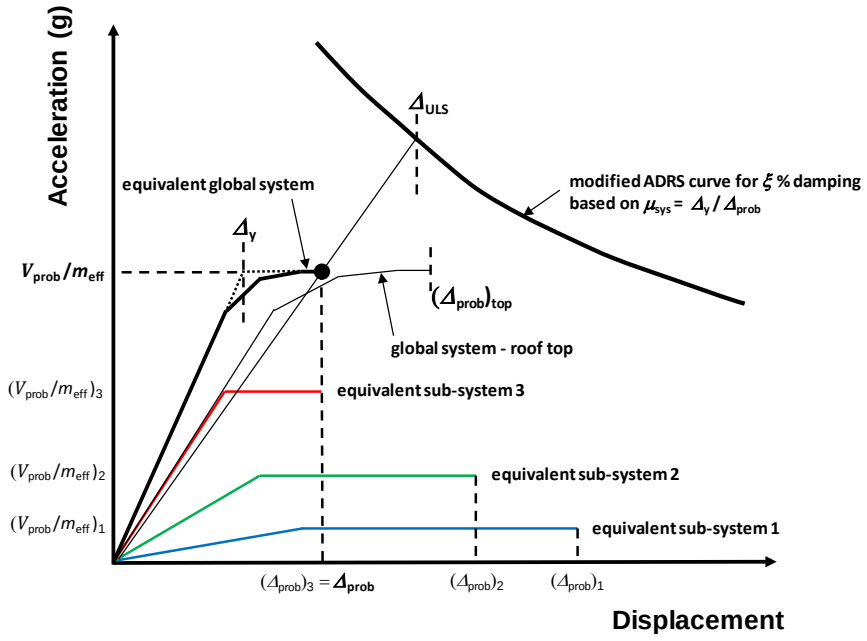


Figure C2.6: Derivation of earthquake score using SLaMA

Note:

Having plotted the capacity and the demand as shown in Figure C2.6 the effect of the various decisions that have been made are readily apparent. For example, if on reflection it is determined that exceeding the capacity of sub-system 3 does not lead to a life safety hazard (say the failure of the bolts in the brace connections of a braced frame) equivalent sub-system 3 can be ignored. This will lower Δ_{prob} but μ_{sys} will increase, bringing the ADRS curve closer to the origin. It is quite possible that the resulting global *%NBS* earthquake score will not be significantly different from when sub-system 3 was included. However, the additional score for the same strength capacity is achieved at higher expected deflections, which may have an impact on secondary structural and non-structural items.

Similarly, if sub-system 3 is retrofitted to achieve a higher displacement capacity the effect is twofold. Firstly the point ($\Delta_{\text{prob}}, V_{\text{prob}}/m_{\text{eff}}$) moves to the right and secondly the ADRS curve moves to the left with the higher μ_{sys}. The effect can be significant. It is also apparent that there may be little point in improving the displacement capacity of sub-system 3 beyond that available from sub-system 2 which may then govern.

Clearly, this representation provides an appreciation of the effect of decisions that may not be readily available from the results of more detailed analyses.

Also score the calculated displacements/drifts against the design limits given for the ULS in NZS 1170.5:2004. Consider carefully if the design displacement/drift limits are appropriate for the particular building and, if not, adjust the score.

Note:

For some buildings the deformation limits provided for the ULS in NZS 1170.5:2004 may be too severe to constitute a significant life safety hazard. For example, the 2.5% drift limit for a portal frame in an industrial building where the axial loads in the portal leg are low is unlikely to be of concern from the perspective of a significant life safety hazard.

The lowest score from consideration of the strength capacity of the primary structure and the deformation limits will provide the score for the global capacity of the primary lateral structure.

The effects of torsion should now be considered as outlined in Section C2.9.3 and Appendix C2F.

Steps D6 - D9

These steps are identical to Steps F6 through to F9 of the force-based procedure provided in Section C2.3.1.

C2.4.3 Nonlinear pushover procedure

This assessment procedure uses a nonlinear static pushover analysis (NLSPA) to determine the probable global displacement capacity of the building. This is then compared with the demand.

NLSPA (described in detail in Section C2.8.2) is generally applicable for the assessment of low to medium rise regular buildings, where the response is dominated by the fundamental (first) mode of vibration. It is less suitable for taller, slender or irregular buildings, where multiple vibration modes affect the behaviour. It is recommended that elastic modal response spectrum analysis is carried out to complete the NLSPA if dynamic response and higher modes are considered to be significant (refer to Section C2.8.2.4).

When using the NLSPA the lateral seismic forces acting on the frame are gradually increased until a mechanism forms. The behaviour of the structure is in the elastic range until the first plastic hinge forms, and then the post-elastic deformations at the plastic hinges need to be taken into account. The number of plastic hinges forming increases with an increase in lateral force until a mechanism develops, giving the probable lateral force capacity.

The nonlinear capacity curve can be generated by nonlinear modelling of the building structure, or a pseudo-nonlinear analysis using iterative elastic analyses, or a simplified method such as SLaMA. Refer to Section C2.6 for further guidance on the various analysis techniques available to generate the nonlinear pushover capacity curve.

The flow chart in Figure C2.8 summarises the generic assessment procedure using nonlinear static pushover analysis. The steps are similar to those for the displacement-based procedure described in Section C2.4.2 (changes from the force-based procedure described in Section C2.3.1 are in blue, and from the general displacement-based procedure are in red). Figure C2.7 illustrates the use of acceleration-displacement response spectrum (ADRS) with nonlinear pushover analysis to determine the *%NBS* earthquake score from the performance point ($\Delta_{\text{prob}}, V_{\text{prob}}$).

There are various ways to estimate Δ_y from the graphical representation. In Figure C2.7 the approach has been to approximate (by eye) the areas between the pushover curve and the assumed stylised initial stiffness (with cracked stiffness) and post mechanism stiffness (along the plateau) representations.

Note:

The theoretical basis for this approach used is the equivalent linearisation approach for capacity spectrum assessment (ATC 40, 1996) and the substitute structure approach used in direct displacement-based design (DDBD) (Priestley et al., 2007).

Other approaches such as the coefficient methods presented in FEMA 440 (2005) and ASCE 41-13 (2014) are available but the work has not been done to correlate them to the objectives of these guidelines. Therefore, they are not specifically covered by these guidelines.

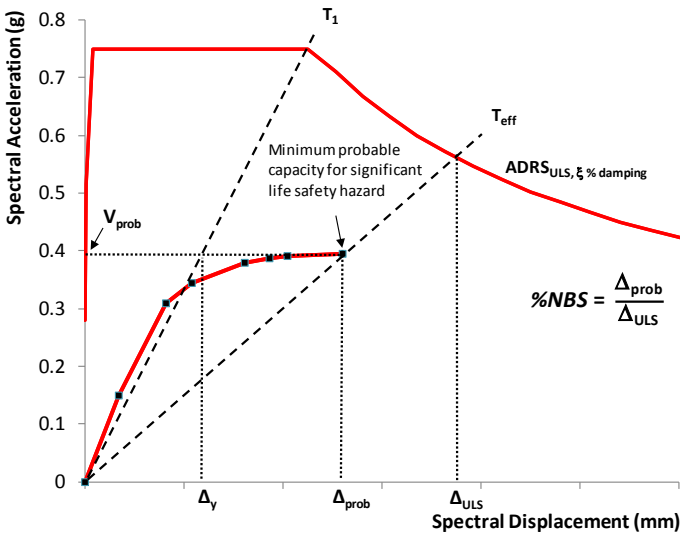


Figure C2.7: Nonlinear static pushover assessment using the ADRS plot

C2.4.4 Nonlinear time history procedure

This procedure uses a series of nonlinear time history analyses (NLTHAs) to evaluate the level of earthquake motion that generates actions in the building less than or equal to those necessary to generate the probable deformation capacity in elements/members and/or the probable global deformation capacity of the building.

Refer to Section C2.8.3 and Appendix C2C for further guidance on NLTHA.

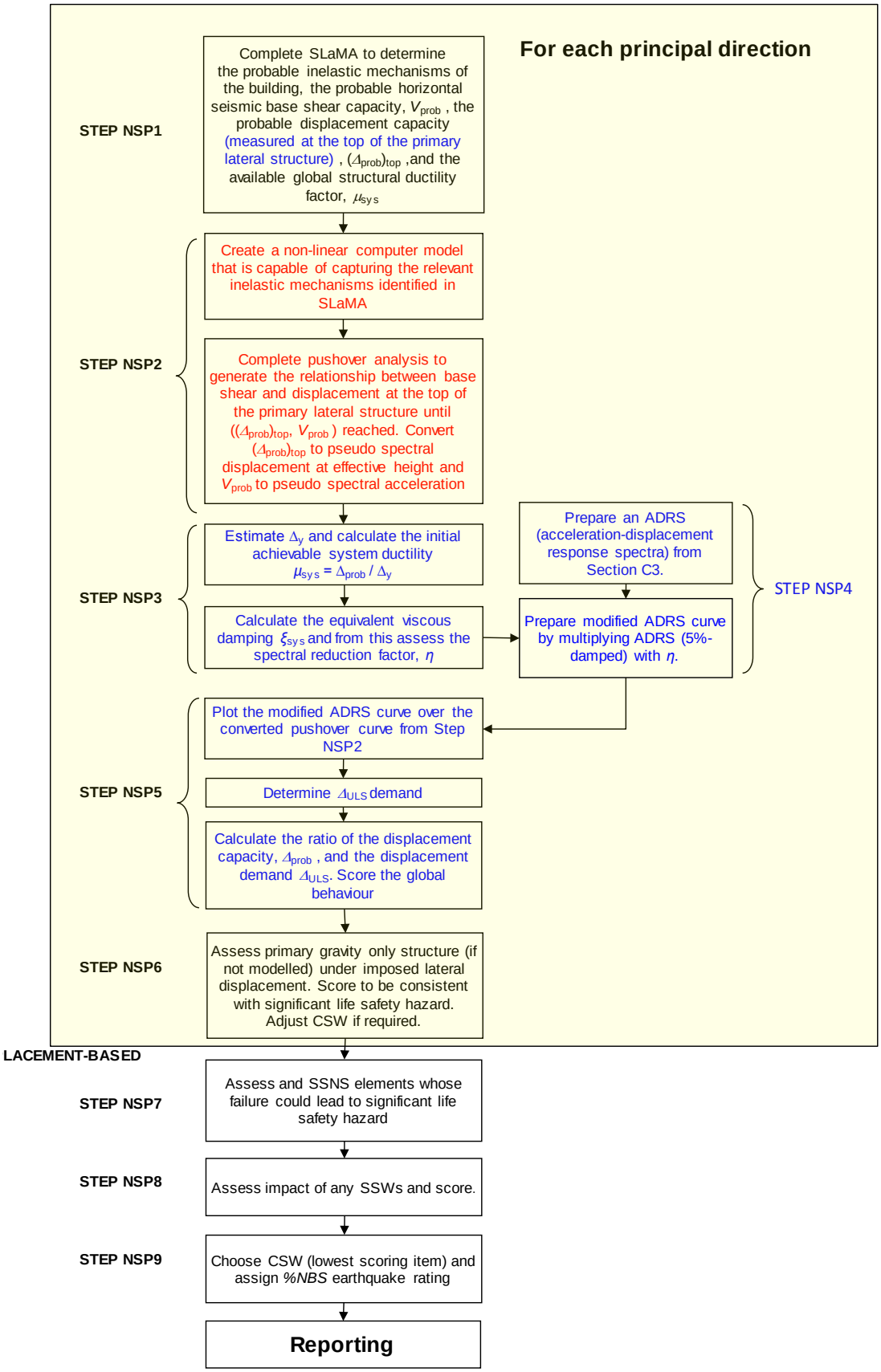


Figure C2.8: Assessment procedure using nonlinear static pushover analysis (changes from the force-based procedure are shown in in blue, and from the general displacement-based procedure in red)

C2.5 Assessment Procedures – Specific Issues

C2.5.1 Strength reduction factors

The probable capacities determined by these guidelines are typically calculated setting the strength reduction factor, ϕ, equal to 1.0. This is in addition to adopting probable material strengths.

For elements/members that are expected to behave in a particularly brittle fashion once the probable capacity is exceeded, the calculated probable capacities should be factored down to provide a margin against the brittle behaviour. This is particularly important when the brittle behaviour is intended to be avoided by a hierarchical load limiting approach (e.g. where a full ductile mechanism is intended to limit actions on brittle elements/ members).

The materials sections (Sections C5 to C9) build this factor into the assessment of probable capacity. In such cases, this is noted in the appropriate section (e.g. shear in reinforced concrete members or ground anchors failing in tension). The material sections also provide other specific adjustments to the reduction factor if it is considered that the calculated probable capacities are subject to high uncertainties and the consequence of exceeding these is significant.

Note:

Care needs to be taken when extracting probable capacities from other sources that capacities reflecting brittle behaviour are appropriately factored down to provide the margins intended in the other provisions of these guidelines.

C2.5.2 Characterising earthquake demand

The earthquake demand is determined in accordance with Section C3.

C2.5.3 Primary gravity structure

While seismic assessment should always include adequate consideration of the full primary structure, a high level of attention should be placed on the gravity system and on how well protected it is by the primary lateral system.

Note:

In the past, many engineers have expended a significant amount of effort in assessing the ability of the primary lateral system to resist the required lateral loads while paying, at best, only scant attention to other building elements (Kam and Jury, 2015).

It is clear that collapses are due to a failure of gravity support and catastrophic collapses due to a significant loss of gravity support. Catastrophic collapses typically occur because the primary lateral system has provided inadequate protection to the gravity system, particularly when the lateral and vertical resisting systems are separate and the gravity systems are heavily loaded and lack ductile capability.

While properly detailed and configured primary lateral systems have rarely failed due to a lack of strength capacity, there are numerous instances of gravity failures due to inadequate deformation capacity in the gravity system to sustain the applied deformations.

Often, the applied deformations have significantly exceeded those that would have been estimated during the design process. This can be due to the size of the earthquake but is often due to unexpected behaviour of the lateral system once it goes inelastic, particularly when this also causes torsional behaviour due to unexpected eccentricities.

Engineers need to recognise that earthquake induced deformations are not constrained in reality to levels assumed in design. Care needs to be taken when assessing low ductile gravity systems and adequately reflecting this in the analyses completed - and finally in the scoring of the primary gravity system. Refer also to Appendix C2G regarding the assessment of SSWs.

C2.5.4 Global versus component assessment

C2.5.4.1 Local component and element mechanisms

It is very important to recognise that the determination of member capacity, overall structural capacity and demand are not entirely separable and there is considerable interaction between these.

An obvious example is the need to know the strength of beam and column cross sections before carrying out an inelastic time history analysis or pushover analysis in order to ensure the correct mechanisms are identified. Another example is the need to correctly assess stiffness of members and the structure when doing modal analysis. Initial assumptions of member properties/capacities will have a bearing on the calculation of structural displacement. This in turn will affect the calculated demand on structural elements.

In the face of this, engineers will need to assess the implications carefully before choosing the most appropriate method of analysis (refer to Section C2.6). Considerable judgement will be needed to achieve a credible assessment to match the circumstances and available budget. For example, it may be possible to quickly identify which members/frames/walls are critical and restrict the analysis to those elements.

C2.5.4.2 Critical and non-critical element deficiencies

While the assessment of a building is focused on the members and elements level, it is important to appreciate that the overall building system performance is quantified by the consequence of the members and elements exceeding their probable capacities. It will generally be too conservative to rate a building on the basis of the first member or element exceeding its probable capacity. As set out in Part A, it is a crucial aspect of the assessment process outlined in these guidelines that the CSW identified must relate to behaviour that would lead to a significant life safety hazard as defined in Part A. In this regard it is recommended that that this test be applied whenever scoring the various aspects of the building.

C2.5.5 Horizontal and vertical irregularity

Horizontal and vertical irregularities, when they are present, can have a significant effect on the behaviour of a building. They can lead to the non-uniform development of ductility

demands in a building's primary lateral structure and high displacement demands on the primary gravity structure.

When significant irregularities are present it is vital that the structural analysis models the effects appropriately and that adequate account is taken of the dynamic amplification effects that irregularities can have on the response once yield occurs in the structure.

Therefore, for buildings with identified horizontal and vertical irregularities it is recommended that nonlinear analysis – even if this is limited to SLaMA and DBA – is carried out to quantify the effects.

C2.5.6 Severe structural weaknesses (SSWs)

SSWs are not readily amenable to reliable assessment using the usual methods, which assume the probable capacity of the system can be determined from the probable capacities of the individual elements/members of that system.

Appendix C2G describes the aspects that need to be assessed as SSWs in a DSA and how to assess their capacity.

C2.5.7 Accidental displacement of the centre of mass

The consideration of an "accidental" displacement of the centre of mass for new building design is used to reflect uncertainties and variations in permanent structure densities, distribution of live loads and superimposed dead loads at the time of the earthquake, member/element/system stiffnesses and strengths, spatial shaking differences, etc.

While an argument can be made that some of these effects can be allowed for explicitly in the seismic assessment of existing buildings, it is difficult to determine the contribution that each makes to the design allowance.

The approach taken in these guidelines is that the allowance for accidental displacement of the centre of mass in a seismic assessment should be at the full design level; i.e. $\pm$ 10% of the width of the building at right angles to the assumed direction of loading.

Note:

The intent of this guidance requirement is not to require mindless assessment at multiple locations of the centre of mass but to note that, in some circumstances (e.g. when the primary lateral system in one direction is on a single line), there may be little resistance to the effects of these uncertainties and this should be reflected in the assessment of the earthquake rating. For any building with primary lateral structure arranged on at least two appropriately spaced lines in each perpendicular direction it is not expected that this allowance will have any significant effect on the earthquake rating.

C2.5.8 Effects of torsion

Accidental eccentricity, plan irregularity and other irregularities can result in torsional amplification, both in the elastic and nonlinear range of behaviour. The effects of torsion should be considered in the assessment using the following analysis techniques.

The torsional amplification due to plan and structural irregularities is additive to the accidental mass eccentricity (refer to Section C2.5.7). The accidental mass eccentricity should always be applied irrespective of the analysis procedures used to considered torsional effects.

Force-based elastic analysis procedure

- Amplify the seismic force to account for accidental torsion and torsional moment. This can be done using the procedure set out in NZS 1170.5:2004, and
- Amplify the seismic displacement response with the amplification factor as per NZS 1170.5:2004. The amplification factor, χ, is the ratio of the maximum displacement at any point on the level x diaphragm to the average displacement:

$$\chi = \delta_{\max}/\delta_{\text{average}} \quad \ldots\text{C2.14}$$

Torsional amplification does need not to be considered if the building is considered torsionally insensitive as per NZS 1170.5:2004.

An alternative approach is the ASCE 41-13 (2014) procedure for torsional amplification multiplier $A\chi$:

$$A\chi = \left(\frac{\chi}{1.2}\right)^2 \leq 3.0 \quad \ldots\text{C2.15}$$

Nonlinear assessment procedures

- Undertake an inelastic torsional assessment as per Appendix C2F, **or**
- Model the accidental torsion effect explicitly by shifting the centre of mass on the diaphragm and running multiple sensitivity analyses (refer to Appendix C2F), **or**
- Apply a blanket torsional amplification factor to NLSPA results to account for accidental torsion. The factor can be derived based on sensitivity analyses as above or on engineering judgement. It is expected the torsional amplification for a NLSPA for a torsionally sensitive building will be in the order of 1.1 to 1.25.

C2.5.9 Concurrent/bi-axial effects

The procedures outlined in Section C2.4 imply that the assessment procedure can be conducted for each principal direction separately to understand the governing inelastic mechanism and probable capacity of the systems. This is similar to the NZS 1170.5:2004 approach for ductile and limited ductile systems. By relaxing the concurrent effects in the analysis, a simpler assessment process can be achieved.

Concurrency effects should be considered for elements and systems where bi-axial effects would significantly change their response and performance.

For example, corner columns and their foundations which form part of two-way moment or braced frames should be assessed for bi-axial effects. This includes assessing the capacity-to-demand ratio from bidirectional or 45 degree diagonal loadings.

Sections C5 to C9 provide further guidance on elements and components that should be assessed for bidirectional effects. If concurrency actions are to be considered, the

NZS 1170.5:2004 provisions for elastic and nominally ductile responding structure should be used (100%X + 30%Y; 30%Y + 100%Y).

For NLTHA, the concurrency effect is modelled explicitly in the analysis by having pairs of horizontal input ground motions and bi-axial properties for elements and components that are affected by concurrency effects (refer to Section C2.8.3 for more on NLTHA).

C2.5.10 Higher mode effects

Higher mode effects can be assumed to be influential in structures where:

- the building's fundamental period exceeds approximately one second, and
- shear in any one storey, calculated from a modal analysis considering sufficient modes to achieve at least 90% mass participation, exceeds 130% of the corresponding storey shear resulting from a second analysis considering only the first mode participation.

Higher modes are not likely to be significant if 60% or more of the mass participates in the first mode in a particular direction.

For structures where higher modes are influential it is recommended that either linear or nonlinear dynamic analyses are carried out in conjunction with other analyses. For example, modal response spectrum analysis (MRSA) can be carried out in conjunction with SLaMA to give a better insight to the building's seismic performance.

Refer to Section C2.8.2 for information about the consideration of high modes for nonlinear static analyses.

For cantilevered reinforced concrete walls responding in a flexural ductile mechanism and for reinforced concrete columns in ductile moment resisting frames, it has been recognised that shear force demand up the building can be amplified by higher mode effects. A shear amplification factor, ω_v, should be applied as follows when considering these systems:

- If there is shear force demand in cantilevered shear walls responding in a flexural ductile mechanism ($\mu \geq 3, S_p = 0.7$) and the building has more than six storeys:

$$\omega_v = 1.3 + \frac{n_t}{30} \leq 1.8 \qquad \text{…C2.16}$$

  where:

  n_t = number of storeys.

- If there is column shear demand in a reinforced concrete moment resisting frame responding in a flexural ductile mechanism ($\mu \geq 3, S_p = 0.7$) and the building has more than six storeys:

$$\omega_v = 1.3 \qquad \text{…C2.17}$$

Note:

Column moment dynamic amplification (as per NZS 3101.2006 Appendix D) is not specifically required.

C2.5.11 Mixed ductility systems

Existing buildings generally have a mixed lateral load resisting system. In the same building there can be two or more structural systems providing lateral bracing but each with its own lateral stiffness, strength and deformation capacities.

When more than one lateral load mechanism is present or when there are components of varying strengths and stiffness, a displacement-based approach is considered essential to ensure that displacement compatibility is achieved and the global capacity is not overstated.

In general, mixed ductility and strength systems cannot be modelled adequately with only elastic analysis. The results can also be misleading, as the stiffer elements generally "attract" more loads while the displacement and ductility demands may be concentrated at the more flexible elements.

Figure C2.9 illustrates this effect: the deformation capacities of elements W3 and W4 cannot be utilised when considering the global deformation capacity as they both exceed the deformation capacity of element W7.

The global system displacement capacity, $\Delta_{cap,}$ and global ductility factor, μ_{sys}, can be determined by considering the sums of the force-displacement capacity curves, as also illustrated in Figure C2.9.

Note:

A common mistake is to assess a building with different deformation capacities allocated to different elements, without considering issues of displacement compatibility.

For example, in the case of a building containing a stiff masonry wall and flexible portal steel frames, the masonry wall would need to exceed its displacement capacity in order to yield the steel frame. Even if the initial stiffness of the two elements are relatively close together, there are still significant issues if one element yields or reaches its deformation capacity prematurely before the other element generates sufficient deformation to justify the higher global structural ductility factor.

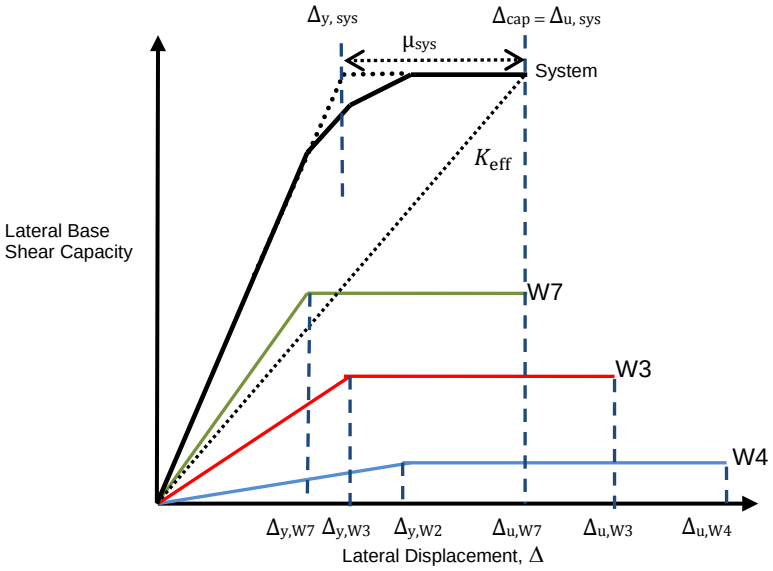


Figure C2.9: Displacement compatibility and mixed ductility

C2.5.12 Damping

The consideration of inherent damping and energy dissipation (hysteretic damping) varies depending on the assessment procedures and associated analysis techniques used:

- For DBA and nonlinear static procedures, the assessment of the equivalent viscous damping, ξ_{sys}, is important as the results are quite sensitive to the choices made.
- For force-based assessments, the capability for energy dissipation is captured within the structural ductility factor, μ, that is assumed and the inherent ductility is included in the assessment of the demand (typically 5%).
- For NLTHA, hysteretic damping and supplementary damping must be explicitly considered and modelled in the analysis.

Refer to Appendix C2D for detailed guidance on the assessment of ξ_{sys}.

C2.5.13 Secondary structural and non-structural (SSNS) elements

SSNS elements should be analysed as individual elements and assessed for imposed deformations in accordance with Section C10.

C2.6 Choosing Suitable Analysis Techniques

C2.6.1 Key principles

Assessing the seismic capacity of an existing building is fundamentally different from designing a new building for seismic actions. Seismic assessment requires a clear understanding and reliable evaluation of the existing load paths, the probable inelastic deformation mechanisms, the probable "collapse mechanism", and the available ductility/displacement capacity of the structure.

This relies on an understanding of the hierarchy of strength and sequence of failure of a structure in an earthquake, which can be gained by undertaking simplified hand calculations such as SLaMA (described in Section C2.2.4 and Appendix C2A). The comparison of capacities of various mechanisms (e.g. flexural versus shear) and within connected elements (e.g. wall to foundation) generally provides an indication of the hierarchy of strength and the likely post-elastic behaviour of a building in a severe earthquake.

Some considerations when selecting the most appropriate analysis techniques for seismic assessment follow:

- **For relatively simple structures** that conform with certain established criteria where complex analysis may not be warranted, the calculated demand and capacity of the building may be modified with appropriate factors based on the identified governing inelastic mechanism; e.g. the use of different allowable μ/S_{p}.
- **For complex/more significant structures** (e.g. with regard to occupancy, consequence of failure), and/or where greater levels of reliability of assessment are sought, the engineer is expected to assess, both qualitatively and quantitatively, the seismic behaviour of the building across a range of seismic shaking and to consider the inelastic behaviour and eventual governing mode(s) of failure, using appropriate methods.

Whatever analysis techniques are used, the consideration of nonlinear behaviour is fundamental for the assessment procedures given in these guidelines. The engineer can mobilise all available inelastic mechanisms – i.e. *%NBS* is not governed by the first element exceeding its strength/strain capacity – provided that local behaviour does not lead to loss of gravity support and/or lateral instability that could reasonably lead to a significant life safety issue.

Some key principles of structural analysis also apply:

- The structural model (computer or hand analysis) should consider and include the appropriate boundary conditions. In particular, the foundation system and the soil-structure flexibility should be modelled if this is deemed to have a significant influence on the behaviour of the building (refer to Section C2.9.2).
- Diaphragm flexibility or the presence of any diaphragm actions should be assessed prior to applying any diaphragm constraint in the model.
- A seismic assessment procedure itself is not tied to a specific analysis technique and the assessment should be informed by a number of analyses. Specific analysis techniques will have their own particular ways of characterising the earthquake demand and specific ways to predict the structure response and ultimate capacity. However, the assessment

procedure used to derive the *%NBS* needs to be holistic and include some aspects of engineering judgement.

- Analysis results, especially those of advanced techniques such as NLTHA, are highly dependent on the input parameters. Each of these is subject to judgement, probabilistic outcome and potential errors (e.g. ground motion records, nonlinear parameters, soil-structure modelling, etc.). There is a fine balance between accuracy, reliability (or precision), cost and complexity in structural analysis, as illustrated in Figure C2.10.

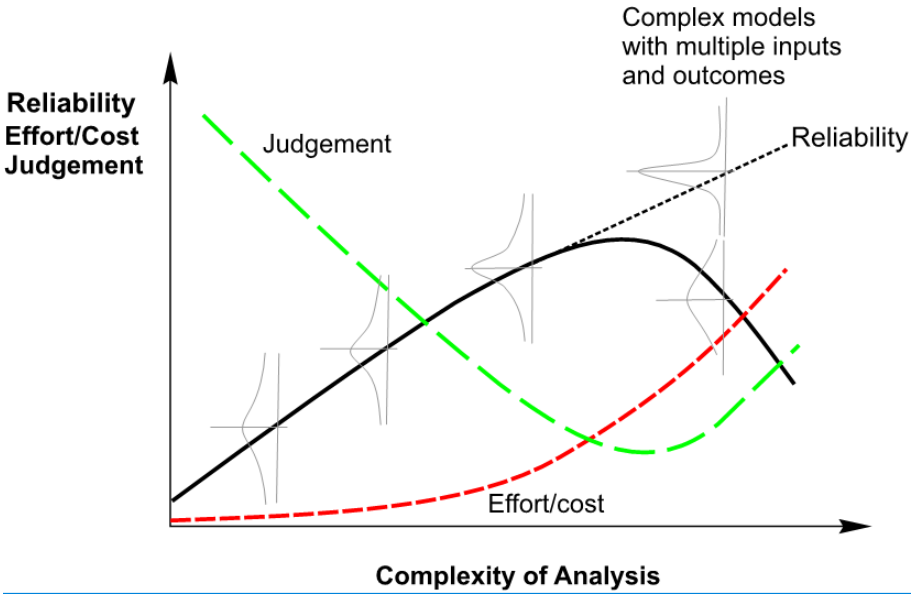


Figure C2.10: Trade-off between reliability, engineering judgement, cost and complexity of structural analysis (modified from Kam and Jury, 2015)

C2.6.2 Recommended techniques, their application and limitations

There are a number of analysis techniques that can be used in the seismic assessment of existing buildings to determine the distribution of member actions due to lateral seismic forces and gravity loading. The primary decision is whether to use a nonlinear analysis technique over the conventional elastic analysis that is more commonly used for designing new buildings.

In general, linear elastic analyses, including equivalent static analysis and modal response spectrum procedures, are applicable when the structure is expected to respond in an elastic or nominally ductile manner at the level of shaking consistent with the *%NBS* earthquake rating or when the nonlinear response and ductility demand is generally uniform throughout the structure (e.g. beam sway of frame structure).

Note:

The level of shaking consistent with the *%NBS* earthquake rating refers to XXX%ULS shaking where XXX is the percentage of new building standard achieved (i.e. XXX*%NBS*). This may require some iteration because whether or not the building remains elastic, or essentially elastic, will depend on the level of shaking that it is assessed against.

The advantage of conventional elastic analysis techniques is that engineers are familiar and comfortable with these and the simplified analysis techniques often allow for analysis inputs to be minimised.

However, the results of linear analyses can be very misleading when applied to buildings with some of the following characteristics:

- highly irregular structural systems, either on plan/horizontal or vertical irregularity
- displacement incompatibility and mixed ductility responses (e.g. ductile steel frames in parallel with unreinforced masonry (URM)), or
- a complex sequence of failure mechanisms and inability of the structure to redistribute the lateral loads without compromising the gravity load path (e.g. premature shear failure of coupling beams which leads to softening of the coupled wall system, so the overall building has higher lateral displacement demand).

The effect of these characteristics may be to concentrate ductility demands to a greater extent than assumed when setting a global ductility factor. As a result the ductility demands in some parts of the building may well exceed the inherent assumptions that have been made.

A more sophisticated analysis does not, by itself, ensure improved accuracy of the building assessment. For example, it may not be possible to model degrading strength characteristics in an explicit nonlinear model; so that the improved precision implied by use of static pushover or time history analysis may be no more reliable than achieved by conducting a simple DBA with SLaMA.

Linear static and dynamic analysis techniques supplemented with SLaMA to consider the governing nonlinear mechanisms are likely to be sufficient for most buildings.

For some complex circumstances where even explicit nonlinear analysis is unable to predict the behaviour well, specific post-processing checks are required to complement the analysis techniques used (e.g. torsional instability of irregular and ductile systems).

Recommendations on the application and limitation of the various analysis techniques are summarised in Table C2.1 and covered in more detail in the following sections.

Table C2.1: Recommended analysis techniques: application and limitations

Analysis techniques	Comments
Elastic analysis	
Equivalent static analysis (linear static)	Useful for preliminary assessment, low-rise assessment and quick calculation • building height not exceeding 30 metres • no significant vertical stiffness or mass irregularly present • no significant torsional stiffness irregularity present, and • orthogonal lateral force-resisting systems present. Either: • elastic responding under 100%ULS shaking, or • low ductility demand/capacity ($\mu \leq 2.0$) under 100%ULS shaking and the following are satisfied: – no in-plane or out-of-plane discontinuities present in primary lateral force-resisting system – no significant weak storey irregularity present – no significant torsional strength irregularity present in any storey.
Modal response spectrum analysis (MRSA) (linear dynamic)	Useful for assessing higher modes and dynamic behaviour prior to nonlinear responses; and useful to assess modern buildings designed using modal response spectrum analysis. This method should not be used for systems with mixed ductility unless appropriate nonlinear assessment (e.g. SLaMA) has been undertaken in addition to the MRSA. Either: • elastic responding under 100%ULS shaking, or • low ductility demand/capacity ($\mu \leq 2.0$)) under 100%ULS shaking and the following are satisfied: – no in-plane or out-of-plane discontinuities present in primary lateral force-resisting system – no significant weak storey irregularity present. While MRSA would consider the elastic torsion responses, it is recommended that inelastic torsion is also assessed via the procedure outlined in Section C2.5.8.
Pseudo-nonlinear using elastic analysis software	Uses elastic analysis but allowance for some non-consequential inelastic behaviour. This procedure is suitable for nonlinear analysis of relatively simple and small structures where the inelastic mechanism can be identified with a high degree of confidence. • Plastic hinge stiffness is removed/reduced artificially – iterative analysis – with plastic moment capacity applied as a moment at locations of plastic hinges. • This method relies on the local inelastic mechanism being well identified and ductile. The method assumes load redistribution with formation of plastic hinges. This condition needs to be confirmed, i.e. ductile hinges are formed. • To be used in conjunction with equivalent static and modal response spectrum analysis.

Analysis techniques	Comments
Nonlinear analysis	
SLaMA (pushover by hand)	SLaMA is recommended to be the first step of any assessment to determine the global inelastic mechanisms. • No significant torsional stiffness irregularity. If torsional irregularity is present then additional inelastic torsional check (Appendix C2F) must be undertaken. • Higher mode effects not critical. Linear dynamic analysis must be used in parallel if higher mode effect is influential. • Uses ADRS or displacement spectra as demand. • DBA is generally not suitable to be used by itself for large complex structures and should be complement by appropriate analysis that would allow the investigation of load redistribution and complex inelastic displacement profiles.
Nonlinear static pushover analysis (NLSPA)	• No significant torsional stiffness irregularity. If torsional irregularity is present then additional inelastic torsional check (Appendix C2F) must be undertaken. • Higher mode effects not critical. Linear dynamic analysis must be used in parallel if higher mode effect is influential. • If higher mode is critical (e.g. mass participation in first translation modes are less than 60%), two or more load/deformed shape vectors should be used for the pushover analysis.
Nonlinear time history analysis (NLTHA) (nonlinear dynamic)	Useful as either a final verification/analysis or in-depth analysis with multiple input ground motions and sensitivity study (research). • Suitable for highly irregular structures and buildings with significant higher mode effects. • Requires suitable nonlinearity models for the identified potential inelastic behaviour. • Preliminary nonlinear assessment using SLaMA, nonlinear pushover or similar analysis is required. • No limitation in terms of structural configuration. • High expertise and independent peer review are required.

C2.6.3 Use of other rational analysis techniques

The use of other alternative analysis techniques (e.g. energy-based methods, finite element model) that are rational and based on sound engineering principles are not precluded but cannot be considered to be specifically covered by these guidelines. If these methods are to be used it is highly recommended that other techniques as outlined in these guidelines are also used for comparison purposes and that alternative techniques (results and method) are peer reviewed by an independent engineer with relevant expertise.

C2.7 Elastic Analysis Techniques

C2.7.1 Equivalent static analysis

Under the equivalent static method, the lateral seismic forces are assumed to be distributed over the building height in accordance with the requirements of Section 6 of NZS 1170.5:2004 and the corresponding internal forces and building displacements are determined using a linear elastic static analysis.

Note:

It will be necessary to carefully consider any vertical irregularities as described in NZS 1170.5:2004. A common issue is the lightweight penthouse or upper storey on a building. This may present a vertical irregularity by virtue of the change in storey mass and/or storey stiffness. The issue of the irregularity may be avoided by considering the upper storey as secondary structure and lumping its mass at the top of the main part of the structure, which is then considered as the primary lateral structure. The actions on the secondary structure are then evaluated in accordance with Section 8 of NZS 1170.5:2004 and scored accordingly.

In this analysis the lateral seismic forces (distributed as above) are varied until the probable strength capacity is reached in a structural element/member that would be considered to be a significant life safety hazard if it were to fail. This is a lower bound estimate of the probable capacity of the primary lateral structure.

Some limited account may be taken of the post-elastic deformation capacity of the structure, to allow use of a system structural (displacement) ductility factor of $\mu > 1.0$. Assessment of this factor should be based on the expected displacement ductility capability of the weaker link components in the structure, but should not be taken greater than $\mu = 2$ unless the engineer is convinced (and can justify) that reliable mechanisms, preferably with a reliable hierarchy of hinge formation, are present.

Note:

Use of equivalent elastic analyses is not generally recommended but may be appropriate in circumstances where there is significant strength capacity to achieve the target *%NBS*.

C2.7.2 Modal response spectrum analysis (MRSA)

Modal response spectrum analysis (MRSA) is a linear dynamic analysis and is commonly used for new building design.

This technique is appropriate for use with structures that are expected to respond elastically to the input seismic action. In addition, MRSA is also suited for structures with well-defined and distributed plastic mechanisms, such as ductile frames, or for assessing recently designed structures (i.e. that meet capacity design or other modern seismic design requirements). It is generally only in such circumstances that an assumed initial level of global ductility will be able to be relied on to limit element/member ductilities to the required limits.

Note:

MRSA is typically used to assess existing multi-storey buildings designed post-1976 in which the original designer may have used MRSA as a basis of design. Engineers should check the appropriate "ductile" mechanism is achievable and that the ductility and inelastic mechanism are consistent with the initial assumption.

Accordingly, it is recommended that a SLaMA is completed before the 3D modelling and MRSA.

MRSA is generally **not appropriate** on its own for mixed-ductility systems. However, it can be a useful method to complement nonlinear static pushover analysis or SLaMA as it enables some consideration of higher mode effects.

The use of MRSA as a nonlinear technique to account for anticipated nonlinear response is generally inappropriate for the assessment of existing structures for the following reasons:

- There is no simple way of assessing the expected inelastic deformations from an MRSA. Common methods, which tend to assume that structure and member ductility levels are identical, are not necessarily correct unless no irregularities are present and the expected behaviour of any nonlinear mechanisms are well understood and ductility demands are well distributed.
- MRSAs underestimate the force levels and local ductility demand associated with higher mode response when member force levels are scaled back to inelastic mechanism strength. This is due to the different between inelastic deformed shapes and the elastic mode shapes used in MRSA (Carr, 1994). Conversely, MRSAs may overestimate torsional response levels for most buildings that respond inelastically.
- MRSAs cannot consider the influence that seismic axial force variations in members may have on their flexural stiffness. This can result in inaccurate estimates of the point at which inelastic action develops in reinforced concrete frame members. The influence of seismic force on member stiffness can be included directly in nonlinear methods.

Accordingly, apart from structural steel and timber structures, and concrete structures that are expected to respond elastically to seismic action, MRSAs should not be used as the sole means of analysis to assess existing structures unless special modifications are made to allow consideration of the above issues.

If MRSA is to be used, the modal response analysis should be carried out in accordance with NZS 1170.5:2004 and good engineering practice (e.g. Carr, 1994). MRSAs are carried out using linearly elastic response spectra, with the resulting forces generally scaled to match the lateral force used in the equivalent static procedure and the components evaluated in the elastic range of strength and serviceability. For any output from the MRSA, the aspect required should be found for each mode before statistical combination methods are applied. The post elastic deformation capacity of the structure is addressed in the same way as for the equivalent static method.

The earthquake demand should be in the form of response spectra derived as required by Section C3.

C2.7.3 Pseudo-nonlinear static pushover analysis

Pseudo-nonlinear static pushover analysis is a technique in which a nonlinear static pushover analysis is completed using a series of progressive elastic analyses.

Using this technique, the equivalent static earthquake forces are increased from zero until the first plastic hinge forms. The lateral seismic force corresponding to the development of the first plastic hinge gives a lower bound to the probable lateral force capacity of the structure, as shown in Figure C2.11. This lower bound estimate will always be less than or equal to the actual lateral force capacity. In reality, moment redistribution will permit higher lateral seismic forces to be resisted while further plastic hinges form until a mechanism develops or a member capacity is exceeded locally.

The lateral load and displacement at the point of the first plastic hinge formation is recorded and plotted on the pushover capacity curve.

The elastic analysis model is updated by releasing the member fixity at the point of the first hinge formation (e.g. the end of the beam or column) and by assigning an external moment (equivalent to the overstrength flexural capacity of the hinge). This will allow any additional moment to redistribute and the overall building softens. The elastic analysis is then re-run with increased force or displacement vectors until the formation of the next plastic hinge.

This sequential analysis is continued until a significant life safety hazard is identified. This marks the end of the pseudo-nonlinear static pushover analysis.

This analysis technique does not automatically track the actual ductility demand at individual hinges and assumes all hinges can sustain some level of ductility. If the hinge is non-ductile in nature (e.g. a shear mechanism on coupling beams) the engineer can elect to release the member fixity without assigning any external moment. This is appropriate as long as the gravity load carrying capacity is not compromised by the local inelastic mechanism.

Note:

This is a manual approach to undertake nonlinear pushover that can be particularly useful for practitioners unfamiliar with software packages capable of nonlinear static pushover analysis.

It can also be used to modify an elastic analysis model for force-based assessment procedures. In particular, the elastic analysis model can be modified to reflect the actual nonlinear response of the building. For example, beams exceeding their flexural capacity can be assigned “hinged” properties to release the moment and allow moment redistribution in the following elastic analysis. Multiple elastic models and iterative analysis to identify various “secondary” inconsequential mechanisms will be required.

When a mechanism can form, the method should yield the same result as the SLaMA.

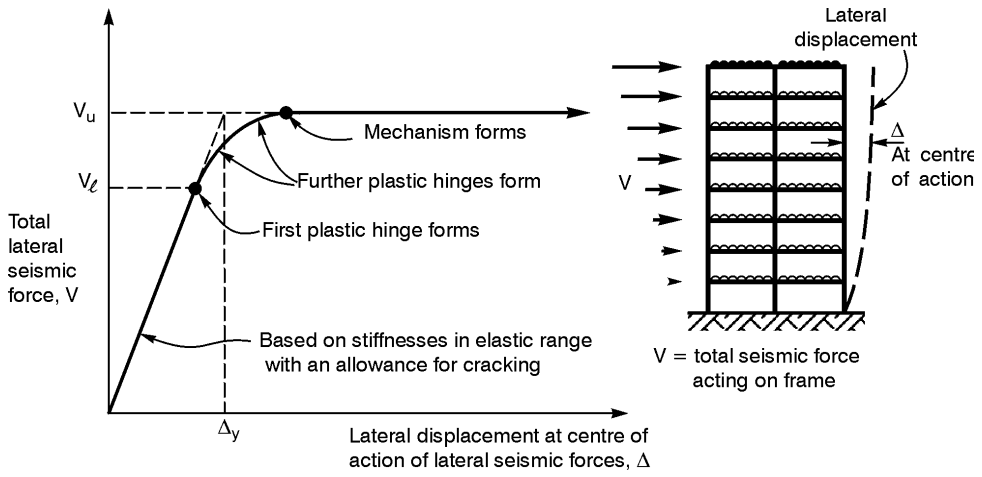


Figure C2.11: Pseudo-nonlinear static pushover analysis with iterative elastic analysis

C2.8 Nonlinear Analysis Techniques

C2.8.1 General

Nonlinear analyses involve significantly more effort to perform and should be approached with specific objectives in mind. Nonlinear analyses require a clear understanding of the probable inelastic behaviour and building response that will depend on both lateral forces and deformation.

The nonlinear models also require the definition of member analytical models that can capture the force-deformation response of these members. In some cases, there are no robust member analytical models available to capture a specific mechanism using commercially available software.

Nonlinear structural analysis models can vary significantly depending on a number of factors that:

- can be controlled (the objective of the analysis, outputs required, the level of structural nonlinearity that is modeled, the level of resources available, and the simplification adopted), and
- cannot be controlled (such as the ability of available analytical models to capture specific mechanisms).

A SLaMA is considered an essential initial stage for any nonlinear modelling to help identify which areas may require more focus and which are unlikely to undergo any inelastic deformation. A SLaMA will also help to provide an appreciation of how the various elements of the building are likely to interact.

A nonlinear analysis technique is appropriate for buildings which contain irregularities and when high levels of nonlinear behaviour are anticipated. If nonlinear pushover analyses are used, the engineer should include appropriate allowances in the analysis for anticipated cyclic strength and stiffness degradation.

Nonlinear time history analyses (NLTHAs), which are dynamic analyses, should be used with care. They require specific expert inputs to account explicitly for such factors as cyclic strength, stiffness degradation, higher modes, and inelastic dynamic behaviour (e.g. torsion).

Note:

The need to model strength and stiffness degradation is intended to pick up the detrimental aspects of these. Where these factors result in a beneficial effect they should be carefully appraised and be subjected to objective peer review.

However, NLTHAs are very complex and their results can be very sensitive to the input parameters, which may be associated with significant variability including within the modelling approach, uncertainties in input ground motions, input hysteresis models, etc. Sensitivity analyses will likely be warranted to “test” the effects of this variability.

Recent research (Krawinkler et al., 2011; Deierlein et al., 2010) has shown that a combination of NLTHA and NLSPA is better for the overall understanding and quantification of a structure’s seismic performance than either technique used by itself.

Therefore, it is advisable to employ a combination of both methods to understand seismic performance and quantify important engineering demand parameters. Both techniques are explained below.

C2.8.2 Nonlinear static pushover analysis (NLSPA)

C2.8.2.1 Description

NLSPA is essentially a refinement of the SLaMA approach which relies on explicit modelling of nonlinear parameters and load distribution within a computer programme.

An incremental inelastic lateral analysis of the structure is carried out under a lateral vector of floor forces, the magnitude of which is gradually increased. The onset of inelastic action of each member can thus be identified and the inelastic deformation of critical members tracked directly. This identifies the structure's probable capacity more reliably than is possible using linear elastic techniques.

NLSPA results in a simplified force-displacement response which can be used with a nonlinear assessment procedure (refer to Section C2.4) to determine *%NBS*.

Most NLSPA computer programs cannot deal with negative structural stiffness (so-called "falling branch behaviour") since the pushover is carried out with incremental force vectors at each analysis step. This means the analysis can terminate just before the lateral displacement capacity is achieved, making it difficult to determine the full structural displacement capacity.

The value of NLSPA is that it allows a detailed inspection of response and is a relatively simple tool for identifying critical regions of a structural system and inelastic mechanism.

The choice of the shape of the lateral force vector will affect the results; possibly including the location and type of inelastic action. Most engineers are familiar with the inverted triangle distribution of floor forces, but a structure developing a soft-storey sway mechanism should have a force vector essentially uniform with height.

As it is difficult to incorporate higher mode effects into NLSPA, in most cases it is still essentially a single mode approach and collapse mechanisms associated with higher modes may be missed.

Note:

As the structural model is being "pushed over", elements/members may experience nonlinear behaviour. The demands on the building (e.g. drifts) and the elements (curvature/rotations) are compared with the probable capacities (e.g. maximum curvature ductility, maximum inter-storey drift) at various steps of the pushover analysis. The governing condition occurs when the probable capacity is exceeded; provided that exceeding this capacity generates a significant life safety hazard.

Note that some elements/members can be allowed to exceed their probable capacity as long as the gravity load carrying capacity is maintained throughout the earthquake.

The base-shear versus centre-of-mass (or roof) displacement – i.e. the pushover capacity curve – is then analysed with a seismic spectral acceleration-displacement demand curve in order to determine the performance points. The assessment using a capacity-spectrum

framework assumes that complex multi-degree-of-freedom models can be simplified into equivalent SDOF systems.

The seismic performance in terms of *%NBS* can be estimated by reducing the percentage of seismic demand such that the response demand parameters do not exceed the acceptable performance criteria.

C2.8.2.2 Modelling and analysis requirements

When carrying out an NLSPA:

- The reference point should be located at the centre of mass at the roof of a building. For buildings with a penthouse, the floor of the penthouse should be regarded as the level of the reference point. The displacement of the reference point in the mathematical model should be determined for the specified lateral loads.
- The relationship between base shear force and lateral displacement of the reference point should be established for reference point displacements ranging between zero and the displacement at which a significant life safety hazard is determined to occur.
- The component gravity loads should be included in the mathematical model for combination with lateral loads as specified in AS/NZS 1170.0:2002. The lateral loads should be applied in both the positive and negative directions.
- The analysis model is discretised to represent the load-deformation response of each member along its length to identify locations of inelastic action. All lateral-force-resisting elements should be included in the model.
- The force-displacement behaviour of all elements can be explicitly included in the model using full backbone curves that include strength degradation and residual strength, if any.
- Alternatively, a simplified analysis can be used. In such an analysis only primary lateral force-resisting elements are modelled, the force-displacement characteristics of such elements are bilinear, and the degrading portion of the backbone curve is not explicitly modelled. Elements not meeting the acceptance criteria but which do not represent a significant life safety hazard can be removed from the mathematical model.

Note:

When using the simplified analysis care should be taken to make sure that the removal of degraded elements from the model does not result in changes in the regularity of the structure that could potentially significantly alter the dynamic response. The simplified analysis does not automatically capture changes in the dynamic characteristics of the structure as yielding and degradation take place.

In order to explicitly evaluate deformation demands on elements that are to be excluded from the model, the engineer may consider including them in the model but with negligible stiffness to obtain deformation demands without significantly affecting the overall response.

C2.8.2.3 Lateral load vector/inelastic deformed shape profile

Lateral loads are applied to the mathematical model in proportion to the distribution of inertia forces in the plane of each floor diaphragm. For all analyses, at least two vertical distributions of lateral load should be applied. One pattern should be selected from each of the following two groups:

- A modal pattern selected from one of the following:
  - A vertical distribution of lateral load proportional to the values of C_{vx} given in Equation C2.18 below. Use of this distribution should be made only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration, and the uniform distribution is also used.

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad \ldots C2.18$$

    where:

    C_{vx} = load distribution factor
    w_i = portion of total building weight W on floor level i
    w_x = portion of total building weight W on floor level x
    h_i = height (in m) from base to floor level i
    h_x = height (in m) from base to floor level x
    k = 2.0 for $T_1 \geq 2.5$ seconds and 1.0 for $T_1 \leq 0.5$ seconds. Linear interpolation is to be used for intermediate values of T_1.

  - A vertical distribution of lateral load proportional to the shape of the fundamental mode in the direction under consideration. Use of this distribution should be used only when more than 75% of the total mass participates in this mode.
  - A vertical distribution of lateral load proportional to the storey shear distribution calculated by combining modal responses from a response spectrum analysis of the building, including sufficient modes to capture at least 90% of the total building mass, and using the appropriate seismic demand spectrum from Section C3. This distribution should be used when the period of the fundamental mode exceeds 1.0 second.
- A second pattern selected from one of the following:
  - A uniform lateral load distribution consisting of lateral forces at each level proportional to the total mass at each level.
  - A lateral load distribution that changes as the structure is displaced. The load distribution is modified from the original load distribution using a procedure that considers the properties of the yielded structure.

Note:

A difficulty with the pushover analysis is that a static representation of the distribution of the seismic forces acting on the frame is required. Conventionally, an inverted triangular distribution of lateral seismic forces up to the height of the frame could be assumed, but this distribution takes no account of higher mode effects or changes in displaced shape post yield.

A sensitivity bound analysis is recommended to assess the differences in lateral force capacity of the frame arising from different distributions of seismic load; for example, uniform up the height.

In lieu of using the uniform distribution to bound the solution, it is possible to use a pushover analysis (Satyarno, 1999; Antoniou and Pinho, 2004) whereby the loading vector is updated at each analysis step to reflect the inelastic deformed shape and the associated redistribution of loading due to stiffness change and progressive damage accumulation in the structure. The changes in the distribution of lateral inertial forces are captured explicitly.

Procedures for developing adaptive load patterns include the use of storey forces proportional to the deflected shape of the structure (Fajfar and Fischinger, 1989), the use of load patterns proportional to the storey shear resistance at each step (Bracci et al., 1997), adaptive load pattern based on modal analysis result (Gupta and Kunnath, 2000) and displacement-based adaptive pushover (Antoniou and Pinho, 2004).

FEMA 440 (2005) and Pinho et al. (2006) provide a good summary of the various pushover techniques of this type. However, at this stage, only several commercially available software packages are capable of such analysis (e.g. Carr, 2007; Seismosoft, 2013; McKenna et al., 2004).

C2.8.2.4 Higher mode effects

Linear dynamic analysis should be used in parallel with NLSPA if higher mode effects are likely to be influential. If the sum of the modal mass participation of the first two translation modes (in each direction) is less than 60% the assessment should include checks of internal actions from MRSA (scaled to the achievable base shear from NLSPA).

The MRSA should have corresponding global ductility and stiffness modifiers to reflect the inelastic mechanism assessed from NLSPA. The MRSA base shear should be scaled to the achievable base shear as determined by NLSPA capacity curve. The MRSA results will be used to assess the displacement and internal actions of elements susceptible to higher mode effects, e.g. upper floors structure in a multi-storey building.

The load/deformed shape vectors from modal analysis can also be used to check the sensitivity of the structure to higher modes. This will be of particular interest for taller structures when higher modes are likely to become important. The lateral load distribution obtained from a modal analysis can provide some allowance for higher modes but will only be completely valid while the structure remains predominantly in the elastic range. Statistical combination of the individual pushover analyses (Goel and Chopra, 2004; Gupta and Kunnath, 2000) is not recommended by these guidelines.

C2.8.3 Nonlinear time history analysis (NLTHA)

NLTHA is a form of dynamic analysis that, in principle, offers the most realistic prediction of seismic response.

The most important value of NLTHA is as an investigative tool to improve the understanding of the overall nonlinear mechanism trend and mean responses. NLTHA offers the ability to track the onset of inelastic response that is obtained from the nonlinear static pushover methods, while at the same time including higher mode effects in a realistic way as well as the manner in which they might vary as the structure becomes nonlinear.

As structural engineers become increasingly familiar with NLTHA and the relevant software becomes more readily available, this technique is expected to become a more popular

analysis technique for structural assessment; particularly for more important structures. Even so, it requires considerable judgement, and the NLTHA results and model should be peer reviewed from a holistic view point by an independent engineer with appropriate expertise.

Refer to Appendix C2C for more about NLTHA and guidance on its use. Further guidance on the use of the ASCE 41-13 (2014) assessment approach using NLTHAs is provided in Section C1.

Note:

A number of guidance documents have been published on NLTHA for performance based seismic design and assessment (e.g. Deierlein et al., 2010; ASCE 41-13, 2014; FEMA 440, 2005). A number of software programs for NLTHA are also commercially available. As is the case for all analyses using proprietary computer programs the user must have a good understanding of methodologies adopted and the inherent limitations of the assumptions that are incorporated.

C2.9 Analysis Techniques – Specific Issues

C2.9.1 Primary, secondary structural and non-structural elements/members

Primary structural elements/members should be checked for earthquake induced forces and for deformations in combination with gravity load effects. Secondary structural elements/members should be checked for deformations imposed by the primary lateral structure in combination with gravity load effects and for seismic loads assuming the element/member is a part in accordance with Section 8 of NZS 1170.5:2004.

Primary structural elements/members that are considered to be part of the primary lateral load resisting system should be modelled in the lateral load analysis. Primary elements/ members that are identified as part of the primary gravity system only can be omitted from the model but should be checked for imposed displacements using post-processor techniques. Judgement will need to be exercised to decide which elements/members should be modelled explicitly.

Refer also to Section C10 for the treatment of SSNS elements.

The displacement capability of SSNS elements may limit the earthquake rating of the building as a whole.

C2.9.2 Soil-structure interaction (SSI) modelling

Close collaboration between structural and geotechnical engineers is needed to clarify the potential soil-structure interaction (SSI) behaviour (also known as soil-structure-foundation-interaction, or SFSI). Of critical consideration for both is the potential impact of geotechnical issues on the building structure in terms of life safety. A critical geotechnical weakness that does not in turn create a significant life safety hazard for the building will not be a potential critical structural weakness for the building and therefore it will not influence the building's earthquake rating.

The degree of SSI analysis and modelling sophistication will vary depending on the potential sensitivity of the superstructure and foundation to the overall SSI system. However, it is expected that a structural engineer assessing the building would consider such aspects as foundation flexibility and whether any step change behaviour is anticipated. If there is any indication that geotechnical issues could influence the behaviour of the building or where there is any doubt about this, a geotechnical engineer should be consulted.

Refer to Section C4 for more information on SSI, its likely influence on the earthquake rating and how to model SSI effects.

Note:

In the past, structural engineers have typically adopted a fixed base model for the interface between the structure and the ground on the basis that, for responses dominated by the first mode, this has been considered to be a conservative assumption. However, foundation flexibility often has a significant effect on the formation of mechanisms and also on the deformation capacity of the building, which can significantly affect the assessment rating determined for the building (e.g. Mylonakis and Gazetas, 2000). Fixed base assumptions

may represent a conservative approach but this should be carefully reviewed before adoption.

C2.9.3 Diaphragm modelling and torsion effects

C2.9.3.1 General approach

Diaphragms are typically suspended floors or roof structures that are relatively thin horizontal structural systems capable of resisting and distributing lateral forces. Diaphragms transfer inertial forces from themselves and connected elements, such as stairs and services connected to them, to the lateral force-resisting structural systems. They may also resist differential in-plane movement of the lateral force-resisting structural systems.

A diaphragm can be classified as flexible or rigid:

- **Flexible diaphragm**: a diaphragm for which the maximum horizontal deformation of the diaphragm along its length is more than twice the average inter-storey drift of the vertical lateral force-resisting elements of the storey immediately below the diaphragm. For diaphragms supported by basement walls, the average inter-storey drift of the storey above the diaphragm should be used. In a URM building it is a diaphragm constructed of timber and/or steel bracing.
- **Rigid diaphragm**: a diaphragm that is not flexible. For assessment purposes, the structural model can assume that the storey mass and storey lateral shear force are concentrated at the centre of mass (including accidental allowance), and a coupled torsion moment is applied at the centre of rigidity.

Figure C2.12 illustrates some of the terminology used for diaphragms and wall displacements.

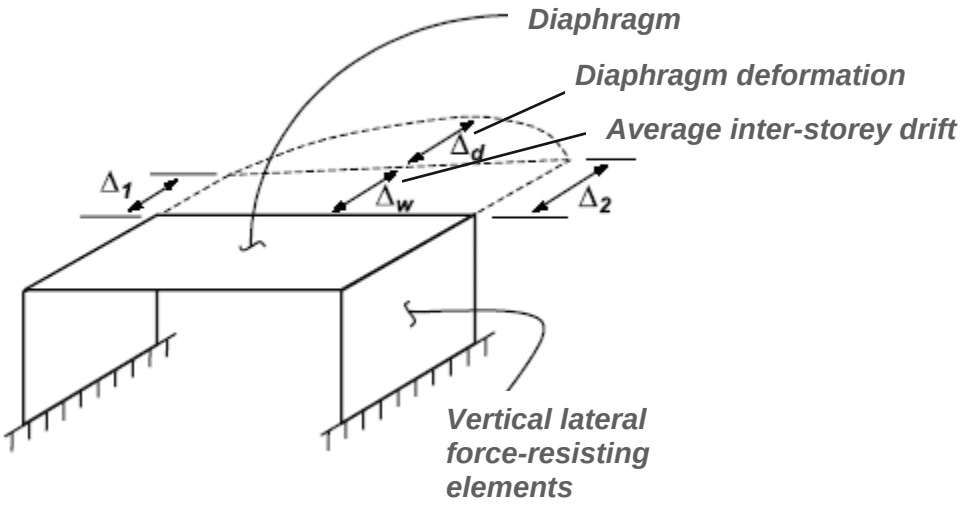


Figure C2.12: Diaphragm and wall displacement terminology

Note:

For the purpose of classifying diaphragms, the inter-storey drift and diaphragm deformations should be calculated using relevant diaphragm inertia loads. The in-plane deflection of the diaphragm should be calculated for an in-plane distribution of lateral

forces consistent with the distribution of mass, and including all in-plane lateral forces associated with offsets in the vertical seismic framing at that diaphragm level.

In modern computer analysis package, the use and definition of flexible diaphragm is straight-forward and is not very demanding computational. Therefore, it is generally recommended to model as flexible diaphragm with the appropriate in-plane stiffness of the diaphragm system.

For flexible diaphragms, each lateral load resisting system can be assessed independently, with seismic mass assigned on the basis of the tributary area. The structural model can assume load distribution by tributary area. The engineer will need to check the displacement compatibility of the overall system and induced transfer forces within the diaphragm to ensure that the diaphragm, even though it is flexible, remains intact.

If the building has flexible diaphragms at each floor level each lateral force-resisting elements in a vertical plane can be assessed independently, with seismic masses assigned on the basis of tributary area. Although the centre of mass should be displaced between the lines of lateral support to reflect the accidental allowance, this will rarely prove to be significant.

For buildings with rigid diaphragms it will be necessary to consider the torsional amplification effect arising from the demand and resistance eccentricities (centre of mass and the location of the centre of stiffness or strength as appropriate).

Please refer to:

- Appendix C2E for more detailed guidance on diaphragm modelling and analysis
- Appendix C2F for more on the torsional amplification effects (for buildings with rigid diaphragms) and ways to assess this.

C2.9.3.2 Influence of infill walls

The potentially detrimental effect on the torsional response of non-uniform loss of infill (due either to in-plane or out-of-plane actions) at one or more storeys or on one or more lateral resistance lines of action should be considered, although any residual capacity of the bounding frames may also be taken when evaluating the lateral and torsional capacity of the building.

Note:

The engineer should recognise that the loss of the infill in all frames at the same storey may not occur and, therefore, an assumption that the residual capacity of the frames alone is available across any storey should not be relied upon.

C2.9.4 P-delta effects

Buildings should be checked for P-delta effects as set out in Section 6.5 of NZS 1170.5:2004.

Note:

P-delta effects are caused by gravity loads acting through the laterally deformed structure and result in increased lateral displacements.

A negative post-yield stiffness may significantly increase inter-storey drift and the displacement demand. Dynamic P-delta effects are introduced to consider this additional drift. The degree by which dynamic P-delta effects increase displacements depends on the:

- ratio α of the negative post-yield stiffness to the effective elastic stiffness
- fundamental period of the building
- structural ductility demand, μ, which is the ratio of the yield displacement to the ultimate displacement
- hysteretic load-deformation relations for each storey
- frequency characteristics of the ground motion, and
- duration of the strong ground motion.

C2.9.5 Seismic pounding

Many existing buildings do not comply with the current requirements for building separation. With insufficient building separation there is a high risk that seismic pounding (building to building impact) will occur, potentially affecting the performance of both structures. However, pounding is not usually an issue for adjacent buildings that are of the same height, have similar configuration and have aligning intermediate floors.

The effects of seismic pounding should be included in the building assessment (refer to Appendix C2B for details).

Note:

Appendix C2B also contains information on how to mitigate the effects of seismic pounding. However, in many cases, resolving pounding issues can be difficult given the different ownership of adjacent buildings.

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Appendix C2A: Simple Lateral Mechanism Analysis (SLaMA)

C2A.1 General

The SLaMA is a simple nonlinear analysis technique that provides an estimate of the global probable capacity of the primary lateral structure of the building as the summation of the probable capacities of the identified individual mechanism/systems. The capacities of the individual mechanisms/systems are typically represented in elasto-plastic form (although post yield stiffness and strength changes can be incorporated) with strength and maximum deformation equal to the assessed probable strength and maximum deformation capacity respectively.

The SLaMA is considered to be a relatively easy way of obtaining an estimate of the nonlinear pushover relationship (strength vs deformation) of reasonably complex structures comprising multiple nonlinear systems of varying ductile capacity. For this reason SLaMA is recommended as the first step in all of the assessment procedures presented in these guidelines.

Although SLaMA is a simplistic process it provides the engineer with the opportunity of observing the contribution that each individual member/element/system has on the capacity of the whole system. Often, the clarity of the simplistic representation will prove more useful in understanding the seismic behaviour of the building than more sophisticated analyses, where the available detail may cloud the individual aspects of the behaviour.

As a SLaMA will not typically allow incorporation of torsional effects these need to be addressed using other techniques.

The steps for completing a SLaMA are outlined in this appendix.

C2A.2 Key Steps

The key steps for a SLaMA are:

Step 1 Assess the structural configuration and load paths to identify key structural elements, potential structural weaknesses (SWs) and severe structural weaknesses (SSWs).

Step 2 Calculate the relevant probable strength and deformation capacities for the individual members.

Step 3 Determine probable inelastic behaviour of elements by comparing probable member capacities and evaluating the hierarchy of strength.

Step 4 Assess the sub-system inelastic mechanisms by extending local to global behaviour.

Step 5 Form a view of the potential governing mechanism for the global building by combining the various individual mechanisms and calculate the probable base shear and global displacement capacity measured at the top of the primary lateral structure. The global displacement capacity will typically be limited to that for the system with the lowest displacement capacity.

Step 6 Determine equivalent SDOF system, seismic demand and *%NBS*.

Step 1 Assess the structural configuration and load paths to identify key structural elements, potential SWs and SSWs

Review the structural drawings and collected as-built structural data thoroughly to understand the structural configuration and lateral load paths.

Separate out the structural members and elements that are part of the primary lateral load resisting system and those that are part of the primary gravity load resisting system. Gain an understanding of when these systems are combined and when they are separate. The primary gravity systems, when not part of the primary lateral system, need to be assessed to ensure they can continue to "go along for the ride" with the primary lateral system.

Step 2 Calculate the probable strength and deformation capacities for individual members

Calculate the relevant probable strength and deformation capacities for individual members with reference to the material sections of these guidelines (Sections C5 to C9) or relevant literature (e.g. EAG, 2012). For example, for members within reinforced concrete moment resisting frames it would be necessary to calculate the flexural and shear capacities for the beams and columns, joint shear capacities and anchorage/lap-splice capacities, if applicable.

Note:

The devil is in the detail! The seismic behaviour of a non-ductile structural system is often governed by the detailing and failure mechanisms not considered by either the original designer or by the code/standard of the day.

In many cases, the absolute strength capacity of the structural member is not necessarily critical. The ability to respond nonlinearly in a ductile manner (i.e. having sufficient deformation and ductility capacity) is more important as it allows load redistribution and mobilisation of other structural elements within the system.

While progress has been made in providing quantitative procedures to calculate the deformation capacity of various non-ductile mechanisms, in many cases the assessment of the achievable local ductility is qualitative in nature and requires significant engineering judgement and understanding of the basis of the detailing requirements in the current standard.

For example, the transverse reinforcement detailing for reinforced concrete columns plays a significant role in their ductile capacity. Some buildings have column tie detailing that does not even satisfy the minimum requirement of transverse steel reinforcement or maximum tie spacing. Engineers need to apply necessary judgement to the quantitative procedures set out in the material sections of these guidelines to estimate the achievable inelastic deformation capacities.

It is often easier and more informative to evaluate the general capacity relationship for a member and then superimpose other deformation limiting issues over the top of the general relationship (e.g. the limiting effect of a reducing shear capacity with increasing ductile flexural behaviour in members within a ductile concrete moment resisting frame). This allows the effect of each aspect on the capacity of the member to be readily observed and has the additional benefit of clearly indicating the effect of undertaking retrofit to address individual aspects.

The capacity of individual URM wall members (piers and spandrels) requires consideration of each of the behavioural modes described in Section C8.

Step 3 Determine probable inelastic behaviour of elements by comparing probable member capacities and evaluating the hierarchy of strength

Determine the potential inelastic behaviour of each element in the critical bracing line by checking the hierarchy of strength of the interconnected members/components.

Figure C2A.1shows an example of the hierarchy of strength assessment of a reinforced concrete beam-column joint. The capacities of the individual elements of beams, columns and joints are assessed separately before an equivalent comparison is made to identify the governing inelastic mechanism within the beam-column joint subassembly.

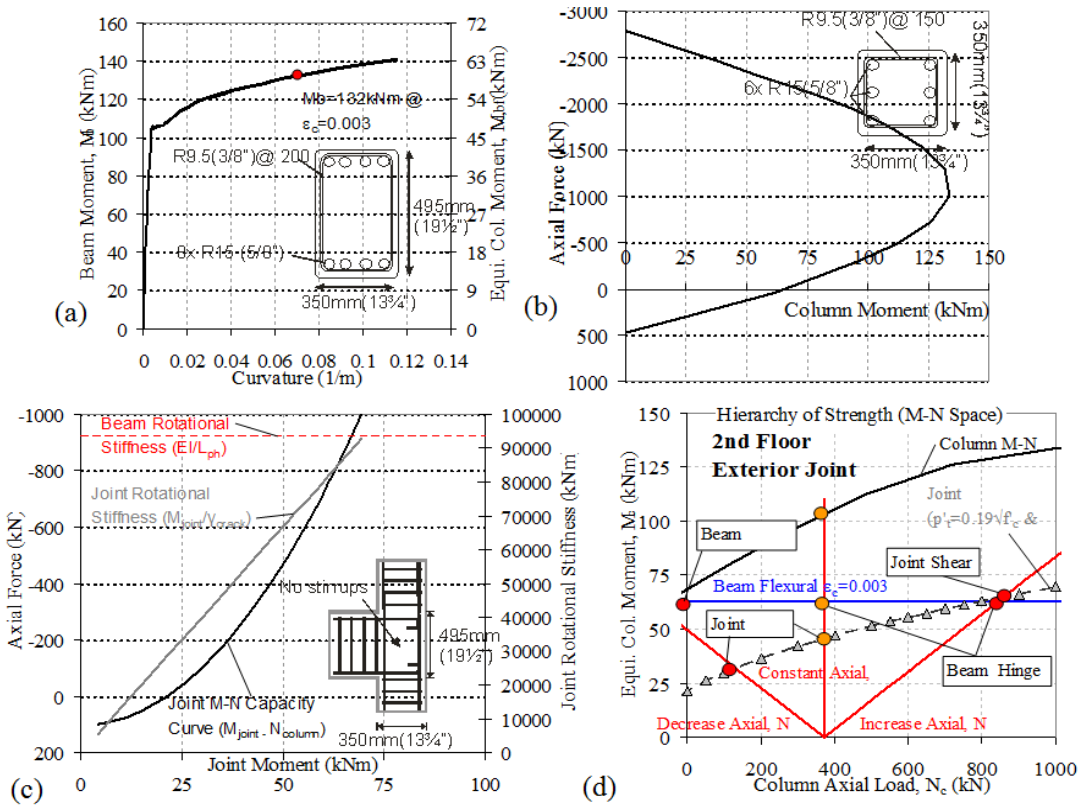


Figure C2A.1: Example of evaluation of element capacity and hierarchy of strength for a non-ductile exterior beam-column joint element as part of a reinforced concrete frame system (Kam, 2010)

For URM wall sub-systems determine the hierarchical behaviour of spandrels and piers. For example, does rocking of the piers between openings occur before the capacity of the spandrels is reached?

Step 4 Assess the sub-system inelastic mechanisms by extending local to global behaviour

Establish the relationship between the local and the global behaviour based on some assumptions of deformed shape profiles and ability to redistribute forces after the formation of "hinges" to determine the limiting mechanism and probable strength and deflection capacity (measured at the top) of each subsystem. The method will depend on the structural configuration and the identified local mechanism:

- For moment resisting frames use the Sway Index (refer to note below) to investigate the likely hierarchy of plastic hinge formation. The inelastic deformed shape profile will depend on the hierarchy of plastic hinge formation as shown in Figure C2A.2.

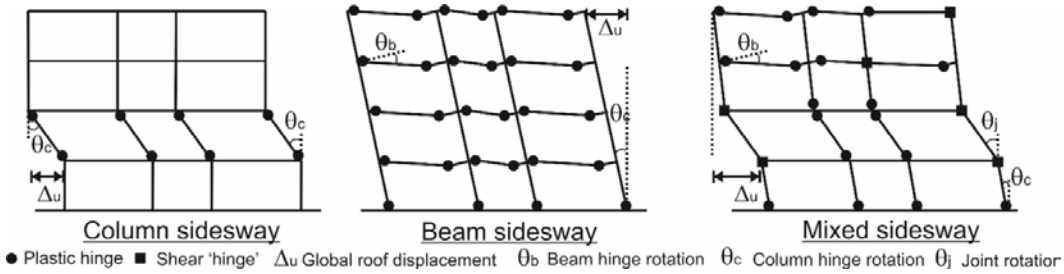


Figure C2A.2: Possible mechanisms of post-elastic deformation of moment resisting frames

Note: Potential inelastic mechanisms in moment frames using Sway Index

At this stage in the analysis, it is important to identify the probable location of post-elastic deformations due to severe earthquake forces and hence to determine the critical mechanism of post-elastic deformation.

This will involve determining whether flexural plastic hinges occur in the beams or the columns at each beam-column joint and/or whether shear failure occurs in the members or joints. The imposed shear forces on members should be those associated with the plastic hinge (flexural) mechanism. The imposed horizontal shear forces on beam-column joint cores should be those associated with the adjacent plastic hinges. The horizontal joint shear force is given conventionally by the sum of the tensile forces in the top and bottom longitudinal beam reinforcement minus the column shear force. Comparisons of these calculated imposed shear forces and the expected shear strengths will determine whether or not shear failures occur before the flexural strengths are reached.

To assess the likely inelastic mechanism the Sway Index, S_{i}, is used. S_{i} compares the overstrength beam flexural capacity to the probable column flexural capacity at the beam-column joint:

$$S_{\mathrm{i}} = \frac{\sum(M_{\mathrm{bl}}+M_{\mathrm{br}})}{\sum(M_{\mathrm{ca}}+M_{\mathrm{cb}})} \qquad \ldots\text{C2A.1}$$

where:

M_{bl}, and M_{br} = beam expected maximum flexural strengths at the left and right of the joint, respectively, at the joint centroid

M_{ca} and M_{cb} = minimum expected column flexural strengths above and below the joint, respectively, at the centroid of the joint.

These are summed for all the joints in the same line at that horizontal level.

The lateral seismic force capacity associated with the critical mechanism of post-elastic deformation can then be calculated.

For a building frame, the critical mechanism is often not simply a beam sidesway mechanism or a column sidesway mechanism (see Figure C2A. above), but is a mixed mechanism involving flexural plastic hinges at some locations combined with shear failures of members and/or joints at other locations.

When $S_i > 1$, column plastic hinges may be expected to form (Sullivan and Calvi, 2011). However, to include the effects of higher modes of vibration, and a possible overestimation of column flexural strength, it is suggested (Priestley, 1996) that column plastic hinges are assumed to form if $S_i > 0.85$. Accordingly, the dynamic magnification factor, ω_v, does not need to be applied in this procedure.

The use of the dynamic magnification factor in the capacity design of new columns is intended to significantly reduce the possibility of column hinge formation. Less conservative measures are appropriate if individual column hinging can be accepted, provided that a full storey column sidesway mechanism does not develop.

- For cantilevered wall systems it can generally be assumed the capacity of the base will govern. For mid to high-rise wall buildings it will be necessary to assess the shear demand at the upper levels as well, as this can be amplified due to higher mode responses and the termination of shear reinforcement with height will not always match the demand requirements to ensure the base governs.
- The various mechanisms for penetrated URM walls can be considered in a similar fashion to that outlined for frames above.

It is important to assess the whole load path as some mechanisms may be limited by another more deficient member/element. For example, the connection capacity often governs the overall lateral load capacity of steel braced frame systems designed pre-1980s. Before the introduction of modern capacity design philosophies for steel structures the connections would have rarely been designed to yield the braces.

Some specific mechanisms can be discounted by inspection based on past experience and understanding of typically observed mechanisms.

Note:

Engineers need to consider whether or not the item/aspect identified as limiting the capacity of a mechanism is likely to present a significant life safety hazard if the capacity of the particular aspect was exceeded (i.e. would this cause loss of gravity load support?). If it is determined that it would not, that aspect is not necessarily material to the assessment

and the probable global capacity of the building. Engineers should either remove it from the analysis or reduce its capacity to its residual value and repeat the analysis of the mechanism.

The potential displacement at every level is:

$$\Delta_i = (\Delta_{elastic})_i + (\Delta_{plastic})_i \quad \text{…C2A.2}$$

where:

$(\Delta_{elastic})_i$ = elastic displacement at level i $\leq (\Delta_y)_i$

$(\Delta_{plastic})_i$ = plastic displacement at level i.

The elastic component of the displacement capacity can be significant for flexible structures and should be accounted for.

The elastic component of the displacement capacity may also be important in regions of low seismicity. For example, for steel portal frames or light timber frames, Δ_{prob} may be less than the yield displacement of the system, indicating the system will remain elastic.

Step 5 Form a view of potential governing mechanism and calculate probable base shear and global displacement capacity

Having identified the mechanisms for the various sub-systems, the next step is to determine how these mechanisms work together to provide the global inelastic mechanism for the building.

It is intended that this step is done by hand and follows the following procedure for a particular considered direction:

- Determine the lowest available deformation capacity of any of the linked sub-systems. This is the available probable global lateral deformation capacity, $(\Delta_{prob})_{top}$.
- Determine the probable base shear capacity of each sub-system at the global deformation capacity determined above and add. The sum is the probable global base shear capacity, V_{prob}.

Note:

It will be apparent that the procedure outlined above requires the deflection of each sub-system to be determined at the same level, the top of the primary lateral structure. It is possible that some sub-systems will not extend to through to this level. In such cases the deflections at the top of the sub-system can be assumed to extend through to the top of the structure.

Figure C2A.4 illustrates a combination of probable force-displacement capacity curves of a dual system (refer to Section C5 for further description of this). Table C2A.1 below illustrates some examples of the derivation of global overturning and lateral base shear capacities for different global systems.

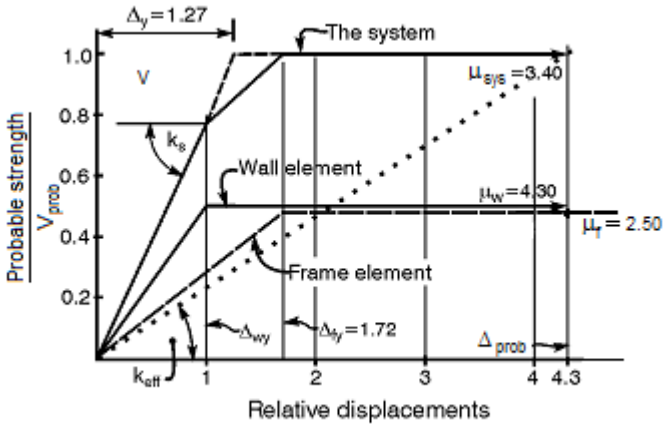


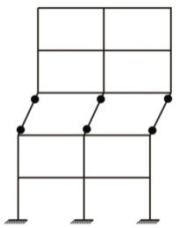
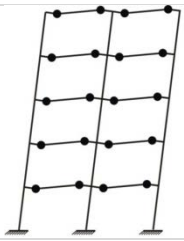
Figure C2A.4: Combination of force-displacement probable capacity curves of a dual system

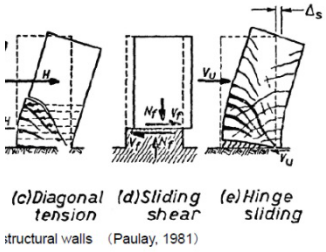
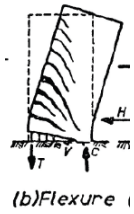
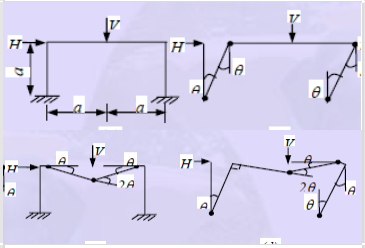
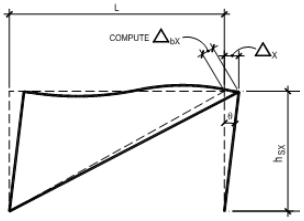
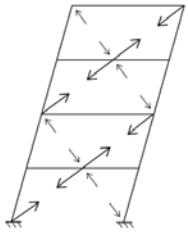
Note:

One of the weaknesses of SLaMA is the potential for overestimating the global capacity by missing the mechanism that has a lower strength and displacement capacity. In some ways, SLaMA has the same underlying principles as the plastic method for steel design, where the lower bound plastic mechanism is used to estimate the upper bound global capacity.

Therefore it is very important that the correct inelastic or "collapse" mechanism is identified so the lateral force capacity is not overestimated. The mechanism that has given the least lateral force capacity is the correct one and must be sought.

Table C2A.1: Calculation of lateral load capacity based on the mechanism

System	Mechanism		Calculation/equation
Frame (steel or concrete)	**Soft storey – column sway** • Concrete: column flexural, lap splice or shear failure • Steel: axial-flexural buckling, web buckling • URM: pier mechanism		$OTM = \sum_{\mathrm{i}} M_{\mathrm{coli}}$ $V_{\mathrm{b}} = \frac{OTM}{h_{\mathrm{eff,col\ sidesway}}}$
	Beam-sway – distributed • Concrete: beam hinging, joint hinging • URM: spandrel mechanism		$OTM = \sum_{\mathrm{i}} M_{\mathrm{coli}} + \left(\sum_{\mathrm{n}} V_{\mathrm{end\ beam.n}}\right) L$ $V_{\mathrm{b}} = \frac{OTM}{h_{\mathrm{eff,beam\ sidesway}}}$

	Mixed mechanism		$OTM = \sum_{\mathrm{i}} M_{\mathrm{coli}} + \left(\sum_{\mathrm{x}} V_{\mathrm{end\ beam.x}}\right) L$ $V_{\mathrm{b}} = \frac{OTM}{h_{\mathrm{eff,mixed\ sidesway}}}$
Wall	**Brittle mechanism** • Global overturning/ instability • Concrete: shear failure , global buckling		$V_{\mathrm{prob}} = 0.85 V_{\mathrm{shear}}$ where: $0.85V_{\mathrm{shear}}$ = probable base shear capacity corresponding to the brittle mechanism
	Ductile/limited ductile mechanism • Concrete: flexural yielding, rocking/uplift • Timber: nail/ply shear		$V_{\mathrm{prob}} = 1.5 \sum M_{\mathrm{wp}} / h_{\mathrm{w}}$ where: h_{w} = height of the walls, which is assumed here to be the same as the height of the building
Steel Portal Frame	**Portal frame mechanisms**		Refer to Section C6 for steel
Braced Frame	**Concentrated brace failure**		Refer to Section C6 for steel
	Distribute brace failure		Refer to Section C6 for steel
URM	**Refer to Section C8 for unreinforced masonry**		

Step 5 provides μ_{sys} , $(\Delta_{\mathrm{y}})_{\mathrm{top}}$ and $(\Delta_{\mathrm{prob}})_{\mathrm{top}}$.

Step 6 Determine equivalent SDOF system, seismic demand and *%NBS*

The procedure for this step is described in Section C2.4.2. It is completed for each direction and can be summarised as follows:

- For each sub-system estimate the relationship between $(\Delta_y)_{top}$ and $(\Delta_{prob})_{top}$ (found from Step 5) and the displacement over the height of the structure from the assumed lateral load distribution with height.
- Estimate the effective height for the equivalent SDOF oscillator for the structure as a whole and determine the simplistic pushover force displacement curve at the effective height. The effective mass, m_{eff}, can be taken as the total mass of the structure. The probable strength is taken as V_{prob}, and the probable displacement capacity, Δ_{prob}, is taken as the lowest value of displacement for all sub-systems calculated at the effective height for the structure as a whole.
- Plot the point (Δ_{prob}, V_{prob}/m_{eff}) over the ADRS curve for 100%ULS shaking and the system viscous damping taken (Priestley et al, 2007) as the weighted average (based on the probable shear capacity) of the sub-system effective viscous damping values, i.e.:

$$\xi_{sys} = \frac{\sum(V_{base})_i \xi_i}{\sum(V_{base})_i} \quad \text{…C2A.3}$$

where $(V_{base})_i$ and ξ_i are the lateral shear capacity and effective viscous damping for each sub-system i.

- Extend a line from the origin through the point Δ_{prob}, V_{prob}/m_{eff} to intersect with the ADRS curve.
- The *%NBS* earthquake score based on SLaMA is the ratio of the spectral displacement at the intersection with the ADRS curve, Δ_{ULS}, and Δ_{prob}.

Appendix C2B: Assessment of Seismic Pounding

C2B.1 Introduction

This appendix provides discussion and guidance on:

- general observations on seismic pounding (Section C2B.2)
- an overall approach to assessment (Section C2B.3)
- qualitative screening for the potential for seismic pounding with significant consequences for a building's seismic performance (Section C2B.4), and
- quantitative assessment for various building configurations (Section C2B.5).

It also lists some alternative mitigation (retrofit) approaches (Section C2B.6).

C2B.2 General Observations

Older buildings have often been built up to property boundary lines, with little or no separation from adjacent buildings. As a result, buildings with inadequate separation may impact on each other or pound during an earthquake. Such impacts will transmit short duration, high amplitude forces to the impacting buildings at any level where pounding occurs. This has the following consequential effects:

- High accelerations within the building in the form of short duration spikes.
- Modification to the dynamic response of the impacting buildings, the pattern and magnitude of inertial demands, and deformations induced on both structures. Response may be amplified or de-amplified and is dependent on the relative dynamic characteristics of the buildings including their relative heights, masses and stiffness, as well as ground conditions that may give rise to soil-structure interaction and the magnitude and direction of travel of the earthquake motions.
- Local degradation of strength and/or stiffness of impacting members.

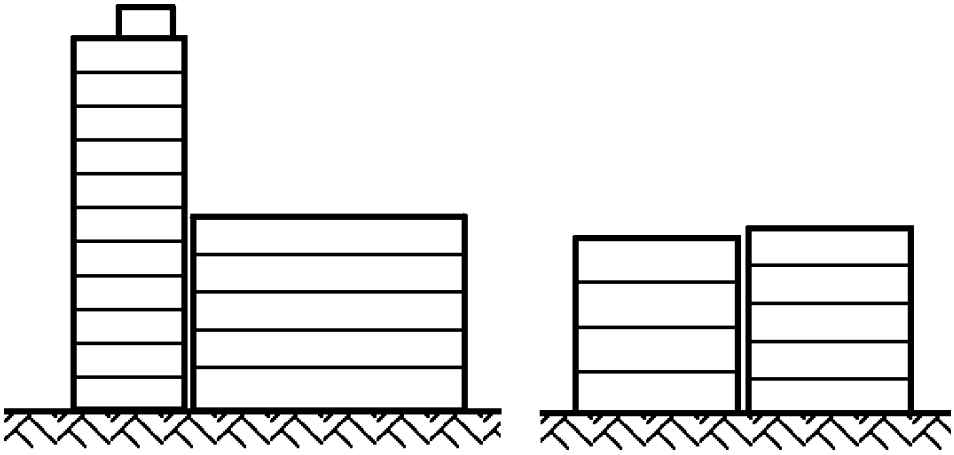
Numerous pounding damage surveys and numerical and analytical pounding studies have been undertaken in recent years, especially after the 1985 Mexico City earthquake (Bertero, 1986) which caused an unusually large number of building failures. In the 2011 Lyttelton (Canterbury) earthquake, seismic pounding was also observed to cause significant damage in a number of URM buildings (Cole et al., 2011).

It is clear that pounding is a complex problem and can occur in a number of circumstances. The results of studies undertaken to date are sensitive to the many parameters related to the building structures (and their numerical modelling) in addition to the prevalent soil conditions and the characteristics and direction of seismic attack. However, based on these studies and evidence from past earthquakes, it is possible to draw the following general conclusions:

- Where buildings are significantly different in height, period and mass, large increases in response from pounding can be expected.
- Differences in height, particularly between neighbouring buildings, can result in significant pounding effects and produce large response increases in the upper part of the taller building (refer to Figure C2B.1(a)). The shears in the impact-side columns for the taller building can be up to 50-70% higher than in the no-pounding case at the levels

immediately above the lower building, and 25-30% at levels higher up. This is because the shorter building acts as a buttress to the taller building. In soft ground conditions where soil-structure interaction and through-soil coupling occurs, the impact-side shears can be enhanced by a further 25-50%.

- For buildings of similar height, mass and stiffness, in most cases the effects of pounding will be limited to some local damage (mostly non-structural and nominally structural), and to higher in-building accelerations in the form of short duration spikes. In such conditions, from a practical viewpoint the effects of pounding on global responses can be considered insignificant.
- Where building floors are at different elevations, the floor slabs of one structure can impact at the mid-storey of the columns of the others, damaging the columns and initiating partial or total collapse (refer to Figure C2B.1(b)). Buildings that are particularly susceptible to such action are those overtopping a shorter neighbouring building whose columns may be impacted at mid-storey by the uppermost level of the shorter building.
- The local high amplitude and short duration accelerations induced by colliding buildings will increase the anchoring requirements for the contents of the buildings as well as for architectural elements.



(a) Buildings of unequal height **(b) Buildings of equal height**

Figure C2B.1: Example of differing floor elevations in adjacent buildings

Note:

When adjacent buildings are of similar height and mass and have matching or similar floor levels, it is not expected that engineers need to account for the effects of pounding, irrespective of the provided separation clearances. The exception is if a building is on the end of a row of buildings without separation (as per item 4 in Table C2B.1).

Similarly, experience from past earthquakes has indicated that solid boundary walls can mitigate the effects of seismic pounding between two buildings with similar stiffness and mass (Anagnostopoulos and Karamaneas, 2008; Kam et al., 2011).

C2B.3 Overall Approach

When pounding is found to be a potential issue it is recommended that the building is first assessed by assuming that pounding does not occur. The next step is to consider any mitigating effects as outlined in Section C2B.4, and then to quantify any remaining issues in accordance with the recommendations in Section C2B.5.

Note:

The quantification of the effects of impact due to pounding is very difficult and is associated with considerable uncertainty. Adjacent buildings of different height and local effects can be scored as outlined in Section C2B.5, but precision should not be assumed.

C2B.4 Screening for Potential for Consequential Seismic Pounding

While seismic pounding between two adjacent buildings in earthquakes is a complex physical phenomenon, it is generally accepted that its effects are more critical for some building configurations than for others. It is also recognised that, in many cases, seismic pounding may only result in localised damage and that the likelihood of pounding is subject to the complex dynamic phasing of two separate structures in an earthquake.

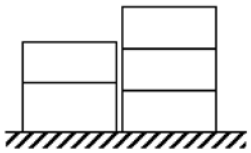
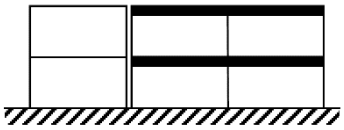
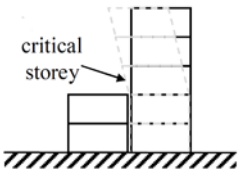
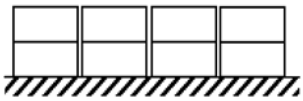
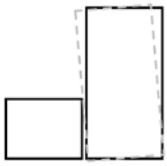
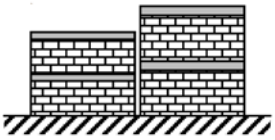
Damage to buildings from seismic pounding can be divided into two categories:

- **local damage** (damage resulting from the magnitude of the force applied during physical contact), and
- **global damage** (damage due to the change in dynamic building properties resulting from momentum transfer during collision). Global damage can increase the lateral response of a stiffer building while reducing the lateral response of a more flexible building, when compared to a standalone structure not affected by pounding.

Local and global damage effects are found to be fundamentally different consequences of collision, with the two categories responding differently to changes in the modelled system.

From observations of earthquake damage, six key building configurations have been identified as vulnerable to seismic pounding (Jeng and Tzeng, 2000; Cole et al., 2011; Kasai et al., 1992). Table C2B.1 describes these configurations and includes notes on their assessment (covered in more detail in the following Section C2B.5).

Table C2B.1: Evaluation of potential pounding vulnerabilities

Scenario		Illustration	Comment	Assessment
1	Column-to-floor		Columns resisting the floor collision are subject to very high shear forces (Karayannis and Favvata, 2005).	Refer scenario 1 in Sections C2B.5.2 and C2B.5.3
2	Floor-to-floor with greatly different masses		The lighter building is more susceptible to collapse. If the lighter building can sustain the imposed drift demand (e.g. a timber building) the pounding effects may be negligible.	Refer scenario 2 in Section C2B.5.2
3	Different building heights		An increase in shear and ductility demands is expected in the taller building in the storey immediately above the top floor of the shorter building.	Refer scenario 3 in Sections C2B.5.2 and C2B.5.3
4	Row of buildings without separation (URM only)		The end building suffers increased damage due to the momentum transfer from the interior buildings, in particular for URM buildings.	For URM only – refer to Section C8 Unreinforced Masonry Buildings *No assessment required for other construction systems*
5	Plan irregularity and pounding		Building configurations can excite torsional response which can lead to amplified local demand.	Depending on the relative stiffness of the buildings, bound the analysis (e.g. assume the long building is being propped by the square building).
6	Pounding of brittle materials, i.e. URM		URM buildings are very susceptible to pounding, which results in amplified lateral demands to the adjacent building. Refer to Cole et al. (2011).	This is generally only critical for the front and rear façade elevations of URM buildings. Refer to Section C8 Unreinforced Masonry Buildings

Note:
Figures are adopted from Cole et al. (2011).

The effects of seismic pounding need to be considered when both of the following criteria apply:

- **Any** of the following conditions exist:
  - adjacent buildings are of different heights and the height difference exceeds two storeys or 20% of the height of the taller building, whichever is the greater, or
  - floor elevations of adjacent buildings differ by more than 20% of the storey height of either building, or

- no boundary reinforced concrete walls are present that would allow transmission and distribution of the localised pounding forces

AND

- Separation between an adjacent building at any height is less than a distance given by:

$$S = \sqrt{\delta_1^2 + \delta_2^2} \quad \text{…C2B.1}$$

where:

δ_1 = estimated lateral deflection of Building 1 relative to ground under the loads used for the assessment

δ_2 = estimated lateral deflection of Building 2 relative to ground under *two-thirds* of the loads used in the assessment.

However, the value of S calculated above does not need to exceed 0.03 times the height of the building at the possible point(s) of impact.

The engineer should calculate S assuming that the building being assessed can be either Building 1 or Building 2.

Note:

The potential or likelihood of pounding needs to be evaluated using calculated drifts for both buildings. The Square Root Sums of Squares (SRSS) combination of structural lateral deflections of both buildings is proposed, as adopted in ASCE 41-13 (2014), to check the adequacy of building separation. This approach has been adopted to account for the low probability of maximum drifts occurring simultaneously in both buildings while they respond completely out of phase. It is not intended for detailed analysis or modelling to be undertaken to determine building drifts; rather, that general estimates are used.

C2B.5 Quantitative Assessment of Pounding Effects

C2B.5.1 Recommended approaches

The effects of pounding effects can be considered using either:

- simplified checks, or
- an approximate approach, or
- a detailed analytical approach.

Note:

Analytical methods have been proposed for assessing the effects of pounding, including time history analyses and elastic response spectrum analyses (Kasai et al., 1990; Cole et al., 2010). However, the use of such approaches may prove impractical for many buildings or may not be within the capability of many design practitioners (Cole et al., 2010).

An alternative simplified approach has been proposed, based on simple factoring of earthquake design forces applicable to the building, to ensure some account of pounding

effects is made. Both moment/shear capacities and P-delta effects need to be considered. A number of studies (Kasai et al., 1990; Jeng and Tzeng, 2000; Carr and Moss, 1994; Karayannis and Favvata, 2005) have shown that column and storey shears in the taller building above the pounding level can be increased by anywhere up to or exceeding 100%. The level of increase is dependent on many factors including initial separation distances and the relative mass and stiffness of the adjacent buildings. A mid-range increase in design shear has been adopted for the simplified approach at this stage.

While it is recognised that this approximate approach is relatively crude, it has the benefit of being easy to apply and does not need the use of, or familiarity with, sophisticated analysis tools. As further research on seismic pounding is undertaken, it is expected that more appropriate and practical means to evaluate and mitigate pounding will become available.

Irrespective of the approach adopted, the *%NBS* score determined for pounding will be based on the %ULS shaking that leads to a significant life safety hazard due to a loss of gravity support (based on probable member/element capacities).

C2B.5.2 Simplified checks

Simplified checks can be performed to estimate the upper and lower bound responses if seismic pounding occurs. Some examples follow. Scenario numbers correspond to Table C2B.1.

Scenario 1 – Misaligned floors and column-to-floor pounding

Assume the columns in collision with the floor have failed in shear. Check if the gravity load path is maintained by the secondary load path (e.g. floor beams or slab cantilevered back to the building or boundary walls exist). If a reliable scenario load path is available (either existing or through seismic retrofit), no further assessment is required.

Scenario 2 – Aligned floors but with mass difference

Assume the stiffer building will "prop" the more flexible building. Assess the stiffer building with 20% or more seismic mass from the adjacent building. If the lighter/less stiff building does not have a rigid diaphragm, the additional seismic inertia to be resisted by the stiffer building can be estimated based on the tributary area.

Scenario 3 – Aligned floors but with building height difference

Carry out an initial assessment of the taller building by assuming its building height is truncated by the shorter building (which would decrease its fundamental period and, therefore, increase its seismic loading). If the shorter building is of concern, assess this against a 20% storey shear from the adjacent building applied at the point of impact.

C2B.5.3 Approximate approach

Scenario 1 – Misaligned floors and column-to-floor pounding

If the floor elevations of adjacent buildings differ and there is potential for mid-storey hammering of each building, the impact-side columns of the building(s) which may be

impacted between storeys should have sufficient strength to resist design actions resulting from imposition of a displacement on the columns, at the point of impact, corresponding to one half of the value of S derived from Equation C2B.1 in Section C2B.4.

The imposed displacements only need to be applied at any one level. However, critical design actions should be derived considering application of the imposed displacements at any level over the building height where impact could occur.

In addition, if the buildings are of unequal heights, in accordance with Section C2B.4 the requirements of Scenario 3 below also apply.

Scenario 3 – Aligned floors but with building height difference

If two buildings are of unequal height but their floor elevations align, the impact-side columns of the taller building should have sufficient strength to resist the following design actions:

- 175% of the column design actions (shear, flexural and axial) occurring in the taller building under the application of the seismic lateral loading in accordance with Section C3, assuming the building is free standing, applied above the height of the shorter building
- 125% of the column design actions occurring in the taller building under the application of the seismic lateral loading in accordance with Section C3, assuming the building is free standing, applied over the height of the shorter building, and
- all other columns remote from the building side suffering impact should have sufficient strength to resist 115% of the column design actions occurring under the application of the seismic lateral loading in accordance with Section C3, assuming the building is free standing, over the full height of the building.

C2B.5.4 Detailed analytical approach

Detailed modelling of the seismic pounding phenomena requires consideration of the transfer of momentum and energy between the buildings as they impact, both in terms of local contact damage and of global building response changes. Possible approaches for a variety of pounding situations and varying levels of model detail are available in e.g. Cole et al., 2011; and Khatiwada et al., 2011.

NLTHA with simplified mass and stiffness and appropriate contact elements appears to be the only appropriate detailed quantitative assessment of pounding between two buildings (Cole et al., 2010).

These guidelines recommend the simplified and approximate checks as outlined in preceding sections in preference to a detailed analytical approach. The limitations of NLTHA and pounding modelling mean this method is not necessarily viable for practitioners.

C2B.6 Mitigation

In some circumstances, rather than carrying out a complex analysis of the seismic pounding phenomenon it may be more cost effective to accept the seismic pounding risks and undertake steps to mitigate its effects.

Retrofit options to mitigate the risk of seismic pounding include:

- tieing adjacent buildings together. This approach may prove practical for a row or block of buildings of similar height and configuration
- providing additional structural members/elements away from the points of impact to compensate for/replace members/elements that may be severely damaged due to impact
- improving individual buildings to reduce displacement or increase resilience to pounding and additional seismic inertia from the adjacent building
- providing robust boundary shear walls to act as buffer elements to protect the rest of the building (Anagnostopoulas and Karamaneas, 2008). The use of collision shear walls would prevent mid-storey impact to columns of adjacent buildings, reducing potential for local damage and partial or total collapse, and/or
- linking adjacent buildings with energy dissipating devices to reduce the severity of pounding and collisions (ULIEGE, 2007).

Appendix C2C: Nonlinear Time History Analysis

C2C.1 General

Nonlinear Time History Analysis (NLTHA) or Response Analysis is a highly specialised analysis technique that provides a real time "snapshot" prediction of the seismic response of a building under earthquake actions. It is particularly important as an investigative tool to improve the understanding of "what happened": i.e. the overall nonlinear mechanism trend and mean responses.

Advanced and sophisticated analyses such as NLTHA are useful in understanding the nonlinear and dynamic behaviour of the building. However, they require significant effort and engineering judgement to ensure the validity of the outputs. While the accuracy may have increased with the use of complex and sophisticated analysis, the uncertainties, precision and reliability remain a function of the level of checking and rigour of the analysis (number of runs, sensitivity analysis and well-defined analysis parameters).

Note:

A number of guidance documents have been published on the use of NLTHA for seismic assessment (e.g. Deierlein et al., 2010; ASCE 41-13, 2014; FEMA 440, 2005; ATC 72, 2010). There are also a number of software programs for NLTHA that are now commercially available.

It is important to recognise that any NLTHA output is only a representation of the building's response to one particular earthquake record and is highly dependent on the ability to model the nonlinear element behaviour adequately. The performance in an actual earthquake is contingent on a number of other variables that may or may not be modelled (NIST, 2013).

Note:

Engineers should resist the temptation to believe that NLTHAs reliably predict the performance of a building in a particular earthquake. The whole assessment approach is based around rating a building's performance against that of a similar new building. Therefore, care should be taken not to overcomplicate a NLTHA in the pursuit of unattainable accuracy; especially if loss of clarity of the behavioural issues in the building is the result.

It is recommended that an NLTHA should not be the sole analysis technique used for a structural assessment but should be supported by the results of simplified approaches. This is for the following reasons:

- Individual results from individual runs are highly dependent on the characteristics of the ground motion and its interaction with the nonlinear characteristics of the building. As such, NLTHA is a poor predictor of the exact performance or the exact magnitude of response for any given earthquake input motion.
- Special care and skill is required to select appropriate modelling approximations. For example, the definition of elastic damping needs careful consideration, as inappropriate definition will result an incorrect estimate of response.

- Typically, the interactions between flexure, shear and axial load are not modelled in NLTHA programs, making it impossible to model the onset of shear failure reliably. Similarly, few NLTHA programs include the influence of axial force in columns on their stiffness. This can influence predictions of onset of inelastic response, and can be critical for structures with brittle failure modes.
- Some NLTHA programs cannot model degrading strength characteristics, and few have special elements representing the strength and degradation characteristics of beam-column joints in concrete or steel structures.
- The refinements of an NLTHA may also be inappropriate when the uncertainty associated with the seismic intensity is considered. The seismic intensity is typically represented by the shape of the response spectrum for the earthquake record but will also be affected by other factors such as ground conditions, site source distance and path, and magnitude and duration of shaking, as discussed below. When NLTHAs are carried out, it is usually necessary to run several analyses with different records representing the design intensity. This is to improve the chance that all potential inelastic mechanisms are identified and appropriately "tested". When it is necessary to determine the actual level of intensity corresponding to a given level of earthquake shaking rather than assessing a pass/fail result for a reference intensity, multiple analyses will be required, scaling the intensity of the records until the required level of shaking is reached.

C2C.2 Input Ground Motions

Where an inelastic time history analysis is carried out, the model representation of the building structure should be subjected to earthquake shaking represented by ground motion time histories in accordance with Section C3. Section C3 refers to Clauses 5.5 and 6.4 of NZS 1170.5:2004.

Note:

Research has shown that consideration of different ground motions is ESSENTIAL to the application of nonlinear response history analyses. The calculated response can be highly sensitive to the characteristics of individual ground motions.

More recent research (Baker and Cornell, 2006; Hancock et al., 2008; Bradley, 2010; Beyer and Bommer, 2007; Kalkan and Chopra, 2010) has indicated that the NZS 1170.5:2004 requirements for input ground motions may need to be updated. These include the minimum number of ground motion records that should be analysed and the method which should be used to assess the results. This is summarised in Table C2C.1.

As this is an area of active research, it is recommended that engineers review the latest literature (e.g. Bradley et al., 2015; Kwong and Chopra, 2015). ASCE 41-13 also provides more up-to-date guidance than available from NZS 1170.5:2004. At this stage, it is recommended that the selection and scaling of input ground motion is independently reviewed.

Table C2C.1: Suggested number of ground motion acceleration history records

Condition	Method of computing results	Number of ground motion records
Far-field (>5 km/3 mi)	Average	Record pairs ≥10
Far-field (>5 km/3 mi)	Maximum	**3 ≤** record pairs ≤ 9
Near-fault (≤5 km/3 mi)	Average	Near-fault record pairs ≥ 5; total number of record pairs ≥ 10
Near-fault (≤5 km/3 mi)	Maximum	**3 ≤** near-fault record pairs ≤ 9

Vertical ground motion should be included in the NLTHA, particularly if the structure has any element or component that is sensitive to the amplification of axial and gravity loadings (such as a cantilevered transfer structure). Similarly, this should be included if the structure's lateral load carrying capacity is largely dependent on the gravity restoring forces (e.g. URM rocking piers).

Note:

Care needs to be taken to ensure that the additional complexity of including the vertical acceleration component is warranted, as each increase in the complexity of the analysis has the potential to cloud the behaviour. In many instances the maximum vertical accelerations have dissipated before the maximum lateral shaking occurs.

The use of record pairs, applied in both directions in the NLTHA, should adequately account for the concurrency effects.

C2C.3 Modelling of Nonlinearity

Inelastic structural element models can be differentiated by the way in which plasticity is distributed through the member cross sections and along its length. For example, Figure C2C.1 shows a comparison of five idealised model types for simulating the inelastic response of beam-columns, ranging from a lump plasticity rotational spring model to a detailed continuum finite element model. All models are empirical as the models are calibrated to experimental results either at the macro level or micro-material level.

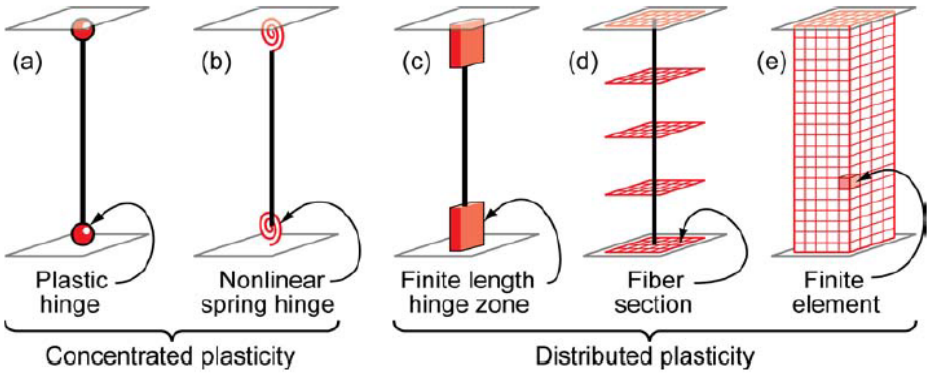


Figure C2C.1: Idealised nonlinearity model (from Deierlein et al., 2010)

It is important to understand the limitations of the modelling type and approximations inherent for each modelling assumption (e.g. whether shear failure is modelled). Engineers should appreciate the trade-off between different nonlinear modelling approaches and apply judgement as appropriate.

In general, lumped plasticity models are recommended for nonlinear analyses of large buildings as these simplify the number of inputs required and can be used to pinpoint the governing local inelastic mechanism. However, effects such as the interaction between axial, flexure, and shear failure in concrete members are difficult to capture using lumped plasticity models. Figures C2C.1 and C2C.2 show an example of a 2D nonlinear lumped-plasticity model for a non-ductile reinforced concrete frame (Kam, 2010).

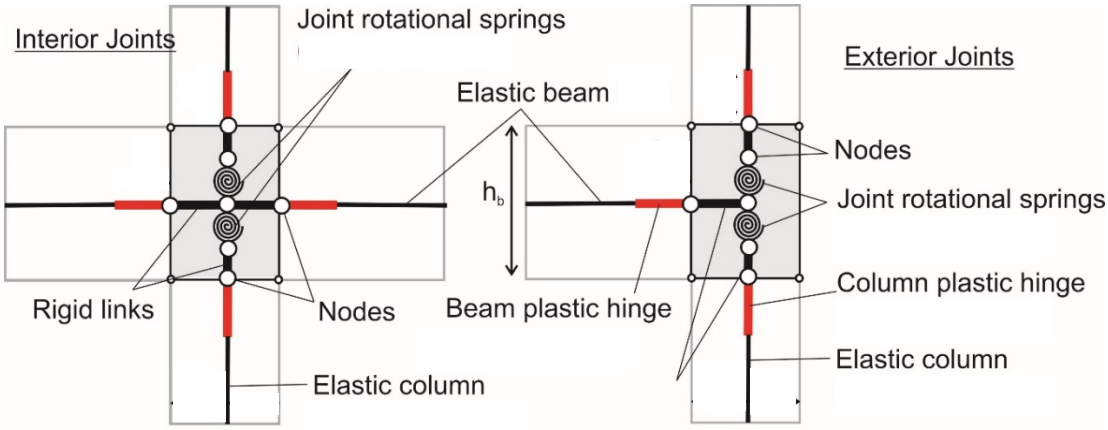


Figure C2C.2: Rotational joint spring model for non-ductile beam-column joint element (adapted from Kam, 2010)

For nonlinear procedures, a connection should be modelled explicitly if the connection is weaker, has less ductility than the connected components, or the flexibility of the connection results in a change in the connection forces or deformations greater than 10% (as found from sensitivity analyses).

There are a number of sources for guidance on the appropriate hysteretic modelling parameters (Carr, 2007; ASCE 41-13, 2014; Deierlein et al., 2010; FEMA 440, 2005).

C2C.4 Nonlinear Hysteretic Model Parameters

Irrespective of whether a concentrated plasticity or distributed plastic modelling approach is adopted, the definition and selection of the parameters for the nonlinear hysteresis model for either lumped plasticity or constitutive fibre/finite element elements is a very important step in nonlinear analysis.

In particular, the selected nonlinear hysteretic parameter should match as closely as possible to empirical data/evidence. This is particularly important for elements with strength and stiffness degradation (e.g. non-ductile reinforced concrete joints or columns) or pinching hysteresis (e.g. debonding and failure of lap splice of reinforcing bars).

Specialist knowledge of the appropriate type of hysteresis model and parameters is required in order to define the nonlinear hysteresis model appropriately. For example, Ibarra et al., 2005 and Pampanin et al., 2003 provide suitable strength and stiffness degrading models for non-ductile beam-column joints.

Based on available supporting literature (e.g. Carr, 2007; Seismosoft, 2013; McKenna et al., 2004; FEMA 440a, 2009a) the engineer should calibrate appropriate hysteresis parameters with appropriate experimental test data of similar structural sub-assemblies/members.

It is important to exercise engineering judgement in selecting the appropriate hysteretic parameters. Sensitivity analyses of key parameters are recommended.

C2C.5 Damping

Refer to Section C2D.4 for guidance on damping in an NLTHA.

C2C.6 Structural Performance Factor, S_p

There are two approaches when S_p is applied to NLTHA:

- **Spectrum reduction method***:* S_p is applied to the seismic hazard demand curve in which the input ground motions are scaled to as per clause 5.5.2 of NZS 1170.5:2004.
- **Base shear reduction method:** A 5%-damped elastic spectrum is used with $S_{p,nltha}$ = 1.0 for the initial scaling of the input ground motions. The NLTHA is implemented with the probable capacities. The resulting governing capacity-to-demand ratio and therefore *%NBS* are multiplied by $1/S_p$ (effectively increasing *%NBS*). The same S_p as per NZS 1170.5:2004 can be applied directly.

In the scenario where a large number of ground motion records and average responses from NLTHA are used, there is an argument that S_p as per NZS 1170.5:2004 may be unconservative. For scenarios where average responses from NLTHA are used, $S_p = 1.0$ should be adopted.

C2C.7 Interpretation of NLTHA Results and *%NBS*

It is expected that *%NBS* will be evaluated from NLTHAs as follows:

- Run analyses for the required number of earthquake strong motion records scaled to represent ULS earthquake shaking as defined in NZS 1170.5:2004.
- Check that all members/elements satisfy the deformation limits defined for the particular material types in Sections C5 to C9 in these guidelines. If they do, the analyses indicate that the building achieves at least 100*%NBS*.
- If they do not, scale all of the records using the same scale factor and re-run the analyses until acceptance is just achieved. The scale factor applied at this point is related to *%NBS* as shown in Equation C2C.1.

$$\%NBS = \text{Scale factor} \times 100 \quad \ldots\text{C2C.1}$$

C2C.8 Peer Review

As NLTHA results are highly sensitive to the input parameters, modelling assumptions and input ground motions, the results and model of an NLTHA should be peer reviewed by an independent engineer with a good knowledge and experience of running this type of analysis. Peer review solely of inputs and outputs by an engineer with little understanding of the limitations of NLTHA will rarely provide the degree of overview expected or required.

Appendix C2D: Damping

C2D.1 Introduction

Assessing the level of damping available is a critical aspect of the assessment procedures outlined in these guidelines.

The treatment of damping and how it is incorporated varies depending on the analysis technique adopted. Damping can be allowed for explicitly (e.g. NLTHA, NLSPA, SLaMA) or implicitly (force-based procedures). Guidance is provided on the intended approach for each analysis technique in the following sections of this appendix.

C2D.2 Force-based Assessment Procedures and Elastic Analysis Techniques

For force-based assessment, energy dissipation and damping is captured as an inherent ductility of 5% in the defined demands and in the defined structural ductility factor, μ, and to some extent, the structural performance factor, S_{p}. These are defined in NZS 1170.5:2004 and relate to the inherent capability of the seismic resisting systems to sustain the ductility demand and dissipate energy. They assume that the system mechanisms will be fully developed, which is not always the situation with existing buildings where the response is invariably limited by deficiencies that would not be present in a new building.

For modal response analysis it is expected that 5%-damped spectra from NZS 1170.5:2004 should be used.

For mixed ductility systems, the appropriate structural ductility factor for the total system needs to be assessed to account for the governing inelastic mechanism and the actual achievable ductility in the system.

If additional damping is present (e.g. viscous dampers) a nonlinear procedure/analysis technique should be used, as the effectiveness of the dampers will be a nonlinear function of the deformations sustained.

C2D.3 Displacement-based Assessment Procedures and Nonlinear Analysis Techniques

C2D.3.1 General

For displacement-based assessments and nonlinear static procedures, these guidelines include damping in the form of an effective system viscous damping, ξ_{sys}. The derivation of ξ_{sys} is presented in Section C2D.3.2.

The intended method of derivation of ξ_{sys} for mixed inelastic systems is given in Section C2D.3.3.

C2D.3.2 Effective system viscous damping, ξ_{sys}

The effective viscous damping for the system, ξ_{sys}, is defined as follows:

$$\xi_{sys} = \xi_0 + \xi_{hy} + \xi_d \quad \text{…C2D.1}$$

where:

ξ_0 = the inherent damping
ξ_{hy} = the hysteretic damping
ξ_d = added damping due to supplemental viscous dampers. This is taken as zero if there are no dampers present.

The inherent damping, ξ_0, present is likely to be in the range of 0.02 (2%) and 0.05 (5%) damping. For the methods outlined in these guidelines ξ_0 may be taken as 0.05 (5%).

Typical values for ξ_{hy} (expressed as a % of critical damping) are shown in Table C2D.1. The assessment of ξ_{hy} is intended to proceed as follows:

- Identify the type of structural system present.
- Evaluate the level of hysteric energy dissipation expected to be available.
- Determine the level of displacement ductility achieved at the displacement when a significant life safety hazard develops in the building, μ_{sys} .
- Obtain the value of ξ_{hy} from Table C2D.1 for the system under consideration.
- Deal with multiple systems with different values of ξ_{hy} in accordance with Section C2D.3.3 to obtain the combined effective system viscous damping, ξ_{sys} , for the building.

Note:

The evaluation of the effective hysteretic damping factor for inclusion in the calculation of ξ_{sys} will necessarily be based on judgement, interpolation of the values provided in Table C2D.1 and use of values for specific structural types from available literature. Recent research has been carried out (e.g. Wijesundara et al., 2011; Sullivan et al., 2012; Sullivan et al., 2013; O'Reilly and Sullivan, 2015) that provides expressions for a wide range of systems. This is still an area of active research (Sullivan, 2016).

Table C2D.1: Typical values of ξ_{hy} for various structural types, materials and levels of hysteretic energy dissipation

Assessed level of hysteretic energy dissipation available	μ_{sys} [2]	ξ_{hy} [1, 2] (%)	Applicable structural systems/ mechanisms
H (High)	6	15	• Concrete frame (deformed bars) • Steel frame with rigid connections
	3	12	
	2	10	
	1.25	3	
	≤1	0	
M-H (Medium to High)	6	12	• Concrete wall • Light weight timber frame • Shallow footing rocking systems • Steel EBFs
	3	10	
	2	7	
	1.25	3	
	≤1	0	
M (Medium)	6	10	• Steel frame with flexible connections • Steel CBFs • Concrete frame (plain bars) • URM system[3]
	3	8	
	2	6	
	1.25	2	
	≤1	0	
L (Low)	6	5	• Hybrid prestressed concrete frame • Rocking system
	3	4	
	2	3	
	1.25	2	
	≤1	0	

Note:

1. After Pekcan et al. (1999), Priestley et al. (2007), Sullivan (2016) and NZSEE (2006).
2. The value of μ_{hy} in the table relates to the displacement ductility experienced at the level of demand considered. Thus, even though a structure may be detailed to achieve μ = 6, the value of ξ_{hy} should be chosen assuming μ_{sys} = 3 if the structure is only loaded to, say, half capacity. Generally, engineers will be interested in performance at the displacement consistent with a significant life safety hazard so only one value of ξ_{hy} will need to be assessed.
3. For unreinforced rocking masonry walls the concept of hysteretic damping is not appropriate. However, it is considered reasonable to use ξ_{sys}= 0.15 (15%) for walls loaded in-plane and ξ_{sys}= 0.05 (5%) for face loaded walls (refer to Section C8) when applying the procedures and analysis techniques set out in these guidelines. These values of damping should be used irrespective of the level of displacement expected.

C2D.3.3 Mixed inelastic systems

These guidelines adopt a weighted approach (based on the proportion of strength capacity contributing to the lateral resistance at the level of displacement being considered) to compute the achieved effective system damping for the building (as a whole) containing mixed-inelastic mechanisms (e.g. a steel braced frame coupled with concrete shear walls).

Note:

It is difficult to extrapolate the local ductility capacity to the global ductility capacity for mixed-inelastic mechanisms. The approach outlined here represents a pragmatic solution to the issue (refer to Section C2.5.11).

C2D.4 Nonlinear Time History Analysis (NLTHA)

The viscous damping in a NLTHA is associated with the reduction in seismic response through energy dissipation other than that modelled explicitly by the nonlinear hysteresis. This inherent damping occurs principally in:

- structural and non-structural items that are treated as elastic or inconsequential but which, as a whole, may contribute to not insignificant damping, and
- foundation radiant damping.

Supplementary energy dissipation devices (e.g. viscous, hysteretic or friction dampers) should be modelled explicitly in the nonlinear model.

In absence of any substantive data, an inherent viscous damping ratio of 0.05 (5%) as per Clause 6.4.6 of NZS 1170.5:2004 should be adopted across all modes of vibration.

The modelling of inherent damping in computer programs varies. Engineers should become very familiar with the method in the particular program being used and ensure that the response is appropriately damped.

Note:

The application of an elastic damping model in the NLTHA itself is a challenging topic. The use of a Rayleigh damping model or damping based on initial stiffness can be problematic and potentially unconservative (Priestley et al., 2007; Charney, 2008). A standard Rayleigh damping model will generate unrealistically high damping force post yield, although the majority of these non-modelled damping sources do not increase post elastic.

In the Rayleigh damping formulation the damping matrix is calculated as a linear combination of mass- and stiffness-proportional damping. Alternatively, a modal damping formulation can be used where percentages of critical damping are assigned to specific periods/elastic modes.

As an existing structure is very likely to experience inelastic behaviour and strength/stiffness degradation, a tangent-stiffness damping model is generally preferred to an initial-stiffness damping model (Carr, 2007; Priestley et al., 2007).

Further information on damping is available in literature (e.g. Carr, 2007; Deierlein et al., 2010).

Appendix C2E: Diaphragm Modelling and Analysis

C2E.1 Modelling

Mathematical modelling of buildings with flexible diaphragms should account for the effects of that flexibility by modelling the diaphragm as an element with an in-plane stiffness consistent with the structural characteristics of the diaphragm system.

Modelling of buildings with rigid diaphragms should account for the effects of horizontal torsion (refer to Appendix C2F).

C2E.2 Flexible Diaphragm Analysis

For buildings with flexible diaphragms at each floor level, each lateral force resisting element in a vertical plane may be permitted to be designed independently, with seismic masses assigned on the basis of tributary area.

Note:

Whether or not a diaphragm is considered to be flexible will depend on the relative flexibility of the diaphragm compared with the lateral stiffness of the members/elements being supported/interconnected.

Figure C2E.1 shows a flexible roof diaphragm arrangement commonly encountered in New Zealand low-rise buildings. In this example the diaphragm will be flexible compared with the connected walls.

Figure C2E.1: Example of a flexible diaphragm

C2E.3 Rigid Diaphragm Analysis

Seismic demands on rigid diaphragms should be determined using an equivalent static analysis.

Note:

Use of modal analysis is not appropriate for determining inertia forces to be resisted by a diaphragm. This analysis technique provides envelopes of maxima actions that will not provide relevant information for assessing load paths across diaphragms. These maxima do not provide actions that occur together at the same point in time. Therefore, they are not in equilibrium and do not produce a vector sense for the action (the outputs are all of one sign).

Actions within the diaphragms should account for higher mode effects and the influence of overstrength actions generated from within the structure as a whole.

Research and calibration of the recommended pseudo-Equivalent Static Analysis (pESA) method (Gardiner, 2011), described in Section C2E.5, account for the overstrength actions generated in the building and for dynamic higher mode effects.

Once the seismic demands from the global analysis is calculated, internal design actions on rigid diaphragms should be determined using a strut and tie analysis.

For concrete diaphragms, refer to the provisions of Section 13 of NZS 3101:2006 and Clause 6.1.4 of NZS 1170.5:2004. Note the added complication of transfer diaphragms, particularly when not originally designed as such.

For diaphragms containing a high degree of precast elements, refer to the interconnection provisions of Section 4.4 of NZS 3101:2006.

C2E.4 Overstrength Factor

The diaphragm floor forces (effectively the inertia of each floor) should be factored up by the building overstrength factor, ϕ_{ob}, as described below:

- The building's lateral overstrength capacity, $V_{\text{o/s}}$, accounting for an inelastic mechanism and strain hardening of the ductile mechanism of the structure, divided by the probable lateral strength of the structure, V_{prob}. The building overstrength factor will typically be in the range of 1.5-2.5.

$$\phi_{\text{ob}}, = V_{\text{o/s}}/V_{\text{prob}} \qquad \text{…C2E.1}$$

- For a structure that is not detailed for capacity design, increase the floor forces above that consistent with V_{prob} by the following amplification factor, $\phi_{\text{ob},\mu\,=\,1.25}$:

$$\phi_{\text{ob},\mu\,=\,1.25} = 0.9/S_{\text{p}} \qquad \text{…C2E.2}$$

where:

S_{p} is the structural performance factor adopted for the assessment of the probable strength, V_{prob}, of the vertical lateral load resisting elements at

μ = 1.25. (The numerator of 0.9 reflects that this is the S_p value for structures with insufficient detailing to provide a reasonable degree of ductility in case a plastic mechanism has formed (part or the whole of a structure). As a diaphragm is not normally detailed for ductility, a higher strength demand is warranted.)

Consider when determining ϕ_{ob} that:

- the plastic mechanism for the structure needs to be recognisable, and
- account should be made for the likelihood of some of the plastic hinge zone not forming (for example, using the R_v factor in NZS 3101:2006).

C2E.5 Pseudo-Equivalent Static Analysis (pESA)

Note:

The pESA method is currently limited to buildings of up to nine storeys (NZS 1170.5:2004).

Using a pESA, the magnitudes and directions of the applied forces where vertical lateral force-resisting structures are connected to the diaphragm are known and are in equilibrium. These connection points include on the perimeter and anywhere within the diaphragm.

The coarseness of a pESA is somewhat mitigated if ties across the diaphragms are present and connected adequately into the vertical structural elements.

Note that, when looking at both the whole building and individual floors, there are shortcomings with the traditional approaches used to determine inertia forces applied to diaphragms:

- The equivalent static analysis (ESA) underestimates the inertia forces at lower levels,
- The floor forces "Parts and Components" (Section 8 of NZS 1170.5:2004), when applied to the structure one at a time, will underestimate actions in the building. It is not intended to apply all the floor forces determined from Section 8 to the building simultaneously as this will overestimate the floor inertia and deflection of the building. This in turn overestimates the deformation compatibility forces ("transfer" effects that are driven by lateral displacements of the structure (Gardiner, 2011)).

Figure C2E.2 illustrates the static floor forces used for the pESA. The pESA floor forces are determined in accordance with Section 6.2 of NZS 1170.5:2004 except that the horizontal seismic shear, V_E, acting at the base of the structure is taken as equal to the probable lateral strength of the structure, V_{prob}.

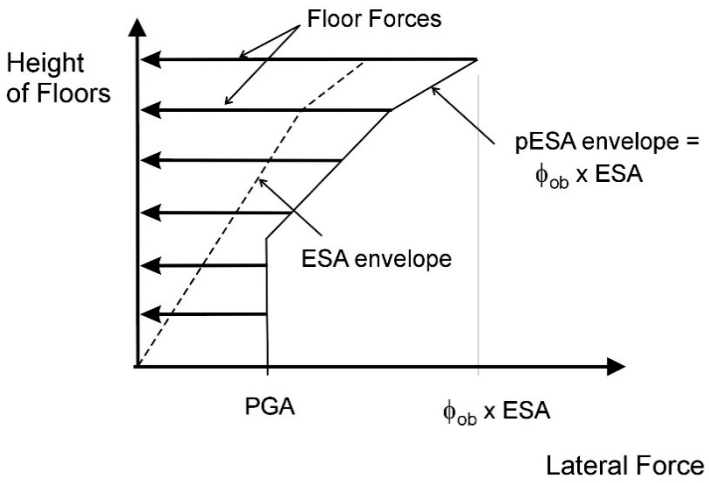


Figure C2E.2: Static forces for ESA and pESA envelopes (NZS 1170.5:2004)

Overstrength floor forces are then determined using the building lateral overstrength factor, $\phi_{ob,}$ calculated in the previous section. As illustrated in Figure C2E.2, in the lower levels of the building the floor accelerations should not be taken lower than the peak ground acceleration (PGA).

At the overstrength of the building the recommended pESA floor forces result in:

- an appropriate deformed shape for the building; hence the deformation compatibility forces are in the right order of magnitude
- a reasonable estimate of the amplified inertias for the upper floors
- PGA inertias that are approximately correct for the lower floors of the building. This has little effect on the overall deformed shape of the building, and hence little effect on the deformation compatibility forces.

The line of action for the floor forces should be taken through the centre of mass of each floor.

Appendix C2F: Torsion

C2F.1 General Approach

For buildings with rigid diaphragms it will be necessary to consider the torsional amplification effect arising from the demand and resistance eccentricities (centre of mass including accidental displacement allowance) and the location of the centre of stiffness or strength as appropriate).

Torsion does not need to be considered in buildings with flexible diaphragms.

There can be several types of torsion response:

- **accidental torsion** arising from the effects of the rotational component of the ground motion, differences between computed and actual stiffnesses, and unfavourable distributions of dead and live load masses (assumed to be covered by the allowance for accidental displacement of the centre of mass)
- **inelastic torsion** arising from the effects of nonlinear behaviour and interaction between systems with different post-yield stiffness, strength degradation and ductility capacity, and
- **torsional amplification** arising in a deteriorating system resulting in plan irregularity; e.g. premature damage of infill panels at one elevation may lead to inelastic torsion response.

Torsional response can lead to amplification of either internal actions or displacement responses, depending on several factors:

- degree of plan stiffness irregularity:

  $e_{\text{stiffness}}$ = distance between centre of mass and centre of rigidity/stiffness

- degree of plan strength irregularity:

  e_{strength} = distance between centre of mass and centre of strength (linear or nonlinear)

- torsionally restrained (TR) systems with torsional resistance from lateral bracing elements in the orthogonal direction or torsional stiffness of the overall lateral load resisting system
- nonlinear behaviour and interaction of the lateral load resisting systems. Systems with low post-yield, high differential strength degradation and ductility capacities are shown to have higher inelastic torsion amplification.
- rotational mass inertia. Torsionally unrestrained (TU) systems can have significant displacement/ductility demand amplification due to rotational mass inertia (Paulay, 2000b).

This appendix provides three assessment methods. The first, Method A, is for systems expected to respond elastically or with nominally ductile demand at the lowest loading that leads to a significant life safety hazard.

However, for most existing structures with nonlinear response – and especially for those with mixed ductility response – an inelastic torsional assessment should be carried out using either:

- Method B: Inelastic torsion response with ductile responding systems (in both directions), or
- Method C: Removing strength eccentricity in inelastic response.

C2F.2 Method A: Elastic Torsion Response

For assessment using the elastic force-based procedure and linear analysis techniques (equivalent static or modal response analysis), only the consideration of accidental torsion will be required. The conventional 10% accidental mass eccentricity assumption as per Clause 6.3.5 of NZS 1170.5:2004 can be used. This method is intended for systems that are expected to respond elastically or with nominally ductile demand at the lowest loading that leads to a significant life safety hazard.

C2F.3 Method B: Inelastic Torsion Response with Ductile Systems in Both Directions

This approach relies on the lateral load resisting system in the orthogonal direction to provide torsional resistance.

In this method, the shear demand in the orthogonal direction is amplified to account for the torsional demand (as outlined below). Alternatively, the engineer could calculate the torsional moment of inertia and amplify the displacement demand corresponding to the torsional demand.

If the lines of the primary lateral system in the direction being considered have some ductility capacity ($\mu \geq 2$) it is considered acceptable to resist the torque resulting from the eccentricities solely by the couple available from the lines of the primary lateral system perpendicular to the direction of loading (refer to Figure C2F.1). If this approach is followed the centre of strength rather than the centre of stiffness should be used when evaluating the eccentricities.

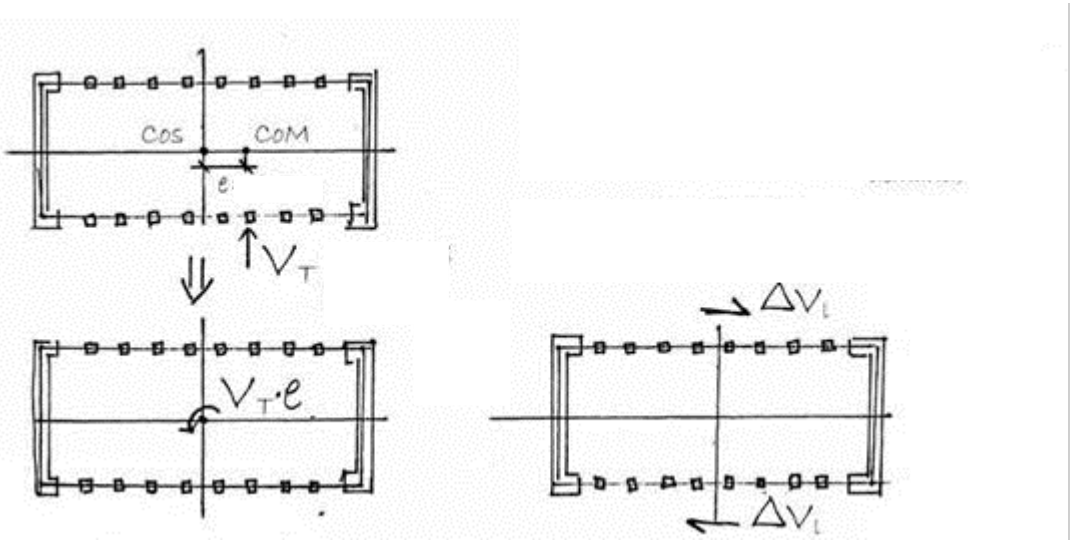
The increase in shear demand in the orthogonal direction can be calculated assuming any torsional demand is resisted by a torsional couple formed by the lateral load resisting members in the orthogonal direction. Therefore, for the example shown in Figure C2F.1(b):

$$\Delta V_{\mathrm{t}} = V_{\mathrm{base}} \text{ x } e_{\mathrm{strength}}/L \qquad \ldots\text{C2F.1}$$

where:

ΔV_{t} = the shear force increase in the lateral load resisting members in the orthogonal direction

L = the distance between the centroids of the lateral load resisting lines in the orthogonal direction.



(a) Torsional demand **(b) Resistance provided by orthogonal wall lines**

Figure C2F.1: Relationship between demand and resistance for a building with rigid diaphragms

Note:

For this method to be valid for nonlinear responding systems, the lateral load resisting systems should be ductile ($\mu \geq 2$) and the elements in the orthogonal wall lines should not become nonlinear under these actions. In addition, this method requires that the diaphragm maintains its structural integrity in order to mobilise the lateral system in the orthogonal direction to provide torsional resistance.

In a building assessed following the Christchurch earthquake sequence (Kam et al., 2011) the lateral load resisting systems were severely compromised due to the combination of inelastic torsion response and diaphragm failure. In this building, the lateral load resisting system comprised perimeter ductile moment resisting frames with mesh-reinforced concrete topping on precast double tees acting as a diaphragm.

In the 4 September 2010 earthquake, the ductile perimeter moment frames in the north-south direction yielded, resulting in a nonlinear system with low post-yield stiffness. In the 22 February 2011 earthquake, the loss of the diaphragm as well as relatively low stiffness of the moment frames in the orthogonal direction resulted in limited to no torsional resistance being provided by the moment resisting frames in the east-west direction.

C2F.4 Method C: Absence of Strength Eccentricity

In this approach, the achievable lateral capacity (e.g. base shear) is reduced to eliminate the strength eccentricity and therefore any inelastic torsion amplification. This method is intended for torsionally unrestrained systems or systems with limited torsional redundancy.

If the strength eccentricity exceeds 2.5% of the relevant lateral dimension of the plan, the probable strength of the system should be revised by reducing the probable strengths of those elements which are responsible for the strength eccentricity until the strength eccentricity is

eliminated. Therefore, the global base shear capacity is artificially reduced to eliminate the strength eccentricity and mitigate any inelastic torsional response.

This procedure is based on the assumption that, in the absence of the strength eccentricity, the response of the system may be considered to be governed primary by translatory displacements (Paulay, 2001; Paulay, 2000a). In terms of ductile response, effects of stiffness eccentricity may be ignored.

This method may provide a conservative estimate of the lateral capacity of the building as it seeks to minimise inelastic torsion response. Castillo et al. (2002) indicates that this method is most suitable for torsionally unrestrained systems for which other methods may be un-conservative.

Note:

Figure C2F.2 illustrates an example where it is found that the relative probable translatory strengths of elements (1), (2) and (3) are 46%, 18% and 36% respectively. These result in a negative strength eccentricity of $e_{vx} \approx 0.10A > 0.025A$.

By only relying on 30% and 13% strength contribution to elements (1) and (2) respectively, the overall strength eccentricity, e_{vx} can be eliminated. This reduces the achievable probable strength to 79%.

The expected displacement ductility demand on the system may then be based on this reduced system strength. Under these circumstances displacement demands on element (3) due to system translations and rotations, while developing 100% of the probable system strength, will not be critical.

In traditional design procedures which were based on elastic structural behaviour, strengths of elements were assigned in proportion to their assumed stiffness. Subsequently, strength redistribution (NZS 3101:2006) within a 30% limit was permitted to be used, provided that the total seismic strength of the building was not reduced.

However, this restriction on the allocation of seismic strength to elements is now considered to be unnecessary. Therefore, reliance on the probable strengths of elements, as constructed, may be made without recourse to analysis of the elastic structure when evaluating the probable global base shear of the system.

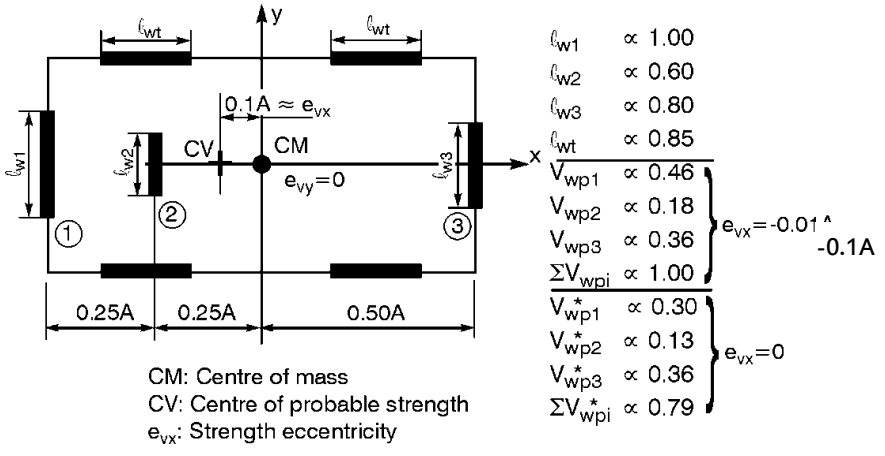


Figure C2F.2: Torsional effects in walled buildings (after Paulay, 2001)

Appendix C2G: Severe Structural Weaknesses

C2G.1 Approach to Assessing SSWs

Aspects that should be evaluated as SSWs in a DSA to meet the requirements of these guidelines and the quantitative means for assessing them are described in Section C1.

Note:

Refer to Section C1 for more about the rationale for assessing SSWs and the criteria used to define these.

The typical approach followed in these guidelines is to determine the capacity of SSWs as follows:

- Assess the probable capacity of the elements/members comprising the system that is considered to be the SSW using the methods outlined in Sections C4 to C9.
- Assess the probable capacity of the system using the methods outlined in this section.
- The capacity (strength and deformation) of the SSW is then taken as one half of the probable capacity.

For the SSWs involving geohazards it may be more convenient to determine the level of shaking that would lead to the levels of deformation in the structure that would create a significant life safety hazard, in accordance with Section C4. The capacity of the SSW is then taken as half of this level of shaking.

C2G.2 Non-ductile Columns with Axial-shear Failure

These are lightly reinforced concrete columns and/or beam-column joints (refer to Section C5 for definition) with axial loads greater than 0.5 $A_g f'_c$ which are part of the primary structural system (typically the gravity system) of buildings where multiple fatalities would be possible if one or more storeys were to suffer full collapse. To be a SSW the failure of a column and/or beam/column joint would need to lead to a progressive collapse scenario for the entire storey.

The capacity of any such non-ductile reinforced concrete columns susceptible to axial-shear failure should be taken as **one half of the probable lateral drift capacity determined in accordance with Section C5**.

Note:

The axial-shear failure of key vertical load carrying members such as primary columns and beam-column joints has been shown to be a critical deficiency that has led to catastrophic collapse of reinforced concrete buildings (Elwood and Moehle, 2005; Boys et al., 2008; Kam et al., 2011).

Some structural systems have the ability to redistribute the axial load for localised axial-load failure. In other cases, progressive collapse can be initiated with the failure of one column or joint. The severity of the axial-shear failure of a particular vertical load carrying member should be assessed.

It is vital that both the overall analysis and the assessment of ductility demand take proper account of the characteristics of short columns. Displacements generated in the structure can have a severe effect on the integrity of these elements by driving up shear forces beyond the capability of the sections.

The most critical aspect of the detailing of reinforced concrete columns for flexural ductility capacity is the amount of transverse reinforcement provided; in particular, the spacing between adjacent reinforcement sets (Park and Paulay, 1975). The transverse reinforcement provides confinement to the core concrete and prevents the longitudinal bars from buckling. In general, the lower the transverse reinforcement ratio the less ductile the column will be under lateral displacements such as those experienced during an earthquake.

The performance of non-ductile reinforced concrete columns with low quantities of transverse reinforcement has been covered extensively in literature (in particular, refer to Boys et al., 2008; Elwood and Moehle, 2005; Kam et al., 2011).

In addition to low quantities of transverse reinforcement, several other characteristics of a column can contribute to its vulnerability in an earthquake. The following is a list of indices (Stirrat et al., 2014) that may suggest the columns are susceptible to non-ductile behaviour. Note that the suggested limits are based on available literature and experience, and have not been extensively tested. The nomenclature is that used in NZS 3101:2006.

- Low or inadequate quantities of transverse reinforcement – spacing, $s > d/2$
- High axial load demand – $P/A_{\mathrm{g}}f'_{\mathrm{c}} > 0.3$
- Low core-to-gross concrete area – $A_{\mathrm{c}}/A_{\mathrm{g}} < 0.7$
- High inelastic inter-storey drift demand – > 1.5% drift
- Detailing – inadequate lap-splice length, lap-splice located in potential plastic hinge zone, poor detailing of transverse reinforcement anchorage (e.g. 90 degree bends), welded detailing, lack of support to longitudinal bars
- Location of column – in location prone to inelastic torsional amplification of displacements (e.g. corner column or column on opposite face to eccentric shear core).

C2G.3 Non-ductile Shear Wall Without Redundancy

This SSW is a shear wall system meeting the following criteria:

- it supports a significant level of axial load where $N^*_{\mathrm{g}} \geq 0.15A_{\mathrm{g}}f'_{\mathrm{c}}$, where N^*_{g} is the axial load under dead and reduced live load (Q_{u}), and
- it has shear-failure dominated force-controlled mechanism (i.e. not flexural governed behaviour), and
- it is a group of interconnected walls acting as a single unit (single core wall) which supports more than 60% of the seismic lateral demand, and
- multiple fatalities would be possible if the building were to suffer full collapse.

The shear capacity for these critical walls should be taken **as one half the probable shear capacity determined in accordance with Section C5**. The shear capacity should be appropriately modified accounting for axial and flexural interaction.

C2G.4 Flat Slab Floor System Susceptible to Punching Shear Failure

This SSW is a flat slab system in a cast insitu concrete without shear reinforcement in the slab and with gravity-only shear demand exceeding 40% of the probable shear capacity (v_c+v_s) at the critical shear interface, and multiple fatalities would be possible if one or more storeys were to suffer full collapse..

The capacity of this SSW is taken as **one half of the probable drift capacity of the axial-shear mechanism determined in accordance with Section C5**. The intent is indicated in Figure C2G.1 Figure C2G.1

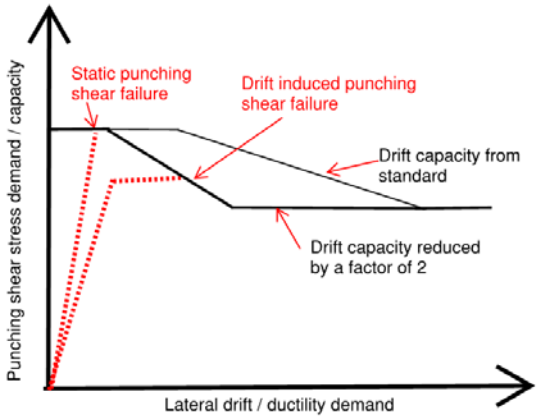


Figure C2G.1: Shear demand versus drift relationship for non-ductile flat slab-column system

Note:

Refer to Section C5 for the assessment of punching shear failure of a non-ductile flat slab by considering both gravity load and drift-induced punching shear demand.

The flat slab-column system is generally used with a rigid lateral load resisting structural system, such as shear walls or moment frames. Irrespective of the primary lateral load mechanism, the slab-column system must maintain its gravity load capacity based on displacement compatibility. As the flat slab system sways laterally, the unbalanced bending moment in the slab-column connection results in increased punching shear demand.

Flat slabs, particularly those with discontinuous bottom reinforcement or that are lightly reinforced, are susceptible to progressive collapse if punching shear failure occurs at a connection (e.g. Robertson and Johnson, 2004; Kang and Wallace, 2006). Many such failures have occurred in past earthquakes and led to significant loss of life. An example of this type of failure is shown in Figure C2G.2

Figure C2G.2: Collapse of flat-slab system observed in Christchurch (from Kam et al., 2011)

C2G.5 Diaphragm Without Redundant Load Path

This SSW is a concrete diaphragm (most likely to be precast concrete) in systems where, if there is a loss of diaphragm connection, there is no ability to redistribute seismic actions through other means (e.g. a core wall building). This can also lead to undesirable inelastic torsional instability as described in Appendix C2F.

This SSW is only intended to apply to a diaphragm system that meets the following criteria:

- there is lack of ductile connection to the vertical lateral load elements; i.e. the failure plane of the diaphragm is either unreinforced or is only reinforced with brittle cold-drawn mesh reinforcement, and
- the diaphragm transfers all seismic inertia into no more than two vertical lateral load elements in a particular direction (i.e. there are no more than two bracing lines), and
- multiple fatalities would be possible if one or more storeys were to suffer full collapse.

The capacity of such a diaphragm is taken as **one half of the probable capacity determined in accordance with Section C5.**

Note:

Refer to Section C4, DBH (2009) draft guidelines and Fenwick et al. (2010) for further information.

C2G.6 Loss of Support Due to Complex Slope Failure

This SSW is a complex slope failure resulting in complete loss of the building platform and support. It applies where more than 50% of the building platform would be affected by slope failure; i.e. where the building is on a slope or cliff edge.

The capacity of the slope is taken as **one half of the peak ground acceleration causing a slope failure that would result in the complete loss of at least 50% of the building platform**. This is calculated using probable soil parameters in accordance with Section C4.

Note:

Refer to Section C4 for guidance on the assessment of slope failure.

C2G.7 Poorly Tied Together Multi-Storey URM Structure On Liquefiable Ground

This SSW applies to a poorly tied together building that:

- is on liquefiable ground, and
- where multiple fatalities would be possible if one or more storeys were to suffer full collapse.

A poorly tied together building is one where the ties within the building score less than the assigned *%NBS* for the SSW.

PART C

Earthquake Demands C3

Document Status

Version	Date	Purpose/ Amendment Description
1	July 2017	Initial release

This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

This document may be downloaded from www.EQ-Assess.org.nz in parts:

1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

This document is managed jointly by the Ministry of Business, Innovation and Employment, the Earthquake Commission, the New Zealand Society for Earthquake Engineering, the Structural Engineering Society and the New Zealand Geotechnical Society.

Please go to www.EQ-Assess.org.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.EQ-Assess.org.nz

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C3. Earthquake Demands

C3.1 General

C3.1.1 Outline of this section

This section sets out the intended method for deriving the Ultimate Limit State (ULS) seismic demand, which is needed to evaluate the *%NBS* earthquake rating in accordance with Part A and Section C1. It also lists the available representations of the ULS seismic demand and explains what is intended for these.

C3.1.2 Definitions and acronyms

100%ULS seismic demand	Ultimate limit state seismic demand for new buildings used in the calculation of *%NBS*. Can be represented in a number of ways depending on the aspect under consideration.
ADRS	Acceleration-displacement response spectrum (spectra)
Importance level (IL)	Categorisation defined in the loadings standard, AS/NZS 1170.0:2002. This is used to define the ULS shaking for a new building based on the consequences of failure and is a critical aspect in determining new building standard.
PGA	Peak ground acceleration
Simple Lateral Mechanism Analysis (SLaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (push-over) relationship for the building as a whole
Site subsoil class	Categorisation of the soil profile under the building in accordance with NZS 1170.5:2004
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings

C3.1.3 Notation, symbols and abbreviations

Symbol	Meaning
%NBS	Percentage of new building standard as assessed by application of these guidelines
g	Acceleration due to gravity
$K_\delta\ (T)$	Displacement spectral scaling factor. Varies depending on the building period, *T*.
k_μ	Inelastic spectrum scaling factor as defined in NZS 1170.5:2004
K_ξ	Spectral damping reduction factor (refer to Section 0)
R	Return period factor. Will typically be R_u determined in accordance with NZS 1170.5:2004.
R_u	Return period factor appropriate for the ULS. Determined in accordance with NZS 1170.5:2004.
S_a	Spectral acceleration

Symbol	Meaning
S_{d}	Spectral displacement
S_{p}	Structural performance factor. Determined in accordance with NZS 1170.5:2004.
T	Period(s) of vibration for the building
T_{eff}	Effective period of vibration of the equivalent single degree of freedom representation of the building
V_{prob}	Probable shear capacity
W	Total weight of the structure
Δ_{cap}	Probable deflection capacity at the effective (equivalent) height
ξ_{sys}	Equivalent viscous damping of the system
π	Refer to Equation C3.2, Section 0 and Equation C3.5

C3.2 Method for Deriving ULS Seismic Demand

C3.2.1 General

The basis for the derivation of ULS seismic demand is the New Zealand earthquake loadings standard NZS 1170.5:2004 and Module 1 of the New Zealand Geotechnical Society and Ministry of Business, Innovation and Employment's Earthquake Geotechnical Engineering Practice series (NZGS/MBIE, 2016). These are assumed to define 100%ULS seismic demand or, in other words, the seismic demand that would be used to design a similar new building for the ULS at the time the assessment is undertaken.

Note:

The basis for setting the ULS seismic demand for determining *%NBS* generally is the demand determined in accordance with the versions of the above documents that are current at the time the assessment is completed.

ULS seismic demand for the purposes of defining an earthquake-prone building in accordance with these guidelines has been set in legislation as that which would have been obtained for the design of a new building from NZS 1170.5:2004 (including Amendment 1) and Module 1 of the Earthquake Geotechnical Engineering Practice series dated March 2016. These documents define the seismic demand that was current at the time the legislation was enacted, which is the relevant basis for the ULS seismic demand used to calculate the earthquake-prone threshold adopted in these guidelines of 34*%NBS.*

The importance level (IL) used for the evaluation of the ULS seismic demand shall be derived from AS/NZS 1170.0:2002 based on the use/intended use of the building.

For the purposes of deriving the ULS seismic demand, the design life shall not be taken as less than 50 years unless a lower design life has been formally established with the relevant building consent authority/territorial authority.

Note:

An argument can be raised that life safety risks should not be affected by the chosen design life of the building. The rationale for this is that the life safety risk exists at any point in time (say, expressed as an annual risk) and is not affected by the total exposure period, whereas the exposure period is relevant when considering the potential economic losses (for example) over the life of the building.

While the concept of a design life less than 50 years is allowable under AS/NZS 1170.0:2002, this is on the assumption that the building will be removed when this period expires and that this intention will be noted on the building file held by the building consent authority/territorial authority. This should also apply if a building is assessed from a regulatory point of view or a consent for alteration (retrofit) is applied for. It is not intended that a chosen design life of less than 50 years is simply rolled over in perpetuity. In accordance with the intent of the New Zealand Building Code a 50 year exposure period (design life) is considered to represent an indefinite design life.

C3.2.2 Available representations

Representation of the ULS seismic demand will vary depending on the method of analysis and the particular aspect being assessed.

The range of available representations includes:

- acceleration response spectra
- displacement response spectra
- acceleration-displacement response spectra (ADRS)
- ground acceleration, velocity or displacement strong motion records
- peak ground acceleration (PGA), ground displacements, characteristic earthquakes, numbers of cycles for geotechnical considerations
- inter-storey drifts and total deformation between supports for elements supported on ledges, and
- applied accelerations and displacements on elements of the building.

When using time history analysis techniques it may be appropriate to determine the *%NBS* by scaling input motions. In these circumstances the scaling should only be applied to the ground accelerations and displacements and not to the duration of shaking, which should remain as appropriate for the ULS.

Likewise, when running traditional analysis for a target *%NBS* (say 34*%NBS* for a simple earthquake-prone check) it is only the response spectral ordinates that are scaled. The duration of shaking remains unchanged from that implied by the 100%ULS seismic demands.

Note:

While it is acknowledged that some engineers will be more familiar with the elastic based representations of NZS 1170.5:2004 and the allowance for ductility through application of an assumed global ductile capability, the thrust of these guidelines is to take account of the nonlinear deformation capability of the building directly using the displacement-based simple lateral mechanism analysis (SLaMA) approach and the ADRS representation of the seismic demand.

C3.3 Horizontal Acceleration Response Spectra

When a horizontal acceleration response spectrum is used to establish the ULS seismic demand, the spectrum shall be derived in accordance with NZS 1170.5:2004 Clauses 5.2.2.1 and 5.2.2.2 including an appropriate value for S_p, which may vary depending on the particular aspect being assessed (refer to Section C3.10.2).

When required, horizontal acceleration response spectra for different damping values may be obtained by multiplying the spectral ordinates of the 5% damped elastic spectrum determined as above (i.e. setting $k_\mu = 1$) by the spectral damping reduction factor, K_ξ:

$$K_\xi = [7/(2 + \xi_{sys})]^{0.5} \quad \text{…C3.1}$$

where:

ξ_{sys} = equivalent viscous damping of the system (refer to Appendix C2D for calculation of ξ_{sys}).

Note:

Priestley et al. (2007) provides some guidance on damping and the resulting reduction in spectral demand for seismic assessment. Equation C3.1 is presented as part of this guidance.

While Kong and Kowalsky (2016) have recently noted that the above equation appears to be quite reasonable for large magnitude events, studies such as those by Akkar et al. (2014) and Rezaeian et al. (2014) indicate that the actual damping-dependent spectral scaling factor should be a function of several factors including magnitude, epicentral distance (and depth) and period of vibration.

Pennucci et al. (2011), on the other hand, demonstrated that more representative inelastic (effective period) spectra for use with the displacement-based design/assessment approach could be obtained by scaling the displacement spectrum using ductility-dependent, as opposed to damping-dependent, spectral scaling factors. However, Pennucci et al. (2011) also point out that scaling factors should be a function of spectral shape and the results presented by Stafford et al. (2016) indicate that such inelastic spectra should again depend on magnitude and period.

For sites affected by near-field ground motions containing velocity pulses, Priestley et al. (2007) recommended changing the exponent within Equation C3.1 from 0.5 to 0.25 to account for the limited benefit of hysteretic energy dissipation characteristics on inelastic displacement demands induced by velocity pulse characterised near-field motions.

However, results presented in Sullivan et al. (2013) suggest that when the effective period of a structure is assessed to be less than the velocity pulse period for the site then no change is required to the scaling recommended for far-field motions. In contrast, when the velocity pulse period is equal to or larger than the pulse period, the inelastic displacement demands tend to be equal to the elastic spectral displacement demands (suggesting no benefit of hysteretic response).

Near-fault effects have traditionally been associated with larger magnitude earthquakes. However, Bradley (2015) indicated that near-fault effects were also discernible in the moderate magnitude Christchurch near-fault events.

NZS 1170.5:2004 currently adjusts the acceleration response hazard spectrum for near-field effects using the near-fault factor. This addresses the increased amplitude of the expected motion for larger magnitude earthquakes (also taking into account the directional nature on the expected frequency of occurrence) but does not otherwise address the effect of the reduction in the ability to dissipate energy, and therefore the reduced effect of the ability of nonlinear behaviour (ductility) to reduce a building's response.

It is clear that additional research is needed to determine how best to account for near-field effects in design and assessment and the extent to which this phenomenon needs to be allowed for. It might be expected that future revisions of NZS 1170.5:2004 will need to address this issue which may increase demand requirements. This could also lead to the need to reconsider the level of damping that might be available and the expected effect of this. However, in the interim, it is recommended that Equation C3.1 continues to be used for all sites.

C3.4 Horizontal Displacement Response Spectra

For displacement based methods, a displacement response spectrum is required. For the purposes of these guidelines it is considered appropriate to derive the 5% damped spectral displacement spectrum by multiplying the ordinates of the 5% damped elastic acceleration spectrum from Section 0 by the factor:

$$K_{\delta}(T) = 9800T^2/4\pi^2 \quad \ldots\text{C3.2}$$

Displacement spectra for different damping values may be obtained by multiplying the 5% damped displacement spectrum by the factor K_{ξ}, calculated using Equation C3.1.

Figure C3.1 illustrates the shape of the resulting displacement spectra for Wellington, Christchurch and Auckland for different subsoil conditions. The effect of the application of K_{ξ} is illustrated in Figure C3.2. These figures show the spectra suitable for general purposes, i.e. not the bracketed values from Table 3.1 in NZS 1170.5:2004.

Examination of the displacement spectra in Figures C3.1 and C3.2 reveals several interesting points.

First, the significance of the soil type is much more apparent when seismicity is expressed in terms of displacement, rather than acceleration, spectra.

Second, apart from some nonlinearity for low periods, the curves are well represented by straight lines from the origin as shown on Figure C3.2. For sites where near-fault effects are not an issue the displacement spectra are well represented by a bilinear relationship pivoting around the displacement at T = 3 seconds and with a horizontal leg beyond 3 seconds. For a site where near-fault effects are specified the displacement spectra can be approximated by a bilinear relationship between T = 0, 3 and 4.5 seconds. These are approximations, the validity of which will need to be confirmed. It is expected that the straight-line approximations indicated are sufficiently accurate to be used as the basis for assessments and design of retrofit works. However, this should not preclude a more precise or direct evaluation should circumstances warrant or allow.

Third, the displacement spectra obtained do not represent the tendency of the spectral displacement to converge to the peak ground displacement at long periods but maintain the spectra conservatively at constant peak displacement response values (or increase these for sites where near-fault effects are specified).

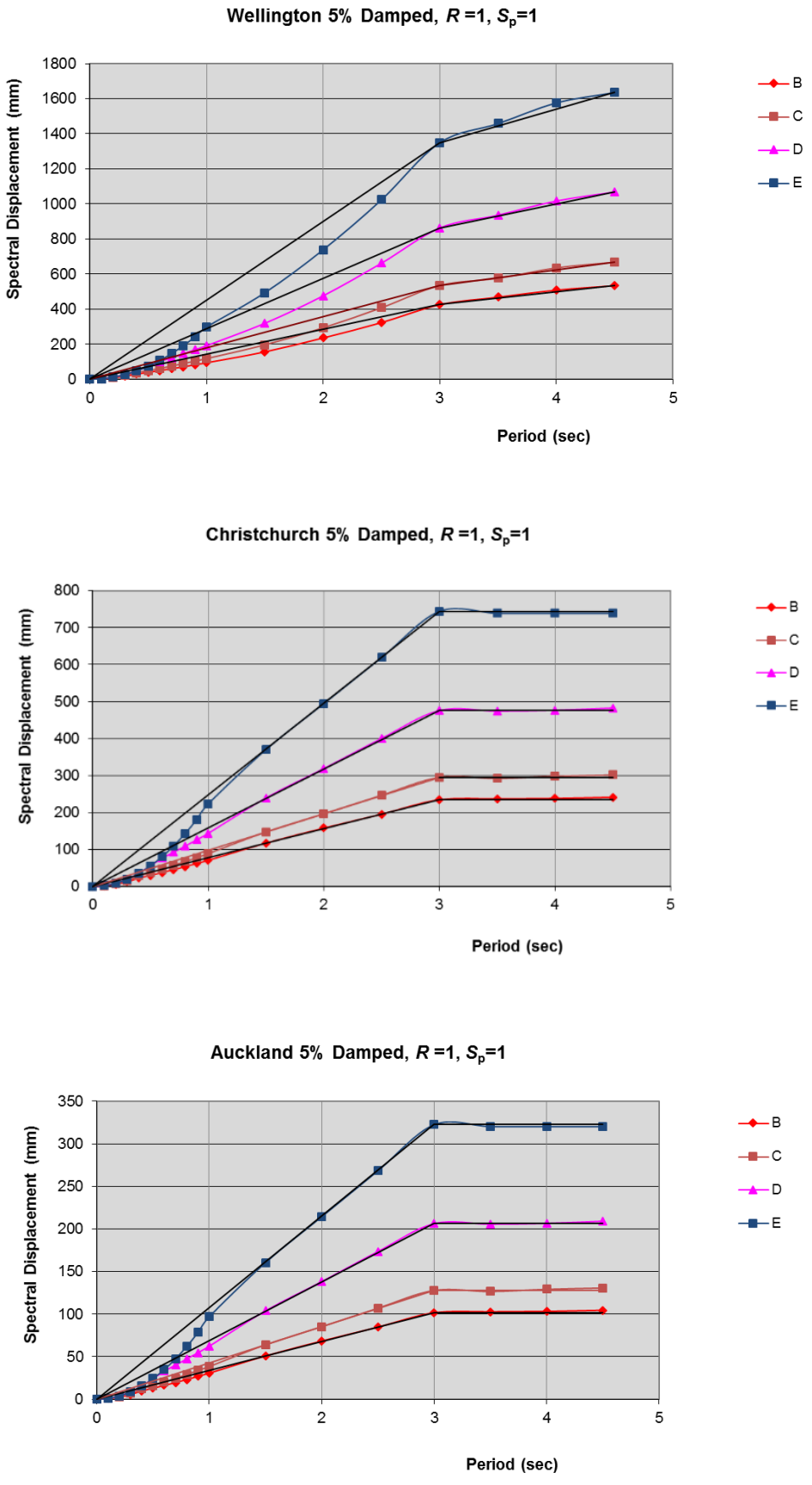


Figure C3.1: Displacement spectra at 5% damping for R = 1, S_p = 1 for various site subsoil classes and including appropriate near fault factor

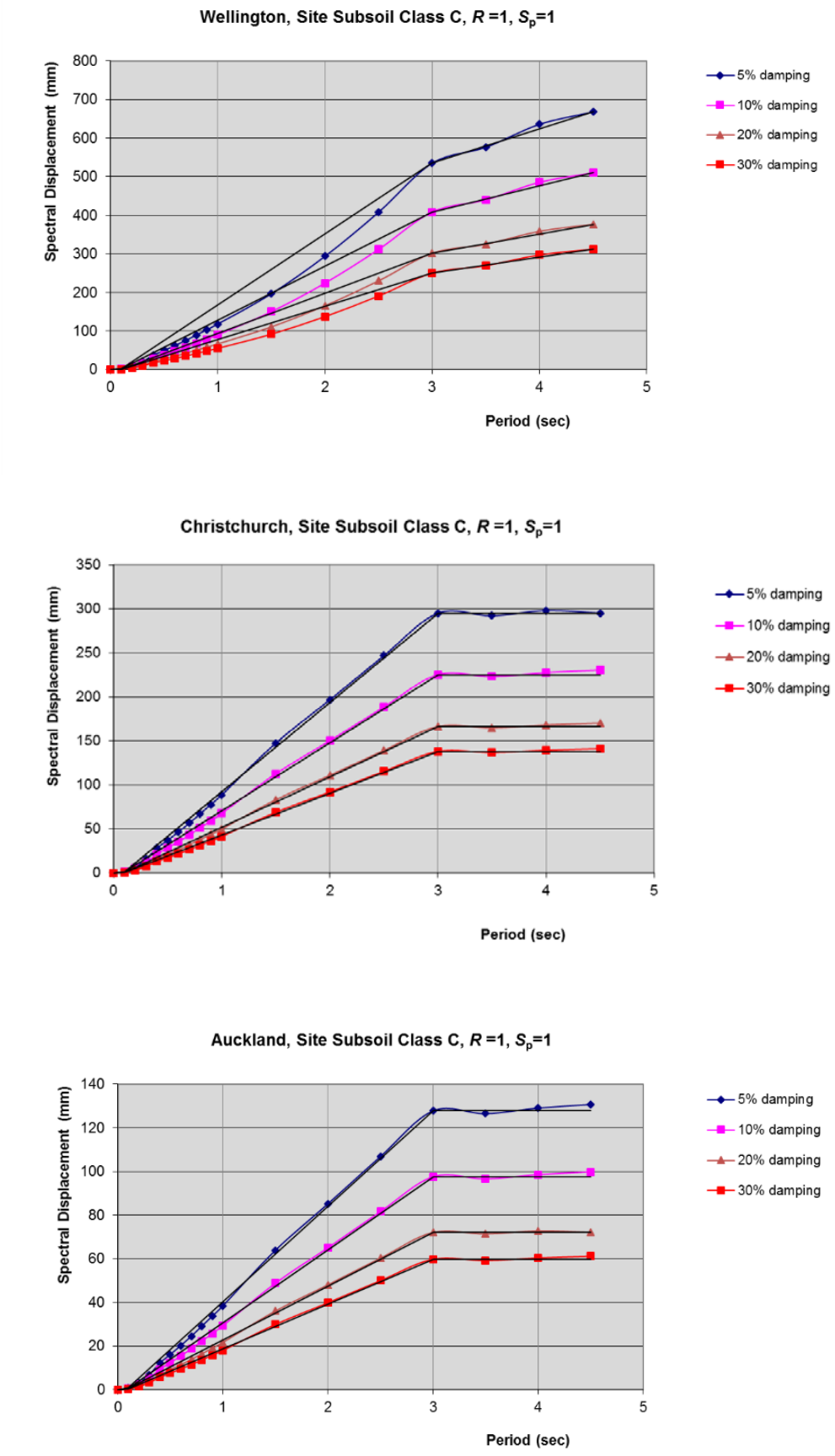


Figure C3.2: Displacement spectra for different damping levels and site subsoil class C and including appropriate near fault factor

C3.5 Horizontal Acceleration-Displacement Response Spectra (ADRS)

The acceleration and displacement spectra derived in the previous two sections for a particular site and level of damping can be usefully presented in the form of an acceleration-displacement response spectrum (Mahaney et al., 1993). The ordinates of such a spectrum are spectral acceleration and spectral displacement. An example of such representations is shown in Figure C3.3 for Wellington, Christchurch and Auckland for a 500 year return period (R_u = 1), S_p = 1 and site subsoil class C.

When constructing an acceleration-displacement spectrum for a particular level of damping both the acceleration and the displacement ordinates must be multiplied by K_ξ and the appropriate value of S_p.

Acceleration-displacement spectra are particularly useful when assessing the *%NBS* of a building from the results of a nonlinear pushover analysis. The acceleration and displacement results from a pushover analysis need to be converted to spectral acceleration and spectral displacement (as described below) before comparisons are possible with the acceleration-displacement spectra described above.

Note:

When a pushover curve has been derived from the combination of various structural systems of different ductile capability (using, for example, the SLaMA method), it may be more useful to incorporate the various S_p factors into the combined system pushover curve and compare against the ADRS calculated assuming S_p = 1 (refer to Section C3.10.2).

The conversion can be carried out as follows, assuming that elastic response is a good predictor of inelastic response and/or response in the first mode dominates (neither will always be the case):

$$S_a = V_{prob}/W \quad \text{…C3.3}$$

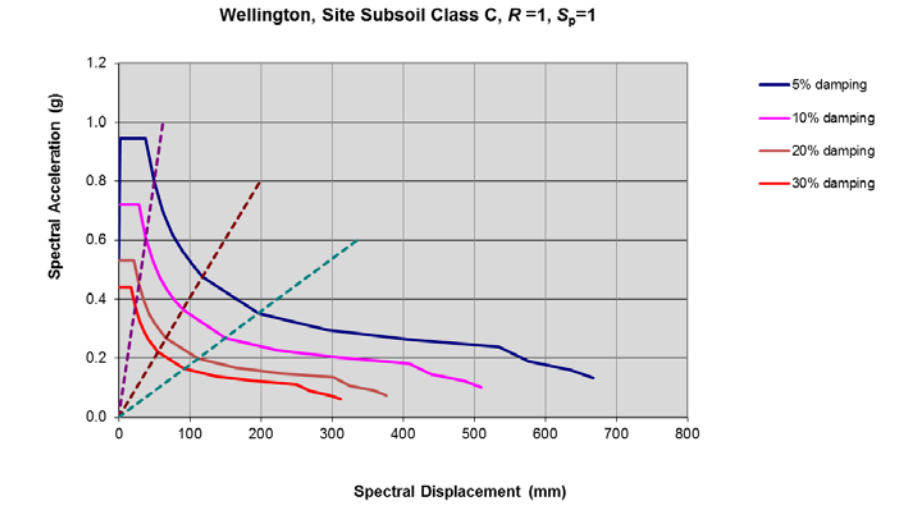
$$S_d = \Delta_{cap} \quad \text{…C3.4}$$

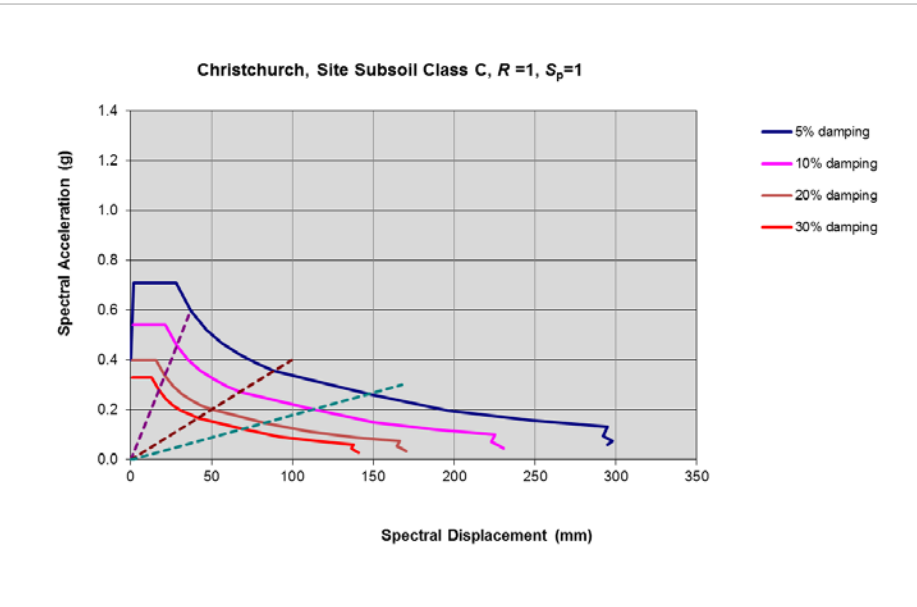
where:

V_{prob} = probable base shear capacity consistent with Δ_{cap} (as calculated in Section C2)

W = total weight of the structure. This can be substituted with effective mass times g as calculated in Section C2. When this substitution is made V_{prob} is the base shear capacity of the first mode.

Δ_{cap} = maximum lateral displacement capacity determined at the effective (equivalent) height (refer to Section C2).





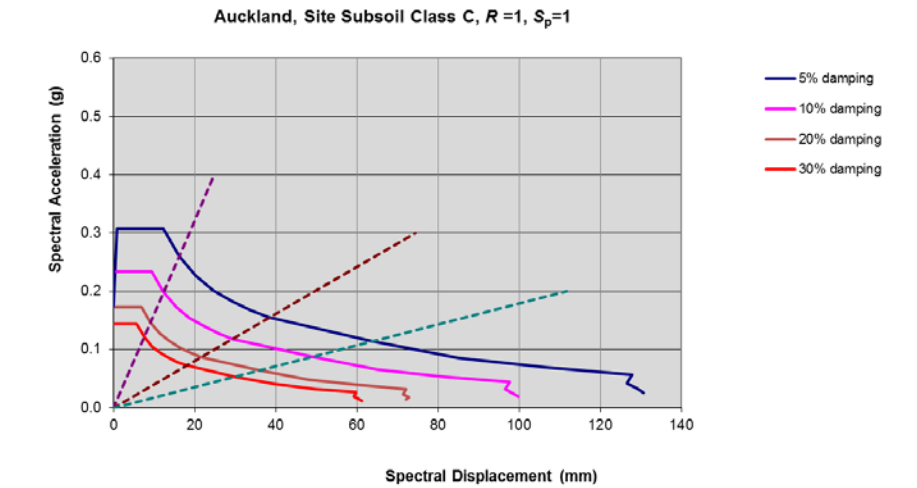


Figure C3.3: Acceleration-displacement spectra for different damping levels for R = 1, S_p = 1 and site subsoil class C

Note that the effective period, T_{eff}, of the equivalent single degree of freedom system can be approximated (assuming predominantly first mode response) from the relationship:

$$T_{\text{eff}} = 2\pi \sqrt{(S_{\text{d}}/S_{\text{a}})} \quad \ldots\text{C3.5}$$

where:

S_{a}, S_{d} are as defined above.

Thus the stiffness of the building (T) can be represented by radiating lines from the origin of the acceleration-displacement spectrum. These lines, for example periods of 0.5, 1.0 and 1.5 seconds, are shown in Figure C3.3.

Note:

ATC 40 (1996) presents an excellent discussion on the way in which the acceleration-displacement spectrum can be derived and used to assess the performance of buildings. Refer to Section C2 for the use of ADRS with nonlinear static pushover analysis and in particular with SLaMA.

C3.6 Vertical Acceleration Response Spectra

When a vertical response spectrum is required to establish the ULS seismic demand, the spectrum shall be derived from NZS 1170.5:2004, Clause 5.4.

C3.7 Acceleration Ground Motion Records

When acceleration ground motion records are required, their selection and scaling shall meet the requirements of NZS 1170.5:2004, Clause 5.5.

The input earthquake records shall either contain at least 15 seconds of strong motion shaking or have a strong shaking duration of at least five times the fundamental period of the structure, whichever is greater.

All three components of any ground motion records should be used where all components are scaled by the same factor which is determined separately for each direction of application of the principal component. When scaled ground motion records are used to establish a *%NBS* other than 100*%NBS*, only the acceleration ordinates should be scaled. The duration of shaking established for the ULS seismic demand should not be changed.

C3.8 Demands on Elements Not Part of the Primary Lateral Structure

The ULS seismic demand on elements not part of the primary lateral structure should be determined in accordance with Section 8 of NZS 1170.5:2004. The demand may be in the form of applied loads/forces or deformations. Further guidance is provided in Sections C2 and C10.

C3.9 Representations for Geotechnical Considerations

The ULS seismic demand for geotechnical considerations, including PGA, representative (effective) earthquake magnitude and number of cycles, should be derived in accordance with the requirements of Module 1 of the Earthquake Geotechnical Engineering Practice series (NZGS/MBIE, 2016).

C3.10 Other Issues

C3.10.1 Site-specific probabilistic seismic hazard analysis

Site-specific probabilistic seismic hazard analyses should be completed in accordance with the requirements of NZS 1170.5:2004 and Module 1 of the Earthquake Geotechnical Engineering Practice series (NZGS/MBIE, 2016) as appropriate. The constraints noted in the Verification Method B1/VM1 (for New Zealand Building Code Clause B1 Structure) regarding the results from a site specific hazard analysis apply.

C3.10.2 Incorporation of the structural performance factor, S_p

The appropriate value of the structural performance factor, S_p, needs to be used when assessing the ULS seismic demand for structural considerations. This may require different values for S_p depending on the level of nonlinear deformation possible from the aspect under consideration, as determined in accordance with NZS 1170.5:2004 and this section.

S_p may be used either to reduce the demand spectral values calculated above (this is the approach adopted in NZS 1170.5:2004) or used to enhance the global capacity as assessed later in these guidelines. If the latter option is used, then for the purposes of establishing the ULS seismic demand S_p would need to be taken as 1.0.

As S_p is dependent on the structural ductility available it is likely that this factor will only be able to be set once the available global ductility has been determined from the global deformation capacity of the building.

S_p is not used for geotechnical considerations.

C3.10.3 Application of ULS loading (actions)

The direction of application of the specified actions and the allowances for accidental eccentricity should meet the requirements of NZS 1170.5:2004, Clause 5.3.

Where the actions for an element are influenced by more than one direction of loading (e.g. a corner column in a moment resisting frame building) and the load on the element cannot be limited by a yielding mechanism, the application of the ULS actions may be as for a nominally ductile structure.

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PART C

Geotechnical Considerations C4

Document Status

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1	July 2017	Initial release

This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

This document may be downloaded from www.EQ-Assess.org.nz in parts:

1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

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Please go to www.EQ-Assess.org.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.EQ-Assess.org.nz

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C4. Geotechnical Considerations

C4.1 General

C4.1.1 Scope and outline of this section

This section provides guidance on the geotechnical considerations for a Detailed Seismic Assessment (DSA). It provides tools to:

- identify the level of influence that ground behaviour (e.g. soil deformation or specific geotechnical hazards such as slope instability) may have on structural performance during earthquake shaking and,
- where possible, to quantify these effects and provide an appropriate level of input to the overall assessment.

All DSAs are expected to include consideration of geotechnical influences on the building's structural behaviour, and will likely require some geotechnical input to the DSA process, Steps 1, 2 and 3, outlined in Figure C1.1 of Section C1. However, the level of consideration will be a function of the detail required for the assessment and the likely sensitivity of the building's seismic behaviour to the geotechnical conditions (assessments are categorised as either "structurally dominated", "interactive" or "geotechnically dominated" for this purpose, as outlined in Section C1).

The geotechnical assessment of earthquake performance of existing buildings requires a high degree of experience**,** competence, local knowledge and engineering judgement to properly:

- understand the scope of work required
- understand the likely vulnerabilities of the soil-structure system being assessed, and
- interpret and act on information acquired during the steps of the assessment process.

The geotechnical assessment is to be led by a CPEng (Geotechnical) with appropriate experience and specific training in seismic assessment.

The approach outlined in these guidelines for including the consideration of geotechnical issues in the DSA represents a fundamental change from the traditional approach to considering these issues for new building design. Accordingly, a geotechnical engineer will need to carefully consider the material in this section to make sure this approach is understood.

The lead engineer (who will likely be a structural engineer) will also need to be familiar with this section as significant interaction between the geotechnical and structural engineer during a DSA is considered essential.

This section contains particular guidance on:

- timing and scope of input, including an outline of the respective roles of the geotechnical engineer and structural engineer depending on the nature of the project
- the approach to be taken for the inclusion of geotechnical issues
- development of an appropriate ground model

- identification and screening of common geotechnical hazards (geohazards) related to seismic activity that are relevant to life safety in structures and the manner in which geohazards from outside the site are dealt with in terms of influencing the earthquake rating for the building
- provision of input to soil-structure interaction (SSI) models and consideration of SSI in seismic assessment
- assessment of geotechnical aspects of foundation behaviour
- inputs to the calculation of *%NBS* (typically in a form relating to geotechnical influences on the assessment of the structure's probable capacity), and
- reporting and peer review.

As outlined in Part A and Section C1 the earthquake rating is not intended to cover issues that arise from outside the site. This includes the effect of adjacent buildings and geohazards. Therefore, while aspects such as fault movement away from the site, slope failure onto a building, rockfall from above, and tsunami are important to note (where known) from a holistic hazard point of view, they should not be included in the assessment of the earthquake rating for the building. This is similar to the approach taken when rating a building when the neighbouring buildings could present a hazard to the building bring assessed.

Note:

The Canterbury earthquake sequence of 2010-11 triggered widespread liquefaction across much of Christchurch as well as rock slides, rockfalls and cliff collapse and other forms of slope instability in the Port Hills, affecting tens of thousands of buildings. About half of the NZ$40 billion total economic loss from these earthquakes (New Zealand Treasury, 2013) could be attributed to the geotechnical impacts caused by liquefaction and rock mass instability.

However, while seismic assessments may include economic considerations, it should be remembered that the assessment of a building's earthquake rating under these guidelines is focussed on those aspects, including geotechnical influences, which will potentially lead to a life safety issue for building occupants and the public outside the building, and damage to adjacent property.

The assessing engineer should be mindful of the differences between assessment and design. In design the focus is on life safety and serviceability, with the objective of providing a "reliable" solution. Assessment focusses primarily on life safety (damage to adjacent property also requires consideration), and has the objective of developing an understanding of the building's expected behaviour in seismic events. Key principles regarding the differing focus and levels of conservatism ("reliable" for design and "probable" for assessment) are set out in Section C4.5.

As the science and practice of geotechnical earthquake engineering continues to evolve it is intended that these guidelines and the joint New Zealand Geotechnical Society/Ministry of Business Innovation and Employment modules (described in Section C4.1.2 below) will be updated periodically to incorporate new advances in the field. However, these updates will, naturally, lag behind the very latest advances. It is important that users of this document familiarise themselves with the latest advances and amend this guidance appropriately.

Note:

Additional material can also be found in the appendices to this section. This material is intended to supplement the material in the modules and provide information/discussion on issues that are particularly relevant to assessment rather than design, which is the primary focus of the modules. The material in some of the appendices is shown as "interim guidance" indicating that the guidance given does not yet appear in the modules.

A comprehensive bibliography and list of references is provided at the end of this section. Engineers are expected to be familiar with the relevant documents and to know what is important for the seismic assessment of existing buildings, particularly as this relates to life safety aspects.

C4.1.2 Relevant publications

C4.1.2.1 New Zealand geotechnical guidance

The New Zealand Geotechnical Society (NZGS) and the Ministry of Business, Innovation and Employment (MBIE) have jointly developed a series of modules for earthquake geotechnical engineering practice ("the NZGS/MBIE modules"). These modules have been published by MBIE as guidance under section 175 of the Building Act 2004 and are summarised in Table C4.1.

While the NZGS/MBIE modules relate primarily to new building design, many of the principles they contain are relevant to the seismic assessment of existing buildings. It is the intent that the requirements set out in these modules are used as the basis for assessment, with appropriate adjustments to reflect the differences between design and assessment outlined in these guidelines (e.g. in the treatment of uncertainties).

Note:

The information regarding the status of each NZGS/MBIE module was correct at July 2017. Please check at www.nzgs.org for updates.

Table C4.1: Summary of joint NZGS/MBIE modules in the earthquake geotechnical engineering practice series

NZGS/MBIE module (publication date)	**Description**
1. Overview of the guidelines (March 2016)	• Provides an overview of the module series • Introduces the subject of geotechnical earthquake engineering, provides context within the building regulatory framework, and provides guidance for estimating ground motion parameters for geotechnical design • Includes guidance on a number of geohazards, including fault rupture
2. Geotechnical investigations for earthquake engineering (November 2016)	• Guidance on planning geotechnical site investigations • Detailed description of various techniques available for sub-surface exploration; discussion of advantages and disadvantages of each • Describes that the primary objective is to understand the ground conditions for the project being undertaken
3. Identification, assessment and mitigation of liquefaction hazards (May 2016)	• Introduces the subject of soil liquefaction; describes the various liquefaction phenomena including lateral spreading • Includes discussion on clay soils and volcanic soils

NZGS/MBIE module (publication date)	**Description**
4. Earthquake resistant foundation design (November 2016)	• Discusses foundation performance requirements during earthquakes in the context of New Zealand Building Code requirements • Describes the different types of foundations in common use and includes a strategy for selecting the most suitable type based on necessary site requirements for each Note: Module 4 is an important reference for the assessment of existing structures. However, not all load and resistance factor design (LRFD) requirements for new design are relevant to the assessment of existing buildings. See later in this section for more on this topic.
5. Ground improvement of soils prone to liquefaction (May 2017)	• Considers the use of ground improvement techniques to mitigate the effects of liquefaction, cyclic softening, and lateral spreading at a site, including the effects of partial loss of soil strength through increase in pore water pressure during earthquake shaking • Guidance on assessing both the need for ground improvement and the extent of improvement required to achieve satisfactory performance for new design and for improvement of existing buildings
5a. Specification of ground improvement for residential properties in the Canterbury region (November 2015)	• Guidance on what should be included in a technical specification when designing and constructing ground improvement for liquefaction mitigation purposes. Four ground improvement techniques are covered: densified crust, stabilised crust, stone columns, and driven timber piles. Note re Modules 5 and 5a: The application of ground improvement methods to enhance the safety of existing buildings may be limited, but important principles are covered in these modules that will lead to greater understanding of dynamic soil behaviour and effects on foundation performance.
6. Earthquake resistant retaining wall design (May 2017)	• Seismic considerations for design of retaining walls Note: MBIE's Guidance on the seismic design of retaining structures for residential sites in Greater Christchurch (Nov 2014) is an existing source of information on retaining walls that is informative for existing structures.
7. Landslides and rockfalls (Planned for future development)	• Will consider landslide and rockfall hazard assessment and mitigation including earthquake effects. Note: GNS Science's wealth of reporting on the Port Hills soil and rock slope stability in the Canterbury earthquake sequence is informative for landslide and rockfall hazard assessment in other parts of New Zealand.

C4.1.2.2 US geotechnical guidance

ASCE 41-13 (2014) – Foundations and geologic site hazards

ASCE 41-13 (2014) Chapter 4 Foundations and Geologic Site Hazards provides useful additional information with respect to the assessment of existing buildings to supplement that provided in these guidelines and the NZGS/MBIE modules.

Chapter 4 of ASCE 41-13 (2014) presents general requirements for consideration of foundation load-deformation characteristics, seismic evaluation and retrofit of foundations, and mitigation of seismic geologic site hazards. It covers:

- definition of seismic geologic site hazards
- data collection for site characterisation
- procedures for mitigation of seismic geologic site hazards
- soil strength and stiffness parameters for consideration of foundation load-deformation characteristics

- procedures for consideration of SSI effects
- seismic earth pressures on building walls, and
- requirements for seismic retrofit of foundations.

Note:

Care is necessary when applying guidelines from other jurisdictions to ensure that the overarching philosophies are consistent. For example, the New Zealand approach is heavily focused on life safety and uses probable (mean) capacities to determine how a building may rate against minimum Building Code (B1) requirements.

Soil-structure interaction (SSI)

There are a number of relevant US references regarding the modelling of SSI effects for the design of new buildings (e.g. NIST GCR 12-917-21, 2012a; FEMA P-1050-1, 2015) and seismic evaluation of existing buildings (ASCE 41-13, 2014).

These documents provide a modelling approach and parameters for foundation flexibility, kinematic effects (i.e. base slab averaging and embedment effects) and foundation damping.

Note:

While the SSI modelling principles are generally applicable to the New Zealand context, the use of SSI to reduce the seismic demand using SSI damping and kinematic effects is not provided for in these guidelines although some aspects of SSI damping could be considered to be included in the NZS 1170.5:2004 structural performance factor, S_p, for the building as a whole. If engineers elect to reduce seismic demand using damping resulting from SSI and kinematic effects (an alternative solution to these guidelines), S_p is likely to require amendment accordingly and care will be necessary to reflect the high level of potential uncertainty in such assessments.

C4.1.3 Definitions and acronyms

CPT	Cone penetration test
Critical structural weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be potential critical structural weaknesses.
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines.
FE	Finite element (refer to Section C4A.3.6)
Geohazard	Geotechnical hazards
Geotechnically dominated	One of three defined project categories, in which the structure response is likely to be governed by geohazards and/or ground behaviour. Step change is often a characteristic of the ground and foundation performance in a geotechnically dominated project.
Interactive	One of three defined project categories, in which geohazards, soil nonlinearity and SSI may have an influence on the critical structural mechanism(s)
LRFD	Load and resistance factor design
MMI	Modified Mercalli Intensity

M-O equation	Mononobe-Okabe equation (refer to Appendix C4B)
MSE	Mechanically stabilised earth
PGA	Peak ground acceleration
Probable capacity (of a foundation/soils)	Assumed probable resistance (i.e. strength) and probable deformation capacity of a foundation/soils/geohazard. The probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Probable deformation capacity/limit δ_{SC} or δ_{L}	The maximum deformation (δ_{SC} or δ_{L}) a foundation can tolerate while continuing to provide resistance R or R_{R} as appropriate
Probable strength (capacity) R	Ultimate geotechnical strength capacity or nominal resistance. Evaluated as it would be for design (refer to NZGS/MBIE Module 4: *Earthquake resistant foundation design*).
Project categories	Assessments are categorised as either structurally dominated, geotechnically dominated or interactive depending on the significance of potential geotechnical influences on the structure (refer to Section C1)
Resistance	Restraint that a foundation provides at a specific level of deformation or level of shaking. Resistance increases with deformation to the maximum value R. See "Probable strength (capacity) R.
Severe structural weakness (SSW)	A defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge
Simple Lateral Mechanism Analysis (SLaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (pushover) relationship for the building as a whole
Serviceability limit state (SLS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings
SPT	Standard penetration test
SSI	Soil- structure interaction
Step change	The point at which the behavior of the structures, the ground or foundation is considered to abruptly deteriorate/reduce
Structural weakness (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100*%NBS*. Note that an aspect of the building structure scoring less than 100*%NBS* but greater than or equal to 67*%NBS* is still considered to be a structural weakness even though it is considered to represent an acceptable risk
Structurally dominated	One of three defined project categories, in which the structural response is unlikely to be significantly influenced by geohazards, foundation soil nonlinearity or SSI
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings
XXX%ULS shaking (demand)	Percentage of the ULS shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.

C4.1.4 Notation, symbols and abbreviations

Symbol	Meaning
%NBS	Percentage of new building standard as calculated by application of these guidelines
A_{loop}	Area contained within the hysteretic curve
B	Width of foundation
c	Soil cohesion
G_{sec}	Equivalent secant modulus
H	Wall height
k_{h}	Earthquake acceleration design coefficient (calculated using W = 1)
R	Ultimate geotechnical resistance/strength capacity (Probable Strength)
$R_{\text{d}} = \phi_{\text{g}} R$	Reliable geotechnical resistance/strength capacity used for design, where ϕ_{g} is the geotechnical strength reduction factor and R is as defined above
R_{R}	Probable residual resistance strength capacity after a step change
S_{u}	Undrained conditions of embedded cantilever walls
S_p	Structural performance factor associated with the detailing and assessed ductile capability of the system as a whole. Determined in accordance with NZS 1170.5:2004. Refer to Section C3.
γ_{c}	Expected amplitudes of shear stress and shear strain respectively
δ_{cap}	Expected limiting deformation
δ_{SC}	Predicted deformation at a step change
ξ_{soil}	Equivalent viscous damping ratio
τ_{c}	Expected amplitudes of shear stress and shear strain respectively
ϕ	Strength reduction factor
ϕ_{g}	Geotechnical strength reduction factor
γ	Unit weight of the backfill

C4.2 Roles and Responsibilities

C4.2.1 General

The roles and responsibilities for structural and geotechnical engineers are outlined in the following sections, together with suggestions on the suitable level of experience for geotechnical engineers involved in DSAs. This is followed by a summary of the roles and responsibilities that can be considered to apply based on the project categorisation; i.e. taking into account the potential impact of the geotechnical hazards on the building structure behaviour.

The effective assessment of structures starts with effective communication between the client/owner/tenant, the structural engineer and the geotechnical engineer (Oliver et al., 2013). A collaborative approach between all parties is essential so that the scope of work undertaken and the final assessment is appropriate for its intended purpose.

A common understanding of the expectations, roles and requirements of each team member at the outset of an assessment is important. Developing an appropriate brief that recognises the potential impact of geotechnical issues will likely require collaboration between the geotechnical engineer and the structural engineer and is an important step in the assessment process (refer to Section C1, DSA process Step 1).

While in some cases the geotechnical input to an assessment may be limited, in many instances the ground and its interactions with the structure at increasing levels of shaking intensity can be complex and nonlinear. In these situations specialist geotechnical advice and close collaboration between the structural and geotechnical engineer during the entire assessment process will be required. Some projects may also warrant special studies, e.g. a site-specific seismic hazard assessment and/or site response, which will require specialist input.

The early decisions regarding the potential impact of geotechnical issues and the complexity of the geotechnical assessment that is warranted to address these will be under the influence of the lead engineer, who will more than likely be a structural engineer. If there is any question regarding whether ground conditions may influence the behaviour of the structure, the lead engineer should seek geotechnical advice, at least as part of formulating the scope of the assessment. This is important as there are a number of geohazards that can have a significant effect on a building's performance but may not be readily apparent to a non-geotechnical engineer.

Note:

All structural assessments are expected to include some consideration of the influences the ground behaviour and foundation systems can have on structural performance. Hence, geotechnical considerations are integral to the DSA process and in particular Steps 1 to 3 (refer to Section C1). Depending on the ground conditions, foundation types and the level of detail of the assessment, the geotechnical input to an assessment may vary significantly.

As this will potentially influence the project briefing, the assessing engineer liaising with the client at the outset should be experienced and aware of the range of interaction that may be required between the structural and geotechnical engineering disciplines.

C4.2.2 Structural engineer's role

The structural engineer:

- is typically the lead consultant for the assessment
- will assess if specialist geotechnical input is required (in most instances in consultation with a geotechnical engineer)
- is responsible for liaison and reporting between the assessment team (structural and geotechnical) and the client. This should include involving the geotechnical engineer with client meetings when appropriate. For example:
  - at briefing meetings so the geotechnical engineer can hear and understand the client's needs and drivers, and
  - at other meetings so the geotechnical engineer can present conclusions, describe uncertainties, respond to questions on geotechnical aspects, and allow for the structural-geotechnical interaction required
- works collaboratively with the geotechnical engineer, and
- identifies structural forms and details which could potentially make the structure sensitive to soil and/or foundation performance.

Note:

At the outset of a project it is important that the structural engineer is aware of potential geotechnical influences and makes the client aware of the potential need for, and value of, the input of a geotechnical engineer at various stages of the project. It should be expected that the scope of the geotechnical input may increase as the assessment proceeds and the impact of geotechnical issues on the expected behaviour of the building becomes clearer.

C4.2.3 Geotechnical engineer's role and required experience

The geotechnical engineer:

- provides advice relating to SSI effects, geohazards and soils as they relate to foundation behaviour
- provides advice relating to geotechnical uncertainties
- recognises when the project would benefit from the geotechnical engineer's involvement with client communication (meetings) and discusses this with the structural engineer if so, and
- works collaboratively with the structural engineer.

The level of advice and judgement that will often be necessary in this role requires knowledge of:

- local ground conditions and geohazards
- the earthquake behaviour of soil and rock
- the interactions and behaviour of building/foundation/soil systems and how these may influence the performance of structures in earthquakes, and
- soil-spring characterisation.

The advising CPEng geotechnical engineer must have relevant experience in geotechnical foundation and earthquake engineering (refer also to the NZGS/MBIE modules) and must have completed training in the assessment of existing buildings in accordance with these guidelines so there is confidence that the underlying principles and approach to assessment taken in these guidelines are understood.

Alternatively, the work may be undertaken by a geotechnical engineer with guidance and appropriate review from a CPEng geotechnical engineer with the experience and training described above.

C4.2.4 Roles by project category

C4.2.4.1 General

On completing Step 3 of the DSA process it is expected that the significance of geotechnical influences will be understood such that project can be categorised as either structurally dominated, interactive or geotechnically dominated as indicated in Figure C4.1 (refer to Section C1 for a description of the project categories and the process).

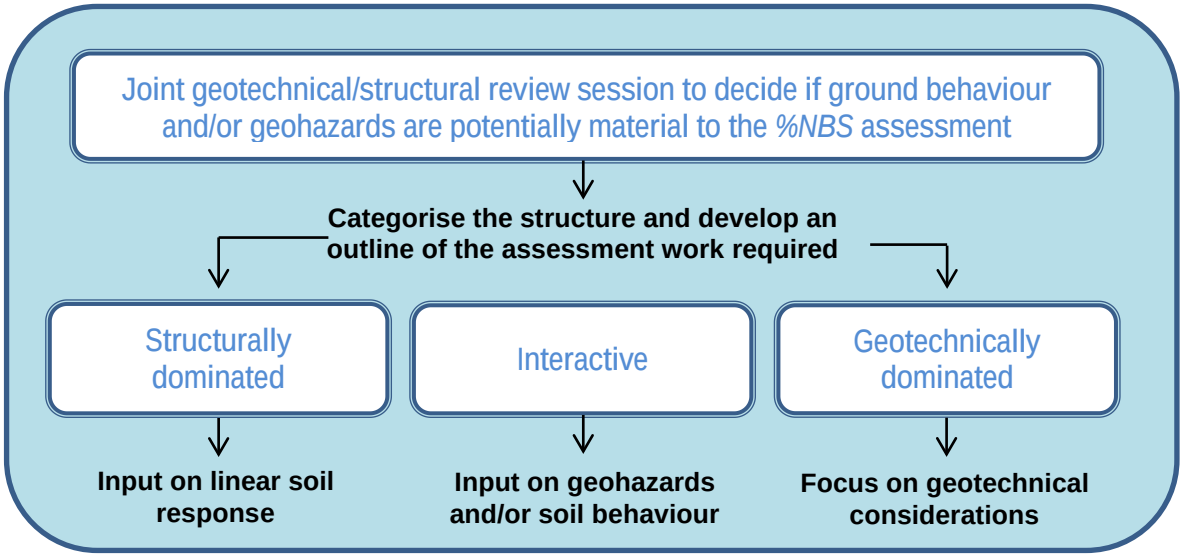


Figure C4.1: Project categorisation to reflect potential impact on the assessment of geotechnical issues

The guidance given below conveys the expected differences in scope for each project category. Specific project requirements will be determined at the outset and may vary as the project progresses.

C4.2.4.2 Structurally dominated

For structurally dominated projects, the structural/geotechnical collaboration should be sufficient to convey the general characteristics of the ground model and to develop an understanding and agreement that the probable range of geotechnical parameters are unlikely to significantly influence the behaviour of the structure.

The geotechnical parameters to be provided include:

- site seismic subsoil class
- near fault (as defined in NZS 1170.5:2004) assessment, and
- soil foundation stiffness (reported as a range of linear spring stiffnesses) and probable resistance available/strength (capacity).

The structural analysis is to include:

- sensitivity analysis across the range of parameters provided. To be "structurally dominated" it will be necessary to conclude that the structural analysis is not likely to be sensitive to the choice of parameters across this range, and
- a feedback loop to the geotechnical engineer, i.e. discussion of the results and conclusions of the analysis with the aim of verifying that geotechnical parameters have been interpreted and applied as intended and expected.

C4.2.4.3 Interactive

Interactive projects generally require substantially more detailed geotechnical input. Significant interaction is expected between the geotechnical and structural engineering disciplines.

A staged approach should be employed, with structural/geotechnical collaboration and re-evaluation on completion of each stage to check that:

- geotechnical parameters have been applied as intended, with results as expected, and
- investigation and analysis is targeted and appropriate for specific building vulnerabilities.

C4.2.4.4 Geotechnically dominated

Geotechnically dominated projects are expected to include those where step change in ground and/or foundation behaviour can occur. In this category, significant interaction is expected between the geotechnical and structural engineering disciplines.

The geotechnical engineer defines the expected onset of the step change as a proportion of the shaking considered in an ultimate limit state (ULS) event (for %ULS shaking refer to Section C4.5.3). The structural engineer then confirms that a brittle structural step change directly follows the geotechnical step change and that this response occurs at a lower shaking level than any other (structural) mechanism.

The geotechnical engineer will convey the details of the geohazard anticipated to result in the critical mechanism. In some cases, spring-type representation of the ground may not be required as the criticality of the geohazard can be defined without detailed structural analysis.

Typically, the emphasis will be on details of the critical geohazard. For example, this may be by an estimate of settlement or displacement from liquefaction or lateral spread. A staged approach can be employed, with re-evaluation on completion of each stage so that investigation is targeted at valid vulnerabilities and gaps in knowledge, as appropriate.

C4.3 Assessment Process

C4.3.1 General

As the seismic assessment of a building should consider the interaction of the soil, foundation and structure, this requires collaboration between the geotechnical and the structural disciplines (as outlined in the previous section).

Figure C4.2 illustrates the three key stages in this process:

- Stage 1 – project definition
- Stage 2 – assessment (including the geotechnical desktop study and geotechnical analysis and assessment), and
- Stage 3 – reporting within the DSA.

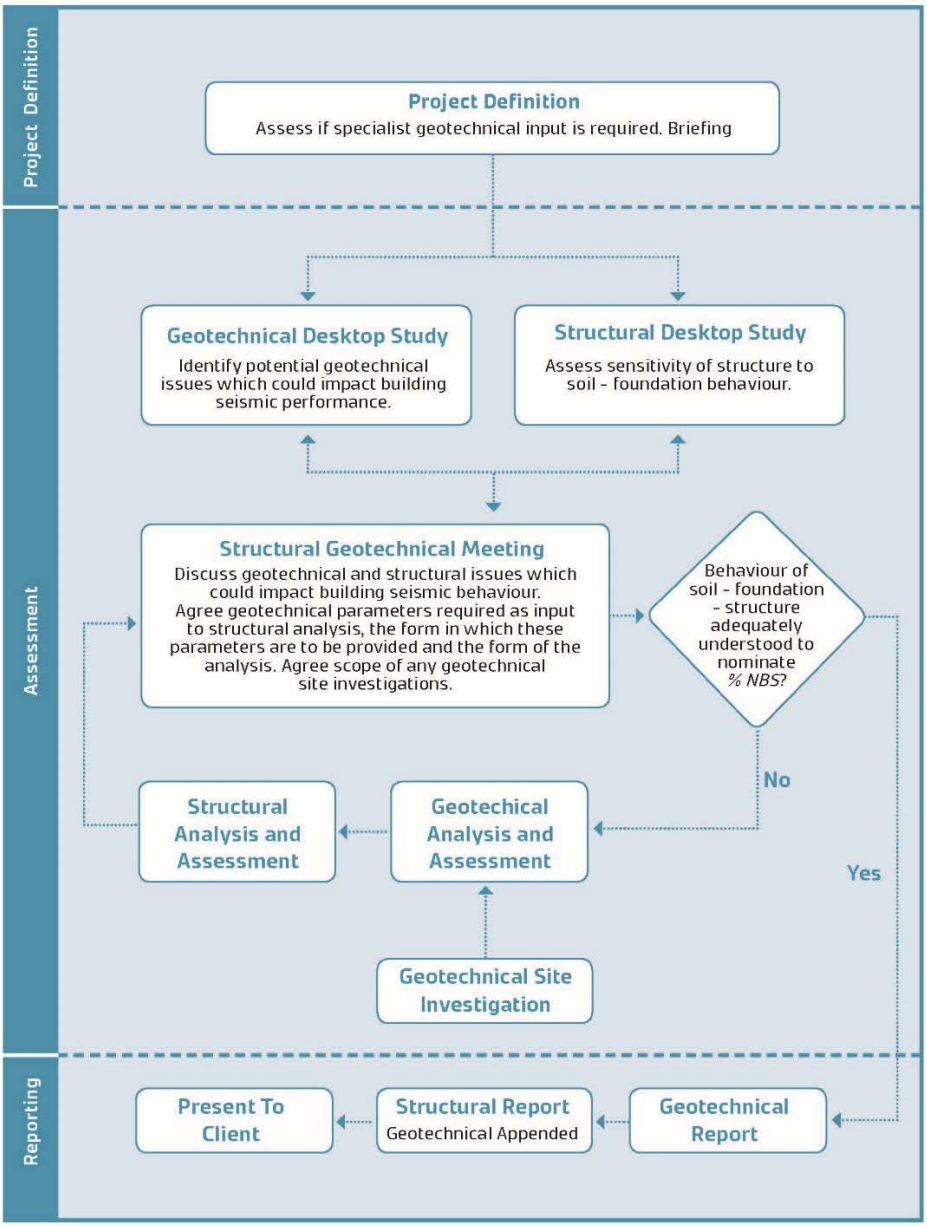


Figure C4.2: Project definition, assessment and reporting stages

These stages are outlined below and discussed in more detail in later sections.

C4.3.2 Stage 1 – Project definition

This first stage of the process outlined in Figure C4.2 is the initial review by the structural engineer, preferably in collaboration with the geotechnical engineer, to assess whether specialist geotechnical input is required and the likely scope of that work.

This involves:

- review of historic drawings and building records
- consideration of the ISA report, where available
- local knowledge of the site, ground conditions and groundwater regime
- judgement/experience
- the client's requirements, and
- initial consideration of potential geohazards and SSI effects, reliability of soil-foundation support and associated uncertainties in the ground model, and the level of sensitivity of the structure to the soil-foundation behaviour.

Note:

Situations where no specialist geotechnical input may be required are where geohazards are absent or are not potentially influential or governing for structural life safety, soil-foundation (SSI) behaviour is well understood and is reliable, and the assessment is expected to be "structurally dominated". However, it is likely that some degree of specialist geotechnical input will be required to confirm that geotechnical issues are not influential. The scope of work for the geotechnical engineer may vary as the assessment proceeds and potential influences on the building behaviour become clearer.

C4.3.3 Stage 2 – Assessment

C4.3.3.1 Desktop study

The initial part of the assessment involves separate preliminary geotechnical and structural desktop investigations.

The geotechnical desktop study is to identify potential geotechnical issues that could affect the building's seismic behaviour. Section C4.4 provides guidance on undertaking this desktop study and reporting its conclusions.

The output of the geotechnical desktop study should include:

- a sketch (cross section) and information to describe the inferred ground model, including the soil profile
- a list of geotechnical issues (including geohazards) that could influence the seismic assessment of the building, and
- an outline of uncertainties.

NZGS Module 2 - *Geotechnical investigations for earthquake engineering* provides guidance on undertaking a desktop study to inform likely site ground conditions and geohazards. For assessment of an existing building, information also needs to be collated and reviewed to inform the likely details of the existing foundations. This includes collating and reviewing historic drawings, and a site inspection to challenge the accuracy of those drawings. Conversations with people involved in the original construction or subsequent site work can be another valuable source of information.

C4.3.3.2 Structural geotechnical meetings

Once the structural and geotechnical engineers have carried out their desktop investigations, they then need to meet to share understanding from these and to explore the scope of subsequent investigation and analysis work (refer to Figure C4.2). An outline of these meetings and collaboration follows:

- **Inputs:**
  - conclusions of geotechnical desktop study (refer to Section C4.3.3.1)
  - results of geotechnical and structural review and analysis, and assessment to date.
- **Initial assessment:**
  - Consider the identified geotechnical issues in conjunction with understanding of structure. Discuss any potential geotechnical step change behaviours. Assess each issue with regard to its impact on *%NBS* and identify those issues which could be material to the assessment.
  - Consider what further analysis and assessment is required and how best to undertake this, focussing on those issues which could be material to the assessment.
  - Consider the current uncertainties associated with issues which could be material to the assessment. Consider how they are likely to impact on the reliability of the assessment of *%NBS* rating and, if appropriate, the cost/benefit of further investigations to reduce these uncertainties (refer to Section C4.4).
- **Output:**
  - agreement of updated list of geotechnical issues identified. Categorise these as:
    a) originating from outside the building footprint and thus not influencing the *%NBS* rating
    b) jointly agreed with the structural engineer as not being critical to the assessment of the *%NBS* rating, and
    c) to be specifically assessed
  - agreement on the project categorisation that best describes the potential behaviour of the building and therefore the type of assessment expected; i.e. structurally dominated, interactive, or geotechnically dominated
  - agreement of the analyses that will be carried out
  - agreement of what, if any, site investigations will be undertaken, and
  - agreement of the geotechnical parameters required as input to the structural analysis and the form in which these parameters will be provided.

Several meetings may be required before an output acceptable to all is achieved, as outlined below.

C4.3.3.3 Investigation, analysis and assessment iterations

As indicated in Figure C4.2 a series of iterations of investigation, analysis and assessment, with collaboration, may follow the initial meeting.

- The geotechnical engineer undertakes investigation, analysis and assessment, and reports the parameters required to the structural engineer.
- The structural engineer applies these parameters to the structural analysis and assessment.

- The structural and geotechnical engineers discuss the results of the analysis and assessment, and consider what further investigation and analysis is required to complete the assessment of *%NBS* rating.

This is an iterative process of reducing uncertainties and increasing understanding of potential building behaviour and, therefore, the *%NBS* earthquake rating. Each stage of the iteration is purposely targeted at those issues which could be material to the *%NBS* rating.

C4.3.4 Stage 3 – Reporting and peer review

As the assessment process (Stage 2) is collaborative and iterative, the geotechnical report cannot be completed until the assessment is finished. As outlined above, the geotechnical engineer will provide inputs during this process.

Refer to Section C4.8 for guidance on reporting and peer review.

C4.4 Site Characterisation

C4.4.1 General

Understanding the site's ground conditions and how these relate to the foundations, and communicating this adequately, is fundamental to the assessment of an existing building.

C4.4.2 The ground model

The geotechnical engineer should develop the ground model from information collated in the desktop study and site investigations, and update this throughout the investigation and assessment process as more information becomes available. However, the ground model only needs to be of sufficient detail to meet the overall needs of the assessment.

The ground model can be a cross section, and possibly a table, clearly summarising the inferred soil profile, groundwater level and foundation details, and presence of geohazards. As part of the ground model, it is also important to also highlight the uncertainties. Refer to Section C4.4.4.

This ground model then becomes the basis for discussions between the geotechnical engineer and the structural engineer. Its clarity will also aid in discussions with non-technical personnel (e.g. a building owner or tenant). As part of the ground model it is important to highlight the uncertainties.

C4.4.3 Identifying geohazards

Geohazards are to be identified as part of developing the ground model. The NZGS/MBIE modules provide guidance on evaluating seismic geohazards as indicated in Table C4.1, Section C4.1.2, including an overview of these in NZGS/MBIE Module 1 - *Overview of the Guidelines.*

Geohazards which could potentially affect the earthquake rating of a building include the following (NZGS/MBIE modules and appendices to this section that will aid the assessment are identified in brackets):

- soil/foundation compression/tension/lateral deformations with loading and the associated effects of deformation of the building (Module 4 and Appendix C4D)
- loss of ground strength and stiffness under the building – liquefaction (sandy soils) and cyclic softening (clayey soils), post liquefaction settlement (Module 3 and Appendices C4E and C4F)
- land instability causing loss of support for the building – lateral spread, slope instability, and instability of retaining walls affecting the support of the structure (Module 1 and Appendices C4B and C4C), and
- fault rupture under the building and complexities of near-fault effects.

The assessing engineer should consider if and how the relevant seismic geohazards could affect the building. The full range of earthquake demand (%ULS shaking) relevant to the assessment needs to be considered.

Note:

NZGS/MBIE Module 1 provides an overview of assessment of slope stability. A future module may be developed to consider this further. In the interim some guidance is provided in Appendix C4C.

NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design* provides valuable information for both design and assessment. Appendix C4B provides supplementary information to be considered in assessment of existing retaining walls and buildings. There is good coverage of retaining wall design in the literature (e.g. Kramer, 1996 and MBIE, 2014), and also insightful coverage of their seismic performance (Wood, 2014).

The location of the surface expression of any future fault movement may not be known with any certainty. It is important that the DSA appropriately discusses the uncertainties involved and the effect these have on the hazard and risks associated with future fault movements on the site.

Geohazards originating beyond the building footprint are not intended to be included in assessment of the earthquake rating. Nevertheless, they may be important considerations if a holistic seismic assessment is to be achieved. This principle is discussed above and in Part A and Section C1. Such geohazards include:

- tsunami or dam break and associated impact and inundation
- tectonic movement leading to flood inundation, and
- rockfall and slope or retaining wall instability from above leading to inundation.

Note:

NZGS/MBIE Module 1 provides general comments on Tsunami: it is not currently planned to include information about the assessment of tsunami hazard within this module series.

C4.4.4 Managing uncertainties

Any investigation of geotechnical issues will involve uncertainties. These should be evaluated and where necessary and appropriate, a targeted investigation programme developed to address them.

These uncertainties could relate to:

- ground conditions
- type and geometry of foundations (shallow, deep or mixed; size; founding level; beam connections and condition, etc.)
- condition of foundations, and
- nature of foundation subgrade (while new builds can include verification testing of foundation subgrades, such information is rarely available for existing buildings).

It is often not economically or technically viable to undertake investigations to resolve all these uncertainties in the assessment process. Due to access constraints these investigations can be considerably more expensive than equivalent investigations for a new build. Therefore, the geotechnical engineer and the structural engineer should collaborate to identify which of these uncertainties could have a material impact on the assessed seismic behaviour and earthquake rating of the building, and develop a targeted investigation in

response. Identified critical uncertainties related to the critical structural weakness (CSW), severe structural weaknesses (SSWs) and other low scoring structural weaknesses (SWs) are likely to require specific investigation.

Identifying critical uncertainties could include the geotechnical engineer identifying a number of possible scenarios for critical soil and foundation properties (and combinations of these), and the structural engineer testing these scenarios for their impact on the structural seismic assessment.

The geotechnical engineer's description of a scenario could include:

- assumed foundation type, size, depth and founding conditions
- assessed behaviour of this foundation (e.g. soil/foundation stiffness, probable strength (capacity), probable deformation limit)
- likelihood of these assumed conditions or worse/better existing, and
- the scope of investigations considered necessary to verify assumed conditions (i.e. if this scenario is based on conservative assumptions no investigation may be required to verify. If this scenario is based on optimistic assumptions, specific investigations will be required to confirm or modify these assumptions.).

In the first round of the process described above it would be appropriate to assume a scenario with geotechnical parameters which can be relied on without further site investigation (necessarily pessimistic), i.e. to test if these conditions are critical to the structure and if investigation is necessary.

C4.4.5 Site investigations

C4.4.5.1 General

NZGS/MBIE Module 2 - *Geotechnical investigations for earthquake engineering* provides guidance on desktop studies and physical investigations. This section of these guidelines should be read in conjunction with Module 2 as it provides additional guidance relating to existing buildings.

The first phase of the investigation, the desktop study (refer Section C4.3.3.1), allows an initial ground model to be developed and likely issues and uncertainties to be identified. If potential issues or uncertainties are identified which could be critical to the assessment of the building targeted physical investigations are likely to be required.

The purpose of the geotechnical investigation of an existing structure is to characterise the ground conditions and foundations that the building is supported on. This includes:

- seismic subsoil class (refer to NZS 1170.5:2004)
- ground conditions and liquefaction potential (refer to NZGS/MBIE Module 2)
- dimensions of existing foundations (refer to Section C4.4.5.2 below)
- foundation load/deformation behaviour (refer to Section C4.4.5.3).

C4.4.5.2 Dimensions of existing foundations

During the desktop stage available information relating to the existing foundations should be collated and reviewed. Sources of information include:

- historic drawings and geotechnical reports, potentially sourced from council property files, building owner's or designer's files

- local knowledge including discussions with those involved in the original construction or subsequent alterations and with the building maintenance personnel, and
- site inspection to check drawings and other information against site observations.

Physical investigation of foundations is sometimes necessary to confirm foundation dimensions and geometry. This may include local excavation around foundations or piles/pile caps by hydro-excavation or other excavation technique. Coring may be used to drill through foundations to confirm foundation dimensions, concrete condition and founding depth, and if extended below the foundation the condition of foundation soils. There are a number of non-intrusive investigation techniques which may provide alternative options or be used in conjunction with intrusive methods. These include the use of:

- a cover meter to check for reinforcement in foundations
- a magnetometer in an adjacent borehole or cone penetration test (CPT) to detect the toe level (or at least the base of reinforcement) in an adjacent pile
- down-hole or cross-hole seismic testing performed adjacent to a pile to detect the toe level (refer to FHA, 1998), and
- pile integrity test methods to estimate the length and condition of a pile.

These can offer relatively convenient and cost-effective investigation methods. However, calibration against independent (preferably physical) methods is recommended, particularly where structure performance is sensitive to results.

C4.4.5.3 Foundation load/deformation behaviour

Where more reliable information on foundation capacity and/or stiffness is required, it may be possible to undertake a load test on an existing foundation. Typically, this is undertaken by physically separating the building from the foundation by cutting through the pile and inserting a jack which then loads the pile against the building. There are published examples of this approach (e.g. Jury, 1993).

C4.5 Key Principles

Some key principles are embodied within the approach to assessment of geotechnical issues contained within these guidelines. These include understanding the objectives of assessment and the differences between these and those for design, the use of probable capacities and the modelling of the resistance versus deformation behaviour for geotechnical issues. These aspects are discussed below.

C4.5.1 Difference between assessment and design

In general terms, building assessment is not the same as design in reverse as they have different objectives and follow different approaches. This is particularly the case for consideration of geotechnical issues.

Seismic assessment of existing buildings is primarily concerned with life safety. Therefore, it is necessary to understand the mechanisms that may lead to partial or full collapse of the structure, as it is generally the failure of the structure and/or its parts that will lead to casualties. Serviceability issues associated with the onset of general damage are not the focus.

For design, the aim is to set limits for geotechnical parameters for which there is a high reliability that support will be achieved without excessive deformation. This is typically a conservative approach, but in new building design this conservatism can be provided for, in most instances, with little cost premium. However, retrofit of foundations in an existing building is typically a disruptive, often difficult and expensive exercise and, as a result, it is not practical to simply adjust the foundation size to meet normal design criteria that are known to be conservative. Therefore, a realistic assessment of the expected foundation/soil behaviour and how this interacts with the structure becomes very important when establishing how well the foundations, as detailed, are likely to meet the assessment objectives.

In design, load and resistance factored design (LRFD) is typically applied. Loads and resistances are factored to provide a level of reliability that yielding or failure of soil will not occur. This also is likely to control deformations. In an assessment this is typically replaced by a displacement-based approach. The acceptable performance for geotechnical behaviour is a function of the consequence of the geotechnical-induced deformation/loads on the superstructure's life safety performance. Typically, large deformations in the soil can be tolerated before life safety in the building becomes an issue. The exception is in the situation where the building structure may not be well tied together.

Gazetas (2015) presents the case for going beyond conventional seismic failure thresholds and provides case studies that illuminate the benefits and limitations of "rocking isolation", for example.

The process of assessment is often iterative and there can be limited geotechnical information available at the early stages while critical mechanisms are being identified for targeted investigation. However, where limited information is available it is important that "consistent crudeness" is applied to the modelling and assessment, i.e. to avoid reporting analysis to a degree of accuracy that is inconsistent with the uncertainty of the input parameters.

The uncertainties and unknowns associated with assessment are typically greater than in they are in design. Often in assessment the dimensions of the existing foundations are uncertain, and rarely is subgrade verification test data from construction available. Section C4.4 discussed these uncertainties and ways they might be managed.

Due to the inherent uncertainty in geotechnical engineering and, in particular, in geotechnical earthquake engineering, engineers needs to draw on precedent, empiricism and well-founded engineering judgement to arrive at likely ranges of ground and foundation deformation.

Note:

The precedent referred to above is not a precedent in terms of "this is how we have always done it" (i.e. ignoring SSI) but in terms of observed behaviour (i.e. case studies with comparable earthquake demand, structural system, loads and ground conditions). In this regard, the experiences of the Canterbury earthquake sequence of 2010-11 (and other well-documented international earthquakes) can be of benefit to the assessment process.

Sensitivity checks on the assumptions made will be an essential part of most seismic assessments. Depending on the sensitivity on the structural performance these checks might include the consideration of both upper and lower range soil strength/stiffness, the effect of different analysis methods, and soil behavioural models and their uncertainties.

The intent of the seismic assessment is to establish holistically the probable capacity of the soil, foundation and structural system. This is also different to what may be used for design. Refer to Section C4.5.2 for further discussion on the use of probable capacity in the context of the geotechnical assessment and the selection of suitable geotechnical parameters.

C4.5.2 Probable capacity for geotechnical issues

These guidelines are based on assessing the structural capacity of the building at a probable level. "Probable" for structure is considered as being at the expected or mean level. It is typically evaluated by using the determined/estimated mean (structural) material properties and setting the capacity reduction factors, applied for the purposes of design, to 1.

The concept of mean soil properties presents some difficulties in the geotechnical field. It may not be possible or appropriate to work with mean soil properties, for example, given the uncertainty and variance possible. At the same time, undue conservatism and the level of reliable behaviour aimed for in design, particularly around deformation capacity, is likely to be inappropriate for seismic assessment, as has been noted in previous sections.

To recognise this situation the following approach has been adopted in these guidelines for assessing the probable capacity/resistance for geotechnical issues. Geotechnical capacity in these terms includes both strength/resistance and deformation and is represented in terms of an assumed relationship between strength/resistance and the resulting deformation, which needs to consider potential behaviour often well into the nonlinear range.

The probable strength (capacity) is taken as the ultimate geotechnical strength as would be assumed in design (refer to NZGS/MBIE Module 4 - *Earthquake resistant foundation design*). In assessment a strength reduction factor is not applied, and the resistance deformation behaviour is assessed and modelled. Section C4.5.3 considers assessment and modelling of resistance/deformation.

C4.5.3 Resistance-deformation/shaking behaviour

C4.5.3.1 General

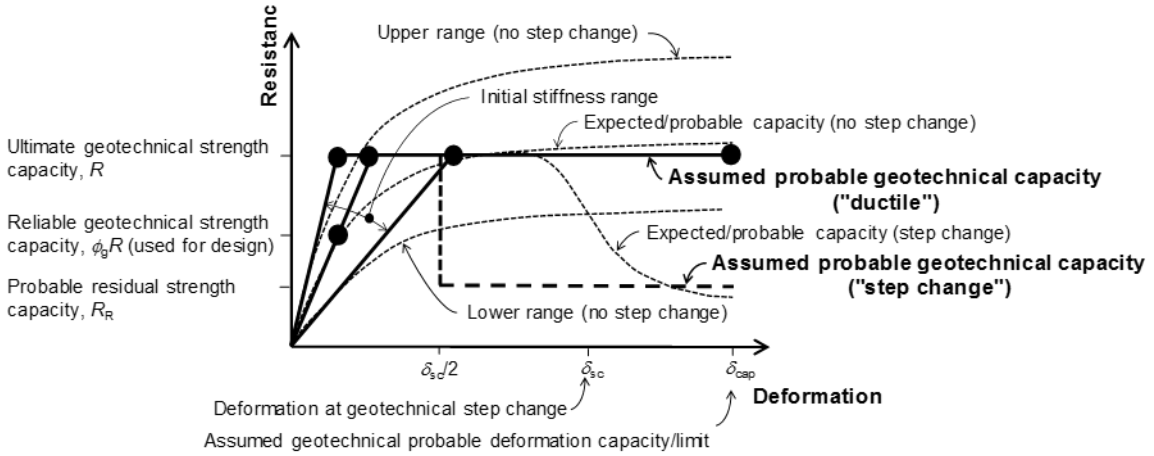
Consider the generic resistance-deformation and shaking relationships/models indicated in Figure C4.3. These might apply to the effect that a foundation soil, a foundation or a geohazard might have on the building, or how the resistance these provide to a building might be affected by increasing imposed deformation or earthquake shaking.

Figure C4.3(a) shows a generic relationship between resistance and increasing levels of deformation. The figure shows the probable geotechnical resistance models that are intended to be assumed for the situation where no "step change" in behaviour is expected and also when it is (refer Section C4.5.3.2 for a description of step change behaviour). The relevant features of these models are as follows:

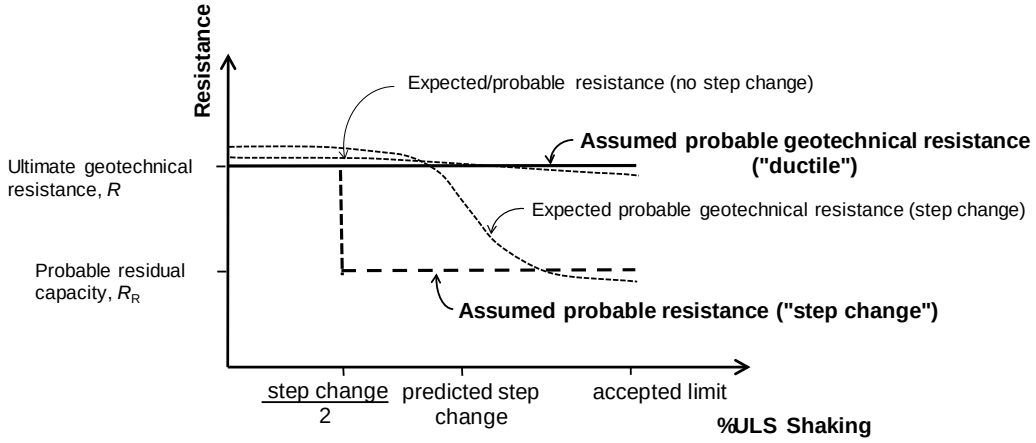
- A bilinear representation is considered adequate for most situations. This is referred to as "ductile" behaviour.
- The maximum resistance (i.e. probable strength capacity) is taken as the ultimate geotechnical strength capacity normally calculated for the purposes of design, but before application of the usual geotechnical strength reduction factors.
- The deformation limit of the model will typically be well beyond the deformations usually considered for design.
- When a step change in behaviour is expected it will be necessary to estimate the deformation at which this is expected and also to consider the probable residual strength capacity that might be available beyond the step change. In line with the assessment philosophy that has generally been adopted in these guidelines around step change behaviours, the deformation at which the step change is indicated is divided by 2 when defining the model. Beyond this halved deformation, the resistance is assumed to be limited to the residual capacity. The objective is to determine a *%NBS* score which has the resilience that is likely to be inherent in current new building design.

The resistance provided by some foundation soils or geohazards (e.g. liquefaction, slope stability, lateral spread) can be influenced by the dynamic effects of the earthquake shaking. Figure C4.3(b) shows a generic relationship between the resistance provided and increasing levels of shaking, illustrated here in terms of increasing %ULS shaking. This figure shows what is intended in the case of a predicted step change where resistance may be lost or significantly reduced, as the shaking level (intensity and duration) reaches a threshold value. A step change factor of 0.5 is also introduced to define this behaviour.

The uncertainties in the relationships/models could be large. It is recommended that the evaluation of the potential sensitivity of geotechnical issues assumes upper and lower ranges of initial stiffness (often twice and half respectively of the estimated values). The geotechnical engineer will need to advise the nature of the uncertainties and when sensitivity analysis of outcomes might be appropriate.



(a) Resistance versus deformation



(b) Resistance versus %ULS shaking

Figure C4.3: Generic resistance-deformation versus shaking relationships for geotechnical issues

When a residual capacity is expected to be maintained after a step change, the geotechnical engineer can either:

- assume the probable resistance is the residual resistance from the outset, or
- if the assessment is to be based on the pre-step change resistance, the deformation should be limited to 50% of the predicted deformation at the step change, as outlined above.

The geotechnical engineer should also nominate the probable deflection/deformation capacity (limit), δ_{cap}, beyond which the relationship is not expected to be valid.

This method is a pragmatic approach to what can be complex issues. It recognises that the primary geotechnical issue is not usually the available level of resistance available but whether or not there is likely to be step change behaviour and whether or not a residual capacity is expected post the step change.

The evaluation of the score for a SSW involving a geotechnical issue may be treated in a similar fashion, but without the expectation of a residual capacity.

C4.5.3.2 Derivation of soil-foundation models

Section C4.5.2 and Figure C4.3 outline the general principles for modelling soil-foundation behaviour for seismic assessment. This section sets out the steps to derive the soil-foundation model parameters.

Step 1 - Qualitative assessment

The first step is a qualitative assessment of the likely soil-foundation behaviour. Is it "ductile" behaviour or could a "step change" be expected?

- Ductile behaviour may be assumed if a step change in resistance is not expected or the resistance is not expected to decrease by more than 20% over the extent of expected deformations.
- Table C4.2 below identifies soil-foundation types which could exhibit step change behaviour.

Step 2 - Selection of parameters

The following guidance is provided for evaluating parameters to be applied in modelling soil-foundation behaviour. In evaluating these parameters due consideration must be given to soil response to the shaking and the dynamic nature of the applied loading (cyclic and reverse loading, push pull). NZGS/MBIE Module 4 - *Earthquake resistant foundation design* considers these factors in its guidance.

Ductile behaviour

Refer to Figure C4.3(a). Ductile behaviour is to be modelled as elastic-plastic. To evaluate this simple model the engineer must establish the following parameters:

- ***R*, ultimate geotechnical (strength) capacity**: this is the assumed limiting resistance provided by the soil-foundation with increasing deformation. It is the same value as is assessed for design before the design strength reduction factor is applied. Strength reduction factors are not applied in assessment of an existing building. NZGS/MBIE Module 4 provides guidance on evaluating *R*.
- **initial stiffness:** the initial stiffness assumptions will rarely prove to be critical in a seismic assessment but stiffness values may be requested by the structural engineer for inclusion in the structural modelling. When requested it is recommended that a range be provided. If it proves critical to the assessment of the behaviour of the building, refinement of the top or bottom end of the range can be undertaken at a later stage. Table C4.3 provides guidance for evaluating initial stiffness for various soil-foundation types.

- **δ_L, deformation limit:** this is the deformation limit over which the soil-foundation can be assumed to provide resistance R. Beyond this limit a reduction of more than 20% in R could be expected, or behaviour cannot be predicted. Table C4.3 provides examples of evaluation of δ_L for various soil-foundation types.

Step change behaviour

Foundation soils that are likely to degrade significantly in strength when subjected to earthquake shaking cannot be considered as "ductile" and will require special consideration, involving both the geotechnical and structural engineer. Such behaviour can lead to sudden loss in building support once a threshold level of shaking is exceeded. The threshold may occur as a result of deterioration in the strength of the soil/foundation and/or deterioration in ability to provide support due to dynamic effects. This is referred to in these guidelines as "step change" behaviour and, if it is judged that it could lead to a significant life safety issue for the building, may result in the limiting score for the building that determines its earthquake rating. It is the identification of potential step change behaviour in the building behaviour that should be the focus of the geotechnical and structural engineer.

Step change may involve a deterioration in resistance to a residual value. In such cases it may be appropriate to carry out the assessment based on the residual strength. If the resistance prior to the step change is to be relied on or is necessary to prevent a significant life safety risk, allowance will need to be made in the scoring to provide confidence that the risk of the step change occurring is at an acceptable level.

In its static condition and during lower levels of earthquake demand the ground is assumed to remain in a competent, stable state. With increasing earthquake demand the ground can gradually deform but at tolerable levels, with the capacity at yield exceeding demand.

In the range of earthquake demand (i.e. combinations of magnitude and peak ground acceleration) being considered there can be a threshold point (or a narrow "bandwidth") up to which gradual ground deformations may have occurred but suddenly, at further increasing demand, the ground or foundation performance abruptly deteriorates. In these guidelines this is termed a "step change" in geotechnical behaviour. The abrupt transition in geotechnical conditions may or may not have significant consequences for the foundation's integrity or the structure's stability (Clayton et al., 2014).

Examples of features that can lead to a geotechnical step change are:

- liquefaction – elevated pore water pressure at lower levels of earthquake demand can occur in liquefaction-prone soils; but over a small "bandwidth" of earthquake demand liquefaction triggering can occur and lead to an abrupt loss of soil shear strength. The consequence can be abrupt, large foundation deformation. For shallow foundations, the step change may manifest as a severe rotation and/or settlement. The severity of the soil and foundation deformation could be significantly exacerbated if lateral spread can also occur.
- slope instability – soil and rock slopes can withstand earthquake shaking with little or no deformation. However, at elevated levels of earthquake shaking they can reach a point where mass movement (e.g. soil slope failure, rockfall or cliff collapse) is expected.

- retaining walls – as for slopes, retaining walls can withstand a degree of earthquake shaking with little or no deformation. However, with increasing earthquake shaking there can come a point at which the wall fails. A wall supporting a foundation could fail leading to a step change in foundation support and large deformations in the structure. Similarly, a wall retaining land upslope of a building could experience abrupt collapse.
- foundation element failure – pull-out of a foundation element, such as an anchor or pile in tension, has the potential to lead to a geotechnical step change. However, there will often be a residual capacity which can be relied on, or the additional deformation that occurs in the structure as a result is tolerable. Step change behaviour could also be experienced with compression loading and sensitive soils.

Failing slopes or retaining walls can either remove foundation support (if the slope or wall is downslope of the structure) or cause soil/rock/debris lateral impact on vertical structural support members. Falling soil/rock/debris can also have direct life safety impacts on life outside and within a structure (e.g. occupants impacted or buried by rockfall) but as noted above this will not affect the earthquake rating for the building itself.

The severity of foundation deformation and consequences for the structure's stability are a function of:

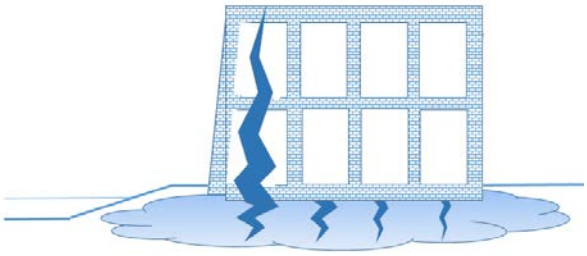
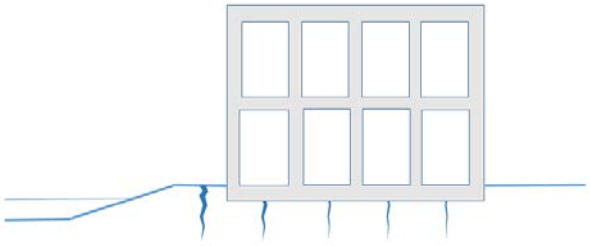
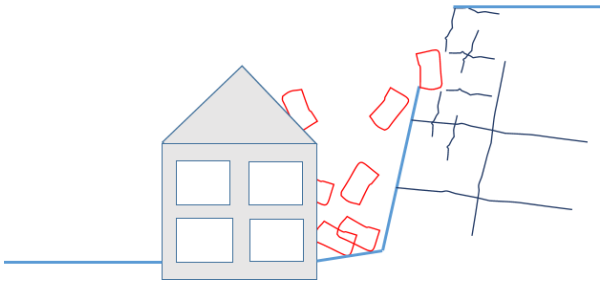
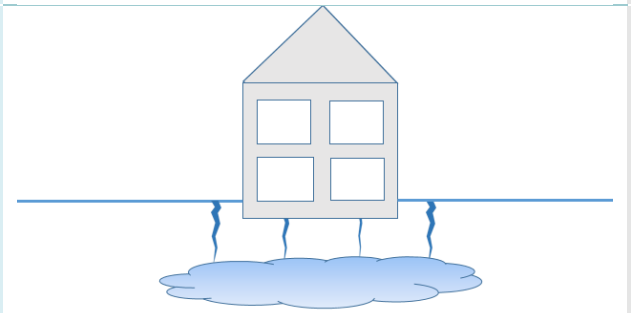
- the severity and nature of the ground deformation; how much of the structure's support system is affected, and
- the structure's resistance to foundation deformation or rupture.

In this regard, a structure on a mat foundation or well-tied footings is more resilient to ground deformation than a structure on discrete footings, although relatively high levels of differential settlement of individual footings may still be tolerable when the structure itself is well tied together.

Geotechnical step change will only be an issue for setting the earthquake rating if it in turn results in a step change behaviour of the building structure, i.e. a structural step change, and then only one that would result in a significant life safety hazard.

Table C4.2 provides some examples of buildings/sites and considers whether or not they have the potential for structural step change behaviour.

Table C4.2: Examples considering the potential for step change

Description		Step change potential?
Unreinforced masonry building on site subject to liquefaction and lateral (flow) spread		Likely to be a structural step change behaviour unless the structure above is well tied together
Building on site subject to coseismic slope movements		**Unlikely** to be a structural step change if the building and/or its foundation is well tied together
Light timber frame dwelling in a rockfall impact zone		Likely to be step change but **not** an earthquake rating issue
Light timber frame building on a site subject to liquefaction		**Unlikely** to be a structural step change

Note:

While many sites may be subject to seismic geohazards, these guidelines anticipate that few of these will result in a true step change in behaviour. In very few cases it is anticipated that a geotechnical step change will, in isolation, set the earthquake rating. More commonly, geohazards may tend to exacerbate pre-existing structural weaknesses or be shown not to have a direct effect on the life safety objective.

Refer to Figure C4.3. Step change behaviour can occur in two situations:

- when a rapid decrease in resistance is expected with increasing imposed deformation, and
- when a rapid decrease in resistance is expected at a particular earthquake shaking threshold.

In both situations it will be necessary to estimate the probable resistance available prior to the step change and the residual resistance available beyond the step change up to the limiting displacement δ_L and also either the deformation or the %ULS shaking at which the step change is predicted. Guidance for their evaluation is as follows:

- R, resistance pre step change: R is evaluated as discussed above and in NZGS/MBIE Module 4.
- deformation, $\delta_{SC,}$ or predicted %ULS shaking triggering step change: this is the deformation or intensity of shaking (%ULS shaking) at which the step change in soil-foundation behaviour is estimated to occur. In modelling, R is assumed to be available up to a deformation of $\delta_{SC}/2$ or %ULS shaking to trigger step change/2. Beyond these levels a residual resistance of R_R is assumed. The halving of the deformation (or %ULS shaking) to trigger the step change is to provide some resilience against the step change occurring. Table C4.3 includes guidance on evaluating $\delta_{SC.}$
- R_R, residual resistance: Table C4.3 provides guidance on evaluating R_R.
- δ_L, deformation limit: δ_L is evaluated as discussed above and in Table C4.3.

Note:

The factor of 2 applied above can be considered as a deformation margin that needs to be applied if reliance is going to be placed on the pre step change resistance/strength capacity.

Example parameters

Table C4.3 provides example parameters. These parameters are not to be relied on for a specific situation. The geotechnical engineer is to consider the soil conditions and foundation details that are appropriate for the particular project and undertake specific assessment of parameters with due consideration of the effects of shaking and dynamic loading. Reference should be made to NZGS/MBIE Module 4 for guidance.

Table C4.3: Indicative soil-foundation modelling parameters

Soil-foundation type	Ductile or step change behaviour	Example initial stiffness, displacement at load = R		Example deformation limit δ_L	Example trigger for step change δ_{SC}	Example residual resistance R_R
		"Stiff" end of range	"Soft" end of range			
Shallow pad or strip foundation on granular soil Foundation width B	Ductile	Elastic analysis based on short term (immediate) soil stiffness	10% of B	30% of B		
Shallow pad or strip foundation on cohesive soil Foundation width B	Ductile Sensitive soils could exhibit step change	Elastic analysis based on short term (immediate) soil stiffness	5% of B	15% of B		
Pile foundation in granular soil Pile base diameter B	Ductile	Lesser of elastic analysis of pile base, or 10 mm. 10 mm assumes load is resisted by shaft resistance alone.	10% of B	30% of B		

Soil-foundation type	Ductile or step change behaviour	Example initial stiffness, displacement at load = R		Example deformation limit δ_L	Example trigger for step change δ_{SC}	Example residual resistance R_R
		"Stiff" end of range	"Soft" end of range			
Pile foundation in cohesive soil Pile diameter B	Ductile Sensitive soils could exhibit step change	Lesser of elastic analysis of pile base or 10 mm. 10 mm assumes load is resisted by shaft resistance alone.	5% of B	15% of B But not greater than 75 mm for shaft resistance		
Screw pile	Ductile	½ X the displaced measured in representative load tests	2x the displacement measured in representative load tests	30% of B		
Grouted ground anchor in tension	Step change with displacement possible depending on loaded soil/rock type.	½ X the displacement measured in representative load tests	2x the displacement measured in representative load tests Or Bar/tendon elastic stretch assuming resistance distributed along full bond length, plus 10 mm			Depends on soil/rock type and method of anchor construction. Could be ½ of the peak resistance in rock.
Foundation in/on soils prone to liquefaction or cyclic softening	Step change possible. Liquefaction potential analysis required along with assessment of consequences of liquefaction to foundation.				Liquefaction triggering analysis. Extent of assessed liquefaction must be sufficient to compromise foundation capacity.	Analysis considering liquefied residual soil strengths
Foundation on or above a slope prone to underslip as a consequence of seismic shaking	Step change possible. Seismic slope stability analysis required.				Seismic slope stability analysis	Zero if slope evacuation from beneath the foundation is predicted to occur. Allow for reduced support adjoining slip scarp.

C4.6 Consideration of SSI Effects

SSI effects may have a significant influence on the seismic behaviour of a building and the way in which some mechanisms might develop in the structure. Accordingly, possible SSI effects should be considered as part of an assessment and a decision made on how detailed and complex the inclusion needs to be.

Engineers should note that it is important to consider the potential for the soil to be stronger/stiffer or weaker/softer and for this variability to be non-uniform in distribution. Similarly, imposed displacements or loads may be uniform or differential. Figure C4.4 illustrates a simple example of the range of structural responses as a consequence of the soil strength/stiffness adopted.

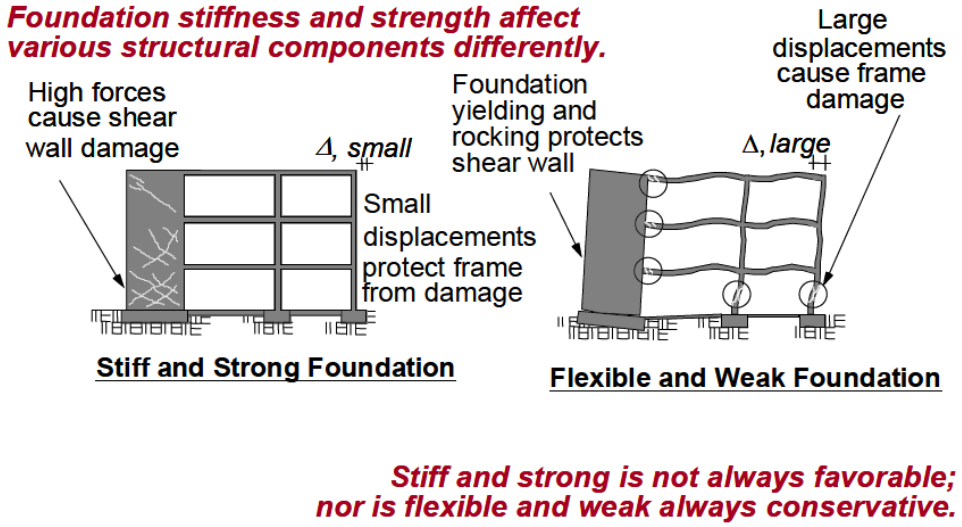


Figure C4.4: Influence of SSI on structural performance (figure adapted from Mahoney, 2005)

Assuming unrealistically stiff soil/foundations (e.g. fixed base assumptions) could result in an unrealistically low natural period of shaking for the structure (unrealistically high seismic loads) or underestimation of structural deformations. The converse also applies.

SSI effects are complex but can often be simplified for assessment; particularly initial screening to assess sensitivity of behaviour.

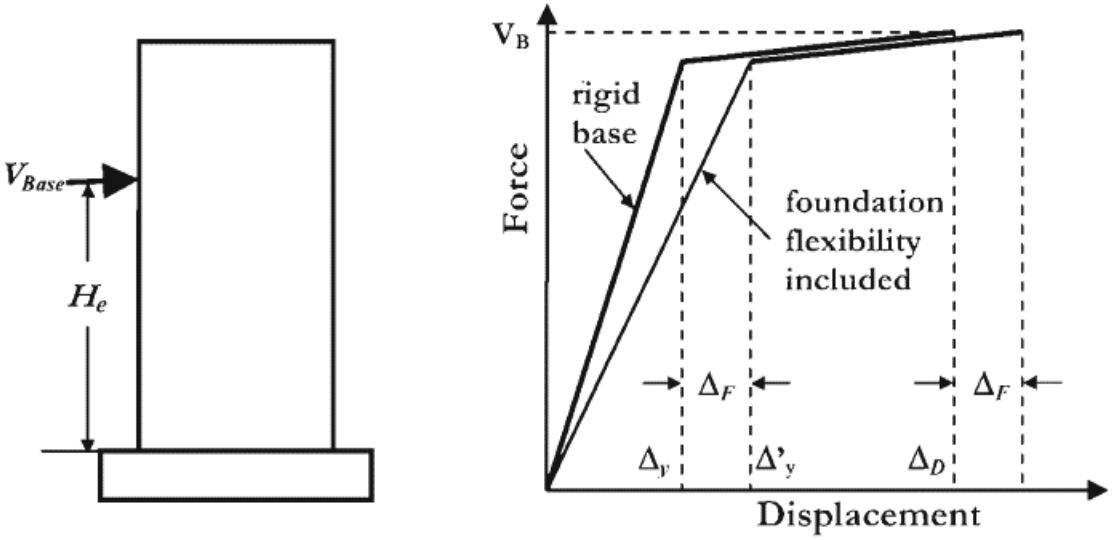
For example, this could be as simple as recognising that the soil support for a footing may not be rigid and reflecting on what this means for the rigidity of a supported column and its ability to receive flexural resistance/restraint at the base. This may influence the possible actions in the column and mechanisms that are possible in the structure. For this example it may be appropriate to at least consider the possibility of varying restraint, within appropriate bounds, when assessing the structure.

Simple hand checks can be undertaken collaboratively with the structural engineer to assess if the building is likely to be sensitive to the deformation demands from foundation flexibility (e.g. Millen et al., 2016). The amount of acceptable deformations for foundations generally depends primarily on the effect of the ground-induced lateral deformation on the structure and ultimately on the life safety hazard that can develop.

Note:

Foundation flexibility may increase the deformation at the soil-foundation interface which could affect the behaviour of the building through additional imposed inter-storey drifts on the gravity framing system. The foundation flexibility may also increase the yielding displacement and effectively reduce the achievable ductility of the system. Refer to Figure C4.5.

While the local effect of SSI should be considered (e.g. effect of soil flexibility on the support to the structure), any beneficial effects of foundation radiation damping and kinematic interaction should only be included in the SSI modelling if there is confidence in the assessment of the parameters used.



(a) Structure **(b) Force-Displacement**

Figure C4.5: Influence of foundation flexibility on displacement and ductility capacity in the structure

Complex analysis including direct nonlinear modelling of the soil and its interaction with the structure is possible and may be warranted in some situations. Table C4.4 provides some further guidance on when to use the next level of sophistication of SSI modelling. For further information on each of the SSI analysis options refer to Appendix C4A. However, in general, specific guidance on such analyses is outside the scope of these guidelines and reference will need to be made to other documents; e.g. NIST 2012a), NIST 2012b) and FEMA P-1050-1, 2015.

Table C4.4: SSI analysis options

SSI analysis option	When to use/not to use	Comments
Fixed base model – no SSI consideration	This should not be used for high rise buildings on piles or slender wall systems with shallow foundations.	The foundation structure will still need to be assessed by hand: • global overturning stability • yielding at the soil-foundation interface.
Simplified flexible base model using linear Winkler springs	Shallow foundations Core walls Basement/part basements	The superstructure needs to be assessed for a fundamental period considering both fixed base and flexible base, i.e. building period shift due to foundation flexibility is to be considered. Consider whether sufficient number of springs have been included.
Simplified flexible base model using compression-only or tension-only Winkler springs	Rocking/uplift foundations The use of tension-only elements in dynamic analysis has risks with respect to stiffness matrix spikes and loss of energy via over-damping.	Examples: Kelly, 2009 for rocking foundation and Wotherspoon et al., 2004 for rocking shallow foundations. Consider a large range of soil spring parameters based on desktop study (e.g. 10,000 kN/m to 100,000 kN/m for vertical stiffness in gravel) in the initial sensitivity runs before specialist geotechnical inputs.
Flexible base model using nonlinear soil springs (either explicit nonlinear or equivalent linear springs) and site response analysis	Shallow foundations Core walls Basement/part basements	Equivalent linear springs need iteration between structural analysis and geotechnical p-y curve analysis. The use of rotational springs or multi-axial springs will need careful consideration of the assumed effective damping and equivalent linearisation of the nonlinear system.
Flexible base – nonlinear dynamic history (e.g. Nonlinear time history analysis computer packages)	Irregular system on complex soils and foundations Soil and foundation could potentially result in catastrophic step change behaviour.	The shape of the hysteresis curve should be realistic and reflective of the ground conditions. No additional damping should be included for foundation radiation damping, etc. Horizontal springs can artificially damp out ground acceleration – these should be used with care.
Advanced geotechnical SSI analyses (e.g. nonlinear finite element analyses)	Where ground deformations are potentially critical and significant, e.g. behaviour of high rise buildings adjacent to a tunnel or steep slope	There needs to be a robust process for interlinking the advanced/complex finite element ground model behaviour with the global structural models.

Note:

Irrespective of the SSI modelling approach adopted, sanity checks of complex model situations (such as the type indicated in Figure C4.6) by approximate calculation and a simplified ground model are essential. The variable nature of the soil and the way in which the building interacts with it means that analysis runs to investigate the sensitivity of the results to the modelling parameter will almost certainly be required.

If SSI behaviour provides a beneficial influence to the structural performance (e.g. period elongation) the SSI analysis and geotechnical considerations should be cautiously appraised and also subjected to appropriate peer review.

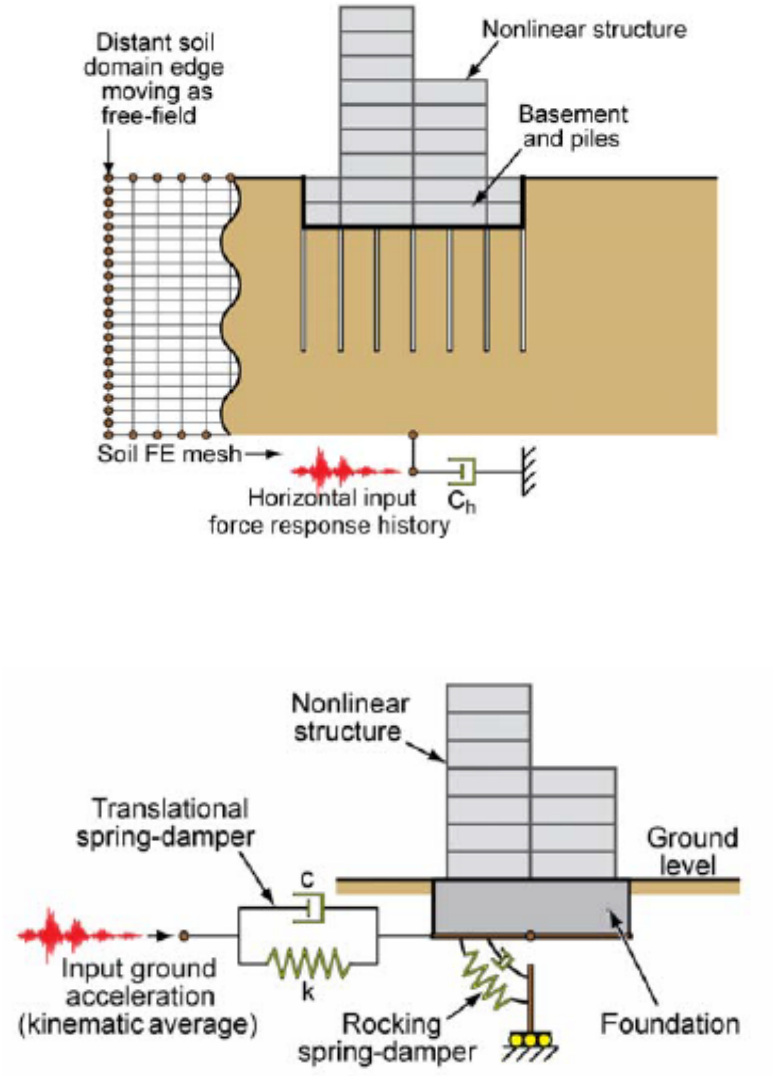


Figure C4.6: Direct and indirect SSI modelling (Deierlein et al., 2010)

Further information on SSI is provided in Appendix C4A.

C4.7 Calculation of *%NBS*

The basis for the earthquake rating for the structure is *%NBS*, which is the ratio of the ultimate (probable/expected/mean) capacity of the lowest scoring element/member/issue compatible with a significant life safety hazard or damage to neighbouring property to the actions expected when the structure is subjected to the demands resulting from the ULS defined loads/deformations for new buildings (refer to Part A and Section C1).

It is clear that if there is to be consistency between the scoring of structural elements and scoring of geotechnical issues there must be consistency in the manner in which *%NBS* is determined for geotechnical issues (soil response and geohazards).

The determination of ULS demand/actions for geotechnical related issues also often uses a slightly different approach to that which is used in the assessment of the structural aspects. Whereas the structural engineer will determine ULS demand actions by loading a model of the structure with the stylised loadings/deflections defined for new buildings, the geotechnical engineer will often consider the demand in terms of particular earthquake parameters such as earthquake magnitude and peak ground acceleration (PGA). This very specific definition of demand can lead to a misunderstanding of what is expected if the shaking levels are higher.

The approach taken in these guidelines for scoring a geotechnical issue when demand must be expressed in terms of a particular level of earthquake shaking is as follows:

Step 1: Determine the earthquake characteristics that would be applied to the design for a new building for that particular geotechnical issue. These could include earthquake magnitudes and PGA. This is defined as ULS shaking.

Step 2: Establish the acceptance criteria (strength/deformation) that would lead to a significant life safety hazard in the structure or damage to neighbouring buildings.

Step 3: Analyse the geotechnical issue for the same magnitude earthquakes as for the ULS shaking to determine the PGAs at which the acceptance criteria are just exceeded. The lowest of these will be the PGA capacity unless a step change in behaviour has been identified for the particular geotechnical issue under consideration.

Step 4: If a step change is indicated, halve the PGA at the step change and take the lower of this value or the value determined in Step 3 as the PGA capacity.

Note:

The intention is that the margin of 2 to any identified geotechnical step change behaviour that could lead to a significant life safety hazard in the structure is reflected in the *%NBS* score for that issue.

Step 5: The *%NBS* score for the particular geotechnical issue is the ratio of the PGA representing the capacity and the ULS shaking.

C4.8 Reporting and Peer Review

C4.8.1 General

Reporting should follow the general requirements set out in Section C1.

In all cases, the *%NBS* will be defined by the structural engineer in their reporting, as detailed elsewhere in these guidelines.

The scope of investigation and analysis by the geotechnical engineer should be acknowledged in the structural engineer's assessment report and the geotechnical report should be appended, together with the peer review report where applicable.

The assessment process is collaborative and iterative (refer to Section C4.3) and, as a consequence, the geotechnical report cannot be completed until this process has been completed. The geotechnical engineer will provide inputs during the process.

C4.8.2 Level of geotechnical reporting

The level of geotechnical reporting should be proportional to the significance of the geotechnical contribution to the building's performance (refer to Section C1 for characteristics of the three project categories and Section C4.8.3.2 for the expected differences in reporting scope).

C4.8.3 Report content

C4.8.3.1 General

All geotechnical reports should document the following:

- an outline of the purpose, scope and limitation of the assessment
- a list of the existing information considered in the desktop study. Relevant information should be included in an appendix where appropriate.
- the scope of any site investigations undertaken. Results and location plan should be included in an appendix.
- table(s) and cross section(s) as appropriate to describe the inferred ground model. Highlight uncertainties in the inferred model.
- a list of geotechnical issues (geohazards) identified. Categorise these as:
  a) originating from outside the building footprint and thus not influencing the *%NBS* rating
  b) jointly agreed with the structural engineer that, because of the soil and structure's expected behaviour, are not likely to be critical to the assessment of the *%NBS* rating
  c) specifically assessed.
- outline of geotechnical analysis and assessment undertaken (expect this to be limited to c) above)

- geotechnical parameters recommended to be adopted by the structural engineer in analysis and assessment
- the significance of any identified geotechnical issues originating from outside the building footprint (i.e. not considered in the assessment of the *%NBS* rating)
- any further recommended investigation/analysis/monitoring, and
- risks and uncertainties.

C4.8.3.2 By assessment category

For **structurally dominated projects** specific content should include:

- potential geohazards identified and the basis for their relevance to the seismic performance of the building. Engineering judgement by a suitably experienced engineer is a valid basis for deeming a geohazard non-relevant.
- geotechnical parameters for use in structural analysis and assessment including bearing capacities and, where required, simplified linear soil/foundation stiffnesses up to the relevant capacities.

For **interactive projects** specific content should include:

- potential geohazards identified, and a summary of their evaluation and relevance to the seismic behaviour of the building. For geohazards that potentially influence the behaviour of the structure the report should provide, as a minimum, probable resistance/deformation, and/or resistance/%ULS shaking relationships (to suit the geohazard).
- geotechnical parameters provided to the structural engineer for use in analysis and assessment
- estimates of soil parameters provided to the structural engineer for before and after initiation of geohazard(s).

For **geotechnically dominated projects** specific content should include:

- potential geohazards identified, a summary of the critical geohazard, details of evaluation, and relevance to the seismic performance of the building. For geohazards that potentially influence the behaviour of the structure the report should provide, as a minimum, probable resistance/deformation, and/or resistance/%ULS shaking relationships (to suit the geohazard) and should specifically address evaluation to ascertain if the geohazard results in a step change.
- where applicable, geotechnical parameters provided to the structural engineer for use in analysis and assessment
- where applicable, estimates of soil parameters provided to the structural engineer for before and after initiation of geohazard(s)
- assessment of the *%NBS* score for the geotechnical issue.

C4.8.4 Peer review

Peer review requirements should be discussed with the structural engineer. Suggested situations where peer review might be considered are summarised in Table C4.5. The peer reviewer's comments and the engineer's responses should be summarised separately and appended to the geotechnical report.

Table C4.5: Situations where peer review might be considered

Case	Peer review recommended
Structurally dominated project (in the absence of any other considerations described below)	X
Interactive project (in the absence of any other considerations below)	X
Interactive project IL4*	✓
Geotechnically dominated project IL4	✓
Site response analysis	✓
Studies that provide geotechnical input to multiple structures simultaneously	✓
Studies that define geohazard risks for multiple sites; e.g. regional liquefaction, tsunami, rockfall studies	✓
Studies where the outcome of the structural assessment is sensitive to one or more of the following: • soil-structure interaction • geophysical investigations • numerical modelling • time-history analyses	✓
Note: * IL = Building importance level as defined in AS/NZS 1170.0:2002	

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Appendix C4A: Modelling of SSI Effects

C4A.1 General

This appendix outlines some general principles of soil-structure interaction (SSI) and discusses various analysis techniques available.

SSI can be assessed by a range of techniques with varying degrees of complexity. This appendix outlines the following techniques, listed below in order of increasing complexity:

- simplified hand analysis to evaluate influence of ground
- simplified flexible base model using linear Winkler springs
- flexible base model using equivalent linear springs
- simplified flexible base model using compression-only or tension-only Winkler springs
- nonlinear pseudostatic analysis with explicit nonlinear soil springs
- direct finite element modelling.

For most assessments only the simplified techniques will be required. If the more complex methods are to be used this should be only if:

- a more simplified method has been applied first to develop an understanding of the likely SSI effects
- the assessment of the simplified analysis indicates that more complex analysis will be beneficial in better understanding the structure's behaviour and meeting the overall objectives of the project, and
- adequate investigation and assessment has been undertaken to define geotechnical and structural input parameters to a level detail consistent with that of the analysis.

It is important to note that the more typical structural engineering approach, which is to adopt a fixed base model for the interface between the structure and the ground, can often lead to a conservative solution for the structure. It assumes that a fixed base translates to a lower first mode period of vibration for the structure and a higher lateral load from design spectra than would be obtained if flexibility was introduced at the base. While this may be true in many cases, in others it can lead to an invalid result (e.g. NIST 2012a and NIST 2012b).

For example, overestimating the restraint available at the base of a column founded on shallow pads may provide an erroneous idea of the bending moment profile in the column and underestimate the deformations in a lateral load mechanism. Equally, assuming a rigid base under a wall may miss the potential for "foundation uplift/wall rocking" and the resulting effects.

However, perhaps more significantly, there is potential for the building response as a whole to be underestimated due to ignoring a possible resonance effect with the ground that is not sufficiently allowed for by the choice of the specified subsoil classification. Multi-storey buildings located on deep soil sites provide an example of this.

C4A.2 Key Principles

In carrying out SSI modelling, precision should not be assumed in any assessment of the interaction. However, the sensitivity to the expected response of the various assumptions should be understood. Parametric analyses to cover uncertainties in soil load-deformation characteristics will generally be required.

When assessing seismic performance both the structural and geotechnical engineers need to recognise and accommodate the potential for nonlinear behaviour of the structure, foundations and the ground. General principles to work by include the following:

- The ground's behaviour cannot be represented by unique parameter values with uniform distributions (e.g. linear springs).
- With close collaboration, the possibility of misinterpretations and abuse of numbers (e.g. spring stiffness, modulus of subgrade reaction) can be significantly reduced and possibly averted. Two effective measures to avoid the risk of misinterpretation are:
  - for geotechnical engineers to provide force/displacement relationships (springs) directly at the locations/spacings/set out that the structural engineers require; e.g. a schedule of pile springs at predetermined lengths along a pile. This avoids the potential for conversion errors from, say, subgrade modulus to springs that might arise if undertaken by the structural engineer.
  - for a reality check of force/displacement outputs performed by geotechnical engineers after structural analysis to verify correct interpretation.
- An iterative process between structural and geotechnical designers has to be established, as soil behaviour is nonlinear and spring stiffness depends on load.
- SSI should consider soil stiffness at the upper range and at the lower range of possible values as assessed by the geotechnical engineer. This could be values of 50% and 200% of the expected value.
- Soil stiffnesses considered are to be those which relate to the short term and magnitude of the seismic loading.
- Serviceability deflections are often critical for the design of new structures but not for the assessment of existing structures.
- Cost and time are associated with more rigorous analysis methods. Therefore, simplified methods should be applied first to develop an initial understanding of behaviour and the likely benefits of further more complex analysis. Complex analysis should only be embarked on when the cost can be justified in terms of improved understanding of behaviour and outcomes for the overall project.

There can be some beneficial influence of SSI on a building's life safety performance (e.g. elongation of building period, concentration of displacement demands in "ductile" foundation rotation, damping resulting from plastic soil behaviour, etc.). However, these beneficial influences are the subject of ongoing research and therefore any reductions in seismic demand resulting from their adoption should be approached with caution.

C4A.3 SSI Modelling Approaches

C4A.3.1 Simplified hand analysis to evaluate influence of ground

The engineer can undertake hand calculation of the capacities of the soil, foundation and structure systems based on preliminary and conservative assumptions of the ground model. A comparison of these capacities in addition to the Simple Lateral Mechanism Analysis (SLaMA) assessment of the superstructure (described in Section C2) will indicate whether an inelastic mechanism will occur in the foundation or soil, or whether SSI flexibility matters to the overall assessment.

If SSI effects are considered to be negligible to the overall building response or the fixed-based analysis is sufficient, no further SSI analysis is required.

A simplified SSI analysis can be undertaken with upper and lower bound geotechnical parameters to determine the most adverse consequences from the probable range of deformations resulting from ground behaviour (e.g. range of foundation flexibility due to pile tension uplift) and step change scenarios (e.g. differential settlements due to liquefaction occurring or not occurring). A desktop-based geotechnical assessment may be sufficient for this.

Due to the simplicity and coarseness of this approach, the engineer should undertake relevant sensitivity analyses and consider the likely effects of the simplifications. The cost and benefit of further more complex analysis needs to be considered before embarking on such analyses. Benefits in terms of improved understanding of behaviour and outcomes for the overall project need to be considered.

In many cases further more complex analysis of SSI will not be necessary.

C4A.3.2 Simplified flexible base model using linear Winkler springs

The SSI is modelled directly by linear soil springs, considering axial, shear and rotational flexibility. The modelling of the soil flexibility will allow a more realistic load distribution and transfer between the structure and supporting ground. This method is appropriate for both shallow and deep foundations (refer to Figure C4A.1). This approach is also referred to as the substitute or indirect method.

This approach is advantageous as it is consistent with how structural engineers typically used to consider SSI in new building design. Linear soil springs can also be incorporated easily into the analysis tools used by most structural engineers. In many cases, the structural response is not very sensitive to the soil spring values used. However, an upper/lower range of the spring flexibility should be considered. This range could be 50% to 200% times the expected value.

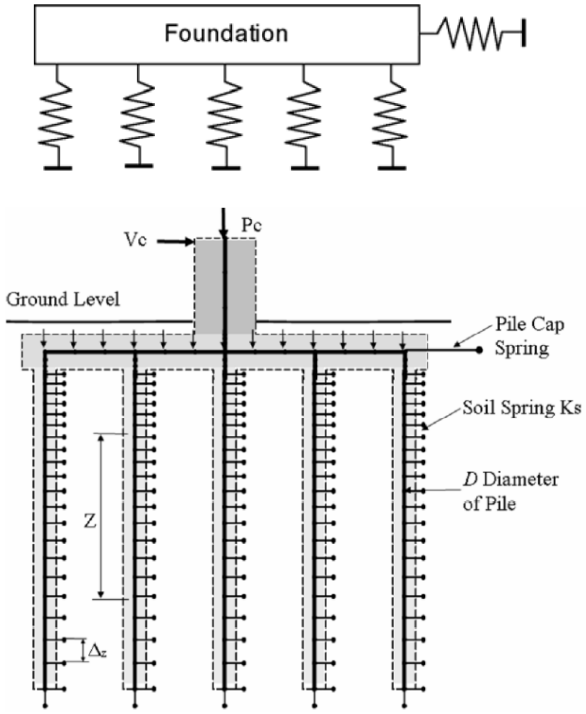


Figure C4A.1: SSI model for flexible base model using Winkler spring for shallow foundation and deep pile foundation

Key issues to consider for **shallow foundations** are**:**

- The definition of linear soil spring modelling parameters requires the geotechnical parameters (soil shear modulus and Poisson's ratio). In absence of robust geotechnical data, values can be used to initially test the sensitivity of the parameters (e.g. Oliver et al., 2013).
- The discretisation of the Winkler spring – typically, vertical springs are applied at 1 m centres. In some analysis packages, line or area springs can be applied.
- The pressure distribution through soils beneath a raft foundation influences the equivalent spring stiffness; i.e. a larger area of loading results in a greater depth of influence and greater settlement (softer springs). This can be addressed by iterations between geotechnical and structural analysis:
  - The geotechnical engineer provides the first estimate of spring stiffnesses.
  - The structural engineer applies these to analysis and reports back to the geotechnical engineer the assessed pressure distribution and settlement distribution.
  - The geotechnical engineer applies the pressure distribution to the surface of the 3D soil model and calculates settlements. Pressures are divided by settlement to give updated spring stiffnesses to be reported to the structural engineer.
  - These iterations are repeated until the pressure/settlement calculated by the structural and geotechnical models converge.
- The clear difference in including horizontal springs from vertical: horizontal springs, which are typically used for friction and/or passive soil resistance should be used with care.

Key issues to consider for **deep piled foundations** are:

- Deep piled foundations can be idealised using a series of uncoupled vertical axial springs along the length of the piles and pile caps being considered as a rigid element.
- Secant stiffness parameters (based on p-y curve at the expected lateral deformation) should be used for elastic analysis.
- Soil spring parameters for the piles spring can be determined using hand analysis (elastic analysis and Brom's method) or by specialist geotechnical analysis software based on nonlinear p-y curve of the soil layers.
- Adding detailed piles and soil springs into the global structural analysis can result in significant numerical complexity to the model, even for a linear analysis. It is common to consider the pile foundation using a refined sub-model of the critical pile-superstructure and pseudostatic nonlinear analysis (refer to Section C4A.3.5 below).
- In some scenarios with significant nonlinearity expected in the piles (e.g. piles with a liquefiable layer), a pseudostatic nonlinear analysis is more appropriate.

C4A.3.3 Simplified flexible base model using compression-only or tension-only Winkler springs

The use of linear Winkler springs is no longer appropriate when the spring goes into tension, as the soil's tensile capacity is generally negligible (unless ground anchors or piles are provided). Using an iterative process, the soil springs in elastic models that are subject to tension forces can be progressively "deactivated" from the model in order to reach an acceptable equilibrium state. This, in effect, allows the shallow foundation to uplift.

If nonlinear analysis methods are used (nonlinear pushover or time history), foundation uplift and soil yielding can be explicitly modelled using compression gap elements and nonlinear springs with asymmetric capacity curves. As the analysis result is very sensitive to the nonlinear springs' parameters, a sensitivity analysis should be carried out. Due to the complexity and time involved, the sensitivity analysis can be carried out using a sub-assembly model.

The nonlinear modelling of rocking foundations can be a complex area resulting in erroneous results. The use of tension-only or compression-only elements in nonlinear dynamic analysis can result in "stiffness matrix spikes" and loss of energy from over-damping. The use of nonlinear contact elements may also lead to over-prediction of the damping and energy dissipation that results from the interaction between the soil and the foundation interface.

C4A.3.4 Flexible base model using equivalent linear springs

The nonlinear behaviour of the soil can be modelled using equivalent linear springs (NIST, 2012a and ASCE 41-13, 2014) for both linear dynamic analysis and nonlinear pushover analysis.

The equivalent linear model simplifies the nonlinear behaviour of soil by characterising the hysteresis loops by:

- an equivalent secant modulus, $G_{\text{sec}} = \frac{\tau_c}{\gamma_c}$ where τ_c and γ_c are the expected amplitudes of shear stress and shear strain respectively

- an equivalent viscous damping ratio, ξ_{soil} that is directly proportional to the hysteretic energy dissipated, where:

$$\xi_{soil} = \frac{1}{2\pi}\frac{A_{loop}}{G_{sec}\gamma_c^2} \quad \ldots C4A.1$$

The G_{sec} values used need to be checked and iterated with analysis results to ensure the equivalent secant modulus is taken at the tangent to the peak shear stress/strain point. In the absence of definitive justification ξ_{soil} = 5% is recommended to be used together with the structural performance factor, S_p, as per the building structural ductility capability.

C4A.3.5 Nonlinear pseudostatic analysis with explicit nonlinear soil springs

Modelling approach

In some scenarios where SSI has a significant influence on the seismic response of the building, nonlinear analysis of the SSI effects will be warranted.

There are a number of relevant articles in the literature on the modelling of nonlinear soil behaviour using bilinear or trilinear capacity curves with substructuring/indirect modelling for the purpose of pseudostatic pushover analysis (FEMA 440 (2005) and Cubrinovski and Bradley, 2009).

Two approaches for shallow and deep foundations are illustrated in the following figures.

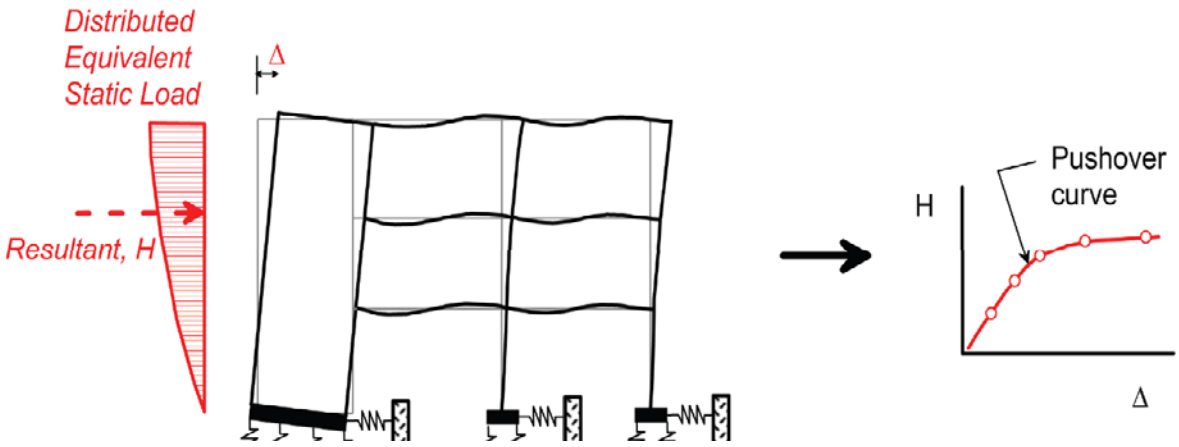


Figure C4A.2: Schematic illustration of a pushover analysis and development of a pushover curve for a structure with a flexible base (NIST, 2012a)

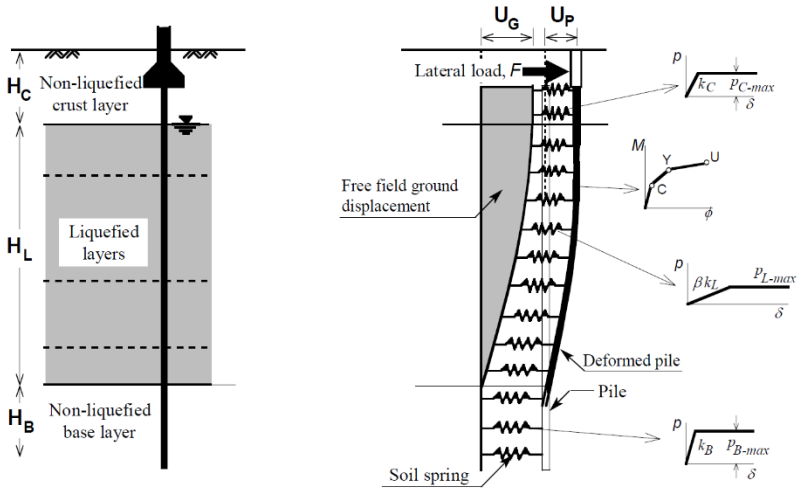


Figure C4A.3: Schematic illustration of a pseudostatic pushover analysis and development of a pushover curve for a bridge pier with a flexible pile base (Cubrinovski and Bradley, 2009)

Damping approach

Damping related to foundation-soil interaction can significantly supplement damping that occurs in a structure due to inelastic action of structural components. The damping from foundation-soil interaction is associated with hysteretic behaviour of soil (not to be confused with hysteretic action in structural components) as well as radiation of energy into the soil from the foundation (i.e. radiation damping). These foundation damping effects tend to be important for stiff structural systems (e.g. shear walls, braced frames), particularly when the foundation soil is relatively soft.

Due to the uncertainty associated with soil hysteretic and radiation damping, ξ_{soil} is limited to 10% and ξ_{soil} = 5% is recommended unless there is strong evident to suggest the use of a higher damping value. Refer to Section C2 for the treatment of additional soil damping (as ξ_{soil}) for nonlinear pseudostatic analysis.

C4A.3.6 Direct finite element modelling

It is possible to undertake a direct simulation of the SSI and the nonlinear responses of the soil and structure using a direct approach, in which the entire SSI system is analysed in a single model/step. SSI using a direct analysis approach can be performed using finite element (FE) computer programmes. Figure C4A.4 shows an example of such analysis.

There are a number of technical challenges related to the use of a direct analysis approach, including the definition of critical input parameters (e.g. a constitutive model for various soil types), the geotechnical information of the underlying soil, the definition of boundary conditions, and the complexity of such a complex nonlinear model.

Methods of this level of complexity would only be considered in exceptional cases where a critical issue has been identified for a larger project requiring specific detailed analysis. Before undertaking direct analysis approach:

- Separate, less complex analyses should be undertaken so the benefits of carrying out a direct analysis can be assessed and also to provide a check against the outputs of the direct analysis.
- Sufficient investigations should be undertaken to provide a level of detail in understanding the geotechnical and structural input parameters in keeping with the detail of the analysis.

There is a greater need for a rigorous checking of the input parameters and analysis assumptions for the FE model given the "black box" nature of such analysis. Independent peer review of the inputs and outputs is recommended.

Note:

Cubrinovski and Bradley (2009) provides an example of the use of effective stress analysis using a direct approach for the analysis of piles in liquefiable ground.

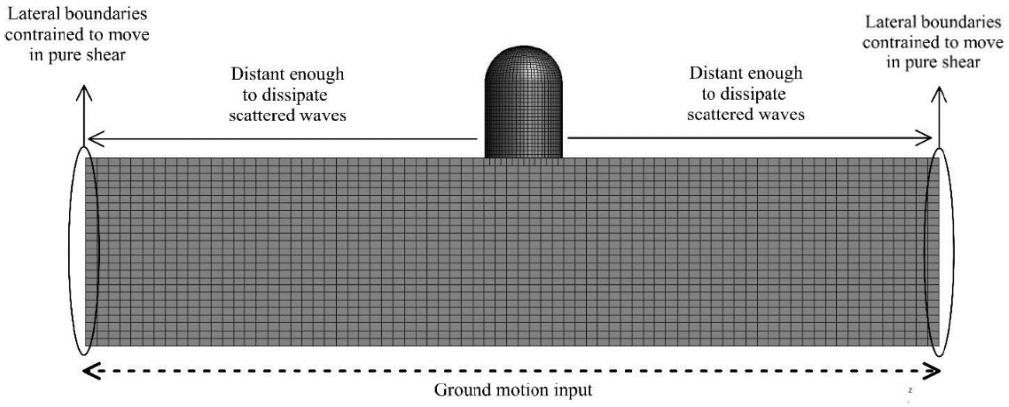


Figure C4A.4: Direct FE modelling (Cubrinovski and Bradley, 2009)

Appendix C4B: Assessment of Retaining Walls

(Supplement to NZGS/MBIE Module 6: Earthquake resistant retaining wall design)

C4B.1 Introduction

Retaining walls are often associated with, or even integral to, a structure under assessment. The assessment of retaining walls may require close collaboration between the structural and geotechnical engineer as these are loaded by, and typically derive their restraint from, the ground but may also contain elements that require structural input.

Note:

NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design* provides relevant guidance. This appendix supplements that guidance with specific information relating to assessment.

C4B.2 Historical Performance

Observations made during the Canterbury earthquake sequence of 2010-11 provide a useful insight into the performance of existing retaining walls under seismic shaking. Refer to Appendix A of NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design* for a commentary on observations from Christchurch. However, care should be exercised in extrapolating these findings to other walls and ground conditions elsewhere in New Zealand. Also, note that there were few, if any, instances of retaining wall performance during the Canterbury earthquake sequence affecting the life safety performance of buildings.

Note:

Other useful references include Anderson et al. (2015) and Kendall Riches (2015).

A number of aspects of retaining wall design contribute to better than expected earthquake performance when walls are apparently loaded beyond their design capacity. In general terms, there is conservatism in static design methods and in simplifications of pseudostatic design methods. In addition, there is the typical robustness of retaining walls.

Where appropriate these aspects (listed below) should be considered while undertaking an assessment of an existing retaining wall:

- the use of strength based design, where wall displacement could have been used to limit seismic loads in the design
- the use of elastic design for wall elements where ductility might be acceptable
- use of the Mononobe Okabe (M-O) equation
- assuming $c = 0$ (cohesion of the soil) to derive loads on a wall supporting ground, but with the shear strength actually due to both c and ϕ (friction angle of the soil)
- considering sloping ground behind the wall where an unrealistically large seismic active earth pressure coefficient was assumed in design

- assuming homogenous soil properties in design, but where actual strength properties increased with depth/distance from the wall but were not taken into account over the extent of theoretical slip; or design was based on the weakest material and/or characteristic (i.e. conservative) parameters
- adopting unrealistically high active earth pressure values for cases with high seismic accelerations or steep back-slopes, and
- ignoring wave scattering and dynamic effects for calculation of seismic pressures on high walls.

Note:

NCHRP, 611 (2008) states: "The overall performance of walls during seismic events has generally been very good, particularly for mechanically stabilised earth (MSE) walls. This good performance can be attributed in some cases to inherent conservatism in the design methods currently being used for static loads".

C4B.3 Identification of Retaining Walls requiring Assessment

C4B.3.1 General

A retaining wall will only need to be assessed if its performance could affect the ability of the structure being supported to meet its own performance criteria.

Accordingly, the focus of any retaining wall assessment should be on the consequence for the supported structure. Even if it indicates that the wall is at risk of "failure" under the earthquake shaking considered, this failure is only considered consequential if it results in:

- the structure not meeting life safety performance criteria, or
- loss of emergency egress from the structure.

In the context of the life safety assessment of existing buildings, the behaviour of supporting retaining walls will often not be the governing issue for the performance of the structure. The following questions are suggested for initial consideration:

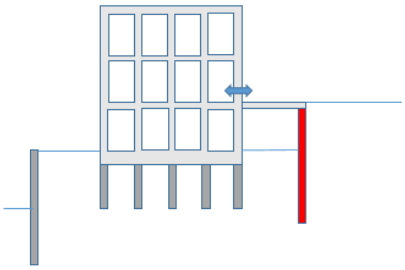
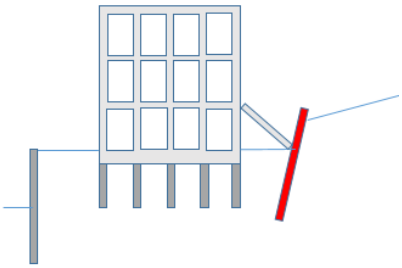
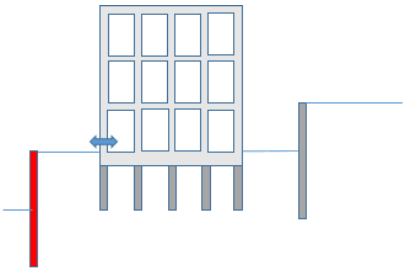
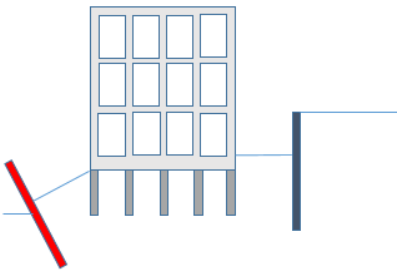
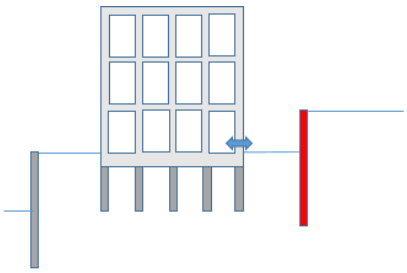
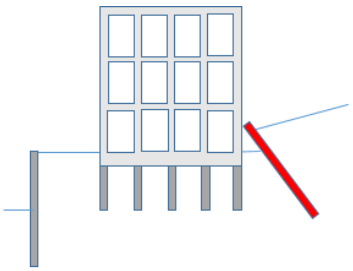
- Is there a significant risk that the wall may be of low capacity? (For example, it is a historic stone/masonry wall with no redundancy, or liquefaction is likely.) If yes, then assess the consequences for the structure's performance on the assumption that the wall may fail.
- Is there a significant risk of excessive (e.g. > 200 mm) horizontal displacement? (For example, it is a historic mass concrete gravity wall with an undersized foundation.) If yes, then assess consequences for the structure's performance.
- Can the structure tolerate horizontal wall displacement of 100 mm? If no, then assess in more detail.

As outlined in Section C4.5.3, the retaining wall's performance should be considered across a spectrum of earthquake demand. There are a number of mechanisms by which a retaining wall can impact on structural seismic performance. Some examples are presented below.

C4B.3.2 Loss of emergency access/egress to the building

Table C4B.1 gives some examples where poor performance in a wall may impact on emergency access/egress and hence on the building's earthquake rating.

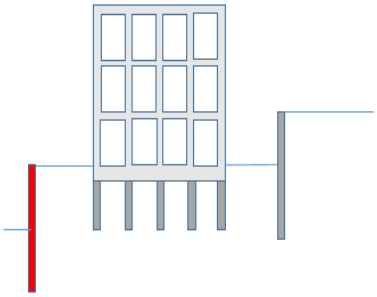
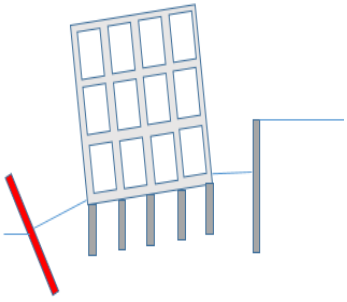
Table C4B.1: Examples of impact on emergency access/egress

Mechanism	As designed	Potentially unacceptable performance
Instability in a retaining wall supporting structure required for building egress		
Instability in a retaining wall supporting ground that provides building egress		
Instability in a wall supporting ground above a building egress		

C4B.3.3 Loss of support to foundation soil

Table C4B.2 gives an example where poor performance in a retaining wall providing support to the building foundations may impact on the building's earthquake rating.

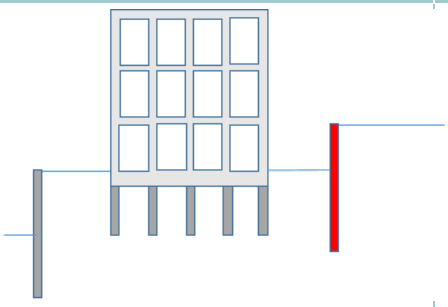
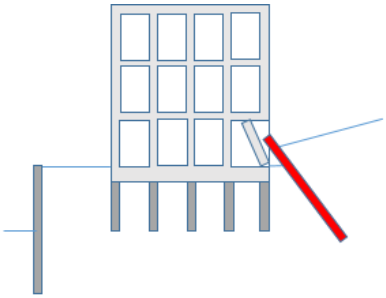
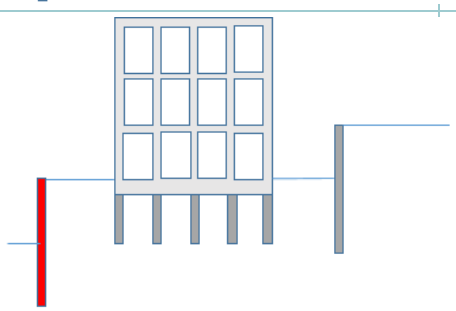
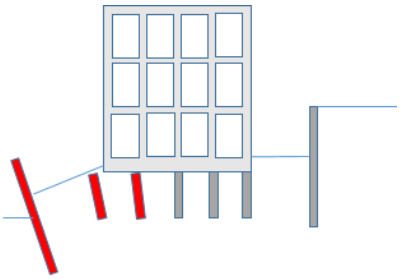
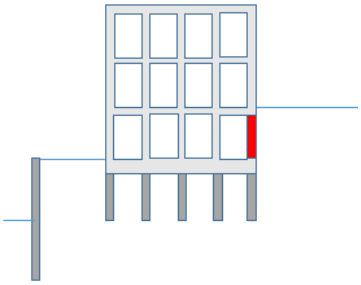
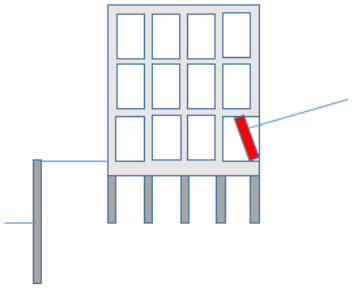
Table C4B.2: Example of loss of support to foundation soil

Mechanism	As designed	Potentially unacceptable performance
Instability in a retaining wall below building foundations removing vertical support		

C4B.3.4 Lateral loading or deflection of a key building element

Selected examples where poor performance of retaining walls that may result in excessive increased lateral loading or reduction in lateral support and, in turn, may impact on the building's earthquake rating are shown in Table C4B.3.

Table C4B.3: Examples of lateral loading or loss of lateral support to foundation soil

Mechanism	As designed	Potentially unacceptable performance
Instability in a retaining wall impacting on building. Does **not** influence the earthquake rating of the building.		
Instability of a retaining wall generating lateral loading on foundations supported at a deeper level		
Instability in a basement retaining wall		

Method A – Force-based assessment

For force-based assessment of retaining walls, refer to NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design.* This provides relevant guidance including how to allow for displacements.

C4B.4 Modes of Deformation

Refer to NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design* for information on the modes of deformation to be considered for various types of retaining walls.

C4B.5 Seismic Loads

Refer to NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design* for guidance on deriving retaining wall seismic loads.

C4B.6 Methods of Assessment

Method B – Displacement-based assessment

If the forced-based assessment indicates that ductile-type behaviour of the wall can be expected but the magnitude of deformations could be an issue for the building, further assessment via a displacement-based assessment is likely to be required. Available methods use the results of Newmark sliding block regression analyses published by researchers such as Bray and Travasarou (2007) and Jibson (2007). In applying these methods the soil parameters assumed need to relate to those at the magnitude of strain/displacement being considered.

Method C – Dynamic numerical analysis method

For complex structures or walls posing a high risk or of significant monetary value, dynamic or time history analysis may be necessary. For further guidance on selecting and scaling appropriate earthquake records refer to NZS 1170.5:2004 and the NZTA Bridge Manual (2013). These complex analyses should only be considered if the results of the methods referred to in Method B above indicate critical issues and uncertainties that require further consideration.

C4B.7 Coincident Building and Earth Pressure Loads

The potential for coincidence of structural loading and retained soil loading should be considered (refer to Table C4B.4).

Factors to consider include:

- physical coupling between structure and walls (e.g. a wall propped by the building)
- the potential for the wall's and structure's natural frequencies to coincide (e.g. squat structure/tall wall), and
- the potential for liquefaction or other time-related effects such as lateral spread-generated kinematic loading coinciding with peak inertial loading.

Table C4B.4: Factors to be considered for loading on retaining walls

Details	Structure/ soil loading likely to coincide	Use conservative assumption or undertake specific analysis	Structure/ soil loading unlikely to coincide	Comments
Basement retaining wall	✓	x	x	
Wall retaining a building platform Structure $> 3H$ behind	x	x	✓	Consider slope of land between wall and structure and the presence of sensitive/ liquefiable ground
Wall retaining building platform Structure between $1H$ and $3H$ behind wall	x	✓	x	
Wall retaining building platform Structure $< 1H$ behind wall.	✓	x	x	
Where liquefaction derived pressures or lateral spread flow loads are already accounted for in design	x	✓	x	
Inertial load from wall elements excluding MSE	✓	x	x	

Note:
H = wall height

C4B.8 Estimation of Backfill Settlement

Loss of foundation support/settlement of backfill behind a retaining wall may occur through a number of mechanisms including erosion, densification and deformation at constant volume due to wall displacement/rotation.

- Erosion of backfill may occur where services carrying water with a significant head are ruptured due to otherwise acceptable seismically induced ground movement. Such effects can typically be assumed to be localised and unlikely to lead to collapse. The associated flooding effect can also cause increase in water pressure on retaining walls leading to collapse or deformation. Further investigation may be warranted in some circumstances.
- Densification will tend to occur during earthquake shaking in granular soils, particularly where this is poorly compacted. This settlement may be damaging to supported structures and can lead to wall deformation. However, unless the structure is particularly sensitive or the backfill is especially loose and deep, the risk of wall collapse can be assumed to be low.
- Significant settlement can be anticipated in retained ground if wall deflections occur during earthquake shaking or due to lateral spread. CIRIA C580 (Gaba et al., 2003) provides methods of estimating the magnitude of retained ground settlement and potential consequences for a range of structural types.

Appendix C4C: Slope Instability Hazard
(Interim Guidance)

C4C.1 Introduction

This appendix provides an overview of slope seismic instability as interim guidance, as there is currently no NZGS/MBIE module on this topic.

Slope stability assessment requires an understanding of a number of key attributes, including:

- slope geometry
- potential defects or structure/zones/planes of weakness in the soil/rock
- the groundwater regime
- soil/rock strength and potential for strength loss, including through liquefaction/cyclic softening, and
- the foundation system and/or retaining wall system embedded in the ground.

Note:

While an unstable slope may interact with the structure, the integrity of the structure or its life safety attributes may not always be compromised. In some cases the structure can withstand the predicted loss of support, displacements, impact or loading that arise from slope instability.

C4C.2 Scale

The scale involved in slope stability can be significantly greater than for other aspects of seismic assessment such as foundations or retaining walls. As a consequence it is important to look beyond the immediate site. Coseismic landslides and rockfalls can range from discrete, localised events up to massive events. Many contemporary examples of seismically induced slope instability can be found, including those associated with the Canterbury earthquake sequence of 2010-11.

C4C.3 Local knowledge

Stability conditions vary widely across New Zealand. Consequently, local knowledge is beneficial, particularly where calibrated by observed behaviour during past earthquakes or inferred from geomorphic evidence. Advice should be sought from an engineering geologist when detailed assessment of slope hazards is warranted.

C4C.4 Influence of ground conditions

Examples of circumstances in which seismically induced slope instability may be an issue include:

- where there is a history of slope instability or a geomorphology that is indicative of historic instability
- when there is no evidence of historic instability but the topography, geology, groundwater conditions and seismic conditions are such that instability is possible

- steep slopes (greater than 35º), such as gorges and cliffs where rockfalls are common
- slopes that have been altered, such as cuttings along roads and quarries, or where vegetation has been removed
- underlying weathered or shattered rocks that weaken the slopes
- soils that have liquefaction potential with sloping ground or a nearby free face
- active landslides or old landslides that might start moving again, and
- in the vicinity of active fault scarps.

Note:

This list has been adapted from the AGS Practice Note Guidelines for Landslide Risk Management, 2007.

C4C.5 Assessment Process

Stage 1 – Initial assessment of stability

A great deal of information on slope stability can usually be obtained via desk study and/or site inspection by a suitably experienced person. Input and review by an engineering geologist is recommended.

It is recommended to start with a natural scale sketch of the system model: the ground, the foundations and the structure. ASCE 41-13 (2014) Clause 8.2.2.4 is a useful guide for screening purposes.

Engineers are referred to geohazard assessments that have been carried out for territorial authorities and regional councils to identify the potential hazards that are likely to be appropriate for the site in question. These are typically in the form of hazard maps. There may also be specific slope hazard reports in urban areas. Additional guidance on this desk study is included in NZGS/MBIE Module 2.

Stage 2 – Site inspection

Input by an engineering geologist is recommended during the site inspection and associated reporting. Relevant geohazard information that is obtained from a walkover of the site, desk study of geohazard references and local knowledge can be combined in a site inspection report. This should include the following information:

- a brief description of the site shape, size, geological profile (refer to maps and memoirs), expected site subsoil class, terrain, vegetation, springs, erosion features, evidence of slope instability on site and on adjoining site(s), where relevant. Comment on depth to groundwater and seasonal fluctuation, if known.
- a description of how the building sits in relation to the site (e.g. with reference to an annotated aerial photo). Comment on proximity of the building footprint to slope edges, slope height and proximity to water courses/river banks (these details are relevant in terms of seismic slope stability and also for potential lateral spread hazard), and
- a description of geohazard sources located outside the site boundaries that could impact on building performance. This is particularly relevant for slope instability uphill of the site or retaining walls on adjacent property.

Stage 3 – Site investigation

If a site investigation is required the site-specific scope should be determined. A CPEng geotechnical engineer or PEngGeol engineering geologist should be engaged for scoping and management of a site investigation. Refer NZGS/MBIE Module 2.

Stage 4 – Analysis

Jibson (2011) provides a useful overview of methods for assessing the stability of slopes during earthquakes, including a list of useful references.

Jibson (2011) describes three families of analyses for assessing seismic slope stability as follows, with each having its own appropriate application:

- **Level 1 – Pseudostatic analysis**
  - only suitable for preliminary or screening analyses because of its crude characterisation of the physical process
- **Level 2 – Permanent deformation analysis**
  - a valuable middle ground between a Level 1 and Level 3 analysis
  - simple to apply and provides far more information than pseudostatic analysis
  - rigid-block analysis suitable for thinner, stiffer landslides, which typically comprise the large majority of earthquake-triggered landslides
- **Level 3 – Stress deformation analysis**
  - best suited to large earth structures such as dams and embankments, as it is too complex and expensive for more routine applications
  - coupled analysis is appropriate for deeper landslides in softer material, which could include large earth structures and deep landslides
  - modelled displacements provide a useful index to seismic slope performance and should be interpreted using judgement and according to the parameters of the investigation.

Note:

Refer to Barbour and Krahn (2004) for insights and guidance on numerical modelling.

C4C.6 Defining Seismic Accelerations for Slope Stability Analysis

Refer to NZGS/MBIE Module 1 - *Overview of the Guidelines*.

Ground shaking can be subject to significant amplification near the crest of steep slopes and ridgelines, such that PGA_{SITE} can be significantly greater than a PGA determined via NZGS/MBIE Module 1. NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design*, MBIE (2014) and Eurocode EN 1998-5:2004 provide information on topographic amplification factors.

Appendix C4D: Seismic Performance of Foundations

(Supplement to NZGS/MBIE Module 4: Earthquake resistant foundation design)

C4D.1 Introduction

NZGS/MBIE Module 4 - *Earthquake resistant foundation design* provides guidance relevant to the assessment of foundations. This appendix supplements that guidance and provides specific information relating to seismic performance of existing foundations and observations from the Canterbury earthquake sequence of 2010-11 and other earthquakes. A description of foundation types historically used in New Zealand and their strengths and weaknesses is also provided.

Following the Canterbury earthquakes, liquefaction-induced ground failure did not result in any direct fatalities in Christchurch's central business district (CBD) despite the widespread damage to residential and commercial buildings (Cubrinovski and McCahon, 2012; Murahidy et al., 2012). However, rockfall and landslides at the fringe of the city resulted in five fatalities (Dellow et al., 2011).

A similar conclusion can be drawn from the 14 representative buildings studied by the Canterbury Earthquakes Royal Commission (CERC Vol 2, 2012). While ground failure (e.g. liquefaction) and foundation damage were observed at a number of sites (e.g. the Town Hall, police headquarters, and 100 Armagh St Apartments), these buildings have generally satisfied the life safety performance required by the New Zealand Building Code.

As a general observation of building performance in Christchurch, if the superstructure was robust (well-tied together), integral and responding in a ductile manner, foundation failure excacerbated the inelastic demand on the superstructure's plastic hinges but did not necessarily result in a uncontrolled displacement response.

C4D.2 Shallow Foundations

Foundation elements are considered to be shallow when the depth to breadth ratio is less than 5 (i.e. $D/B < 5$). Some behaviours of shallow foundations to be considered in assessment are outlined below.

Some foundations have suffered from non-uniform aspects such as basements under only parts of the building, irregular footprints with differential movements in plan, or piles installed to provide tension capacity under only parts of a shallow foundation. Particular attention should be given to the areas around such features in looking for damage, differential movement, etc. A number of buildings have suffered differential movement due to uplift of basements under part of the ground floor.

Basements can be exposed to high uplift pressures generated in liquefied sands or in loose gravels. This can result in vertical displacement as well as damage to the basement floor, depending on the construction as a raft or slab between footings or piles. Uplifted basements, particularly those on gravels rather than liquefied sands, may have large voids below them. Basement walls may have been subjected to lateral earth pressures much higher than normal static loading. Many basements were partially flooded after the 22 February 2011 Canterbury earthquake because of damage to walls, floor or tanking.

Gapping has occurred adjacent to footings as a consequence of cyclic lateral displacement during the shaking.

Where rocking of foundations has occurred (or is suspected to have occurred) gaps may have developed underneath foundation elements or under the edges of elements.

C4D.3 Deep Foundations

Foundation elements are considered to be deep when the depth to breadth ratio is greater than or equal to 5 (i.e. $D/B \geq 5$). Some behaviours of deep foundations to be considered in assessment are outlined as follows:

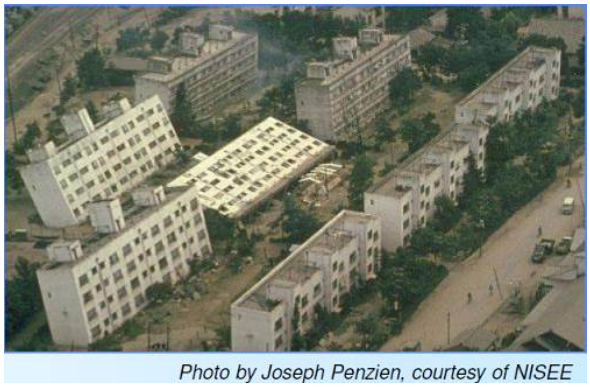
- Common issues for deep foundations that need to be considered include the loss of side resistance (skin friction) in piles, which may occur from pore water pressure increase during shaking, even if full liquefaction does not trigger. Where full liquefaction is triggered at depth all side resistance above may be effectively lost or reversed because of settlement of the overlying strata. In such cases, so-called "negative skin friction" may contribute to pile settlement.
- Unless they are adequately embedded in dense soils, bored cast-in-place piles are perhaps the most susceptible to settlement caused by pore water pressure rise and liquefaction above the base of the pile, because the gravity loads are carried initially almost entirely by side resistance. If this mechanism is overloaded, the pile will settle until the end bearing mechanism is mobilised (which could be as much as 5 to 10 percent of the pile diameter). This can potentially be exacerbated if poor construction has left a zone of disturbed material at the base of the piles.
- Cyclic axial loading during the earthquake may cause loss of capacity and settlement, especially for piles that carry only light gravity loads and rely mainly on side resistance.
- Pile settlement may also be from liquefaction of sand layers below the founding layer. For example, many parts of Christchurch have dense gravel or sand layers that may be several metres thick but underlain with much looser sands. Deeper liquefaction may not have been considered in the pile design, particularly of older buildings.
- Damage to foundations may not always be evident from the surface, particularly where a large area has been subject to lateral displacements. Where there is evidence of relative motion between the structure and the ground, pile heads and the connection to the structure should be checked for overload in shear. Shear transfer from the ground to the building is typically assumed to be carried by friction underneath the building and by passive resistance of the soil against buried foundation beams and walls, etc. The friction mechanism will typically fail quickly with any settlement of the ground and the passive mechanism degrades rapidly with development of gapping. For this reason, and because the earthquake shaking was stronger than design levels, it is likely that the piles may have carried far more shear than the designer ever intended.
- Kinematic interactions between the ground and the piles need to be carefully considered. Ground deformations are known to have been significant around many parts of Christchurch, including both dynamic and permanent deformations. These ground deformations may impose significant strains within piles resulting in pile damage and permanent deformation well below the ground surface. Physical investigation of such damage is difficult and expensive and may be impractical. Analytical procedures are available as a first step to try and estimate the pile strain levels and therefore likelihood of damage.

C4D.4 Soil-Structure Interaction

Reconnaissance reports of past earthquakes confirm that the seismic performance of buildings can be significantly influenced by the geotechnical performance of the supporting ground. Buildings have collapsed or been significantly damaged due to either foundation (shallow or deep) "failure" and/or liquefaction-induced settlements. Similarly, there are buildings that could have collapsed but have not done so due to the beneficial effect of SSI.

Figure C4D.1 shows overseas examples of (a) building collapse and (b) brittle pile shear failure, both as consequences of ground liquefaction and foundation failure from the 1964 earthquake in Niigata, Japan. Both mechanisms would not have been identified by an engineer undertaking a simple pinned/fixed-based structural analysis. It is noted the level of understanding of liquefaction risk was minimal at the time of this earthquake.

The building in Figure C4D.1(b) remained in service for 20 years after the earthquake despite the hidden shear failure of the piles, illustrating the difficulty in predicting foundation performance and identifying foundation damage post-earthquake (Yoshida and Hamada, 1990).



(a) Niigata 1964 – tilt of housing blocks due to liquefaction-induced bearing capacity failure

(b) Pile shear failure observed in an excavation 20 years after the Niigata 1964 earthquake

Figure C4D.1: Significant building damage and collapse due to ground failure (Yoshida and Hamada, 1990)

There are several notable examples where the geotechnical foundation system's step change behaviour led to a brittle failure mode in the substructure and superstructure.

Figure C4D.2 illustrates the example of a five storey building damaged in the Christchurch earthquake of 22 February 2011 (Kam et al., 2011). The site (in Madras St, central Christchurch) showed evidence of moderate liquefaction surface manisfestation.

The foundation of the core wall on the southern elevation lost its bearing capacity, possibly during or after the earthquake event, and the wall had settled about 450 mm vertically. The settled core wall appeared to have pulled the floor slab and the rest of building towards it. The external ground beam connected to the wall, and a number of frame beam-column joints had failed in a brittle shear mechanism (refer to Figure C4D.2I) which is likely to be a consequence of both seismic shaking and induced vertical displacement demand from the wall's foundation failure. The building's lateral load system was severely compromised due to the foundation-wall system failure and it partially collapsed in a subsequent aftershock.

(a) Plan **(b) South east elevation** **(c) Shear failure of ground beam connected to shear wall**

Figure C4D.2: Five storey building with shallow foundation failure beneath core walls (adopted from Kam et al., 2011)

Figure C4D.3 presents several examples of significant building residual deformations due to foundation "failure" observed in the Christchurch CBD (Kam et al., 2011).

(a)-(b) 1980s high rise on basement and raft foundation; with beam plastic hinges observed throughout the building **(c) 1980s low rise on shallow foundation with significant differential settlement and sliding movement**

Figure C4D.3: Building foundation "failure" (Cubrinovski and McCahon, 2012)

C4D.5 Information on Foundation Types used in New Zealand (Potential Strengths and Weaknesses)

The table below summarises the foundation types likely to be encountered in New Zealand buildings, together with their likely strengths and weaknesses.

Note:

This information is for general guidance only. Each site and structure should undergo site-specific engineering assessment.

Table C4D.1: Summary of traditional foundation types

Foundation type	Era	Brief description	Likely strengths	Likely weaknesses
Driven timber piles	1890 -	Round poles top driven to a set	• Durable when quality hardwood used, especially when submerged • Consistent capacity	• Degradation/rot, especially at top • Poor engagement into foundation
Driven timber tip armoured	1890 - 1920	End tapered and protected with steel to penetrate stiffer layers		
Driven steel piles			• Consistent capacity • Could be driven through stiff layers • High shear capacity • (Can be) ductile in bending • Degradation less of an issue due to large area-to-surface ratio	• Rusting/degradation potentially very significant • Variable engagement into foundation
I or H sections	Typical post-1970s	Commonly bare steel, sometimes galvanised or coated		
Tube/pipe	Typical post-1970s			
Railway irons	1890s -	Cast iron prior to ~1910		
Driven concrete piles			• Base bearing capacity consistent • Side friction variable dependent upon installation technique, but should be calculated considering it as a displacement pile	• Shear failure. Existing piles often have few stirrups and can fail in a brittle manner during ground lurch or lateral spreading. • Franki/bulb piles are likely to have poor curtailment of reinforcement into the consolidated base, and so little tension capacity. They also may have “necked” shafts. • Top fixity: does this work in both directions? Is it truly fixed at the top?
Precast	1915-			
Franki/bulb piles	1960s-1980s	Drilled pile, concrete poured at base and driven to provide consolidated end bearing and spread		
Driven precast plug	1970s-	Drilled pile with precast pile driven out through base		
Bottom driven steel tube	1980s-	Permanent steel tube liner driven by dropping a weight on a plug of dry mix concrete in the base of the tube. Reinforcing cage and concrete placed after driving		
Driven cast-insitu	1980s-	Driven tube with sacrificial steel base, casing withdrawn during casting		
Bored piles			• Very old (<1910) piles may have high quantities of non-Portland cement and hence be very durable • Often large robust sizes • Reinforcing easy to curtain into foundation beams • Be careful for distribution between skin friction and end bearing (relative stiffness and strength)	• Base cleanout quality critical for end-bearing dependent piles (esp. bells) • Shear may be critical for piles with fewer stirrups underground lurch or lateral spreading • Top fixity? • Belling quality (collapse)?
Straight	1860-	Multiple drilling techniques		
Straight grooved	1990-	Sides grooved with special tool after drilling		
Belled	1960-	Specialist technique		

Foundation type	Era	Brief description	Likely strengths	Likely weaknesses
Steel screw piles	1990-	Specialist technique	• Records correlate capacity with installation torque • Testing results should be available • Robust against shear • Small drag-down effect if liquefaction settlements in upper layers	• Helices very flexible: vertical displacements often govern for seismic loads (soil/structure interaction • Small contribution to base-shear resistance
Ground anchors			• High capacity can be installed in small space • Free length gives controlled plastic elongation if required • Testing records may be available • Can often be re-tested to prove capacity	• Poorer performance under cyclic load • Limited compression capacity: critical if building settles due to liquefaction • Little to no shear capacity: vulnerable to lurch or lateral spreading • Durability critical, especially around anchorages (esp. for both ends of deadman anchorages) • Potential "brittle" behaviour (reduced grout to country bond with strain)
Drilled and inserted	1960-	Drilled and grouted hole, bar or strand anchors		
Pressure grouted/drilled	1990-	Proprietary bar drilled specialist technique		
Deadman	18??-	Relies on steel bars back to mass or reinforced concrete passive acting blocks		
Mechanical expansion	1970-	Rock bolts with expansive ends		
Grout expansion	1990-	Proprietary grouted tubes which "unroll"		
Mechanical tip	1990-	Proprietary bearing engagement e.g. "Duckbill/Manta Ray"		
Shallow			• Predictable, well tested behaviour in "good ground" • Pads often oversized for older buildings • Rafts can mitigate differential displacement	• Affected significantly by liquefaction • Strip footings often undersized/highly stressed under brick walls • Pre-1930s footings may not have continuous reinforcement
Brick strip	1840-	Nominal widening, sometimes incorporating site concrete		
Concrete strip	1840-	Reinforced or unreinforced		
Ground beam	1950-	Reinforced, likely spreading point loads		
Isolated pad caisson	1840-	Reinforced		
Raft	1970-	Reinforced		
Domestic			• Typically small loads per unit	• Degradation of timber with time • Often lack of distributed resistance • Ensure structure fixed to foundations • Shallow piles have little or no cantilever capacity
Timber ordinary	1840-	Rounds or squares excavated and concreted in place		
Timber anchor	1980-	Square excavated and concreted in place		
Timber driven	1960-	Round or square		
Concrete ordinary	1920-	Precast, sometimes cast in "kerosene tins"		
Concrete strip	1930-	Typical subfloor walls		
Brick strip	1860-	Single or two courses wide, sometimes in site concrete		

Appendix C4E: Initial Screening for Liquefaction Susceptibility

(Supplement to NZGS/MBIE Module 3 - Identification, assessment and mitigation of liquefaction hazards)

NZGS/MBIE Module 3 - *Identification, assessment and mitigation of liquefaction hazards* provides guidance on assessing susceptibility of soils to liquefaction. This appendix supplements that guidance and provides an initial screening tool.

Soils susceptible to liquefaction may substantially lose vertical load-bearing capacity during an earthquake. Loss of vertical support for the foundation causes large differential settlements and induces large forces in the building superstructure. These forces are concurrent with all existing gravity loads and possibly seismic forces during the earthquake.

ASCE 41-13 (2014), Table 8-1, reproduced below in Table C4E.1, provides a means of **initial screening only** of the ground profile to determine the site's liquefaction susceptibility. Refer to ASCE 41-13 (2014), Section 8.2.2 for further details.

Table C4E.1: Estimated susceptibility to liquefaction of surficial deposits during strong ground shaking (ASCE 41-13 (2014), Table 8-1)

Type of deposit	General distribution of cohesionless sediments in deposits	Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction (by geologic age)			
		Modern <500 years	Holocene <11,000 years	Pleistocene <2 million years	Pre-Pleistocene >2 million years
(a) Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Flood plan	Locally variable	High	Moderate	Low	Very low
Alluvial fan, plain	Widespread	Moderate	Low	Low	Very low
Marine terrace	Widespread	-	Low	Very low	Very low
Delta, fan delta	Widespread	High	Moderate	Low	Very low
Lacustrine, playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dune	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	Low	Unknown	Unknown
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low

Type of deposit	General distribution of cohesionless sediments in deposits	Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction (by geologic age)			
		Modern <500 years	Holocene <11,000 years	Pleistocene <2 million years	Pre-Pleistocene >2 million years
(b) Coastal Zone Deposits					
Delta	Widespread	Very high	High	Low	Very low
Estuarine	Locally variable	High	Moderate	Low	Very low
Beach, high energy	Widespread	Moderate	Low	Very low	Very low
Beach, low energy	Widespread	High	Moderate	Low	Very low
Lagoon	Locally variable	High	Moderate	Low	Very low
Foreshore	Locally variable	High	Moderate	Low	Very low
(c) Fill Materials					
Uncompacted fill	Variable	Very high	-	-	-
Compacted fill	Variable	Low	-	-	-

Note:
Adapted from Youd and Perkins, 1978

Appendix C4F: Influence of Shaking Levels on Ground Stability and Liquefaction Triggering

NZGS/MBIE Module 1 - *Overview of the guidelines* includes guidance on assessing shaking hazard at a site. NZGS/MBIE Module 3 - *Identification, assessment and mitigation of liquefaction hazards* includes guidance on assessing intensity of shaking to trigger liquefaction. This appendix supplements those modules by providing an overview of slope instability and liquefaction potential at various intensities of earthquake shaking.

When discussing triggers for seismic slope instability or liquefaction, peak ground acceleration (PGA) is often referred to. This is a measure of ground acceleration at a particular site by instruments.

The Modified Mercalli Intensity (MMI) scale uses personal reports and observations to measure earthquake intensity and is therefore more subjective. As an indication of PGA force, an earthquake that results in 0.2 g may cause people to lose their balance and is approximately equivalent to MM7 (Dowrick et al., 2008).

An important step is for the engineer to determine how the land deformation may impact on the integrity of the foundation and structure in terms of life safety protection. Land damage on its own is not the problem per se: it is the effects on the performance of the structure and people that should be established. Understanding if and how the land may deform is an initial step in the assessment process.

As an initial screening tool to appreciate whether a particular PGA at a site could trigger instability or liquefaction, correlation can be made between the PGA in question (refer to NZGS/MBIE Module 1), modified for terrain amplification effects as appropriate, and the MMI, and then onto generic descriptors of land stability and building behaviour (Dowrick et al., 2008). Refer to Table C4F.1 below for examples of the correlation. The MMI-PGA correlation is extracted from Saunders and Berryman (2012).

The following table provides an approximate correlation between PGA, MMI and land damage descriptors provided by Dowrick et al., 2008. Additional comments have been added based on experiences from the Canterbury earthquake sequence of 2010-11 (comments by Dowrick et al., 2008 that are not representative of recent experience are retained in italics for reference).

Table C4F.1: Approximate correlation between PGA, MMI and land damage descriptors

PGA, g	MMI	Land descriptors*
<0.03	<MM5	Land/slope issues are unlikely.
0.03-0.08	MM5	Loose boulders may occasionally be dislodged from steep slopes.
0.08-0.15	MM6	Loose material may be dislodged from sloping ground, e.g. existing slides, talus and scree slopes. A few very small (≤10^3 m^3) soil and regolith slides and rockfalls from steep banks and cuts. A few minor cases of liquefaction (sand boil) in highly susceptible alluvial and estuarine deposits.
0.15-0.25	MM7	Small slides such as falls of sand and gravel banks, and small rockfalls from steep slopes and cuttings common. Instances of settlement of unconsolidated, or wet, or weak soils. Very small (≤10^3 m^3) disrupted soil slides and falls of sand and gravel banks, and small rockfalls from steep slopes and cuttings are common. Fine cracking on some slopes and ridge crests. A few small to moderate landslides (10^3 - 10^5 m^3), mainly rockfalls on steeper slopes (>30º) such as gorges, coastal cliffs, road cuts and excavations. Small discontinuous areas of minor shallow sliding and mobilisation of scree slopes in places. Minor to widespread small failures in road cuts in more susceptible materials. A few instances of non-damaging liquefaction (small water and sand ejections) in alluvium.
		Added comment: Widespread damaging liquefaction in alluvial soils experienced across Christchurch and environs including lateral spread.
0.25-0.45	MM8	Cracks appear on steep slopes and in wet ground. Significant landsliding likely in susceptible areas. Small to moderate (10^3-10^5 m^3) slides widespread; many rock and disrupted soil falls on steeper slopes (steep banks, terrace edges, gorges, cliffs, cuts, etc.). Significant areas of shallow regolith landsliding, and some reactivation of scree slopes. A few large (10^5-10^6 m^3) landslides from coastal cliffs, and possibly large to very large (≥10^6 m^3) rock slides and avalanches from steep mountain slopes. Larger landslides in narrow valleys may form small temporary landslide-dammed lakes. Roads damaged and blocked by small to moderate failures of cuts and slumping of road-edge fills. Increased instances of settlement of unconsolidated, or wet, or weak soils. Evidence of soil liquefaction common, with small sand boils and water ejections in alluvium, and localised lateral spreading (fissuring, sand and water ejections) and settlements along banks of rivers, lakes and canals etc.
		Added comment: Widespread severely damaging liquefaction in alluvial soils experienced across Christchurch and environs including severe lateral spread and wide-area damage to structures on shallow foundations.

PGA, g	MMI	Land descriptors*
0.45-0.60	MM9	Cracking of ground conspicuous. Landsliding widespread and damaging in susceptible terrain, particularly on slopes steeper than 20º. Extensive areas of shallow regolith failures and many rockfalls and disrupted rock and soil slides on moderate and steep slopes (20º-35º or greater), cliffs, escarpments, gorges, and man-made cuts. Many small to large (103-106 m^3) failures of regolith and bedrock, and some very large landslides (106 m^3 or greater) on steep susceptible slopes. Very large failures on coastal cliffs and low-angle bedding planes in Tertiary rocks. Large rock/debris avalanches on steep mountain slopes in well-jointed greywacke and granitic rocks. Landslide-dammed lakes formed by large landslides in narrow valleys. Damage to road and rail infrastructure widespread with moderate to large failures of road cuts and slumping of road-edge fills. Small to large cut slope failures and rockfalls in open mines and quarries. Liquefaction effects widespread with numerous sand boils and water ejections on alluvial plains, and extensive, potentially damaging lateral spreading (fissuring and sand ejections) along banks of rivers, lakes, canals, etc.). Spreading and settlements of river stop-banks likely.
		Added comment: Widespread severely damaging liquefaction in alluvial soils experienced across Christchurch and environs including severe lateral spread.
0.60-0.80	MM10	Landsliding very widespread in susceptible terrain. Similar effects to MM9, but more intensive and severe, with very large rock masses displaced on steep mountain slopes and coastal cliffs. Landslide-dammed lakes formed. Many moderate to large failures of road and rail cuts and slumping of road-edge fills and embankments may cause great damage and closure of roads and railway lines. Liquefaction effects (as for MM9) widespread and severe. Lateral spreading and slumping may cause rents over large areas, causing extensive damage, particularly along river banks, and affecting bridges, wharfs, port facilities, and road and rail embankments on swampy, alluvial or estuarine areas.
080-0.90	MM11	Environmental response criteria have not been suggested for MM11 as that level of shaking has not been reported in New Zealand or (definitively) elsewhere.
> 0.90	MM12	As above.

Note:

***** Land descriptors are based on Dowrick et al. (2008). Comments that do not reflect recent experience are retained (in italics) for reference. Refer to Dowrick et al. (2008) for full descriptors of building damage.

Additional comments (in bold) are based on experiences from the Canterbury earthquake sequence.

PART C

Concrete Buildings C5

Document Status

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1	July 2017	Initial release

This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

This document may be downloaded from www.EQ-Assess.org.nz in parts:

1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

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Please go to www.EQ-Assess.org.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.EQ-Assess.org.nz

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C5. Concrete Buildings

C5.1 General

C5.1.1 Scope and outline of this section

This section provides guidelines for performing a Detailed Seismic Assessment (DSA) for existing reinforced concrete (RC) buildings from the material properties to section, member, element/component, sub-assembly, and ultimately the system level. Unreinforced concrete structures are not addressed.

The overall aim is to provide engineers with:

- an understanding of the underlining issues associated with the seismic response of RC buildings (including the presence of inherent vulnerabilities or weaknesses), and
- a set of assessment tools based on different levels of complexity (not necessarily corresponding to different levels of reliability) for the DSA of the behaviour of RC buildings, with particular reference to evaluation of *%NBS*.

Note:

This section is based on the latest information and knowledge relating to the seismic behaviour of existing RC buildings which has been developed and gained over the last 15 years at both the national and international level. It also draws on international standards and guidelines on seismic assessment and strengthening/retrofitting, with the aim of adapting and integrating best practice to best suit New Zealand conditions.

Increased knowledge in relation to RC buildings has been obtained through extensive experimental and analytical/numerical investigations, and also through damage observations and lessons learned following major earthquakes. In particular, there have been two significant projects relating to New Zealand construction practice:

- the Foundation of Research Science and Technology (FRST) research project 'Retrofit Solutions for New Zealand Multi-storey Buildings' , which was carried out jointly by the University of Canterbury and University of Auckland from 2004 to 2010, and
- the 'SAFER Concrete Technology' Project (2011-2015), funded by the Natural Hazard Research Platform (NHRP).

These projects have provided very valuable evidence-based information on the expected seismic performance of concrete buildings designed and constructed according to New Zealand practice and Building Code provisions. (For an overview of these findings refer to Pampanin, 2009, and for more details refer to Marriott, 2009; Kam, 2011; Akguzel, 2011; Genesio, 2012; and Quintana-Gallo, 2014.)

More recently, the Canterbury earthquake sequence of 2010-2011 has represented a unique "open-air laboratory" and an important source of information for assessing and evaluating the actual seismic performance of New Zealand RC buildings of different structural type, age, construction practice and design details. The effects of the 2016 Kaikoura Earthquake on taller RC buildings in Wellington, particularly those containing precast floor systems, also represent yet another opportunity to consider the actual seismic performance of this type of building.

Recent experience has highlighted a number of key structural weaknesses and failure mechanisms, either at an element level or at a global system level. It has not only confirmed that pre-1970s RC buildings – as expected – have a potentially high inherent seismic vulnerability, but also that some modern (e.g. post-1980s) RC buildings can be expected to perform poorly. In some cases, this has led to catastrophic collapses or "near misses". This has been a wake-up call as it has identified a "new generation" of potentially vulnerable buildings that need to be scrutinised with care.

This section of the guidelines attempts to capture these new learnings and provide up to date procedures for evaluating the vulnerability of existing RC buildings and for determining their earthquake rating. It dedicates specific effort to describing, both qualitatively and quantitatively, key aspects of the local and global mechanisms and their impact on the building response. This is to provide engineers with a more holistic understanding of the overall building capacity and expected performance, which is essential when determining the earthquake rating for a building.

Note:

Most RC buildings designed post-1976 can be expected to have a relatively low probability of collapse under ULS level earthquake shaking.

However, some of these buildings can still have structural weaknesses – even severe structural weaknesses, such as non-ductile gravity columns with low drift capacity – which could lead to a progressive and catastrophic collapse in severe earthquakes.

This section covers in turn:

- typical building practices, structural deficiencies and observed behaviour of RC buildings in earthquakes (refer to Sections C5.2 to C5.3)
- material properties and testing, element probable capacities and global system capacities (Sections C5.4 to C5.8), and
- brief comments on improving RC buildings (Section C5.9).

Given their importance in the overall behaviour of a building system, as emphasised by the lessons learnt in recent earthquakes, RC floor/diaphragms and their interactions with the main vertical lateral load-resisting systems are covered in some detail in Section C5.5.4.

This material should be read in conjunction with the more general guidance outlined in Section C2.

Note:

An appreciation of the observed behaviour of a building in the context of its age and the detailing present is considered an essential part of assessing its earthquake rating.

Sections C5.2 and C5.3, referred to above, provide important context for any assessment of RC buildings and include findings from the Canterbury earthquake sequence of 2010-11. It is expected that an engineer, having read these sections and being familiar with them, will thereafter be able to concentrate on Sections C5.4 to C5.8 and their associated appendices, which contain the specific assessment requirements.

The appendices to this section summarise:

- the evolution of New Zealand concrete design standards and code-based reinforcing requirements (refer to Appendix C5A)
- historical concrete property requirements, design specifications and strength testing in New Zealand (Appendix C5B)
- the evolution of steel reinforcing standards in New Zealand, including reference values for the mechanical properties of the reinforcing steel depending on the age of construction (Appendix C5C)
- material test methods for concrete and reinforcing steel (Appendix C5D), and
- the evolution of standard based design details for reinforcement and detailing (Appendix C5E).

The appendices also discuss:

- diaphragm grillage modelling (Appendix C5F)
- assessing the deformation capacity of precast concrete floor systems (Appendix C5G)
- assessing the buckling of vertical reinforcement in shear walls (Appendix C5H)
- procedure for evaluating the equivalent "moment" capacity of a joint (Appendix C5I)
- establishing the internal hierarchy of strength and sequence of mechanisms in a column (Appendix C5J).

Note:

The impact of masonry infills on the performance of the primary structural systems is covered in Section C7. The effects of Soil-Structure Interaction (SSI) in terms of seismic performance, modifications of demand and development of mixed mechanisms are discussed in Section C4.

C5.1.2 Useful publications

A short list of key publications follows. A more comprehensive list is provided at the end of this section and is referenced throughout.

ASCE 41-13 (2014). *Seismic evaluation and retrofit of existing buildings,* American Society of Civil Engineers, and Structural Engineering Institute, Reston, Virginia, USA.

ATC 78-3 (2015). *Seismic evaluation of older concrete frame buildings for collapse potential,* Applied Technology Council (ATC), Redwood City, California, USA.

FEMA P-58 (2012). *Seismic performance assessment of buildings,* Applied Technology Council (ATC), Redwood City, California, USA.

EN 1998-3:2005. *Eurocode 8: Design of structures for earthquake resistance*, *Part 3: Assessment and retrofitting of buildings*, European Committee for Standardization (CEN), Updated in 2005.

FEMA-547 (2006). *Techniques for the seismic rehabilitation of existing buildings*, Federal Emergency Management Agency, Washington, DC.

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Pampanin, S. (2009). *Alternative performance-based retrofit strategies and solutions for existing R.C. buildings*, Series "Geotechnical, Geological, and Earthquake Engineering, Volume 10" Chapter 13 within the Book *Seismic risk assessment and retrofitting - with special emphasis on existing low rise structures* (Editors: Ilki, A., Karadogan, F., Pala, S. and Yuksel, E.,), Publisher Springer, 267-295.

C5.1.3 Definitions and acronyms

ADRS	Acceleration-displacement response spectrum
Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable yield capacity is exceeded. A brittle structure has little tendency to deform before it fractures.
Critical structural weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be *potential* CSWs.
Damping	The value of equivalent viscous damping corresponding to the energy dissipated by the structure, or its systems and elements, during the earthquake. It is generally used in nonlinear assessment procedures. For elastic procedures, a constant 5% damping as per NZS 1170.5:2004 is used.
Design level/ULS earthquake	Design level earthquake or loading is taken to be the seismic load level corresponding to the ULS seismic load for the building at the site as defined by NZS 1170.5:2004 (refer to Section C3)
Detailed Seismic Assessment (DSA)	A seismic assessment carried out in accordance with Part C of these guidelines
Diaphragm	A horizontal structural element (usually a suspended floor or ceiling or a braced roof structure) that is strongly connected to the vertical elements around it and that distributes earthquake lateral forces to vertical elements, such as walls, of the primary lateral system. Diaphragms can be classified as flexible or rigid.
Ductile/ductility	Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
Elastic analysis	Structural analysis technique that relies on linear-elastic assumptions and maintains the use of linear stress-strain and force-displacement relationships. Implicit material nonlinearity (e.g. cracked section) and geometric nonlinearity may be included. Includes equivalent static analysis and modal response spectrum dynamic analysis.
Flexible diaphragm	A diaphragm which for practical purposes is considered so flexible that it is unable to transfer the earthquake loads to shear walls even if the floors/roof are well connected to the walls. Floors and roofs constructed of timber, and/or steel bracing in a URM building, or precast concrete without reinforced concrete topping fall in this category. A diaphragm with a maximum horizontal deformation along its length that is greater than or equal to twice the average inter-storey drift. In a URM building a diaphragm constructed of timber and/or steel bracing.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.
Nonlinear analysis	Structural analysis technique that incorporates the material nonlinearity (strength, stiffness and hysteretic behaviour) as part of the analysis. Includes nonlinear static (pushover) analysis and nonlinear time history dynamic analysis.
Non-structural item	An item within the building that is not considered to be part of either the primary or secondary structure. Non-structural items such as individual window glazing, ceilings, general building services and building contents are not typically included in the assessment of the building's earthquake rating.
OTM	Overturning moment

Primary gravity structure	Portion of the main building structural system identified as carrying the gravity loads through to the ground. Also required to carry vertical earthquake induced accelerations through to the ground. May also incorporate the primary lateral structure.
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be the primary gravity structure.
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects this is determined using probable material strengths. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Rigid diaphragm	A diaphragm that is not a flexible diaphragm
Secondary structure	Portion of the structure that is not part of either the primary lateral or primary gravity structure but, nevertheless, is required to transfer inertial and vertical loads for which assessment/design by a structural engineer would be expected. Includes precast panels, curtain wall framing systems, stairs and supports to significant building services items
Serviceability limit state (SLS)	Limit state as defined in AS/NZS 1170.0:2002 (or NZS 4203:1992) being the point at which the structure can no longer be used as originally intended without repair
Severe structural weakness (SSW)	A defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge
Simple Lateral Mechanism Analysis (SLaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (pushover) relationship for the building as a whole
Single-degree-of-freedom (SDOF)	A simple inverted pendulum system with a single mass
Structural element	Combinations of structural members that can be considered to work together; e.g. the piers and spandrels in a penetrated wall, or beams and columns in a moment resisting frame
Structural member	Individual items of a building structure, e.g. beams, columns, beam/column joints, walls, spandrels, piers
Structural sub-system	Combination of structural elements that form a recognisable means of lateral or gravity load support for a portion of the building: e.g. moment resisting frame, frame/wall. The combination of all of the sub-systems creates the structural system.
Structural system	Combinations of structural elements that form a recognisable means of lateral or gravity load support; e.g. moment resisting frame, frame/wall. Also used to describe the way in which support/restraint is provided by the foundation soils.
Structural weakness (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100*%NBS.* Note that an aspect of the building structure scoring less than 100*%NBS* but greater than or equal to 67*%NBS* is still considered to be a SW even though it is considered to represent an acceptable risk.
Ultimate limit state (seismic)	A term defined in regulations that describes the limiting capacity of a building for it to be determined to be an earthquake-prone building. This is typically taken as the probable capacity but with the additional requirement that exceeding the probable capacity must be associated with the loss of gravity support (i.e. creates a significant life safety hazard).
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings

XXX%*NBS*	The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage. Intended to reflect the expected seismic performance of a building relative to the minimum life safety standard required for a similar new building on the same site by Clause B1 of the New Zealand Building Code.
XXX%ULS shaking (demand)	Percentage of the ULS shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.

C5.1.4 Notation, symbols and abbreviations

Symbol	Meaning
$\%NBS$	Percentage of new building standard as calculated by application of these guidelines
a	Depth of the compression stress block (=βc)
A_{bb}	Displacement at the onset of bar buckling
A_{g}	Gross area of the member section
A_{r}	Wall aspect ratio
A_{s}	Area of reinforcement in tension
A_{s}'	Area of reinforcement in compression
A_{sp}	Area of spiral or circular hoop bar
A_{st}	Area of transverse reinforcement parallel to the applied shear
A_{st}	Area of transverse reinforcement parallel to the applied shear
A_{t}	Area of the transverse stirrups
A_{v}	Area of transverse shear reinforcement at spacing s
$As_{,\mathrm{eff}}$	Area of the effective steel of the slab
b_0	Effective width of the spandrel for torsion
b_{b}	Beam width
b_{c}	Column width
b_{core}	Width of column core, measured from centre to centre of the peripheral transverse reinforcement in the web
b_{eff}	Effective width of the slab
b_{j}	Effective width of the joint
b_{w}	Web width
b_{w}	Width of beam web
C	Neutral axis depth
C	Resultant of compression stresses in concrete
C'	Resultant of compression stresses in compression reinforcement
D	Section effective depth
d"	Depth of the concrete core of the column measured in the direction of the shear force for rectangular hoops, and the diameter of the concrete core for spirals or circular hoops
s	Spacing of transverse shear reinforcement
c_0	Cover to longitudinal bars
c_{u}	Neutral axis depth at ultimate curvature
d_{b}	Average diameter of longitudinal reinforcement
E_{s}	Steel elastic modulus

Symbol	Meaning
f'_c	Probable concrete compressive strength
f'_{cc}	Probable confined concrete compressive strength
f_{st}	Stress in the steel related to the maximum tensile strain in the first part of the cycle
f_u	Probable ultimate strength of the longitudinal reinforcement
f_v	Normal stress in the vertical direction
f_y	Probable yielding strength of the longitudinal reinforcement
$f_{y/slab}$	Yielding stress of the slab steel in tension
F_{yt}	Yielding stress of the transverse steel
f_{yt}	Probable yield strength of the transverse reinforcement
H	Height of the member
h_b	Beam height
h_c	Column height
h_{cr}	Vertical height of inclined crack
h_t	Height of the transverse beam or spandrel
h_w	Wall height
J_d	Internal couple lever arm
K	Shear stress degradation factor
K_d	Neutral axis depth when tension steel reaches the strain at first yield, ε_y
k_j	Coefficient for calculating the shear capacity of a joint
k_{lp}	Coefficient related to the plastic hinge calculation
k_{wall}	Shear coefficient related to concrete mechanism
l_b	Half of the length of the beam
L_c	Shear span, distance of the critical section from the point of contra flexure
l_c	Total length of the column
l_{cr}	Horizontal length of inclined crack
l_d	Theoretical development length
$l_{d,prov}$	Provided lap length
$l_{d,req}$	Required lap length
L_p	Plastic hinge length
L_{sp}	Strain penetration length
l_w	Wall length
M	Bending moment
M_b	Moment in the beam (at the interface with the column)

Symbol	Meaning
M_{col}	Equivalent moment in the column (at the level of the top face of the beam)
M_{f}	Residual moment capacity of an element
M_{lap}	Moment capacity of a lap splice
M_{n}	Probable flexural moment capacity of an element
$M_{\mathrm{p,wall}}$	Wall probable flexural strength
N	Axial load
N^*	Total axial load: gravity plus seismic.
p_{t}, p_{c}	Tensile and compressive average principal stresses in the joint panel
S_{n}	Nominal strength capacity
S_{o}	Overstrength capacity
S_{prob}	Probable strength capacity
s_{t}	Spacing in between stirrups in the spandrel
T	Resultant of tension stresses in tension reinforcement
V	Shear
V	Maximum nominal shear stress
V_{b}	Shear force in the beam
V_{c}	Shear resisted by the concrete mechanisms
V_{c}	Shear force in the column
v_{c}	Nominal shear stress carried by concrete mechanism
$V_{\mathrm{c,wall}}$	Shear resisted by the concrete mechanisms
v_{ch}	Nominal horizontal joint shear stress carried by a diagonal compressive strut mechanism crossing joint
V_{jh}	Average shear stress in the joint panel
V_{jh}	Horizontal joint shear force
V_{n}	Shear resisted as a result of the axial compressive load
$V_{\mathrm{n,wall}}$	Shear resisted as a result of the axial compressive load
V_{p}	Probable shear strength capacity of an element
$V_{\mathrm{p,wall}}$	Wall probable shear strength
V_{pjh}	Probable horizontal joint shear force
V_{s}	Shear resisted by the transverse shear reinforcement
$V_{\mathrm{s,wall}}$	Shear resisted by the horizontal transverse shear reinforcement
α'	Shear coefficient related to section aspect ratio
α, β	Stress block parameters
α'_{wall}	Shear coefficient related to section aspect ratio
β'	Shear coefficient related to longitudinal reinforcement ratio

Symbol	Meaning
β'_{wall}	Shear coefficient related to longitudinal reinforcement ratio
γ	Inclination angle of axial load compressive truss
$\gamma_{\text{bb}} l_{\text{w}}$	Wall core length
Δ_{p}	Plastic displacement
Δ_{u}	Ultimate displacement
Δ_{y}	Yielding displacement
δ^*_{p}	Plastic displacement at the onset of bar buckling
ε^+_0	Tensile strain in the steel at zero stress
$\varepsilon^{\text{r}}_{\text{cm}}$	Concrete strain at the onset of bar buckling (reversed actions)
ε^*_{p}	Steel plastic strain at the onset of bar buckling
ε_{cu}	Concrete ultimate compressive strain
ε_{s}	Tension steel strain
$\varepsilon_{\text{s.cr}}$	Steel tensile strain at the onset of bar buckling (cyclic actions)
ε_{sh}	Strain at the end of the yielding plateau
ε_{st}	Maximum tensile strain in the steel in the first part of the cycle
$\varepsilon_{\text{su,b}}$	Steel tensile strain at the onset of bar buckling (monotonic actions)
ε_{su}	Steel ultimate tensile strain
ε_{y}	Strain at first yield of the longitudinal tension reinforcement
θ	Rotation (or drift ratio)
θ_{cr}	Average cracking angle
θ_{p}	Plastic rotation (or drift ratio)
θ_{u}	Ultimate rotation (or drift ratio)
θ_{y}	Yielding rotation (or drift ratio)
μ_{Δ}	Displacement ductility
$\mu_{\Delta\text{c}}$	Displacement ductility capacity
$\mu_{\Delta\text{d}}$	Displacement ductility demand
μ_{ϕ}	Curvature ductility
ρ_{eff}	Effective confinement ratio
ρ_{ℓ}	Longitudinal reinforcement ratio
ρ_{s}	Volume of transverse reinforcement to volume of concrete core ratio
ϕ	Curvature
ϕ^*_{u}	Curvature at the onset of bar buckling
ϕ_{p}	Plastic curvature

Symbol	Meaning
ϕ_u	Ultimate curvature
ϕ_y	First yield curvature
Ψ_1	Coefficient for calculating the development length
Ψ_2	Coefficient for calculating the development length
Ψ_a	Coefficient for calculating the development length
Ψ_b	Coefficient for calculating the development length

C5.2 Typical Concrete Building Practices in New Zealand

C5.2.1 General

Construction methods for RC buildings in New Zealand have changed significantly over the years since their first appearance in the early 1900s. The evolution of construction methods matches the evolution of the relevant codes and standards in line with increasing understanding of the behaviour of these buildings in earthquakes.

An understanding of the development of seismic design provisions for RC buildings is relevant for the engineer as it often provides valuable insight into why certain detailing decisions were made and the need to recognise the presence of SSWs (refer to Section C1), particularly where deformation capacity might be limited.

Developments in the design requirements for RC buildings and the corresponding evolution of loading standards are summarised in Appendix C5A, along with some pointers on what to look for in RC buildings of the corresponding eras. An overview of the key historical code developments is given in this section.

Note:

For a more detailed comparison of New Zealand standards used for seismic design of RC buildings refer to Fenwick and MacRae, 2009. A summary of the evolution of earthquake engineering codified requirements in New Zealand has also been provided by Kam and Pampanin (2012).

C5.2.2 1920s to 1950s: early years of seismic design

The first known New Zealand publication on earthquake design was written by C. Reginald Ford (Ford, 1926) in 1926, several years before the 7.8 magnitude Napier earthquake of 1931 that dramatically changed New Zealand construction practice. Ford's description drew heavily from the state of knowledge and lessons following the San Francisco (1906) and Kanto, Japan (1923) earthquakes. However, the significant loss of lives and devastation following the 1931 Napier earthquake provided the government with the impetus to legislate building construction in relation to earthquake resistance. A Building Regulations Committee was set up and reported on a draft earthquake building by-law, which was presented to the New Zealand Parliament in June 1931 (Cull, 1931). This draft building by-law was subsequently published by New Zealand standards as the 1935 New Zealand Standard (NZS) Model Building By-Law (NZSS 95:1935) and the 1939 NZS Code of Building By-Laws (NZSS 95:1939).

The 1935 by-law (NZSS 95:1935) was not compulsory and depended on adoption by local territorial authorities. There were no specific recommendations for the design of concrete buildings. However, it is interesting to note that 135 degree hooks were already shown for stirrups in reinforced construction (clause 409 of NZSS 95:1935).

The 1955 revision of the NZS Standard Model Building By-Law (NZSS 95:1955) introduced changes but lacked significant improvement in terms of seismic structural detailing. For example, while it gave explicit definitions for deformed bars (which were only

introduced in New Zealand in the mid-1960s) and plain round bars, it only specified 10% higher allowable bond stresses for deformed bars. The provisions for shear resistance of concrete elements were tightened and the requirement of 135° anchorage for stirrups was included. However, no other specific seismic details for reinforced concrete structures were specified.

C5.2.3 1960s to mid-1970s: advent of structural ductility

The NZS 1900:1964 code (NZS 1900.8:1965; NZS 1900.9:1964) was a significant evolution from its predecessors. It showed increased understanding of RC seismic design, and was also based on best international practice and knowledge (ACI 318-63, 1963, CEB-1964, 1964).

This code introduced the concept of structural ductility with the stated assumption of 5-10% damping for structural ductility $\mu = 4$ for RC structures. However, no provision for ductile RC detailing or modern capacity design considerations (yet to be developed) was included.

Notably, NZS 1900:1964 was still based on the working (allowable) stress concept for member design while the international trend, in particular for RC design provisions or Model Codes (fib), was starting to move towards the introduction of limit state design concepts (ACI 318-63, 1963; CEB-1964, 1964).

In 1961, work by Blume, Newmark and Corning (Blume, et al., 1961) had pioneered the concept of ductile RC buildings and introduced detailing for ductile RC elements. As the 1960s and 1970s progressed, there were significant developments in earthquake engineering internationally, as summarised in the 1966-1973 Structural Engineers Association of California (SEAOC) recommendations (SEAOC, 1966; SEAOC, 1973) and the 1971 ACI-318 concrete code (ACI 318-71, 1971). The need for beam-column joint seismic design, different ductility coefficient for different lateral-resisting systems and ductile RC detailing were identified in these documents.

However, the 1971 ACI-318 code (ACI 318-71, 1971) did not contain any of the capacity design provisions which were developed in New Zealand in the late 1960s-1970s (Park and Paulay, 1975). As a result, without explicit design for lateral-force resistance, for example, buildings constructed before the NZSS 95:1955 provisions were introduced – or pre-1970s RC frames more generally – are unlikely to have sufficient lateral strength capacity or adequate lateral stiffness because of small column dimensions (proportioned primarily for gravity loads).

In 1969, J.P. Hollings (Hollings, 1969) introduced a step-by-step design procedure to achieve a beam-hinging inelastic mechanism in RC frames under seismic loading, which was a precursor of the concept of capacity design. The 1968-1970 Ministry of Work's Code of Practice for Design of Public Buildings (Fenwick and MacRae, 2009; Megget, 2006; MOW-NZ, 1968-1970) adopted many ductile detailing recommendations from the 1966 SEAOC recommendations (SEAOC, 1966) and the 1971 ACI-318 code (ACI 318-71, 1971).

Park and Paulay's seminal publication of 1975 (Park and Paulay, 1975) outlined many concepts of modern seismic RC design and detailing, including a rigorous design procedure of RC frames under the capacity design philosophy and quantification of the ductility capacity of RC beam, column, wall and joint elements. These innovations were quickly

disseminated in New Zealand engineering practice and building standards (NZS 3101:2006) from the mid-1970s onwards.

C5.2.4 Mid-1970s onwards: modern seismic design

The introduction of the NZS 4203:1976 loading standard represented a quantum change in the approach to seismic design. The limit state approach using defined Ultimate Limit State (ULS) and Serviceability Limit State (SLS) was codified as the preferred design method over the working stress approach. Ductility was required to be explicitly allowed for (as per the 1966 SEAOC recommendations). Structures without any ductile detailing were required to be designed for higher seismic loading.

In the same period, the provisional NZS 3101 concrete standard, published in 1972 (NZS 3101:1970P) also adopted many parts of the 1971 ACI-318 code (ACI 318-71, 1971) and some recommendations from the draft of Park and Paulay's publication (Park and Paulay, 1975). It introduced some detailing of plastic hinge regions with a focus on shear reinforcement, lapping of bars and column confinement.

However, it was not until the revamp of the New Zealand loading code NZS 4203 in 1976, the update of the ACI-318 code in 1977 and the various drafts of the 1982 edition of the NZS 3101 concrete design standard (NZS 3101:1982) that modern seismic design for RC buildings was fully codified in New Zealand.

NZS 3101:1982 provided improved requirements in the detailing of plastic hinge regions, including shear, confinement and anti-buckling reinforcement. Lapped bars were not permitted at any floor levels in columns where there was a possibility of yielding. Column ties were required to be anchored by 135 degrees in cover concrete. Improved methods of determining spacing of transverse reinforcement for seismic columns were provided. A strong-column weak beam mechanism was explicitly specified in the commentary of this standard, with requirements to account for overstrength moments including flange effects from the slab.

NZS 3101:1982 was reviewed and updated in 1995 and 2006 to reflect further knowledge from research, the revisions of the NZS 4203 loading standard (NZS 4203:1976) in 1992 and the introduction of the NZS 1170 loading standard (NZS 1170.5: 2004) in 2004.

As an example of key improvements between 1982 and 1995, both in conceptual design and required details, a potential "deficiency" in the 1982 code relating to the design of gravity columns (now typically referred to as pre-1995 '"non-ductile" columns) was removed when improved provisions were included in NZS 3101:1995.

Note:

The period from the late 1970s through to the 1990s is one in which the knowledge of seismic performance of buildings improved significantly. As a result, the development of standards over this period often lagged behind the published research. In New Zealand the Bulletin of the New Zealand National Society for Earthquake Engineering, BNZSEE, published a number of papers that were the precursor of provisions which ultimately translated into design requirements. Designers often incorporated these refinements into their designs long before the provisions were cited in the standards.

For this reason any assumptions regarding detailing that are based solely on the date of design/construction should be approached with care. Non-invasive and/or intrusive investigations will be required to confirm such assumptions when these are found to be key to the assessed behaviour of the building.

C5.3 Observed Behaviour of Reinforced Concrete Buildings

C5.3.1 General

Extensive experimental and analytical investigations into the seismic vulnerability and response/performance of RC buildings, together with observations of damage in past earthquakes (including the Canterbury earthquake sequence of 2010/11) have highlighted a series of typical structural deficiencies in RC buildings.

These include:

- inadequate transverse reinforcement for shear and confinement in potential plastic hinge regions
- insufficient transverse reinforcement in beam-column joint core regions
- insufficient and inadequate detailing of column longitudinal and transverse reinforcement
- inadequate anchorage detailing in general, for both longitudinal and transverse reinforcement
- insufficient lap splices of column reinforcement just above the floor or at the foundation level, or of beam reinforcement in regions where the gravity moments are high
- insufficient shear, anti-buckling and confining/restraining reinforcement in wall systems
- insufficient longitudinal reinforcement ratio in walls, combined with higher than expected tensile strength in the concrete, leading to single crack opening when compared to a spread plastic hinge, resulting in failure in tension of the rebars
- inadequate capacity of the foundations to account for overturning moment caused by lateral loading
- lower quality of materials (concrete and steel) when compared to current practice; in particular:
  - use of low grade plain round (smooth) bars for both longitudinal (until the mid-1960s) and transverse reinforcement
  - low-strength concrete (below 20-25 MPa and, in extreme cases, below 10 MPa)
- potential brittle failure mechanisms at both local and global level due to interaction with spandrel beams, masonry infills, façades causing shear failure in columns (due to short/captive column effects) and/or potential soft-storey mechanisms
- lack of (horizontal and vertical) displacement compatibility considerations between the lateral load resisting systems (either frames, walls or a combination of these), the floor-diaphragms, and gravity load bearing systems (e.g. non-ductile columns with limited confinement details and drift capacity)
- inadequate design of diaphragm actions and connection detailing; particularly in the case of precast concrete floor systems which became common from the 1980s onwards
- inadequate protection against punching shear between columns and flat-slab connections
- plan and vertical irregularity, resulting in unexpected amplification and concentration of demands on beams, walls and columns

- limited and inadequate consideration of bidirectional loading effect on critical structural elements (e.g. columns, walls, or beam-column joints), and
- lack of, or inadequate consideration of, capacity design principles. While this is more typical of pre mid-1970s RC buildings (before the introduction of NZS 4203:1976 and the capacity design concept itself), it can also arise in later buildings as this concept was under continuous refinement in further generations of building standards.

It is worth noting that often structural deficiencies are not isolated. Brittle failure mechanisms can be expected either at local level (e.g. shear failure in the joints, columns or beams) or global level (e.g. soft-storey mechanisms). The presence of multiple structural deficiencies and lack of an alternative robust load path – i.e. lack of redundancy/robustness – can trigger progressive collapse with catastrophic consequences, as evident in the 22 February 2011 Christchurch (Lyttleton) earthquake.

Note:

While the deficiencies listed above have been shown to reduce the performance of RC buildings, non-compliance with current standards is not necessarily an indication of inadequate performance when compared against the minimum requirements of the Building Code. The effect of the deficiencies on the building behaviour and therefore its earthquake rating will depend on their location and criticality and the assessed impact of failure on life safety.

The following sections discuss the behaviour of non-ductile columns and shear walls, and also include observations made following the Canterbury earthquake sequence.

C5.3.2 Non-ductile (gravity) columns

Gravity columns are common in structural systems that contain shear walls, seismic frames, or a combination of both as the lateral load resisting system. These columns are generally required to support often significant areas of floor, while not being relied upon to contribute to the strength of the lateral system. In order to perform this function they must remain capable of carrying axial load while undergoing the required lateral displacements of the structural system.

If these displacements are particularly large and/or the axial loads in the columns are large, there is the potential for the gravity columns to be a severe structural weakness (SSW) with potentially catastrophic consequences.

The poor performance of reinforced concrete columns with inadequate detailing, such as inadequate transverse reinforcement, lap-splices in the plastic hinge region and possibly longitudinal rebars ‘cranked’ at the end of the lap splices, has been observed in various past earthquakes (refer to Figure C5.1) and investigated in recent literature (in particular, Boys et al., 2008; Elwood and Moehle, 2005; and Kam et al., 2011).

(a) Indian Hills Medical Centre (1994 Northridge earthquake)

(b) Olive View Hospital (1971 San Fernando earthquake)

Figure C5.1: Examples of failure of inadequately reinforced columns in past earthquakes

In addition to older (pre-1970s) details, which were expected to have a number of deficiencies, a potential loophole in the NZS 3101:1982 design standard was identified for the detailing of columns designed according to post-1982 and pre-1995 code specifications.

Note:

Experimental tests conducted at the University of Canterbury by Boys et al. in 2008 (and therefore before the Canterbury earthquake sequence of 2010/11), which reflected New Zealand construction and design detailing, highlighted the potentially high vulnerability of gravity columns with inadequate/poor detailing to sustain lateral displacements.

These tests comprised both unidirectional and bidirectional loading testing regimes. They showed the low displacement/drift capacity of such columns, which was exacerbated by a bidirectional loading regime (more realistically representing the actual response of a building under a ground motion).

Figure C5.2 presents examples of axial-shear failure of non-ductile gravity columns simulated in this laboratory testing under unidirectional cyclic loading.

In general, the (limited) experimental tests that were carried out confirmed that the equations proposed for axial-shear failure of columns according to the Elwood-Moehle model (Elwood and Moehle, 2005) capture the displacements at which shear-dominated RC columns subject to unidirectional loading lose their axial load carrying capacity (Boys et al., 2008).

However, in many cases, and particularly when subjecting the column specimens to bidirectional loading, failure with loss of axial load capacity occurred at very low lateral drift levels: in the range of 1-1.5%.

Figure C5.2: Performance of poorly detailed and confined gravity columns designed according to NZS 3101:1982 code provisions (after Boys et al., 2008)

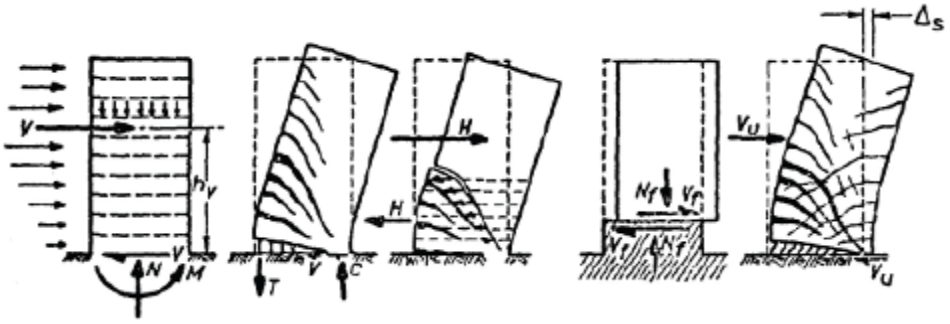
C5.3.3 Failure mechanisms for shear walls

Depending on the geometric and mechanical characteristics (reinforcing details and layout) and on the demand (unidirectional or bidirectional, level of axial load and moment/shear), structural (shear) walls can develop alternative and complex mechanisms as demonstrated in extensive experimental testing in structural laboratories as well as by damage observed following major earthquakes.

Figure C5.3 gives an overview of the most commonly expected and analysed failure mechanisms in shear walls under unidirectional loading (Paulay, 1981).

In addition to the most desirable flexural yielding of the longitudinal reinforcement in the plastic hinge region (b), alternative failure modes such as diagonal tension (c) or diagonal compression due to shear, instability of thin walled sections or buckling of the main compression reinforcement (refer to Appendix C5H), sliding shear along the construction joints (d) and shear or bond failure along lapped splices or anchorage can occur and should be assessed.

Poor or inadequate detailing can lead to a severe and sudden strength degradation; potentially at relatively low levels of lateral displacement/drift demand.



(a) Wall actions **(b) Flexure** **(c) Diagonal tension** **(d) Sliding shear** **(e) Hinge sliding**

Figure C5.3: Various failure modes of cantilevered shear walls (Paulay, 1981)

Note:

Concrete walls in buildings constructed before the importance of the ductile capacity was recognised will typically have low levels of shear and confinement reinforcing.

Anti-buckling and confinement stirrups and ties were not required before NZS 3101:1982. Compression zone ductile detailing was introduced at that time, with specific requirements to limit the extreme fibre compressive strain or provide boundary confining stirrups.

Furthermore, pre-1970s concrete walls were typically constructed as infill panels in between concrete columns and perforated with multiple openings. Typical pre-1970s walls (for low to mid-rise buildings) were 6" to 8" thick (approx. 150-200 mm) and lightly reinforced with 3/8" or ¼" bars at 8" to 12" centres (approx. 200-300 mm). However, the increase in flexural capacity of the wall including the longitudinal reinforcement of the boundary columns may result in increased shear demands and a brittle shear-dominated inelastic mechanism.

The major Chile earthquake of 2010 and the Canterbury earthquake sequence of 2010-2011 provided real examples of most, if not all, of the "traditional" mechanisms referred to earlier (NZSEE 2010-2011 and EERI/NZSEE 2015 Special Issues dedicated to the Canterbury Earthquake sequence; e. g. Kam et al., 2010, 2011; Fleischman et al., 2014; Sritharan et al.; 2014 and Bech et al., 2014).

In addition, a number of alternative failure mechanisms have been observed. These include:

- out-of-plane instability of doubly reinforced, well confined and not necessarily "thin" (as typically considered) walls
- diagonal compression-shear failure of walls due to interaction (displacement compatibility) with the floor system during the uplifting
- out-of-plane shear/sliding failure at lap-splice level, in part due to bidirectional loading effects, and
- flexural tension failure of singly reinforced walls with low-reinforcement ratios.

The key parameters controlling the behaviour and alternative mechanisms of walls are both geometrical and mechanical:

- element shear span ratio (V/M), i.e. squat vs. tall
- section aspect ratio (L_w/t_w)
- slenderness ratio (H/t_w)
- longitudinal reinforcement ratio in the boundary elements and in the core (ρ_l)
- transverse reinforcement and confinement details in the boundary regions, and
- axial load ratio ($N/f'_c A_c$).

Note:

Following observations of the relatively poor performance of existing walls in the aftermath of the Chile and Canterbury earthquakes, there is an ongoing and internationally coordinated research effort under the name of "Wall International Institute". The purpose of this research, which is based on experimental, numerical and analytical investigations, is to improve the understanding of shear wall building behaviour (at local, member and

global system level) in order to refine current provisions both for new design and the assessment of existing walls (Wallace et al., 2016).

The methods described in these guidelines (either in the core text or in the appendices) are based on the latest knowledge and will be updated as new research evidence becomes available in the near future.

C5.3.4 Typical deficiencies in beam-column joint design and detailing

Older RC buildings can be characterised by a number of different construction practices and structural detailing for beam-column connections. Typical inadequacies can be related to the:

- lack or absence of horizontal and/or vertical transverse reinforcement
- non-ductile anchorage of beam longitudinal bars into the joint, and
- lack of reliable joint shear transfer mechanism beyond diagonal cracking.

The primary deficiency of older beam-column joints, particularly before the 1970s, was the inadequate joint shear reinforcement. In fact, in older construction practice beam-column joints were treated either as construction joints or as part of the columns. Consequently, these beam-column joints would have no, or very few, joint stirrups.

As demonstrated in laboratory testing (Hakuto et al., 2000; Pampanin et al., 2002-2003) and post-earthquake observations, different types of damage or failure modes are expected to occur in beam-column joints depending on the:

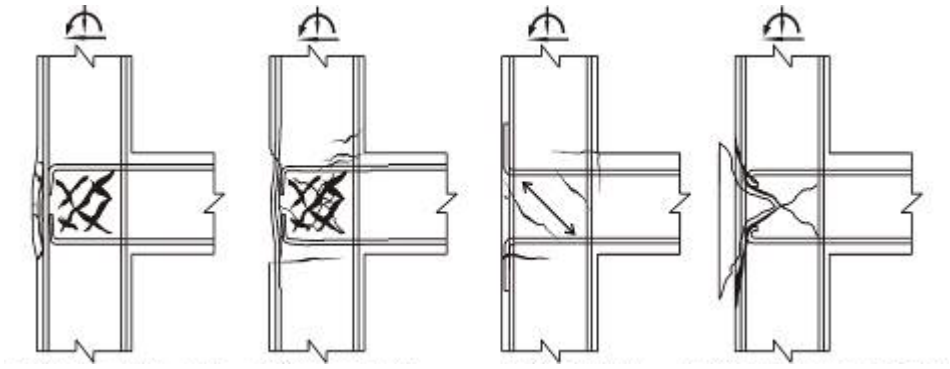
- typology (i.e. exterior or interior joints, with or without transverse beams) and
- structural details; i.e.:
  - lack or insufficient transverse reinforcement in the joint
  - type of reinforcement, i.e. plain round or deformed
  - alternative bar anchorage solutions; i.e. bent in, bent out, end-hooked, or a combination of these.

Figure C5.4 illustrates possible damage mechanisms of exterior tee-joints with no or minimal transverse reinforcement in the joint regions and alternative beam anchorage details.

Alternative damage mechanisms for exterior tee-joints are shown in Table C5.4:

- beam bars bent inside the joint region – (a) and (b)
- beam bars bent outside the joint region – (c), and
- plain round beam bars with end-hooks: “concrete wedge” mechanism – (d).

All of these solutions have been used in New Zealand.



(a) Beam bars bent in – cover cracking at back to joint

(b) Beam bars bent in – loss of joint integrity

(c) Beam bars bent away from the joint

(d) Plain round beam bars with end-hooks: concrete wedge mechanism

Figure C5.4: Alternative damage mechanisms expected in exterior joints depending on the structural detailing: (a) and (b) beam bars bent inside the joint region; (c) beam bars bent outside the joint region; (d) plain round beam bars with end-hooks

Note:

Referring to the basic strut-and-tie theory for beam-column joints (Park and Paulay, 1975; Paulay and Priestley, 1995), it is expected that exterior joints of older construction practice (i.e. with poor or no transverse reinforcement in the joints and poor anchorage detailing of the beam bars) are usually more vulnerable than interior beam-column joints.

After diagonal cracking, the shear transfer mechanism in a joint with no or very limited shear reinforcement must essentially rely on a compression diagonal strut. This mechanism can be maintained up to a certain level of compression stress in an interior beam-column joint. However, when dealing with exterior beam-column joints the strut efficiency is critically related to the anchorage solution adopted for the longitudinal beam reinforcement.

When the beam bars are bent into the joint (refer to Figure C5.4(a) and (b)) they can provide a limited resistance against the horizontal expansion of the joint. This is until the hook opens under the combined action of the diagonal strut and the pulling tension force in the beam reinforcement, which then leads to a rapid joint degradation. When the beam bars are bent away from the joint (refer to Figure C5.4(c)), as is more typical of older construction practice in New Zealand, no effective node point is provided for the development of an efficient compression strut mechanism unless a significant amount of transverse column hoops is placed immediately above the joint core. In this case, rapid joint strength degradation after joint diagonal cracking is expected.

Arguably, the worst scenario is provided by the solution shown in Figure C5.4(d), which is more common in pre-1970s buildings and consists of plain round bars with end-hook anchorage. The combination of an inefficient diagonal strut action and a concentrated compression force (punching action) at the end-hook anchorage due to slippage of the longitudinal beam bars can lead to the expulsion of a ‘concrete wedge’ and rapid loss of vertical load capacity.

C5.3.5 Damage observations following the Canterbury earthquakes

Tables C5.1 (pre mid-1970s RC buildings) and C5.2 (post mid-1970s RC buildings) provide a pictorial overview of the main structural deficiencies and observed damage of reinforced concrete buildings following the Canterbury earthquake sequence of 2010-2011.

For a more detailed overview of the seismic performance of RC buildings following the 4 September 2010 (Darfield Earthquake) and the 22 February 2011 (Lyttleton earthquake) events, refer to the NZSEE, 2010, 2011 and EERI/NZSEE 2014 Special Issues dedicated to the Canterbury Earthquake sequence (e. g. Kam et al., 2010, 2011; Fleischman et al., 2014; Sritharan et al., 2014; and Bech et al., 2014).

Note:

As the mid-1970s threshold cannot be taken as a rule to define earthquake risk buildings or earthquake-prone buildings, it can be also argued that post mid-1970 concrete buildings are not expected to suddenly have superior seismic performance. In fact, research carried out under the FRST-funded 'Retrofit Solutions' project in New Zealand has confirmed that typical weaknesses of pre-1970s buildings were consistently adopted for several years afterwards (Pampanin et al., 2006-2010; Ingham et al., 2006).

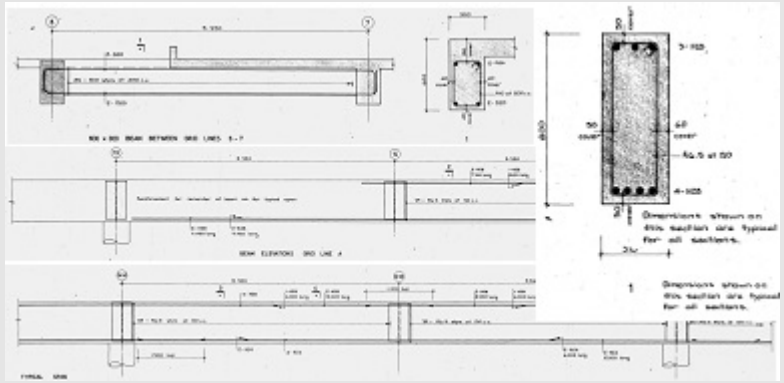
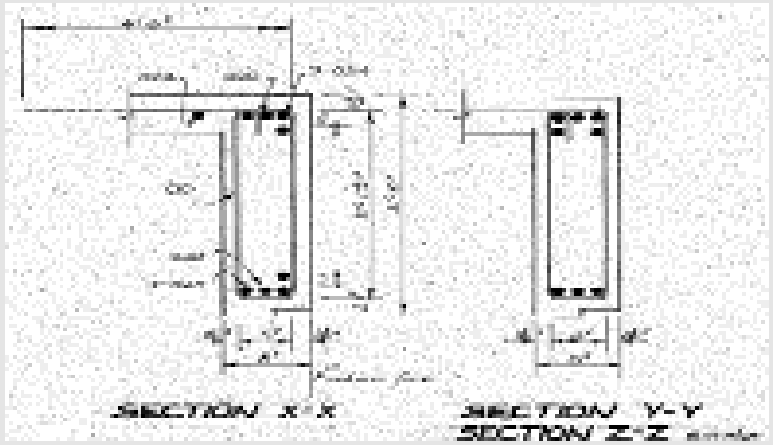
For example, the issue of potentially inadequate transverse reinforcement observed in columns constructed since the 1960s was not completely addressed with the provisions of NZS 3101:1982. Accordingly, many buildings designed and constructed prior to the 1995 standards were introduced can be expected to have inadequate levels of confinement in their columns (a potential SW) when compared to current standards. When confinement is low, loss of cover concrete combined with buckling of the longitudinal bars could occur, particularly in the lap-spliced regions, leading to unexpected failure.

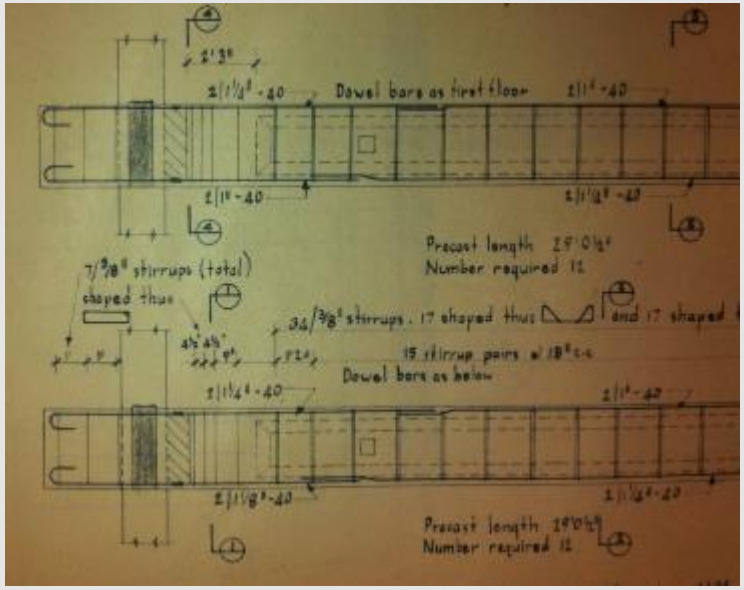
Moreover, recent focus on displacement incompatibility issues between lateral load resisting systems (i.e. walls or floors) and floor systems has shown potential SWs. Inadequate structural details could favour local damage and failure mechanisms due to beam elongation and vertical displacement incompatibilities (refer to Section C5.5.4).

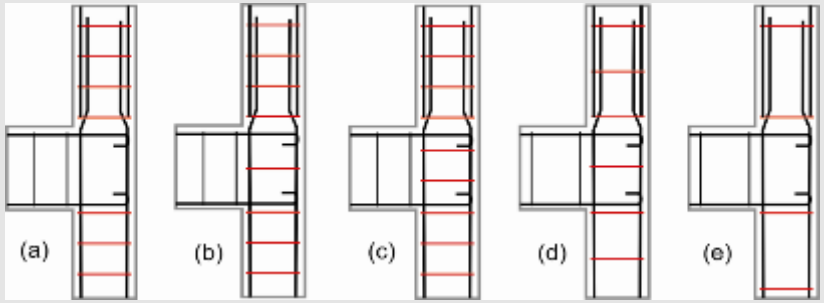
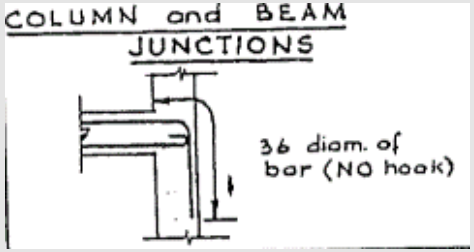
Irregularities in plan and elevation leading to torsionally-prone response, concentrated failure mechanisms, and/or ratcheting response have also been found as recurrent issues in post mid-1970 buildings.

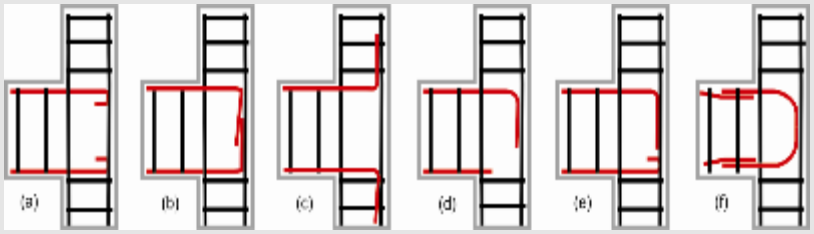
Notwithstanding these comments, modern design philosophies were also being incorporated in buildings from the late 1960s as discussed in Section C5.2.3.

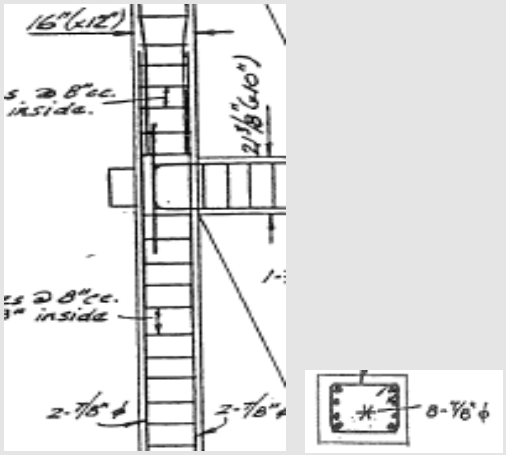
Table C5.1: Typical/expected structural deficiencies and observed damage/failure mechanism in pre mid-1970s Canterbury RC buildings

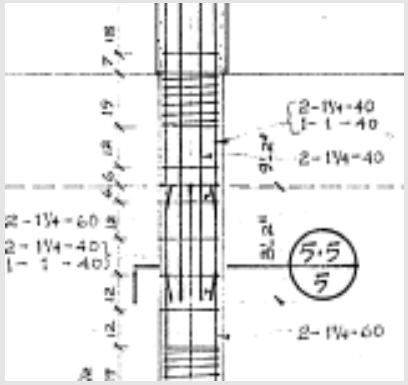
Component or global structure	Typical deficiency	Observed damage
Beams	Poor confinement details and transverse reinforcement in beams  Structural drawings of beam reinforcement and confinement details. Often the stirrups were 'opened' with a 90 degree angle instead of the more modern 135 degrees.  Structural drawings of beam reinforcement and confinement details	Flexural plastic hinge in beams, often characterised by single crack opening (refer to photo below) - especially when plain round bars adopted. This would lead to higher deformability (fixed end rotation), lower moment capacity at a given drift demand and possibly excessive strain demand in the reinfrocing steel bars. Also due to the poor confinement and transverse reinforcement details, higher level of demand could lead to premature compression-shear damage and failure in the plastic hinge region.

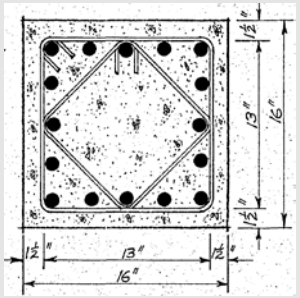
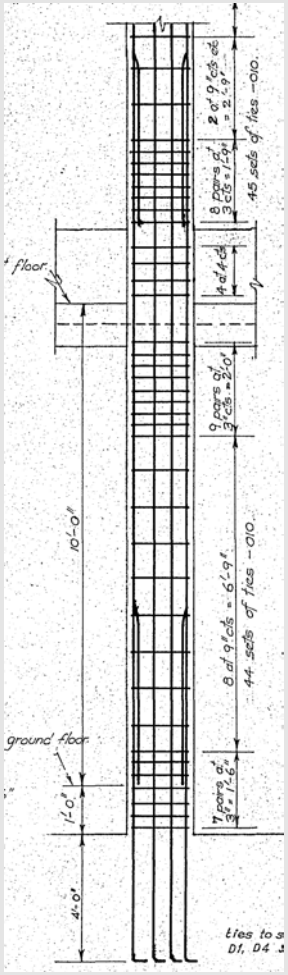
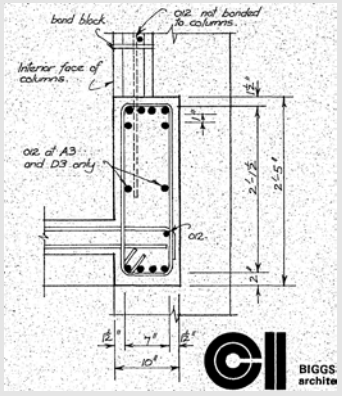
Component or global structure	Typical deficiency	Observed damage
	Inadequate anchorage of beam bars into the joint	(Refer to Joint section)
	Inadequate splice detailing (short development length, L_d, well below 40 diameters  Photo: Splices: 21” lap for D32 L_d =16 diameters); shear: 3/8” (R10) stirrups @ 18” centres (457 mm)	 Photo: Observed lap-splice failure in beams due to limited splice length Lapping was probably done at expected point of contraflexure due to gravity loading, without considering seismic effects.
	Use of plain round (smooth) bars	Development of single crack instead of a wider plastic hinge region. Concentration of strain and stresses in the reinforcing bars with possible premature failure in tension. Bond degradation and slip with reduced flexural capacity and energy dissipation (pinched hysteresis loop).

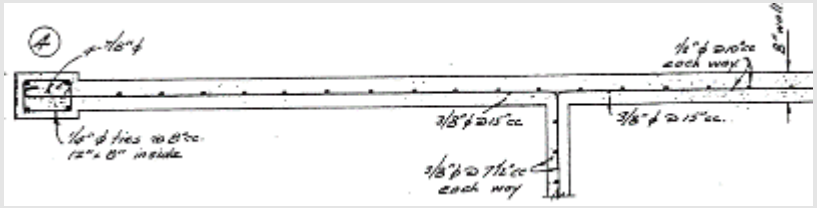
Component or global structure	Typical deficiency	Observed damage
Beam-column joints	Lack or total absence of horizontal and/or vertical transverse reinforcement in the joint panel zone.  Figure: Schematic illustrations of joint traverse reinforcement in pre-1970s buildings related to column stirrups and design assumptions: (a)-(b) Joint neglected in design or considered as a construction joint (c)-(d)-(e) Joints treated as part of column, therefore quantity of joint stirrups depended on column stirrup spacing and beam depth	Shear damage/failure in joint area with potential loss of gravity load bearing capacity in column  Figure and Photo: Structural drawing of joint reinforcing details and observed shear failure of exterior joints. (It is worth noting that the failure in this case was due to a combination of lateral loading and vertical settlement due to failure of a foundation beam.)

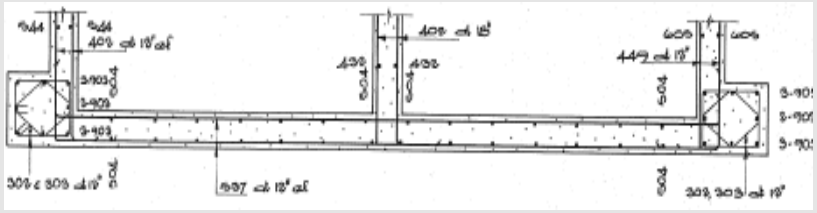
Component or global structure	Typical deficiency	Observed damage
	Inadequate anchorage of beam longitudinal bars into the joint Lack of reliable joint shear transfer mechanism beyond diagonal cracking  Figure: Alternative structural detailing of non-ductile beam-column joint: (a) 180° hooks (typical of plain round bars) (b) beam bars bent into the joint with 90° inward bends (c) beam bars bent out with 90° outwards bends (d) top beam bars bent in at 90°, bottom bars stop short with no anchorage hook or bend (e) top beam bars bent in at 90° bottom bars with hook anchorage (typically of plain round bars), and (f) U-shaped bar splice into the joint core.	

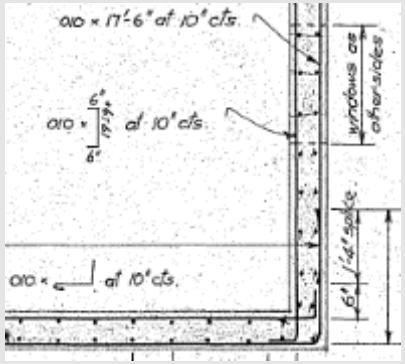
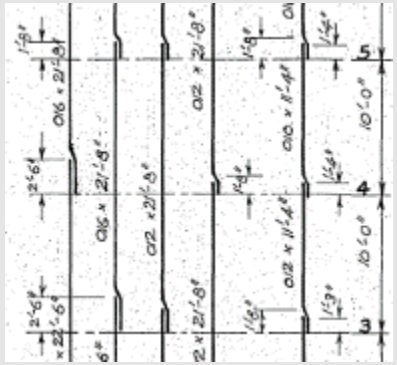
Component or global structure	Typical deficiency	Observed damage
Columns	Inadequate confinement detailing in the plastic hinge region: • not all of the bars of the longitudinal reinforcement are confined with stirrups • inadequate spacing for anti-buckling.  Figure: Structural drawings of column confinement details	Shear failure of the column at the plastic hinge Buckling of the longitudinal reinforcement at the plastic hinge Photo: Example of shear failure and bucking of column in plastic hinge region

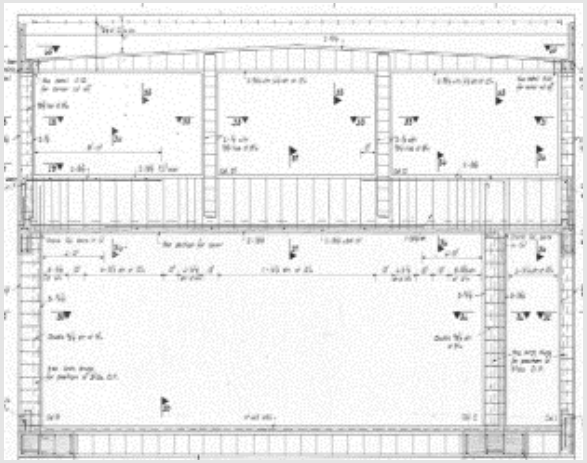
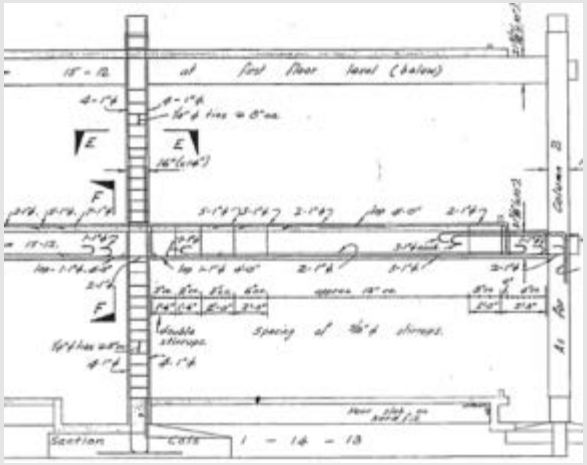
Component or global structure	Typical deficiency	Observed damage
	Inadequate lap-splice details Inadequate shear reinforcement  Figure: Structural drawing showing poor shear reinforcement details and lap splices	Potential for weak-column/strong-beam mechanism due to significant decrease in the flexural capacity of the plastic hinge Potential shear failure Photo: Shear failure of the columns due to short-column phenomenon

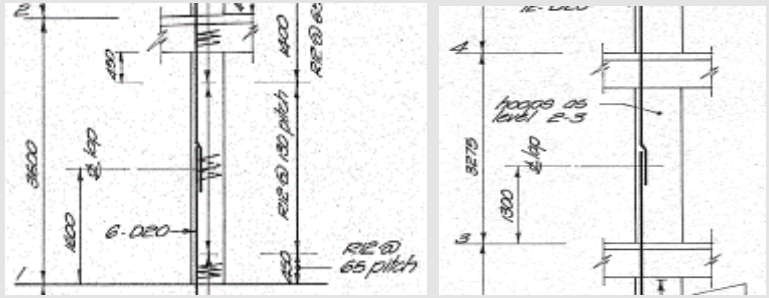
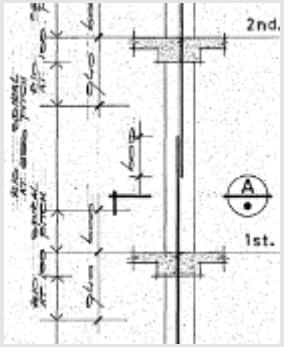
Component or global structure	Typical deficiency	Observed damage
	Short (captive) columns effects – effective shortening of the clear shear span of the columns due to presence of masonry or concrete infills, heavy spandrel beams or stiff non-structural facades   	Shear failure of columns Photo: Short column effect and shear failure due to presence of masonry infills Photo: Short column effect due to presence of spandrel elements (bottom)

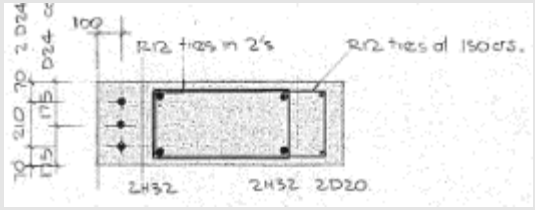
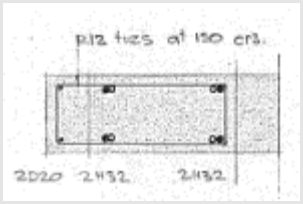
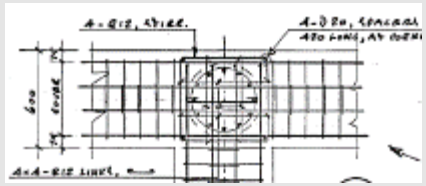
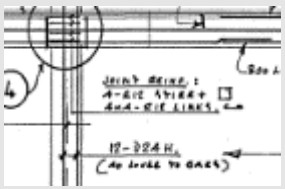
Component or global structure	Typical deficiency	Observed damage
Walls	Inadequate longitudinal reinforcement ratio  Figure: Structural drawing of a thin and singly reinforced wall	Opening of single crack in the plastic hinge region, with concentration of strain demand and potential tensile failure of longitudinal bars Photo: Tensile failure of longitudinal rebars hidden behind a single and small (residual) crack

Component or global structure	Typical deficiency	Observed damage
	Inadequate confinement and shear reinforcement in walls  Figure: Structural drawing of confinement and shear reinforcement details in a wall	Crushing and buckling failure in the boundary regions Photo: Wall failure due to buckled longitudinal reinforcements Photo: Combination of buckling, single crack opening and shear sliding due to inadequate detailing

Component or global structure	Typical deficiency	Observed damage
		 Photo: Crushing of end connection in boundary regions
	Inadequate lap-splice detailing   Figure: Structural drawings of reinforcing details at lap-splices	
	Excessive wall slenderness ratio (wall height-to-thickness ratio)	Out-of-plane (lateral) instability Refer to example of associated observed damage in the following table (related to post mid-1970s walls)

Component or global structure	Typical deficiency	Observed damage
Global structure	Lack of capacity design: weak-column, strong beam mechanism, soft-storey prone   Figure: Structural drawings of weak-column, strong beam mechanisms	Severe damages to columns or joints, which can lead to global brittle failure mechanism Photos: Severe shear damage and failure in columns

Component or global structure	Typical deficiency	Observed damage
Columns	Lap-splicing with not sufficient length and confinement. More often away from the plastic hinge region.  Figure: Structural drawings showing inadequate lap-splicing  Figure: Structural drawings showing inadequate lap-splicing	Damage due to the compromised continuity of the element, loss of moment-capacity, potential soft-storey mechanism

Component or global structure	Typical deficiency	Observed damage
	Inadequate confinement at the plastic hinge region of columns with high axial load ratio   Figure: Structural drawings of column confinement details	Shear-axial failure of columns Photo: Compression-shear failure in columns
	Inadequate transverse reinforcement in circular columns to resist torsion   Figure: Structural drawings showing transverse reinforcement details in circular column	Torsional cracks Photo: Torsional cracking of column

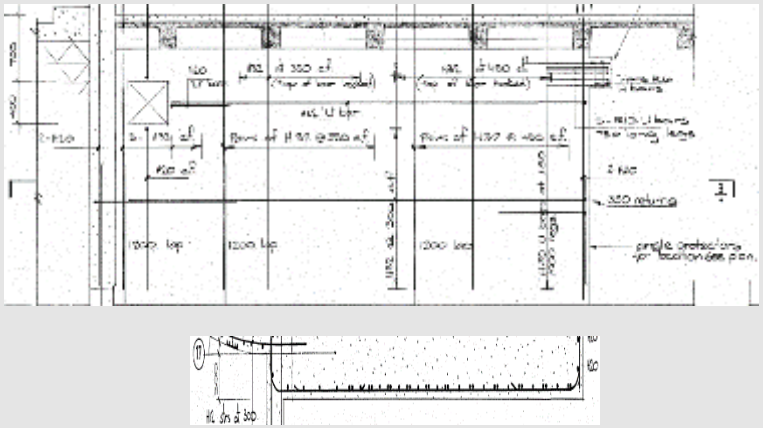
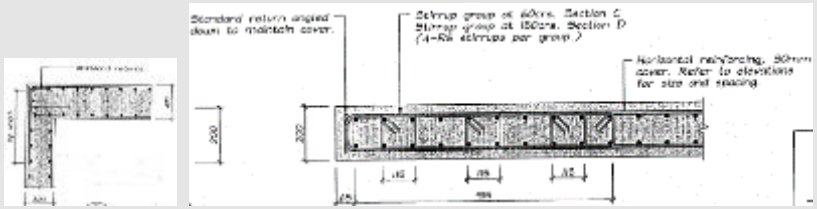
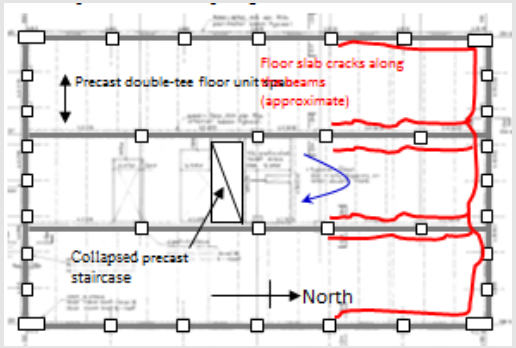
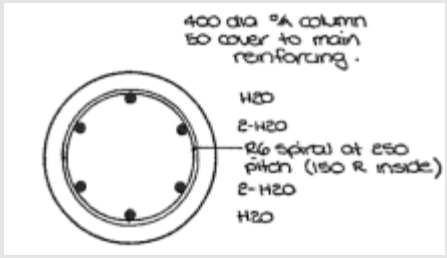
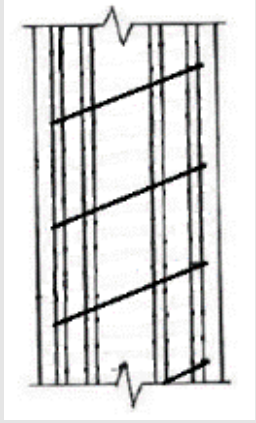
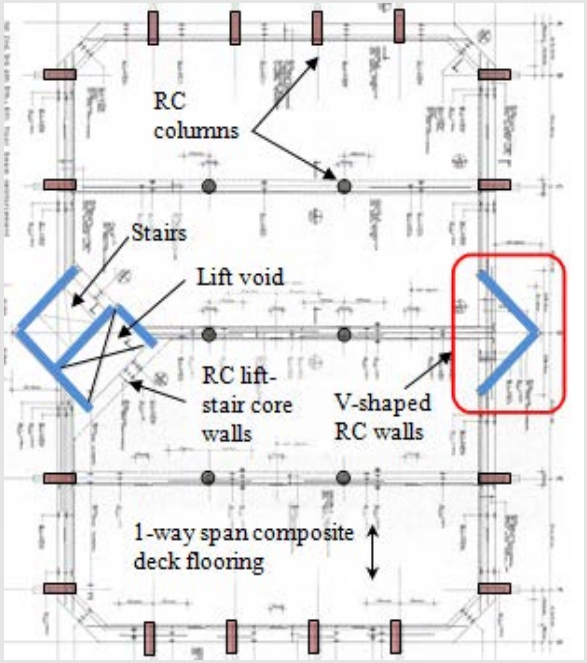
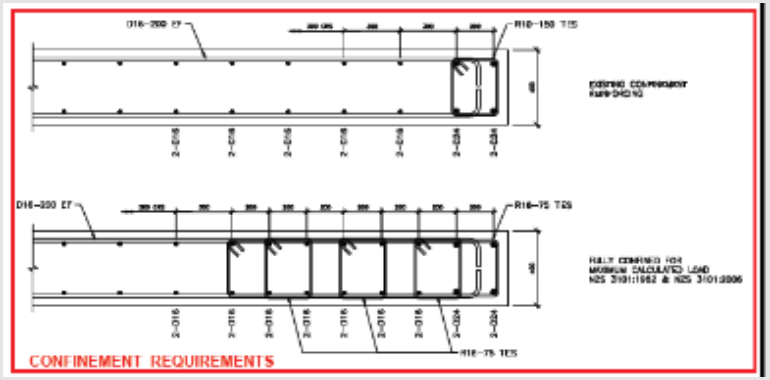
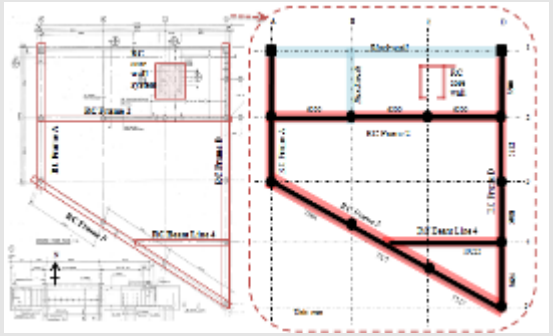
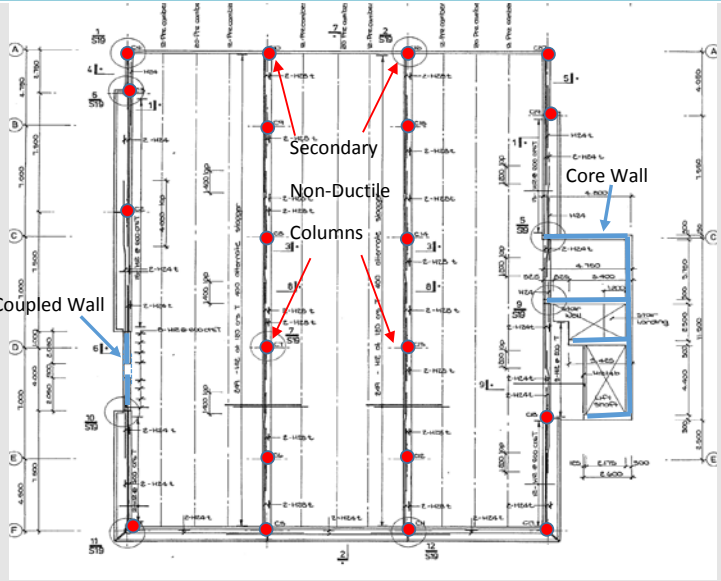
Component or global structure	Typical deficiency	Observed damage
Walls	Inadequate confinement in boundary elements as well as core area  Figure: Structural drawings of wall reinforcement and confinement details  Figure: Structural drawings of confinement details at wall corner and boundary element	Crushing, spalling of concrete; bar buckling; out-of-plane failure Photos: Spalling of concrete at wall end, and buckling failure Photos: Shear failure at ground floor wall

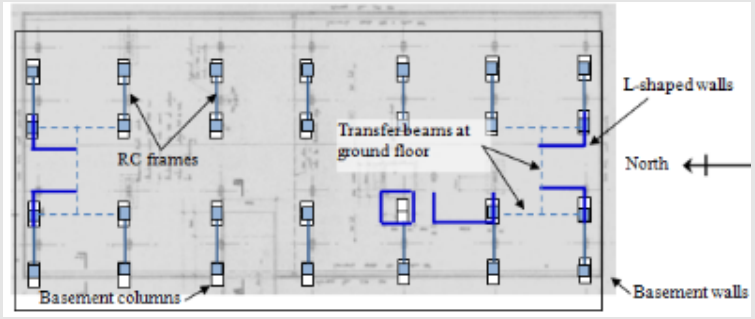
Table C5.2: Typical/expected structural deficiencies and observed damage/failure mechanism in post mid-1970s Canterbury RC buildings

Component or global structure	Typical deficiency	Observed damage
Floor/diaphragm	Beam elongation effects and lack of seating in precast floor diaphragms 	Tearing/damage to diaphragm and potential loss of seating Photos: Damage in the diaphragm due to beam elongation; potential unseating of floor units.
Non-ductile columns	Inadequate structural detailing to provide required ductility Inadequate confinement and shear reinforcement, poor lap splices, excessive cover concrete   Photo: Example of details of pre-1995 non-ductile (secondary) columns. Large cover concrete, inadequate stirrups spacing.	Lack of capacity to sustain the imposed displacement-drift compatibly with the 3D response of the system Loss of gravity load bearing capacity at earlier level of inter-storey drift Potential catastrophic collapse of the whole building Photos: Shear failure of pre-1995 non-ductile column details

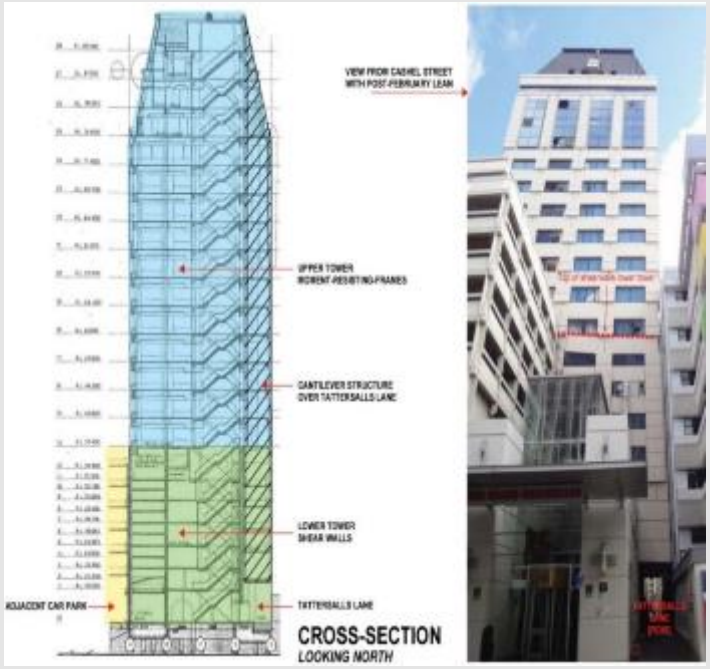
Component or global structure	Typical deficiency	Observed damage
Walls	 Flanged or irregular shaped walls Figure: Quasi-symmetric configuration of flanged-walls, yet leading to asymmetric response and inelastic torsion	Local lateral instability and concentration of damage in compression region Photos: Crushing of well confined boundary regions and lateral instability

Component or global structure	Typical deficiency	Observed damage
	Under-designed boundary region, lack of ties in the web, inadequate design against bidirectional loading, including out-of-plane shear  Figure:Example of actual details (top) of a 1980s shear wall and equivalent redesign according to latest NZS 3101:2006 design	 Photo: Out-of-plane shear-buckling failure of shear wall
Global structure	Plan irregularity  Figure: Irregular plan	Damage due to torsional effect to components Photos: Torsional cracks on columns

Component or global structure	Typical deficiency	Observed damage
	 Figure: Plan irregularity	Photo: Complete progressive collapse of the building as a result of a combination of a number of structural deficiencies including plan irregularity, non-ductile columns, weak diaphragm-to-lateral resisting system connection, etc.

Component or global structure	Typical deficiency	Observed damage
	Vertical irregularity  Figure: Schematic plan of an 11-storey building with plan and vertical irregularity	 Photos: Vertical irregularity resulting in: (a) Severe basement columns shear-axial failure; (b) Transfer beam damage and repair; (c) and (d) Ground floor transfer slab and basement wall damage

Component or global structure	Typical deficiency	Observed damage
Global structure	Vertical irregularity and set backs Photo: Vertical irregularity: set back	 Photo: Axial compression failure of ground floor column at the boundary of the setback. Transverse reinforcement: R6 spirals @ ~300-400 mm Photo: Captive column failure at building set-back level

Component or global structure	Typical deficiency	Observed damage
Global structure	Vertical irregularity and set-backs  Photo: Multi-storey building built mid-1980s with vertical irregularity due to first floor set-back and number of floors hanged on a transverse beam.	Asymmetric behaviour leading to ratcheting response, concentration of damage in gravity load-bearing elements; e.g. base wall at the boundary with the set-back and columns under transfer beam Photo: Axial-shear failure of columns under transverse beam due to ratcheting response

C5.4 Material Properties and Testing

C5.4.1 General

For reinforced concrete structures, key material-related data for the assessment include:

- concrete strength (its probable strain capacity being indirectly derived/assumed)
- steel yield strength, probable tensile strength, probable strain capacity and the expected variation in its properties.

Information on the mechanical properties of concrete and steel reinforcing can be sourced from:

- the construction drawings, and/or
- the original design specifications, and/or
- original test reports, and/or
- knowledge of the practices of the time, and/or
- site observations of quality, and/or
- in-situ testing.

In the absence of specific information, default values for the mechanical properties of the reinforcing steel and concrete may be assumed in accordance with the relevant standards and practices at the time of construction, after first making an assessment on general material quality (particularly in relation to the concrete work). The following sections provide the intended default values.

More details on the historical material properties specifications and design requirements in New Zealand can be found in the appendices.

Note:

The extent of any in-situ testing must be based on a careful assessment of the tangible benefits that will be obtained. It will never be practical to test all materials in all locations. In-situ testing may be justifiable in situations where the critical mechanism is highly reliant on material strengths, or perhaps relative material strengths (e.g. steel grade in interconnected beams and columns) but only when judgement based on an assumed range of possible material strengths cannot indicate an appropriate outcome. “Spot” testing to ascertain the material types in generic locations might be appropriate but it is not intended that it be necessary to determine the range of properties present for a particular material.

Appendix C5D provides destructive and non-destructive techniques for gathering further information on concrete and reinforcing steel material properties if this is considered necessary.

Note:

Use of probable and overstrength member and element capacities as outlined in these guidelines is considered to provide the required level of confidence that a mechanism will be able to develop with the required hierarchy if the material strengths can be reasonably

ascertained. This means tit is not intended that the engineer applies any additional factors to account for natural variation in material strengths when assessing the hierarchy within a particular mechanism.

C5.4.2 Concrete

C5.4.2.1 General

Regardless of the information provided on the drawings, the actual properties of concrete used in the building might vary significantly. This can be due to factors such as:

- construction practice at the time the building was constructed; e.g. poor placement and compaction, addition of water for workability
- the fact that the concrete may have been subject to less stringent quality control tests on site, and
- concrete aging.

Appendix C5A summarises the evolution of concrete property requirements and design specifications in New Zealand. Appendix C5B summarises the tests used for quality control of concrete as contained in the New Zealand standard for specification for concrete production, NZS 3104, from 1983 to the present day.

Notwithstanding the potential inherent variability in concrete properties, which will be impossible to determine precisely (even with extensive investigation), it is intended that a seismic assessment is based on reasonably established generic concrete properties.

C5.4.2.2 Probable compressive strength of concrete

In the absence of specific information, the probable compressive strength of concrete, $f'_{c,prob}$, may be taken as the nominal 28 day compressive strength of the concrete specified for the original construction, f'_c, factored by 1.5 for strengths less than or equal to 40 MPa and 1.4 for strengths greater than 40 MPa.

Table C5.3 presents suggested default values for the probable compression strength of concrete when the actual specified values cannot be ascertained. These are based on typical 28 day compressive strengths specified over different time periods. If inspection indicates poor compaction, these default values may need to be reduced for column strength calculations.

Table C5.3. Default probable concrete compressive strengths

Period	Generic assumed 28 day compressive strength (MPa) f'_c	Default probable compressive strength (MPa) $f'_{c,prob}$
1970-1981	20	30
1982-1994	25	40
1995-2005	30	45
2006-present	30	45

Note:

The actual compressive strength of old concrete is likely to exceed the specified value as a result of conservative mix design, the aging effect, and the coarser cement particles that were used. Furthermore, probable strength values should be used for assessment instead of the fifth percentile values (or lower bound of compression strength) typically adopted for design.

There is a lack of experimental in-situ testing of New Zealand structures, and of buildings in particular, to allow the strength of aged concrete to be reliably determined.

As an indicative reference only, tests on the concrete of 30-year-old bridges in California consistently showed compressive strengths approximately twice the specified strength (Priestley, 1995). Concrete from the columns of the Thorndon overbridge in Wellington had a measured compressive strength of about 2.3 times the specified value of 27.5 MPa about 30 years after construction (Park, 1996).

Similarly, concrete from collapsed columns of the elevated Hanshin Expressway in Kobe, Japan after the January 1995 earthquake had a measured compressive strength of about 1.8 times the specified value of 27.5 MPa almost 30 years after construction (Park, 1996; Presland, 1999).

Eurocode 2 Part 1, 2004 provides an expression to evaluate the aging factor as a function of the strength class of cement adopted. The aging factor tends almost asymptotically after 10-20 years to values in the range of 1.2-1.4 depending on the cement strength class.

This limited evidence, at least, would suggest that the use of factors of between 1.4 and 1.5 depending on the originally specified concrete strength (lower bound – fifth percentile) to obtain the probable current concrete strength is a reasonable approach. Generally accepted relationships for concrete strength gain with age indicate that enhanced strength can be expected for structures of relatively young age (beyond a year), so distinguishing for age is not considered necessary.

Recourse to default generic values is considered a reasonable approach when considered against the extent (and cost) of in-situ testing required to generate an appropriate statistical sample with no certainty of identifying areas of under-strength concrete.

C5.4.3 Reinforcing steel

C5.4.3.1 General

The mechanical properties of reinforcing steel will vary depending on the source, targeted grade and age.

The historical overview provided in Appendix C5C should provide a useful basis for selecting the expected mechanical characteristics of reinforcing steel if more specific information is not available from the building's structural and construction drawings. Whenever practicable, samples of steel from the structure should be tested to at least provide an indication of the likely grade of reinforcement that is present.

C5.4.3.2 Probable yield strength of reinforcing steel

The probable yield strength of the reinforcing steel may be taken as the mean of the upper characteristic (95^{th} percentile value) and the lower characteristic (5^{th} percentile value) yield strength.

Where the lower and upper yield strength bounds are not known, the probable yield strength of the reinforcing steel may be taken as 1.08 times the lower characteristic yield strength value.

Note:

The ratio between the upper and lower characteristic yield strengths will typically be in the range of 1.17 to 1.3 depending on source and age. The 1.08 factor is based on the lower end of this expected range.

Refer to Section C5C.1 for indicative values of the lower and upper bounds of the yield stress for specified post-1970 reinforcing steels.

Chapman (1991) reports that site sampling and testing has found the structural grade reinforcement in New Zealand structures built during the 1930s to 1970s is likely to possess a lower characteristic yield strength (5^{th} percentile value) that is 15-20% greater than the specified values. Therefore, for pre-1970 reinforcing steels the probable yield strength may be taken as 1.08 x 1.15 = 1.25 times the specified yield stress values indicated in Appendix C5C.

C5.4.3.3 Probable modulus of elasticity of reinforcing steel

The probable modulus of elasticity of reinforcing steel may be taken as 200,000 MPa.

C5.4.3.4 Probable tensile capacity of reinforcing steel

The probable tensile capacity of reinforcing steel may be taken as the tensile strength given in Tables C5C.1, C5C.2 and C5C.3 appropriate for the expected age and grade of steel.

C5.4.3.5 Probable strain at tensile capacity

The probable strain in reinforcing steel at the probable tensile capacity may be taken as 0.1.

C5.4.3.6 Bar size

Typical bar sizes available before and after the mid-1970s are shown in Tables C5.4 and C5.5.

Table C5.4: Available diameters of steel reinforcement bars – before the mid-1970s

NZS 1693:1962		NZS 1879:1964		NZS 3423P:1972	
Bar designation	**d inch (mm)**	**Bar designation**	**d inch (mm)**	**Bar designation**	**d inch (mm)**
3	3/8 (9.525)	3	3/8 (9.525)		3/8 (9.525)
4	1/2 (12.7)	4	1/2 (12.7)		1/2 (12.7)
5	5/8 (15.875)	5	5/8 (15.875)		5/8 (15.875)
6	3/4 (19.05)	6	3/4 (19.05)		3/4 (19.05)
7	7/8 (22.225)	7	7/8 (22.225)		7/8 (22.225)
8	1.000 (25.4)	8	1.000 (25.4)		1.000 (25.4)
9	1 1/8 (28.575)	9	1 1/8 (28.575)		1 1/8 (28.575)
10	1 1/4 (31. 75)	10	1 1/4 (31. 75)		1 1/4 (31. 75)
11	1 3/8 (34.925)	11	1 3/8 (34.925)		1 3/8 (34.925)
12[1]	1 1/2*(38.1)	12[1]	1 1/2*(38.1)		1 1/2(38.1)
					2 (50.80)

Note:
1. Introduced in 1970

Table C5.5: Available diameters of steel reinforcement bars – from the mid-1970s onward

NZ 3402P:1973 (Stage 1)				NZ 3402P:1973 (Stage 2)			NZS 3402:1989			AS/NZS 4671:2001		
Bar designation		**d (inch)**	**d (mm)**	**Bar designation**		**d (mm)**	**Bar designation**		**d (mm)**	**Bar Designation**		**d (mm)**
R10	D10	-	10	R10	D10	10	R6	D6	6	R6	D6	6
R13	D13	½	12.7	R12	D12	12	R8	D8	8	R10	D10	10
R16	D16	-	16	R16	D16	16	R10	D10	10	R12	D12	12
R20	D20	-	20	R20	D20	20	R12	D12	12	R16	D16	16
R22	D22	7/8	22.23	R24	D24	24	R16	D16	16	R20	D20	20
R25	D25	-	25.4	R28	D28	28	R20	D20	20	R25	D25	25
R28	D28	-	28	R32	D32	32	R24	D24	24	R32	D32	32
R32	D32	-	32	R40	D40	40	R28	D28	28	R40	D40	40
R38	D38	1 ½	38.1				R32	D32	32			
							R40	D40	40			

C5.4.4 Cold-drawn welded wire mesh

The properties for cold drawn wire mesh may be taken from Appendix C5C. The maximum available strain at maximum stress should be taken as 1.5%.

C5.5 Element Probable Capacities

C5.5.1 General approach

This section sets out the procedures for evaluating the probable strength and deformation capacities of beams, columns, beam-column joints, walls and diaphragms.

The general approach taken to determine RC probable member/element capacities is to evaluate:

- the probable flexural strength and deformation (curvature) capacity relationships for RC sections/regions and where necessary extend the relationships to rotations and inter-storey drifts
- the probable shear strength and
- the limiting effect, if any, of shear in flexural regions subjected to nonlinear deformations, reinforcing steel laps, buckling of vertical reinforcement in columns and walls, out-of-plane stability in walls, and sliding shear.

These are discussed in the sections below.

It is considered acceptable to determine probable strength capacity as the nominal strength determined from NZS 3101 using the probable material properties obtained from Section C5.4.

Note:

Member/element capacities will be dependent, in many situations, on the actions in the member/element (e.g. axial loads in columns and walls and shear in regions subjected to nonlinear deformations). Therefore, an iterative approach is likely to be employed whereby some analysis is undertaken in parallel with assessing the capacities to gain an appreciation of the likely range of actions. In this way the quantum of work required to evaluate capacities can be kept to a minimum, with a focus on only those members/elements that are likely to limit the capacity of the subsystems and systems within the building.

The probable capacity of a member/element calculated simply from consideration of section capacities may be significantly overstated if issues such as deterioration of reinforcing steel laps (particularly for round bars), buckling of poorly restrained longitudinal reinforcing steel (axially loaded members), lateral stability (thin walls) and deterioration of shear capacity in nonlinear regions are not taken into account. Guidance on how to allow for these issues is provided below.

Where specific requirements are not covered in these guidelines the probable strength capacities may be taken as the nominal capacities from NZS 3101 (i.e. ϕ =1) determined using probable material strengths. Such an approach is likely to be conservative compared with the requirements outlined below and therefore may be used in lieu of those requirements.

C5.5.1.1 Key terms

The following key terms are used in the derivation of probable element capacities outlined in the following sections.

Nominal capacity

For reinforced concrete the nominal strength capacity, S_{n}, is the theoretical strength of a member section based on established theory, calculated using the section dimensions as detailed and the lower characteristic reinforcement yield strengths (fifth percentile values) and the specified (nominal) compressive strength of the concrete.

The nominal strength capacity gives a lower bound to the strength of the section and is the value typically used for design.

Similarly, for design, the nominal deformation capacity is determined in accordance with the concrete design standard NZS 3101:2006.

For assessment, the probable values as defined below should be used.

Probable capacity

The probable strength capacity, S_{prob}, which is also referred to as expected strength capacity, is the theoretical strength of a member section based on established theory, calculated using the section dimensions as detailed and the probable (mean values) material strengths and a strength reduction factor as noted below.

The probable or expected deformation capacity is determined as indicated in the following sections.

Overstrength

The overstrength capacity takes into account factors that may contribute to an increase in strength, such as: higher than specified strengths of the steel and concrete, steel strain hardening, confinement of concrete, and additional reinforcement placed for construction and otherwise unaccounted for in calculations.

For beams, the overstrength in flexure, when tension failure is controlling the behaviour, is mainly due to the steel properties along with the slab flange effect and possibly the increase in axial load due to beam elongation. For current New Zealand manufactured reinforcing steel, an upper bound for the yield strength can be taken as the upper characteristic (95^{th} percentile value).

A further 8% increase in steel stress due to strain hardening should be assumed (e.g. refer to Andriono and Park, 1986).

Hence, as a first approximation – i.e. as a quick check before more comprehensive calculations – and indicatively, the ratio of overstrength in flexure to:

- nominal flexural strength, $M_{\mathrm{o}}/M_{\mathrm{n}}$, can be taken as 1.25 (for both Grade 300 and Grade 430 steel) and 1.35 for Grade 500
- probable flexural strength, $M_{\mathrm{o}}/M_{\mathrm{prob}}$, can be taken as 1.16.

Note:

While adequate confinement can cause an increase in the concrete compressive strain and ultimate deformation capacity for columns, the effect on the increase in flexural strength is limited. For poorly detailed and confined columns this enhancement in flexural strength is further limited, such that neglecting it would be on the conservative side. The actual overstrength of the concrete section can be established using a moment curvature analysis, stress/strain assumptions for material strengths as noted later in this Section, and the range of expected axial loads.

Strength reduction factors

For the purposes of calculating the probable strength capacity, no strength reduction factor ϕ should be used for either flexure or shear (i.e. ϕ =1.0). Where considered necessary, a factor to provide a safety margin against shear failure has been included in the derivation of the shear capacity equations.

Bounds of flexural strength

The lower and upper bounds of flexural strength can be important when assessing hierarchy of strength mechanisms for post-elastic deformation (e.g. moment resisting frames). The lower bound of flexural strength can be taken as the probable strength, and the upper bound as the overstrength.

When the hierarchy of strength mechanisms is critical to the assessment result or relied on to limit actions, the overstrength should be taken as the full overstrength at ϕ_{cap}, irrespective of the maximum curvature calculated under XXX%ULS shaking.

Note:

For lateral sway mechanisms (e.g. frame action) reliant on a hierarchy of strength it is important to also account for the variation in strength due to resulting axial loads and/or due to displacement incompatibility issues (e.g. vertical restraint from floor during lifting up of wall or horizontal restraint to beams due to beam elongation effects).

The full overstrength should be used in assessing strength hierarchies to reflect the underlying philosophy of these guidelines that shaking is not limited to XXX%ULS shaking, and also to maintain relativity with design.

C5.5.2 Beams, columns and walls

C5.5.2.1 Flexural (moment) capacity

The probable flexural capacity at a member/element section for a beam, column or wall is represented by the generalised relationship shown in Figure C5.5. For column and wall sections the relationship shown is for a particular axial load and for all members is shown unlimited by flexural-shear action.

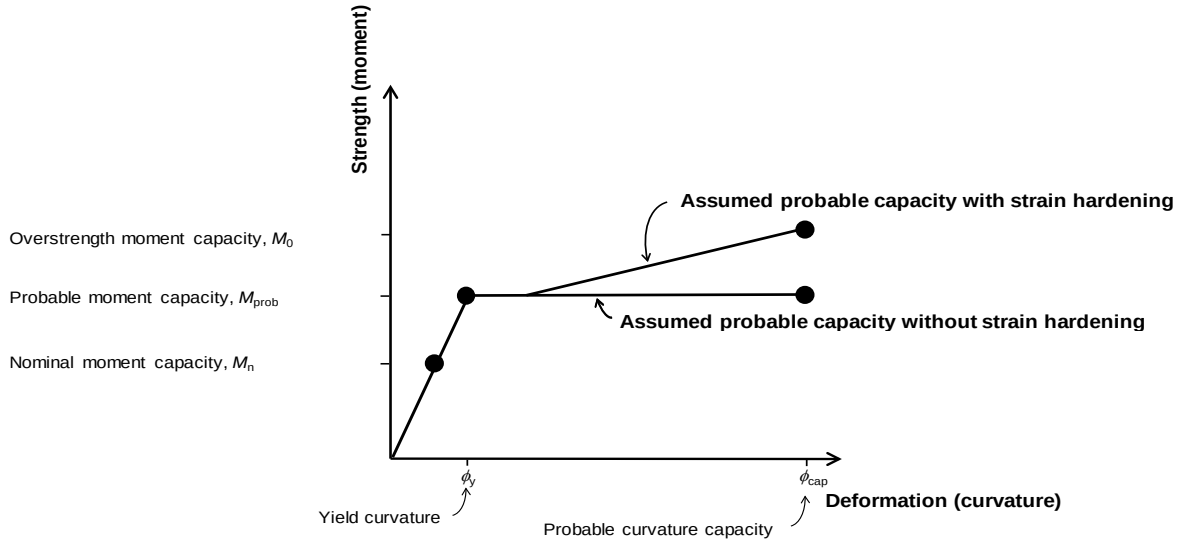


Figure C5.5: Generalised flexural strength - deformation relationship for reinforced concrete sections

The parameters that need to be determined are:

- probable flexural (bending moment) strength
- probable yield curvature
- probable curvature capacity
- overstrength capacity (when required).

The derivation of these is covered in the following sections.

Probable flexural (moment) strength

General

The probable flexural strength of member sections should be calculated using the probable material strengths determined in accordance with Section C5.4 and the standard theories for flexural strength of RC sections (Park and Paulay, 1975).

Note:

It is worth recalling that the basic theory for RC section flexural strength relies upon key assumptions such as:

- plane section remain plane (Hooke 1678, also known as Bernoulli-Navier theory), and
- fully bonded conditions between steel and concrete (i.e. no or negligible bond slip).

While these assumptions are generally valid for modern and relatively well designed members, issues can arise when dealing with older construction detailing; in particular, inadequate anchorage/development length and/or use of plain round bars.

In these cases, the flexural capacity as well as the probable curvature and ductility capacity of the beams and columns can be reduced. In turn, this can affect the hierarchy of strength within a beam-column joint connection/sub-assembly as discussed in subsequent sections.

The plastic hinges in beams normally occur at or near the beam ends in seismically dominated frames (whilst in gravity-dominated frames these could occur away from the column interface). Therefore, the longitudinal beam reinforcement is at or near the yield strength at the column faces.

This can result in high bond stresses along beam bars which pass through an interior joint core, since a beam bar can be close to yield in compression at one column face and at yield in tension at the other column face. During severe cyclic loading caused by earthquake actions, bond deterioration may occur in the joint. If the bond deterioration is significant, the bar tension will penetrate through the joint core, and the bar tensile force will be anchored in the beam on the far side of the joint.

This means that the compression steel will actually be in tension. As a result, the probable flexural strength and the probable curvature capacity of the beam will be reduced.

Hakuto et al. (1999) have analysed doubly reinforced beam sections at the face of columns of a typical building frame constructed in New Zealand in the late 1950s. The effect of stress levels in the "compression" reinforcement on the moment capacity of the beam was not found to be significant. When the bond had deteriorated to the extent that the "compression" reinforcement was at the yield strength in tension, the decrease in moment capacity was up to 10% for positive moment and up to 5% for negative moment compared with beams with perfect bond along the beam bars (Hakuto et al., 1999).

Based on this evidence and in order to provide a simplified procedure, the effect of bar slip on flexural strength of beams can be neglected in the assessment.

Similarly, for the first approximation the reduced level of ductility demand can be calculated by ignoring the compression reinforcement (in case a tension failure mechanism is expected).

Note that the bond-slip could actually introduce additional sources of deformability, increasing the deformation demand in the structural system.

The flexural strength of columns within a beam-column joint is similarly affected due to bond-slip of the longitudinal vertical reinforcement.

The probable flexural strength of a wall should be determined based on the effective vertical reinforcement at the base and the gravity loads. The neutral axis depth to wall length ratio, c/l_{w}, which is derived as a by-product of this calculation, is used subsequently when checking the curvature ductility capacity of each wall section. A traditional section analysis can be carried out. This should take into account the distributed reinforcement and assume a linear strain profile based on "plane sections remaining plane" assumption and a full bond condition between the steel rebars and the concrete.

It has recently been shown that, depending on the structural detailing and key mechanical/geometrical parameters, an assumption of a linear strain profile might not be valid; particularly for post yield behaviour. Strain and stress concentrations (both tension and compression) can thus occur and develop not only along the section depth but also across the thickness, leading to more complex out-of-plane or localised failure mechanisms as outlined in Appendix C5H. More information can be found in Dashti et al., 2015.

In general terms, consideration of the upper and lower bounds of flexural strength of beams and columns is important when assessing the behaviour of moment resisting frames, for example, to determine the likely hierarchy of strength and global mechanism, and therefore whether plastic hinging can occur in the beams or columns or both.

The axial load demands due to gravity loads and seismic actions should be accounted for when assessing the flexural strength of columns and walls.

Note:

When the axial load demands in columns and walls vary, a range will need to be considered when assessing the flexural capacity of these elements. This could have particular relevance for the development of some mechanisms dependent on a strength hierarchy.

Slab and transverse beam contributions to negative flexural capacity of beams

When calculating the probable flexural capacity of beams in negative moment regions it is important to account for the potential "flange-effect" contribution from the slab reinforcement (refer to Figure C5.6). This is particularly important when cast-in-place floor slabs (which are integrally built with the beams) are used. However, it should not be underestimated when precast floors with topping and starter bars are used.

Experimental evidence has also revealed the influence of the transverse beam torsion resistance on the magnitude of the effective width due to flange effect, b_{eff}, in exterior beam-column joints of cast-in-place two-way frames (Durrani and Zerbe, 1987; Di Franco et al., 1995).

A higher-than expected strength of the beam could modify the hierarchy of strength in a beam-column joint, possibly resulting into an increased risk of a column-sway mechanism when compared to a more desirable beam-sway mechanism.

As a first approximation the slab can be assumed to provide a 50% increase in the beam negative probable moment capacity and corresponding overstrength capacity, as shown in the figure below. However, experimental research has shown that the presence of a slab and transverse beam can increase the negative flexural strength of a beam by up to 1.7 to 2 times (Durrani and Zerbe, 1987; Ehsani and Wight, 1985; Di Franco, Shin and La Fave, 2004).

Therefore, it is recommended that the overstrength capacity should be more properly evaluated in cases where the hierarchy of hinge formation within the mechanism is important to the assessment result.

In addition to increasing the flexural capacity, the slab reinforcement reduces the ultimate ductility of curvature of a beam section.

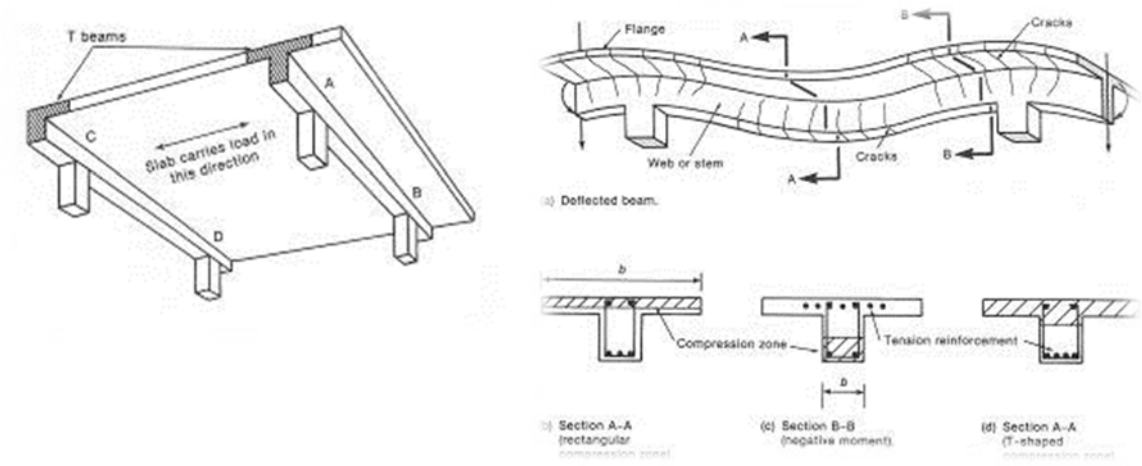


Figure C5.6: (a) Schematic monolithic one-way floor slab with beams (b) T-beam in double-bending (c) X-sections of T-beam showing different tension and compression zones (MacGregor, 1997)

Note:

The actual contributions of slab reinforcement to the negative moment flexural strength of a beam are dependent on: (1) the type of floor system, (2) the boundary conditions of the slab, (3) the level of imposed deformation on the beam-slab section, (4) the torsional resistance of transverse beams, if any, and (5) the quality of the anchorage of the reinforcing bars to develop full tensile strength.

In absence of further analysis, the recommendations provided by a new building standard (such as NZS 3101:2006) to evaluate the width of the slab contributing, with its reinforcement, to the flexural capacity under negative moments of T and L beams built integrally with the slab can be taken as a reference.

In poorly detailed beam-column joints where the joint and column are weaker than the beam-slab section, an effective width of the slab $b_{\text{eff}} = 2.2h_{\text{b}}$ (which includes the width of the beam) can be also used as a reference, based on the experimental research conducted by Kam et al., 2010.

To account for the torsional effects of a transverse beam, these guidelines recommend an effective width $b_{\text{eff}} = b_{\text{c}} + 2h_{\text{t}}$, where b_{c} is the width of the column and h_{t} is the height of the transverse beam or spandrel.

Flexural strength at lap splices

If the lap length is sufficient to develop yield (e.g. L_{ds} (approx. $20d_{\text{b}}$) for deformed bars and $2L_{\text{ds}}$ for plain bars) then the probable flexural strength capacity can be attained. For lesser lap lengths, exceedance of the capacity of the lap quickly degrades the bond strength and within one cyclic of loading the lap splice may be assumed to have failed.

When a lap splice in a beam fails in bond the contribution of those bars will need to be assumed to have been lost. However, bond failure in laps in columns and walls does not generally lead to a catastrophic failure, as the member is still able to transfer moment due to the presence of the eccentric compression stress block that arises as a result of the axial load in the column. However, the hierarchy of strength at that floor level can change to the extent that the mechanism may also change from a weak-beam to a weak-column mechanism, potentially leading to a soft-storey.

On the other hand, premature lap-splice failure can protect against failure of a more brittle mechanism. Therefore, it is recommended to use full flexural capacity (without reduction due to lap spice failure) when assessing shear behaviour.

The probable moment capacity of a lap splice, M_{lap} , may be determined by interpolation as follows:

$$M_{\text{lap}} = M_{\text{prob}}{}^{L_{\text{ds,prov}}}\Big/_{L_{\text{ds}}} \qquad \ldots\text{C5.1}$$

where:

$L_{\text{ds,prov}}$ = provided lap length
L_{ds} = the development length determined from NZS 3101.

The strength of lap splices in longitudinal reinforcement in plastic hinge regions will tend to degrade during imposed cyclic loading in the post-elastic range.

In general, a structural ductility factor of greater than 2 should not be assumed possible if lap splices in deformed longitudinal reinforcement exist in plastic hinge regions; unless these are heavily confined.

If plain round longitudinal bars are lapped the available structure ductility factor should be limited to 1.0 (Wallace, 1996).

Development length, anchorage details and lap splices can represent potential issues in buildings designed to earlier standards. In older frames, column lap-splice connections can often be found immediately above the floor level, where the potential location of moment reversal plastic hinges cannot be precluded.

Note:

At a first step, and on a conservative level, the plastic rotation demand on the column, θ_{p}, can be taken as the one calculated for a pure flexural failure mechanism.

Similarly, the axial load force on the column can be estimated assuming a beam sway mechanism which would lead to the highest variation of the axial load.

In terms of reference values for the development length, l_{d}, the NZS 3101:2006 recommendations for basic calculation for l_{d} in tension and compression can be adopted for deformed bars. For plain round bars it is recommended to take $l_{\text{d,req}}$ as twice the specification for l_{d} in NZS 3101:2006.

More detailed information on bond capacity and development length of plain round bars can be found in (Fabbrocino et al., 2002).

In older shear walls, lap splice often occurs within the plastic hinge regions and can develop for a significant length (e.g. one full storey or more) depending on the full wall height and section depth. The wall capacity should be checked not only at the base of the wall but also at the top of the lap splice. If necessary, an appropriate reduction in moment capacity should be accounted for.

Probable yield curvature

The probable yield curvature, ϕ_y, can be evaluated using a section analysis but may be taken as:

$$\phi_y = \frac{\varepsilon_y}{d - kd} \quad \text{…C5.2}$$

where:

ε_y = strain at the point of probable yield of the longitudinal tension reinforcement (= f_y/E_s)
d = effective depth of longitudinal tension reinforcement
kd = neutral axis depth when tension steel reaches the strain at first yield, ε_y.

In principle, and particularly for multiple layers of reinforcement in beams (and more commonly for columns), ϕ_y should be defined using a bilinear approximation (refer to Figure C5.7). The yield point so defined can be referred to as the equivalent yield point.

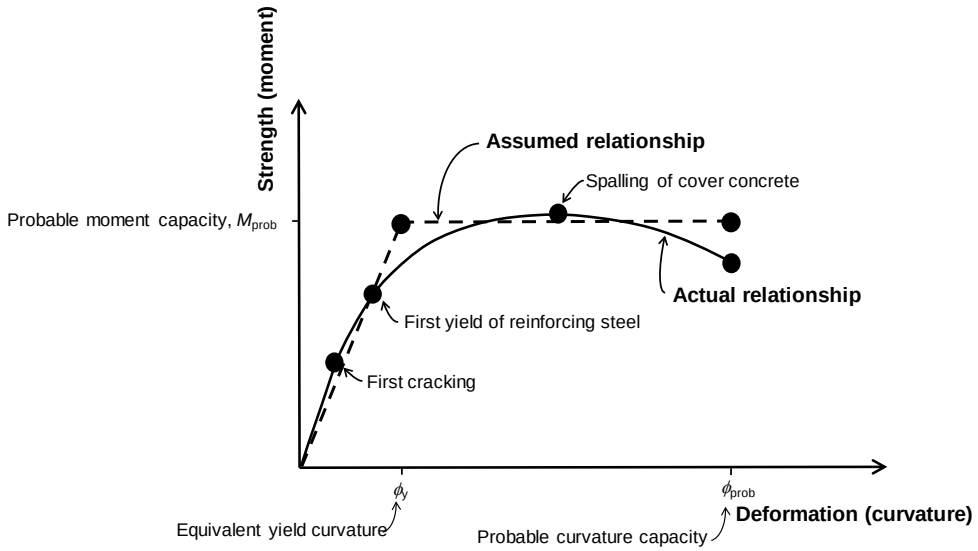


Figure C5.7: Bilinear representation of moment-curvature relationship

Priestley and Kowalsky (2000) have shown that the (equivalent) yield curvature can be well approximated with dimensionless formulae, with minimal variations due to the axial load and reinforcement ratio as follows.

For rectangular-section beams and columns:

$$\phi_y = \frac{2.0\,\varepsilon_y}{h_b} \quad \text{…C5.3}$$

For T-section beams:

$$\phi_y = \frac{1.7\varepsilon_y}{h_b} \quad \text{…C5.4}$$

For rectangular shear walls:

$$\phi_{\mathrm{y}} = \frac{2\varepsilon_{\mathrm{y}}}{l_w} \quad \text{...C5.5}$$

For flanged shear walls:

$$\phi_{\mathrm{y}} = \frac{1.5\varepsilon_{\mathrm{y}}}{l_w} \quad \text{…C5.6}$$

where:

h_{b} = beam or column depth

l_{w} = wall length.

Probable curvature capacity, ϕ_{cap}

The probable curvature capacity for a beam can be taken as the lesser of:

$$\phi_{\mathrm{cap}} = \frac{\varepsilon_{\mathrm{c,max}}}{c_{\mathrm{prob}}} \quad \text{…C5.7}$$

and:

$$\phi_{\mathrm{cap}} = \frac{\varepsilon_{\mathrm{s,max}}}{d - c_{\mathrm{prob}}} \quad \text{…C5.8}$$

where:

c_{prob}	=	neutral axis depth at probable capacity
$\varepsilon_{\mathrm{c,max}}$	=	the accepted maximum concrete compressive strain, at the extreme fibre of the section or of the confined core region, depending on the extent of confinement of the concrete (as defined in Table C5.6 and further explained below)
$\varepsilon_{\mathrm{s,max}}$	=	the maximum accepted strain of the reinforcing steel in tension (as defined in Table C5.6)
d	=	effective depth of longitudinal tension reinforcement.

Table C5.6: Concrete and steel strain limits for calculation of probable curvature capacity

		Strain limits
Concrete	**Unconfined (including cover concrete)**	$\varepsilon_{c,max} = 0.004$
	Confined core	$\varepsilon_{c,max} = 0.004 + \frac{1.4\rho_{st} f_{yh} \varepsilon_{ten}}{f'_{cc}} \leq 0.015$ where ρ_{st} = volumetric ratio of confinement reinforcement $= \frac{1.5A_v}{b_c s}$ for beams and columns f_{yh} = yield strength of the confinement reinforcement ε_{ten} = available strain at the tensile strength of the reinforcing steel f'_{cc} = compression strength of the confined concrete A_v = total area of confinement reinforcement in a layer s = spacing of layers of transverse reinforcement b_c = width of core, measured from centre to centre of the peripheral transverse reinforcement
Steel		$\varepsilon_{s,max} = 0.06 \leq 0.6\ \varepsilon_{ten}$ where ε_{ten} = available strain at the tensile strength of the reinforcing steel

Note:

In general terms, for assessment purposes, the probable deformation capacity is not assumed to be limited to a value of compression strain in the extreme fibre $\varepsilon_c = 0.003$ which is the typical approach used for ULS design of new elements, but rather when either:

(i) an overall reduction in strength of more than 20% occurs, or

(ii) the confined concrete-core reaches the defined confined concrete strain limit, or

(iii) the steel reaches a much higher level of strain (e.g. $\varepsilon_s = 0.06$).

These potential deformation capabilities of an existing beam element beyond crushing and spalling of the cover concrete, $\varepsilon_c = 0.004$, can be appreciated in the moment-curvature example given in Figure C5.8 below.

Moment-curvature analyses will show that, while the yield curvature is not greatly affected by axial load level (particularly when yield curvature is expressed in terms of equivalent elasto-plastic response), the probable curvature is strongly dependent on axial load.

This is illustrated in Figure C5.9 further below, where a poorly confined (transverse reinforcement D10@400, 2 legs only) end column of a frame with nominal axial load of $N = 0.2f'_c A_g$ is subjected to seismic axial force variations of $N_E = \pm 0.2f'_c A_g$. The yield curvatures differ by less than 10% from the mean, while the ultimate curvatures at $N = 0$ and $N = 0.4f'_c A_g$ are 61% and 263% of the value at $N = 0.2f'_c A_g$.

On the other hand, especially in columns with high axial load ratios, poor confinement detailing and large cover concrete, the loss of cover concrete (resulting from or combined with buckling of the longitudinal rebars) can correspond to the onset of full loss of axial load capacity refer below.

In general terms, the evaluation of ultimate curvature for walls can be carried out in a similar manner to that presented for columns. Special care should be taken in relation to the particular mechanisms of wall elements.

The main hypothesis of 'plane sections remain plane', i.e. linear strain profile along the wall section length, l_w, might not be valid at the probable moment capacity due to higher concentration of strains in both tension and compression area. Therefore, a traditional section analysis approach may lead to unconservative results and overestimate the curvature/rotation/displacement demand of walls.

However, while acknowledging the limitations of section analysis, it can still be a valuable approach to determine an upper bound of the deformation capacity of an existing wall under an ideal flexurally dominated behaviour.

Interaction with shear (either before or after yielding), local bar buckling or out-of-plane (lateral global) instability can lead to premature failure or achievement of ULS. More information on these failure mechanisms are described in the following sections and in Appendix C5H.

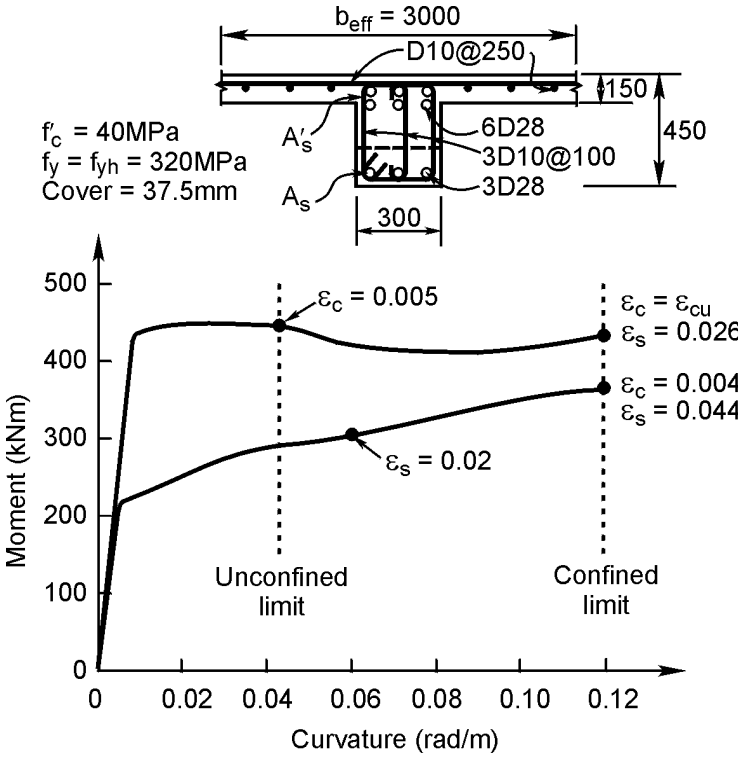


Figure C5.8: Example of a moment-curvature curve for a flanged (T or L) beam

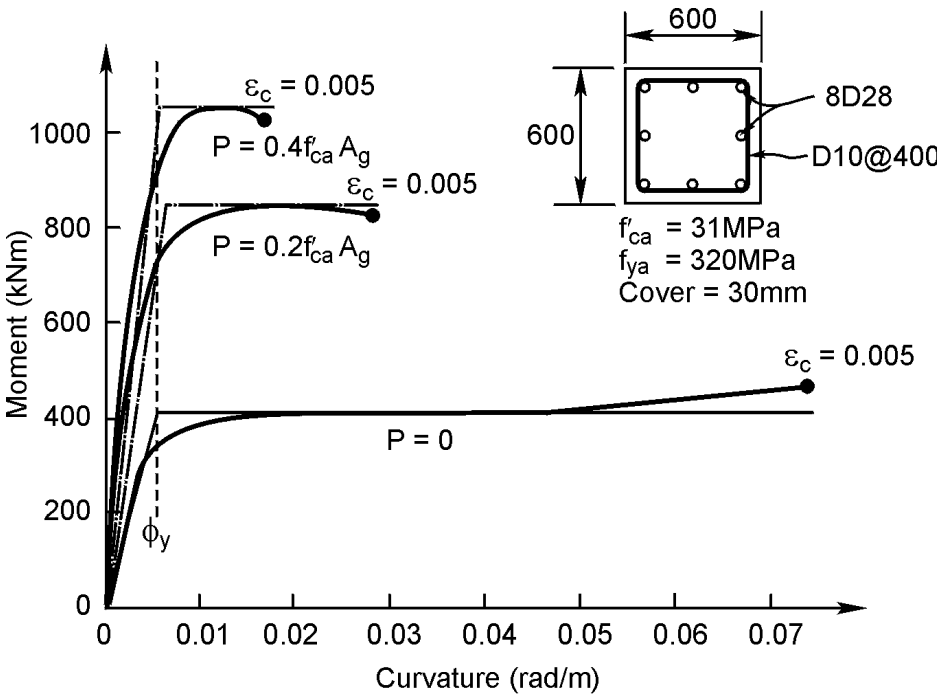


Figure C5.9: Example of a moment-curvature response of a column with poor confinement

Unconfined conditions are assumed to be present if at least one of the following applies:

- only corner bars restrained against buckling by a bend of transverse reinforcement, or
- hoop stirrup ends not bent back into the core (i.e. 90° hooks), or
- spacing of hoop or stirrup sets in the potential plastic hinge such that:

$$s \geq d/2$$
or
$$s \geq 16d_{\mathrm{b}}$$

where:

d = effective depth of the section
d_{b} = diameter of longitudinal reinforcement

When the section appears poorly confined (which is most likely to be the case for older construction) it is suggested that the confining effects on the concrete strength are neglected and $f'_{\mathrm{cc}}/f'_{\mathrm{c}} = 1.0$.

A confined core may be assumed in the presence of a good level of transverse reinforcement such that:

$$s < d/2$$
or
$$s < 12d_{\mathrm{b}}$$

and values of $f'_{\mathrm{cc}} = 1.2f'_{\mathrm{c}}$, may be assumed (Scott et al., 1982; Priestley et al., 1996).

Note:

The original formulation of the expression for confined concrete presented by Mander et al. (1988) can predict high levels of confined concrete strain, depending on the assumed value for the ultimate steel strain, of the transverse reinforcement. The modified expression suggested in fib Bulletin 25 (2003) provides a correction.

However, it is recommended that an upper bound value for the ultimate steel strain of 0.06 (i.e. 6%) is assumed and the values of confined concrete strain are limited to 0.015 (1.5%) in ordinary situations.

When using a moment-curvature analysis to establish, M_{prob}, ϕ_{y} and ϕ_{prob} the modelling of the materials (concrete, including the effect of confinement, and the reinforcing steel) should conform with the material properties established in Section C5.4 and the limitations set out above. To achieve consistency, modelling of the concrete should be in accordance with the modified Mander et al. expression referred to above.

Non-ductile columns

The probable capacity of non-ductile columns within the primary structure, which are described in Section C5.3.2, should be assessed in a similar manner as that recommended above for columns forming part of the lateral load resisting system.

However, given their critical role of gravity-load-carrying capacity and the lack of adequate detailing which could lead to brittle failure mechanisms, special care must be taken when assessing their capacity and performance. This acknowledges the higher level of uncertainty in the prediction of displacement/drift capacity associated with shear failure, particularly when bidirectional loading is considered.

Buckling of reinforcing steel in columns and walls

When the spacing of the transverse reinforcement restraining buckling of the vertical reinforcement in a wall is greater than $6d_{\text{b}}$, cyclic loading and strains in the vertical reinforcing bars greater than yield are expected, the probable curvature capacity of the wall section should be limited to:

$$\phi_{\text{cap}} = \frac{\varepsilon_{\text{p}}^{*}}{\gamma l_{\text{w}}} \qquad \text{...C5.9}$$

where:

$$\varepsilon_{\text{p}}^{*} = \frac{11-(5/4)(\text{s}/d_{\text{b}})}{100} \qquad \text{...C5.10}$$

γl_{w} is shown in Figure C5.10.

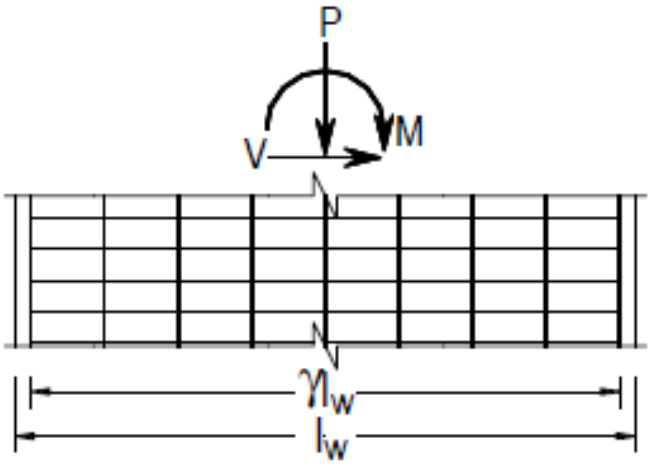


Figure C5.10: Definition of γl_w according to Rodriguez et al. (2013)

Note:

Buckling of reinforcing bars in RC elements is a complex phenomenon and, although the seismic design standards contain general detailing requirements to postpone or avoid this, there is currently limited information for assessing existing buildings. Appendix C5H discusses buckling of reinforcing bars in walls in more detail.

The following expression derived from that proposed by Berry & Eberhard (2005) can be employed to estimate the probable lateral displacement, Δ_{cap}, at which buckling of the longitudinal bars in a flexure-governed non ductile column is initiated.

$$\Delta_{cap} = 0.0325 L_c \left(1 + k_{e_bb} \rho_{eff} \frac{d_b}{D}\right) \left(1 - \frac{N}{A_g f'_c}\right) \left(1 + \frac{L_c/2}{10D}\right) \quad \ldots C5.11$$

where:

k_{e_bb} = transverse reinforcement co-oefficient
0 for columns with $s \geq 6d_b$
40 and 150 for rectangular columns and spiral-reinforced columns, respectively

L_c = distance of the critical section from the point of contraflexure

ρ_{eff} = effective confinement ratio

d_b = average diameter of longitudinal reinforcement

D = section effective depth

It is worth noting that the original expression proposed by Berry & Eberhard (2005) was calibrated on the drift ratio (Δ_u/L_c) obtained from experimental results. The dispersion of such expressions, when applied directly to derive displacement, is quite high and should be treated with care.

Out of plane (lateral) instability of walls

Out-of-plane (or lateral) instability is currently identified as one of the common failure modes of slender rectangular RC walls. This 'global' mode of failure, which involves a large portion of a wall element as opposite to the 'local' bar buckling phenomenon where a single rebar is affected, was previously observed in experimental studies of rectangular walls. However, it was not considered as a major failure pattern until the recent earthquakes in Chile (2010) and Christchurch (2011). Appendix C5H provides an overview of the issue and a description of current knowledge on the topic.

C5.5.2.2 Evaluation of moment-rotation and force–displacement curves for members and elements

Once the key points of the moment-curvature of a structural element (beams, columns or walls) have been evaluated, the corresponding moment-rotation curve can be derived by integrating the curvature profile (elastic and plastic) along the equivalent cantilever length and after defining a plastic hinge length. The plastic hinge length in this context is the portion of the member length over which the plastic behaviour is assumed to be concentrated and the plastic curvature is assumed to be constant.

The probable rotation capacity is defined as the sum of the yield rotation and plastic rotation capability:

$$\theta_{\text{cap}} = \theta_{\text{y}} + \theta_{\text{p}} \qquad \text{Probable rotation capacity} \qquad \text{…C5.12}$$

where:

$$\theta_{\text{y}} = \phi_{\text{y}} \left(\frac{H}{3}\right) \qquad \text{Yield rotation} \qquad \text{...C5.13}$$

$$\theta_{\text{p}} = \left(\phi_{\text{cap}} - \phi_{\text{y}}\right) L_{\text{p}} \qquad \text{Plastic rotation capability} \qquad \text{…C5.14}$$

The force-displacement response can then be derived (Figure C5.11) by:

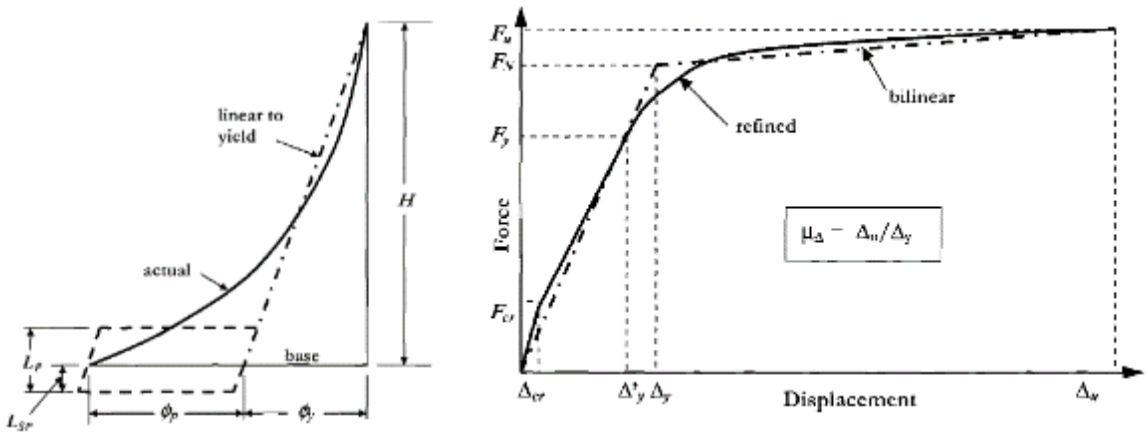
$$F = \frac{M}{H} \qquad \text{…C5.15}$$

$$\Delta_{\text{cap}} = \Delta_{\text{y}} + \Delta_{\text{p}} \qquad \text{Probable displacement capacity} \qquad \text{…C5.16}$$

where:

$$\Delta_{\text{y}} = \phi_{\text{y}} \frac{H^2}{3} \qquad \text{Yield displacement} \qquad \text{…C5.17}$$

$$\Delta_{\text{p}} = \left(\phi_{\text{cap}} - \phi_{\text{y}}\right) L_{\text{p}} H \qquad \text{Plastic displacement capability} \qquad \text{…C5.18}$$



(a) (b)

Figure C5.11: Idealisation of: (a) curvature distribution in a cantilever scheme and (b) force-displacement curve and its bilinear approximation

Note:

It is worth remembering that the axial load level critically affects the ultimate curvature and thus the ultimate rotation capacity of a column. A proper estimation of the expected level of axial load due to gravity loads and the variation due to the application of lateral seismic loads should be carried out. More details are provided in the following sections on beam-column joints, hierarchy of strength, and determination of the "seismic" axial load contribution from a frame sway mechanism.

In fact, from a rotation capacity point of view the critical column will be the one with highest axial compression, while from a moment capacity point of view the critical column will be the one with the lowest axial load.

Plastic hinge lengths

The estimation of the plastic hinge length, L_{p}, is a delicate step in the evaluation of the probable rotation and displacement capacity of a member. A number of alternative formulations are available in literature to predict the plastic hinge length in beams, columns and walls.

The equivalent plastic hinge length, L_{p}, may be approximated (Priestley et al., 2007) as:

$$L_{\mathrm{p}} = kL_{\mathrm{c}} + L_{\mathrm{sp}} \qquad \ldots\text{C5.19}$$

where:

$$k = 0.2\left(\frac{f_{\mathrm{u}}}{f_{\mathrm{y}}} - 1\right) \leq 0.08 \qquad \ldots\text{C5.20}$$

L_{c} = distance of the critical section from the point of contraflexure
L_{sp} = strain penetration = $0.022 f_{\mathrm{y}} d_{\mathrm{b}}$
f_{y} = probable yield strength of longitudinal reinforcement
d_{b} = diameter of longitudinal reinforcement
f_{u} = probable ultimate strength of the longitudinal reinforcement.

The first term, kL_c, represents the spread of plasticity due to tension-shift effects and the second term, L_{sp}, represents the strain penetration into the supporting member (e.g. beam-column joint).

Note:

The values presented above for the evaluation of the plastic hinge length are typically based on experimental results with reference to relatively well detailed plastic hinge regions and use of deformed bars.

However, when dealing with older construction practice, poorer detailing, low longitudinal reinforcement ratio (lightly reinforced elements), construction (cold) joints, high tensile strength of concrete, and possibly plain round bars, experimental tests as well as on-site observations from the 2010-11 Canterbury earthquake sequence have shown that the plastic hinge length may not develop to be as long as expected. Instead, it may be concentrated in a very short region, leading to a single crack opening and concentration of tensile strain demand in the reinforcement.

Such effects should be accounted for in the evaluation of the plastic hinge length, L_p, assuming much smaller values of the plastic hinge length, and assessing the effects on the overall behaviour (limited ductility/deformation capacity).

It is recommended that a plastic hinge length equal to $L_p/5$ is adopted (with L_p derived from the expressions above) in the presence of either:

- plain round bars
- a low longitudinal reinforcement ratio, i.e. $\rho_\ell \leq \sqrt{f'_c}/(4f_y)$ l , or
- an inadequately constructed cold joint, e.g. smooth and unroughened interfaces.

As suggested by Priestley et al. (2007), the plastic hinge length of shear walls is more likely to be influenced by tension shift effects than is the case with beams or columns.

Therefore, when compared to the expression for plastic hinge length in beams and columns, an additional term in the plastic hinge equation should be included as a function of the wall length as follows:

$$L_P = k.L_C + 0.1l_w + L_{SP} \quad \text{...C5.21}$$

$$k = 0.2\left(\frac{f_u}{f_y} - 1\right) \leq 0.08 \quad \text{...C5.22}$$

$$L_{SP} = 0.022 f_y d_b \quad \text{...C5.23}$$

where:

L_C = distance from the critical section to the point of the contraflexure
l_w = wall length.

Note:

The values presented above for walls are typically based on experimental results with reference to relatively well detailed plastic hinge regions and use of deformed bars.

However, as observed following the Canterbury earthquake sequence (Kam, Pampanin and Elwood, 2011; Structural Engineering Society of New Zealand (SESOC) 2011; Sritharan & al., 2014), when dealing with older construction practice, and in the specific case of walls, with:

- a low longitudinal reinforcement ratio – i.e. lightly reinforced walls
- construction (cold) joints
- high tensile strength of concrete, and possibly
- plain round bars,

the plastic hinge length may be concentrated in a very short region with mostly a single main flexural crack, as opposed to distributed cracking over a length. This concentration of tensile inelastic strain demand in the reinforcement resulted in premature fracture of vertical reinforcement.

In fact, while primary cracks occur as a result of the global flexural action on the wall, if low vertical reinforcement ratio is provided the tension force generated by the reinforcing steel – and thus the tensile stress generated in the surrounding concrete – may be insufficient to develop secondary flexural cracks.

Recent studies suggests that even recent design provisions (including NZS 3101:2006 with a specified minimum reinforcement ratio of $\rho_{\mathrm{n}} \geq \sqrt{f'_{\mathrm{c}}}/(4f_{\mathrm{y}})$) may not be sufficient to ensure distributed cracking in the ductile plastic hinge regions, thus potentially resulting in premature bar fracture, and lower-than expected drift capacities (Henry, 2013).

More specifically, not only the total reinforcement ratio along the full section but also the amount (or lack of) longitudinal reinforcement concentrated in the boundary region can facilitate the formation (or impairment) of secondary cracks.

As part of the assessment procedure, such effects should be accounted for in the evaluation of the plastic hinge, L_{p}.

A simple and practical approach would be to assume much smaller values of the plastic hinge length, as $L_{\mathrm{p}}/5$, and evaluate its effects on the overall behaviour (limited ductility/ deformation capacity).

Also note that large crack openings at the wall base can cause additional problems such as large axial elongations, wall sliding, or out-of-plane wall instability.

C5.5.2.3 Probable shear capacity

The probable shear capacity of reinforced concrete beams, columns and walls can be taken as:

$$V_{\mathrm{prob}} = 0.85(V_{\mathrm{c}} + V_{\mathrm{s}} + V_{\mathrm{n}}) \qquad \ldots \text{C5.24}$$

where V_{c}, V_{s} and V_{n} are the shear contributions provided by the concrete mechanism, steel shear reinforcement and (where present) the axial compressive load respectively, as described below.

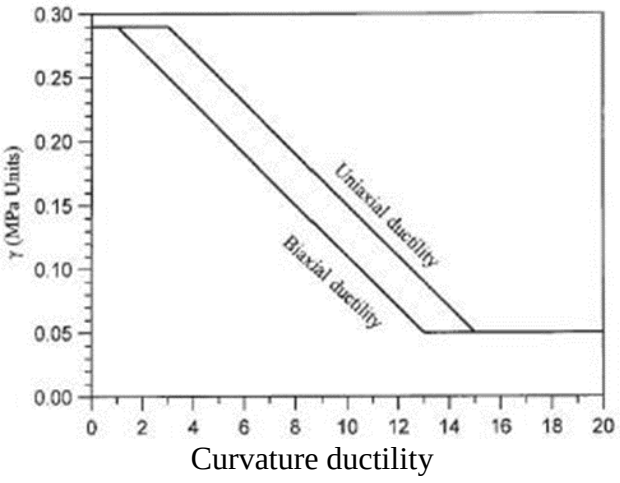
The shear contribution from the concrete, V_c, can be evaluated as:

$$V_c = \alpha\beta\gamma\sqrt{f'_c}(0.8A_g) \quad \ldots C5.25$$

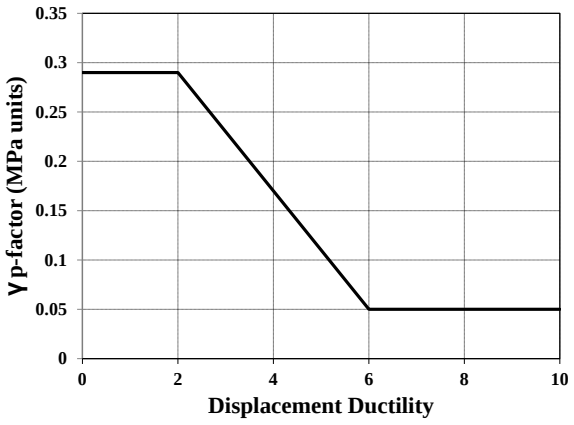
where:

$$1 \le \alpha = 3 - \frac{M}{VD} \le 1.5$$

β = $0.5 + 20\rho_l \le 1$
γ = shear strength degradation factor (refer to Figure C5.12)
A_g = gross area of the member section ($b_w d$ for a beam)
b_w = width of section web
d = effective depth of section
M/V = ratio of moment to shear at the section
D = total section depth or the column diameter as appropriate
ρ_l = ratio of the total area of the longitudinal reinforcement.



(a) Beams and columns



(b) Walls

Figure C5.12: Concrete shear strength degradation factor as a function of ductility: curvature ductility for beams and columns and displacement ductility for walls.

The shear contribution from the shear reinforcing steel, $V_{s,}$ may be evaluated as follows.

For beams assuming that the critical diagonal tension crack is inclined at 45° to the longitudinal axis of the beam:

$$V_s = \frac{A_v f_{yt} d}{s} \quad \text{…C5.26}$$

where:

A_v = total effective area of hoops and cross ties in the direction of the shear force at spacing s
f_{yt} = probable yield strength of the transverse reinforcement
d = effective depth of the beam.

For columns assuming that the critical diagonal tension crack is inclined at 30° to the longitudinal axis of the column:

- For rectangular hoops:

$$V_s = \frac{A_v f_{yt} d''}{s} \cot 30º \quad \text{…C5.27}$$

- For spirals or circular hoops:

$$V_s = \frac{\pi}{2} \frac{A_{sp} f_{yt} d''}{s} \cot 30º \quad \text{…C5.28}$$

where:

A_v = total effective area of hoops and cross ties in the direction of the shear force at spacing s
A_{sp} = area of spiral or circular hoop bar
f_{yt} = expected yield strength of the transverse reinforcement
d'' = depth of the concrete core of the column measured in the direction of the shear force for rectangular hoops and the diameter of the concrete core for spirals or circular hoops.

For walls the shear contribution of the effective horizontal reinforcing steel, V_s, may be evaluated as follows:

$$V_s = \frac{A_v f_{yh} h_{cr}}{s} \quad \text{…C5.29}$$

where:

$$h_{cr} = \frac{l'}{\tan \theta_{cr}} \leq h_w \quad \text{…C5.30}$$

$$l' = l_w - c - c_0 \quad \text{…C5.31}$$

$$\theta_{cr} = 45 - 7.5 \left(\frac{M}{V.l_w} \right) \geq 30^{\circ} \quad \text{…C5.32}$$

A_{v} = horizontal shear reinforcement
f_{yh} = yield strength of transverse reinforcement
s = centre-to-centre spacing of shear reinforcement along member
h_{w} = wall height
c = the depth of the compression zone
c_0 = the cover to the longitudinal bars
l_{w} = wall length.

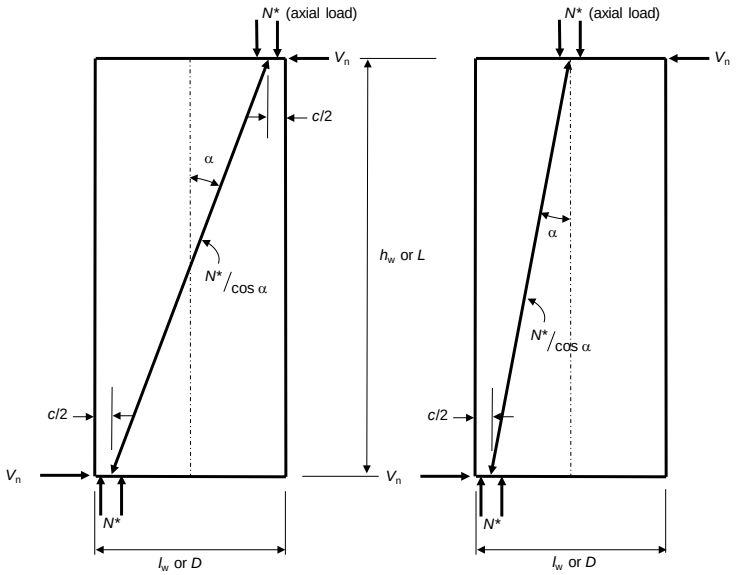
The shear resisted as a result of the axial compressive load N^* is given by:

$$V_{\mathrm{n}} = N^* \tan \alpha \qquad \ldots \text{C5.33}$$

where:

α = for a column or wall in double curvature (reverse bending), the angle between the longitudinal axis of the member and the straight line between the centroids of the concrete compressive forces of the member section at the top and bottom of the column (refer to Figure C5.13(a)).
for a cantilever column or wall (single bending), the angle between the longitudinal axis of the member and the straight line between the centroid of the member section at the top and the centroid of the concrete compression force of the member section at the base (refer to Figure C5.13(b))

N = axial compressive load.



(a) Reverse Bending **(b) Single Bending**

Figure C5.13: Contribution of compressive axial force to shear strength in a column or wall based on Priestley et al. (1994, 1995, 2007)

Therefore, for a cantilever:

$$V_{\mathrm{n}} = N^{*}\left(\frac{l_{\mathrm{w}}-c}{2h_{\mathrm{w}}}\right) = N^{*}\left(\frac{D-c}{2L}\right) \quad \ldots\text{C5.34}$$

or for a member with reverse bending:

$$V_{\mathrm{n}} = N^{*}\left(\frac{l_{\mathrm{w}}-c}{h_{\mathrm{w}}}\right) = N^{*}\left(\frac{D-c}{L}\right) \quad \ldots\text{C5.35}$$

where:

h_{w} = wall height
L = column height
D = column depth
c = the depth of the compression zone
l_{w} = wall length.

Note:

The formulation of shear capacity for walls herein reported has been proposed by Krolicki et al. (2011) and is based on the modified UCSD (University of California, San Diego) shear model proposed by Kowalsky and Priestley (2000) and updated by Priestley et al. (2007) for the evaluation of the shear capacity of columns.

Displacement/drift capacity due to flexure-shear failure mechanism

Exceeding the shear capacity of RC columns in a flexure-shear mode does not necessarily imply loss of axial load carrying capacity. In such a mixed mode, when shear capacity is exceeded, axial load can still be supported by the longitudinal reinforcing bars and force transfer through shear friction.

When a column behaviour is characterised by a flexural-shear behaviour with shear strength reduction due to ductility demand, the ultimate displacement capacity can be estimated as the point at which the probable shear strength degradation curve intersects the probable flexural strength curve.

The displacement of a column at the point that the shear capacity is reached, Δ_{s}, can be roughly estimated from (Elwood and Moehle, 2005). In the context of these guidelines, Δ_{s}, is to be considered as the probable drift/displacement based limit associated with the evaluation of *%NBS*:

$$\Delta_{\mathrm{s}=} L_{\mathrm{c}}\left(0.03 + 4\rho_{\mathrm{s}} - 0.024\frac{v}{\sqrt{f'_{\mathrm{c}}}} - 0.025\frac{P}{A_{\mathrm{g}}f'_{\mathrm{c}}}\right) \geq 0.01L_{\mathrm{c}} \quad \ldots\text{C5.36}$$

Details-dependent drift levels are calculated for the yielding of the section, shear failure and post shear-failure loss of axial load carrying capacity.

Note:

Shear mechanisms, particularly the post-peak displacement behaviour of columns dominated by shear failure mechanisms, is a complex area of research that is still under development. Different models have been proposed (e.g. Elwood and Moehle, 2005; Yoshimura, 2008), which can provide a significant scatter in terms of predicted values.

Given the dramatic impact that shear failure of columns in particular can have, as this can lead to loss of gravity bearing capacity, it is recommended that the assessment of their ultimate capacity is treated with care and that specific remedial (retrofit) interventions are considered to eliminate such potentially significant critical structural weaknesses (CSWs).

C5.5.3 Beam-column joints

C5.5.3.1 Probable shear strength of beam-column joints

Joints without any shear reinforcement

For interior and exterior beam-column joints without shear reinforcement, as typical of pre-1970s buildings, the probable horizontal joint shear force that can be resisted is:

$$V_{\text{prob,jh}} = 0.85 v_{\text{prob,ch}} b_{\text{j}} h$$

$$= 0.85 k_{\text{j}} \sqrt{f'_{\text{c}}} \sqrt{\left(1 + \frac{N^*}{A_{\text{g}} k \sqrt{f'_{\text{c}}}}\right)} b_{\text{j}} h \leq 1.92 \sqrt{f'_{\text{c}}} b_{\text{j}} h \qquad \text{…C5.37}$$

where:

$v_{\text{prob,ch}}$	=	probable horizontal joint shear stress capacity of the diagonal compressive strut mechanism crossing the joint
b_{j}	=	effective width of the joint (being normally the column width as per NZS 3101:2006)
h	=	depth of column
k_{j}	=	Coefficient for calculating the shear capacity of a joint

The following values for k_{j} should be used:

- for interior joints, $k_{\text{j}} = 0.8$ (note that compression failure rather than tensile failure would govern in an interior beam-column joint)
- for exterior joints with beam longitudinal (deformed) bars anchored by bending the hooks into the joint core, $k_{\text{j}} = 0.4$
- for exterior joints with beam longitudinal (deformed) bars anchored by bending the hooks away from the joint core (into the columns above and below), $k_{\text{j}} = 0.3$
- for exterior joints with beam longitudinal (plain round) bars anchored with end hooks, $k_{\text{j}} = 0.2$.

Note:

These recommended values for k_{j} are based on experimental testing from Hakuto et al., 1995-2000 (mostly focusing on deformed bars with no variation of axial load) and Pampanin et al., 2000-2010 (mostly focusing on plain round bars and variation of axial load).

v_{ch} indicates the estimated maximum nominal horizontal joint core shear stress, calculated the conventional way, resisted by beam-column joints in tests without joint shear reinforcement and without axial load.

The term indicating the influence of axial load, $\sqrt{\left(1+\frac{N^*}{A_g k\sqrt{f'_c}}\right)}$ was obtained by assuming that the diagonal (principal) tensile strength, p_t, of the concrete was $p_t = k_j\sqrt{f'_c}$ and using Mohr's circle to calculate the horizontal shear stress required to induce this diagonal (principal) tensile stress when the vertical compressive stress is N^*/A_g (Hakuto et al., 2000, Pampanin et al., 2002).

The factor of 0.85 has been included in Equation C5.37 to account for the higher uncertainty (and impact) of a shear failure mechanism when compared to a flexural one.

In fact, it has been demonstrated (Priestley, 1997; Pampanin, 2002) that principal tensile and compression stresses, p_t and p_c, are more appropriate indicators of joint damage than the probable shear stress $v_{prob,jh}$, as they can take the variation of axial load into account.

Principal tensile stresses, p_t, would tend to govern the failure mechanism of exterior beam-column joints (tensile cracking), while principal compression stresses, p_c, would tend to govern interior beam-column joints where higher levels of axial load are expected and the damage/failure mechanism is more correlated to the degradation of the diagonal compression strut.

Figure C5.14 shows strength degradation curves p_t versus γ (shear deformation) as well as p_t versus drift presented in literature and based on extensive experimental tests.

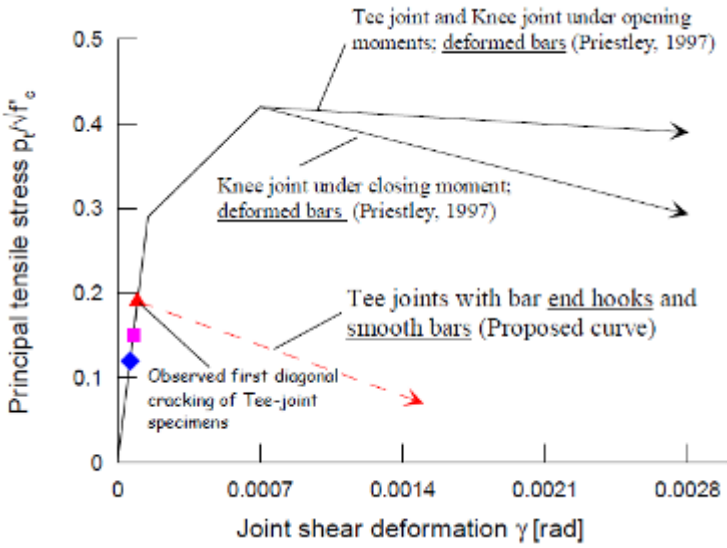


Figure C5.14: Strength degradation curves for exterior joints (Pampanin et al., 2002)

Indicative deformation limits at ULS for exterior beam column joints with no shear reinforcement, expressed in terms of shear deformation, γ [rad], and inter-storey drift, θ, are reported in Table C5.7.

In the case of interior joints, given the possibility to develop a joint shear transfer mechanism via diagonal compression strut the deformation limits of Table C5.7 can be increased by approximately 50%.

Table C5.7: Suggested ULS deformation limits for exterior joints with no shear reinforcement (modified after Magenes and Pampanin, 2004)

	ULS deformation limits
Shear deformation, γ [rad]	$0.01 \leq \gamma < 0.015$
Drift, θ [%]	$1.2\% \leq \theta < 1.8\%$

Note:

The deformation limits proposed above are based on experimental and numerical investigations on beam-column joint subassemblies and frame systems.

It is worth noting that the inter-storey drift corresponding to a specific damage level in the joint panel zone would depend on the elastic and plastic contribution of beams and column and thus would need to be checked on a case-by-case basis.

Joints with some shear reinforcement

For interior and exterior beam-column joints with some shear reinforcement (stirrups), the probable horizontal joint shear force that can be resisted is:

$$V_{\text{prob,jh}} = 0.85 v_{\text{prob,jh}} b_{\text{j}} h \qquad \text{…C5.38}$$

For joints with interior stirrups the joint shear stress can be computed, based on similar considerations on Mohr's Circle approach, as:

$$v_{\text{prob.jh}} = 0.85\, k_{\text{j}} \sqrt{f'_{\text{c}}} \sqrt{1 + k\sqrt{f'_{\text{c}}}(f_{\text{v}} + f_{\text{h}}) + f_{\text{v}} f_{\text{h}}} \quad \text{for exterior joints} \qquad \text{…C5.39}$$

$$v_{\text{prob,jh}} = 0.85\, k_{\text{j}} f'_{\text{c}} \sqrt{1 + k f'_{\text{c}}(f_{\text{v}} + f_{\text{h}}) + f_{\text{v}} f_{\text{h}}} \quad \text{for interior joints} \qquad \text{…C5.40}$$

where:

k_{j} = Coefficient for calculating the shear capacity of a joint

f_{v} = $\frac{N}{A_{\text{g}}}$ is the axial load stress on the joint

f_{h} = $\frac{A_{\text{st}} f_{\text{sy}}}{b_{\text{j}} h_{\text{b}}}$ represents horizontal confinement effects due to the stirrups in the joint and is calculated as the maximum tension stress that the stirrups can develop at yield.

Note:

The expression above is used in Eurocode 8 to determine the required amount of stirrups in a joint.

For $f_{\text{h}} = 0$ (and after substituting the definition of principal tensile stress, p_{t}, as a function of nominal shear stress, $v_{\text{prob,jh}}$ and the axial load stress f_{v}) the general equation for joints with shear reinforcement (Equation C5.38) converges to the equation for joints with no shear reinforcement (Equation C5.36).

Taking a rigorous approach, the joint capacity would be evaluated considering both principal tensile and compression stresses approach. However, in practical terms and considering that exterior joints are mostly governed by tensile cracking failure and interior joints by compression (crushing) failure, the expression presented above (based on principal tensile stress $p_t = k\sqrt{f'_c}$) can be used for exterior joints.

For interior joints a similar expression based on principal compression stresses is obtained by replacing $p_t = k\sqrt{f'_c}$ with $p_c = k_c f'_c$ and assuming $k = 0.6$ for critical damage level.

Effects of bidirectional cyclic loading on joint capacity

The effects of bidirectional loading can significantly affect the response of poorly detailed beam-column joints and modify the hierarchy of strength and sequence of events of the sub-assembly – and thus possibly the overall global response of the frame.

Conceptually, the shear (or equivalent moment) strength reduction due to bidirectional loading is similar to that expected in a column (both in flexure and shear) when subjected to bidirectional loading (refer to Figure C5.15).

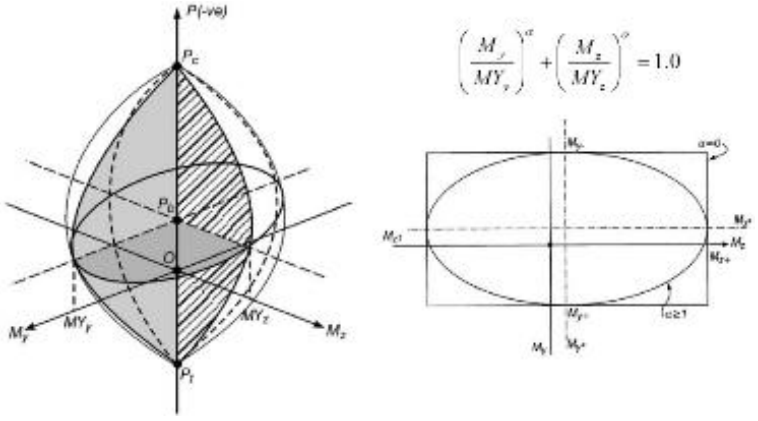


Figure C5.15: Conceptual moment-axial load ($M_y - M_z - P$) or shear-axial load ($V_y - V_z - P$) interaction surface for a reinforced concrete element (including beam-column joint) subjected to bi-axial loading

In the absence of more detailed study or evidence, a reduction of 30% on the probable joint shear (strength) capacity within the sub-assembly hierarchy of strength may be made when the joint is subjected to bidirectional loading. Also, it is suggested that the lower bounds of the deformation limit states indicated in Table C5.7 are adopted to account for the effect of bidirectional loading.

Note:

Overlooking the effects of bidirectional loading on the local and global response and the performance of an RC structure can significantly impair the efficiency of a retrofit intervention.

Most of the available studies available on the seismic assessment and retrofit of existing poorly detailed frame buildings have concentrated on the two-dimensional response, thus subjecting the specimen or subassemblies to unidirectional cyclic loading testing protocols. Even when the 3D response under combined bidirectional loading has been taken into account in experimental testing, the focus has been typically on interior (fully or partially confined) joints.

As part of a more extensive research programme on seismic retrofit solutions for New Zealand RC buildings, the effects of bidirectional loading – which is more representative of the actual seismic response of a building structure – on the assessment and design of the retrofit intervention have been investigated (Akguzel and Pampanin, 2010).

C5.5.4 Concrete floor diaphragms

C5.5.4.1 General

For most concrete diaphragms the in-plane deformations associated with diaphragm actions will be negligible. Therefore, the assumption of rigid diaphragm behaviour is likely to be generally satisfactory.

One notable exception to this is that stiffness of transfer diaphragms should typically be included explicitly in the analysis (e.g. in the common situation of a suspended ground floor above a basement) to avoid potentially unrealistically large diaphragm forces.

Note:

When assessing buildings it is important to recognise that there is an inherent difference between the performance and integrity of precast flooring systems and traditional cast-in-situ concrete floors. Precast floors with cast-in-situ concrete topping are not as robust or tolerant to racking movements under earthquake actions as cast-in-situ floors. These will require additional assessment to determine that adequate performance can be achieved.

C5.5.4.2 Diaphragm analysis

Design actions on concrete diaphragms should be determined using a strut and tie analysis.

For buildings that are essentially rectangular, have a relatively uniform distribution of vertical lateral force-resisting systems across the plan of the building, and have no significant change of plan with height, simple hand-drawn strut and tie solutions can be used (refer to Figure C5.16).

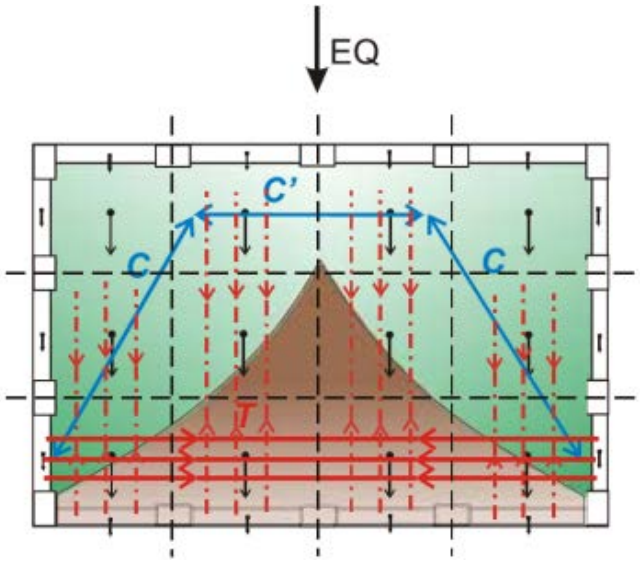


Figure C5.16: Example of a hand-drawn strut and tie solution for simple building (Holmes, 2015)

However, buildings with significant asymmetry in the location of lateral force-resisting elements (distribution across the building plan, termination up the height of the building, varying stiffness and/or strength between vertical elements) may require a more sophisticated analysis.

For these types of structures, a grillage method can be used to obtain diaphragm design actions (Holmes, 2015). The key steps for this method are as follows and are also shown in Figures C5.17and C5.18. Further details of the diaphragm grillage modelling methodology are provided in Appendix C5F.

Step 1

Determine the geometric properties of the diaphragm elements (i.e. topping thickness, beam sizes, etc.) from available structural drawings and site measurements.

Step 2

Identify areas of potential diaphragm damage which may limit diaphragm load paths (i.e. floor separation due to beam elongation, etc.) (refer to Section C5.5.4.3 below).

Step 3

Calculate probable capacities of diaphragm collector, tie and strut elements using available structural drawings and site investigation data (refer Section C5.5.4.4).

Step 4

Determine grillage section properties and complete the grillage model.

Next, for each principal direction of earthquake loading to be considered complete the following steps.

Step 5

Calculate building overstrength factor, ϕ_{ob}, and overstrength diaphragm inertia forces using the pseudo-Equivalent Static Analysis (pESA) procedure detailed in Section C2.

Step 6

Determine "floor – forces", F_{Di}, from the pESA and apply these to the nodes in the grillage model associated with vertical lateral load resisting elements.

Step 7

Determine vertical element out-of-plane "floor – forces", F_{OPi}, from the pESA and apply these to the nodes in the grillage model.

Step 8

Run the grillage model analysis to determine the seismic demands on the diaphragm elements.

Step 9

Check the capacity of the diaphragm elements against the seismic demands.

Step 10

If the diaphragm has enough capacity to resist the seismic demands, go to Step 12. Otherwise, if the seismic demands on selected diaphragm elements exceed their capacity, redistribution can be used to utilise other load paths which may exist.

Step 11

Re-check the capacity of the diaphragm elements against the redistributed building seismic demands. If, after redistribution, the diaphragm does not have adequate capacity to resist the seismic demands then reduce the diaphragm inertia forces and return to Step 6. If the diaphragm has adequate capacity to resist the redistributed seismic demands proceed to Step 12.

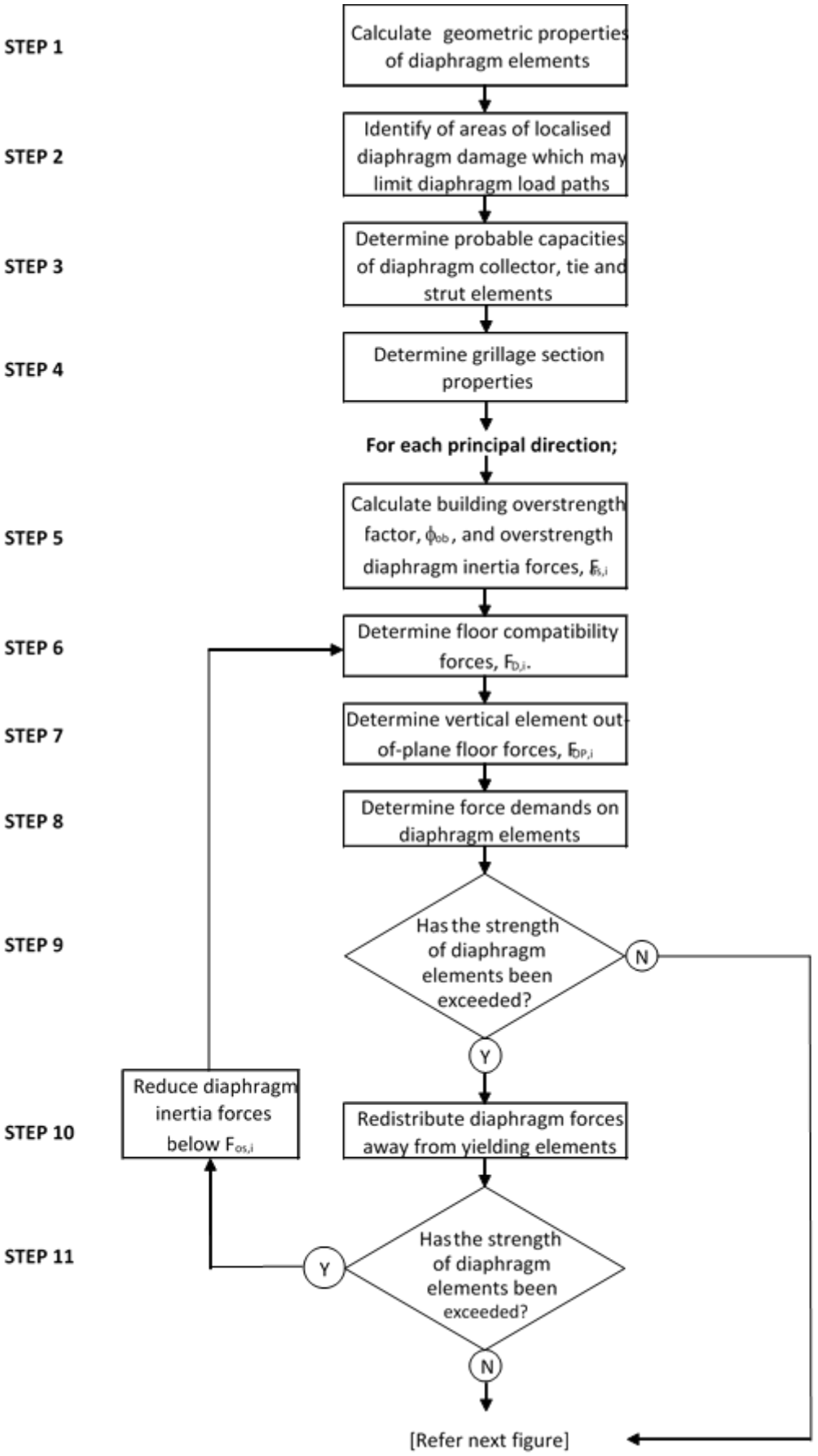


Figure C5.17: Summary of diaphragm assessment procedure – Steps 1 to 11

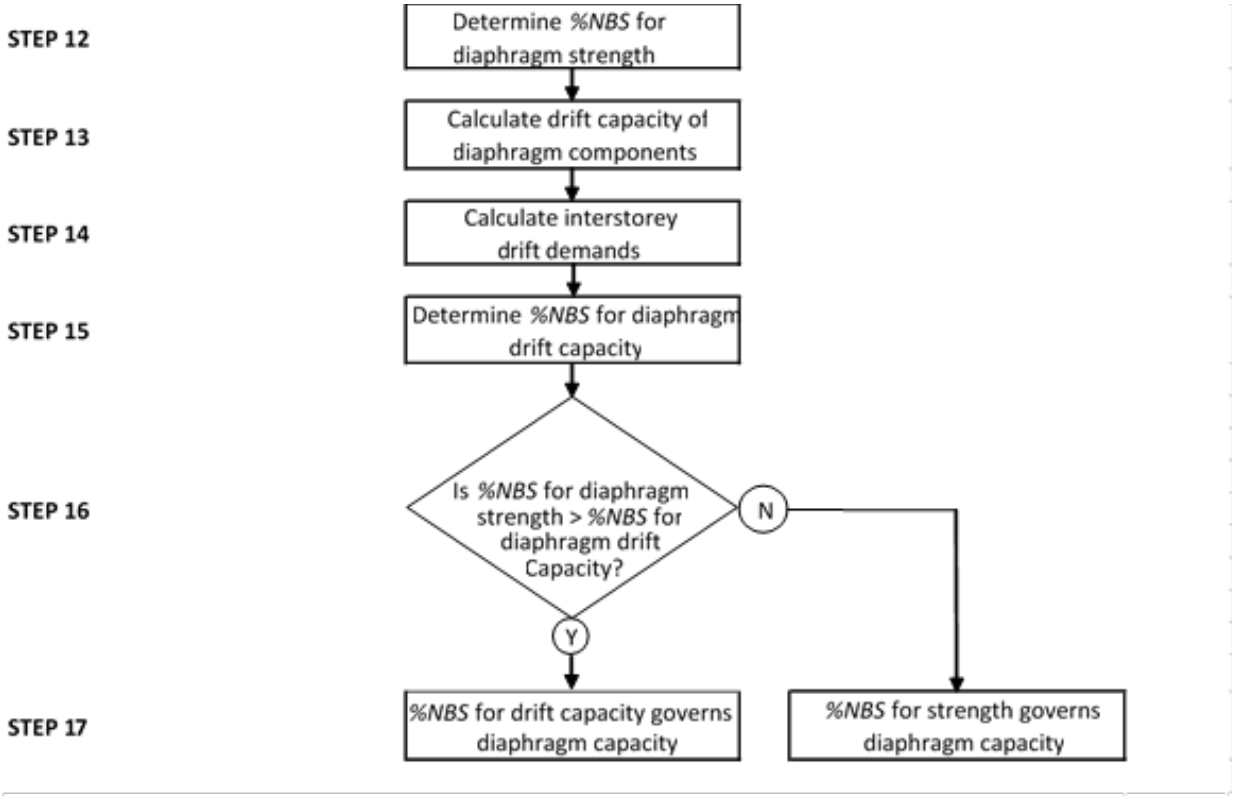


Figure C5.18: Summary of diaphragm assessment procedure – Steps 12 to 17

Step 12

Determine *%NBS* for the diaphragm in terms of strength (refer to Section C5.5.4.4). If the capacity of the diaphragm is greater than the seismic demands calculated using the building overstrength factor, ϕ_{ob}, the diaphragm can be taken as 100*%NBS*. If the diaphragm demands were reduced below the building overstrength demands in Step 11, the *%NBS* for each diaphragm element should be determined as follows:

$$\%NBS = 100\frac{0.9R_{\text{prob}}}{K_{\text{d}}R_{\text{E},\mu=1.25}} \quad \text{…C5.41}$$

where:

R_{prob} = probable capacity of diaphragm element calculated in Step 3

$R_{\text{E},\mu=1.25}$ = diaphragm element demand calculated using the pESA procedure detailed in Section C2, with the base shear V_{E} calculated from Section 6.2 of NZS 1170.5:2004 using $\mu = 1.25$ and $S_{\text{p}} = 0.9$

K_{d} = demand-side multiplier such that K_{d} = 1.5 for diaphragm collector elements and K_{d} = 1.0 for all other ties and struts.

Redistribution between diaphragm elements is permitted. The *%NBS* for the diaphragm in terms of strength is the minimum of the *%NBS* values assessed for each individual diaphragm element.

Note:

A higher demand side multiplier of 1.5 is applicable to collector elements recognising that these elements are force controlled, and typically have low redundancy and a high consequence of failure. The demand side multiplier of 1.5 is intended to provide a margin of resilience.

Step 13

Calculate the probable inter-storey drift capacity, $\theta_{\text{prob,SC}}$, of diaphragm components. This includes assessing any precast concrete floor units for loss of support and assessing the seismic capacity of the units themselves (refer to Section C5.5.4.3).

Step 14

Calculate inter-storey drift demands, θ_{SD}, in accordance with Section C2 of these guidelines. Section C5.5.4.5 below provides additional guidance on how the NZS 1170.5:2004 structural performance factor, S_{p}, should be applied.

Step 15

Determine *%NBS* for the diaphragm in terms of inter-storey drift. The *%NBS* for each diaphragm element should be determined as follows:

$$\%NBS = 100\frac{\theta_{\text{SC}}}{K_{\text{d}}\theta_{\text{SD}}} \quad \text{…C5.42}$$

where:

$\theta_{\text{prob,SC}}$ = probable inter-storey drift capacity of diaphragm component
θ_{SD} = inter-storey drift demand on diaphragm component
K_{d} = demand-side multiplier such that K_{d} = 1.5 for precast concrete diaphragm elements and their support, and K_{d} = 1.0 for in-situ concrete diaphragm elements.

The *%NBS* for the diaphragm in terms of inter-storey drift is the minimum of the *%NBS* values assessed for each individual diaphragm element.

Step 16

Check if the *%NBS* for the diaphragm in terms of strength calculated in Step 12 is greater than the *%NBS* for the diaphragm in terms of inter-storey drift calculated in Step 15.

Step 17

The *%NBS* for the diaphragm is the minimum of the two *%NBS* values considered in Step 16.

C5.5.4.3 Diaphragm damage due to deformation compatibility

Deformation demands of the primary lateral force-resisting systems can cause damage to the diaphragm structure (as a result of beam elongation or incompatible relative displacements between the floor and adjacent beams, walls or steel braced frames). Figure C5.19 illustrates an example of diaphragm damage due to beam elongation.

The assessment of inter-storey drift capacity of diaphragms consisting of precast concrete components needs to consider the following:

- loss of support of precast floor units, and
- failure of precast floor units due to seismic actions, including the consideration of incompatible displacements.

Figure C5.19: Observed separation between floor and supporting beam due to beam elongation in 2011 Canterbury earthquakes (Bull)

Appendix C5G provides an assessment procedure for precast floors with cast-in-situ concrete topping.

Note:

Precast floors with cast-in-situ concrete topping are not as robust or tolerant to racking movements as traditional cast-in-situ concrete floors. Failure of a precast floor unit in the upper level of a building is likely to result in progressive collapse of all floors below that level. Therefore, additional assessment is recommended to ensure that adequate performance can be achieved during an earthquake.

C5.5.4.4 Assessment of diaphragm capacities

The capacity of diaphragm strut and tie elements can be calculated in accordance with Appendix A of NZS 3101:2006 using probable material strengths and a strength reduction factor, ϕ, equal to 1.0. Reduction factors β_n and β_s should be taken as specified in NZS 3101:2006.

C5.5.4.5 Inter-storey drift demands on diaphragm components

Inter-storey drift demands on diaphragm components can be determined in accordance with one of the applicable analysis methods detailed in Section C2 except as modified below (Fenwick et al., 2010):

- When calculating member elongations the structural performance factor, S_p, adopted for the primary lateral resisting system can be used to determine the plastic hinge rotations.

- When assessing brittle failure modes of precast concrete components (i.e. web-splitting of hollowcore floor units, loss of support, etc.) the peak displacements determined from the analysis of the primary lateral load resisting system should be increased by $1/S_{\mathrm{p}}$, where the value of S_{p} is that used in the analysis of the primary lateral load resisting system.

C5.6 Global Capacity of Moment Resisting Concrete Frame Buildings

C5.6.1 Evaluation of the hierarchy of strength and sequence of events for a beam-column joint sub-assembly

Once the flexural and shear capacity of the components are evaluated, the hierarchy of strength and expected sequence of events within a beam-column joint can be carried out by comparing capacity and demand curves within an M-N (moment-axial load) performance domain.

Figure C5.20 illustrates an example of the M-N performance domain adopted to predict the sequence of events and the level of damage in the joint panel zone of a 2D exterior beam-column joint sub-assembly. According to such a procedure, the capacities of beams, columns and joints need to be evaluated in terms of a common parameter. This is recommended to be the equivalent moment in the column, based on equilibrium considerations corresponding to the selected limit state (e.g. cracking/"yielding" or peak capacity in the joint versus yielding of beams and columns).

The order and "distance" of the events (e.g. beam hinging, joint shear, column hinging) can also strongly depend on the axial load demand. If a constant axial load was assumed, as often done for simplicity, an erroneous sequence of events might be predicted leading to the potential implementation of an incorrect retrofit strategy.

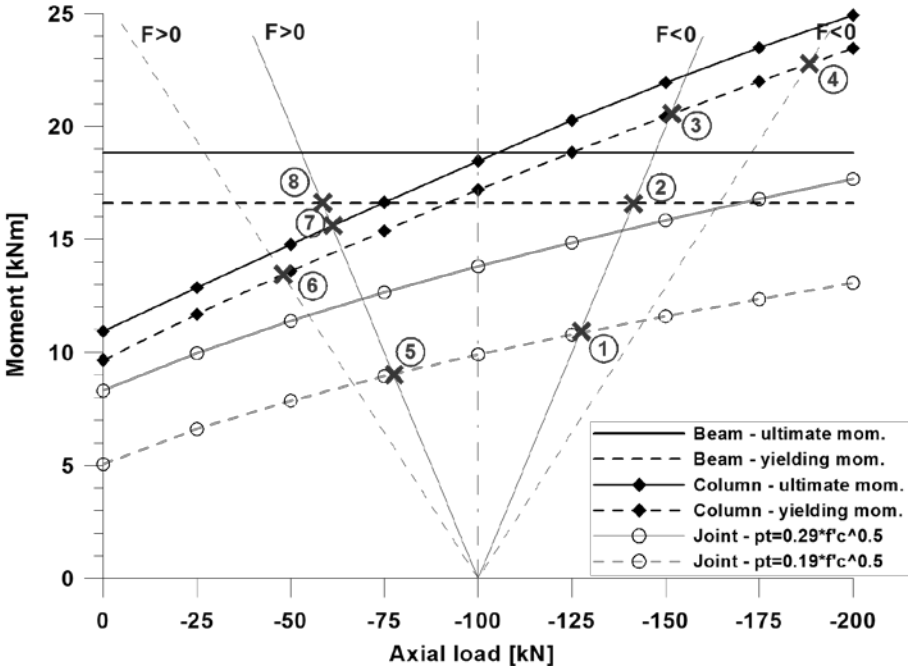
Note:

In the case of the exterior joint shown as an example in Figure C5.20, a shear hinge mechanism with extensive damage of the joint before any hinging of beams or columns was expected and predicted, using a proper demand curve (refer to the table in Figure C5.20) and later confirmed by the experimental tests.

However, as anticipated, the order and "distance" of the events strongly depend on the assumption on the axial load demand curve.

If a constant axial load curve is used (in this case N = -100 kN as shown in Figure C5.20), as is often selected in experimental tests and analytical assessment methodology, only a relatively small increase in the joint strengthening would appear necessary for the retrofit intervention.

However, in reality such a strengthening solution would lead to the formation of a column hinging before any beam hinging. This would possibly result in the development of a soft-storey mechanism in spite of the (generally quite expensive and invasive) retrofit intervention already implemented.



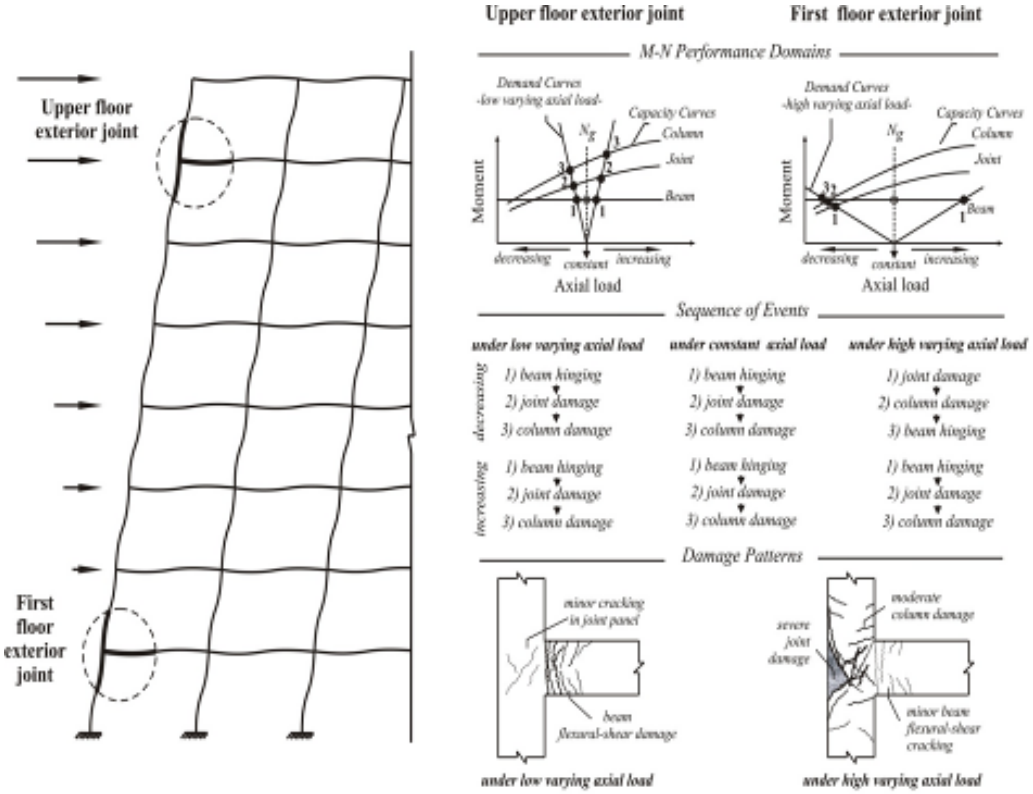
Specimen T1 (as-built)			
Type of lateral force	N°	Event	Lateral force [kN]
Open joint F<0	1	Joint cracking and deterioration starting $p_t = 0.19\sqrt{f_c'}$	-10.94
	2	Beam yielding	-16.59
	3	Upper column yielding	-20.50
	4	Lower column yielding	-22.75
Close joint F>0	5	Joint failure	9.37
	6	Lower column yielding	13.50
	7	Upper column yielding	14.50
	8	Beam yielding	16.59

Figure C5.20: Example of evaluation of hierarchy of strengths and sequence of events: moment-axial load, M-N, performance domain for an exterior beam-column joint in as-built configuration, (after Pampanin et al., 2007)

C5.6.2 Effect of varying axial load on joint capacity

The capacity of a beam-column joint, particularly when characterised by poor detailing and lack of transverse reinforcement as typically found in older buildings, is strongly affected by the variation of the axial load. This was anticipated above when introducing principal stresses instead of nominal shear stress as a more realistic damage indicator.

Therefore, appropriate demand curves for beam-column joint systems should account for the variation of axial load due to the lateral sway mechanism, for either opening and closing of the joint (refer to Figure C5.21). Otherwise, incorrect and non-conservative assessment of the sequence of events can result, which can lead to inadequate – and not necessarily conservative – design of any retrofit intervention.



(a) Laterally loaded frame **(b) Hierarchy of strength and sequence of events for two types of exterior joints**

Figure C5.21: Variation of axial load due to frame sway mechanism and its effects on the hierarchy of strength of beam-column joint subassemblies

Note:

Most of the experimental cyclic tests on joint subassemblies (as well as column-to-foundation connections) are carried out, for simplicity, under a constant axial load regime in the column/joint.

While this simplified testing procedure is not expected to have a substantial effect on the behaviour of well-designed specimens, in the case of poorly detailed subassemblies the effect on damage level and mechanisms could be significant.

In general, the axial load on a column can be expressed as:

$$N = N_{\mathrm{g}} \pm \alpha\, F \qquad \ldots\text{C5.43}$$

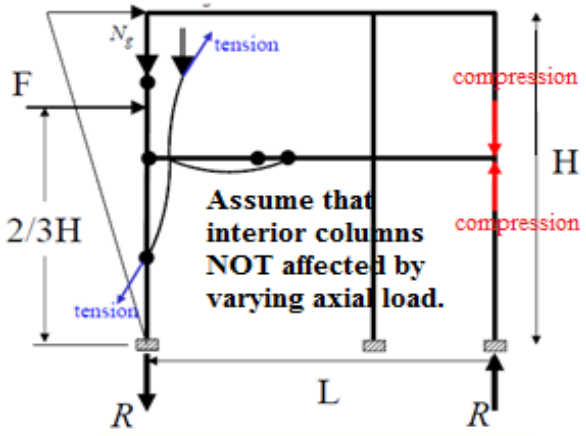
where:

N_{g} = the axial load due to gravity load
F = the lateral force (base shear capacity), and

α depends on the global geometry of the building (height and total bay length, L, as shown in Figure C5.22).

Such variation of axial load due to the seismic action can be substantial for exterior beam-column joints. It can be 30-50% or higher, with a further increase when considering bidirectional loading.

On the other hand, as a first approximation (especially if there are only two or three bays) the variation of axial load in interior beam-column joints can either be neglected or assumed to be in the order of 10-20%.



$$F\left(\frac{2}{3}H\right) = RL \Rightarrow R = \frac{2}{3}\frac{H}{L}F \therefore N = N_g \pm \underbrace{\frac{2}{3}\frac{H}{L}}_{\alpha}F$$

Figure C5.22: Example of evaluation of variation of axial load in a frame

C5.6.3 Upper and lower bounds of base shear capacity and force-displacement curves

C5.6.3.1 General

Once the hierarchy of strength and sequence of events of all the beam-column joint subassemblies within a frame have been evaluated, the global mechanism of the frame can be analysed.

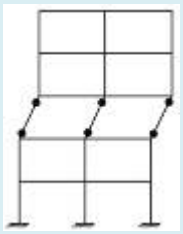
In general, as shown in Table C5.8, upper and lower bounds of the lateral load capacity (i.e. base shear or overturning moment) will be given by a soft-storey mechanism and a beam sway mechanism respectively. Any mixed sidesway mechanisms, including possible shear hinging in the joint, would provide an in-between capacity curve.

Note:

The overall Overturning Moment (OTM) in a frame is given by the sum of the moments at the column bases and the contribution of the axial load variation in the columns "collected" from the shear contribution of the beam. Therefore, each mixed mechanism can be evaluated by estimating the moment in each beam resulting from the equilibrium of the sub-assembly, as follows:

$$OTM = \sum_i M_{\text{coli}} + \left(\sum_x V_{\text{end beam,x}}\right)L \qquad \ldots\text{C5.44}$$

Table C5.8: Upper and lower bounds of frame capacity due to column and beam sway mechanisms, and in-between capacity due to mixed sway mechanism

Upper bound	Lower bound	In between
Beam sidesway mechanism	Column sidesway mechanism 	Mixed sidesway mechanism
$OTM,1 = \sum_{i} M_{coli} + \left(\sum_{n} V_{end\ beam,n}\right) L$ $V_{b,1} = \frac{OTM}{H_{eff,beam\ sidesway}}$	$OTM,2 = \sum_{i} M_{coli}$ $V_{b,2} = \frac{OTM}{H_{eff,col\ sidesway}}$	$OTM = \sum_{i} M_{coli} + \left(\sum_{x} V^{*}_{end\ beam,x}\right) L$ $V_{b,3} = \frac{OTM}{H_{eff,mixed\ sidesway}}$

Note:

$\sum_{i} M_{coli}$ = sum of base column moments

$\sum_{n} V_{end\ beam,n}$ = sum of end beam shears for all n levels

L = frame full span

C5.6.3.2 Beam sidesway mechanism

$$OTM,1 = V_{b,1} * H_{eff} = \sum_{i} M_{coli} + \left(\sum_{n} V_{end\ beam,n}\right) L \quad \text{…C5.45}$$

where:

$V_{end\ beam}$ = the additional column axial load due to the beam shear (evaluated as corresponding to maximum flexural capacity).

This provides an upper bound of the lateral load resistance capacity.

C5.6.3.3 Column sway mechanism

$$OTM,2 = \sum_{i} M_{coli} = V_{b},2 * 0.5h$$

where:

$\sum_{i} M_{coli}$ = Sum of Moment of the columns at the base

0.5h = point of contraflexure of one floor …C5.46

This provides a lower bound of the lateral load resistance capacity.

C5.6.3.4 Mixed mechanism

$$OTM, 3 = V_{b,3} * H_{eff} = \sum_i M_{coli} + \left(\sum_x V^*_{end\ beam,x}\right)L \quad \ldots C5.47$$

where:

$V^*_{end\ beam}$ is determined from the minimum value (expressed as equivalent beam moment) between the beam flexural capacities, joint equivalent moments, column flexural capacities, and column shear capacities, depending on strength hierarchy at local level.

This base shear value, corresponding to a mixed mechanism, $V_{b,3}$, should be in between the upper and lower bound determined from a beam sway, $V_{b,1}$, and a column sway, $V_{b,2}$, mechanism respectively.

When combining the information on yielding and ultimate (limit states) drift displacement of the frame corresponding to the most critical mechanism, the global force-displacement curve of this frame can be evaluated as shown in Figure C5.23.

The structure's performance can thus be assessed against any given level of earthquake intensity, using an Acceleration Displacement Response Spectrum (ADRS) approach as described in Section C2.

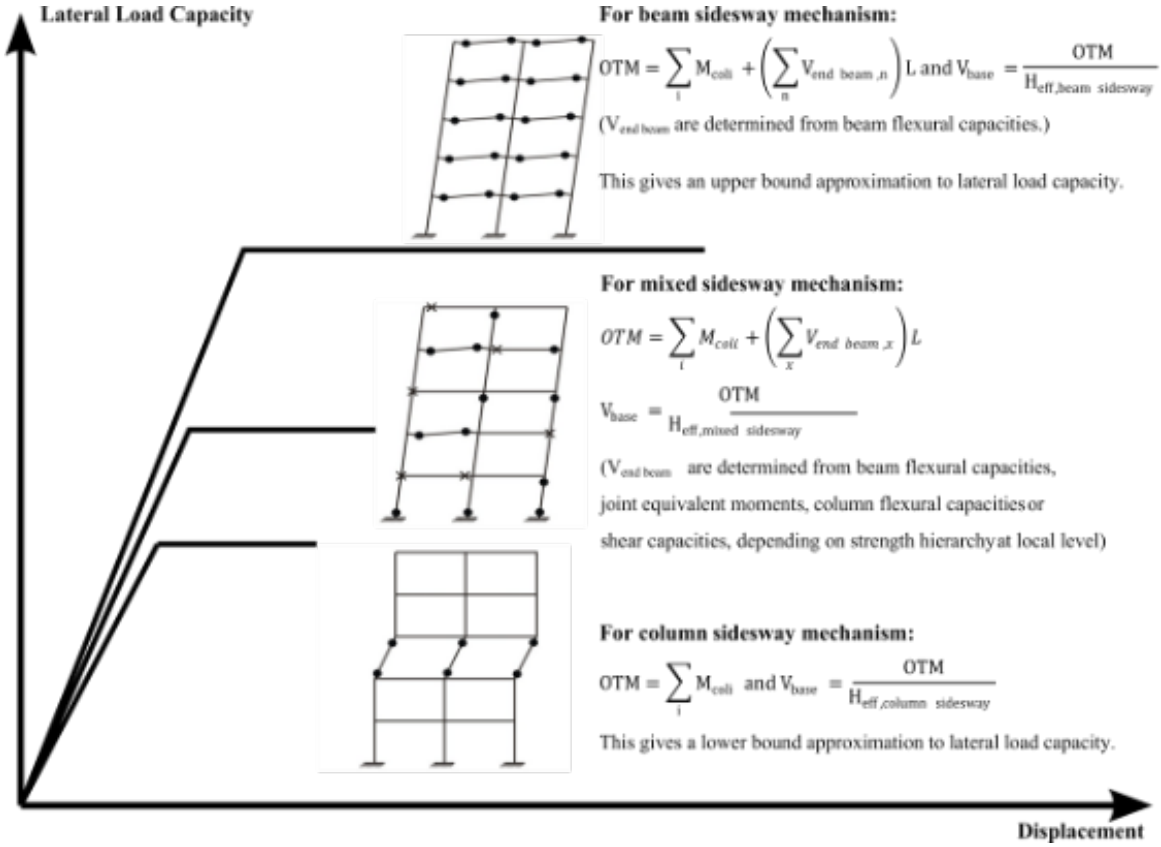


Figure C5.23: Lateral load capacity versus displacement for different global mechanisms

C5.7 Global Capacity of Wall Buildings

C5.7.1 General

The assessment of the overall behaviour of a building's structural system in which seismic resistance has been assigned to reinforced concrete structural walls will probably be less elaborate than that for frame systems.

In the presence of robust walls, the contribution to seismic resistance of other elements with a primary role of supporting gravity loads may often be neglected at a first stage. The detailing of such frame components only needs checking to satisfy any displacement compatibility issues with the overall 3D response (including torsion) of the building system.

In such cases, it is important to check the displacement-drift capacity of non-ductile columns for displacement demand higher than that corresponding to the ULS displacement capacity of the main wall-lateral resisting system (refer to Section C2 for details of this Critical Structural Weakness).

The presence of alternative load paths and overall redundancy characteristics should be checked in order to avoid progressive and catastrophic collapse, as observed in the CTV building after the 22 February 2011 Christchurch earthquake.

Note:

If the contribution of such frame systems to seismic capacity is judged to be more significant or the system needs to rely on their seismic contribution to satisfy seismic performance criteria, the building should be treated as a dual frame-wall building and assessed as outlined in Section C5.8.

C5.7.2 Evaluation approach

The first step is to evaluate the total force-displacement capacity curve of the wall system in each orthogonal direction (i.e. assuming 2D response) as the sum in parallel of all walls contributing in that direction. This is shown in Figure C5.25 with reference to the layout of a wall system shown in Figure C5.24.

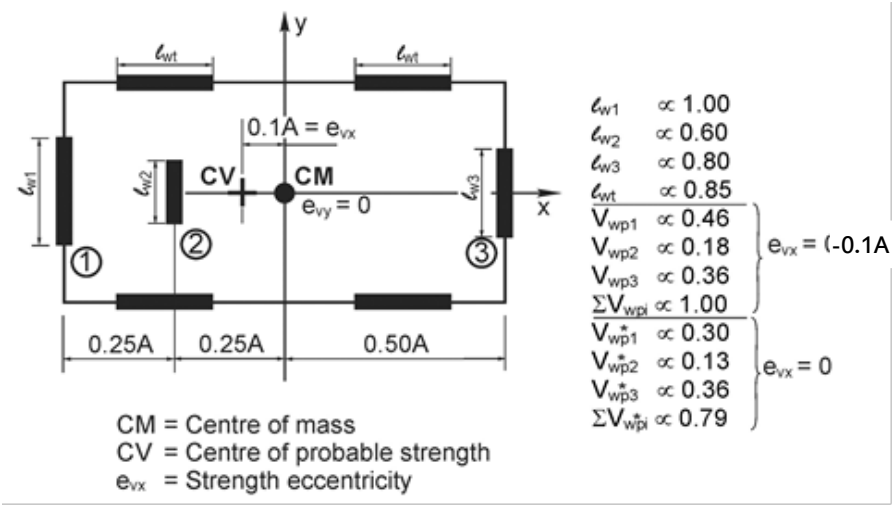


Figure C5.24: (Elastically calculated) torsional effects in a walled building

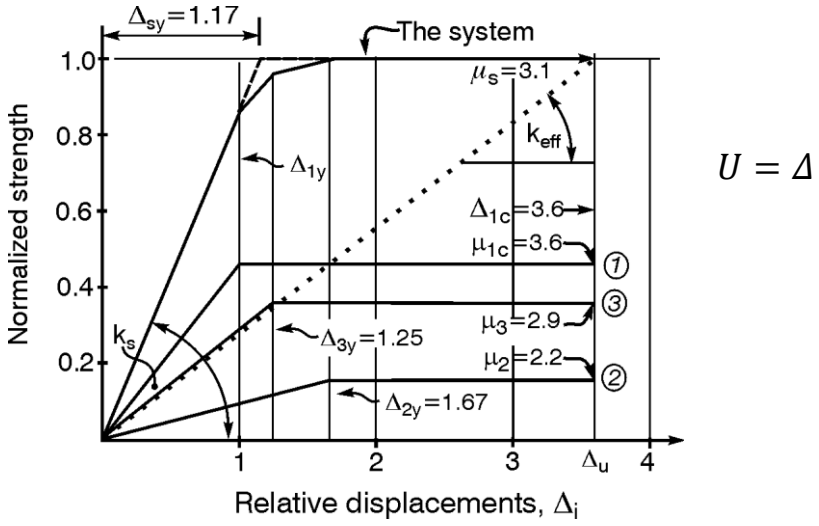


Figure C5.25: Bilinear idealisation of ductile element and system response for a wall building shown in Figure C5.24

Figure C5.25 shows the global capacity curve and the individual contribution of each wall system.

The relationship between ductilities developed in walls with different dimensions and that of the wall system as a whole can be appreciated. As the wall with greatest length will yield first, it is likely that, assuming a flexurally dominated behaviour, the associated displacement capacity of such walls will govern the overall displacement capacity of the system. However, other brittle mechanisms can occur first on individual walls and should be carefully checked.

This procedure is based on the use of a simplified analytical approach where the two orthogonal directions are, at a first stage, considered to be decoupled.

This approximation is more appropriate when dealing with rectangular walls and is acceptable, as a first step, when considering C-shape or T-shape walls with poor connection details in the corner/regions.

When good connection between web and flange are present in T- or C-shaped walls, the actual behaviour of the walls in both longitudinal and transverse directions should be evaluated.

In any case, the 3D response effects should then be accounted for. These include, for example:

- slab coupling effects between walls oriented orthogonally but close to each other, and
- possible response amplifications to the displacement/ductility demand due to inelastic torsional effects (refer to Section C2 for details of procedures to account for inelastic torsional effects).

C5.8 Global Capacity of Dual Frame-Wall Concrete Buildings

C5.8.1 General

In dual systems, elements resisting lateral forces in a given direction of the building may have significantly different behaviour characteristics. Mechanisms associated with their ductile response may also be very different. Typical examples are buildings where lateral forces in different parallel vertical planes are resisted by either ductile frames or ductile walls. Walls forming a service core over the full height of the building are common. They may be assigned to resist a major part of the lateral forces, while primarily gravity load carrying frames may also be required to provide a significant fraction of the required seismic strength.

Regardless of whether elastic or post-yield behaviour is considered, displacement compatibility requirements (Paulay and Priestley, 1992) over the full height of the building need to be considered. Figure C5.26 shows the interaction that may occur between a relatively flexible frame and a wall in a multi-storey building due to the need to achieve displacement compatibility at each level. The presence of a rigid diaphragm, with an ability to transfer significant in-plane dynamically induced floor forces to the different vertical elements, is a prerequisite. Therefore, the examination of diaphragm-wall connections is particularly important (refer to Section C5.5.4 for more details).

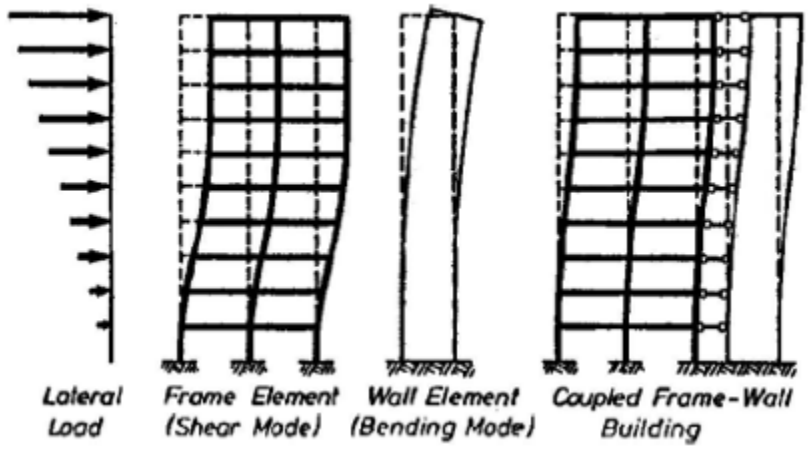


Figure C5.26: Deformation of frame-wall system (Paulay and Priestley, 1992)

During the ductile dynamic response of such dual systems, very different displacement ductility demands may arise for each of the two types of individual lateral resisting system. One purpose of the assessment procedure is to identify the element with the smallest displacement capacity. Wall elements, often representing significant fractions of the probable lateral strength of the system, are typical examples. They control the displacement capacity of the system.

Major advantages of such dual systems are that displacement ductilities imposed on frames are generally very moderate, and that dynamic displacement demands are not sensitive to modal effects, as in the case of frame systems. Moreover, in comparison with frame (-only) or wall (-only) systems, dual systems provide superior drift control. Provided that potential

plastic hinges are detailed for moderate curvature ductility demands, column sway mechanisms in any storey of the frames are acceptable.

The assessment procedure outlined is applicable to any combination of walls and frames, provided that no gross vertical irregularities, such as discontinuities in walls, exist. It is based on displacement-focused or displacement-based treatment of ductile reinforced concrete systems introduced in Paulay and Restrepo (1998); Paulay (2000, 2001b and 2002) and on a redefinition of strength-dependent component stiffness (Paulay, 2001a).

Note:

For more recent information on displacement-based design for dual systems that can be used for the assessment procedure refer to Sullivan et al., 2012.

This enables the same assessment procedure to be carried out for strength and displacement-based performance criteria. The displacement ductility capacity of a dual system needs to be made dependent on the displacement capacity of its critical element.

C5.8.2 Derivation of global force-displacement capacity curve

C5.8.2.1 Assessment approach

As the walls are expected to govern the behaviour of the dual system, both in terms of strength and stiffness, it is recommended to start the assessment of a dual system from the assessment of the wall system(s).

In fact, because the wall remain essentially elastic above the plastic region at the base during ductile system response, their deformations will control that of the overall system. Moreover, in general, the displacement capacity of the walls rather than that of the frames should be expected to control the performance limit state.

Hence, wall displacement capacity should be estimated and compared with the corresponding displacement ductility demands generated in the frames.

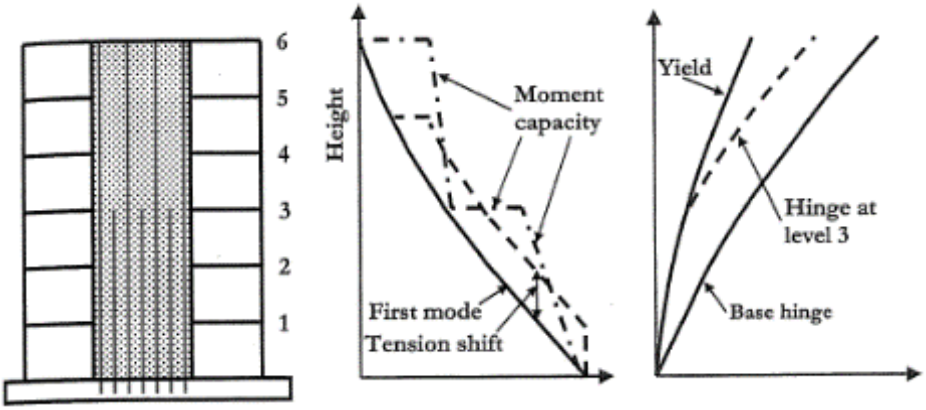
C5.8.2.2 Step by step procedure

Step 1 Estimate the post-elastic mechanism of walls and their contribution to lateral force resistance

The nonlinear mechanism of the walls of a dual system is expected to comprise plastic hinges at the base of each wall. A detailed study of the wall capacity along the height, as outlined in Section C5.5.2.1, is required to verify this.

Based on the procedure presented in this section for single cantilever walls, moment-curvature analyses of the wall cross sections can be computed at each level accounting for the axial load variation and change in longitudinal and transverse reinforcements. The wall flexural strength should be checked against the shear strength to detect premature shear failure along the wall height. This failure is likely to govern the behaviour of walls more than columns.

As shown by the dash/dot line in Figure C5.27, the moment capacity gradually reduces along the height as a consequence of the reduced axial load and longitudinal reinforcement amount.



(a) Structure **(b) Moment profiles** **(c) Displacement profiles**

Figure C5.27: Displacement response of a wall structure (Priestley et al., 2007)

Assuming a typical first-mode distribution of lateral forces (i.e. inverted triangular), determine the distribution of the bending moment up the wall height corresponding to the wall-base flexural strength (the solid line in Figure C5.27(b)).

Determine the extent of the wall region over which the shear stress is such that diagonal cracking is to be expected. Over this region, tension shift effects resulting from diagonal cracking will increase the apparent moment. This influence can be reasonably represented by shifting the moment profile over the affected region up by a distance equal to half the wall length, $l_w/2$ (dashed line in Figure C5.27(b)).

The critical section of the wall can be identified comparing the capacity and demand moment envelope (dash/dot and dashed line in Figure C5.27(b)). If the capacity exceeds the demand at all the levels above the base, such as in the example in Figure C5.27(b), the inelastic response can be assumed as concentrated at the base only. Otherwise, plastic hinging is expected at the level where the demand is higher than the capacity.

Characterise the pushover curve of the single-degree-of-freedom (SDOF) system assuming, in first instance, a cantilever wall scheme with $H = H_{\text{eff}}$.

Based on the probable strength of the examined sections of all walls of the system, quantify the total overturning moment that can be carried by these walls, $M_{\text{w,b}}$ (subsequently referred to as the wall element).

With this evaluation of the overturning moment capacity of the wall element, $M_{\text{w,b}}$, (refer to Figure C5.29(a)), its probable base shear strength can be estimated from:

$$\sum V_{\text{wp}} = M_{\text{w,b}}/H_{\text{eff}} \qquad \text{…C5.48}$$

The effective height of the wall element, H_{eff}, is given by the approximate position of its point of contraflexure Figure C5.29(a). As a first approximation it can be assumed that $H_{\text{eff}} = 0.67H_{\text{w}}$.

When a more slender wall element is used, its probable base strength will be smaller and the point of zero wall moment will be at a lower level, resulting in $H_{\text{eff}} < 0.67H_{\text{w}}$.

While the storey shear strength provided by the frames can be evaluated with a relatively high degree of precision, the likely shear demand on the walls is less certain. This is because walls are significantly more sensitive to differences between estimated and real seismic demands.

Therefore, comparisons of probable wall storey shear strength should be conducted with caution as these are largely dependent on the horizontal shear reinforcement which has been provided.

The displacement capacity at the yielding and ULS conditions can be computed according to Section C5.5.2.2.

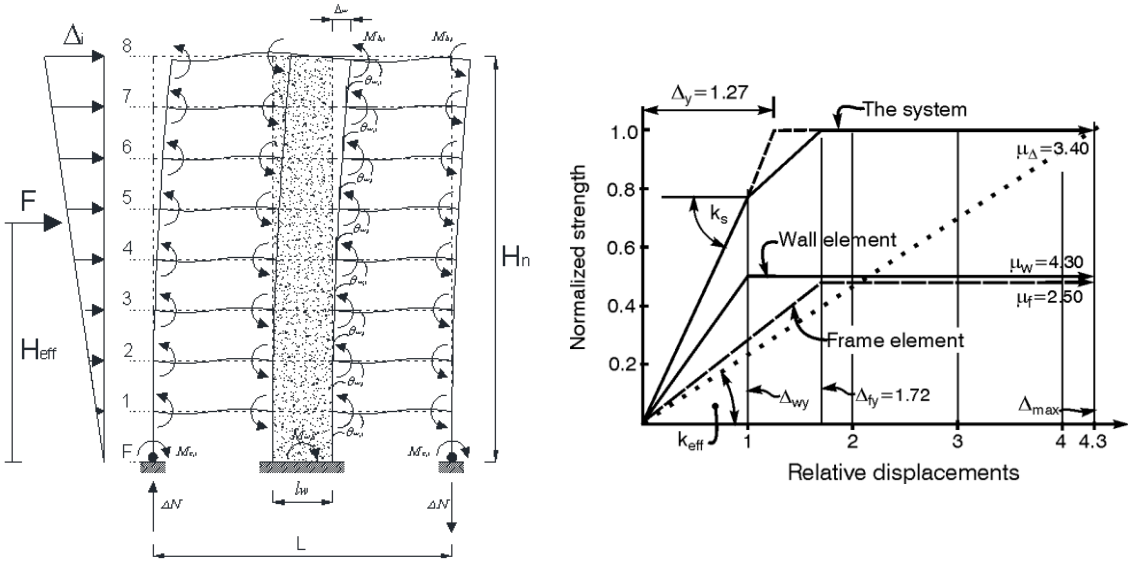
Step 2 Establish the post-elastic mechanism of frames and their contribution to lateral force resistance

Following the procedure outlined in Section C5.5 the probable strength of beams, column and joints are evaluated as well as the hierarchy of strength of column/beam/joint and the overall probable mechanism.

The contribution of the frame members at each floor can therefore be computed imposing the drift corresponding to the yielding and ultimate limit state in the wall on the weaker frame, as illustrated in Figure C5.28.

This allows the computation of the distribution of bending moment, shear and axial load on the frames, and the corresponding actions transmitted to the wall.

To obtain a more refined assessment of the wall behaviour and failure mode, the shear and flexural strength previously calculated in Step 1 can be now compared with a more refined estimation of the shear and bending moment demand determined accounting for the contribution of the frames at each floor.



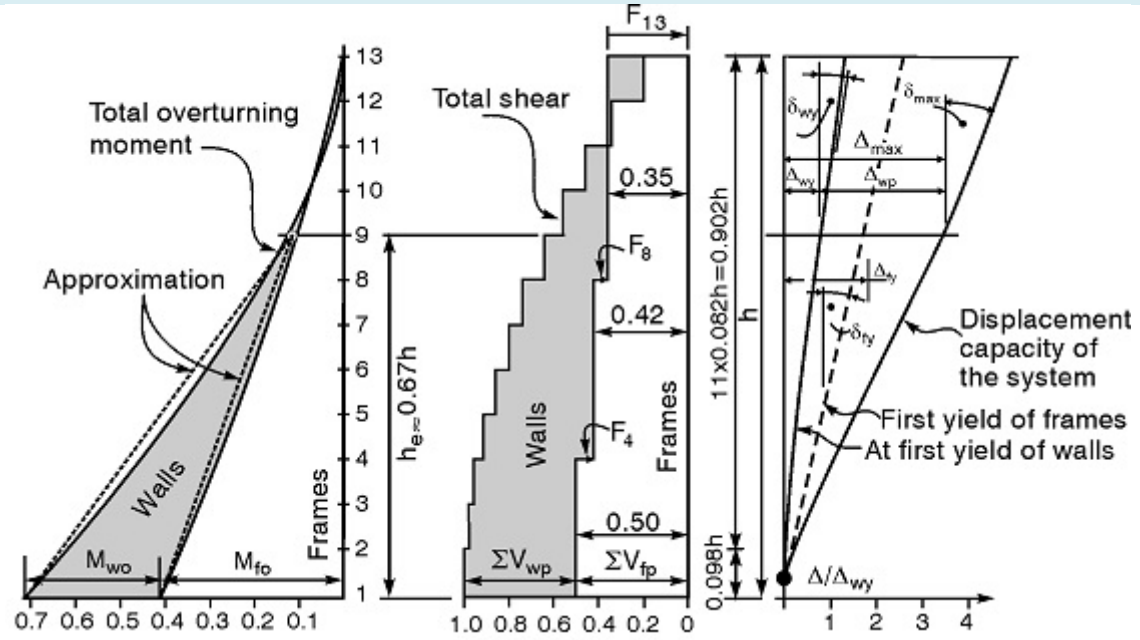
(a) **(b)**

Figure C5.28: Contribution of frame and wall to the global force-displacement capacity curve

Note:

Figures C5.28 and C5.29 illustrate the procedure described at Step 2, with a kinematically admissible sway mechanism. Plastic hinges introduce a total moment of $\sum M_{\mathrm{pi}}$ to the four (equivalent) columns at the level of the beams. This is proportional to the storey shear force, V_{pi}. Note that the overturning moments transmitted from storeys above by means of axial forces in the columns are not shown here.

These figures also illustrate the stepwise estimation of the contribution to total probable overturning moment capacity and storey shear force of both the frames and the walls.



(a) Normalised overturning moments (M/hV_{b}) **(b) Normalised storey shears (V_{s}/V_{b})** **(c) Displacement profiles**

Figure C5.29: Stepwise estimation of the contribution of a frame and a wall element to probable lateral strength and corresponding displacements of a dual system

Step 3 Determine the stiffness and displacement capacity of dual systems

Once the strength contribution of frame members at specific levels of drift has been assessed, the base shear contribution of the frame, wall and resultant dual system can be computed by dividing the total overturning moment by the effective height, H_{eff}, as suggested in Step 1. In the case of dual systems, the effective height of the frame can be assumed to be equal to the effective height of the wall.

Alternatively, and more practically, the base shear of the dual system can be obtained by:

- summing directly (in parallel, thus assuming equal displacement) the pushover curves of the SDOFs of the wall and the frames, or
- estimating the OTM of the dual system considering the contribution of wall and frame elements (refer to Figure C5.28(b)).

Note:

Figure C5.28(b) presents the overall simplified (bilinear modelling) force-displacement capacity curve of the dual system, summarising the procedure discussed in Step 3 and is similar to that shown in Section C2 on mixed ductility systems. Note that this figure represents the expected behaviour of the schematic dual system shown in Figure C5.27 (i.e. a dual system comprising of a central wall and beams coupling to two external columns) as specific assumptions were made to illustrate the simple details of these calculations.

As Figure C5.29(b) shows, an approximately equal contribution (50-50) to the probable base shear strength of the dual system, $V_{\text{dual,p}}$, was found to be provided by the wall and the frame elements.

The relative nominal yield displacements at level H_{e}, were found to be:

- Δ_{wy} = 1.00 displacement units for the wall element, and
- Δ_{fy} = 1.72 displacement units for the frame element.

Therefore, the normalised stiffness of the wall and frame elements are, respectively:

$$k_{\text{w}} = V_{\text{wp}}/\Delta_{\text{wy}} = 0.5/1.0 = 0.5$$

$$k_{\text{f}} = V_{\text{fp}}/\Delta_{\text{fy}}\ 0.5/1.72 = 0.29$$

Hence the relative nominal yield displacement of the dual system is:

$$\Delta_{\text{y}} = V_{\text{dual,p}}/(K_{\text{w}} + K_{\text{f}}) = 1.00/(0.5 + 0.29) = 1.27 \text{ displacement units}$$

The bilinear idealisation of the force-displacement curve for frame, wall and dual system behaviour, shown in Figure C5.29(b), confirms these quantities.

C5.9 Improving the Seismic Performance of Concrete Buildings

Alternative seismic retrofit and strengthening solutions for concrete buildings have been studied and adopted in practical applications ranging from conventional techniques (e.g. using braces, walls, jacketing or infills) to more recent approaches including base isolation, supplemental damping devices or involving advanced materials such as fibre reinforced polymers (FRPs) and shape memory alloys (SMAs). Refer to international guidelines such as fib (2003), EC8-part 3 (2003) FEMA 547 (2006); ASCE-41-13 (2014).

Most of these retrofit techniques have evolved into viable upgrades. However, issues of cost, invasiveness, architectural aesthetics, heritage protection and practical implementation still remain the most challenging aspects of any intervention.

Based on lessons learned from recent major earthquakes and on extensive experimental and analytical data, it is increasingly evident that major – and sometimes controversial – issues can arise in, for example:

- deciding whether the retrofit is actually needed and, if so, in what proportions and to what extent
- assessing and predicting the expected seismic response pre- and post-intervention by relying upon alternative analytical/numerical tools and methods
- evaluating the effects of the presence of infills, partitions or general "non-structural" elements on the seismic response of the overall structure, which is more typically and improperly evaluated considering only the "skeleton"
- deciding, counter-intuitively, to "weaken" one or more structural components in order to "strengthen" the whole structure
- adopting a selective upgrading to independently modify strength, stiffness or ductility capacity
- relying upon the deformation capacity of an under-designed member to comply with the displacement compatibility issues imposed by the overall structure, and/or
- defining a desired or acceptable level of damage that the retrofit structure should sustain after a given seismic event: i.e. targeting a specific performance level after the retrofit.

Regardless of what technical solution is adopted, the efficiency of a retrofit strategy on a reinforced concrete building depends strongly on a proper assessment of the internal hierarchy of strength as well as on the expected sequence of events and damage/failure mechanisms within:

- a frame system (i.e. shear damage and failure in the joint region, flexural hinging or shear failure in beam and column elements, or
- a wall system (i.e. sliding, flexural or shear failure, lateral instability, etc.), or
- a combination of these (dual system).

Following a conceptually similar procedure included in these guidelines, and in particular the SLaMA method, the overall lateral force vs. displacement curve of the building system can be computed before and after alternative retrofit interventions and the performance point of the structure under different earthquake intensity computed, including the new level of *%NBS* achievable when improving the behaviour of individual elements.

This approach allows to gain a direct appreciation of the incremental benefits achievable when implementing specific retrofit interventions or combination of them.

The retrofit strategy can follow a selective intervention, i.e. strength-only, ductility-only, stiffness-only, as well as selective weakening, or a combination of the above.

An overview of alternative performance-based retrofit strategies and technical solutions for Reinforced Concrete buildings, developed and/or refined in the past decade few years as part of the multi-year research project "Retrofit Solutions for NZ multi-storey Buildings", funded by the FRST (Foundation of Research Science and Technology from 2004-2010) can be found in Pampanin, 2009. Pampanin et al., 2010).

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Appendix C5A History of New Zealand Concrete Design Standards and Code-based Reinforcing Requirements

C5A.1 Introduction

This appendix provides a historical overview of New Zealand's concrete design standards. It also summarises the history of the country's code-based reinforcement requirements for:

- beams
- columns
- beam-column joints, and
- walls.

C5A.2 Evolution of Concrete Design Standards

The following table sets out key milestones in the development of New Zealand concrete design standards, from pre-1957 to the present day.

Table C5A.1: Summary of key milestones in the evolution of New Zealand concrete design standards (modified after Fenwick and MacRae, 2011)

Period	Loading Standard	Concrete Standard	Major changes
Pre-1957	1935 Model Bylaws	No seismic provisions	While there were no specific seismic requirements, 135 degree hooks were already shown for stirrups in RC construction (clause 409). Maximum spacing of stirrups was 2/3 of the internal lever arm (clause 616). Development of plain round longitudinal bars was often by 180 degree hooks.
1957-1964	NZSS 95 - Pt IV Basic Loads to be used and methods of application (1955)	UK concrete Code of Practice, CP114:1957 (No seismic provisions) and NZSS 95, Pt V (1939)	Section properties of members were permitted to be based on gross sections, transformed un-cracked sections, or transformed cracked sections (Fenwick and MacRae, 2009).
1964-1968/71	NZSS 1900 Basic Design Loads Chapter 8 (1965)	Design and Construction, Concrete, Chapter 9.3, 1964 (No seismic provisions)	Essentially, no seismic details were specified. It is likely that reinforcement was inadequately anchored for seismic actions, particularly in columns. Plain round bars were used extensively during this period.
1968/71-1982	Ministry of Works Code of Practice: 1968	Ministry of Works Code of Practice: 1968	Ultimate Limit State (ULS)/Limit State Design (LSD) recommended. Detailing requirements introduced for (i) beam-column joints; (ii) column confinement. **Capacity design** introduced between beams and columns (though no allowance for beam overstrength due to slab reinforcement contribution).

Period	Loading Standard	Concrete Standard	Major changes
	NZ4203:1976	ACI 318:1971 or provisional NZ Concrete Standard, NZS 3101:1970	Ultimate Strength Design used Strength Reduction Factors of 0.9 for beams, 0.75 for confined columns and 0.7 for unconfined columns. Member stiffness for seismic analysis recommended as 75% gross section stiffness. Provisions for detailing potential plastic hinge regions introduced: • some shear reinforcement to resist the gravity induced shear and the shear corresponding to flexural strength in the potential plastic hinge region • lapping of bars in specified potential plastic hinge regions not permitted • some column confinement required where axial load ratio bigger than 40% N_b (balanced condition). Capacity design required to ensure sum of column strengths greater than the sum of beam strengths (with no minimum ratio).
1982-1995	NZS 4203:1984	NZS 3101:1982	Modifications to strength reduction factors: 0.9 for flexure in beams and confined columns; 0.7 for unconfined column with axial load higher than $0.1A_g f'_c$; and 0.9 for zero axial load (clause 4.3.1) Member stiffness 0.5 times the gross section stiffness for beams and 1.0 for columns (clause C3.5.5.1) **Detailing** • Confinement of all potential column plastic hinges required, depending on the maximum design axial load level in the column due to the gravity and earthquake actions (clause 6.5.4.3). It was greater than in the previous standards. • lapped bars not permitted at floor levels in columns where there was a possibility of yielding • shear reinforcement requirements in plastic hinge zones more conservative • specific anti-buckling bars in potential plastic hinge regions • joint shear reinforcement development requirements and reinforcing increased • column ties anchored by 135 degrees in cover concrete • beam bars in external joints likely to be bent away from the joint core • columns not designed for earthquake with ϕ=0.7 were permitted to have 6 mm reinforcement at spacing no greater than (i) the minimum column cross sectional dimension, (ii) 16 times the longitudinal diameter. **Capacity design** Capacity design requirements • Over-strength moments in beams were taken as 1.25 or 1.4 times the ideal flexural strength of beams with grade 275 and 380 steel respectively (clause C3.5.1.3).

Period	Loading Standard	Concrete Standard	Major changes
			• Design for a Strong Column Weak-Beam frame mechanism was specified in the commentary (refer to NZS 3101:1982, Appendix C3A). This encouraged potential primary plastic regions to be in the beams, except at the column bases. To obtain the column design actions for flexure, shear and axial force, this included considering: – the maximum beam overstrength moments that could be applied to a joint which affected the corresponding static column demands – changes in distribution of column moments due to higher elastic and inelastic mode behaviour, with a dynamic magnification factor – bi-axial moments on columns which were part of two orthogonal frames, and – effects of beams yielding simultaneously over the frame. The required minimum ratio of the sum of the nominal column flexural strengths to the sum of the nominal beam flexural strengths at beam-column joint centreline in one way frames ranged from 1.6 to 2.4. In many cases the minimum ratios were exceeded as the flexural strengths of the column changed between the top and bottom of the joint zone; and for practical purposes the same longitudinal reinforcement was used in the column on each side of the joint zone. This method of designing columns for seismic actions was adopted into NZS 3101:1995 and retained with minor modifications in NZS 3101:2006. An effective width of floor slab (usually 2 to 4 times the depth of the slab measured from the column faces) was assumed to contribute to beam overstrength (clause 6.5.3.2 (e)), which was smaller than that in later standards. **Diaphragm Design** (refer to Section 10.5.6). Floors are designed for the smaller of the maximum forces that could be resisted by the lateral force system, or for the forces from the “parts and portions” section of the loadings standard. Nominal requirements were given for reinforcement to tie the floor into the building and for the use of precast flooring elements.
1995-2006	NZS 4203:1992	NZS 3101:1995	Ultimate Strength Design used. Building Classifications (4.4.1) are: • elastically responding • limited ductile, and • ductile. **Strength reduction factor** The strength reduction factor for flexure in beams and flexure and axial load in columns was 0.85. (The option of using a nominally unconfined column with a strength reduction factor of 0.7 was removed – clause 3.4.2.2.) The maximum ductility was set as 6 for concrete structures. This overrode the larger values permitted by NZS 4203:1992.

Period	Loading Standard	Concrete Standard	Major changes
			Member stiffness Recommended section stiffness for seismic analysis was 0.4 times the gross section stiffness for rectangular beams and 0.35 for T and L beams. For columns the value varied from 0.4I g for an axial tension of ratio ($N^*/(A_g f'_c)$) of -0.05, 0.6I g at a ratio of 0.8, with interpolation for intermediate axial load ratios (clause C3.4.3.3). Bay elongation effects (i.e. elongation of plastic hinges in the beams pushing the columns apart). Requirements for the minimum length of support ledges for precast floor components to minimise the possibility of units supported on small ledges and/or on cover concrete (clause 4.3.6.4). Effective width of slab to contribute to beam moment flexural strength was increased and assumed to be the same in both loading directions (clause 8.5.3.3). Effective anchorage of slab reinforcement required (clause 4.3.6.6). Considerations were made for increase in shear force in the first storey columns and the formation of a plastic hinge forming in the columns adjacent to the first level beams (although these are not likely to govern) (Fenwick and MacRae, 2009). **Details** Confinement of columns increased for columns with a high axial load (refer to Section 7.5) Confinement for gravity columns, which were not designed to resist seismic actions, was required (clause 8.4.7). Here, among other requirements, the spacing of transverse steel is no greater than (i) one third the minimum column cross sectional dimension, (ii) 10 times the longitudinal bar diameter. Beam-column joint reinforcement requirements revised and reduced compared with the 1982 edition (clause 11.3.7) Minimum seating lengths for precast floor components after reasonable allowance for construction tolerances were set as the larger of 1/180 of the clear span or 50 mm for solid slabs or hollow-core units and 75 mm for ribbed members (clause 4.3.6.4) Stairs consider the seating lengths of NZS 4203:1992 (clause 4.4.13.2)
2006-	NZS 1170.5: 2004	NZD3101:2006	**Building classifications** For consistency with NZS 1170.5:2004 three classifications were defined for buildings. These relate to the value of the structural ductility factor used to determine the seismic design actions. They are: • nominally ductile, using a design ductility of 1.25, • limited ductile, and • ductile buildings. Three classifications of potential plastic regions were defined. Each of these have different detailing requirements and inelastic capacities (clause 2.6.1.3).

Period	Loading Standard	Concrete Standard	Major changes
			They are: • nominally ductile plastic regions • limited ductile plastic regions, and • ductile plastic regions. There is no direct connection between the type of plastic region and classification of a building. • Design of brittle elements is excluded from this standard. • Values for structural ductility factor of less than 1.25 are not given. • S_p values given in NZS 1170.5:2004 were replaced by 0.9 for a structural ductility factor, μ, of 1.25, and 0.7 for a structural ductility factor of 3 or more, with linear interpolation between these limits (clause 2.6.2.2). **Materials** Welded wire fabric, with a strain capacity less than 10%, is permitted only in situations where it will not yield in ULS shaking or when, if it does yield or rupture, the integrity of the structure is not affected (clause 5.3.2.7). **Member stiffness** Minor revisions were made to the section stiffness where a high grade reinforcement was used (clause C6.9.1). **Capacity design** (clause 2.6.5) Contribution of prestressed floor components to overstrength of beams is considered (clause 9.4.1.6.2). The difference in effective widths of floor slabs contributing to nominal negative moment flexural strength of beams and to overstrength of beams is considered (clauses 9.4.1.6.1 and 9.4.1.6.2). Two methods are permitted for assessing capacity design actions in columns: • The first method is based on the one contained in NZS 3101:1995 Appendix A with modifications to consider bi-axial actions more directly and to allow for the effects of elongation of beams on plastic hinge locations. In this method, each column above the primary plastic hinge located at its base of the column is proportioned and detailed with the aim of minimising inelastic deformation that may occur (Method A in Appendix D, clause D3.2 in the NZS 3101:2006). • The second method permits a limited number of potential plastic hinges in the columns provided the remaining columns have sufficient nominal strength to ensure that the storey column sway shear strength exceeds the storey beam sway shear strength in each storey by a nominated margin. The beam-sway storey shear strength is calculated assuming overstrength actions are sustained in all the potential plastic regions associated with the storey being considered (refer to Appendix D, clause D3.3 in the NZS 3101:2006). This method has more restrictions on the lap positions of longitudinal bars and requiring more confinement reinforcement than the first method.

Period	Loading Standard	Concrete Standard	Major changes
			The significance of elongation of plastic hinges in beams on the actions in columns is recognised. In particular, elongation can cause plastic hinges, which are not identified in standard analyses, to form in columns immediately above or below the first elevated level. This can increase the shear forces induced in the columns. However, as the requirement for confinement reinforcement is generally more critical than shear reinforcement this is unlikely to be critical for the shear strength of these columns (refer to 10.4.7.1.2, B8.4, C2.6.1.3.3, C5.3.2, C10.4.6.6, C10.4.7.2.1 in the NZS 3101:2006). In calculating overstrength actions in beams, allowance needs to be made for the possible material strengths and the increase in stress that may be sustained due to strain hardening. Strain levels are much higher in overstrength conditions than in normal ultimate strength design conditions. As strain levels increase the width of floor slab that acts with a beam increases. Consequently a greater width of slab needs to be assumed to contribute to overstrength than to design strength. This effect is recognised in the NZS 3101:2006 (clauses 9.4.1.6.1 and 9.4.1.6.2) but it was not recognised in earlier standards. Precast prestressed floor units in a floor slab, which span past potential plastic hinges in a beam, can make a very significant difference to the overstrength capacity of plastic hinges. A method of assessing the strength due to this source is given in the Standard (clause 9.4.1.6.2). **Strength design** Primary plastic hinges detailed in terms of likely ULS inelastic demands. These demands are written in terms section curvature for a specified plastic hinge length, which is similar to specifying a plastic rotation (refer to clause 2.6.1). **Serviceability limit state (SLS) with earthquake** New requirements for *fully ductile* (but not *nominal* or *limited ductile* structures) (clause 2.6.3.1). The structural ductility that can be used in the ULS is limited to 6 for buildings of normal importance; and in some cases a lower value is required (clause 2.6.1.2d). For the SLS a structural ductility factor of 1 is required for SLS1, but a value of 2 may be used for SLS2 (clause 2.6.2.3.1). However, SLS2 is only applied to buildings of high importance (NZS 1170:2004, clause 5, 2.1.4). Clause 2.6.3.1 requires either that: • the serviceability design strength is equal to, or exceeds, the serviceability design actions, or • analysis shows that crack widths and deflections remaining after a SLS earthquake are acceptable considering the effect of inelastic deformation caused by moment redistribution and other shake down effects associated with repeated inelastic displacements during an earthquake. Strength requirements for the SLS are related to the average strength of structural sections. This is taken as the nominal strength with a strength reduction factor of 1.1 (clause 2.6.3.2) to correspond to average material strengths.

Period	Loading Standard	Concrete Standard	Major changes
			Diaphragm Design Similar material to NZS 3101:1995 Strut and tie analysis required for forces induced in the diaphragms associated with the ultimate limit-state, or with actions associated with overstrength in potential plastic regions (clause 13.3.3) Floors containing precast prestressed units have special requirements (NZS 3101: 2006 plus Amendment 2) relating to (Fenwick and MacRae, 2009): • limiting the possibility of the floors falling off supports (clause 18.7.4) • limiting the possibility of brittle failure by: – requiring for low friction bearing strips with hollow-core units (clause 18.7.4) – requiring a thin linking slab between a precast unit and a parallel structural element, such as a beam or wall, which may deflect in a vertical direction relative to the precast unit. This is required to prevent the load transfer between the structural elements causing the precast units to fail (clause 18.6.7.2) – specifying requirements for shear strength of precast units in zones where overstrength actions can cause tensile stresses to be induced on the top surface of the precast units. In this situation the shear strength is reduced to a value comparable with a non-prestressed beam of the same dimensions (clause 19.3.11.2.4) – specifying the position where reinforcement connecting the precast unit to the supporting structure is cut off or reduced is based on the capacity of the floor to sustain the negative moments and axial tension. These may be induced in the floor when overstrength actions act at the supports and vertical ground motion induces negative moments in the floor (clause 19.4.3.6) – cautioned against supporting precast units on structural elements that may deform and induce torsional moments as these may lead to torsional failure of the floor unit. This situation can be critical for hollow-core flooring (clause C19.4.3.6).

C5A.3 History of Code-based Reinforcement Requirements for Beams in New Zealand

If structural and/or construction drawings for the building are not available, it may be useful to refer to the New Zealand standards of the time. Appendix C5Esummarises the structural detail requirements for beams according to the NZS 3101:2006 standards from 1970 onwards (1970, 1982, 1995 and 2006).

Figure C5A.1 illustrates the evolution of structural design requirements and detailing layout for beams according to the New Zealand concrete standard from the 1970s onwards.

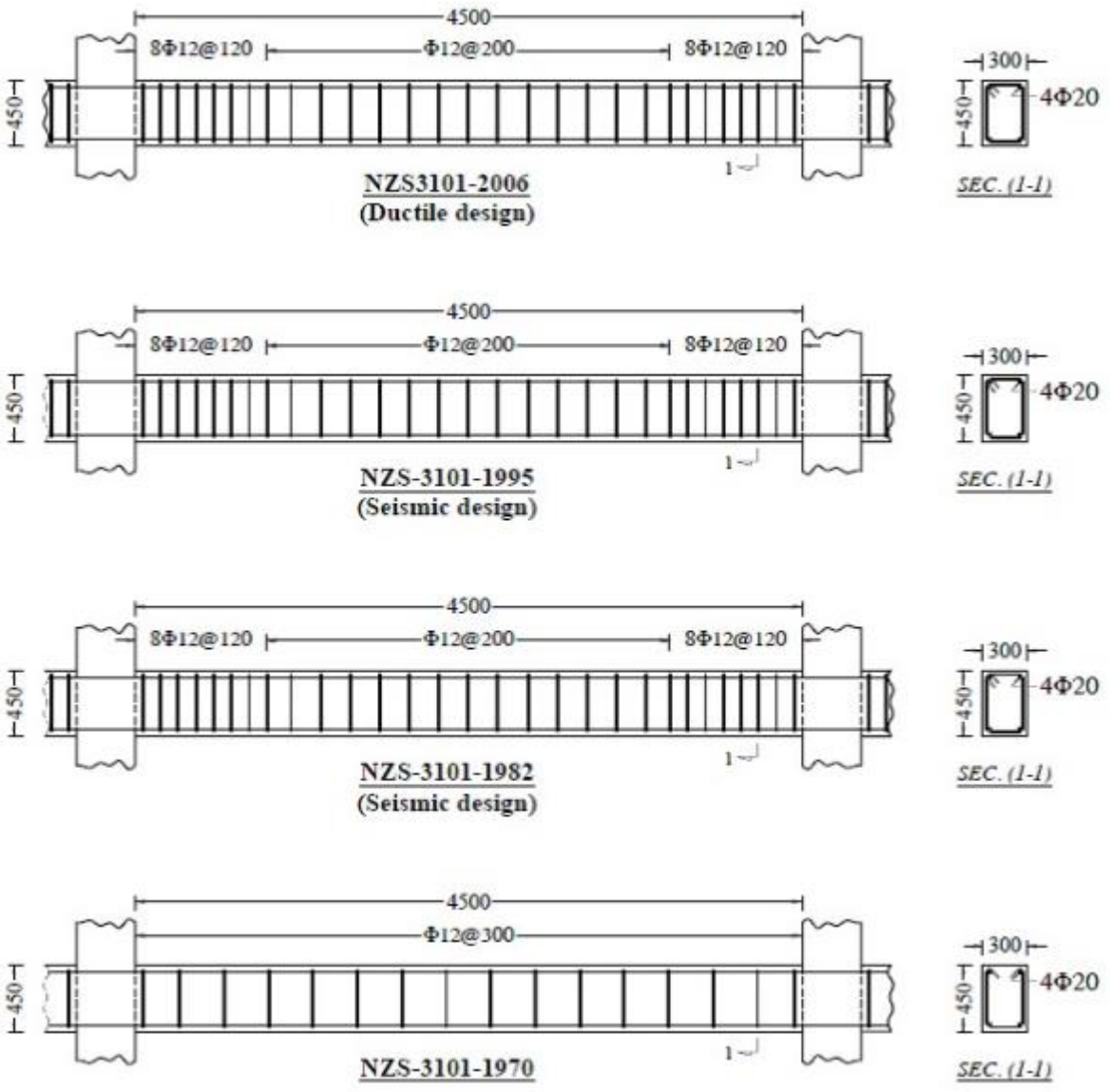


Figure C5A.1: Example of typical beam layouts according to different versions of NZS 3101:2006 (Cuevas et al., 2015)

C5A.4 History of Code-based Reinforcement Requirements for Columns in New Zealand

If structural and/or construction drawings for the building are not available, it may be useful to refer to the New Zealand standards/codes of the time. Appendix C5E compares minimum design/details requirements for columns (either designed for gravity only or for seismic loading) in New Zealand according to NZS 3101:1970, 1982, 1995 and 2006. More information can be found in Niroomandi et al., 2015.

Figures C5A.2 and C5A.3 illustrate the evolution of structural design requirements and detailing layout for gravity column and seismic columns respectively according to the New Zealand concrete standards from the 1970s onwards.

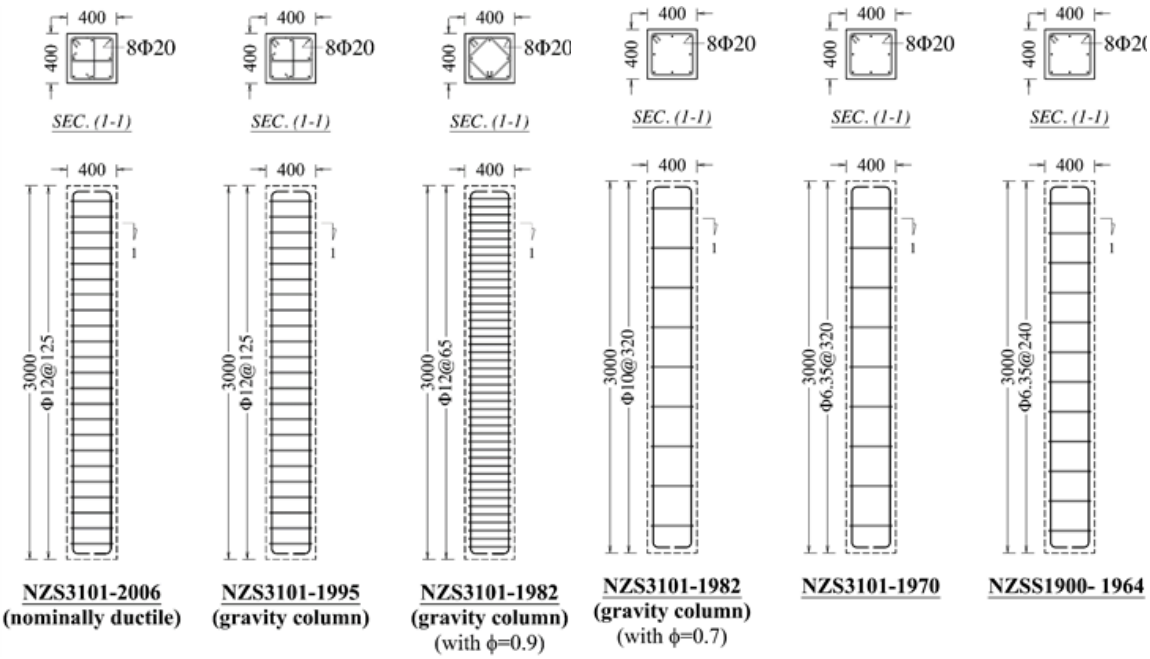


Figure C5A.2: Example of typical gravity column layouts according to different New Zealand concrete standards from the mid-1960s on (Niroomandi et al., 2015)

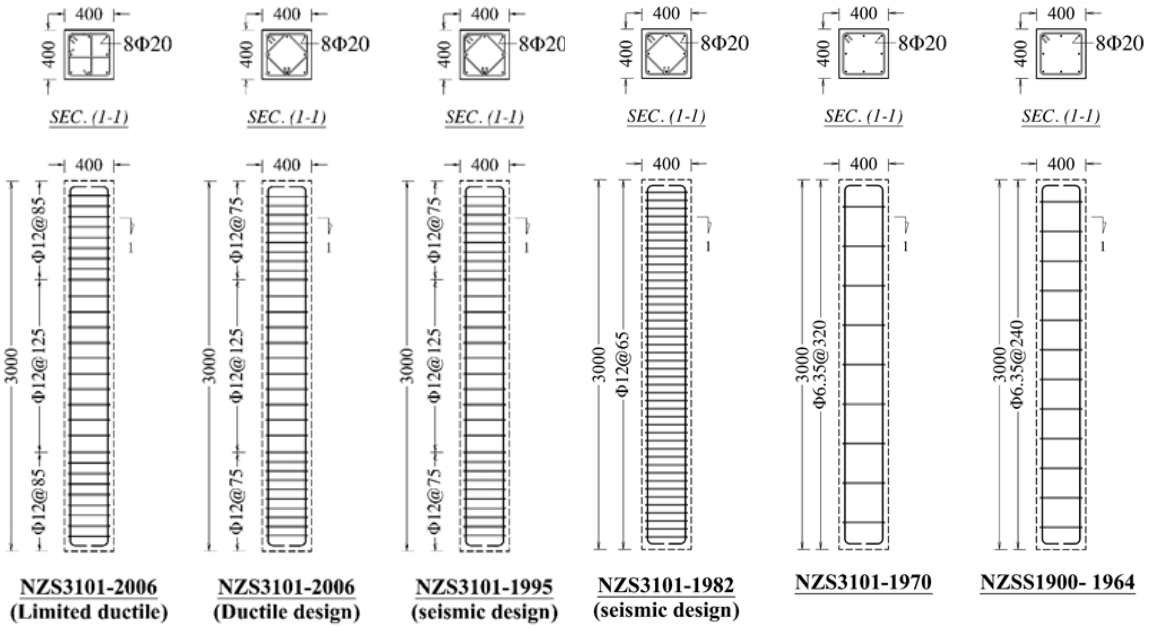


Figure C5A.3: Example of typical column layouts with seismic design according to different New Zealand concrete standards from the mid-1960s onwards (Niroomandi et al., 2015)

The CERC report (CERC, 2012) highlighted the possibility of concrete columns not assumed to form part of the primary seismic system (referred to as gravity only columns) being inadequately detailed to accommodate the displacement demand of the building by the way in which particular clauses in the concrete structures standard NZS 3101:1982 were interpreted by designers when classifying these columns as secondary elements.

Note:

The interpretation of clause 3.5.14 of NZS 3101:1982 may have led some designers to incorrectly classify gravity columns within the general category of secondary structural elements. NZS 3101:1982 provided three options for the level of ductile detailing that was to be used in a secondary element; non-seismic provisions, seismic provisions for limited ductility, and seismic provisions.

Clause 3.5.14 specified which of these provisions should be selected, based on the level of design displacement at which the column reaches its elastic limit. If the column could be shown to remain elastic "when the design loads are derived from the imposed deformations, $\upsilon\Delta$, specified in NZS 4203", the non-seismic provisions could be used. However, the clause was open to interpretation and in practice it appears it was applied in an inconsistent manner. Caution should be applied when making any assumptions as to the design approach that may have been employed in the original design of a building designed to these provisions.

From the mid-1980s it became more common to include the gravity system in the analysis modelling together with the seismic system. If this had been done there would be a higher chance that the secondary elements were designed with some attention to imposed deformations in mind.

In any case, it should be recognised that the imposed deformations in the design codes of the 1980s were much lower than would currently be specified. Furthermore, the deformation demand estimated from modal analysis approach (most common numerical approach used at that time) might have been inaccurate and unconservative.

The following table provides a comparison between the minimum transverse reinforcement spacing requirements of the previous standard (NZSS 1900 Chapter 9.3:1964) and the three levels of ductile detailing available in NZS 3101:1982 and subsequent versions (NZS 3101:1995 and NZS 3101:2006).

Note:

The primary focus of this table is on columns designed to the non-seismic and limited-ductile provisions of the 1982 standard. More detailed information on the evolution of seismic design specifications and requirements for columns in New Zealand from 1970 onwards can be found in Appendix C5E (Niromaandi et al., 2015).

Table C5A.2: Comparison of transverse reinforcement spacing requirements in concrete structures standards (Stirrat et al., 2014)

Design standard	Non-seismic spacing limit	Limited-ductile spacing limit	Ductile spacing limit
NZS 1900 Chapter 9.3:1964	For spirally-wound columns, min. of 75 mm or d_c/6		
NZS 3101:1982	Min. of h, b_c, $16d_b$, $48d_{bt}$	Min. of h, b_c, $10d_b$, $48d_{bt}$	Min. of $h/5$, $b_c/5$, $6d_b$, 200 mm
NZS 3101:1995 and NZS 3101:2006	Min. of $h/3$, $b_c/3$, $10d_b$	Min. of $h/4$, $b_c/4$, $10d_b$	Min. of $h/4$, $b_c/4$, $6d_b$

While the requirements for shear, anti-buckling and confinement lead to adequate transverse reinforcement detailing of the moment resisting frame (MRF) columns in NZS 3101:1982, the 'gravity' columns did not have matching requirements. This is a considerable oversight as the columns, while not specifically considered to contribute to the lateral force-resisting mechanism, still undergo the same displacement demands as the lateral resisting system.

Note:

Even the 1964 standard and the non-seismic provisions in NZS 3101:1995 and 2006 required a fairly close spacing of transverse reinforcement sets. This means that columns designed using the non-seismic or limited-ductile provisions of NZS 3101:1982 are likely to be the primary concern.

It is also worth noting that the requirements in NZS 3101:1982 were more stringent for seismic conditions compared to the non-seismic and limited-ductile conditions.

There are also relevant concerns for secondary columns from other eras (pre-1982 and post-1995). This is even though the investigation by the Department of Building and Housing (now the Ministry of Business, Innovation and Employment) following the Canterbury earthquake sequence was on non-ductile columns in buildings designed to the NZS 3101:1982 (i.e. between 1982 and 1995).

In addition to low quantities of transverse reinforcement, several other characteristics of a column can contribute to its vulnerability in an earthquake. The following list provide indicative-only boundaries for key parameters that may suggest columns are susceptible to non-ductile behaviour:

- Low or inadequate quantities of transverse reinforcement – spacing (e.g. $s > d/2$)

- High axial load demand (e.g. $P/A_g f'_c > 0.3$)
- Low core-to-gross concrete area (e.g. $A_c/A_g < 0.77$)
- Detailing – inadequate lap-splice length, lap splice located in potential plastic hinge zone, poor detailing of transverse reinforcement anchorage (e.g. 90 degree bends), welded detailing, lack of support to longitudinal bars
- High inelastic inter-storey drift demand (e.g. drift $> 1.5\%$) Location of column – in location prone to inelastic torsional amplification of displacements; e.g. corner column or column on opposite face to eccentric shear core.

This list is based on available literature and experience as proposed by (Stirrat et al., 2014). However, more experimental and numerical investigations are required to gain more confidence regarding the actual ranges.

C5A.5 History of Code-based Reinforcement Requirements for Beam-column Joints

If structural and/or construction drawings for the building are not available it may be useful to refer to the requirements of the New Zealand standards of the time. Appendix C5E summarises the minimum design requirements for beam-column joint reinforcement and details according to NZS 3101:1970, 1982, 1995 and 2006.

Figure C5A.4 illustrates the evolution of structural design requirements and detailing layout for beams according to these standards.

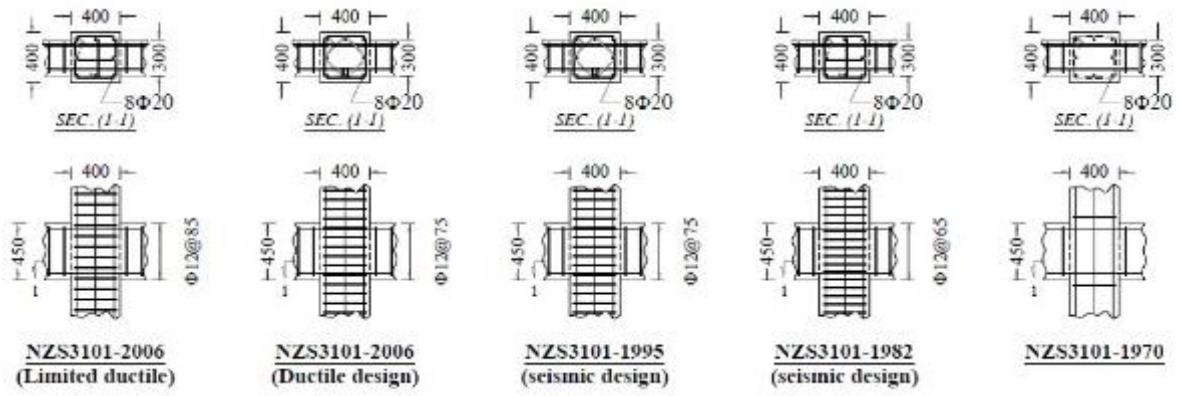
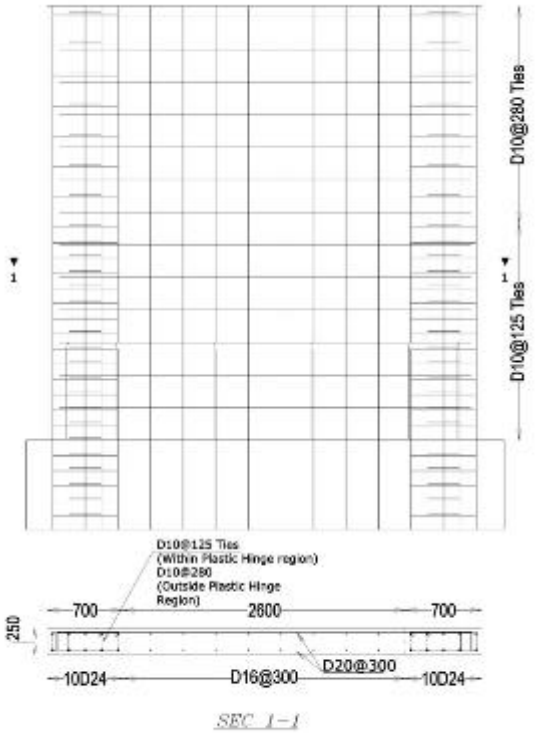


Figure C5A.4: Example of typical beam-column joint layouts according to different New Zealand standards (Cuevas et al., 2015)

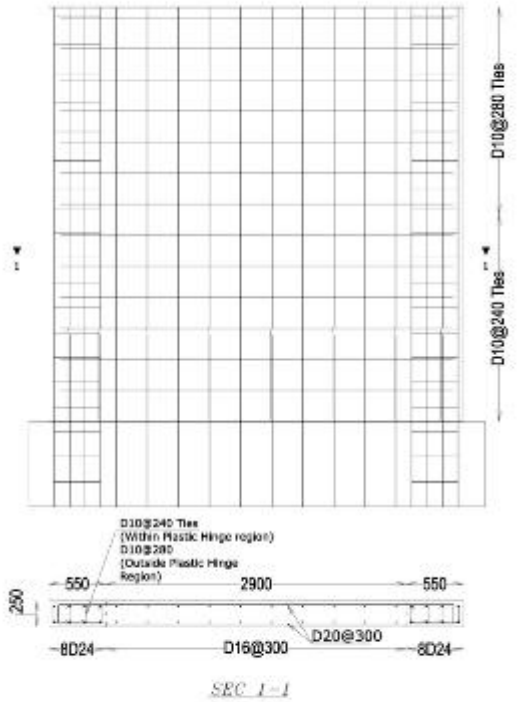
C5A.6 History of Code-Based Reinforcement Requirements for Walls

If structural and/or construction drawings for the building are not available it may be useful to refer to requirements of the New Zealand standards of the time. Refer to Appendix C5E for a comparison of minimum design and detail requirements for walls according to NZS 1900:1964 and NZS 3101:1970, 1982, 1995, 2006. More information can be found in Dashti et al., 2015.

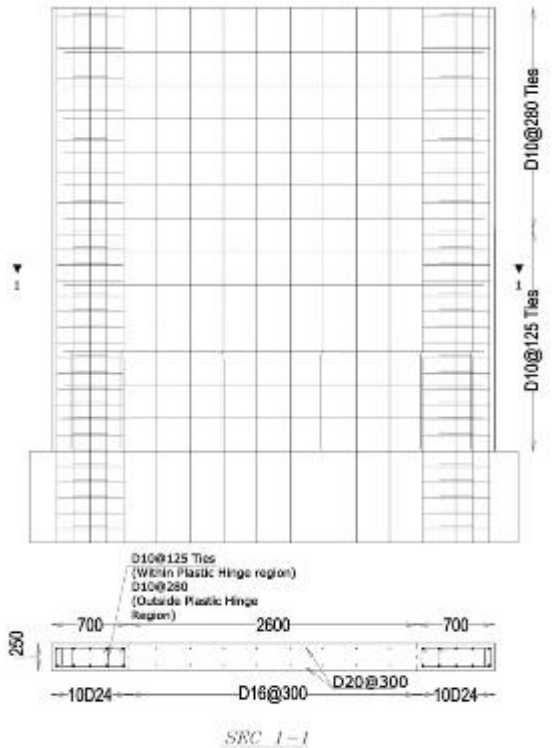
The following figure illustrates an example of the evolution of structural design requirements and detailing layout for shear walls according to these standards.



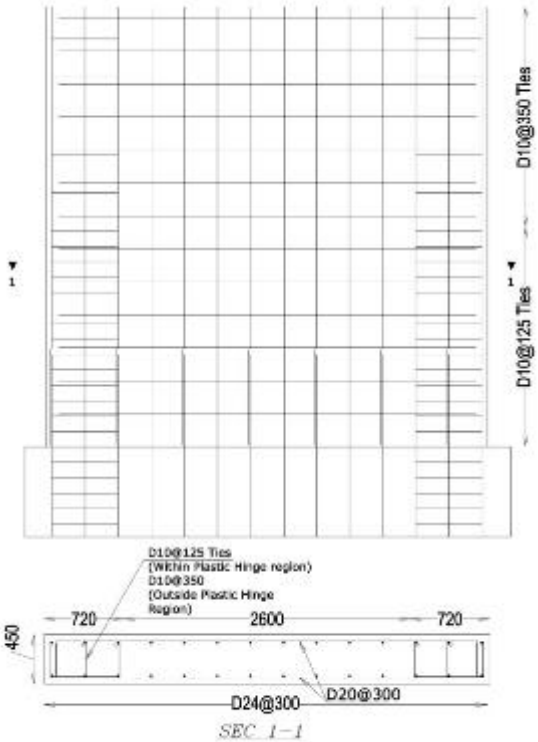
NZS3101: 2006 DPR



NZS3101: 2006 LDPR



NZS3101: 1995



NZS3101: 1982

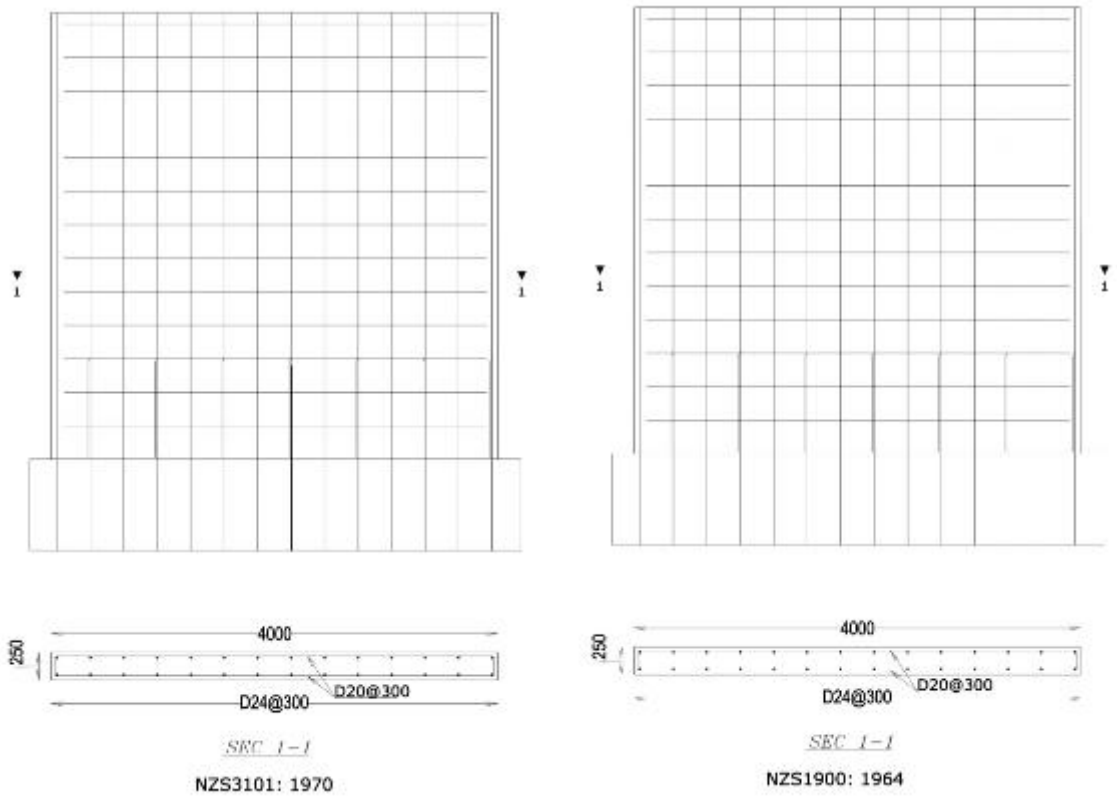


Figure C5A.5: Example of typical reinforcement layouts for shear walls designed according to different New Zealand concrete standards from mid-1960s on (Dashti et al., 2015)

Appendix C5B Historical Concrete Property Requirements, Design Specifications and Requirements for Concrete Strength Testing in New Zealand

This appendix provides tables comparing the:

- concrete property requirements and design specifications from NZS 3101:1970 to NZS 3010:2006, and
- concrete strength tests for quality control from (NZS 3104:1983 to NZS 3104:2003.

Table C5B.1: Comparison of concrete property requirements and design specifications from four generations of New Zealand standards post-1970

Standard / Concrete Property	(1) NZS 3101:1970	NZS 3101:1982	NZS 3101: 1995	NZS 3101: 2006
Specified compressive strength (MPa)	f'_c = 17.2 MPa, 20.7 MPa, 27.6 MPa, 34.5 MPa	20 MPa < f'_c < 55 MPa	17.5 MPa < f'_c < 100 MPa	25 MPa ≤ f'_c < 100 MPa 25 MPa ≤ f'_c < 75 MPa (for ductile elements and elements of limited ductility)
Modulus of rupture (MPa)			For normal weight concrete: f_r=0.8 $\sqrt{f'_c}$ For lightweight concrete: • where f_{ct} is specified and the concrete mix designed in accordance with NZS 3152: f_r=0.8 ×1.8 f_{ct} (the value of 1.8 f_{ct} shall not exceed $\sqrt{f'_c}$) • where f_{ct} is not specified, f_r shall be multiplied by 0.75 (for all-light-weight concrete) 0.85 (for sand-light-weight concrete)	• f_r=0.6 λ $\sqrt{f'_c}$ (for the purpose of calculation deflections) λ = 0.85 (normal weight sand, lightweight coarse aggregate) λ = 0.75 (lightweight sand, lightweight coarse aggregate) λ = 1.0 (concrete with no lightweight aggregates) f_r=1.12 f_{ct} (when the indirect tensile strength of concrete, f_{ct} , specified and lightweight concrete is used, but no more than 0.6 λ $\sqrt{f'_c}$) • from testing – modulus of rupture test (AS 1012: Part 11); or – indirect tensile strength test (AS 1012:Part 10)

Direct tensile strength (MPa)				($0.36\sqrt{f'_c}$) or (0.54 × indirect tensile strength obtained from Brazil test according to AS 1012:Part 10)
Elastic Modulus	$E = 0.043\, w^{1.5}\sqrt{f'_c}$ (for $1450 < w^{(2)} < 2500$ kg/m^3)	$E = 0.043\, w^{1.5}\sqrt{f'_c}$ (for $1400 < w < 2500$ kg/m^3) $E=4700\sqrt{f'_c}$ (for normal weight concrete)	$E = (3320\sqrt{f'_c}+6900)\left(\frac{\rho}{2300}\right)^{1.5}$ (for $1400 < \rho < 2500$ kg/m^3) $E=(3320\sqrt{f'_c}+6900)$ (for normal weight concrete)	Testing of plain concrete $E = (3320\sqrt{f'_c}+6900)\left(\frac{\rho}{2300}\right)^{1.5}$ (for $1400 < \rho < 2500$ kg/m^3) $E = (3320\sqrt{f'_c}+6900)$ (for normal weight concrete) $E \geq$ value corresponding to (f'_c+10) MPa (when strain induced action are critical) Note: For the SLS, this value may be used in lieu of above expression.
Poisson ratio			0.2	0.2 (for normal density concrete) Shall be determined (for lightweight concrete)
Coefficient of thermal expansion (/°C)			12×10^{-6}	For concrete of an aggregate type: • Greywacke ($9.5 - 11 \times 10^{-6}$) • Phonolite ($10.0 - 11.0 \times 10^{-6}$) • Basalt ($9.0 - 10.0 \times 10^{-6}$) • Andesite ($7.0 – 9.0 \times 10^{-6}$) The coefficient of thermal expansion may be taken as 12×10^{-6}/°C or determined from suitable test data for other aggregate types. For self-compacting concrete these values shall be increased by 15%.
Shrinkage				The design unrestrained shrinkage strain may be determined by testing to AS 1012 Part 13, or appropriate published values.
Creep				The creep coefficient used for design may be determined by testing to AS 1012 Part 16, or to ASTM C512, or assessed from appropriate published values.

Stress-strain curves				• Assumed to be of curvilinear form defined by recognised simplified equations; or • Determined from suitable test data.
Applicable density range (kg/m^3)				1800 to 2800

Note:

1. Formulas have been converted to metric units.
2. w: weight of concrete.

Table C5B.2: Concrete strength tests for quality control

Standard / Control Tests	NZS 3104:1983	NZS 3104:1991	NZS 3104:2003
Number of test specimens	3 specimens made from one sample of concrete 2 specimens when the number of tests > 20 and the 28-day compressive testing mean has a within-test coefficient of variation of the test series of less than 4%.	Same as NZS 3104:1983	Same as NZS 3104:1983
Frequency of testing	• Ready-mixed concrete: – 1/75 m^3 (up to 15,000 m^3 per annum), with an additional test for every 250 m^3 above 15,000 m^3 – At least 120 tests per annum • Site-mixed concrete: – 1 sample (each day/75 m^3)	Same as NZS 3104:1983	• Ready-mixed concrete: – Same as NZS 3104:1983 – At least 10 tests per month (6 tests per month in the case of plants producing less than 9000 m^3 per annum) • Site-mixed concrete: – Same as NZS 3104:1983

Appendix C5C Historical Reinforcing Steel Properties in New Zealand

C5C.1 General

The first New Zealand standard to regulate the mechanical properties of steel bars for reinforcing concrete is likely to have been NZS 197:1949 (based on BS 785:1938) "Rolled steel bars and hard drawn steel wire". This standard only referred to plain round bars.

Before NZS 197:1949 (BS 785:1938), there was apparently no specific national standard to cover reinforcing steel. However, it can be reasonably assumed that steel reinforcement was regulated by BS 165:1929, which was the previous version of BS 785:1938 used in New Zealand from 1949.

Deformed bars were introduced in 1963 with NZSS 1693:1962 "Deformed steel bars of structural grade for Reinforced Concrete". A 227 MPa (33,000 psi) yield stress steel bar was first introduced and then replaced in 1968 (Amendment 1 of NZSS 1693:1962) by a 275 MPa (40,000 psi yield stress steel bar).

Note:

It can therefore be assumed that plain round bars were used in concrete buildings at least until the mid-1960s. The required development length for plain round bars can be taken as not less than twice that for deformed bars specified in NZS 3101 (2006).

Also note that during cyclic loading the bond degradation for plain round bars is more significant than for deformed bars (Liu and Park, 1998 and 2001; Pampanin et al., 2002). Hence, old structures reinforced with plain round longitudinal bars will show a greater reduction in stiffness during cyclic loading. As a reference value, as part of quasi-static cyclic load tests of beam-column joint subassemblies reinforced by plain round longitudinal bars at the University of Canterbury, the measured lateral displacements were approximately twice those of similar assemblies reinforced by deformed longitudinal bars at similar stages of loading (Liu and Park, 1998 and 2001).

Often plain round bars were terminated with hooks to provide reliable development of the bars, but this was not always the case.

In 1964 another standard relating to deformed steel bars was issued: NZSS 1879:1964 "Hot rolled deformed bars of HY 60 (High yield 60,000 psi) for Reinforced Concrete". This standard introduced a higher yield steel bar with a yield stress of about 414 MPa (60,000 psi). At this stage, there were three standards for steel reinforcing bars: one for plain round bars (NZS 197) and two for deformed bars (NZSS 1693 and NZSS 1879).

Note:

Reinforcing steel from the pile caps of the Thorndon overbridge in Wellington constructed in the 1960s had a measured mean yield strength of 318 MPa with a standard deviation of 19 MPa (Presland, 1999).

In 1972 the old NZS 197 was replaced by a temporary standard NZS 3423P:1972 “Hot rolled plain round steel bars of structural grade for reinforced concrete” but this was only valid for a year. In 1973, all three standards (NZSS 1693:1962, NZSS 1879:1964 and NZS 3423P) were superseded by NZS 3402P:1973 “Hot rolled steel bars for the reinforcement of concrete” which regulated both plain round and deformed bars.

Metric units for steel bars were slowly introduced in 1974 and became the only units used by steel manufacturers from 1976 onwards. Steel grades used at that time were Grade 275 and Grade 380.

In 1989, NZS 3402P was superseded by NZS 3402:1989. This replaced Grades 275 and 380 with new grades, 300 and 430.

In 2001, the current version of the standard for reinforcing steel, AS/NZS 4671:2001, was introduced. Steel grades proposed for New Zealand in this standard are Grade 300E (Earthquake ductility) and Grade 500E.

Table C5C.1 summarises the evolution of these standards, while Tables C5C.2 to C5C.4 in the next section list available diameters for steel reinforcing bars. Also refer to Appendix C5E for a summary of the historical evolution of the mechanical properties of steel reinforcing over different time periods.

Table C5C.1: Evolution of reinforcing steel material standards in New Zealand

<table>
<tr><th colspan="3">1949</th><th>1962</th><th>1964</th><th>1968</th><th>1972</th><th>1973</th><th>1989</th><th>2001</th></tr>
<tr><td colspan="5">NZS 197:1949 (BS 785:1938)
Rolled steel bars and drawn steel wire for concrete reinforcement
(Yield stress varied with diameter, minimum value was 227 MPa, Refer to Table C5D.2)</td><td>NZS 3423P:1972
Hot rolled plain round steel bars of structural grade for reinforced concrete
“Grade” 40,000 psi (275 MPa)</td><td rowspan="4">NZS 3402P: 1973
Hot rolled steel bars for the reinforcement of concrete
Grade 275 MPa
Grade 380 MPa</td><td rowspan="4">NZS 3402: 1989
Steel bars for the reinforce-ment of concrete
Grade 300 MPa
Grade 430 MPa</td><td rowspan="4">AS/NZS 4671: 2001
Steel reinforcing material
Grade 300 MPa
Grade 500 MPa</td></tr>
<tr><td colspan="2"></td><td colspan="2">NZSS 1693:1962
Deformed steel bars of structural grade for reinforced concrete
“Grade” 33000 psi (227 MPa)</td><td colspan="2">NZSS 1693:1962
(Amendment 1:1968)
Deformed steel bars of structural grade for reinforced concrete
“Grade” 40000 psi (275 MPa)</td></tr>
<tr><td colspan="3"></td><td colspan="4">NZS 1879:1964
Hot rolled deformed bars of HY 60 (High Yield 60,000 psi) for reinforced concrete
Grade” 60,000 psi (415 MPa)</td></tr>
</table>

C5C.2 Mechanical Properties of Steel Reinforcing Bars Over Different time Periods

The evolution of standards for the mechanical properties of steel reinforcement bars is summarised in the following tables.

Table C5C.2: Mechanical properties of steel reinforcement bars – pre-1960s

Standard / Steel Property	NZS 197:1949 (BS 785:1938)			
Type of steel	Plain round bar *Mild steel (MS)* *Medium tensile (MT)* *High tensile (HT)*			
Yielding stress	*Bar size (diameter)*	*MS*	*MT*	*HT*
	Up to 1 inch	Not Specified	19.5 tsi (≈270 MPa)	23.0 tsi (≈317 MPa)
	Over 1 to 1½ inch		18.5 tsi (≈255 MPa)	22.0 tsi (≈303 MPa)
	Over 1½ to 2 inch		17.5 tsi (≈241 MPa)	21.0 tsi (≈290 MPa)
	Over 2 to 2½ inch		16.5 tsi (≈227 MPa)	20.0 tsi (≈275 MPa)
	Over 2½ to 3 inch		16.5 tsi (≈227 MPa)	19.0 tsi (≈262 MPa)
Tensile strength		≥ 28 tsi (≈ 386 MPa)	≥ 33 tsi (≈ 455 MPa)	≥ 37 tsi (≈ 510 MPa)
		≤ 33 tsi (≈ 455 MPa)	≤ 38 tsi (≈ 524 MPa)	≤ 43 tsi (≈ 593 MPa)
Elongation at fracture (%)	Up to 1 inch	≥ 20(1)	≥ 18(1)	≥ 18(1)
	Over 1 to 1½ inch	≥ 16(1)	≥ 14(1)	≥ 14(1)
	Under ⅜ inch	≥ 24(2)	≥ 22(2)	≥ 22(2)

Note:

psi = pounds per square inch

tsi = tons per square inch

1 Measured on a minimum 8 diameters gauge length.

2 Measured on a minimum 4 diameters gauge length.

Table C5C.3: Mechanical properties of steel reinforcement bars – 1960s to mid-1970s

Standard / Steel Property	NZS 197:1949 (BS 785:1938)			
Type of steel	Plain round bar *Mild steel (MS)* *Medium tensile (MT)* *High tensile (HT)*			
Yielding stress	*Bar size (diameter)*	*MS*	*MT*	*HT*
	Up to 1 inch	Not Specified	19.5 tsi (≈270 MPa)	23.0 tsi (≈317 MPa)
	Over 1 to 1½ inch		18.5 tsi (≈255 MPa)	22.0 tsi (≈303 MPa)
	Over 1½ to 2 inch		17.5 tsi (≈241 MPa)	21.0 tsi (≈290 MPa)
	Over 2 to 2½ inch		16.5 tsi (≈227 MPa)	20.0 tsi (≈275 MPa)
	Over 2½ to 3 inch		16.5 tsi (≈227 MPa)	19.0 tsi (≈262 MPa)
Tensile strength		≥ 28 tsi (≈ 386 MPa)	≥ 33 tsi (≈ 455 MPa)	≥ 37 tsi (≈ 510 MPa)
		≤ 33 tsi (≈ 455 MPa)	≤ 38 tsi (≈ 524 MPa)	≤ 43 tsi (≈ 593 MPa)
Elongation at fracture (%)	Up to 1 inch	≥ 20(1)	≥ 18(1)	≥ 18(1)
	Over 1 to 1½ inch	≥ 16(1)	≥ 14(1)	≥ 14(1)
	Under ⅜ inch	≥ 24(2)	≥ 22(2)	≥ 22(2)

Note:

psi = pounds per square inch

tsi = tons per square inch

1 Measured on a minimum 8 diameters gauge length.

2 Measured on a minimum 4 diameters gauge length.

Table C5C.4: Mechanical properties of steel reinforcement bars – 1970s onwards

Steel Property \ Standard	NZ 3402P:1973		NZS 3402:1989		AS/NZS 4671:2001	
Type of steel	Grade 275	Grade 380	Grade 300	Grade 430	Grade 300	Grade 500
Yielding stress (MPa) • Lower bound • Upper bound	275	380	≥ 275(min) (300(k)) ≤ 380(max) (355(k))	≥ 410(min) (430(k)) ≤ 520(max) (500(k))	≥ 300(k) ≤ 380(k)	≥ 500(k) ≤ 600(k)
Tensile Strength (MPa)	≥ 380 ≤ 520	≥ 570*	Not specified		Not specified	
Ratio R_m/R_e (TS/YS)	Not specified		1.15 ≤ $\frac{TS}{YS}$ ≤ 1.50	1.15 ≤ $\frac{TS}{YS}$ ≤ 1.40	1.15 ≤ $\frac{R_m}{R_e}$ ≤ 1.50	1.15 ≤ $\frac{R_m}{R_e}$ ≤ 1.40
Elongation at maximum force A_{gt} (%)	Not specified		Not specified		≥ 15	≥ 10
Elongation at fracture (%)	≥ 20(1)	≥ 12(1)	≥ 20(1)	≥ 12(1)	Not specified	

Note:

* But not less than 1.2 times the actual yield stress

1. Measured on a minimum 4 diameters gauge length.

k characteristic value

TS = tensile strength

YS = yield stress

R_m = value of maximum tensile strength *(determined from a single tensile test in accordance with AS 1391)*

R_e = value of the yield stress or 0.2% proof stress (*determined from a single tensile test in accordance with AS 1391)*

C5C.3 Mechanical Properties of Mesh

The evolution of Standards for hard drawn steel wire and mesh for concrete reinforcement is shown in Table C5C.5.

Table C5C.5: Evolution of hard drawn steel wire and mesh for concrete reinforcement standards in New Zealand

1949	1972	1975	2001
NZS 197:1949 (BS 785:1938) Rolled steel bars and hard drawn steel wire for concrete reinforcement	NZS 3421:1972 Hard drawn steel wire for concrete reinforcement (metric and imperial units)	NZS 3421:1975 Hard drawn steel wire for concrete reinforcement (metric units)	AS/NZS 4671:2001 Steel reinforcing material
	NZS 3422:1972 Welded fabric of drawn steel wire for concrete reinforcement (metric units)	NZS 3422:1975 Welded fabric of drawn steel wire for concrete reinforcement (metric units)	

Steel wire for concrete reinforcement was originally regulated in New Zealand by the first local steel code NZS 197:1949 (BS 785:1938). The tensile strength limits were between 37 ton/in^2 (510 MPa) and 42 ton/in^2 (580 MPa). The elongation limit was 7.5 % measured over a gauge length of 8 times the diameter. This standard remained valid until 1972.

In 1972, NZS 3421:1972 and NZS 3422:1972 replaced the old standard. The first of these provided specifications for hard drawn steel wire; the second, for welded fabric hard drawn steel wire. Hard drawn steel wires were normally available in diameters not greater than 0.1 inches (12.7 mm) and not less than 0.08 inches (2.0 mm). The minimum 0.2 percent proof stress limit was 70,000 lbf/in^2 (483 MPa) while the minimum tensile strength was 83,000 lbf/in^2 (572 MPa). The mechanical property limits of welded fabric of drawn steel wires were similar to the ones specified for hard drawn steel wires. A maximum tensile strength limit was introduced equal to 124,000 lbf/in^2 (855 MPa) for diameters up to and including 0.128 in (3.25 mm) and 112,000 lbf/in^2 (772 MPa) for diameters over 0.128 in.

In 1975 NZS 3421:1972 and NZS 3422:1972 were superseded by the metric units versions NZS 3421:1975 Hard drawn steel wire for concrete reinforcement (metric units) and NZS 3422:1975 Welded fabric of drawn steel wire for concrete reinforcement (metric units). The first was applied to plain and deformed wires while the second only to plain ones. The available diameters ranged between 2.5 mm and 8 mm. The mechanical property limits were similar to those prescribed in the 1972 standards: 485 MPa for minimum 0.2 percent prof stress; 575 MPa for minimum tensile strength and 855 MPa maximum tensile strength (for diameters up and including 3.15 mm) and 775 MPa (for diameters over 3.15 mm).

The current AS/NZS 4671:2001 (Steel reinforcing materials) replaced the old NZS 3421:1975 and NZS 3422:1975. This standard provides specifications for steel reinforcing bars and mesh. The steel grades are Grade 300E and Grade 500E. The commonly available mesh diameters are 6 mm, 7 mm, 8 mm and 9 mm for structural mesh and 4 mm and 5.3 mm for non-structural mesh. The most common mesh pitch size for is 200 by 200 mm for structural mesh and 150 by 150 mm for non-structural mesh.

Appendix C5D Test Methods for Investigating Material Properties

C5D.1 Concrete

The following table summarises test methods for investigating concrete material properties.

Table C5D.1: Overview of destructive, semi-destructive and non-destructive tests for investigating concrete material properties (De Pra, Bianchi and Pampanin, 2015; Malek et al., 2015)

Method	Capability/Use	Advantages	Disadvantages
DESTRUCTIVE TESTS			
Compressive test	Strength of concrete	Direct evaluation of concrete strength from compressive tests on cylindrical specimens	Disturbance of the sample, so excessive damage to obtain a representative core of concrete Previous test with pacometer necessary to individuate the regions without bars
SEMI-DESTRUCTIVE TESTS			
Pull-out	In-place estimation of the compressive and tensile strengths	In-place strength of concrete can be quickly measured	Pull-out device must be inserted in a hole drilled in the hardened concrete Only a limited depth of material can be tested
Pull-off/tear-off	Direct tension test	In situ tensile strength of concrete Determining bond strength between existing concrete and repair material	Sensitivity to rate of loading
Penetration probe (Windsor probe)	Estimation of compressive strength, uniformity and quality of concrete Measuring the relative rate of strength development of concrete at early ages	The equipment is easy to use (not requiring surface preparation) The results are not subject to surface conditions and moisture content	Minimum edge distance and member thickness are requested Not precise prediction of strength for concrete older than 5 years and where surface is affected by carbonation or cracking
NON-DESTRUCTIVE TESTS			
Visual tests	The first step in investigating concrete material	Quick evaluation of damage	No detailed information
Rebound hammer	Measuring surface hardness of concrete to estimate compressive strength	The assessment of the surface layer strength	Results can only suggest the hardness of surface layer
Durability test – Concrete electrical resistivity	Measuring the ability of the concrete to conduct the corrosion current	Inexpensive, simple and many measurements can be made rapidly	Not reliable at high moisture content

Method		Capability/Use	Advantages	Disadvantages
	Permeability	To evaluate the transfer properties of concrete (porosity)	Useful method to evaluate the risk of leaching, corrosion and freezing	Thickness limitation Age, temperature dependent Sufficient lateral sealing
Fiberscope		To check the condition of cavities, and honeycombing in reinforced concrete Voids detection along grouted post-stressed tendons	Direct visual inspection of inaccessible parts of an element	Semi destructive as the probe holes usually must be drilled Needs additional fibre to carry light from an external source
Stress-Wave propagation methods	Ultrasonic pulse velocity	Evaluation of concrete strength and quality Identification of internal damage and location of reinforcement	Excellent for determining the quality and uniformity of concrete; especially for rapid survey of large areas and thick members	The measure can be distorted by the presence of lesions in the concrete The test requires smooth surfaces for a good adhesion of the probes No information about the depth of suspected flaw
	Ultrasonic echo method	Quality control and integrity of concrete	Access to only one face is needed Internal discontinuities and their sizes can be estimated	Limited member thickness
	Impact echo method	Defects within concrete element such as delamination, voids, honeycombing	Access to only one face is needed	The ability of instrument is limited to less than 2 m thickness
	Spectral analysis of surface waves	Determining the stiffness profile of a pavement Depth of deteriorated concrete	Capability of determining the elastic properties of layered systems such as pavement and interlayered concrete	Complex signal processing
Nuclear methods	Gamma radiography	Location of internal cracks, voids and variations in density of concrete	Simple to operate Applicable to a variety of materials	X-ray equipment is bulky and expensive Difficult to identify cracks perpendicular to radiation beam
	Backscatter radiometry	Determining in-place density of fresh or hardened concrete	Access only to surface of test object Since this method's measurements are affected by the top 40 to 100 mm, best for assessing surface zone of concrete element	The accuracy of this method is lower than direct transmission Measurements are influenced by near surface material and are sensitive to chemical composition
	CT scanning	Concrete imaging	3D crack/damage monitoring	Sophisticated software for analysis Not in situ application Access to CT scanner needed

Method	Capability/Use	Advantages	Disadvantages
Infrared thermography	Detecting delamination, heat loss and moisture movement through concrete elements; especially for flat surfaces	Permanent records can be made Tests can be done without direct access to surface by means of infrared cameras	Expensive technique Reference standards are needed Very sensitive to thermal interference from other heat sources The depth and thickness of subsurface anomaly cannot be measured
Ground penetrating radar	Identification of location of reinforcement, depth of cover, location of voids and cracks Determination of in situ density and moisture content	Can survey large areas rapidly	Results must be correlated to test results on samples obtained Low level signals from targets as depth increases
Acoustic emission	Real time monitoring of concrete degradation growth and structural performance	A few transducers are enough to locate defects over large areas Can detect the initiation and growth of cracks in concrete under stress	Passive technique, could be used when the structure is under loading
Ultrasonic tomography (MIRA)	Uses high frequency (greater than 20,000 Hz) sound waves to characterise the properties of materials or detect their defects	Thickness measurement, reinforcement location, and distress evaluation	Significant efforts and user expertise are required for measurement and data interpretation of large scale application
Petrography	Forensic investigation of concrete Determining the composition and identifying the source of the materials Determination of w/c Determining the depth of fire damage	Microscopic examination of concrete samples	Laboratory facilities as well as highly experienced personnel are needed to interpret the result
Sclerometric method	Determination of compressive strength	Determination of a sclerometric index connected to compressive strength	The instrument must be in the horizontal direction or the reliability of results is reduced Empirical formulas, based on probabilistic methods, are used to obtain the concrete strength The preparation of the test surface is laborious and expensive
SonReb method	Determination of compressive strength	The concomitant use of sclerometric and ultrasonic methods can reduce mistakes due to the influence of humidity and aging of concrete	Risk of regression on a small statistically representative sample

C5D.2 Reinforcing Steel

The following table summarises test methods for investigating reinforcing steel material properties.

Table C5D.2: Destructive and non-destructive tests for investigating reinforcing steel material properties (De Pra, Bianchi and Pampanin, 2015)

Method	Capability/Use	Advantages	Disadvantages
DESTRUCTIVE TESTS			
Tensile test	Steel strength (yield strength, tensile strength and elongation on 5 diameters gauge length)	Direct evaluation of steel strength	The test is limited to areas that are easily accessible The interpretation of the results is subjective and depends on the operator`s experience
NON-DESTRUCTIVE TESTS			
Hardness stress with Leeb method	Evaluation of hardness and tensile strength	Low cost The device is portable, so particularly useful in difficult operative conditions	A previous survey with pacometer is required to identify the regions with less cover
Penetrating liquids	Deterioration of steel	Simple to apply	The surface must be cleaned before the test to remove all extraneous substances Not applicable on too porous surfaces
Measure of potential corrosion of reinforcement	Evaluation of potential corrosion	Possibility to measure the potential corrosion of the bars	The electrode must be dampened 12 hours before the test A previous survey with a pacometer is required to individuate the presence of bars
Survey with pacometer	Identification of bars (cover, bar free interface, spacing of stirrups, diameters of bars)	Identification of the areas without bars in order to identify where it is possible to carry out concrete tests	The device is sensitive to the presence of the ferromagnetic material The method is slow and laborious
Georadar	Determination of dimensions and depth of foundations	Possible to have information on foundations	Calibration of the instrumentation is required before the data acquisition, investigating two directions

Appendix C5E Evolution of Standard Based Design Details for Reinforcement and Detailing

C5E.1 Beams

The following table summarises the evolution of standard-based details requirements for beams, from NZS 3101P:1970 to NZS 3101:2016.

Table C5E.1: Evolution of standard-based details requirements for beams

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970
Lateral support spacing	$50b_w$	$50b_w$ (for earthquake)	$50b_w$	
ρ_{max}	$\rho_{max} = 0.75\rho_{bal}$	$\rho_{max} = 0.75\rho_{bal}$	$\rho_{max} = 0.75\rho_{bal}$	$\rho_{max} = 0.75\rho_{bal}$ (for USD)
ρ_{min}	$\rho_{min} = \frac{\sqrt{f'_c}}{4f_y} \geq \frac{1.4}{f_y}$	$\rho_{min} = \frac{\sqrt{f'_c}}{4f_y} \geq \frac{1.4}{f_y}$	$\rho_{min} = \frac{1.4}{f_y}$	$\rho_{min} = \frac{200}{f_y}$
ρ_{min} (alternatively)	$\rho_{min} = \frac{4}{3}\rho_{reqd}$ (for gravity only)	$\rho_{min} = \frac{4}{3}\rho_{reqd}$	$\rho_{min} = \frac{4}{3}\rho_{reqd}$	$\rho_{min} = \frac{4}{3}\rho_{reqd}$
Maximum d_b in internal beam-column joints (for nominally ductile structures)	$\frac{d_b}{h_c} = 4\alpha_f \frac{\sqrt{f'_c}}{f_y}$ $\alpha_f = 0.85$ (two-way) $\alpha_f = 1.00$ (one-way)			
Minimum requirements for transverse reinforcement	5 mm in diameter $f_{yt} \leq 500MPa$		6 mm in diameter (for earthquake)	
Maximum nominal shear stress	$v_n \leq 0.2f'_c$ or $8MPa$	$v_n \leq \begin{cases} 0.2f'_c \\ 1.1\sqrt{f'_c} \\ 9MPa \end{cases}$	$v_n \leq 0.2f'_c$ or $6MPa$	$v_n \leq 5\sqrt{f'_c}$ $v_n \leq 8.5\sqrt{f'_c}$ (USD)
Spacing limits for shear reinforcement	$S_{max} \leq \begin{cases} 0.5d \ , b_w \\ 500\ mm \\ 16d_b \end{cases}$	$S_{max} \leq \begin{cases} 0.5d \\ 600\ mm \end{cases}$	$S_{max} \leq \begin{cases} 0.5d \\ 600\ mm \end{cases}$	$S_{max} \leq \begin{cases} 0.75d \\ 450\ mm \end{cases}$
	($0.5d$ and $500\ mm$ reduced by half if $v_s \geq 0.33\sqrt{f'_c}$)	($0.5d$ and $500\ mm$ reduced by half if $v_s \geq 0.07f'_c$)	($0.5d$ and $500\ mm$ reduced by half if $v_s \geq 0.07f'_c$)	($S_{max} \leq 0.25d$ if $v_n \geq 3\sqrt{f'_c}$, or $v_n \geq 5.1\sqrt{f'_c}$ for USD)
Minimum area of shear reinforcement	$A_v = \frac{1}{16}\sqrt{f'_c}\frac{b_w s}{f_{yt}}$	$A_v = 0.35\frac{b_w s}{f_{yt}}$	$A_v = 0.35\frac{b_w s}{f_{yt}}$	$A_v = 0.0015 b_w s$
Dimension of beams (for earthquake)	$\frac{L_n}{b_w} \leq 25$ $\frac{L_n h}{b_w^2} \leq 100$ $b_w \geq 200\ mm$	$\frac{L_n}{b_w} \leq 25$ $\frac{L_n h}{b_w^2} \leq 100$ $b_w \geq 200\ mm$	$\frac{L_n}{b_w} \leq 25$ $\frac{L_n h}{b_w^2} \leq 100$ $b_w \geq 200\ mm$	

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970
ρ_{max} (for earthquake, within plastic hinge region)	$\rho_{max} = \frac{f'_c + 10}{6f_y}$ ≤ 0.025	$\rho_{max} = \frac{f'_c + 10}{6f_y}$ ≤ 0.025	$\rho_{max} = \frac{1 + 0.17\left(\frac{f'_c}{7} - 3\right)}{100}$ $\left(1 + \frac{\rho'}{\rho}\right) \leq \frac{7}{f_y}$	
ρ_{min} (for earthquake, within plastic hinge region)	$A'_s > 0.5A_s$ for ductile plastic regions. $A'_s > 0.38A_s$ for limited ductile plastic regions. $\rho_{min} = \frac{\sqrt{f'_c}}{4f_y}$	$A'_s > 0.5A_s$ $\rho_{min} = \frac{\sqrt{f'_c}}{4f_y}$	$A'_s > 0.5A_s$ $\rho_{min} = \frac{1.4}{f_y}$	
Maximum longitudinal beam bar diameter to column depth (for earthquake)	$\frac{d_b}{h_c} \leq 3.3\alpha_f\alpha_d\frac{\sqrt{f'_c}}{1.25f_y}$ $f'_c \leq 70MPa$ $\alpha_d = 1.00$ (ductile) $\alpha_d = 1.20$ (limited ductile)			
Minimum area of shear reinforcement (for earthquake)	$A_v = \frac{1}{12}\sqrt{f'_c}\frac{b_w s}{f_{yt}}$			
Spacing limits for shear reinforcement (for earthquake)	$S_{max} = \begin{cases} 12d_b \\ d/2 \end{cases}$	$S_{max} = \begin{cases} 16d_b \\ b_w \end{cases}$	$S_{max} \leq \begin{cases} b_w \\ 48d_v \\ 16d_b \end{cases}$	
Minimum area of shear reinforcement in plastic hinge regions (for earthquake)	$A_{te} = \frac{\sum A_b f_y}{96f_{yt}}\frac{s}{d_b}$	$A_{te} = \frac{\sum A_b f_y}{96f_{yt}}\frac{s}{d_b}$	$A_{te} = \frac{\sum A_b f_y}{160f_{yt}}\frac{s}{d_b}$	
Spacing limits for shear reinforcement in plastic hinge regions (for earthquake)	$S_{max} = \begin{cases} 6d_b \\ d/4 \end{cases}$ (ductile) $S_{max} = \begin{cases} 10d_b \\ d/4 \end{cases}$ (limited ductile)	$S_{max} = \begin{cases} 6d_b \\ d/4 \end{cases}$	$S_{max} = \begin{cases} 6d_b \\ d/4 \end{cases}$	
Maximum nominal shear stress (for earthquake)		$v_n \leq \begin{cases} 0.16f'_c \\ 0.85\sqrt{f'_c} \end{cases}$		

Note:

NZS 3101P:1970 units of [psi]

USD: Ultimate Strength Design

C5E.2 Columns

The following table summarises the evolution of standards-based details requirements for columns, from NZSS 1990 (1964) to NZS 3101:2006.

Table C5E.2: Evolution of standards-based details requirements for columns (Niroomandi et al., 2015)

Requirement	NZS 3101: 2006	NZS 3101: 1995	NZS 3101: 1982	NZS 3103: 1970	NZSS 1900 (1964)
Strength reduction factor (ϕ)	0.85	0.85	0.9 for conforming transverse 0.7 for others	0.75 for spirally reinforced 0.7 for tied	-
f_c'	$25-100\ MPa$ For DPRs[5] and LDPRs[6]: $25-70\ MPa$	-	-	-	-
f_y	$<500\ MPa$	-	-	-	-
f_{yt}	$<500\ MPa$ for shear $<800\ MPa$ for confinement	$<500\ MPa$ for shear $<800\ MPa$ for confinement	$<400\ MPa$	$<414\ MPa$	-
Maximum axial compressive load	$0.85\phi N_{\mathrm{n,max}}$[1] For DPRs and LDPRs: $0.7\phi N_{\mathrm{n,max}}$[1]	$0.85\phi N_{\mathrm{n,max}}$[1] For DPRs: $0.7\phi N_{\mathrm{n,max}}$[1]	$0.85\phi N_{\mathrm{n,max}}$[1] for conforming, otherwise $0.8\phi N_{\mathrm{n,max}}$[1] For DPRs: Min of ($0.7\phi f_c' A_g$ and $0.7\phi N_{\mathrm{n,max}}$[1])	P_0[18]	For tied columns: $P = cA_c + ncA$ For spirally columns: $P = cA_k = ncA + 2t_b A_b$
Dimension of column	For DPRs and LDPRs: $b_w \geq L_n/25$ $b_w \geq \sqrt{L_n h/100}$	For DPRs: $b_w \geq L_n/25$ $b_w \geq \sqrt{L_n h/100}$	For DPRs: $b_w \geq L_n/25$ $b_w \geq \sqrt{L_n h/100}$	25.4 mm for circular 20.32 for rectangular or $A_g > 413\ \mathrm{mm}^2$	-
Extend of ductile detailing length, l_y, for detailing purposes	For DPRs and LDPRs: $l_y = h$ for $N_0^* \leq 0.25\phi f_c' A_g$ $l_y = 2h$ for $0.25\phi f_c' A_g < N_0^* \leq 0.5\phi f_c' A_g$ $l_y = 3h$ for $N_0^{*2} > 0.5\phi f_c' A_g$	For DPRs: $l_y = h$ for $N_0^* \leq 0.25\phi f_c' A_g$ $l_y = 2h$ for $0.25\phi f_c' A_g < N_0^* \leq 0.5\phi f_c' A_g$ $l_y = 3h$ for $N_0^{*2} > 0.5\phi f_c' A_g$	For DPRs: $l_y = h$ for P_e[13] $\leq 0.3\phi f_c' A_g$ $l_y = 1.5h$ for $P_e > 0.3\phi f_c' A_g$	-	-
Minimum longitudinal reinforcement ratio	$0.008A_g$	$0.008A_g$	$0.008A_g$	0.01	0.008

Requirement	NZS 3101: 2006	NZS 3101: 1995	NZS 3101: 1982	NZS 3103: 1970	NZSS 1900 (1964)
Maximum longitudinal reinforcement ratio	$0.08A_g$ For DPRs and LDPRs: $18\,A_g/f_y$	$0.08A_g$ For DPRs: $18\,A_g/f_y$	$0.08A_g$ For DPRs: $0.06A_g$ for Grade 275 $0.045A_g$ for Grade 380	0.08	0.08
Maximum longitudinal reinforcement ratio at splices	$0.08A_g$ For DPRs and LDPRs: $24\,A_g/f_y$	$0.08A_g$ For DPRs: $24\,A_g/f_y$	For DPRs: $0.08A_g$ for Grade 275 $0.06A_g$ for Grade 380	0.12	-
Minimum number of longitudinal bars	8 bars, but may be reduced 6 or 4 if clear spacing is less than 150 mm and $N^* \leq 0.1\phi f_c' A_g$	6 bars in a circular arrangement 4 bars in a rectangular arrangement	6 bars in a circular arrangement 4 bars in a rectangular arrangement	Same as nominally ductile (1995)	Same as nominally ductile (1995)
Maximum spacing between longitudinal bars requiring restraint	Circular columns, larger of one quarter of a diameter or 200 mm Rectangular, larger of one third of column dimension in direction of spacing or 200 mm, spacing can be increased in centre of column when $h/b > 20$	Larger of one third of column dimension in direction of spacing or 200 mm for Rectangular column	200 mm	-	-
	For DPRs and LDPRs: Larger of one-quarter of the column dimension (or diameter) in direction of spacing or 200 mm In protected plastic hinge regions and outside plastic hinge regions same as nominally ductile	For DPRs: Larger of one-quarter of the column dimension (or diameter) in direction of spacing or 200 mm			

Requirement	NZS 3101: 2006	NZS 3101: 1995	NZS 3101: 1982	NZS 3103: 1970	NZSS 1900 (1964)
Maximum longitudinal column bar diameter	For DPRs and LDPRs: $\frac{d_b}{h_b} \leq 3.2\frac{\sqrt{f'_c}}{f_y}$ Bar diameter can be increased by 25% when plastic hinges are not expected to develop in column end zones and need not be met when bars remain in tension or compression over the length of the joint	-	-	12.7 mm (minimum)	50.8 mm (maximum) 12.7 mm (minimum)
Minimum diameter for transverse reinforcement (outside of the potential plastic hinge region)	Rectangular hoops and ties 5 mm for $d_b < 20$ 10 mm for $20 \leq d_b < 32$ 12 mm for $d_b > 32$ Spiral or hoops of circular shape, 5 mm	-	Rectangular hoops and ties 6 mm for $d_b < 20$ 10 mm for $20 \leq d_b < 32$ 12 mm for $d_b > 32$ Spiral or hoops of circular shape, 6 mm	6.35 mm	6.35 mm $> d_b/3$
Maximum vertical spacing of ties (outside of the potential plastic hinge region)	Smaller of $h_{min}/3$ or $10d_b$	Smaller of $h_{min}/3$ or $10d_b$	If using $\phi = 0.9$ smaller of $h_{min}/5$ or $16\ d_b$ If using $\phi = 0.7$ smaller of $h_{min}, 16d_b$ or $48\ d_s$	For Spirally columns, $d_c/6$ For tied columns, min $\{h_{min}, 16d_b$ and $48\ d_s\}$	For Spirally columns, max $\{1$ in. and $3d_s\}$, min $\{3$ in and $d_c/6$ and also $\rho_s > 0.004A_g\}$
		For DPRs: It shouldn't be lower than 70% of the ones within the plastic hinge region	For DPRs: Smaller of $2h_{min}/5$, $12d_b$ or 400 mm		For tied columns, min $\{12d_b$, 1 in. and $2h_{min}/3\}$
Anti-buckling reinforcement (outside of the potential plastic hinge region)	Rectangular hoops and ties $A_{te} = \frac{\sum A_b f_y}{135 f_{yt}} \frac{s}{d_b}$ Spirals or hoops of circular shape $\rho_s = \frac{A_{st}}{155 d''} \frac{f_y}{f_{yt}} \frac{1}{db}$	Rectangular hoops and ties $A_{te} = \frac{\sum A_b f_y}{135 f_{yt}} \frac{s}{d_b}$ Spirals or hoops of circular shape $\rho_s = \frac{A_{st}}{155 d''} \frac{f_y}{f_{yt}} \frac{1}{db}$	-	-	-

Requirement	NZS 3101: 2006	NZS 3101: 1995	NZS 3101: 1982	NZS 3103: 1970	NZSS 1900 (1964)
Confinement reinforcement (outside of the potential plastic hinge region)	Rectangular hoops and ties A_{sh}[3] Spirals or hoops of circular shape ρ_s[4]	Rectangular hoops and ties A_{sh}[9] Spirals or hoops of circular shape ρ_s[10]	If using φ=0.9 then for Rectangular hoops and ties A_{sh}[11] Spirals or hoops of circular shape ρ_s[12]	Spirals shape ρ_s[12]	-
Minimum shear reinforcement (outside of the potential plastic hinge region)	$A_v = \frac{1}{16}\sqrt{f'_c}\frac{b_w S}{f_{yt}}$ For DPRs and LDPRs: $A_v = \frac{1}{12}\sqrt{f'_c}\frac{b_w S}{f_{yt}}$	-	-	-	-
Maximum shear force (outside of the potential plastic hinge region)	$V_n \leq 0.2 f'_c b_w d, or\ 8\ b_v$	-	-	-	-
Minimum diameter for transverse reinforcement (within potential plastic hinge region)	Same as outside plastic hinge region	-	-	Same as outside plastic hinge region	-
Maximum vertical spacing of ties (within potential plastic hinge region)	For DPRs: Smallest of $h_{min}/4$ or $6\ d_b$ For LDPRs: Smallest of $h_{min}/4$ or $10\ d_b$	For DPRs: Smallest of $h_{min}/4$ or $6\ d_b$	For DPRs: Smaller of $h/5$, diameter, $/5$ $6d_b$ or $200\ mm$	Same as outside plastic hinge region	-
Anti-buckling reinforcement (within potential plastic hinge region)	For DPRs and LDPRs: For rectangular hoops and ties $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}}\frac{S_h}{d_b}$ For spirals or hoops of circular shape $\rho_s = \frac{A_{st}}{110 d''}\frac{f_y}{f_{yt}}\frac{1}{db}$	For DPRs: For rectangular hoops and ties $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}}\frac{S_h}{d_b}$ For spirals or hoops of circular shape $\rho_s = \frac{A_{st}}{110 d''}\frac{f_y}{f_{yt}}\frac{1}{db}$	-	-	-
Confinement reinforcement (within potential plastic hinge region)	For DPRs and LDPRs: Rectangular hoops and ties for DPRs, A_{sh}[7]	For DPRs: Rectangular hoops and ties for DPRs, A_{sh}[7]	For DPRs: Spirals or hoops of circular shape ρ_{s1}[14] or ρ_{s2}[15]	Same as outside plastic hinge region	-

Requirement	NZS 3101: 2006	NZS 3101: 1995	NZS 3101: 1982	NZS 3103: 1970	NZSS 1900 (1964)
	for LDPRs, $0.7\ A_{sh}$[7] Spirals or hoops of circular shape for DPRs, ρ_s[8] for LDPRs, $0.7\ \rho_s$[8]	for LDPRs, $0.7\ A_{sh}$[7] Spirals or hoops of circular shape for DPRs, ρ_s[8] for LDPRs, $0.7\ \rho_s$[8]	Rectangular hoops and ties A_{sh1}[16] or A_{sh2}[17]		
Minimum shear reinforcement (within potential plastic hinge region)	Same as outside plastic hinge region	-	-	-	-
Maximum shear force (within potential plastic hinge region)	Same as outside plastic hinge region	-	-	-	-

Note:

1. $N_{n,max} = \alpha_1 f'_c(A_g - A_{st}) + f_y A_{st}$
2. $N_0^* = 0.7\phi N_{n,max}$
3. $A_{sh} = \frac{(1-\rho_t m)S_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0065 S_h h''$ (N^* = design axial load at ultimate limit state)
4. $\rho_s = \frac{(1-\rho_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0084$ (N^* = design axial load at ultimate limit state)
5. *DPR* = Ductile Potential Plastic Region
6. *LDPR* = Limited Ductile Potential Plastic Region
7. $A_{sh} = \frac{(1.3-\rho_t m)S_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N_0^*}{\phi f'_c A_g} - 0.006 S_h h''$
8. $\rho_s = \frac{(1.3-\rho_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N_0^*}{\phi f'_c A_g} - 0.0084$
9. $A_{sh} = \frac{(1-\rho_t m)S_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0065 S_h h''$ $\left(N^* = 0.85\phi\alpha_1 f'_c(A_g - A_{st}) + f_y A_{st}\right)$
10. $\rho_s = \frac{(1-\rho_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0084$ $\left(N^* = 0.85\phi\alpha_1 f'_c(A_g - A_{st}) + f_y A_{st}\right)$
11. $A_{sh} = 0.3 S_h h'' \left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}$
12. $\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}$
13. P_e = Maximum design axial load in compression at a given eccentricity
14. $\rho_{s1} = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}\left(0.5 + 0.125\frac{P_e}{\phi f'_c A_g}\right)$
15. $\rho_{s2} = 0.12\frac{f'_c}{f_{yh}}\left(0.5 + 0.125\frac{P_e}{\phi f'_c A_g}\right)$
16. $A_{sh} = 0.3 S_h h'' \left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right)$
17. $A_{sh} = 0.12 S_h h''\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right)$
18. $P_0 = \phi\left(0.85 f'_c(A_g - A_{st}) + A_{st} f_y\right)$

C5E.3 Beam-Column Joints

The following table summarises the evolution of standards-based design/details requirements for beam-column joints, from NZS 3101:1982 to NZS 3101:2006.

Table C5E.3: Evolution of standards-based beam-column joints design/details requirements (Cuevas et al., 2015)

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982
Maximum horizontal shear stress	$v_{jh} \le \begin{cases} 0.20 f'_c \\ 10\text{MPa} \end{cases}$	$v_{jh} \le 0.25 f'_c$	
Minimum horizontal transverse confinement reinforcement	For spirals or circular hoops: $\rho_s = \frac{(1-p_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0084$ $\rho_s = \frac{A_{st}}{155 d''}\frac{f_y}{f_{yt}}\frac{1}{d_b}$ For rectangular hoop and tie reinforcement: $\frac{A_{sh}}{} = \frac{(1-p_t m) s_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0065 s_h h''$ $A_{te} = \frac{\sum A_b f_y}{135 f_{yt}}\frac{s_h}{d_b}$ With $\begin{cases} A_g/A_c \le 1.50 \\ p_t m \le 0.40 \end{cases}$ (reduce by half when joints connecting beams at all four column faces)	For spirals or circular hoops: $\rho_s = \frac{(1-p_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0084$ $\rho_s = \frac{A_{st}}{155 d''}\frac{f_y}{f_{yt}}\frac{1}{d_b}$ For rectangular hoop and tie reinforcement: $\frac{A_{sh}}{} = \frac{(1-p_t m) s_h h''}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{\phi f'_c A_g} - 0.0065 s_h h''$ $A_{te} = \frac{\sum A_b f_y}{135 f_{yt}}\frac{s_h}{d_b}$ With $\begin{cases} A_g/A_c \le 1.20 \\ p_t m \le 0.40 \end{cases}$ (reduce by half when joints connecting beams at all four column faces)	For spirals or circular hoops: $\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}$ For rectangular hoop and tie reinforcement: $A_{sh} = 0.3 s_h h''\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}$ where $f_{yh} \le 500MPa$
Spacing limits	$S_{max} = \begin{cases} (D,b,h)/3 \\ 10 d_b \\ 200\ mm \end{cases}$	$S_{max} = \begin{cases} (D,b,h)/3 \\ 10 d_b \\ 200\ mm \end{cases}$	$S_{max} = \begin{cases} (D,b,h)/5 \\ 10 d_b \\ 200\ mm \end{cases}$
Design yield strength (for earthquake)	$f_{yh} \le 500\text{MPa}$ $f_{yv} \le 500\text{MPa}$	$f_{yh} \le 500\text{MPa}$ $f_{yv} \le 500\text{MPa}$	
Maximum horizontal shear stress (for earthquake)	$v_{jh} \le \begin{cases} 0.20 f'_c \\ 10\text{MPa} \end{cases}$	$v_{jh} \le 0.20 f'_c$	$v_{jh} \le 1.5\sqrt{f'_c}$
Minimum horizontal joint reinforcement (for earthquake)	For spirals or circular hoops: $\rho_s = \frac{(1.3-p_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{f'_c A_g} - 0.0084$ (*) $\rho_s = \frac{A_{st}}{110 d''}\frac{f_y}{f_{yt}}\frac{1}{d_b}$	For spirals or circular hoops: $\rho_s = 0.70\left\{\frac{(1.3-p_t m)}{2.4}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{f'_c A_g} - 0.0084\right\}$ $\rho_s = \frac{A_{st}}{110 d''}\frac{f_y}{f_{yt}}\frac{1}{d_b}$	For spirals and circular hoops, the greater of: $\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right)$

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982
	For rectangular hoop and tie reinforcement: $A_{sh} = \frac{(1.3-p_t m)s_h h"}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{f'_c A_g} - 0.006 s_h h"$ (*) $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}}\frac{s_h}{d_b}$ With $\begin{cases} A_g/A_c \le 1.50 \\ p_t m \le 0.40 \end{cases}$ (*) 70% reduction for limited ductile	For rectangular hoop and tie reinforcement: $A_{sh} = 0.70\left\{\frac{(1.3-p_t m)s_h h"}{3.3}\frac{A_g}{A_c}\frac{f'_c}{f_{yt}}\frac{N^*}{f'_c A_g} - 0.006 s_h h"\right\}$ $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}}\frac{s_h}{d_b}$ With $\begin{cases} A_g/A_c \le 1.20 \\ p_t m \le 0.40 \end{cases}$ (*) 70% reduction not allowed at the joint of the columns of the first storey	$\rho_s = 0.12\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right)$ For rectangular hoop and tie reinforcement, the greater of: $A_{sh} = 0.3 s_h h"\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right)$ $A_{sh} = 0.12 s_h h"\frac{f'_c}{f_{yh}}\left(0.5 + 1.25\frac{P_e}{\phi f'_c A_g}\right)$
Spacing limits (for earthquake)	$S_{max} = \begin{cases} (D,b,h)/4 \\ 6d_b \\ 200\ mm \end{cases}$ (ductile) $S_{max} = \begin{cases} (D,b,h)/4 \\ 10d_b \\ 200\ mm \end{cases}$ (limited ductile)	$S_{max} = \begin{cases} (D,b,h)/4 \\ 6d_b \\ 200\ mm \end{cases}$	$S_{max} = \begin{cases} (D,b,h)/5 \\ 6d_b \\ 200\ mm \end{cases}$
Spacing limits for vertical reinforcement (for ductile members adjacent to the joint)	$S_{max} = \begin{cases} (D,h,b)/4 \\ 200\ mm \end{cases}$ (at least one intermediate bar in each side of the column in that plane)	$S_{max} = \begin{cases} (D,h,b)/4 \\ 200\ mm \end{cases}$ (at least one intermediate bar in each side of the column in that plane)	$S_{max} = 200\ mm$ (at least one intermediate bar in each side of the column in that plane)
Maximum diameter of longitudinal beam bars passing through joints (for ductile members adjacent to the joint)	$\frac{d_b}{h_c} \le 3.3\alpha_f\alpha_d\frac{\sqrt{f'_c}}{1.25 f_y}$ $f'_c \le 70MPa$ $\alpha_d = 1.00$ (ductile) $\alpha_d = 1.20$ (limited ductile)		
Maximum diameter of column bars passing through joints (for ductile members adjacent to the joint)	$\frac{d_b}{h_b} \le 3.2\frac{\sqrt{f'_c}}{f_y}$ (1) $\frac{d_b}{h_b} \le 4.0\frac{\sqrt{f'_c}}{f_y}$ (2) For columns designed by Method B or by Method A (and the joint is below the mid height of the second storey)		

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982
	For columns designed by Method A and the joint is above the mid height of the second storey		

Note:

NZS 3101P:1970, clause 1.2.6 states that "…The reinforcing spiral shall extend from the floor level in any storey or from the top of the footing to the level of the lowest horizontal reinforcement in the slab, drop panel, or beam above." Therefore, no spiral, hoop or tie is required in the beam-column joint.

C5E.4 Walls

Table C5E.4 summarises the evolution of the New Zealand standards-based design/details requirements for walls, while Table C5E.5 provides a key to the notation used throughout the various Standards.

Table C5E.4: Evolution of standards-based design/details requirements for walls

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P: 1970	NZS 1900:1964 (bylaw)
Minimum thickness-general	100 mm	100 mm for the uppermost 4 m of wall height and for each successive 7.5 m downward (or fraction thereof), shall be increased by 25 mm	150 mm for the uppermost 4 m of wall height and for each successive 7.5 m downward (or fraction thereof), shall be increased by 25 mm	6 in.	5 in.
Limitations on the height to thickness ratio	If $N^* > 0.2f'_c A_g$ $\frac{K_e L_n}{t} \leq 30$ L_n: the clear vertical distance between floors or other effective horizontal lines of lateral support	If $N^* > 0.2f'_c A_g$ $\frac{L_n}{t} \leq 25$	$\frac{L_n}{t} \leq 10$ UNLESS: 1- the neutral axis depth for the design loading $\leq 4b$ or $0.3l_w$ 2- Any part of the wall within a distance of $3b$ from the inside of a continuous line of lateral support provided by a flange or cross wall	$\frac{L_n}{t} \leq 35$ L_n: the distance between lateral supports (Horizontal or Vertical)	$\frac{L_n}{t} \leq 24$ L_n: the distance between lateral supports (Horizontal or Vertical)
Singly reinforced walls Limitations on the height to thickness ratio to prevent flexural torsional buckling of in-plane loaded walls	$\frac{\frac{k_{ft} L_n}{t}}{\leq 12 \sqrt{\frac{L_n / L_w}{\lambda}}}$ where: $N^* \leq 0.015 f'_c A_g$ and $\frac{L_n}{t} \leq 75$ and $\frac{k_{ft} L_n}{t} \leq 65$	No limitations	No limitations	No limitations	No limitations
Doubly reinforced walls Moment magnification required when:	$\frac{k_e L_n}{t} \geq \frac{\alpha_m}{\sqrt{\frac{N^*}{f'_c A_g}}}$	No requirements	No requirements	No requirements	No requirements

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970	NZS 1900:1964 (bylaw)
Minimum thickness for prevention of instability within plastic hinge region	$b_{\mathrm{m}} = \frac{\alpha_{\mathrm{r}} k_{\mathrm{m}} \beta (A_{\mathrm{r}} + 2) \mathrm{L_w}}{1700\sqrt{\xi}}$ $\beta = 7\ (DPR)$ $\beta = 5\ (LDPR)$	$b_{\mathrm{m}} = \frac{k_{\mathrm{m}} (\mu + 2)(A_{\mathrm{r}} + 2}{1700\sqrt{\xi}}$	No requirements	No requirements	No requirements
Ductile detailing length - special shear stress limitations	$\max\{L_{\mathrm{w}}, 0.17\frac{M}{V}\}$ Measured from the 1st flexural yielding section Need not be greater than $2L_{\mathrm{w}}$	$\max\{L_{\mathrm{w}}, \frac{h_{\mathrm{w}}}{6}\}$ Measured from the 1st flexural yielding section Need not be greater than $2L_{\mathrm{w}}$	$\max\{L_{\mathrm{w}}, \frac{h_{\mathrm{w}}}{6}\}$ Measured from the 1st flexural yielding section Need not be greater than $2L_{\mathrm{w}}$	No requirements	No requirements
Limitation on the use of singly reinforced walls	$\rho_{\mathrm{l}} \leq 0.01$ $b \leq 200\ \mathrm{mm}$	$b \leq 200\ \mathrm{mm}$ $\mu \leq 4$	$b \leq 200$ mm or if the design shear stress $\leq 0.3\sqrt{f_{\mathrm{c}}'}$	Earth retaining walls: $b < 10$ in. Other walls: $b < 9$ in.	$t < 10$ in.
Minimum longitudinal reinforcement ratio	$\rho_{\mathrm{n}} = \frac{\sqrt{f_{\mathrm{c}}'}}{4f_{\mathrm{y}}}$	$\rho_{\mathrm{l}} = \frac{0.7}{f_{\mathrm{y}}}$	$\rho_{\mathrm{l}} = \frac{0.7}{f_{\mathrm{y}}}$	$\frac{9000}{f_{\mathrm{y}}}\%$ $\geq 0.18\%$ Note: f_{y} in units of [psi]	0.0025 (mild steel) 0.0018 (high tensile steel)
Maximum longitudinal reinforcement ratio (ρ_l)	$\frac{16}{f_{\mathrm{y}}}$	$\frac{16}{f_{\mathrm{y}}}$	$\frac{16}{f_{\mathrm{y}}}$	No requirements	No requirements
Maximum spacing of longitudinal reinforcement	Min $\{Lw/3, 3t$, or $450\ \mathrm{mm}\}$	Min $\{2.5b, 450\ \mathrm{mm}\}$	Min$\{2.5b, 450\ \mathrm{mm}\}$	Min $\{2.5b, 18\mathrm{in.}\ (457\ \mathrm{mm})\}$	$2.5b$
Anti-buckling reinforcement (Outside of the potential plastic hinge region)	Where: $\rho_{\mathrm{l}} > \begin{Bmatrix} \frac{2}{f_{\mathrm{y}}}\ \mathrm{DPR} \\ \frac{3}{f_{\mathrm{y}}}\ \mathrm{LDPR} \end{Bmatrix}$ $d_{\mathrm{tie}} > d_{\mathrm{b}}/4$ $\mathrm{Spacing} < 12\mathrm{d_b}$	Where: $\rho_{\mathrm{l}} > \frac{2}{f_{\mathrm{y}}}$ $d_{\mathrm{tie}} > d_{\mathrm{b}}/4$ $\mathrm{Spacing} < 12d_{\mathrm{b}}$	Hoop or tie sets Spacing $\leq \min \begin{cases} \text{least lateral dime} \\ 16\mathrm{d_b} \\ 48d_{\text{transverse b}} \end{cases}$	No requirements	No requirements

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970	NZS 1900:1964 (bylaw)
Anti-buckling reinforcement (Within the potential plastic hinge region)	Where: $\rho_l > \left\{ \begin{matrix} \frac{2}{f_y}\ DPR \\ \frac{3}{f_y}\ LDPR \end{matrix} \right\}$ $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{d_b}$ Spacing $\leq \left\{ \begin{matrix} 6d_b\ (DPR) \\ 10d_b\ (LDPR) \end{matrix} \right\}$	Where: $\rho_l > \frac{2}{f_y}$ $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{d_b}$ Spacing $\leq 6d_b$	Where: $\rho_l > \frac{2}{f_y}$ $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{100}$ Spacing $\leq 6d_b$	No requirements	No requirements
Confinement reinforcement	Where neutral axis depth $> c_c = \frac{0.1\phi_{ow} L_w}{\lambda}$ $\lambda = 1.0$ (DPR) $\lambda = 2.0$ (LDPR) $A_{sh} = \alpha s_h h'' \frac{A_g^*}{A_c^*} \frac{f_c'}{f_{yh}} \left(\frac{c}{L_w} - 0.07 \right)$ $\alpha = 0.25$ (DPR) $\alpha = 0.175$ (LDPR)	Where neutral axis depth $> c_c = \left(\frac{0.3\phi_o}{\mu} \right) L_w$ $A_{sh} = \left(\frac{\mu}{40} + 0.1 \right) s_h h'' \frac{A_g^*}{A_c^*} \frac{f_c'}{f_{yh}} \left(- 0.07 \right)$	Where neutral axis depth $> c_c = \left\{ \begin{matrix} 0.1\phi_o S l_w \\ \text{or} \\ \frac{8.6\phi_o S l_w}{(4-0.7S)\left(17+\frac{h_w}{l_w}\right)} \end{matrix} \right\}$ $A_{sh} = \max \left\{ \begin{matrix} 0.3 s_h h'' \left(\frac{A_g^*}{A_c^*} - 1 \right) \frac{1}{f} \\ 0.12 s_h h'' \frac{f_c'}{f_{yh}} \left(\end{matrix} \right.$	No requirements	No requirements
Maximum spacing of confinement reinforcement	DPR: $\min \{6d_b, 0.5t\}$ LDPR: $\min \{10d_b, t\}$	Min $\{6d_b, 0.5t,$ 150 mm$\}$	Min $\{6d_b, 0.5t,$ 150 mm$\}$	No requirements	No requirements
Minimum confinement length	Max $\left\{ \begin{matrix} c - 0.7c_c \\ 0.5c \end{matrix} \right\}$ c: neutral axis depth	Max $\left\{ \begin{matrix} c - 0.7c_c \\ 0.5c \end{matrix} \right\}$ c: neutral axis depth	$0.5c$	No requirements	No requirements
Maximum nominal shear stress	$v_n \leq 0.2 f'_c$ or 8 MPa	$v_n \leq \left\{ \begin{matrix} 0.2 f'_c \\ 1.1\sqrt{f'_c} \\ 9 \text{ MPa} \end{matrix} \right.$	$v_n \leq 0.2 f'_c$ or 6 MPa	$v_u \leq (0.8 + 4.6 \frac{H}{D}) \phi \sqrt{f'_c}$ $v_u \leq 5.4\phi \sqrt{f'_c}$ for $H/D < 1$ $v_u \leq 10\phi \sqrt{f'_c}$ for $H/D > 2$ $\phi = 0.85$	$v = \frac{f_c}{1 + \frac{h^2}{49t^2}}$
Concrete shear strength (simplified)	$V_c = \min \left\{ \begin{matrix} 0.17\sqrt{f} \\ 0.17 \left[\sqrt{f'_c} \right. \end{matrix} \right.$	$v_c = \min \left\{ \begin{matrix} 0.2\sqrt{f'_c} \\ 0.2 \left[\sqrt{f'_c} + \right. \end{matrix} \right.$	$v_c = \min \left\{ \begin{matrix} 0.2\sqrt{f'_c} \\ 0.2 \left[\sqrt{f'_c} + \frac{P_u}{A_g} \right] \end{matrix} \right\}$	The shear stress carried by the concrete shall not exceed:	No requirements

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970	NZS 1900:1964 (bylaw)
				$v_c = \left(3.7 - \frac{H}{D}\right) 2\phi\sqrt{f'_c}$ $v_c \leq 5.4\phi\sqrt{f'_c}$ for $H/D < 1$ $v_c \leq 2\phi\sqrt{f'_c}$ for $H/D > 2.7$ $\phi = 0.85$	
Shear reinforcement	$A_v = V_s \frac{s_2}{f_{yt} d}$	$A_v = \frac{(v_n - v_c) b_w s_2}{f_{yt}}$	$A_v = \frac{(v_n - v_c) b_w s_2}{f_{yh}}$	$A_v = \frac{V'_u s}{\phi f_y d \left(\frac{H}{D} - 1\right.}$	No requirements
Minimum shear reinforcement	$A_v = \frac{0.7\, b_w\, s_2}{f_{yt}}$	$A_v = \frac{0.7\, b_w\, s_2}{f_{yt}}$	$A_v = \frac{0.7\, b_w\, s_2}{f_{yh}}$	$A_v = \frac{V'_u s}{\phi f_y d}$ or Ratio (%): $\frac{9000}{f_y} \geq 0.18$	0.0025 (mild steel) 0.0018 (high tensile steel)
Maximum spacing of shear reinforcement	$\text{Min}\left(\frac{L_w}{5}, 3t, \text{or } 450\right.$	$\text{Min}\left(\frac{L_w}{5}, 3t, \text{or } 450\right.$	$\text{Min}\left(\frac{L_w}{5}, 3t, \text{or } 450 \text{ mm}\right)$	$2.5t, 18\text{in}$ (457 mm)	$2.5t$
Vertical reinforcement	$\rho_n \geq \frac{0.7}{f_{yn}}$ Spacing $\leq \min\{\frac{L_w}{3}, 3t,$ $450 \text{ mm}\}$	$\rho_n \geq \frac{0.7}{f_{yn}}$ Spacing $\leq \min\{\frac{L_w}{3}, 3t,$ $450 \text{ mm}\}$	$\rho_n \geq \frac{0.7}{f_{yn}}$ Spacing $\leq \min\{\frac{L_w}{3}, 3t,$ $450 \text{ mm}\}$	No requirements	No requirements
Maximum shear strength provided by the concrete in ductile detailing length	$V_c = \left(0.27\lambda\sqrt{f'_c} + \frac{N^*}{4A_g}\right) b_w d \geq 0.0$ $\lambda = 0.25DPR$ $\lambda = 0.5LDPR$	V_c shall not be taken larger than: $v_c = 0.6\sqrt{\frac{N^*}{A_g}}$ Total nominal shear stress shall not exceed: $v_n = \left(\frac{\phi_{ow}}{\mu} + 0.15\right)\sqrt{f'_c}$	V_c shall not be taken larger than: $v_c = 0.6\sqrt{\frac{P_e}{A_g}}$ Total nominal shear stress shall not exceed: $v_n = (0.3\phi_o S + 0.16)\sqrt{f'_c}$ S: structural type factor as defined by NZS 4203	No requirements	No requirements

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970	NZS 1900:1964 (bylaw)
Splicing of flexural tension reinforce-ment	One-third (*DPR*) and one-half (*LDPR*) of reinforcement can be spliced where yielding can occur	One-third of reinforcement can be spliced where yielding can occur	One-third of reinforcement can be spliced where yielding can occur	One-half of reinforcement can be spliced where yielding can occur	No requirements
Maximum compress-ive stress in concrete	No requirements	No requirements	No requirements	$\left[1 - \left(\frac{h}{35d}\right)^3\right]0.2$ h: distance between supports d: thickness of wall	Direct loading: $\frac{kf_{cu}}{k} = \frac{p}{5} - 0.007\frac{h}{t} + 0.2$ f_{cu}: minimum crushing strength P: total percentage of vertical reinforcement $0.25 \leq p \leq 0.5$ $\frac{h}{t} \geq 10$ Seismic bending + direct stress: $1.25k$
Maximum stress in the tensile steel	No requirements	No requirements	No requirements	No requirements	15000 psi for mild steel 20000 psi for the special types of reinforcement covered by the First Schedule hereto

Note:

NZS 3101P:1970 units of [psi]

Table C5E.5: Notation used in New Zealand standards for walls

Notation	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970*	NZS 1900:1964 (bylaw)
Design axial load at the ultimate limit state	N^*	N^*	P_u	N/A	N/A
The clear vertical distance between floors or other effective horizontal lines of lateral support, or clear span	L_n	L_n	L_n	h	h
Wall thickness	t, b	b	b	d, b	t
Effective length factor for Euler buckling	k_e	N/A	N/A	N/A	N/A
Effective length factor for flexural torsional buckling	k_{ft}	N/A	N/A	N/A	N/A
Horizontal length of wall	L_w	L_w	l_w	D	N/A
Thickness of boundary region of wall at potential plastic hinge region	b_m	b_m	N/A	N/A	N/A
Total height of wall from base to top	h_w	h_w	h_w	H	N/A
Aspect ratio of wall (h_w/L_w)	A_r	A_r	N/A	N/A	N/A
Yield strength of non-prestressed reinforcement	f_y	f_y	f_y	f_y	N/A
Yield strength of transverse reinforcement	f_{yh}	f_{yh}	f_{yh}	N/A	N/A
Yield strength of shear reinforcement	f_{yt}	f_{yt}	f_{yh}	f_y	N/A
Ratio of vertical (longitudinal) wall reinforcement area to gross concrete area of horizontal section	$\rho_n = \frac{A_t}{A_g}$	N/A	N/A	N/A	N/A
The ratio of vertical wall reinforcement area to unit area of horizontal gross concrete section	$\rho_l = \frac{A_s}{ts_v}$	$\rho_l = \frac{A_s}{bs_v}$	$\rho_l = \frac{A_s}{bs_v}$	N/A	N/A
Diameter of longitudinal bar	d_b	d_b	d_b	N/A	N/A
Centre-to-centre spacing of shear reinforcement along member	s	s	s	N/A	N/A
Computed distance of neutral axis from the compression edge of the wall section	c	c	c	N/A	N/A
A limiting depth for calculation of the special transverse reinforcement	c_c	c_c	c_c	N/A	N/A
Overstrength factor	ϕ_{ow}	ϕ_o	ϕ_o	N/A	N/A
Area of concrete core	$A_c{}^*$	$A_c{}^*$	$A_c{}^*$	N/A	N/A
Gross area of concrete section	$A_g{}^*$	$A_g{}^*$	$A_g{}^*$	N/A	N/A

Notation	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970*	NZS 1900:1964 (bylaw)
Dimension of concrete core of rectangular section measured perpendicular to the direction of the hoop bars to outside of peripheral hoop	h''	h''	h''	N/A	N/A
Centre-to-centre spacing of hoop sets	s_{h}	s_{h}	s_{h}	N/A	N/A
Structural type factor	----	----	S	N/A	N/A
Displacement ductility capacity relied on in the design	N/A	μ	N/A	N/A	N/A
Area used to calculate shear area	A_{cv}	N/A	N/A	N/A	N/A
Total nominal shear strength	V_{n}	V_{n}	V_{n}	N/A	N/A
Design shear force	V^*	V^*	V_{u}	V_{u}	N/A
Concrete shear strength	V_{c}	N/A	N/A	N/A	N/A
Nominal shear strength provided by shear reinforcement	V_{s}	N/A	N/A	N/A	N/A
Shear stress provided by concrete	v_{c}	v_{c}	v_{c}	v_{c}	N/A
Centre-to-centre spacing of horizontal shear reinforcement	$s2$	$s2$	$s2$	s	N/A

Appendix C5F Diaphragms Grillage Modelling/Analysis Methodology

C5F.1 Assessment Approach

For buildings that are essentially rectangular with relatively uniform distribution of vertical lateral force-resisting systems across the plan of the building, and no significant change of plan with height, simple, hand-drawn strut and tie solutions can be used.

However, buildings with significant asymmetry in the location of lateral force-resisting elements (distribution across the building plan, termination up the height of the building, varying stiffness and/or strength between vertical elements) may require a more sophisticated analysis. For these types of structures, a grillage method can be used to obtain diaphragm design actions. Details of a simple grillage method appropriate for design office use are given below (Holmes, 2015).

C5F.2 Grillage Section Properties

Grillage members are typically modelled as concrete elements, without reinforcement modelled, in an elastic analysis program. Figure C5F.1 illustrates a grillage model developed for a complicated podium diaphragm.

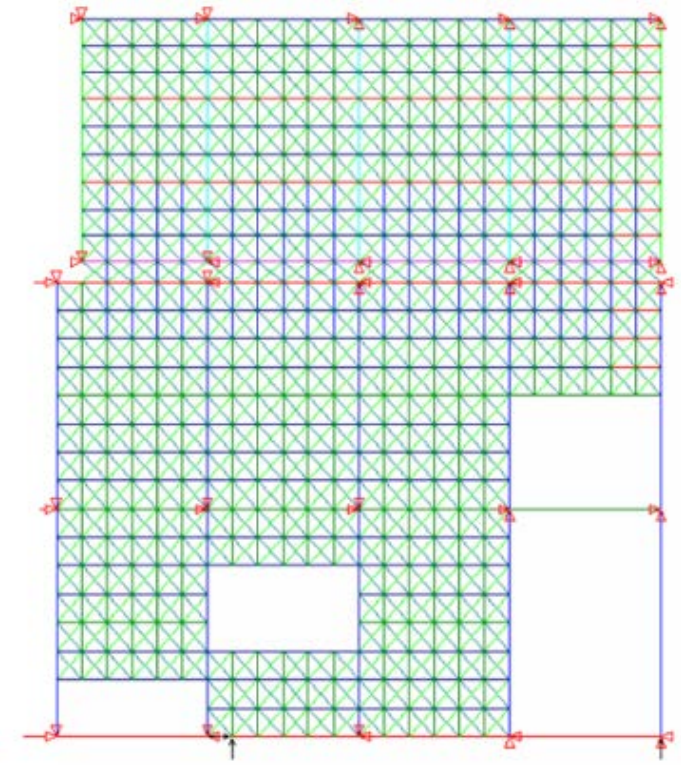


Figure C5F.1: Example of a grillage model for podium diaphragm (Holmes, 2015)

The recommended dimensions of the grillage elements for the modelling of a flat plate are based on work completed by Hrennikoff (1941), as shown in Figure C5F.2. This solution is based on a square grillage (with diagonal members). Rectangular grillages can also be used; the dimensions of the grillage beams will vary from those given for the square grillage solution (Hrennikoff, 1941).

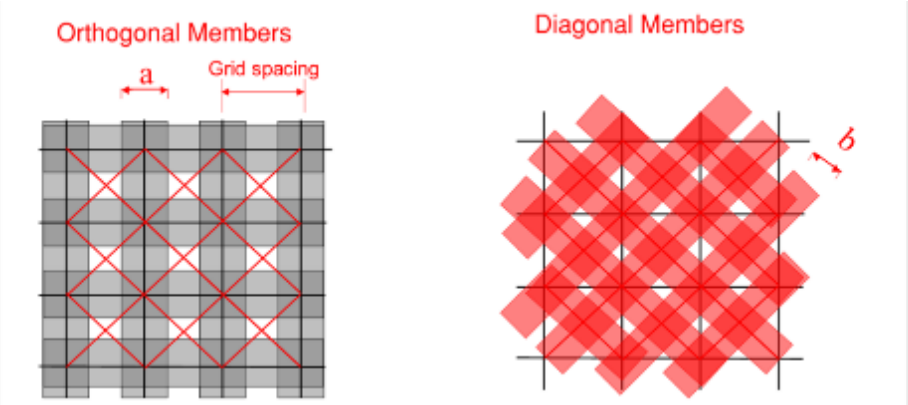


Figure C5F.2: Grillage beam dimensions for the square grillage (Hrennikoff, 1941)

Floors can be assumed to be uncracked for the purposes of diaphragm assessments. Given that diaphragms typically contain low quantities of longitudinal reinforcing steel and considering transformed section effects, it is not considered necessary to include longitudinal reinforcement when determining grillage section properties. An exception to this is the determination of the section properties for collector elements.

It is recommended that the effective stiffness of collector elements is based on the transformed section of the concrete plus:

- the bars reinforcing the collector element, or
- the structural steel beam acting in a collector.

Typically, when a collector is stretched and the strain in the steel approaches the yield strain, there will be significant cracking of the concrete that contributes to the collector. The effective stiffness of the collector, in tension, will reduce. However, for the typical steel contents of collector elements this reduction in stiffness is relatively small.

Note:

The collector is also typically required to resist compression forces due to the cyclic nature of seismic loading. Therefore, for modelling the collector element it is generally satisfactory to use either the transformed section of concrete and steel or the steel without the concrete. The combined concrete and steel option is stiffer than the steel-only option, so will attract more force.

C5F.3 Effective Width of Grillage Members

The recommended effective grillage member widths for orthogonal and diagonal members are as follows (Hrennikoff, 1941):

- Orthogonal members:
  - width A = 0.75 x grid spacing
  - carries both tension and compression forces
- Diagonal members:
  - width B = 0.53 x grid spacing
  - carries compression forces only.

C5F.4 Effective Thickness of Grillage Members

The recommended thickness of the grillage beams depends on the floor construction as follows:

- Hollow-core and Tee units:
  - parallel to the units: average thickness (per metre width) to match the combined areas of the topping plus unit
  - perpendicular to the units: the average thickness (per metre width) of the combined areas of the topping and the top flange of the units
- Rib and timber in-fill:
  - parallel to the ribs: average thickness (per metre width) of combined areas of the topping and ribs
  - perpendicular to the ribs: average thickness (per metre width) of the topping only
- In situ slabs and flat slabs:
  - combined thickness of the topping and units (if present) parallel and transverse to the units (if present)
- Steel profile composite floors:
  - parallel to the webs: average of cross-section flange and web
  - transverse to the webs: thickness of the flange
- Spaced hollow-core units with in situ slabs:
  - following the concepts above, the designer should rationalise the effective thickness, parallel and perpendicular to the units.

C5F.5 Spacing of Grillage Members

It is recommended that a grillage beam spacing of 1.0 m is typically adequate to produce reasonable distribution of forces (Gardiner, 2011). It is advisable to try larger and smaller grid spacings to determine if the model is sufficiently refined.

In general terms, the point of sufficient refinement for the grid spacing is when the actions reported in the beams of the grillage change very little from the previous trial.

In order to get a desirable, higher resolution of forces, grillage spacings should be reduced while maintaining the square format (divide the main square grillage into sets of smaller squares) for the following situations:

- Around the nodes where vertical structures (e.g. beams, columns, walls and eccentrically braced frames (EBFs)) would be connected to the floor plate. This applies to vertical elements, both on the perimeter of the floor as well as within the interior of the floor:
  - internal frames
  - frames, walls or EBFs, etc. next to floor penetrations (typically stairs, escalators and lifts)
- Around floor penetrations (typically stairs, escalators and lifts)
- At re-entrant corners in the floor plate

- For collectors, smaller sets of square grillages may be used either side of a collector (a grillage member with properties relevant to the collector performance). If a collector is relatively wide (say, greater than half the typical grillage spacing) consider modelling the collector as a small grillage/truss along the length of a collector, with the smaller set of squares either side of this.

C5F.6 Supports, Nodes and Restraint Conditions

The grillage is set up as a framework of struts. The junctions of the strut grillage framework are called "nodes". Floor inertia loads will typically be applied to all of the nodes of the grillage. Each vertical structural element will be associated with one or more nodes in the grillage as follows:

- Columns – typically a single node
- Walls – typically a number of nodes along the length of the wall.

The vertical translational degree of freedom of nodes which coincide with vertical structural elements (i.e. columns or wall elements) should be fixed. The horizontal translational degrees of freedom of these nodes should be left unrestrained. The reasons for this are as follows:

- Forces going in to or out of the nodes associated with the vertical elements are in equilibrium with the inertia and transfer or deformation compatibility forces within the floor plate.
- If the horizontal degree of freedom was fixed, the loads applied to these nodes would go directly to the support point and would not participate in the force distribution of the floor plate.
- Transfer or deformation compatibility forces are internal forces and must balance at the vertical supports and across the floor plate.

Note:

If all of the horizontal degrees of freedom are left unrestrained in a computer analysis model the analysis will not run. Therefore, it is recommended that two nodes are fixed; with both horizontal degrees of freedom fixed at one node and with fixity only in the direction of the applied inertia at the second node (i.e. free to move in the perpendicular direction).

C5F.7 Loss of Load Paths due to Diaphragm Damage

Modify the grillage to account for anticipated diaphragm damage/deterioration. For example, where floor to beam separation similar to that illustrated in Figure C5F.3 is anticipated due to beam elongation, the diagonal strut in the grillage should be removed recognising that the compression struts may not be able to traverse the damaged area (refer also to Figure C5F.3).

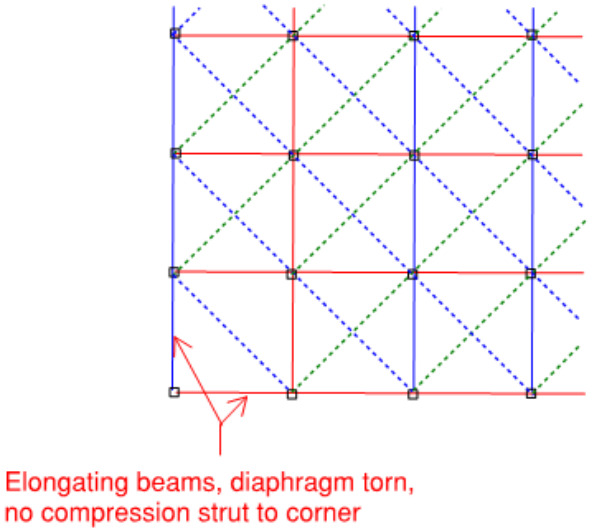


Figure C5F.3: Recommended grillage modelling at corner columns when frame elongation is anticipated (Holmes, 2015)

C5F.8 Application of Inertia Forces Introduced into the Grillage Model

Inertia of the floor, determined from pseudo-Equivalent Static Analysis (pESA) (refer to Section C2), is distributed over the framework of grillage elements, at the nodes of the orthogonal members of the grillage and in accordance to the tributary mass at each node:

- Tributary mass attributed to each node will include the seismic mass of the floor and any of the vertical structures attached to that node or nodes of the floor (i.e. walls, columns, beams, braces etc.).
- As a result of the "weighted" distribution of inertia associated with the appropriate mass attributed to each node, the distribution of inertia will not be uniform across the floor. There are concentrations of mass at frame lines, for example (beams, columns and cladding), and a more even distribution of inertia over the floor areas.
- Note that no inertia is placed where the diagonal member cross, because there is no node where the diagonal members pass. The diagonal members run between the nodes of the orthogonal grillage.

Inertia forces, applied to the structure, will be balanced by the forces at the supports/nodes of the floor plate. Other "internal" forces that balance the remaining portion of the forces at supports/nodes arise from deformation compatibility between the vertical structural systems being constrained to similar lateral displaced shapes. The largest of these compatibility forces are traditionally called "transfer" forces. Deformation compatibility forces occur in all buildings on all floors to varying degrees. All forces, applied and internal, must be in equilibrium.

C5F.9 Application of "Floor Forces"

Forces entering or leaving the floor where the floor is connected to the vertical lateral force-resisting structures have been called "floor forces", F_{Di}. Floor forces can be determined from the results of the pESA (refer to Section C2) and, as illustrated in Figure C5F.4, are equal to the difference in shears in vertical lateral load resisting elements above and below the diaphragm being assessed.

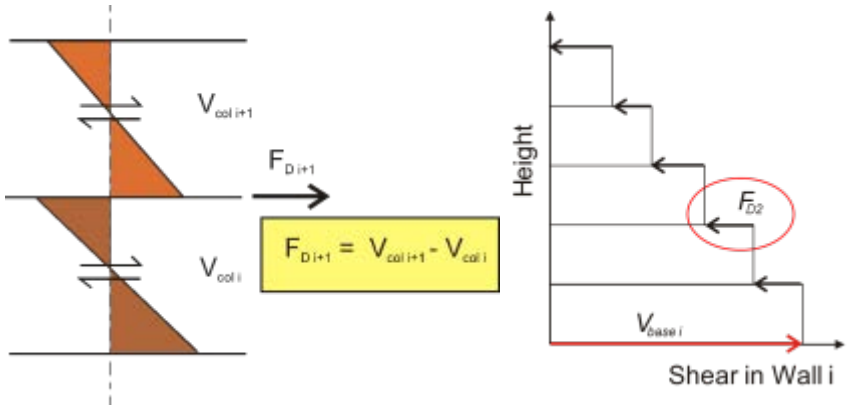


Figure C5F.4: Floor forces, F_{Di}, determined from pESA (Holmes, 2015)

It is important that members of the vertical lateral force-resisting systems in the pESA analysis model have in-plane and out-of-plane stiffness and that the analysis model has been enabled to report both major and minor axis actions of vertical elements.

Outputs for such elements should report actions in the X and Y directions. Therefore, for a given direction of earthquake attack, at each node there will be forces to be applied in the X and Y directions (refer to Figure C5F.5). Care is required to ensure that sign conventions (i.e. input and output of actions) are maintained.

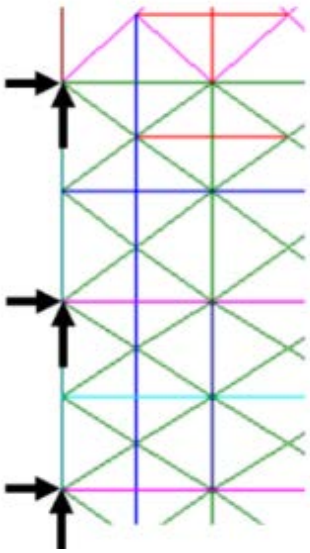


Figure C5F.5: Floor forces F_{Di} in both X and Y directions at nodes connected to vertical elements – for one direction of earthquake attack (Holmes, 2015)

C5F.10 Out-of-Plane Push and Pull of Vertical Elements

Vertical elements (i.e. walls, columns, braced frames) are pushed out-of-plane at some stage during a seismic event. Depending on the magnitude of the inter-storey drift demands, these elements may yield, exhibiting a permanent displacement out-of-plane. On reversal of the direction of seismic displacement, the element will need to be pulled back the other way (into the building). This action will subject the diaphragm to out-of-plane floor forces, $F_{\mathrm{OP,i}}$, which can be significant.

Consideration is required of when and where the push or pull forces develop. One side of a building has columns being pushed out of the building, while the other side is pulling the columns back in to the building.

A recommended methodology for assessing the out-of-plane forces, $F_{\mathrm{OP,i}}$, is as follows:

- Determine the out-of-plane displacement profile for a column, etc. from the pESA.
- Using a linear elastic analysis program impose this displacement profile on the element.
- Determine the out-of-plane bending moment at the base of the element. If the displacement is sufficient to yield the base of the element then scale the moments determined by the linear elastic analysis to the overstrength of the element base.
- Determine the shear force distribution for this overstrength moment.

At each floor level, the difference in this shear force distribution is to be added to the pESA model, which is then re-run and the out-of-plane forces, $F_{\mathrm{OP,i}}$, determined accordingly (i.e. taking the difference in out-of-plane shear in the vertical elements above and below the diaphragm being assessed).

C5F.11 Redistribution of Diaphragm Loads

It is probable that the reinforcing steel in the diaphragm may be insufficient to resist the tensions determined from the pESA.

One method to account for floor regions that may have yielding and to allow for a redistribution (plastic) of forces within the diaphragm is to adjust the section properties of the yielding members. Accordingly, adjust the stiffness of the yielding members until the yield forces are the outputs from the elastic pESA.

For each load case, it may take a couple of iterations to stabilise the redistribution of forces within the diaphragm.

For those situations when connections between the vertical lateral load resisting elements and the diaphragm are grossly overloaded (i.e. if very limited connectivity is provided) both the global building model (i.e. the analysis model used to assess the capacity of the vertical lateral load resisting elements) and the pESA analysis model may need to be adjusted so the affected vertical lateral load resisting elements are disconnected from the diaphragm.

Appendix C5G Deformation Capacity of Precast Concrete Floor Systems

C5G.1 General

Deformation demands of the primary lateral force-resisting systems can cause damage to the diaphragm structure (as a result of beam elongation or incompatible relative displacements between the floor and adjacent beams, walls or steel braced frames). Figures C5G.1 and C5G.2 illustrate two common examples of incompatible deformations between primary structure and a floor system.

Note:

The material in this section has largely been sourced from the University of Canterbury Research Report 2010-02 by Fenwick et al. (2010).

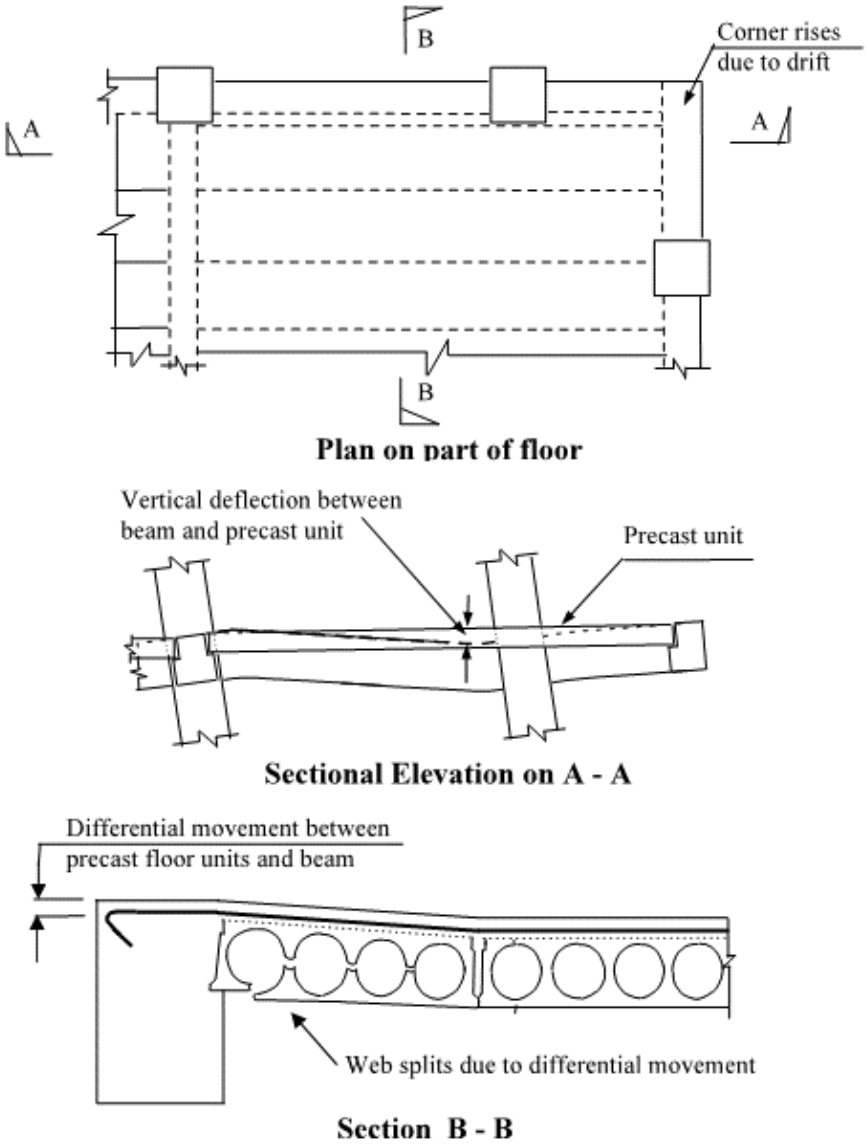


Figure C5G.1: Incompatible displacements between precast floor units and beams (Fenwick et al., 2010)

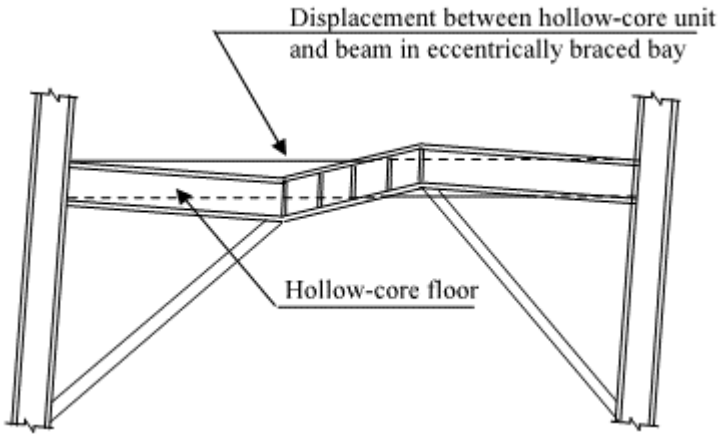


Figure C5G.2: Incompatible displacements between precast floor units and braced bay (Fenwick et al., 2010)

When present, precast concrete floor units effectively reinforce blocks of a diaphragm and concentrate any movement into cracks, which open up at the weak section between the floor and supporting structural elements. Where beams may form plastic hinges in a major earthquake, elongation within the plastic hinges can create wide cracks by pushing apart the beams or other structural components supporting the precast floor units. This can lead to the formation of wide cracks around the perimeter of bays of floor slabs containing prestressed precast units (refer to Figures C5G.3 and C5G.4).

Compression forces (struts) and tension forces (ties) may not be able to traverse damaged areas of floor. When assessing diaphragms, due allowance needs to be made for the loss of load paths, anticipating localised damage within the diaphragm.

Tests have shown that a wide crack does not develop where a linking slab is located between the first precast unit and a column in a perimeter frame – provided it does not have a transverse beam framing into it and the column is tied into the floor with reinforcement that can sustain the tension force given in NZS 3101:2006, clause 10.3.6 (Lindsay, 2004). Refer to Figure C5G.3(c).

C5G.2 Extent of Diaphragm Cracking

Figures C5G.3 and C5G.5(a) show the locations of wide cracks, which may limit strut and tie action in a floor. The length of these cracks round a perimeter frame (lines 1 and A in Figure C5G.5(a)) depends on the relative strength of the perimeter beams in lateral bending to the strength of reinforcement tying the floor into the beams. A method of assessing the lengths of these cracks is presented below.

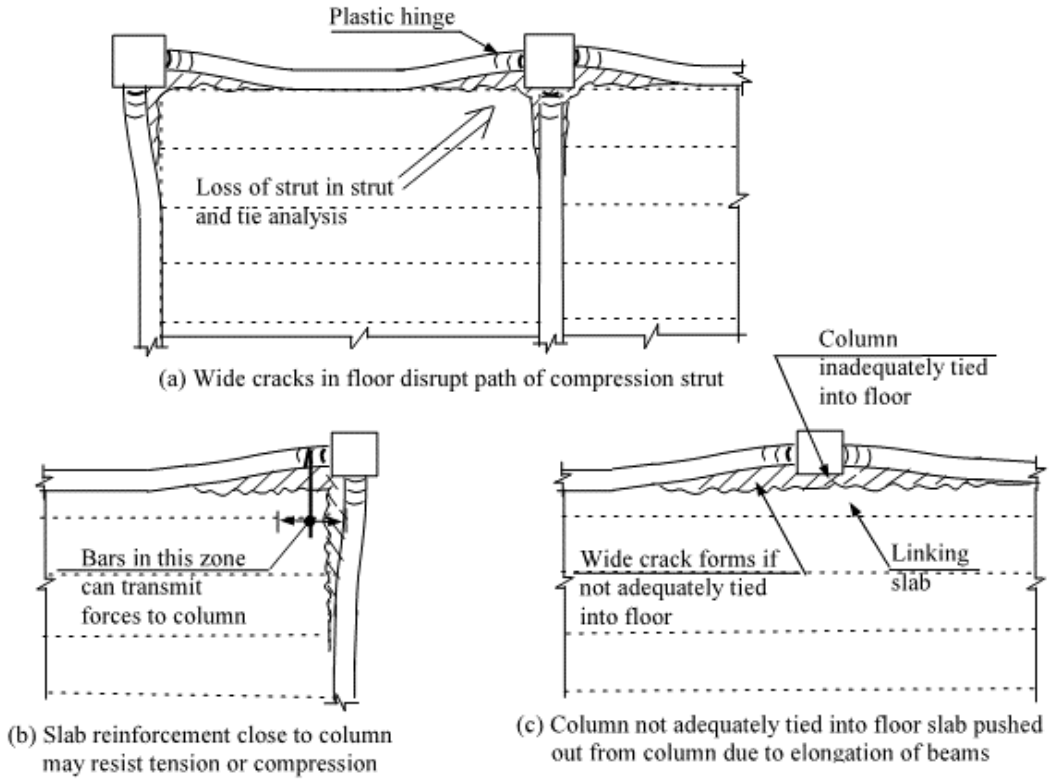
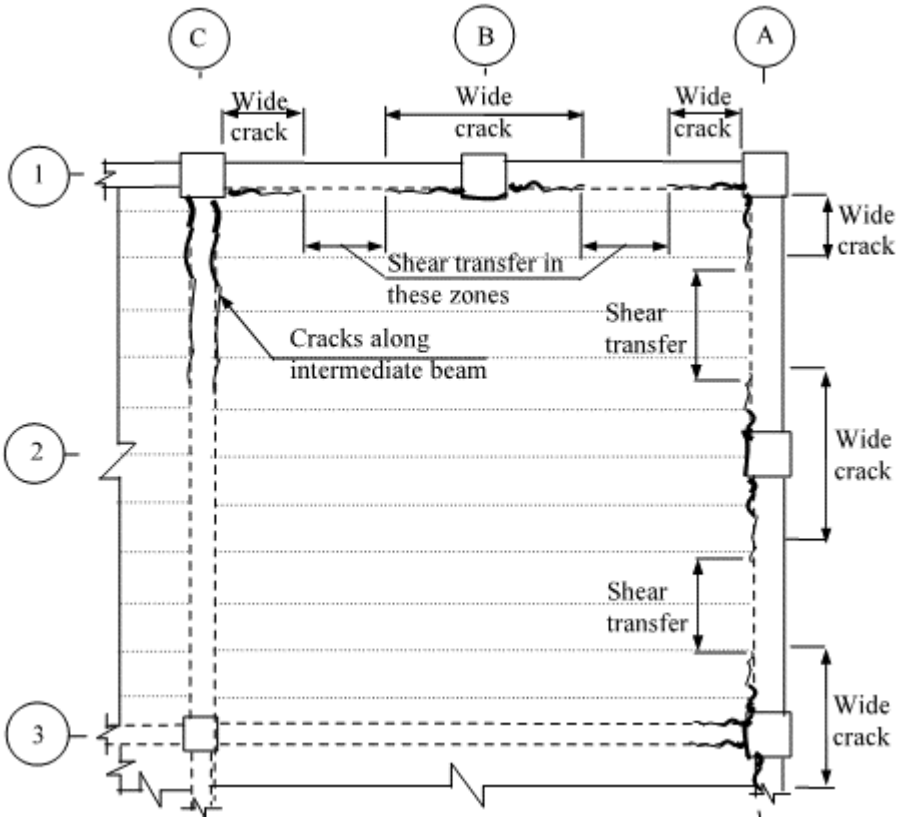


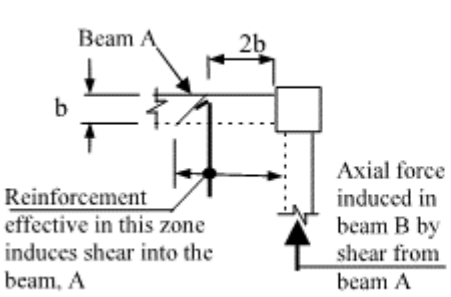
Figure C5G.3: Separation crack between floor and supporting beam due to frame elongation (Fenwick et al., 2010)

Figure C5G.4: Observed separation between floor and supporting beam due to frame elongation in 2011 Canterbury earthquakes (Des Bull)

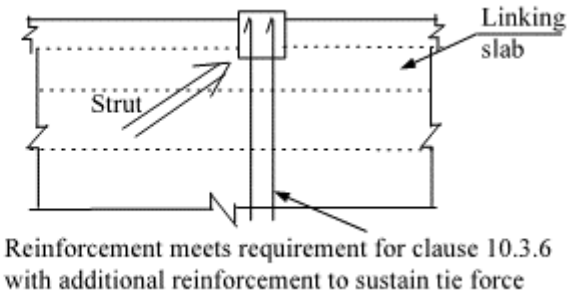
A wide crack is assumed to be one where the reinforcement tying the floor to a beam, or other structural element, has been yielded. In these zones shear transfer by conventional strut and tie type action is likely to be negligible.



(a) Plan on part of a floor showing areas where shear can be transferred to perimeter frames



(b) Effective zone for reinforcement acting near a column



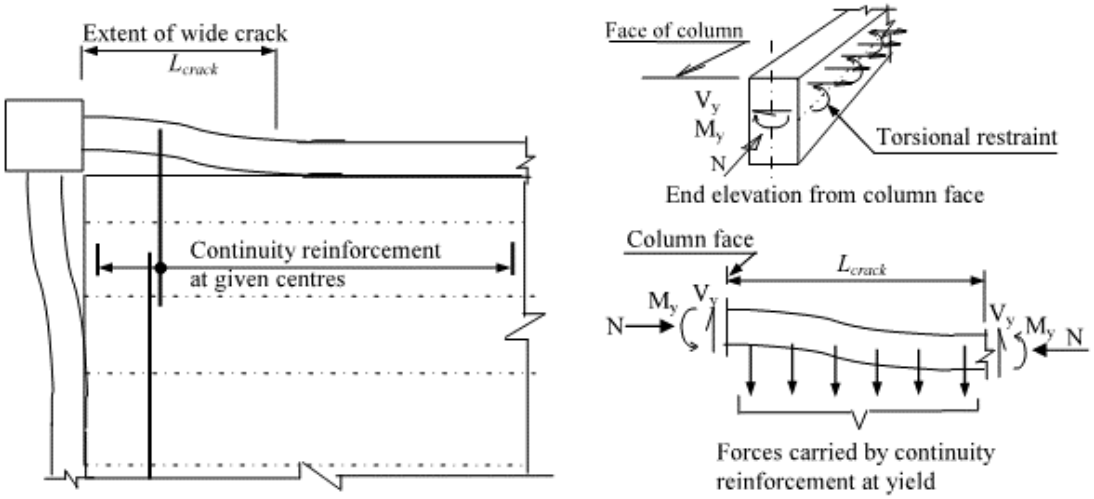
(c) Intermediate column acts as node for strut and tie forces to transfer shear to frame

Figure C5G.5: Location of cracks and strut and tie forces in a diaphragm (Fenwick et al., 2010)

The extent of cracking along an intermediate beam, such as the beam on line C in Figure C5G.5 depends on the relative magnitudes of inelastic deformation sustained in the perimeter frame (such as the frame on line 1) and an adjacent intermediate frame (such as frame on line 3 in Figure C5G.5(a)). Where the intermediate frame is flexible compared to the perimeter frame, extensive inelastic deformation together with the associated elongation may occur in the perimeter frame with no appreciable inelastic deformation in the intermediate frame.

C5G.3 Method for Assessing Crack Length

The length over which a wide crack may develop between a perimeter beam and an adjacent floor slab can be assessed from the lateral flexural strengths of the beam and the continuity reinforcement tying the floor to the beam. Figure C5G.6 shows the separation of a corner column due to elongation in beams framing into the column.



(a) Plan on floor showing separation of beams from floor

(b) Plan of length of wide crack

Figure C5G.6: Separation between floor and supporting beam (Fenwick et al., 2010)

The beams are displaced laterally, opening up a wide crack at the interface between the floor slab and beam such that the strain in the reinforcement tying the beam to the floor is in excess of the yield strain. The length of the wide crack is determined by the lateral strength of the beam. If the floor slab is assumed to provide restraint to torsion the critical length, L_{crack}, is given by:

$$L_{\text{crack}} = \sqrt{\frac{2M_{\text{o}}}{F}} \qquad \text{…C5G.1}$$

where:

M_{o} = flexural overstrength of beam about the vertical axis
F = yield force of continuity reinforcing per unit length

When calculating the flexural overstrength of the beam, M_{o}, the effects of strain hardening and axial load should be included. The axial load can be taken equal to the tension force carried by outstanding portion of the effective flange, i.e. the contribution of slab reinforcement to overstrength of plastic hinge region, as defined in NZS 3101:2006, clause 9.4.1.6.2.

Note that when the equation is applied to an intermediate column, where the precast floor units span past potential plastic hinges (such as column B on line 1 in Figure C5G.5) the axial load can be high and this can make a very considerable contribution to the flexural strength. In the calculation of M_{o} it should be assumed that the floor slab provides torsional restraint to the beam as this gives a conservative assessment both of the flexural strength and of the length of the wide crack.

C5G.4 Inter-storey Drift Capacity of Diaphragm Components

C5G.4.1 General

The assessment of inter-storey drift capacity of diaphragms containing precast concrete components needs to consider the following:

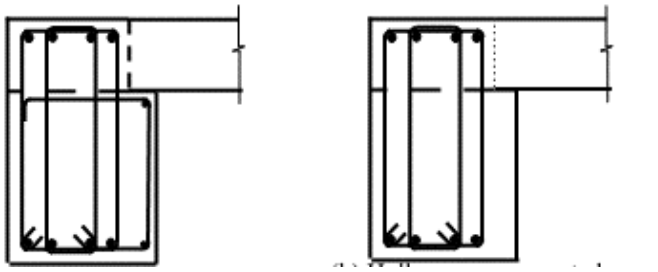
- loss of support of precast floor units, and
- failure of precast floor units due to seismic actions, including the consideration of incompatible displacements.

C5G.4.2 Loss of support

Overview

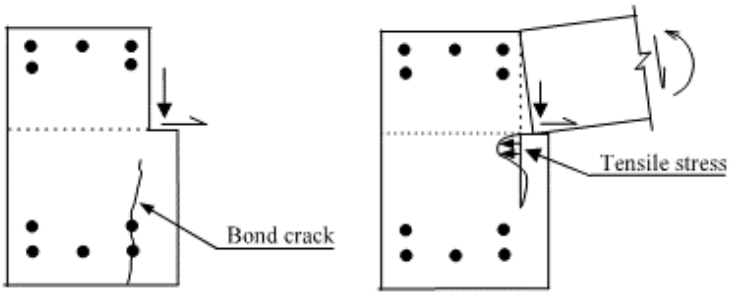
There are two key aspects to consider when assessing precast concrete floor units for loss of support:

- loss of support due to spalling of concrete near the front face of the support ledge and near the back of the precast floor unit, together with the movement of precast floor unit relative to the supporting beam, and
- loss of support due to failure of an unreinforced, or inadequately reinforced, supporting ledge Figure C5G.7(b). This may occur due to structural actions in the supporting elements, prying action of the precast floor unit on the support ledge, and the development of bond cracks associated with longitudinal beam reinforcing Figure C5G.8.



(a) Support ledge tied into beam **(b) Hollowcore supported on cover concrete**

Figure C5G.7: Support on concrete ledge tied into the supporting element or on cover concrete (Fenwick et al., 2010)



(a) Bond cracks **(b) Prying action**

Figure C5G.8: Bond cracks and tensile stresses due to prying action of precast floor units (Fenwick et al., 2010)

Loss of support does need not to be considered for a precast hollow-core floor unit if two cells at the end of the unit have been broken out and filled with reinforced concrete such that the yield force of the reinforcement exceeds twice the maximum shear force sustained by the unit. In addition, this reinforcement must be adequately anchored to sustain the yield force both in the hollowcore cells and in the supporting beam.

When assessing loss of support due to spalling and relative movement the methodology in Section C2 should be followed. When assessing the adequacy of existing seating widths for loss of the support the following needs to be considered:

- inadequate allowance for construction tolerance
- movement of precast floor unit units relative to the ledge providing support due to elongation and rotation of support beams
- spalling of concrete from the front face of support ledge and back face of the precast floor unit
- creep, shrinkage and thermal movement of the floor, and
- crushing of concrete resisting the support reaction due to bearing failure.

Allowances for each of these actions are detailed below.

Inadequate allowance for construction tolerance

In general, precast units have been constructed on the short side to reduce problems in placing the units on supporting beams. In an assessment, ideally the construction tolerance should be measured. Where these measurements are not available it is recommended that a construction tolerance of 20 mm is assumed. This gives an initial contact length between the precast floor unit and support ledge of the dimensioned length of the support ledge minus 20 mm.

Relative movement of floor unit due to elongation and rotation

Elongation of plastic hinges can push beams supporting precast floor units apart and reduce the contact length between the precast units and support ledge. However, as elongation is related to the mid-depth of the beam containing the plastic hinge it is also necessary to allow for further movement between precast units and support ledge due to rotation of the supporting beam (i.e. geometric elongation) as illustrated in Figure C5G.9.

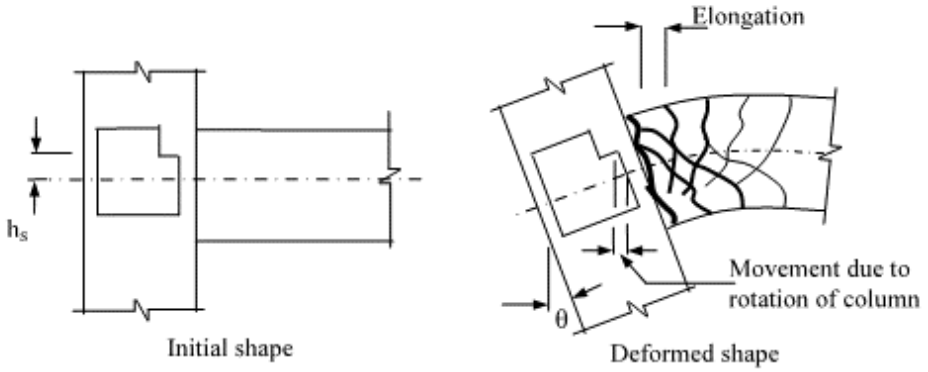


Figure C5G.9: Displacement at support of precast unit due to elongation and rotation of support beam (Fenwick et al., 2010)

Displacement of structural members due to frame elongation can be calculated using the following procedure, which is based on experimental measurements. Experimental testing on structures with hollow-core floor units (Fenwick, et al., 1981; Mathews, 2004; MacPherson, 2005; Lindsay, 2004) has demonstrated that frame elongation is partially restrained by precast concrete floor units when they span parallel to the beams. Figure C5G.10 illustrates three plastic hinge elongation types.

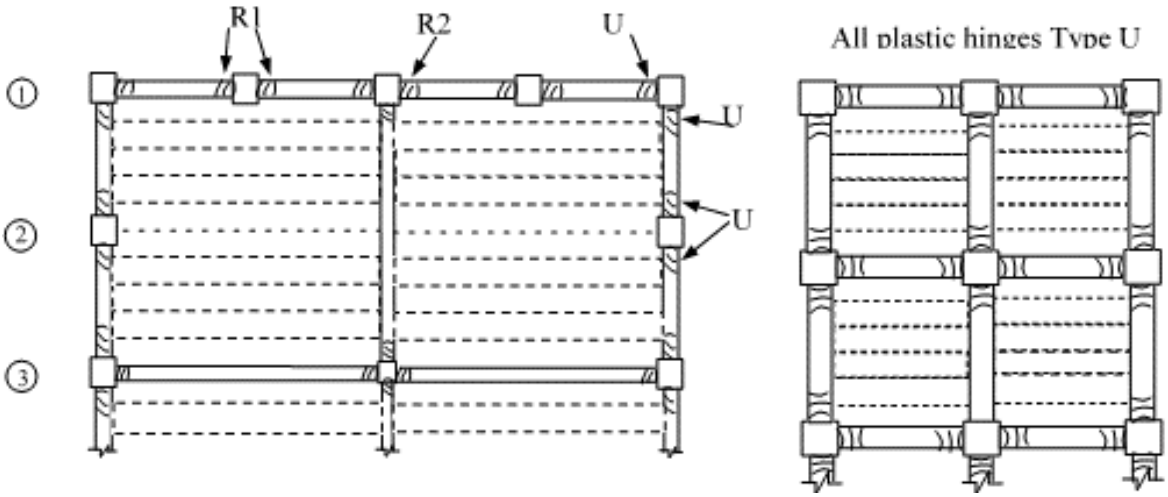


Figure C5G.10: Part plan of floor showing plastic hinge elongation types U, R1 and R2 (Fenwick et al., 2010)

For type U and R1 plastic hinges little restraint is provided by the floor slab and the elongation at mid-depth of the beam, Δ_L, can be calculated as:

$$\Delta_L = 0.0014 h_b \frac{\phi_u}{\phi_y} \leq 0.037 h_b \qquad \text{…C5G.2}$$

where:

h_b = beam depth
ϕ_y = beam first yield curvature
ϕ_u = ultimate curvature demand on beam determined using plastic hinge lengths specified in Section C5.5.2.2.

For type R2 plastic hinges where there is a transverse beam framing into the column the elongation at mid-depth of the beam, Δ_L, can be calculated in accordance with Equation C5G.3 where the terms are as defined above:

$$\Delta_L = 0.0007 h_b \frac{\phi_u}{\phi_y} \leq 0.02 h_b \qquad \text{…C5G.3}$$

Equations C5G.4 and C5G.5 are applicable to reinforced concrete beams that are sustaining inelastic deformations. Some recoverable frame elongation can still be expected at yield. Pending further study, a value in the order of 0.5% beam depth is considered appropriate for assessing the performance of nominally ductile frames.

Geometric elongation associated with movement between precast units and support ledge due to rotation of the supporting beam can be calculated as:

$$\Delta_g = \left(\frac{h_b}{2} - h_L\right)\theta \quad \text{…C5G.4}$$

where:

h_b = beam depth
h_L = ledge height (i.e. vertical distance between top of beam and height at which precast floor unit is supported)
θ = beam rotation.

Total movement of precast floor unit units relative to the ledge providing support due to elongation and rotation of support beams, Δ_{rot}, is calculated as:

$$\Delta_{rot} = \Delta_L + \Delta_g \quad \text{…C5G.5}$$

where:

Δ_L and Δ_g are as defined above.

Spalling at support

Spalling of unarmoured concrete occurs from the front of the support ledge and the back face of the hollowcore units, reducing the contact length available to support the precast units. Tests have indicated that the loss in seating length due to spalling and prying action of precast units increases with the contact length between the unit and support ledge. Assessed loss due to spalling, Δ_{spall}, is given by:

$$\Delta_{spall} = 0.5L_s \leq 35\ mm \quad \text{C5G.6}$$

where:

L_s is the initial contact length between precast unit and support ledge.

Where a low friction bearing strip has been used the value given by Equation C5G.6 can be reduced by multiplying it by 0.75.

Spalling does not need to be consider if both the unit and the ledge are armoured.

Creep, shrinkage and thermal actions

Shortening of a precast floor unit due to creep, shrinkage and/or thermal strains may occur at either or both of the supports. Once a crack has been initiated at one end it is possible that all the movement in the span will occur at that end. Hence, two limiting cases should be considered: all the movement occurs at the end, or no movement occurs at the end.

Opening up a crack due to creep and shrinkage movement reduces the shear transfer that can develop across the crack. This reduces the potential prying action of the hollowcore unit on the beam. In this situation the reduction in prying action can either reduce or eliminate the spalling that occurs from the back face of the hollowcore unit.

Note:

In recognition of this action, the calculated movement due to creep, shrinkage and thermal strain is not added to the loss of length due to spalling. The greater loss in contact length due to spalling or to creep, shrinkage and thermal strain is assumed to apply.

For practical purposes it is recommended that the loss in support length due to creep, shrinkage and thermal strain may be taken as 0.6 mm per metre of length of the precast unit.

Bearing failure

Sufficient contact length should remain between each hollowcore unit and the supporting ledge, after allowance has been made for the loss of supporting length identified above, to prevent crushing of concrete due to this reaction.

The critical reaction is likely to arise due to gravity loading plus the additional reaction induced by vertical seismic movement of the ground. The required bearing area can be calculated from the allowable bearing stress in NZS 3101: 2006, clause 16.3.

C5G.4.3 Failure of precast floor units

When assessing the capacity of a precast floor unit the following potential failure modes need to be considered:

- positive moment failure near support
- negative moment failure near support
- shear failure in negative moment zones
- incompatible displacements between precast floor units and other structural elements, and
- torsional failure of precast floor units.

Consideration of vertical seismic loading, calculated using Section 8 of NZS 1170.5:2004, should be included. Detailed guidance on how to assess the above failure modes for floors with precast concrete hollow-core units is provided in the University of Canterbury Research Report 2010-02 by Fenwick et al. (2010). Similar principles can be used to assess the performance of other types of precast concrete floor units.

Appendix C5H Buckling of Vertical Reinforcement and Out-of-Plane Instability in Shear Walls

This appendix outlines a possible approach to assessing buckling of reinforcing bars in RC elements with emphasis on shear walls. It also provides background information on the out-of-plane instability of shear walls.

C5H.1 Buckling of Vertical Reinforcement

Please note that although there has been a significant amount of research into this phenomenon (Mander et al., 1984; Mau and El-Mabsout, 1989; Mau, 1990; Pantazopoulou, 1998; Rodriguez el al., 1999; Bae et al., 2005; Urmson and Mander, 2011; Rodriguez et al. 2013), guidance for assessing existing buildings is currently limited.

In particular, the effect of the cycles (reflected in the dependence of the critical strain at the onset of buckling ($\varepsilon_{s,cr}$) on the maximum tensile strain experienced by the bar before the cycle reversal takes place (ε_{st}) has not been incorporated in design or assessment codes or standards.

Note:

An illustration of this phenomenon and a possible definition of the buckling critical strain is shown in Figure C5H.1 with reference to a schematic strain profile in the critical section and to the stress-strain hysteresis loop of a bar located close to the extreme fibre of the wall section. Four stress-strain states (1-4) are described.

The maximum tensile strain reached in the first part of the cycle is identified as point 1 ($\varepsilon_s = \varepsilon_{st}$, $f_s = f_{st}$). Two strain levels are used for the same point, representing large and moderate initial elongations of the steel: ε_{st} = 4.0% and ε_{st} = 2.5%, respectively. If the failure is not reached at this point and the strain reversal occurs, the steel follows the descending branch of the hysteresis loop from point 1 to the zero stresses point 2 ($\varepsilon_s = \varepsilon_0^+$, $f_s = 0$).

The strain associated with point 2, ε_0^+, can be estimated using Equation C5H.1 where f_{st} is the stress in the steel at maximum elongation (point 1) and E_s is the modulus of elasticity of the steel. In the non-trivial case, where the steel has entered the inelastic range in tension, f_{st} can be conservatively taken as f_y. As a result, Equation C5H.1 becomes Equation C5H.2

$$\varepsilon_0^+ = \varepsilon_{st} - f_{st}/E_s \quad \text{...C5H.1}$$

$$\varepsilon_0^+ = \varepsilon_{st} - \varepsilon_y \quad \text{...C5H.2}$$

From point 2 towards point 3, the zero strain point (ε_s = 0, fs < 0), the bar is subjected to compression stresses, but it remains under tensile strains. If point 3 can be reached (i.e. the bar does not buckle beforehand), the bar can withstand increasing compression strains until point 4 is reached; the point where the onset of buckling occurs (Rodriguez et al., 1999, 2013). The horizontal distance between points 2 and 4, ε_p^*, can be calculated with

Equation C5H.3 (Rodriguez et al., 2013), as a function of the restraining ratio s_v/d_b. If the critical buckling strain, $\varepsilon_{s,cr}$, is defined with reference to the zero strain axis, it can be calculated with Equation C5H.4.

$$\varepsilon_p^* = \frac{11-(5/4)(s_v/d_b)}{100} \quad \text{...C5H.3}$$

$$\varepsilon_{s,cr} = \varepsilon_{st} - \varepsilon_y - \varepsilon_p^* \quad \text{...C5H.4}$$

Consider, as an example, the damage developed at the free end of the walls W1 and W2 presented in Figure C5H.1. These walls were part of two buildings constructed in Christchurch and were damaged during the 22 February 2011 earthquake.

In Figure C5H.1(d), it can be observed that the spacing of the confinement hoops used in W1 was large (about 300 mm), and the restraining ratio was of the order of s_v/d_b. = 17, as the vertical bar had a diameter d_b = 18 mm. However, for such a large restraining ratio the formula proposed by Rodriguez et al. (2013) is no longer valid and buckling will inevitably occur before the zero strain point can be reached. That point is represented by point 5.

As shown in the same figure, in W2 the confinement hoops were spaced at a much smaller distance, preventing the vertical bar from buckling and effectively confining the concrete.

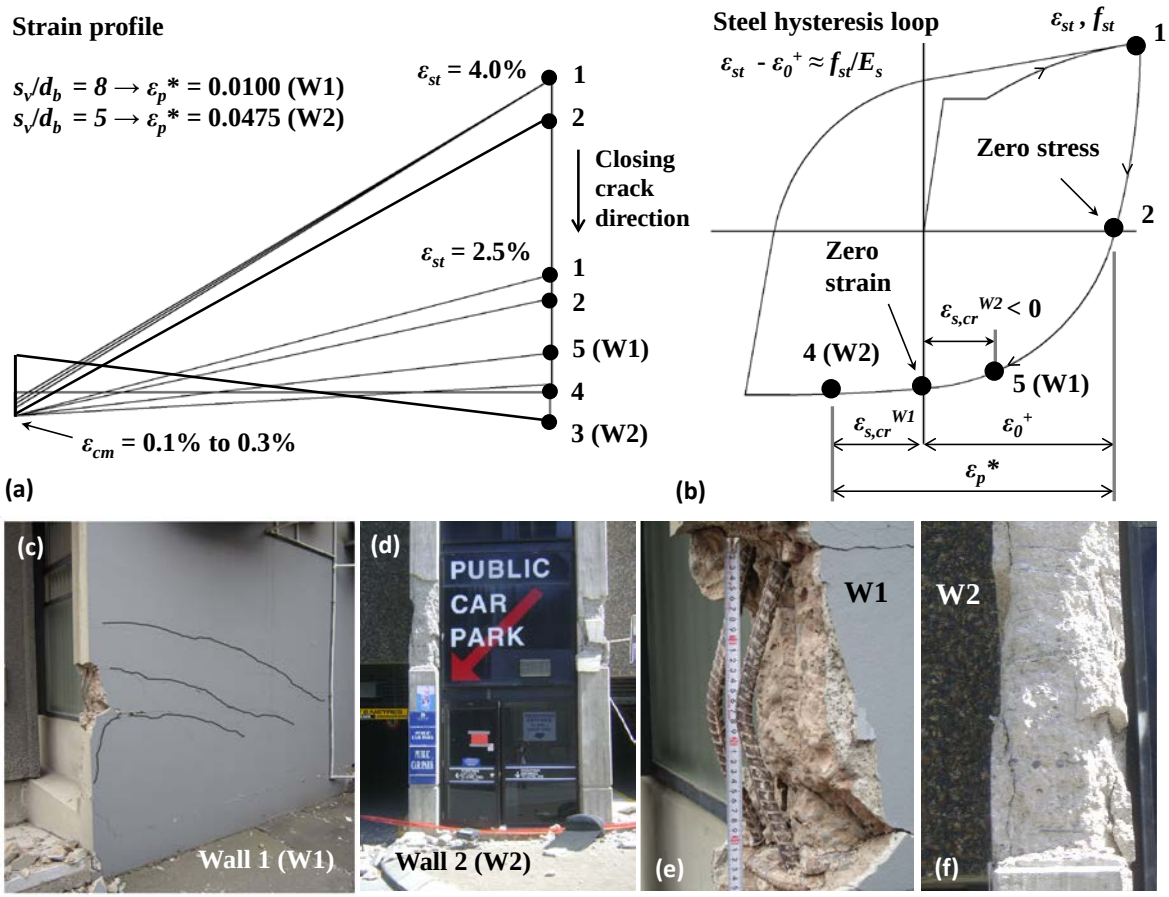


Figure C5H.1: Buckling critical strain definition (Quintana-Galo et al. 2016)

A strain limit for buckling to be used in monotonic moment-curvature analysis can be established in two ways.

- The first approach is to use the maximum strain associated with the ultimate curvature, ε_{sm} to obtain $\varepsilon_{s,cr}$, such that $\varepsilon_{st} = \varepsilon_{sm}$ in Equation C5H.5. The strain $\varepsilon_{s,cr}$ should be compared with the maximum compression strain in the inverse direction of the moment. If the section is symmetric in geometry and reinforcement, $\varepsilon_{s,cr}$, can be compared with the maximum compression strain in the concrete at ε_{cu} or $\varepsilon_{cu,c}$ as corresponds, which is the strain that governs in most of the cases. If the maximum strain of the steel ε_{sm} controls, as occurs in members with large flanges acting in tension, there is no need to check for buckling as the reversal cannot occur. For large values of ε_{sm} smaller, but closer to 6%, $\varepsilon_{s,cr}$ takes positive values, indicating that buckling will occur while the bar experiences tensile strains (point 5 in Figure C5H.1).
- The second approach is to set the ultimate strain of the concrete as $\varepsilon_{cm} = \varepsilon_{cu}$ or $\varepsilon_{cu,c}$ as corresponds, and calculate the maximum tensile strain $\varepsilon_{st} = \varepsilon_{su,b}$, which is the maximum tensile strain that a bar can develop such that buckling of that bar under reversed bending actions occurs at the same time than crushing of the concrete. Setting $\varepsilon_{s,cr} = -\varepsilon_{cm}$, and $\varepsilon_{st} = \varepsilon_{su,b}$ in Equation C5H.5:

$$\varepsilon_{su,b} = -\varepsilon_{cm}^{r} + \varepsilon_{y} + \varepsilon_{p}^{*} \qquad \text{…C5H.5}$$

  The superscript r is used to indicate that this concrete strain corresponds to the cross section under reversed actions.

Note:

As a general rule, if the spacing of confinement stirrups is greater than $7d_b$, as is typical of older construction practice, buckling is likely to control the capacity of the member, as the reinforcement bar after buckling does not follow a stable stress-strain path in compression (Mau, 1990).

Typical stress-strain curves for different values of s_v/d_b (6.5, 10 15) are presented in Figure C5H.2 (Mau and El-Mabsout, 1989). Figure C5H.3 shows the maximum compression normalised stress and the lateral displacement of the bar for different restraining ratios.

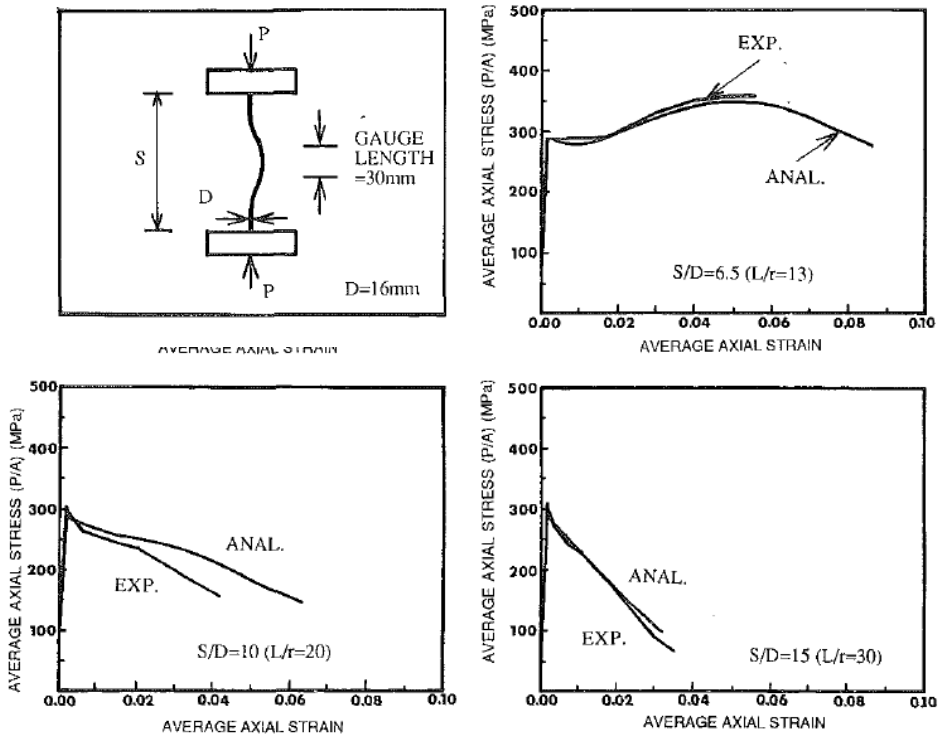


Figure C5H.2: Stress-strain curves of a steel bar in compression for s_v/d_b = 6.5, 10 and 15 (Mau and El-Mabsout, 1989)

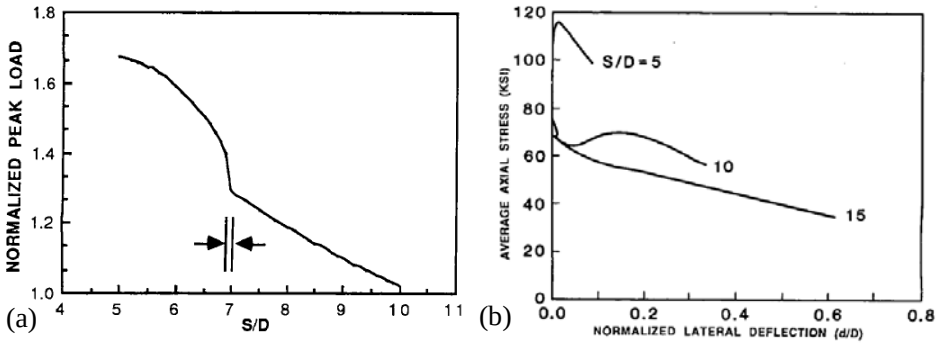


Figure C5H.3: (a) normalised peak load (relative to buckling stress) and critical restraining ratio s_v/d_b = 7, (b) lateral displacement of the bar for different restraining ratios s_v/d_b (Mau, 1990)

Note:

An indicative limit of the buckling strain limit in the steel rebars can be taken as the maximum tension strain in the steel that, given the s/d_b ratio, will produce buckling at a compression strain equal to the maximum (ultimate) strain of the concrete.

As an example, for s/d_b= 6, ε_y= 0.25%, and assuming a well confined concrete ε_{cu}= 1%, the maximum strain in the steel governing the buckling would be about 3%.

Considering that older construction details are likely to be worse than the assumed value of s/d_b= 6 and ε_{cu} = 1% this would suggest that ε_{su}= 3% represents a simplistic upper limit to be adopted to account for buckling effects in section analysis. This is instead of $\varepsilon_{s,max}$= 6% assumed in Table C5.8 considering a flexural-dominated and ideal behaviour.

Based on Equation C5.9, the **probable curvature at the onset of buckling, $\phi^*_{\mathbf{prob}}$,** and the corresponding plastic displacement, δ^*_p, can be estimated using Equations C5H.6 and C5H.7 respectively.

$$\phi^*_{\mathrm{prob}} = \frac{\varepsilon^*_{\mathrm{p}}}{\gamma l_{\mathrm{w}}} \quad \text{...C5H.6}$$

$$\delta^*_{\mathrm{prob}} = L_{\mathrm{p}}\left(\phi^*_{\mathrm{prob}} - \phi_{\mathrm{y}}\right)\left(h_{\mathrm{w}} - 0.5L_{\mathrm{p}}\right) \quad \text{...C5H.7}$$

where:

γl_{w} is shown in Figure C5H.4.

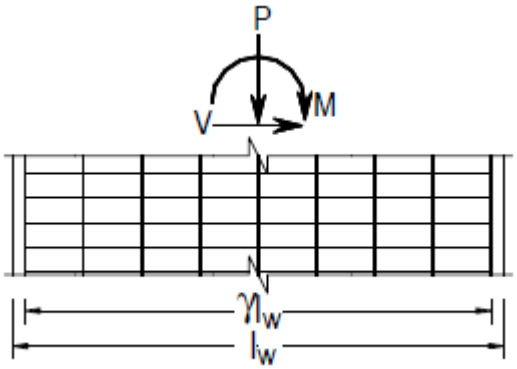


Figure C5H.4: Definition of γl_{w} according to Rodriguez et al. (2013)

C5H.2 Out-of-plane Instability

Out-of-plane (or lateral) instability is currently identified as one of the common failure modes of slender rectangular RC walls. This 'global' mode of failure, which involves a large portion of a wall element as opposite to the 'local' bar buckling phenomenon where a single rebar is affected, was previously observed in experimental studies of rectangular walls. However, it was not considered as a major failure pattern until the recent earthquakes in Chile (2010) and Christchurch (2011).

Note:

Following the Canterbury earthquake sequence extensive numerical and experimental investigations are being carried out to scrutinise the effect of key parameters assumed to be influential in the formation of out-of-plane instability, such as residual strain and peak tensile strain at previous cycle, wall slenderness ratio, wall length, axial load ratio and cumulative inelastic cycles experienced during the earthquake.

The final aim is to develop recommendations consistent with the approach followed in this document and integrate this failure mode within the derivation of the force-displacement capacity curve of the assessed wall.

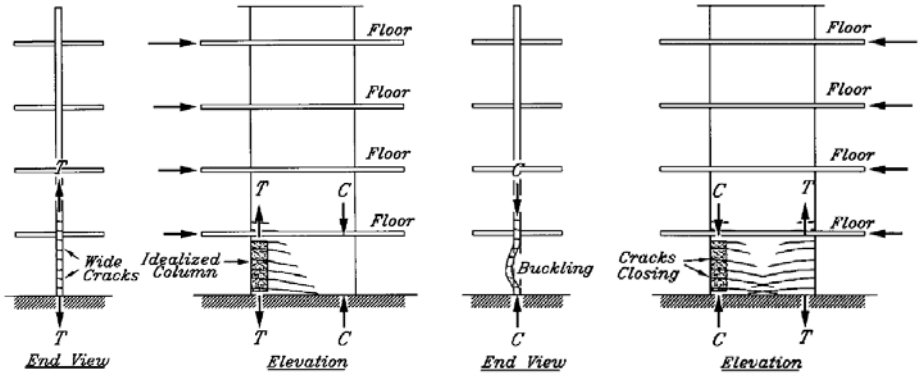
For more detailed information and preliminary results refer to Dashti et al. (2015, 2016).

Note:

Paulay and Priestley (1993) made recommendations for the prediction of the onset of out-of-plane instability based on the observed response in tests of rectangular structural walls and theoretical considerations of fundamental structural behaviour.

Because of very limited available experimental evidence, engineering judgement was relied on extensively. It was concluded that properties for inelastic buckling are more affected by wall length than by unsupported height and the major source of the instability was postulated to be the tensile strain previously experienced by the rebar rather than the maximum compression strain.

Chai and Elayer (1999) studied the out-of-plane instability of ductile RC walls by idealising the end-region of the wall as an axially loaded reinforced concrete column, as shown in Figure C5H.5. They conducted an experimental study to examine the out-of-plane instability of several reinforced concrete columns that were designed to represent the end-regions of a ductile planar reinforced concrete wall under large amplitude reversed cyclic tension and compression.



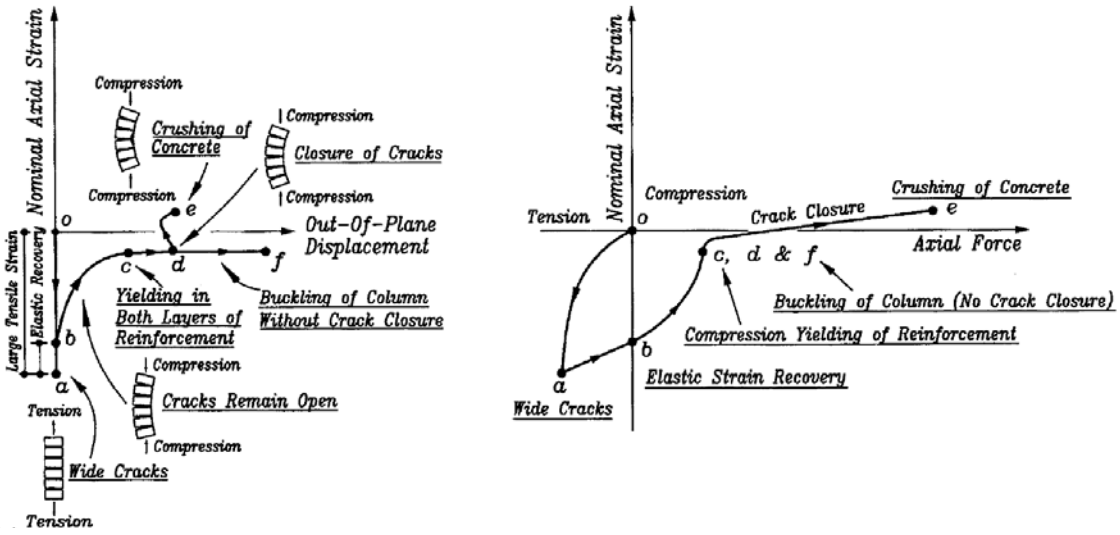
(a) Opening of cracks under tension cycle

(b) Closing of cracks under compression cycle

Figure C5H.5: Idealisation of reinforced concrete wall in end regions: (a) opening of cracks under tension cycle; and (b) closing of cracks under compression cycle (Chai and Elayer, 1999)

Note

Based on this study, the critical influence of the maximum tensile strain on the lateral instability of slender rectangular walls was confirmed and the basic behaviour of the wall end-regions under an axial tension and compression cycle was described by axial strain versus out-of-plane displacement and axial strain versus axial force plots, as shown in Figure C5H.6. Also, based on a kinematic relation between the axial strain and the out-of-plane displacement, and the axial force versus the axial strain response, a model was developed for the prediction of the maximum tensile strain. Points (a) to (f) display different stages of the idealised column response and are briefly described in Table C5H.1.



(a) nominal axial strain versus out-of-plane displacement

(b) nominal axial strain versus axial force

Figure C5H.6: Axial reversed cyclic response of reinforced concrete slender wall (Chai and Elayer, 1999)

Table C5H.1: Behaviour of wall end-region under the loading cycle shown in Figure C5H.6

	Loading	Unloading	Reloading			
Path	**o-a**	**a-b**	**b-c**	**c-d**	**d-e**	**d-f**
	Large tensile strain	Elastic strain recovery mainly in reinforcing steel	Reloading in compression on the cracked concrete column accompanied by an out-of-plane displacement; yielding of the reinforcement closer to the applied axial force resulting in a reduced transverse stiffness of the column and an increased out-of-plane displacement	Compression yielding in the second layer of the reinforcement, and a rapid increase in the out-of-plane displacement	Closure of cracks at point d and decrease of out-of-plane displacement and increase of out-of-plane displacement after significant compressive strain is developed in the compressed concrete	An excessive crack opening where subsequent compression would not result in the closure of the cracks but a continued increase in the out-of-plane displacement and eventual buckling of the column

As can be seen in Figure C5H.6 and Table C5H.1, the idealised column was assumed to consist of the loading stage where a large tensile strain was applied to the specimen (Path o-a), the unloading branch (Path a-b) corresponding to elastic strain recovery mainly in reinforcement steel and the reloading in compression which can be either Path b-c-d-e or Path b-c-d-f.

During Path b-c, when the axial compression is small, the compressive force in the column is resisted entirely by the reinforcement alone as the cracks are not closed, and a small out-of-plane displacement would occur due to inherent eccentricity of the axial force. The increase in axial compression would lead to yielding of the reinforcement closer to the

applied axial force resulting in a reduced transverse stiffness of the column and an increased out-of-plane displacement.

Path c-d corresponds to compression yielding in the second layer of the reinforcement due to further increase in the axial compression which could rapidly increase the out-of-plane displacement. Response of the idealised column after Point d depends on the initial tensile strain. If the initial tensile strain is not excessive, the cracks could close at Point d resulting in decrease of out-of-plane displacement (Path d-e). The crack closure would cause significant compressive strain to develop in the compressed concrete accompanied by increase of out-of-plane displacement. In case of excessive crack opening, the following compression would not be able to close the cracks before the increase in the out-of-plane displacement results in eventual buckling of the column.

Appendix C5I Procedure for Evaluating the Equivalent "Moment" Capacity of a Joint, M_j

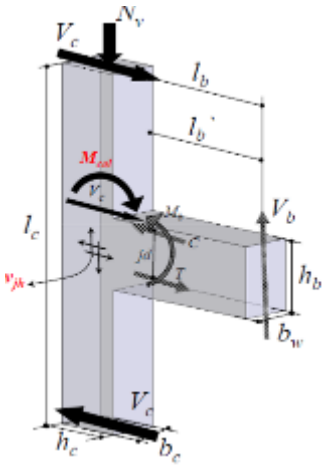
In order to compare the hierarchy of strength and determine the expected sequence of events within beam–column joint subassemblies (refer to Section C5.6.1 for the full procedure) the joint shear capacity can be expressed as a function of a comparable parameter to the capacity of beams and columns.

As a benchmark parameter, it is suggested to take an equivalent moment in the column (based on equilibrium considerations).

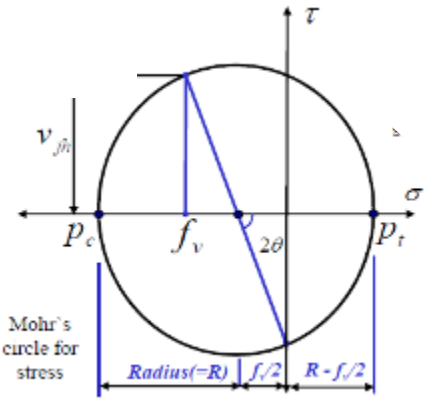
In Table C5I.1 and Figure C5I.1 below the probable shear force $V_{\mathrm{prob,jh}}$ is expressed as a function of the moment in the column, leading to the expression of M_{col} as the equivalent moment in the column corresponding to the given joint parameter.

Table C5I.1: Step-by-step procedure to express the joint capacity as a function of equivalent column moment M_j or M_{col}

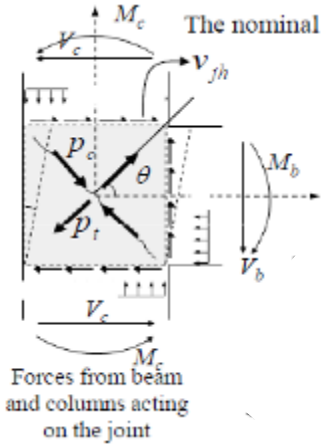
Horizontal shear force acting on the joint core	$V_{jh} = T - V_c$	…C5I.1
Equilibrium of the external action	$V_c l_c = V_b l_b$	…C5I.2
Rearrange to get V_b	$V_b = \frac{V_c l_c}{l_b}$	…C5I.3
Moment acting at the face of the joint core	$M_b = V_b\left(l_b - \frac{h_c}{2}\right) = Tjd$	…C5I.4
Rearrange to get T	$T = \frac{M_b}{jd} = \frac{V_b\left(l_b - \frac{h_c}{2}\right)}{jd} = \frac{V_c l_c\left(l_b - \frac{h_c}{2}\right)}{l_b jd}$	…C5I.5
Substitute into the 1st equation	$V_{jh} = T - V_c = \frac{V_c l_c\left(l_b - \frac{h_c}{2}\right)}{l_b jd} - V_c = V_c\left[\frac{l_c}{l_b jd}\left(l_b - \frac{h_c}{2}\right) - 1\right]$	…C5I.6
Rearrange to get V_c	$V_c = \frac{V_{jh}}{\left[\frac{l_c}{l_b jd}\left(l_b - \frac{h_c}{2}\right) - 1\right]}$	…C5I.7
Joint capacity in terms of the column moment	$M_{col} = V_c\left(\frac{l_c - h_b}{2}\right) = \frac{V_{jh}}{\left[\frac{l_c}{l_b jd}\left(l_b - \frac{h_c}{2}\right) - 1\right]}\left(\frac{l_c - h_b}{2}\right)$	…C5I.8
Assume $j = 0.9d$ and $A_e = b_j \times h_c$	$M_{col} = \frac{\nu_{jh}(1000)}{\phi}\ kNm\ and\ \phi = \frac{2l_b' l_c - 1.8dl_b}{0.9dl_b A_e(l_c - h_b)}$	…C5I.9
Nominal horizontal shear stress at the mid-depth of the joint core	$\nu_{jh} = \frac{V_{jh}}{b_j \times h_c}$	…C5I.10
Effective width of the joint	$b_j = \min(b_c, b_w + 0.5h_c)\ \ if\ b_c \geq b_w$ $b_j = \min(b_w, b_c + 0.5h_c)\ \ if\ b_c \leq b_w$	…C5I.11 …C5I.12
Principal tensile and compressive stresses	$p_t = p_c = -\frac{f_v}{2} \pm R$	…C5I.13
Substitute $R = \sqrt{\left(\frac{f_v}{2}\right)^2 + {\nu_{jh}}^2}$ from Mohr's Circle Theory	$p_t = -\frac{f_v}{2} + \sqrt{\left(\frac{f_v}{2}\right)^2 + {\nu_{jh}}^2}$	…C5I.14
Rearrange to get horizontal shear	$\nu_{jh} = \sqrt{{p_t}^2 + p_t f_v}$	…C5I.15
Substitute into the joint capacity equation	$M_{col} = \frac{\sqrt{{p_t}^2 + p_t f_v}(1000)}{\phi}\ kNm$	…C5I.16
Principal tensile stress	$p_t = k\sqrt{f_c'}$	…C5I.17
Stress due to axial load	$f_v = \frac{N_v}{A_e}$	…C5I.18



(a)



(b)



(c)

Figure C5I.1: (a) Free-body diagram of a beam-column joint sub-assembly; (b) Mohr's circle theory applied to calculate joint shear and principal tensile/compression stresses; (c) Moment, shear and stresses at joint region (modified after Pampanin et al., 2003; Akguzel and Pampanin, 2010; Tasligedik et al., 2015)

For an interior joint the same procedure can be followed by:

- introducing the contribution from the compression steel, $C's$, of the other beam in the first equation in Table C5I.1:

$$V_{jh} = T + C's - V_c \quad \ldots C5I.19$$

  assuming $M_b = M_c$ for interior beam-column joints, instead of $M_b = 2M_c$ for exterior joints, and

- checking that l_b' and l_b are to be taken as the beam clear span and full span respectively, consistent with an interior beam-column joint.

Note:

This procedure is intended to be a simple analytical approach to determine the hierarchy of strength and the global mechanism as part of a SLaMA method. The full procedure to evaluate the hierarchy of strength and sequence of events for a beam-column joint sub-assembly is presented in Section C5.6.1.

The example provided assumes a point of contraflexure at mid height of the column, which might in fact vary during the sway mechanism; in particular when yielding columns or joint shear damage and failure occur at one level requiring redistribution and due to the dynamic effects.

Refer to Section C2 for more information on the limitations of alternative analysis methods.

Appendix C5J Establishing the Internal Hierarchy of Strength and Sequence of Mechanisms in a Column

Once the various failure mechanisms for a column have been evaluated, including flexural, shear, lap-splice failure and bar buckling, the (force-based) hierarchy of strength and expected sequence of events can be visualised within an M-N interaction diagram or performance-domain (Pampanin et al., 2002) in order to account for the variation of axial load during the frame sway mechanism.

As an example of the M-N interaction diagram for a column with poor detailing Figure C5J.1 shows:

- conventional tensile and compressive flexural failures
- shear capacity/failure and shear degradation at various ductility levels (μ = 2 and μ = 4)
- lap-splice failure of the column longitudinal reinforcement.

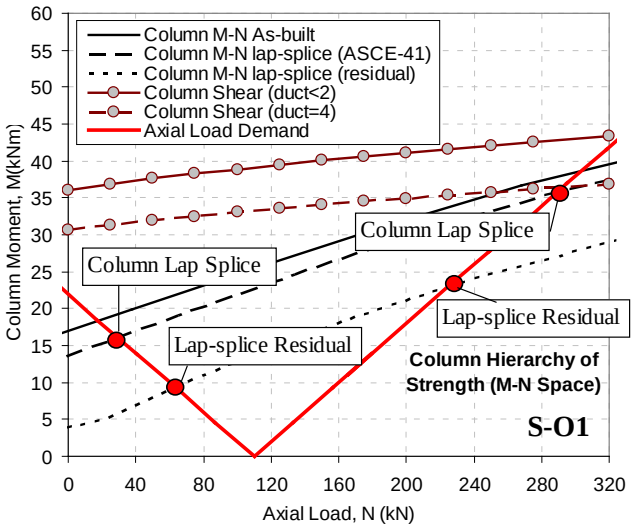


Figure C5J.1: Internal hierarchy of strength of column failure modes within an M-N interaction diagram (Kam, 2011)

Such force-based hierarchy of strength and sequence of event information should be integrated with the information on the rotation or displacement capacities associated with each mechanism, as discussed in Section C5.6.

Ultimately, by combining the flexural capacity curve with the shear degradation capacity curve, an overall force-displacement capacity curve for the column can be derived and will highlight the occurrence of the various mechanisms at different curvature/rotation/ displacement (and therefore the inter-storey drift) level, as shown in Figure C5J.2.

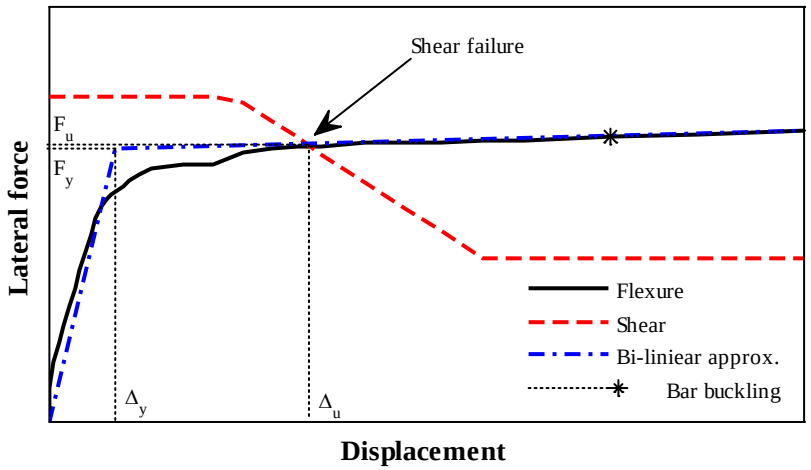


Figure C5J.2: Example of the combined flexural-shear mechanisms within a force-displacement capacity curve for a column (Stirrat et al., 2014)

PART C

Structural Steel Buildings C6

Document Status

Version	Date	Purpose/ Amendment Description
1	July 2017	Initial release

This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

This document may be downloaded from www.EQ-Assess.org.nz in parts:

1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

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Please go to www.EQ-Assess.org.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.EQ-Assess.org.nz

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C6. Structural Steel Buildings

C6.1 General

C6.1.1 Scope and outline of this section

This section provides guidance on the Detailed Seismic Assessment (DSA) of existing steel framed buildings. It does not address earthquake damaged steel framed buildings or the retrofitting of existing buildings.

The following topics are addressed in this section:

- Factors affecting the seismic performance of steel buildings and their observed behaviour in past earthquakes (Sections C6.2 and C6.3)
- Structural steel material properties and testing (Section C6.4)
- Assessment of member and connection probable strength and deformation capacities (Sections C6.5 and C6.6)
- Philosophy and assumptions for the evaluation of existing steel seismic-resisting systems, including the evaluation procedure for steel moment resisting frames (MRFs), steel MRFs with infill panels, and braced frame buildings (Sections C6.7 and C6.8).

C6.1.2 Useful publications

The following publications will be of particular assistance to designers making seismic assessment of steel framed buildings.

ASCE 41-13 (2014). *Seismic evaluation of existing buildings*, American Society of Civil Engineers, and Structural Engineering Institute, Reston, Virginia, USA.

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C6.1.3 Definitions and acronyms

Category 1 buildings	Fully ductile buildings ($\mu > 3$)
Category 2 buildings	Limited ductile buildings ($1.25 < \mu \leq 3$)
Category 3 buildings	Nominally ductile buildings ($1 < \mu \leq 1.25$)
Category 4 buildings	Elastic buildings ($\mu = 1$)
Concentrically braced frame (CBF)	A braced frame where the members are subjected primarily to axial forces
Connection	The entire assemblage of connection components and connectors where two members intersect
Connector	An item within a connection that transfers forces from one member or connection component to another (e.g. bolts, rivets and welds)
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines
Eccentrically braced frame (EBF)	A braced frame in which at least one end of each brace frames only into a beam in such a way that at least one stable, deformable link beam is formed in each beam if the elastic limit of the frame is exceeded. In this event, energy is dissipated through shear and/or flexural yielding in the link beams (termed the active link regions) and the bracing members and columns have sufficient capacity to remain essentially elastic.
Full restraint against lateral buckling (FLR)	Restraint that effectively prevents lateral deflection and twist of a member
Lateral force-resisting system	The part of a structural system that provides resistance to earthquake induced forces
Lateral restraint	An element that prevents lateral movement of the critical flange of a member
Local buckling	A local instability involving a change of shape of the member cross section along a relatively short length of member under compression
Moment resisting frame (MRF)	A building frame system in which lateral loads are resisted by shear and flexure in members and joints of the frame
Overstrength	The maximum strength that a member or a connection can develop due to variations in material strengths, and strength gain due to strain hardening, if applicable
Plate slenderness	The ratio of the critical unsupported width of a steel plate to the average plate thickness
Primary seismic-resisting member	An energy dissipating member of a seismic-resisting system
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects this is determined using probable material strengths. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Rolled steel joist (RSJ)	I-sections that have tapered flanges
Segment	The length between adjacent cross sections which are fully, partially or laterally restrained, or the length between an unrestrained end and the adjacent cross section which is fully or partially restrained
Tensile strength	The probable breaking strength in tension

Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings
XXX%ULS shaking (demand)	Percentage of the ULS shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.
Yielding region	The region of a member which is anticipated to be subjected to nonlinear deformations under earthquake induced forces

C6.1.4 Notation, symbols and abbreviations

Symbol	Meaning
a	Distance between the centre of connectors and a flange cleat angle leg
a_1	Distance between the centre of connectors and the top edge of a flange cleat angle
A_g	Gross area of the cross section
A_n	Net area of the cross section
A_o	Plain shank area of a rivet
A_s	Tensile stress area
A_w	Area of a web
b	Distance between the centroid of tension and compression forces in a web cleat
b_1	Width of contact between beam flange and welds and column
b_{eff}	Effective beam flange width
B_f	Length of an angle
b_{fb}	Beam flange width
b_{fc}	Column flange width
C_s	Factor that accounts for the potential for deterioration in performance of CBFs with increasing inelastic demand
d	Depth of a steel section
D_b , d_b	Depth of a beam section
d_c	Depth of a column section
d_p	Depth of a web
E	Modulus of elasticity
e	Clear length of an active link

Symbol	Meaning
f	Residual capacity factor
f_{u}	Probable tensile strength
f_{uf}	Probable tensile strength of a rivet
f_{uw}	Tensile strength of weld metal
f_{y}	Probable yield strength
f_{yb}	Yield strength of a beam flange
f_{yc}	Yield strength of a column flange
f_{yw}	Yield strength of a web
G	Shear modulus of elasticity for steel, 80,000 MPa
G	Permanent action
h	Storey height
h_{eq}	Effective height of a frame
H_{i}	Height of floor i
H_{v}	Vickers Hardness
I_{b}	Second moment of area of a beam
I_{c}	Second moment of area of a column
k	Distance between bolt centreline and a web cleat angle leg
k	Column base flexural stiffener modifier
k_{e}	Member effective length factor
k_{f}	Form factor for members subject to axial compression
k_{r}	Rotation restraint reduction factor for lap connections
k_{te}	Correction factor for distribution of stresses in a tension member
k_{θ}	Rotational stiffness of column bases
L	Width of the braced bay
l	Member length
l_{a}	Length of a web cleat angle face
L_{b}	Length of critical brace
l_{b}	Clear span of beam
L_{bi}	Bay width
l_{c}	Clear length of column
L_{eq}	Total width of frame
L_{j}	Length of a bolted lap-splice connection
m	Distance from centre of bolt hole to radius root at web
m	Number of columns fixed at the base
m	Number of braces

Symbol	Meaning
M^*	Bending moment demand
M_{b}	Member flexural strength
$M_{\mathrm{bi,l}}$	Probable beam flexural strength to the left of a joint
$M_{\mathrm{bi,r}}$	Probable beam flexural strength to the right of a joint
m_{i}	Mass of floor i
M_{prob}	Probable flexural strength
$M_{\mathrm{prob,bare}}$	Probable flexural strength of a bare connection
$M_{\mathrm{prob,bl}}$	Probable beam flexural strength to the left of a joint
$M_{\mathrm{prob,br}}$	Probable beam flexural strength to the right of a joint
$M_{\mathrm{prob,c}}$	Probable member flexural strength
$M_{\mathrm{prob,ca}}$	Probable column flexural strength above a joint
$M_{\mathrm{prob,cb}}$	Probable column flexural strength below a joint
$M_{\mathrm{prob,encased}}$	Probable flexural strength of an encased connection
$M_{\mathrm{prob,s}}$	Probable section flexural strength
$M_{\mathrm{prob,w}}$	Probable tensile strength of a web cleat angle
$M_{\mathrm{prob,web}}$	Probable flexural capacity of a beam web to column connection
M_{ri}	Probable flexural strength at the base of column i
$M_{\mathrm{ri,b}}$	Probable flexural strength at the base or bottom of column i
$M_{\mathrm{ri,t}}$	Probable flexural strength at the top of column i
n	Number of connectors
n	Number of storeys
n_1	Length obtained by a 45° dispersion though half of the depth of a column
n_2	Length obtained by a 1:2.5 dispersion though column flange and root radius
N^*	Axial force, compressive or tensile
N_{eq}^*	Earthquake induced axial force
N_{fbc}	Probable compression capacity of beam flange
N_{fbc}^*	Compression demand on beam flange
N_{fbt}	Probable tension capacity of beam flange
N_{fbt}^*	Tension demand on beam flange
N_{fct}	Probable tension capacity of column flange
$N_{\mathrm{G+\psi_E Q}}^*$	Axial force demand due to gravity load
$N_{\mathrm{prob,c}}$	Probable member capacity in compression
$N_{\mathrm{prob,ci}}$	Probable compression capacity of brace i
$N_{\mathrm{prob,cr}}$	Probable limiting axial force

Symbol	Meaning
$N_{\text{prob,s}}$	Probable section capacity of a compression member
$N_{\text{prob,tf}}$	Probable tension capacity of a connector
N_{t}	Probable section capacity of a tension member
N_{tfw}	Probable tension capacity of beam flange weld
N_{w}^{*}	Axial force acting on a web panel
N_{wcc}	Probable compression capacity of column web
N_{wct}	Probable tension capacity of column web
n_{x}	The number of connector shear planes intercepting a shear plane
Q	Imposed action
r	Radius of gyration; or transition radius; or the root radius of a section
r_{c}	Column root radius
S	Plastic section modulus
s_{f}	Weld leg length to beam tension flange
S_{i}	Sway potential index
T	Tensile force in web cleat bolts/rivets
t	Thickness
t_1	Flange cleat angle leg thickness
t_2	Web cleat angle leg thickness
T_{c}	Probable tensile strength of column flange
t_{c}	Thickness of column flange
t_{fb}	Beam flange thickness
t_{fc}	Column flange thickness
t_{p}	Total thickness of doubler plates
t_{w}	Thickness of a web
t_{wc}	Column web thickness
V_{base}	Probable base shear capacity
V_{bi}	Storey i beam seismic shear demand determined from beam probable capacity
V_{c}	Probable panel zone shear capacity
V_{prob}	Probable shear capacity
$V_{\text{prob,f}}$	Probable shear capacity
V_{v}	Shear capacity of a web
V_{w}	Shear capacity of a web
α'_{c}	Residual compressive strength factor
α'_{ci}	Residual strength factor for brace i

Symbol	Meaning
α_b	Compression member section constant
γ	Rotation angle of an active link
γ_p	Plastic rotation of an active link
γ_u	Ultimate rotation of an active link
γ_y	Yield rotation of an active link
δ/t	Dimensionless transverse deflection of plate
Δ	Displacement
Δ_b	Displacement capacity of a brace
Δ_c	Displacement at buckling of a brace
Δ_{cap}	Probable displacement capacity
Δ_i	Lateral displacement of floor i
Δ_p	Probable plastic displacement before deterioration
Δ_t	Displacement at tension yield
Δ_y	Probable yield displacement
θ	Chord rotation
θ_{cap}	Probable plastic hinge rotation capacity
θ_i	Angle between a brace and beam at the top end of the brace
θ_p	Plastic hinge rotation before deterioration
θ_y	Probable yield rotation
λ_n	Modified compression member slenderness
μ	Structural displacement ductility factor
μ_{act}	Actual structural displacement ductility demand
ϕ	Strength reduction factor
Ψ_E	Earthquake combination factor

C6.2 Factors Affecting the Seismic Performance of Steel Buildings

C6.2.1 General

Structural steel members are generally considered capable of dissipating significant amounts of energy when subjected to inelastic demands as the base material is inherently ductile. Because of this expected ductile response of the members, steel buildings are considered suitable for regions of high seismicity. However, the seismic performance of steel buildings can be affected by factors such as:

- imperfections and the fabrication process
- load paths through connections
- building condition (deterioration over time)
- member restraints
- P-delta effects
- slab participation, and
- building age (materials and design procedures).

Each of these factors is discussed below. Also refer to Appendix C6A for general guidance on the typical pre-1976 steel building systems used in New Zealand.

C6.2.2 Imperfections and fabrication process

Imperfections in structural steel generally cause stress concentrations that may result in a sudden loss in strength and hence a poor seismic performance. Imperfections may be created during fabrication processes, such as welding, or may be already present in the base material. It is rare for fabrication imperfections to be sufficiently severe in themselves to cause building failures during earthquakes.

Note:

The weld materials used and fabrication processes adopted were some of the minor factors that led to brittle fractures of welded connections in over 200 buildings during the 1994 earthquake in Northridge, California.

C6.2.3 Load paths through connections

Inadequate load paths through connections is the most common cause of local failures in steel buildings during earthquakes. Inadequate load paths through connections was the principal cause of welded connections failures during the 1994 Northridge earthquake (refer to Section C6.3.2.1 for more details).

Inadequate load paths through connections was also considered to be the principal cause of most local failures in multi-storey steel buildings in the 2010/2011 Canterbury earthquake sequence.

Note:

When undertaking a seismic assessment of a steel framed building, assessing load paths through connections is likely to be the most important aspect of the evaluation process.

C6.2.4 Building condition (deterioration over time)

Deterioration due to environmental effects such as corrosion may have a major effect on the seismic performance of steel framed buildings. When exposed to aggressive environments that facilitate corrosion, structural steel members/connections may sustain significant deterioration such as reduction in member strength due to loss of base material to oxidation. The ductile capacity of corroded members may be significantly reduced if the members sustain localised corrosion as the zone of yielding will be limited to the reduced cross section.

Column bases and hold down bolts are the elements most prone to severe localised loss of material due to long term corrosion. There were several reported failures of industrial structural systems in the 1987 Edgecumbe earthquake due to column failures at the base from corrosion. In addition, reduction in member strengths due to corrosion was reported as one of the main factors contributing to failure of braces during this earthquake (Butcher et al., 1998).

Note:

A condition assessment, particularly of pre-1976 steel framed buildings, is recommended as part of the DSA. Refer to Section C6.4 for more details.

C6.2.5 Member restraints

Structural steel members are made up of plates that are hot rolled, cold formed, welded, bolted, or riveted together. The slenderness and the boundary conditions of the constituting plates may significantly affect the seismic performance of a steel member by limiting the local and lateral torsional buckling capacity of the member.

Local buckling of steel members occurs due to plate slenderness, while lateral torsional buckling of steel members occurs when there is inadequate lateral bracing of compression flanges. The elastic resistance to lateral buckling of a steel member is influenced by several factors such as: unbraced length of the compression flange, geometric and material properties of the member, and moment gradient along the member.

Experimental evidences indicate that local plate buckling generally results in a gradual degradation of strength and stiffness in compact cross sections, while lateral torsional buckling causes a rapid loss of strength and stiffness (Gupta and Krawinkler, 1999). Local buckling of slender members causes a rapid loss of section and hence member capacity.

C6.2.6 P-delta effects

Steel MRF buildings are generally more flexible than other building types and hence are subjected to relatively large lateral displacement demands. Therefore, gravity induced loads acting on a laterally displaced building (P-delta effects) can be pronounced on flexible steel MRFs.

Note:

When large ductility demands that may result in significant deterioration in member strength and stiffness are likely, P-delta effects will be worsened.

C6.2.7 Slab participation

Typically, floor slabs have been constructed with no separation from columns. This causes the slab to contribute to the seismic capacity of framed buildings. Slab participation results in development of increased seismic demands in columns due to increased beam flexural overstrength capacity.

Slab participation may induce column flexural yielding, column shear failure or beam shear failure modes in steel MRFs, depending on the relative strength of the members and the connections. Slab participation may also cause damage to floor slabs and compromise the capacity of the floor system to transfer seismic demands to the lateral force resisting members; although the evidence from the 2010/2011 Canterbury earthquake sequence is that the influence on composite slabs (concrete on steel deck on steel or concrete supporting beams) is minimal.

Note:

When the connections of a steel framed building are semi-rigid, slab participation may considerably increase the stiffness and strength of the connections (Roeder et al., 1994). Slab participation may be beneficial in such buildings if it does not result in localised column failures.

C6.2.8 Building age (materials and design)

C6.2.8.1 Materials

The earliest steel framed buildings in New Zealand are believed to have been constructed in the 1880s, with steel being the preferred ferrous material for structural members from then onwards.

Cast iron columns are found in some of the oldest New Zealand buildings and, until the early 1900s, were often used as gravity carrying elements. Cast iron is a low strength and brittle material not suitable for use in a seismic-resisting system or in a gravity system that is required to sustain significant deformations. The tensile strength of cast iron is significantly less than its compressive strength due to the presence of voids and cracks within the iron matrix (Rondal and Rasmussen, 2003). The consequence of these non-ductile characteristics is that the performance of cast iron columns is likely to be poor if they are part of the lateral force resisting system and/or are subjected to significant lateral displacements.

Cast iron columns can be dependably retained in an existing building if they are used as a propped gravity column, with the supports for the beams assessed and reinforced if necessary (e.g. with steel bands) to avoid local fracture under seismic-induced rotations. However, the strength of a cast iron column cannot be determined using the provisions for steel columns in these guidelines as cast iron has a different stress-strain relationship to steel. Guidance on the assessment of cast iron columns can be found in Bussell (1997) and Rondal and Rasmussen (2003).

Wrought iron was also used to a limited extent for structural members in early New Zealand buildings. However, its use largely ended around the 1880s and 1890s as these items were costly to manufacture. The principal disadvantage of wrought iron as a building material was the small quantities made in each production item (bloom), being only 20-50 kg. This meant

that the use of wrought iron in structural members required many elements to be joined by rivets.

Wrought iron has good compressive and tensile strength, good ductility, and good corrosion resistance. The performance of wrought iron members is considered comparable to that of steel members from the same era.

C6.2.8.2 Design

Despite their apparent advantage over other building types of the same era such as unreinforced masonry buildings, steel buildings designed before the introduction of NZS 4203:1976 suffer from the fundamental drawback of being not designed according to capacity design procedures.

Note:

Pre-1976 design methods generally assumed an elastic response, with no consideration given to likely failure modes and with no ductile detailing requirements to ensure that potential plastic hinge regions can dependably accommodate earthquake induced ductility demands. In addition, no attention was generally given to load paths through connections under inelastic response. Structural members of these buildings that should remain elastic to avoid undesirable failure mechanisms may not have the capacity to resist overstrength actions originating from potential plastic hinge regions and slab participation. Additionally, structural members and connections that are provided to resist gravity induced loads may not have the capacity to accommodate earthquake induced displacement demands; although most early gravity systems with bolted or riveted connections are considered to have high ductility capacity but very limited strength.

The pattern of damage observed during the 1995 earthquake in Kobe, Japan indicates that three factors play a significant role in ensuring a good overall seismic performance of a steel frame building not designed following the capacity design method:

- The beam-column connections of the frames of a building should be able to retain their shear and axial force carrying capacity when the connections are sustaining flexural actions from earthquake demands.
- The inelastic demand in the columns should be kept to a minimum. This demand is principally due to local buckling or crippling failure, and also to general plastic hinge formation.
- The inelastic response of the building should be essentially symmetric in nature and not lead to a progressive movement of the building in one direction only.

Note:

Details of the damage sustained during the Kobe earthquake are provided in reconnaissance reports such as that by Park et al. (1995).

In buildings constructed before the 1950s the structural members of steel frames are usually encased in lightly reinforced concrete as fire protection (refer to Figure C6.1). The reinforcement of the encasement is often inadequate and poorly detailed (Bruneau and Bisson, 2000), which results in a significant increase in stiffness and a relatively modest increase in strength of the encased members.

Spalling of the encasement concrete, particularly in the end regions of members, has the potential to increase the nonlinear demands in the steel members if they are required to be loaded beyond yield.

Figure C6.1: A typical riveted beam-column connection

Even older steel framed buildings constructed before the 1936 New Zealand standard model building by-law introduced seismic design requirements typically contain beams that are deeper than the columns. The frames of these buildings generally contain simple and semi-rigid riveted connections that have a modest flexural capacity. In addition, these connections generally exhibit poor energy dissipation capability with lack of adequate strength and stiffness and may serve as the weakest link during inelastic earthquake demands. However, the seismic performance of similar structures dating back to the 1906 San Francisco earthquake has generally been high.

C6.3 Observed Behaviour of Steel Buildings in Past Earthquakes

C6.3.1 Overall performance

Steel buildings have been observed to perform generally well during major international earthquakes. The only steel framed buildings to have been reported to have collapsed were during the 1985 Michoacan, Mexico earthquake. However, these collapses were attributed to factors such as resonance and local soil conditions. The collapsed buildings were between 10 and 15 storeys high, in the resonance range of the strongly harmonic earthquake that struck Mexico City. Another source of collapse was very light welds between built-up members that "unzipped" during the earthquake.

Consequently, steel framed buildings have been generally regarded as ductile and resilient against earthquake induced collapse. However, the significant damage observed during the Northridge (1994) and Kobe, Japan (1995) earthquakes emphasises the vulnerability of even recently constructed steel framed buildings and the need for attention to load paths.

C6.3.2 Moment resisting frame buildings

C6.3.2.1 Performance in the 1994 Northridge earthquake

The 1994 Northridge earthquake caused considerable damage to steel MRFs that had been designed on the basis that they would behave in a ductile manner. The rigidly welded connections of these frames were observed to have fractured at low levels of ductile demand.

Although hundreds of MRF buildings suffered this unexpected overload form of connection damage, most of the buildings displayed no visible signs of distress after the earthquake (such as permanent lateral deflections); nor was there significant damage to non-structural components and contents. However, the capacity of these buildings to resist further earthquake induced demands was significantly compromised and costly repairs were required.

The main reason for the unexpectedly poor performance was the inability of the load paths between the beams and the columns of the frames to transfer actions generated by plastically responding beams into the columns. The inadequacy of these load paths caused fractures of the beam flange to column flange connections. The majority of the fractures were observed to occur at the bottom beam-column flange connections due to slab participation. In some instances these bottom fractures were even observed to trigger web connection failures (Krawinkler, 1995). Refer to Figure C6.2.

Note:

Details of the damage sustained during the Northridge earthquake have been widely reported in reconnaissance reports such as that by Norton et al. (1994).

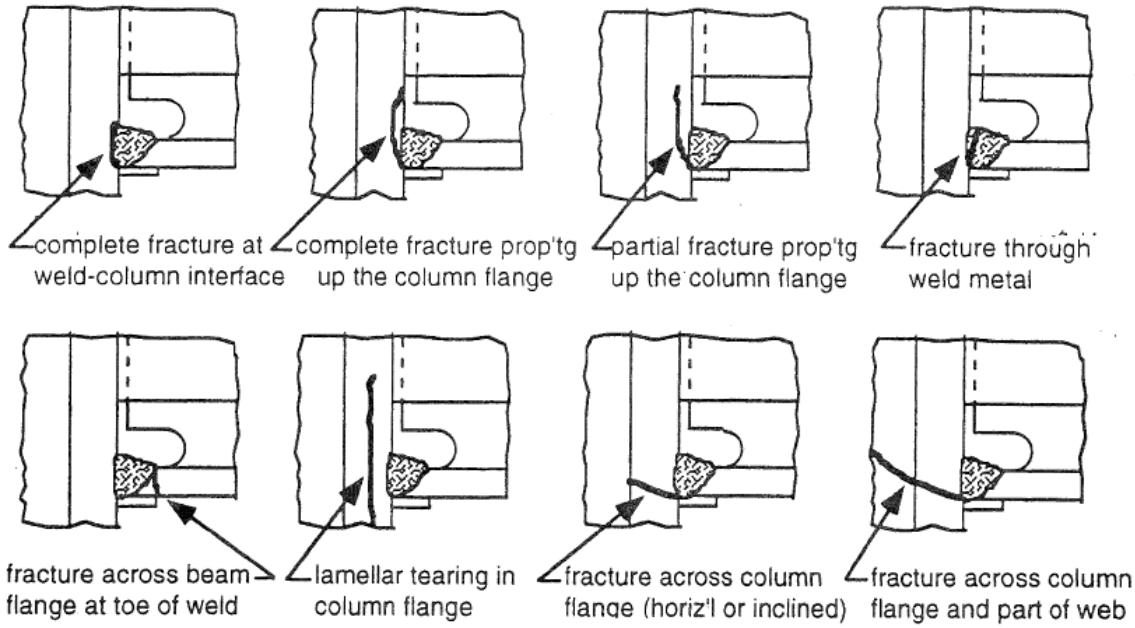


Figure C6.2: Welded connection fracture modes observed during the 1994 Northridge earthquake (Krawinkler, 1995)

The inadequacy of the load paths of "Pre-Northridge" connections meant that even the best fabricated beam to column connections were not able to develop plastic hinges in beams that exceeded a depth of approximately 360 mm. However, the following factors were considered to have minor contributions to the failures of "Pre-Northridge" connections (FEMA 355E, 2000):

- The welding practice was such that bottom flange weld passes were interrupted at beam webs, resulting in weld defects that served as crack initiators.
- The configuration of the connections made detection of hidden bottom weld defects difficult, particularly at the beam webs.
- The filler metal employed was typically developed for high deposition rate welding and had very low notch toughness as a result.
- There was use of large size beams in buildings that had few lateral force resisting frames. The deeper the beam, the greater the web contribution to flexural strength and therefore the greater the likelihood of ductile overload of the beam flange to column flange connection. The use of large size beams also meant higher deposition rate large welds which were more prone to fractures than small size welds (Krawinkler, 1995).
- The mean yield strength of members fabricated in the 1980s was observed to be generally significantly greater than the nominal values.
- The geometry of weld access holes was, in some cases, observed to hinder ease of filler metal deposition and weld inspections.

Immediately after the Northridge Earthquake, the New Zealand Heavy Engineering Research Association (HERA) and the University of Auckland looked at the possibility of similar types of failures in New Zealand buildings and found no examples of this type of construction. A series of large scale beam/column inelastic cyclic tests were performed on typical New Zealand type MRF connections which showed that they were not vulnerable to this type of failure (Butterworth, 1995).

C6.3.2.2 Performance in the 2010-11 Canterbury earthquake sequence

During the Canterbury earthquake sequence of 2010/11 no significant damage appeared to have been sustained by any post-1976 MRFs. Minor panel zone yielding of an MRF (refer to Figure C6.3) was observed in a 12 storey, predominantly eccentrically braced frame (EBF) building.

Provided the beams adjacent to the panel zone did not exhibit any signs of yielding, the yielding of the panel zone was not expected. The yielding of this panel zone was considered to result from the combination of elevated levels of compression force in the columns due to high vertical ground accelerations and the expected and significant bending demands imposed on the adjoining beams.

Figure C6.3: Panel zone of an MRF showing minor inelastic action (Clifton and El Sarraf, 2011)

C6.3.3 Braced steel frame buildings

C6.3.3.1 Eccentrically braced frame buildings

EBF multi-storey buildings generally performed very well during the Canterbury earthquake sequence. Generally, the observed damage was minor and limited principally to non-structural items. A 22 storey EBF building required replacement of seven active links due to nonlinear overload and, in one case, brittle fracture (refer to Figure C6.4(a)). Another 35 active links were replaced due to the steel having unacceptably low Charpy impact energy. More active links would have been expected to be replaced as the magnitude of the excitation during the February 22, 2011 earthquake was such that it was significantly above the 500 year return design spectrum of NZS 1170.5:2004 that is the basis for ULS design of typical new buildings. One 12 storey EBF building was returned to service with no structural repairs needed. It was the only multi-storey building in the Christchurch CBD for which this was the case, including base isolated structures.

The good performance of multi-storey EBF buildings in the Canterbury earthquake sequence can be attributed to:

- the significant effects of soil-foundation-structure-interaction (on reducing the seismic demand on the superstructure of these relatively heavy multi-storey buildings built on soft soil (Storie et al., 2014))
- factors contributing to overstrength in steel frames such as actual yield strengths significantly exceeding nominal values, modelling assumptions, etc.
- the contribution of the composite floor slab action to the shear resistance that was not allowed for in the design of the frames, and
- the contribution of solid partition walls and non-structural items.

A fractured active link of the 12 storey EBF building is presented in Figure C6.4(a). This active link appeared to have undergone at least one full cycle of web panel yielding prior to fracture. The fracture appeared to have propagated from one top corner across the active link region and resulted in significant residual deformations. Detailed evaluations of this and other links in the EBF braced bay concerned showed that the Charpy impact energy of this steel was well below that specified by NZS 3404:1997, with the material having a transition temperature of around 12^{o}C. This particular link also had a shear stud welded to the flange immediately above the left hand visible stiffener, which is believed to have acted as a crack initiation site.

(a) A fractured active link in a 12 storey building

(b) A fractured active link in a low-rise parking building

Figure C6.4: Fractured EBF active links during the February 22, 2011 Christchurch earthquake (Clifton and El Sarraf, 2011)

Fractures of two active links in a low-rise EBF building (refer to Figure C6.4(b)) were attributed to detailing/fabrication errors. The flanges of the two braces were observed to be offset from the stiffeners of the active links. The offset lead to fracture of unstiffened collector beam flanges located between the active link stiffeners and the flanges of braces.

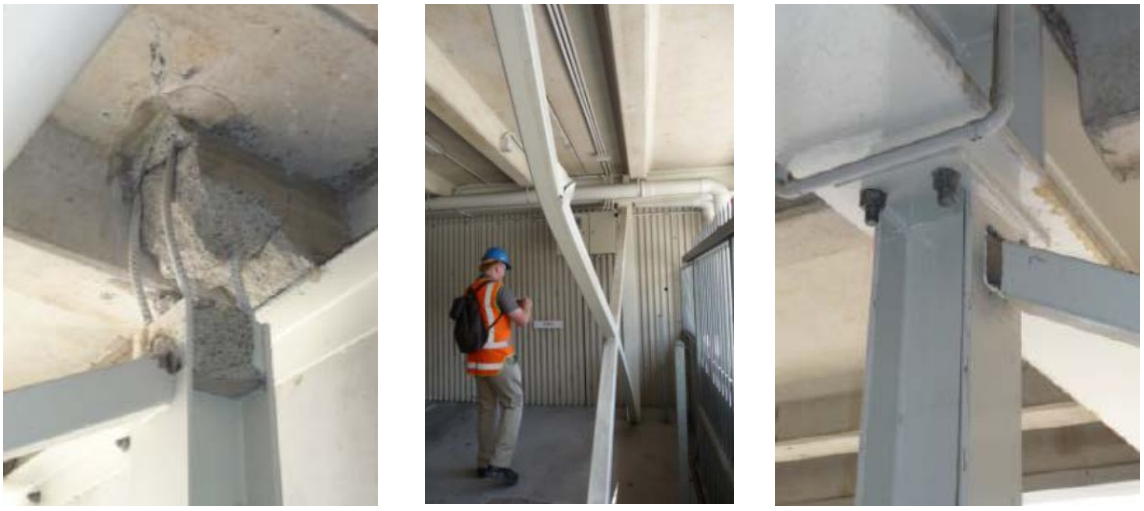
C6.3.3.2 Concentrically braced frame buildings

Observations made during the 1995 Kobe earthquake have reinforced the expectation that concentrically braced frames (CBFs) that are not designed following the capacity design method are not likely to perform as intended in the event of an earthquake.

In New Zealand, non-capacity designed (pre-1976) CBFs are typically X-braced, while very few are believed to be V-braced. Pre-1976 CBFs in New Zealand were typically designed to

resist lower levels of lateral forces than required by NZS 1170.5:2004. In Kobe, several such CBF buildings were reported to sustain buckled braces or failed connections during the 1995 earthquake. However, none of these buildings were reported to have collapsed (Clifton, 1996).

Most CBF buildings performed as expected during the 1994 Northridge earthquake, but with no collapses reported. Similar to the connection weld fractures of MRFs discussed in Section C6.3.2, fractures of brace-collector beam and column-base plate welded connections were prevalent. In addition, excessive local buckling of thin-walled tubular braces of CBFs was observed (Krawinkler, 1995).



(a) A poorly detailed connection **(b) A buckled brace** **(c) A fractured connection**

Figure C6.5: Damaged CBFs in a single-storey car park building during the February 22, 2011 Christchurch earthquake (Clifton and El Sarraf, 2011)

Significant damage to a single-storey CBF building was observed during the Canterbury earthquake sequence (refer to Figure C6.5). However, the connections of the CBFs to the columns appeared to have been poorly detailed. One of the CBFs appeared to have been connected to a column that had a non-ductile reinforced concrete extension (refer to Figure C6.5(a)), while the welded connection of the second CBF did not appear to have been designed by following capacity design principles (refer to Figure C6.5(b)).

C6.3.4 Portal frame buildings

Most portal frame buildings generally performed well during the Canterbury earthquake sequence. Observed damage was mainly attributed to ground instability or limited to failure of bracing systems, while frame moment connections exhibited no visible signs of damage.

Many of the portal frame buildings in Christchurch were industrial facilities designed to resist high wind induced forces, which were typically the controlling design case. These buildings typically have light roofs that are braced using light rod braces with proprietary end fittings. A few fractures and thread stripping of the proprietary brace connectors were reported following the February 22, 2011 earthquake (refer to Figure C6.6).

Figure C6.6: Proprietary brace connectors that failed during the February 22, 2011 Christchurch earthquake (Clifton and El Sarraf, 2011)

Figure C6.7: Roof bracing that failed during the February 22, 2011 Christchurch earthquake (Clifton and El Sarraf, 2011)

In one building, failure of a roof bracing was observed following the February 22, 2011 earthquake (refer to Figure C6.7). This failure was considered to be a result of excessive movements of tilt up panels that were likely to have been caused by ground liquefaction (Clifton and El Sarraf, 2011).

C6.4 Material Properties and Testing

C6.4.1 General

Note:

Assessments in accordance with these guidelines are intended to be carried out using probable material strengths. Typically, the probable material strengths may be taken as the (lower) characteristic/nominal (as referred to in design standards) material strengths but enhanced by the material strength modification factors given in Section C6.4.4.

Mechanical properties of the steelwork within existing structural steel framed buildings may be determined from:

- drawings, specifications or other construction records
- historical steel grades and nominal strengths, and/or
- steel material tests.

The mechanical properties of structural steelwork are best determined from original construction records supplemented by laboratory or in-situ tests of selected critical components to confirm the expected steel grade.

If the source of steelwork can be confirmed from the designations on original construction records, but the steel grade is not identified and testing is not practicable, default mechanical properties corresponding to the source and age of the steelwork can be adopted from those outlined in historical specifications. Refer to Appendix C6B for typical sources of historical New Zealand structural steelwork.

In the absence of construction records, the source of a structural steelwork can be identified from the mill markings generally present on historical structural steel sections and from section geometric properties contained in literature on historical structural steelwork (e.g. Bates, 1991; Bussell, 1997; and Ferris, 1954).

Note:

Older steelwork exhibits greater variability than modern steelwork. Accordingly, a minimum degree of non-destructive testing is recommended to gain assurance of the mechanical properties for the members in the primary structure. This is particularly the case when the steel is “of unknown origin”.

If the steelwork cannot be identified from construction records, mill markings or section geometric properties, the default yield strengths for steel “of unknown origin” provided in Appendix C6B may be adopted.

If tensile tests are undertaken, default strengths corresponding to the grade, potential source and age of the steelwork should be adopted from Appendix C6B.

Members with steel of unknown origin may exhibit non-ductile behaviour if all of the following conditions apply:

- from an assessment of the strength hierarchy of the building, the members with steel of unknown origin are the weakest links, not the connections, and
- the members with steel of unknown origin are located in an external steelwork or on the cold side of the building envelope so that the members could be below their transition temperature at the time of an earthquake, and
- a notch, a significant crack, or any stress raiser is present in a critical location.

If all these conditions contributing to potential member brittle responses are present and potential brittle failure has not already been ruled out through physical testing, fracture toughness tests should be undertaken on selected critical members as per Section C6.4.5 to rule out potential brittle responses.

Note:

Another key concern with members with "steel of unknown origin" is the undefined upper bound on yield strength, which may be significantly greater than the characteristic values. Primary members of unknown origin may develop strengths that are significantly higher than allowed for using overstrength factors. Large member overstrengths may lead to overloading other aspects of the structure and loss of assumed hierarchical behaviours and/or protection.

C6.4.2 Identifying the building materials: are they cast iron, wrought iron or steel?

As outlined in Section C6.2.8.1 the earliest steel framed buildings in New Zealand are believed to have been constructed in the 1880s. While steel was the preferred ferrous material for structural members from then onwards, cast iron and, to a lesser extent, wrought iron were also used in New Zealand buildings before the early 1900s. Identifying the building materials and their age is an important aspect of the seismic assessment process.

Cast iron

The use of cast iron from the 1880s until its discontinuance around 1910 was limited to columns. Cast iron columns would have been used typically for gravity load carrying purposes. These columns are typically "chunky" with thick sections, often having ornate or complex profiles (fluted, plain hollow circular, or cruciform shaped). The surface of these columns is typically pitted with small blowholes.

Wrought iron

If a building is constructed before 1900 and contains members built up from many short-length I-sections, channels and/or flats, then the possible use of wrought iron in these members should be considered. Guidance for the assessment of wrought iron members is provided in Bussell (1997).

Note:

Detailed visual assessment criteria for iron and steel members are presented in Bussell (1997).

C6.4.3 Cast iron and wrought iron: probable strengths

In the absence of specific material data, the probable yield strengths of cast iron and wrought iron should be taken as the values provided in Table C6.1 if members in buildings constructed before the early 1900s are identified to be made of cast iron or wrought iron.

Table C6.1: Probable strengths of historical cast iron and wrought iron

Material	Tensile strength (MPa)	Compressive strength (MPa)	Modulus of elasticity (GPa)
Cast iron	47	247	93
Wrought iron	162	124	185

Note:

Cast iron and wrought iron are generally only found in buildings constructed prior to 1900. Due to the lack of available specific data, the probable strength of cast iron and wrought iron is taken as one half of published breaking strengths such as those by Fidler (1879). Table C6.1 is based on the lower characteristic strength values published in 1879 (Bates, 1991).

C6.4.4 Structural steel: historical grades and probable strengths

Before the 1960s most structural steelwork was imported from Australia (historical evidence indicates this was from the late 1930s onwards) and the UK. A small quantity of steel is also believed to have been imported from the USA and continental Europe.

From the 1960s on most rolled sections have been manufactured in Australia, while plates and welded sections have been mainly produced in New Zealand.

The structural steel properties outlined in relevant historical standard specifications are summarised in Appendix C6B. Default characteristic/nominal strengths are also provided for steel of unknown origin.

Note:

The first New Zealand structural steel standard specifications are believed to be NZS 309 and NZS 310, published in 1941. These standards and their subsequent editions were based on their British equivalents until the first joint AS/NZS standard specifications were introduced in the mid-1990s. The joint specifications were revisions of previous Australian standard specifications.

Mechanical properties provided in construction documentation and default mechanical properties specified in standard specifications should be taken as (lower) characteristic or nominal strengths. Probable strengths can be determined from these by applying the appropriate strength modification factor from Table C6.2. The factors provided in this table are applicable to steelwork produced in New Zealand and to steelwork imported from Australia or the United Kingdom.

Table C6.2: Factors to convert lower characteristic material strengths to probable strengths (based on tests undertaken by Baker, 1969; Erasmus, 1984; and Erasmus and Smaill, 1990)

Period	Steel grade	Factor
Pre-1960	All	1.1
1960-Now	300 and below	1.15
	350 and above	1.1

C6.4.5 Test methods to determine the mechanical properties of structural steel

C6.4.5.1 General

Testing to determine the mechanical properties of structural steel components of an existing building is generally recommended. This is especially the case when the properties of the primary structure cannot be identified from original construction records and mill markings.

Tests should at least identify the likely steel grades. They should also identify unexpectedly high or low strength materials and materials that may exhibit brittle behaviour when subjected to earthquake loading.

Note:

If the intention is to strengthen an existing steel building and the strengthening involves welding to an existing steel, the weldability of the existing steel parent material also needs to be determined.

C6.4.5.2 Tensile strength tests

The probable tensile strength of a structural steel component can be determined from tensile tests undertaken on a representative material removed from the component. Alternatively, hardness tests may be undertaken on the component in situ.

There is an approximate relationship between material hardness and probable tensile strength. The best relationship for the range of steel material strengths of interest (400 to 700 MPa) is given by Vickers Hardness, H_v. The relationship between Vickers Hardness and tensile strength of a steel material is tabulated in ASM International (1976) and can be expressed in equation form as:

$$f_u = 3.09\, H_v + 21.2 \qquad \text{…C6.1}$$

where:

H_v = Vickers Hardness from test
f_u = probable tensile strength

This expression is valid for $100 \le H_v \le 300$, corresponding to $330 \le f_u \le 950$ MPa.

Note:

Testing for Vickers Hardness is carried out to AS 1817:1991 Metallic Materials – Vickers Hardness Test (1991). There are a number of materials testing organisations in New Zealand that can undertake Vickers Hardness tests.

The key steps for determining what components to test and how many tests to conduct are as follows:

Step 1

Determine the members/elements to be tested, i.e. beams, columns, critical connection components and connectors. The elements identified as critical from the connection evaluation in Section C6.6.1 and the strength hierarchy evaluation in Section C6.7 should be subjected to the most detailed testing.

Step 2

Determine the frequency of testing. The aim is to cover at least 5% of the total sample of each type of critical component.

Step 3

Use Equation C6.1 or refer to Nashid et al. (2015) for the relationship between Vickers Hardness and tensile strength.

Note:

Nashid et al. (2015) presents the findings of comprehensive recent research on the hardness-tensile strength relationship of structural steel members.

Step 4

Compare the tensile strengths with the expected steel grades. Any material with $H_v < 100$ or $H_v > 230$ should be investigated more thoroughly by tensile sampling and visual inspection. Any material with $H_v > 230$ should also be treated as potentially prone to brittle fracture.

Note:

There is no direct relationship between tensile strength and brittle fracture. However, the susceptibility to brittle fracture increases with increasing tensile strength. The elongation capacity of steel also decreases with increasing strength. Accordingly, the guidance provided above is a threshold requiring more appropriate testing for potential brittle fracture performance.

C6.4.5.3 Fracture toughness tests

As discussed above and in Section C6.4.1, the potential for member brittle fracture in an existing building becomes an issue for further investigation if the structural components are the weakest links and if any of the following are applicable:

- the components are “steel of unknown origin” and are located in an external steelwork or on the cold side of the building envelope, or

- the Vickers Hardness test of the components identifies steel with $H_v > 230$, or
- the thickness of any component is > 32 mm.

If any of these apply, material from those components should be removed for Charpy impact tests, as specified in NZS 3404:1997, to determine whether the steelwork satisfies energy absorption requirements. Test material may be removed from the less critical regions of a member/element; e.g. from the web of beams away from high shear zones.

A minimum of three Charpy impact tests should be undertaken on material removed from each type of critical component. For the energy absorption requirements to be satisfied, the average Charpy impact energy absorption capacity of a steelwork from the three tests should exceed 27 J at 0°C, while the minimum of the three tests should exceed 20 J at 0°C.

If the steel does not satisfy the above energy absorption requirements a more detailed evaluation should be undertaken.

Note:

For brittle fracture of steel to occur during an earthquake, the steel has to have a low Charpy impact energy absorption capacity at service temperature (or the steelwork has to be below its transition temperature at the time of the earthquake) and a stress raiser has to be present in a critical location.

C6.4.6 Probable yield and tensile strengths of fasteners and weld metals

In the absence of any physical test data, probable strengths of fasteners and weld metals provided in Table C6.3 can be used.

Note:

In the absence of specific data, the probable strengths shown in Table C6.3 have been taken as the lower characteristic strengths based on Bussell (1997) and ASCE 41-13 (2014) except for pre-1961 rivets which have been taken as 1.1 times their characteristic/ nominal strength.

Table C6.3: Probable strengths of fasteners and weld metals

Time period	Material	Origin	Yield strength (MPa)	Tensile strength (MPa)
1901–60	Rivets	USA	228	380
1934-42	Rivets to BS 548 (High tensile steel)	UK	*	510
1948-61	Rivets to BS 15 (Mild steel)	UK	*	425
All	Bolts	All	240	400
All	Weld metals	All	-	410

Note:
*The probable yield strength of these rivets can be taken as half of their probable tensile strength.

C6.5 Component Capacities

C6.5.1 General

This section covers the assessment of the probable strength and deformation (rotation) capacities of members/elements of moment resisting and braced steel frames including:

- beams
- columns
- concrete encased steel beams and columns
- braces
- active links of eccentrically braced frames.

The probable strength of structural steel members/elements should be determined using the probable material strengths as outlined in Section C6.4. A strength reduction factor is not required to be applied (i.e. a strength reduction factor, ϕ, of 1.0 is used).

C6.5.2 Beams

C6.5.2.1 General

The probable strength of steel beams of seismic-resisting frames is generally governed by flexural strength.

The flexural strength of a steel beam is dependent on the length of the beam between adjacent cross sections that may be either restrained or unrestrained (segments) and the restraint condition provided at the ends of the segments (full, partial or lateral restraint).

Guidance provided in NZS 3404:1997 and guidelines such as those outlined in Clifton (2009) provide methods to determine bracing required against lateral torsional buckling and plate slenderness limits to ensure local and lateral buckling of steel members do not occur prematurely.

The effect of combined actions of shear and flexure should be assessed at cross sections where both shear and flexure are expected to be high.

C6.5.2.2 Shear strength

For a stocky web of a structural steel section satisfying a web panel slenderness ratio ($d_\mathrm{p}/t_\mathrm{w}$) of:

$$d_\mathrm{p}/t_\mathrm{w} \leq 82 \Big/ \sqrt{f_\mathrm{y}/250} \qquad \text{…C6.2}$$

where:

d_p = depth of web
t_w = thickness of web
f_y = probable yield strength

the probable shear yield capacity of the web (V_v) should be taken as (NZS 3404:1997):

$$V_v = 0.6 f_y A_w \quad \text{…C6.3}$$

where:

f_y = probable yield strength
A_w = area of web.

If the above web slenderness criterion is not satisfied and the web is slender, the web is likely to buckle instead of yielding in shear. The probable shear buckling strength of slender webs should be determined from Clause 5.11.5 of NZS 3404:1997.

C6.5.2.3 Flexural strength

The probable section flexural strength, $M_{prob,s}$, and probable member flexural strength, $M_{prob,c}$, of steel beams that are subjected to bending about their major principal axis should be determined from Clause 5.2 and Clause 5.3 of NZS 3404:1997 using probable material strengths.

The sections of a steel beam should be compact and not prone to local plate buckling in order to have flexurally yielding regions in the beam that are able to develop and maintain their full plastic section strength until the deformation capacity is reached.

In addition to having compact sections, steel beams or segments of steel beams need to have full restraint against lateral buckling (FLR) to develop and maintain their full section plastic strength.

Beams supporting a concrete slab are considered to have FLR and develop their section flexural strength, while beams supporting timber floors generally achieve member flexural strength only. Restraint offered to steel beams by timber floors or other lateral restraint conditions are provided in HERA Report R4–92 (Clifton, 1997).

The probable strength of beams having slender sections is limited to their probable yield strength due to local plate buckling. However, unlike elastic buckling of compression members, buckling of slender plates of flexural members does not lead to immediate loss of load-carrying capacity or excessive deflections, as shown in Figure C6.8, as redistribution of in-plane stresses occurs within the plates.

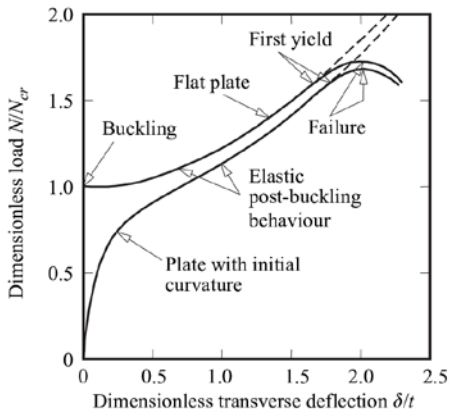


Figure C6.8: Post-buckling behaviour of thin plates (Trahair et al., 2008)

A generic relationship between the probable flexural strength, M_{prob}, and probable chord rotation, θ, capacity of steel beams with FLR is provided in Figure C6.9. The parameters for the generic relationship for this type of beam should be taken from Table C6.4 using the highest possible member category. The member category for steel beams should be determined based on steel material and section geometry requirements outlined in Clause 12.4 and Clause 12.5 respectively of NZS 3404:1997.

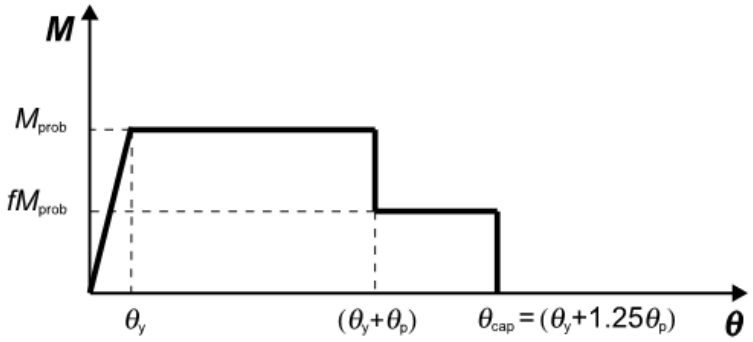


Figure C6.9: Moment-rotation relationship for steel beams and columns with FLR

Table C6.4: Parameters for the moment-rotation relationship for steel beams with FLR

Category of member	$(\theta_y + \theta_p)$ (mrad)	Residual strength factor f
1 & 2	45	0.5
3	30	0.5

Note:
If the beam under consideration cannot support gravity loading $(G + \Psi_E Q)$ in a simply supported condition, halve the rotational capacity of the beam for both the full and the residual strength capacity.

The probable yield rotation, θ_y, of seismic governed steel beams that are rigidly connected to columns at both ends can be determined from:

$$\theta_y = \frac{M_{\text{prob,s}}\, l_b}{6EI_b} \qquad \text{…C6.4}$$

where:

$M_{\text{prob,s}}$ = probable section flexural strength
l_b = clear span of beam
E = modulus of elasticity
I_b = second moment of area of beam.

C6.5.3 Columns

Steel columns in seismic-resisting buildings are generally subjected to a combination of flexure and axial forces. Both axial tension and compression reduce the flexural capacity of steel columns, while axial compression reduces the local buckling capacity.

The probable strength of steel columns may be limited by the various member shear and flexural yield mechanisms outlined in Section C6.5.2. In addition, the flexural capacity of steel columns may be limited by column buckling. The probable section and member flexural capacities of steel columns should be determined from Clause 8.3 and Clause 8.4 of NZS 3404:1997 using probable material strengths.

When determining the rotation capacity of steel columns, the axial force used should be that from the gravity load associated with earthquake action ($N^*_{\mathrm{G}+\psi_{\mathrm{E}}\mathrm{Q}}$) and the seismic contribution should be ignored.

Note:

Experimental tests (MacRae, 1990 and Brownlee, 1994) have shown that the inelastic behaviour and rotation capacity of a steel beam-column subject to compression and major axis bending is dependent on the magnitude of the constant component of the compression force – i.e. that from $N^*_{\mathrm{G}+\psi_{\mathrm{E}}\mathrm{Q}}$ – rather than on the total compression force that includes the seismic component.

Steel columns that are subjected to inelastic demand should satisfy the axial load limitations of Clause 12.8.3 of NZS 3404:1997. This clause is intended to ensure that the level of compression in a column is not too high to compromise the capacity of the column to dependably accommodate inelastic earthquake demands.

A typical moment-rotation relationship for steel columns that have FLR is provided in Figure C6.9. The parameters for the generic relationship for this type of column should be taken from Table C6.5 using the highest possible member category. The member category for steel columns should be determined based on steel material and section geometry requirements outlined in Clause 12.4 and Clause 12.5 respectively of NZS 3404:1997.

Table C6.5: Moment-rotation parameters for steel columns with FLR

Category of member	$(\theta_y + \theta_p)$ $(mrad)$				Residual strength factor, f
	$N^*/N_{\mathrm{prob,s}} \leq 0.15$	$0.15 < N^*/N_{\mathrm{prob,s}} \leq 0.3$	$0.3 < N^*/N_{\mathrm{prob,s}} \leq 0.5$	$0.5 < N^*/N_{\mathrm{prob,s}} \leq 0.8$	
1 & 2	50	45	20	15	0.5
3	35	30	15	10	0.5

The probable yield rotation of steel columns that have a point of contraflexure at mid height and are subjected to both flexure and compression can be determined from:

$$\theta_{\mathrm{y}} = \frac{M_{\mathrm{prob,s}}\, l_{\mathrm{c}}}{6EI_{\mathrm{c}}}\left(1 - \frac{N^*}{N_{\mathrm{prob,c}}}\right) \qquad \ldots \mathrm{C6.5}$$

where:

$M_{\text{prob,s}}$	=	probable section flexural strength
l_{c}	=	clear length of column
E	=	modulus of elasticity
I_{c}	=	second moment of area of column
N^*	=	axial force from analysis
$N_{\text{prob,c}}$	=	probable member capacity in compression.

Comparisons of the provisions in NZS 3404:1997 with physical tests undertaken recently in Canada (Clifton (not published at time of preparation)) on a medium heavy I-section column type cross section and an I-section beam type cross section indicated that the following modifications needed to be made to the provisions in NZS 3404:1997 when determining the probable capacity of steel members:

- The rotation restraint factor (k_{r}) should be taken as 0.85, consistent with a plastic hinge forming at one end only at a particular point in time.
- The member effective length factor (k_{e}) should be taken as 0.85 instead of the NZS 3404:1997-specified 1.0, consistent with a plastic hinge forming at one end only at a particular point in time.

The physical tests undertaken in Canada also showed that the moment-rotations parameters presented in Table C6.5 for highly axially loaded columns are on the conservative side.

Members that are subjected to bending about their minor principal axis should be considered capable of developing their probable plastic section flexural strength about their minor principal axis.

C6.5.4 Concrete encased steel beams and columns

C6.5.4.1 General

If the concrete encasement of steel members complies with the requirements of NZS 3404:1997 for composite member action, the assessment of such members should be undertaken in accordance with NZS 3404:1997, consistent with the determination of probable strength as specified in these assessment guidelines.

The probable capacity of encased steel members not satisfying the requirements of NZS 3404:1997 should be determined as discussed below.

Note:

The structural members of old steel frames are generally encased in lightly reinforced concrete. In some cases the concrete encasement is unreinforced and has low compressive strength, and is therefore generally considered to play a fire protection role only (Bruneau and Bisson, 2000).

If the concrete encasement of old steel frames is reinforced, the reinforcement is often nominal and consists of plain round bars and thin wire meshes. Inadequately reinforced concrete encasement results in a significant increase in stiffness and a relatively small increase in strength of the encased members.

C6.5.4.2 Concrete encased steel beams, solid sections

The concrete encasement should be assumed to suppress local buckling. The probable strength of such beams should be based on the strength of the steel member only, with slight strength enhancement allowed for due to the concrete encasement:

$$M_{\text{prob,s}} = 1.1 S f_{\text{y}} \quad \text{…C6.6}$$

where:

S = plastic section modulus
f_{y} = probable yield strength.

The moment-rotation relationship of concrete encased steel beams is similar to that provided in Figure C6.9. The parameters for the generic relationship for this type of member should be taken from Table C6.4 and the probable yield rotation from Equation C6.4.

C6.5.4.3 Concrete encased steel columns, solid sections, small changes in cross section area or moment of inertia of the encased steelwork within a storey height

The concrete encasement should be assumed to suppress local buckling of the encased steel elements and lateral buckling for moment. However member buckling in compression needs to be considered in accordance with Clause 6.3 of Wood (1987). Alternatively, use the column design curve from NZS 3404:1997 for $\alpha_{\text{b}} = 0.0$ to determine the slenderness reduction factor, with the effective length factor $k_{\text{e}} = 1$ in accordance with Clause 12.8.2.4 of NZS 3404:1997.

The probable flexural capacity of such columns should be based on the probable flexural capacity of the steel members only.

The moment-rotation relationship of concrete encased steel columns is similar to that provided in Figure C6.9. The probable plastic hinge rotation capacity (θ_{cap}) should be determined from Table C6.4 (Category 2) and the probable yield rotation capacity from Equation C6.5. However, for the columns to be considered to have ductile capacity they should satisfy the axial load limitations of Clause 12.8.3 of NZS 3404:1997.

C6.5.4.4 Concrete encased steel columns, laced and battened sections or solid sections with significant changes in the cross section area or moment of inertia of the encased steelwork within a storey height

The response of encased laced and battened columns is considered nominally ductile and, as such, θ_{p} should be limited to that for a Category 3 member in Table C6.4.

The probable flexural capacity of this type of column should be based on the probable flexural capacity of the steel elements only, while the probable yield rotation may be taken as 5 mrad in lieu of a detailed analysis.

The probable compression capacity of laced and battened columns should be determined from Clause 6.4 of NZS 3404:1997 using probable material strengths.

C6.5.5 Braces

C6.5.5.1 Compression capacity

The performance of braces that are subjected to earthquake induced compression forces principally depends on the slenderness ratio of the braces.

Braces with a slenderness ratio $\frac{k_e l}{r}\sqrt{\frac{f_y}{250}}$ that is:

> 120	generally do not have the capacity to carry compressive inelastic earthquake demand and their capacity is exceeded typically through elastic buckling
$\leq$ 120	should be expected to buckle inelastically through local yielding under the combined actions of compression and bending.

where:

k_e	=	member effective length factor
l	=	member length
r	=	radius of gyration
f_y	=	probable yield strength

The probable capacity of braces in compression, $N_{\text{prob,c}}$, should be determined from Chapter 6 of NZS 3404:1997 using probable material strengths. Note that the flexural demand due to the self-weight of the brace and any other gravity load acting on the brace should be allowed for when determining the capacity of a brace acting in a horizontal plane (e.g. roof bracing).

When a compression brace buckles inelastically the same peak compression capacity as achieved in a previous cycle is generally not likely to be achieved during subsequent cycles of loading. A typical force-displacement relationship for a brace in compression is presented in Figure C6.10. The values of the parameters in the figure are provided in Table C6.6.

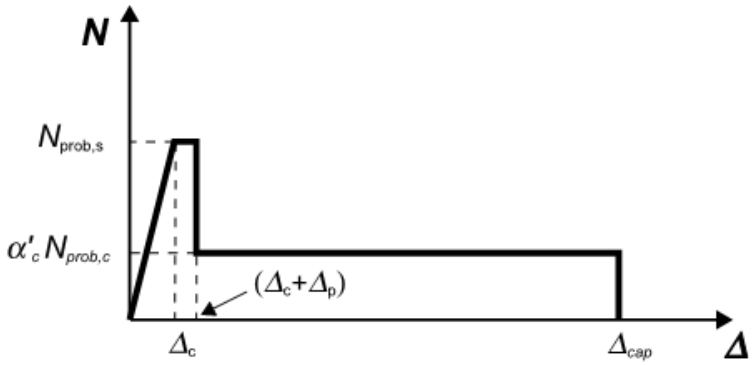


Figure C6.10: Force-displacement relationship for steel braces in compression

Table C6.6: Force-displacement parameters for steel braces in compression

Modified slenderness ratio λ_n	Component type	Deformation		Residual strength factor α_c'
		Δ_p	Δ_{cap}	
≤ 60	I-section, double angle (2L) in-plane	Δ_c	$8\Delta_c$	$*$
	Hollow section, double angle (2L) out-of-plane	Δ_c	$7\Delta_c$	$*$
≥ 120	I-section, double angle (2L) in-plane	$0.5\Delta_c$	$9\Delta_c$	$*$
	Hollow section, double angle (2L) out-of-plane	$0.5\Delta_c$	$8\Delta_c$	$*$
	Single angle	$0.5\Delta_c$	$10\Delta_c$	$*$
$60 < \lambda_n < 120$	All	Linearly interpolate		$*$

Note:

$*$ given by Equation C6.7 or C6.8 as appropriate

λ_n should be determined from Clause 6.3.3 of NZS 3404:1997 as: $\lambda_n = \left(\frac{k_e l}{r}\right)\sqrt{k_f}\sqrt{\frac{f_y}{250}}$

Δ_c is the probable elastic axial deformation of a brace at buckling (N^*=$N_{prob,c}$)

Δ_p is the probable plastic axial deformation capability of a brace before degradation of strength

Δ_{cap} is the probable deformation capacity of a brace.

The residual strength factor (α_c') for compression braces that are likely to remain nominally ductile due to XXX%ULS shaking is given as:

$$\alpha_c' = 7.7/\lambda_n^{0.6} \leq 1.0 \qquad \ldots C6.7$$

and for compression braces that are likely to be subjected to ductile demand at XXX%ULS shaking as:

$$\alpha_c' = 42.15/\lambda_n^{1.1} \leq 1.0 \qquad \ldots C6.8$$

C6.5.5.2 Tension capacity

A typical force-displacement relationship of braces in tension is provided in Figure C6.11. The values of the parameters in the figure are given in Table C6.7. The probable capacity of braces in tension $N_{prob,s}$ should be determined from Chapter 7 of NZS 3404:1997.

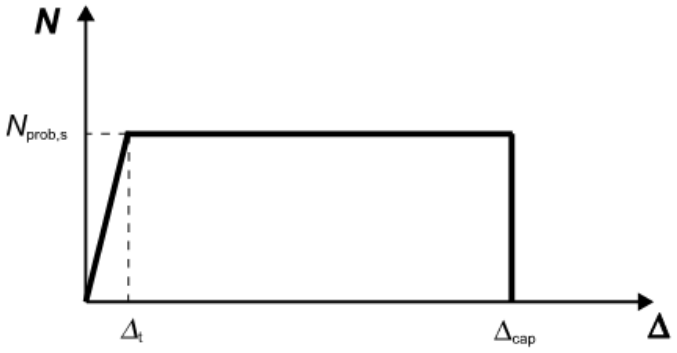


Figure C6.11: Force-displacement relationship for steel braces in tension

Braces in tension are not considered to have any residual capacity after their displacement capacity, Δ_{cap}, is exceeded.

Note:

The probable tensile capacity of a brace should not be taken as greater than the probable capacity (in tension and/or in shear) of the connections at either end.

Table C6.7: Force-displacement parameters for steel braces in tension

Component type	Deformation
	Δ_{cap}
I-section	$11\Delta_t$
Hollow section	$8\Delta_t$
Single angle	$9\Delta_t$
Double angle (2L)	$10\Delta_t$
Rod bracing	$8\Delta_t$
Note: Δ_t is the axial deformation of a brace at yield (N^*=$N_{prob,s}$).	

C6.5.6 Active links of eccentrically braced frames

When subjected to earthquake induced forces, an active link of an EBF responds in either a shear ($e \leq 1.6M_s/V_w$), flexural ($e \geq 3M_s/V_w$) or combined shear and flexural ($1.6M_s/V_w < e < 3M_s/V_w$) mode depending on the clear length of the active link (e).

The probable shear and flexural capacities of an active link should be determined from Section C6.5.2.

The force-rotation relationship of active links is similar to that provided in Figure C6.11. However, $N_{prob,s}$, Δ_y and Δ_{cap} in Figure C6.11 should be replaced with V_w, γ_y and γ_{cap} respectively to obtain the force-rotation relationship for active links. The values of the parameters in the force-rotation relationship are provided in Table C6.8.

Table C6.8: Force-rotation parameters for active links of EBFs

Active link length, e	Deformation* $mrad$
	γ_{cap}
$e \leq 1.6M_s/V_w$	$\gamma_y + 140$
$e \geq 3M_s/V_w$	Same as beams
$1.6M_s/V_w < e < 3M_s/V_w$	Linearly interpolate
Note: γ_y is the rotational deformation of an active link at yielding.	

C6.6 Connection Capacities

C6.6.1 General

Assessing the capacity of a steel frame connection involves determining the load path through the connection, identifying weak links, and then evaluating the probable strength and ductility capacity of those weak links.

The following advice should help when determining the load path through a connection and the weakest link in a load path:

- Determine the internal forces that could be generated in the attached members during an earthquake.
  - An I-section beam responding elastically under flexure will deliver axial forces through the flanges (tension and compression) and vertical shear through the web.
  - An I-section beam responding inelastically under flexure will deliver axial yield forces through the flanges and axial yield forces plus vertical shear through the web.
  - A brace will deliver axial forces (tension is critical) through all its elements.
- Trace the transfer of forces from elements of the supported member into elements of the supporting member that lie parallel to the incoming force. For example, the incoming axial forces from an I-section beam flange connected to an I-section column should be transferred through the column flange into the column web.
- Calculate the probable capacity of all elements along the identified load path in accordance with the provisions of Sections C6.6.3 and C6.6.4.
- If there are no tension and compression stiffeners in a column adjacent to incoming beam flanges in a moment-resisting beam to column connection, then tensile distortion of the flange of the column or compression buckling of the web of the column web are likely to occur before the beam can develop its full flexural capacity.
- The strength and ductility capacity of a load path is determined by the strength and ductility capacity of the weakest component in the load path.
- If various load paths exist through a connection, the stiffest of the load paths will attract the most force.
- Be particularly aware of situations where the connectors (rivets, bolts or welds) may be the weakest component, as their ductility capacity will be limited. One sided fillet welds in tension or bending are particularly vulnerable in this regard, showing no ductility.
- Be aware of component forces introduced when an applied force changes direction along the load path.

Note:

The article by Blodgett (1987) on welds explains the concept of load paths through welded connections and illustrates this with a number of examples.

C6.6.2 Strength modification coefficients

Probable strength of structural steel connections should be taken as the values determined using probable material strengths reduced by the strength modification coefficients provided in Table C6.9.

Table C6.9: Strength modification coefficients for steel connections

Component	Action	Strength modification coefficient	
Bolted connections	Ply in bearing	1.0	
	Bolt shear, tension, and combined actions	0.9	
Pin connections	Ply in bearing	1.0	
	Pin shear, tension, and combined actions	0.9	
		SP	GP
Welded connections	Complete penetration butt welds	1.0	0.7
	Incomplete penetration butt, fillet, plug and slot welds	0.9	0.7

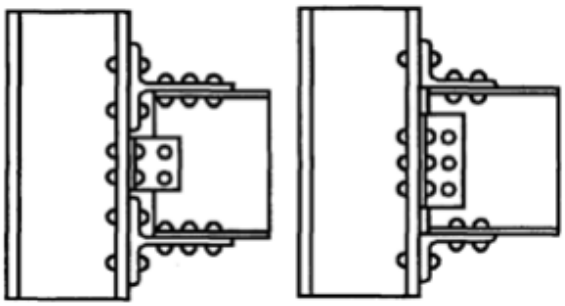
Note:

Strength modification coefficients are not to be confused with strength reduction factors, which for assessment are taken as 1.0. Strength modification coefficients are intended to better define the probable capacity of the defined components.

C6.6.3 Bolted and riveted connections

C6.6.3.1 General

Most old riveted or bolted beam to column connections in New Zealand are believed to be clip angle connections (refer to Figure C6.12(b)). While riveted connections were common in many pre-1950s steel frame buildings, rivets were gradually phased out after this and replaced with bolts as riveting was labour intensive.



(a) Tee-stub **(b) Clip angle**

Figure C6.12: Typical riveted connections (Roeder et al., 1996)

A simplified procedure for determining the moment-rotation relationship of clip angle connections is provided in Section C6.6.3.2. Assessment of other types of historical bolted and riveted connections may be determined using the procedure outlined by Roeder et al. (1996).

The assessment of bolted and riveted connections should be undertaken in accordance with the following:

- Probable shear capacity of rivets, $V_{\text{prob,f}}$, can be determined from Barker (2000). The key equation is derived from the bolt shear capacity provisions of NZS 3404:1997 and is given as:

$$V_{\text{prob,f}} = 0.75 f_{\text{uf}} k_{\text{r}} n_{\text{x}} A_{\text{o}} \quad \text{…C6.9}$$

where:

f_{uf} = probable tensile strength of the rivet
k_{r} = reduction factor given in Table 9.3.2.1 of NZS 3404:1997 to account for the length of a lap connection (L_{j}). $k_{\text{r}} = 1.0$ for $L_{\text{j}} < 300$ mm and for all other type of connections
n_{x} = number of connector shear planes intercepting the shear plane
A_{o} = nominal plain shank area of the rivet.

- Probable tension capacity of rivets should be determined using Clause 9.3.2.2 of NZS 3404:1997, with the value of probable tensile strength of the rivet (f_{uf}) determined from Section C6.4.6 as:

$$N_{\text{prob,tf}} = A_{\text{s}} f_{\text{uf}} \quad \text{…C6.10}$$

where:

A_{s} = gross tensile stress area of the rivet.
f_{uf} = probable tensile strength of the rivet

- The diameter of a rivet shank should be determined from the diameter of the rivet head in accordance with Figure C6.13.
- Be aware that some less scrupulous erectors made up some dummy rivets from moulded putty covered in paint on larger groups of rivets. Hitting each rivet with a hammer will soon identify any dummy ones!
- Assume that concrete encasement, if present and with any amount of confining reinforcement, will prevent local buckling of the steel members. This assumption may not hold for members in regions subject to significant inelastic demand and will need to be assessed more closely for such regions.

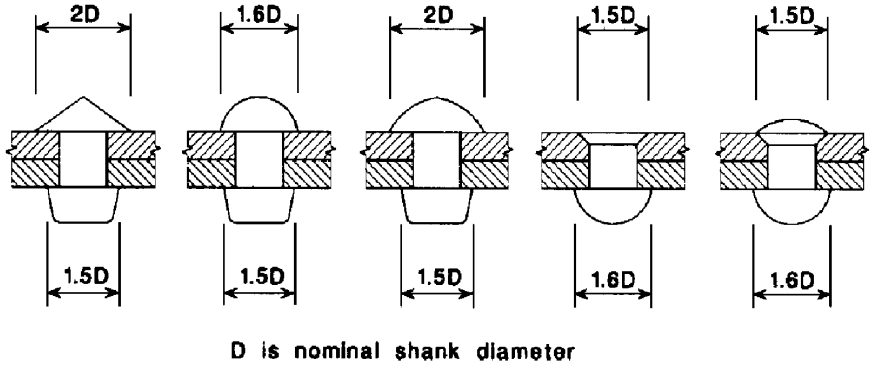


Figure C6.13: Typical rivet shank and head diameters (Bussell, 1997)

- When determining the capacity of a connection, assume that:
  - the connections to beam flanges develop and transfer flexure-induced axial forces from the beam to the column
  - the connections of the beam web to the column flange transfer gravity and earthquake-induced vertical forces and will also transfer horizontal forces, if a suitably stiff and strong horizontal load path from the beam web into the column flange is available, and
  - if there is a direct connection between the beam web and the column flange via welded or bolted plates or cleats, and if this connection is independent of the beam flange to column connection, then for seismic assessment the vertical shear capacity can be assumed to be adequate.

C6.6.3.2 Behaviour of clip angle connections

Clip angle connections are generally weaker and more flexible than other semi-rigid connections and behave as partially restrained connections. The hysteretic behaviour of clip angle connections is relatively poor, but the connections are often able to sustain large deformation demands (Roeder et al., 1996).

The experimental tests undertaken on historical riveted connections by Roeder et al. (1996) revealed that the mode of failure of clip angle connections under cyclic loading was similar to that under monotonic loading. Both monotonic and cyclic load tests deteriorate and fail at similar levels deformation demands, as shown in Figure C6.14. The monotonic tests typically provided an upper bound envelope for the cyclic tests.

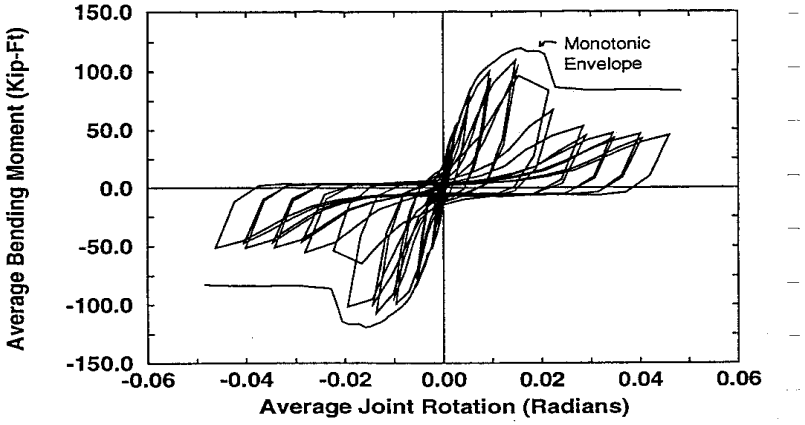


Figure C6.14: Comparison of monotonic and cyclic moment–rotation behaviour

Both concrete encased and bare connections were observed to experience strength degradation at rotations in the order of 20-25 milliradians. It was also observed that concrete encasement improved performance by suppressing any local deformation until the concrete was crushed at larger deformation demands due to lack of adequate confinement.

The capacity enhancement provided by the composite action of concrete encasement and floor slabs to connection capacity was observed to be substantial and in the range of 30-100%. Concrete encasement significantly increased the strength and stiffness of the weaker and more flexible connections such as clip angle connections (refer to Figure C6.15). The capacity of the bare connections was observed to deteriorate significantly when the clip angles to the beam flanges failed. However, flexural capacity was not completely lost because of the resistance provided by the web cleat angle connections.

It should be noted that bolted clip angle connections would be stiffer and would have more rotational capacity than comparable riveted connections. However, the limits on the overall system inelastic displacement would be such that bolted connections cannot attain their full capacity. For example, when the connections are the weakest element, rotational demand on the connections will be around 30 milliradians maximum for an inter-storey drift of 2.5%. Therefore, a 40 milliradians limit on rotation is considered a practical upper limit for the system as a whole, even if the individual connection is capable of greater rotations while maintaining a dependable level of flexural capacity.

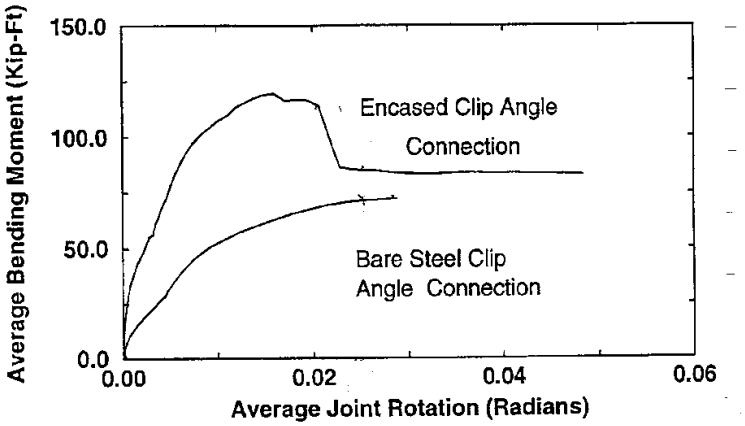


Figure C6.15: Comparison of bare steel and encased moment–rotation behaviour of clip angle connections (Roeder et al., 1994)

C6.6.3.3 Simplified assessment procedure for clip angle connections

General

The strength and rotation capacity of bolted and riveted clip angle connections (illustrated in Figure C6.16) can be determined from first principles and using the guidance presented in this section. The procedure includes a method for determining the probable flexural strength, along with expressions for estimating the probable rotational capacity. Both flexural strength and degradation threshold are considered to be a function of the expected mode of failure of the connections to the beam flanges.

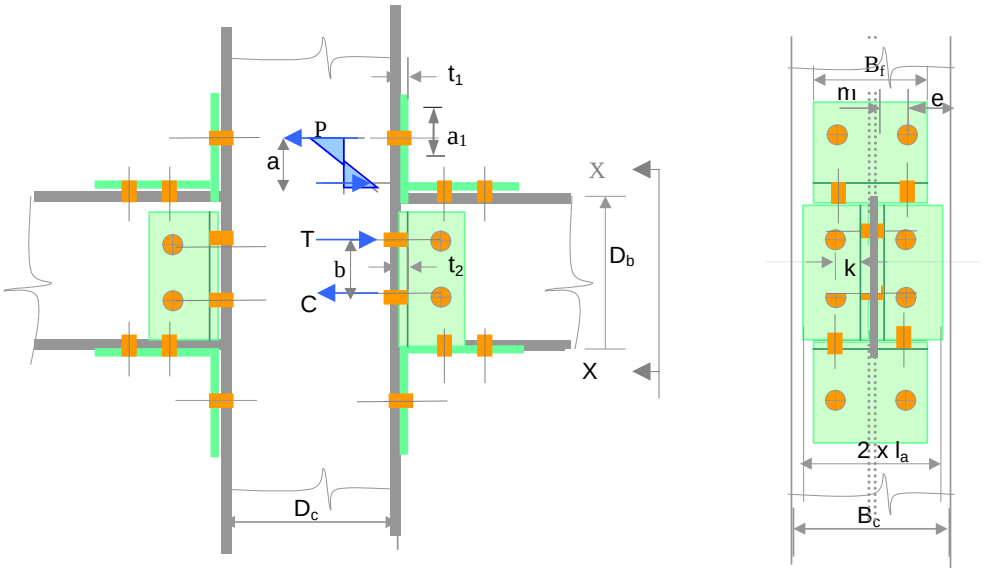


Figure C6.16: A clip angle riveted connection

The flexural strength of a clip angle connection is limited by the smallest demand required to form one of the following four yielding/shear failure modes (Roeder et al., 1996):

- shear yielding/failure of the connectors
- tensile capacity of flange cleat angles
- tensile capacity of connectors, or
- flexural yielding of connection elements (flange cleat angles and/or web cleat angles).

Shear yielding/failure of connectors

Shear yielding/failure of connectors that are provided between the horizontal leg of the flange cleat angles and the beam flange often dictates the flexural capacity of clip angle connections in old buildings.

The probable shear strength of connectors, $V_{\text{prob,f}}$, can be determined from Equation C6.9. The probable flexural strength of a clip angle connection limited by the shear strength of the connectors, M_{prob}, can be determined from:

$$M_{\text{prob}} = nV_{\text{prob,f}}D_{\text{b}} \quad \text{…C6.11}$$

where:

n = the number of connectors
D_{b} = the depth of the beam.

Tensile capacity of flange cleat angles

The strength of the horizontal leg of the flange cleat angle in tension may limit the flexural capacity of clip angle connections. The probable tensile strength of a flange cleat angle $N_{\text{prob,t}}$ can be determined from (NZS 3404:1997):

$$N_{\text{prob,t}} = A_{\text{g}}f_{\text{y}} \leq 0.85k_{\text{te}}A_{\text{n}}f_{\text{u}} \quad \text{…C6.12}$$

where:

A_{g} = gross area of the cross section
f_{y} = probable yield strength of the section
k_{te} = tcorrection factor in accordance with Clause 7.3 of NZS 3404:1997
A_{n} = net area of the cross section
f_{u} = probable tensile strength of the section.

The probable flexural strength of a clip angle connection limited by the tension capacity of the flange angles, M_{prob}, can be determined from:

$$M_{\text{prob}} = N_{\text{prob,t}}\left(D_{\text{b}} + {}^{t_1}/_{2}\right) \quad \text{…C6.13}$$

where:

t_1 = the thickness of the flange cleat angle leg.

Tensile capacity of connectors

The tensile capacity of the connectors provided between the vertical leg of the flange cleat angle and the column flange may also control the flexural strength of a clip angle connection. Experimental tests have shown that this failure mode is the least ductile with a rapidly deteriorating capacity.

The probable tensile strength of connectors, $N_{\text{prob,tf}}$, can be determined from Equation C6.10 and the probable flexural strength of a clip angle connection limited by the probable tensile strength of the connectors, M_{prob} , can be determined from:

$$M_{\text{prob}} = nN_{\text{prob,tf}}\,(d_{\text{b}} + a) \qquad \text{…C6.14}$$

where:

n = the number of connectors
a = the distance between the centre of the connectors and the flange cleat angle leg.

Flexural yielding of flange cleat angles

Flexural yielding of the vertical leg of the flange cleat angle connected to the column flange is the fourth mode that may limit the flexural strength of clip angle connections.

Flexural yielding of the flange cleat angle requires development of prying actions. However, the prying forces that develop in connections that use mild steel connectors are typically not likely to cause the capacity of the connectors to be exceeded.

The probable flexural strength of a clip angle connection reduced by prying actions, M_{prob}, is given as:

$$M_{\text{prob}} = \left(\frac{\frac{B_{\text{f}} t_1^2}{4a} f_{\text{y}} + \frac{a_1}{a} nN_{\text{prob,tf}}}{1 + \frac{a_1}{a}} \right) \left(D_{\text{b}} + {t_1}/{2} \right) \qquad \text{…C6.15}$$

where:

B_{f} = the length of the angle
a_1 = the distance between the centre of the connectors and the top edge of the flange cleat angle.

If the connectors are strong enough to induce flexural yielding of the flange cleat angles, the probable flexural strength can be determined from:

$$M_{\text{prob}} = \frac{B_{\text{f}} t_1^2}{2a} f_{\text{y}}\,(D_{\text{b}} + t_1/2) \qquad \text{…C6.16}$$

Flexural yielding of web cleat angles

If flexural yielding of the flange cleat angle governs the probable flexural strength of a clip angle connection, the flexural strength of the web cleat angle can be considered to contribute to the overall connection strength.

The probable flexural capacity of the web cleat angle can be determined from:

$$M_{\text{prob}} = \frac{l_{\text{a}} t_2^2}{2} f_{\text{y}} \quad \text{…C6.17}$$

where:

l_{a} = the length of the web cleat angle face
t_2 = thickness of the web cleat angle leg
f_{y} = probable yield strength.

From Equation C6.17, the tensile force in the web cleat bolts/rivets is:

$$T = \frac{2M_{\text{prob}}}{k} \quad \text{…C6.18}$$

where:

k = the distance between bolt centreline and the web cleat angle leg.

Probable tensile strength of the column flange is given as:

$$T_{\text{c}} = (4m + 1.25e) t_{\text{c}} f_{\text{yc}} \quad \text{…C6.19}$$

where:

m = distance from centre of bolt hole to radius root at web
e = distance from rivet centre to flange edge
t_{c} = thickness of the column flange
f_{yc} = probable yield strength of the column flange.

The contribution of the web cleat angle to the probable flexural strength of the connection is:

$$M_{\text{prob}} = Qb \quad \text{…C6.20}$$

where:

Q = either T from Equation C6.18 or T_{c} from Equation C6.19, whichever is less
b = the distance between the centroid of tension and compression forces in the web cleat.

C6.6.3.4 Moment-rotation behaviour of riveted clip angle connections

The moment-rotation behaviour of riveted clip angle connections is provided in Figure C6.17 based on the experimental studies undertaken by Roeder et al. (1996). The values of the parameters in the figure are provided in Table C6.10.

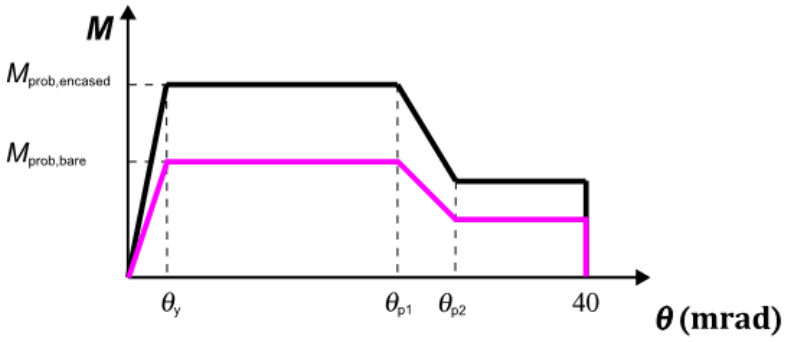


Figure C6.17: Moment-rotation behaviour of clip angle connections

Table C6.10: Moment-rotation parameters for clip angle connections

Mode of failure	Probable yield rotation ($mrad$)	Plastic rotation ($mrad$)		Residual strength
	θ_y	θ_{p1}	θ_{p2}	
Tensile yielding of connectors	5	$3.75/d_b$	$\theta_{p1} + 5$	$0.5M_{prob}$
Shear yielding of connectors	5	$7.5/d_b$	$\theta_{p1} + 5$	$0.5M_{prob}$
Flexural yielding of connecting elements	5	$12.5/d_b$	$\theta_{p1} + 5$	$0.5M_{prob}$

Note:
d_b is depth of beam (m)
$M_{prob,encased} = 2M_{prob,bare}$

C6.6.4 Welded connections

Welded connections are able to transfer the moment-induced beam actions into columns if the various components along the load path have the required capacity as indicated in Table C6.12. The required checks are outlined in Figure C6.18 and Table C6.11.

Note:

As discussed in Section C6.3.2, fractures of welded beam-column connections were widely reported after the 1994 Northridge earthquake, with the majority of these fractures observed at the bottom beam-column flange connections. Refer to that section for more discussion, including a list of the factors considered to have contributed to the brittle failures of "Pre-Northridge" connections (FEMA 355E, 2000).

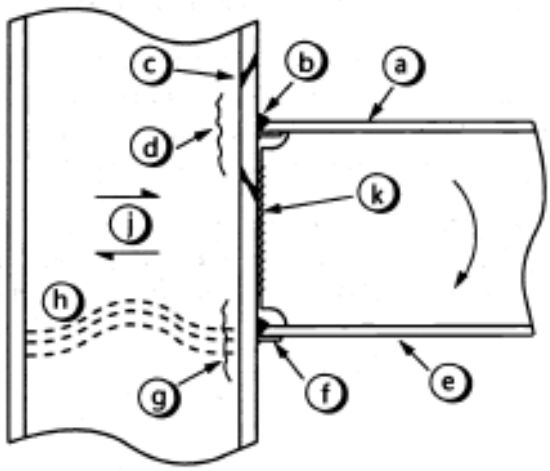


Figure C6.18: Components of welded connections requiring checks (SCI, 1995)

Table C6.11: Components of welded connections requiring checks (SCI, 1995)

Zone	Reference on Figure C6.18	Checklist item
Tension	a	Beam flange
	b	Flange weld
	c	Column flange in bending
	d	Column web in tension
Compression	e	Beam flange
	f	Flange weld
	g	Column web crushing
	h	Column web buckling
Horizontal shear	j	Column web panel shear
Vertical shear	k	Fin plate or direct weld to column

Table C6.12: Probable capacities of components of welded connections requiring checks (NZS 3404:1997; SCI, 1995)

Item	Equation	Equation number
Beam flange capacity	$N_{\text{fbt}} = N_{\text{fbc}} = 1.2[\min(b_{\text{fb}}, b_{\text{fc}})]t_{\text{fb}}f_{\text{yb}}$	…C6.21
Column flange tension capacity*	$N_{\text{fct}} = b_{\text{eff}}t_{\text{fb}}f_{\text{yb}}$ $b_{\text{eff}} = t_{\text{wc}} + 2r_{\text{c}} + 7t_{\text{fc}}$ $b_{\text{eff}} \le t_{\text{wc}} + 2r_{\text{c}} + 7\left[\frac{t_{\text{fc}}^2 f_{\text{yc}}}{t_{\text{fb}}f_{\text{yb}}}\right]$ $b_{\text{eff}} \le b_{\text{fb}} \le b_{\text{fc}}$	…C6.22
Column web tension capacity	$N_{\text{wct}} = [t_{\text{fb}} + 2s_{\text{f}} + 5(t_{\text{fc}} + r_{\text{c}})]t_{\text{wc}}f_{\text{yc}}$	…C6.23
Column web crushing	$N_{\text{wcc}} = (b_1 + n_2)t_{\text{wc}}f_{\text{yc}}$ $b_1 = t_{\text{fb}} + 2s_{\text{f}}$ $n_2 = 5(r_{\text{c}} + t_{\text{fc}})$	…C6.24
Column web buckling	$N_{\text{wcc}} = (b_1 + n_1)t_{\text{wc}}f_{\text{yc}}$ $n_1 = d_{\text{c}}$	…C6.25
Column web panel shear (unstiffened web)+	$V_{\text{c}} = 0.6d_{\text{c}}t_{\text{wc}}\,f_{\text{yc}}\,\eta\left[1 + \frac{3b_{\text{fc}}\,t_{\text{fc}}^2}{d_{\text{b}}d_{\text{c}}t_{\text{wc}}}\right]$ $\eta = \sqrt{(1.15 - (N^*/N_{\text{s}})^2} \le 1$	… C6.26
Beam flange weld	$N_{\text{tfw}} = b_{\text{eff}}f_{\text{uw}}$	…C6.27

Note:

*If $b_{\text{eff}} < 0.7b_{\text{fb}}$, tension stiffeners are necessary to avoid weld tearing at the point of peak stress.

b_1, n_1, & n_2 should be reduced if the column projection is insufficient for full dispersal.

+ If doubler plates are provided, $(t_{\text{wc}} + t_{\text{p}})$ should replace t_{wc} in Equation C6.26.

where:

b_1	=	width of contact between beam flange and welds and column
b_{eff}	=	effective beam flange width
b_{fb}	=	beam flange width
b_{fc}	=	column flange width
d_{b}	=	beam depth
d_{c}	=	column depth
f_{yb}	=	probable yield strength of beam
f_{yc}	=	probable yield strength of column
f_{uw}	=	probable strength of weld metal
n_1	=	length obtained by a 45° dispersion though half the depth of the column
n_2	=	length obtained by a 1:2.5 dispersion though column flange and root radius
N^*	=	axial load in column below joint
N_{fbc}	=	probable compression capacity of beam flange
N_{fbt}	=	probable tension capacity of beam flange
N_{fct}	=	probable tension capacity of column flange
N_{s}	=	probable column section compression capacity
N_{tfw}	=	probable tension capacity of beam flange weld
N_{wcc}	=	probable compression capacity of column web
N_{wct}	=	probable tension capacity of column web
r_{c}	=	column root radius
s_{f}	=	weld leg length to beam tension flange (when available)
t_{fb}	=	beam flange thickness
t_{fc}	=	column flange thickness
t_{p}	=	total thickness of doubler plates
t_{wc}	=	column web thickness
V_{c}	=	probable shear capacity of panel zone.

The demand on beam flanges of welded beam-column connections is determined from:

$$N^*_{\text{fbt}} = \frac{M^*}{d_{\text{b}} - t_{\text{fb}}} - \frac{N^*}{2} \quad \text{…C6.28}$$

$$N^*_{\text{fbc}} = \frac{M^*}{d_{\text{b}} - t_{\text{fb}}} + \frac{N^*}{2} \quad \text{…C6.29}$$

where:

N^*_{fbt} = tension demand on beam flange
N^*_{fbc} = compression demand on beam flange
N^* = axial load in column below joint
M^* = moment in beam.

If the various components of a welded connection do not have the required capacity to resist beam/column overstrength demand, as would be the case for an unstiffened column that is typical of old buildings, the moment-rotation behaviour of the connection should be taken from Table C6.13 and the general shape of the moment-rotation curve should take the form of Figure C6.17.

Table C6.13: Moment-rotation parameters for welded connections

Mode of failure	Probable yield rotation ($mrad$)	Plastic rotation ($mrad$)		Residual strength
	θ_y	θ_{p1}	θ_{p2}	
Flange weld failure	3	$3.75/d_{\text{b}}$	$\theta_{p1} + 5$	$M_{\text{prob,w}}$

Note:
d_{b} is depth of beam (m)
$M_{\text{prob,encased}} = 1.3 M_{\text{prob,bare}}$

$M_{\text{prob,w}}$ is the probable flexural capacity of the beam web column connection and needs to be determined from the particular connection detail adopted. This capacity is determined from:

- the probable capacity of the connection, if the beam web is connected to the column flange using clip angles, or
- the probable plastic flexural capacity of the beam web, if the beam web is connected using balanced, double sided fillet welds, or butt welds of sufficient strength to yield the web in tension.

If a beam-column connection is suspected of being welded but the connection is not visible (e.g. due to concrete encasement) and if no drawings are available, the encasement material should be removed from a representative connection so that a reasonable assessment can be undertaken. The difference in connection moment-rotation capacity between a connection that can transfer the beam flange axial forces induced by inelastic beam action dependably into the column and one that cannot is significantly large that the capacity should be determined and not guessed.

Similarly, the existing state of welds needs to be assessed using visual inspection techniques. Engineers undertaking weld inspections should be familiar with visual inspection techniques such as those outlined by Hayward and McClintock (1999).

C6.7 Global Capacity

C6.7.1 Assumptions

Guidance provided in this section for determining the global capacity of steel framed buildings assumes the following:

- The form of the connections is such that the strengths and the elastic and post-elastic stiffness of the connections can be determined by rational assessment.
- The steel members consist of either solid I-sections or sections built up by plates, which are connected by rivets, bolts or welds, and where the strength of the connectors can be determined by rational assessment.
- The member sizes and connection details can be ascertained with sufficient accuracy to undertake the assessment. This will typically require the availability of structural drawings containing critical details or selective removal of non-structural and concrete encasements surrounding the frames to expose critical members and connections.
- Concrete encasement to the steel frames is considered to play a fire protection role only and is not sufficiently reinforced to contribute significantly to the strength or stiffness of the frames.

If the concrete encasement is well reinforced and is likely to contribute to the strength and stiffness of the steel frame, the contribution of the composite section should be determined. Note that this is very unlikely in pre-1976 building encased beams and is more likely in pre-1976 building encased columns. Column encasement is advantageous as it increases column strength relative to beam strength.

C6.7.2 Global capacity of steel moment resisting frames

C6.7.2.1 General procedure

Determining the global capacity of a steel MRF principally involves identifying the governing inelastic mechanism and the associated deformation capacity, which entails assessing the strength hierarchy throughout the frame.

The influence of inelastic response on overall response is considered to be insignificant on steel MRFs exhibiting the following “good features”:

- The strength hierarchy at all floor levels is beam sidesway except on the uppermost seismic mass level.
- If the connections are the weakest links, the evaluation of the connections in accordance with Section C6.6 shows the following:
  - The weakest components of the connections are not the connectors (welds, rivets and/or bolts). In addition, the capacity of the connections is not limited by the net tension failure of components.
  - When the peak flexural strength of the connections is exceeded, the connections are able to retain their integrity and maintain their shear and axial force carrying capacity.
- None of the beam to column connections has the potential to introduce local buckling or tearing failure in the columns (e.g. lack of stiffeners adjacent to an incoming beam flange in a welded beam to column connection).

- The assessed inelastic response of the system (this assessment is qualitative rather than quantitative) is essentially symmetrical in nature and does not contain features that will inevitably lead to a progressive deformation of the building in one direction only.

If the ductility demand on a steel frame due to XXX%ULS shaking is not significant ($\mu \leq 1.5$) and the frame exhibits the four "good features" listed above, the inelastic response of the frame does not need to be assessed.

A step-by-step hand procedure is provided below on a rapid determination of the global capacity of steel MRFs having either beam sidesway or column sidesway as the governing inelastic mechanism. This procedure is applicable to regular frames that have similar bay widths, floor heights, and floor seismic weights. Refer to Section C2 for the assessment of irregular frames.

Step 1

Determine the probable material strength of the members, the elements of the connections and the connectors. Use probable strengths provided in Section C6.4 in the absence of original construction documentation and physical test data.

Step 2

Determine and assemble the probable capacity of the individual members and connections located on potentially critical floor levels. Refer to Sections C6.5.2 and C6.5.3 for beams and columns respectively, and Section C6.6 for connections.

If the individual beams of the frame on each level under consideration cannot support gravity loading ($G + \psi_{\mathrm{E}} Q$) in a simply supported condition, then halve the plastic rotation capacity of the beams (refer to Section C6.5.2) and of the connections (refer to Section C6.6). The reduction in rotational capacity reflects the monotonic, cumulative nature of inelastic demand on the yielding regions of such members.

If the slab is placed in contact with the columns of a frame or insufficient separation is provided, the contribution of the slab to the flexural strength of the beams should be taken into account.

The assessment should include the first level above the seismic ground level, the uppermost seismic mass level, and floor levels where member sizes and/or connection types change.

Step 3

Determine the governing inelastic mechanism of the frame: i.e. beam sidesway or column sideway mechanism.

A sway potential index (S_{i}) can be employed to determine the potential sway mechanism of a frame. A sway potential index can be defined at a storey of the frame by comparing the sum of the probable flexural strengths of the beams (or connections, whichever are smaller) and the columns at the centroid of every joint:

$$S_{\mathrm{i}} = \frac{\sum(M_{\mathrm{bl}} + M_{\mathrm{br}})}{\sum(M_{\mathrm{ca}} + M_{\mathrm{cb}})} \qquad \ldots \text{C6.30}$$

where:

M_{bl} = probable beam (or connection, whichever is smaller) flexural strength to the left of the joint, extrapolated to the centroid of the connection

M_{br} = probable beam flexural strength (or connection, whichever is smaller) to the right of the joint, extrapolated to the centroid of the connection

M_{ca} = probable column flexural strength above the joint, extrapolated to the centroid of the connection

M_{cb} = probable column flexural strength below the joint, extrapolated to the centroid of the connection.

If:

- $S_{\mathrm{i}} < 0.85$, a beam sidesway mechanism is likely to form. It should be noted that a significant change in storey heights increases the likelihood of a column sidesway mechanism.
- $0.85 < S_{\mathrm{i}} < 1$, either a beam sidesway or column sidesway mechanism is likely to form. The effect of both mechanisms need to be assessed.
- $S_{\mathrm{i}} > 1$, a column sidesway mechanism is likely to form.

Note:

When a frame has semi-rigid connections and these connections are flexurally weaker than the beams or the columns, a beam sidesway mechanism forms.

Step 4

Determine the probable base shear capacity, V_{prob}, of the frame.

If the potential inelastic mechanism of a frame is beam sidesway, the probable base shear capacity, V_{prob}, of the frame can be determined from (refer to Figure C6.19):

$$V_{\mathrm{prob}} = \frac{\sum_{\mathrm{i=1}}^{\mathrm{m}} M_{\mathrm{ri}} + \sum_{\mathrm{i=1}}^{\mathrm{n}} V_{\mathrm{bi}} L_{\mathrm{eq}}}{h_{\mathrm{eq}}} \quad \ldots \text{C6.31}$$

where:

$M_{\mathrm{prob,I}}$ = probable flexural strength of column i at the base

V_{bi} = storey i beam seismic shear demands determined from probable beam flexural strengths as:

$$V_{\mathrm{bi}} = \frac{M_{\mathrm{prob,il}} + M_{\mathrm{prob,ir}}}{L_{\mathrm{b}}} \quad \ldots \text{C6.32}$$

$M_{\mathrm{prob,il}}$ = probable beam (or connection, whichever is smaller) flexural strength to the left of an internal joint on floor i, extrapolated to the centroid of the connection

$M_{\mathrm{prob,ir}}$ = probable beam (or connection, whichever is smaller) flexural strength to the right of an internal joint on floor i, extrapolated to the centroid of the connection

L_{b} = bay width

n = number of storeys

m = number of columns that are fixed at the base

L_{eq} = total width of frame

h_{eq} = effective height of frame to be determined from the displaced shape of the frame as:

$$h_{eq} = \frac{\sum_{i=1}^{n} m_i H_i \Delta_i}{\sum_{i=1}^{n} I} \quad \text{…C6.33}$$

mI_i = mass of floor i
I_i = lateral displacement of floor i
I = height of floor i.

Note:

Equation C6.31 provides an upper bound base shear capacity. For the frame to achieve this upper bound base shear capacity, all of the beams and the bases of the columns should start to yield before the rotational capacity of the critical hinge is exceeded.

If the rotational capacity of the critical hinge is likely to be exceeded before some of the beams and/or the bases of the columns start to yield, the flexural resistance developed in the potential plastic hinges that have not started to yield should replace probable flexural strengths in Equation C6.32.

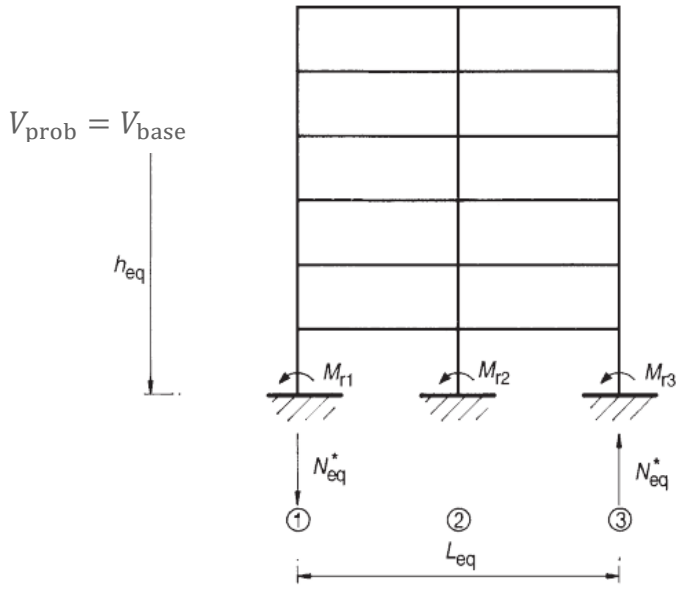


Figure C6.19: Base overturning demand on a beam sidesway governed frame

If the potential inelastic mechanism of a frame is column sidesway, the probable base shear capacity, V_{prob}, of the frame can be estimated from:

$$V_{prob} = \frac{\sum_{i=1}^{m} M_{ri,b} + \sum_{i=1}^{m} M_{ri,t}}{h} \quad \text{…C6.34}$$

where:

$M_{prob,ib}$ = probable column flexural strengths at the base or bottom of column i extrapolated to the centroid of the connection
$M_{prob,it}$ = probable column flexural strengths at the top of column i extrapolated to the centroid of the connection
h = storey height.

Step 5

Ensure the axial force demand on the external columns does not exceed the probable limiting axial force (N_{cr}) from Section 12.8.3.1 of NZS 3404:1997:

$$N^*_{\text{eq}} + N^*_{\text{G}+\psi_E\text{Q}} \leq N_{\text{cr}} \qquad \text{…C6.35}$$

where:

N^*_{eq} = earthquake-induced axial force demand
$N^*_{\text{G}+\psi_E\text{Q}}$ = axial force demand due to gravity loading.

If the above equation is not satisfied, reduce the base shear capacity of the frame until it is.

Step 6

Determine the deformation capacity of the frame.

Refer to Section C2 for methods on determining the deformation capacity of frames.

C6.7.2.2 Steel moment resisting frame systems with infill panels

The interaction between steel MRFs and infill panels should be assessed using the guidance provided in Section C7.

The assessment of infilled steel MRFs should allow for the stiffening effect of infill panels on the overall system response. In addition, the presence of infill panels induces increased shear demands on the frame members by creating short column effects. The increased shear demands are unlikely to exceed the capacity of bare steel or concrete encased solid section columns. However, elements of encased laced and battened members may not have sufficient shear capacity. In addition, if the columns have a better shear capacity than the infills and the infills are likely to sustain significant damage, the potential for a soft-storey formation should be taken into consideration.

Steel moment-resisting infilled frames with weak connections should be assessed for the potential for diagonal compression struts formed in infill panels pulling apart beam to column connections as the frames deform laterally. External beam to column connections are likely to be more critical than internal connections.

Note:

The assessment of weak beam to column connections involves comparison of the tension capacity of the connections with the peak compression capacity of the infill panels (capacity prior to deterioration due to panel crushing/shear failure). If the infill panel compression strut capacity is greater than the beam to external column connection tension capacity, failure of this connection needs to be considered for the response of that end bay.

C6.7.3 Global capacity of concentrically braced steel buildings

Concentrically braced frames (CBFs) are braced frames where the centrelines of the braces intersect at a node. CBFs are commonly X-braced or V-braced (refer to Figure C6.20) and rely primarily on the axial strength and stiffness of the braces to resist lateral forces.

The lateral force capacity of CBFs is dependent on:

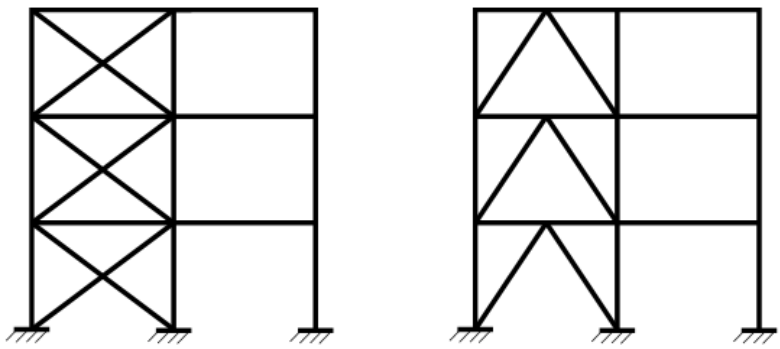
- bracing configuration – X-braced CBFs have an advantage over V-braced CBFs (refer to Figure C6.20, as the inelastic capacity of V-braced CBFs is likely to be governed by the capacity of the collector beam and the post-buckling capacity of the braces only
- the slenderness ratio of the braces – as discussed in Section C6.5.5.1, the slenderness ratio has a significant influence on the deformation capacity and residual strength of compression braces, and
- the capacity of brace connections to the beams and columns of the frame – the connections of the braces should have sufficient capacity to resist demand due to braces yielding in tension or buckling in compression.

Note:

When a compression brace of a V-braced CBF buckles, the capacity of the tension brace may not be fully utilised as the collector beam may not have the capacity to resist the unbalanced vertical force acting at the brace-collector beam joint. Note that the collector beam will have to resist demands due to gravity loads in addition to the unbalanced vertical force.

The buckling of compression braces of V-braced CBFs results in significant reductions in frame lateral stiffness and strength, as the system changes to a D-braced EBF with a long flexural link. In such situations a plastic hinge is likely to form in the collector beam before the tension brace yields in tension.

During the subsequent reversing cycle of earthquake demand, the previously tension brace generally buckles before the braces that buckled during the preceding half cycle fully straighten up (Tremblay and Robert, 2000). Therefore, the inelastic capacity of V-braced frames is limited by the post-buckling capacity of the braces.



(a) X-braced CBFs **(b) V-braced CBFs**

Figure C6.20: Concentrically braced frames (CBFs)

The following steps outline an assessment procedure for X-braced and V-braced CBFs.

Step 1

Determine and assemble the probable capacity of the individual members and connections located on potentially critical floor levels. The capacities to be determined are:

- axial force capacity of the braces
- post-buckling capacity of the braces
- flexural and compression capacity of collector beams
- axial force and flexural capacity of columns
- axial force capacity of connections and splices.

Step 2

Determine the weakest member and the expected mode of failure; i.e. brace, brace connection, collector beam, column, etc.

Step 3

Check whether the frame exhibits the following good features:

- The strength hierarchy involves weak braces at all levels except the uppermost seismic level (rather than weak columns or weak collector beams).
- The columns are continuous over two consecutive storeys.
- The collector beams, columns and the beam to column connections have sufficient capacity to resist the loads generated by the system at the point of brace yielding in tension and brace buckling in compression. In many old braced buildings the brace to beam/column connections are likely to be the weakest components.
- For all beam to column connections the connections should not be of a type that has the potential to introduce local buckling or tearing failure in the column under inelastic rotation due to lack of column tension/compression stiffeners.
- The assessed inelastic response of the system (this assessment is qualitative rather than quantitative) should be essentially symmetrical in nature and not contain features that will inevitably lead to a progressive displacement of the building in one direction.

Step 4

If the ductility demand on the frame due to XXX%ULS shaking is not significant ($\mu \leq 1.5$) and the frame exhibits the above “good features”, the inelastic response of the frame need not be assessed.

If the braces or brace connections are not the weakest component, the capacity of the frames should be limited to the capacity of the weakest member/element if the failure of that member/element constitutes loss of gravity load carrying capacity.

If the brace connections are the weakest component resulting in a rather low lateral force capacity, the frame can be assessed as a moment resisting frame. However, the failure of the brace connections is unlikely to lead to loss of gravity load carrying capacity on their own.

If the ductility demand on the frame due to 100%ULS shaking is significant ($\mu > 1.5$) and the braces are the weakest component, proceed to the next step.

Step 5

Determine the probable base shear capacity of the frame.

The capacity of CBFs can be determined from first principles and the brace member capacity relationships provided in Section C6.5.5.

There is an inherent potential for soft-storey formation in CBFs constructed without following the provisions of NZS 3404:1997. For a typical case of a soft storey forming in the bottom storey of a CBF, the probable base shear capacity, V_{prob} , of a CBF that is effective both in tension and compression can be determined from the post-buckling capacity of the braces in the bottom storey as:

$$V_{\text{prob}} = \sum_{\text{i=1}}^{\text{m}} \alpha'_{\text{ci}} N_{\text{prob,ci}} \; cos\; \theta_{\text{i}} \qquad \text{…C6.36}$$

where:

α'_{ci}	=	residual strength factor for brace i from Section C6.5.5.1
$N_{\text{prob,ci}}$	=	probable compression capacity of brace i
θ_{i}	=	angle between brace i and beam at the top end of the brace
m	=	number of braces.

If a soft storey forms in one of the upper storeys of a CBF, the calculated base shear capacity should allow for the resistance mobilised in the braces that are located in the storeys below the soft-storey level before the capacity of the critical brace is exceeded.

Step 6

Determine the displacement capacity of the frame.

The yield displacement, Δ_{y} , of a CBF may be determined from an elastic analysis of the frame based on the displacements at which the first brace yields in tension (if a tension-only brace) or buckles in compression and a mechanism develops in a storey.

The probable displacement capacity of a single-storey CBF or the probable inter-storey displacement capacity of a multi-storey CBF that is likely to form a soft storey can be determined from the displacement capacity of the critical brace as:

$$\Delta_{\text{cap}} = \sqrt{(L_{\text{b}} + \Delta_{\text{cap,b}})^2 - h^2} - L \qquad \text{…C6.37}$$

where:

L_{b}	=	length of the critical brace
$\Delta_{\text{cap,b}}$	=	displacement capacity of the critical brace from Section C6.5.5.1
h	=	storey height
L	=	width of the braced bay.

C6.8 Assessment of Steel Framed Buildings

C6.8.1 General

Detailed seismic assessments of steel framed buildings, especially those that are considerably old, should not rely solely on drawings. A condition assessment is recommended as part of the DSA and may include inspections to determine:

- any deterioration due to environmental effects
- the physical conditions of members and connections
- configuration and presence of members and connections
- load paths through connections, splices and between members, and
- workmanship.

The global assessment of steel framed buildings may be undertaken using either a displacement or force based assessment procedure as appropriate. This section covers factors specific to the analysis and assessment of these buildings. Refer to Section C2 for overall procedures and appropriate global analysis methods.

C6.8.2 Stiffness of frames

The rotational stiffness of column base connections should be taken into account when undertaking an analysis of steel framed buildings. Fixed column base connections are never infinitely stiff, while pinned column base connections have some rotational stiffness.

Rotational stiffness of column base connections can be determined from NZS 3404:1997 as:

$$k_\theta = \frac{kEI_c}{L_c} \quad \text{…C6.38}$$

where:

k = 1.67 for fixed base connections
k = 0.1 for pinned base connections
I_c = second moment of area of the column about the direction under consideration
L_c = length of column.
E = modulus of elasticity

Note:

Experimental tests undertaken on typical seismic-resisting system foundations have confirmed that the fixed base rotational stiffness recommendation of NZS 3404:1997 is a reasonable value to adopt (AISC, 2012; Borzouie et al., 2016).

When undertaking an elastic analysis of a steel framed building, rigid end blocks having dimensions equal to one half of the beam depth and one half of the column depth should be used at each beam to column connection of the lateral force resisting frame. The use of one half the depth as a rigid end block instead of the full member depth takes account of the flexibility of the panel zone of the connections.

C6.8.3 Seismic actions to ensure ductile mechanisms

As discussed in Section C6.7 a key step in the assessment of an existing steel framed buildings is to check whether non-yielding members and connections (members and connections located outside potential plastic hinge regions) of the primary structure of the building are protected from undergoing inelastic deformations if an overall ductile response of the building is to be assumed.

Comparison of the probable strength of the assumed non-yielding members and connections with the actions generated by the overstrength of potential plastic hinge regions determines whether the members and connections outside these hinge regions of the building are protected. If they are not, development of the full ductile mechanism may not be possible and the overall capacity of the mechanism may reduce.

To ensure that a ductile mechanism can develop as assumed, the assumed non-yielding members and connections should have a probable capacity that is greater than required to resist the actions resulting from yielding the plastic hinge regions at overstrength.

In order to meet the objectives of these guidelines the overstrength actions should be determined using the maximum overstrength factor defined for the particular steel grade in Table 12.2.8(1) of NZS 3404:1997 irrespective of the Category designation.

C6.8.4 Actions on concentrically braced systems

C6.8.4.1 Vertical concentrically braced frames

Non capacity designed CBFs with inelastically responding braces are vulnerable to soft-storey formation. The C_s factor, which needs to be included when determining seismic demand on CBFs in accordance with NZS 3404:1997, accounts for this potential for soft-storey formation and the deterioration in inelastic performance of compression braces with increasing slenderness. The application of the C_s factor limits the ductility demand on CBFs and therefore pushes the seismic response of capacity designed CBFs towards a reliable overall mechanism.

The inelastic demand on braces of Category 3 CBFs is expected to be minimal. Therefore, the C_s factor may be taken as 1.0 when assessing single-storey Category 3 CBFs. The C_s factor for multi-storey Category 3 CBFs and all Category 1 and Category 2 CBFs should be determined in accordance with the provisions of NZS 3404:1997.

C6.8.4.2 Roof X-bracing

The seismic performance of X-braced roof diaphragms is likely to be better than similar vertical CBFs as the roof sheeting system potentially contributes significantly to the stiffness and strength of such diaphragms, especially in light weight systems. However, quantifying this contribution is not straightforward, particularly when the sheeting system has significant openings such as skylights or translucent sheeting.

An X-braced roof diaphragm that remains close to elastic is likely to fulfil its role more reliably than one that yields (noting that there will be a level of earthquake that will cause actions beyond elastic levels unless the actions are limited by a reliable mechanism in the

lateral force resisting system supporting the roof). It should be also recognised that a diaphragm that can reliably yield will likely perform better than one that cannot.

Maximum actions on an X-braced roof diaphragm are principally dependent on the capacity of the lateral force resisting system supporting the roof and the capacity of the connections of the roof bracing system.

They may be taken as one of the following:

- The nominally ductile ($\mu = 1.25$) diaphragm actions determined in accordance with Section C2, in which case $C_s = 1$ should be used.
- If the diaphragm has brace connections capable of yielding the braces in tension at overstrength, diaphragm actions corresponding to a limited ductile response ($\mu = 3$) and $C_s = 1.35$ should be used.
- If a ductile mechanism exists in the lateral force resisting system supporting the roof that would limit the actions in the roof system, the overstrength actions generated by the lateral force resisting system including demand due to out-of-plane response of walls, etc. (if there are any) may be used, provided that the diaphragm is then assumed to be only capable of nominally ductile behaviour.

C6.8.5 Concurrency effects

Columns and their foundations that are part of a two-way seismic-resisting frame should be assessed against concurrent actions as specified in Clause 12.8.4 of NZS 3404:1997.

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Appendix C6A: Typical Pre-1976 Steel Building Systems Used in New Zealand

C6A.1 General

This section gives general guidance on the typical pre-1976 steel building systems used in New Zealand.

Note:

This information is based on published material and details supplied by design engineers.

C6A.2 Moment Resisting Frames

C6A.2.1 Beams

Beams were typically rolled steel joist (RSJ) sections. These are I-sections where the inside face of the flanges is not parallel to the outside face, being at a slope of around 15%. This makes the flanges thicker at the root radius than at the tips.

The flange slenderness ratios of RSJ sections are always compact when assessed to NZS 3404:1997.

These beams were typically encased in concrete for fire resistance and appearance. This concrete contained nominal reinforcement made of plain round bars or, sometimes, chicken wire.

C6A.2.2 Columns

Columns formed from hot-rolled sections

These columns were either RSJs used as columns or box columns formed by connecting two channels, toes out, with a plate to each flange. The columns were encased in lightly reinforced concrete containing nominal reinforcement made of plain round bars.

Compound box columns

These columns were also formed from plates, joined by riveted or bolted angles into a box section and encased in concrete. Examples of this type of construction are shown in Figures C6A.1 and C6A.2.

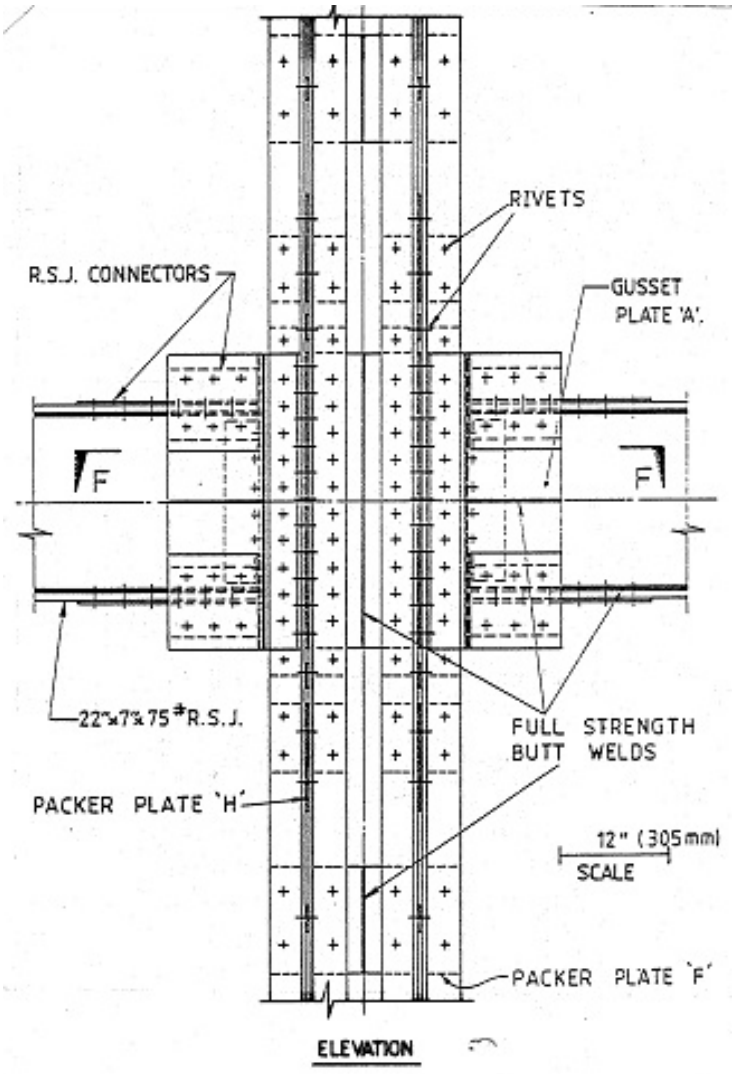

Figure C6A.1: Riveted steel fabrication details, Government Life Insurance Building, 1937 (Wood, 1987)

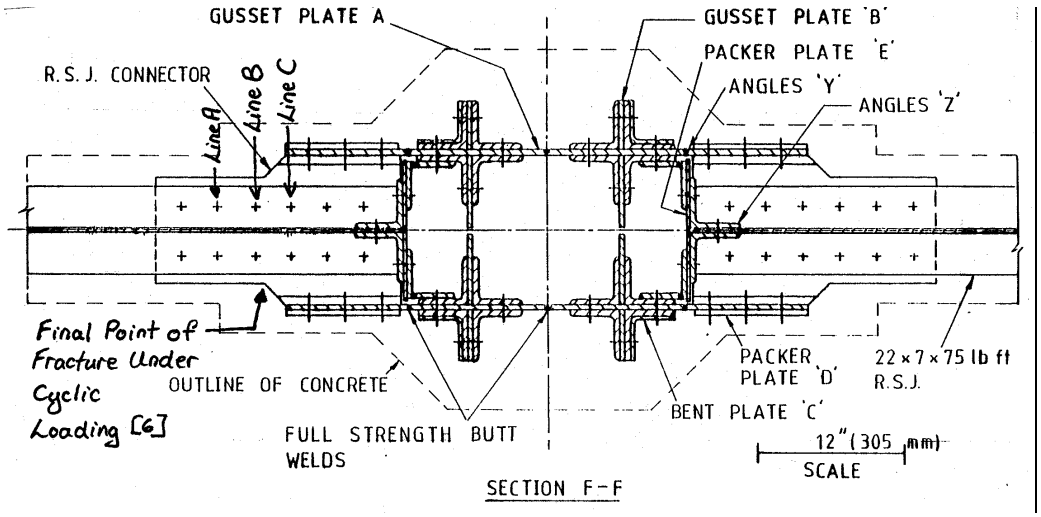

Figure C6A.2: Riveted steel fabrication details, Government Life Insurance Building, 1937 (Wood, 1987)

C6A.2.3 Beam to column connections

Rivets and bolts

Beam to column connections in the earlier moment frames typically comprised semi-rigid riveted or bolted connections. The RSJ beam flanges were bolted to Tee-stubs or angles bolted to the column flanges or to lengths of RSJ bolted to side extensions of the column plates. An example of the latter is shown in Figure C6A.2.

The RSJ beam web was connected by a double clip angle connection to the column flanges, also as shown in Figure C6A.2.

A simpler version of a semi-rigid connection used in some pre-1976 buildings is shown in Figure C6.12.

These joints generally involved the use of rivets up to 1950 and HSFG bolts after 1960, with a changeover from rivets to bolts from 1950 to 1960.

Arc welding

Beam to column connections from about 1940 onwards were also arc welded.

The strength and ductility available from welded connections will need careful evaluation and attention to load path. This topic is addressed in Section C6.6.1 and its importance is illustrated in Figure C6A.3. This figure is taken from a building that collapsed in the Kobe earthquake of 1995 (while this example is from Japan, the details are relevant to some early New Zealand buildings and the concept is certainly relevant). It shows a failed beam to column minor axis connection, forming part of a moment resisting frame in that direction. The beam was welded to an endplate which was fillet welded to the column flange tips.

Unlike the connection detail shown in Figure C6A.2, there was no way to transfer the concentrated axial force in the beam flanges induced by seismic moment reliably from the beam into the column. As a result, the weld between endplate and column flange unzipped under the earthquake action.

Figure C6A.3: Failed beam to column weak axis welded connection from the 1995 Kobe earthquake

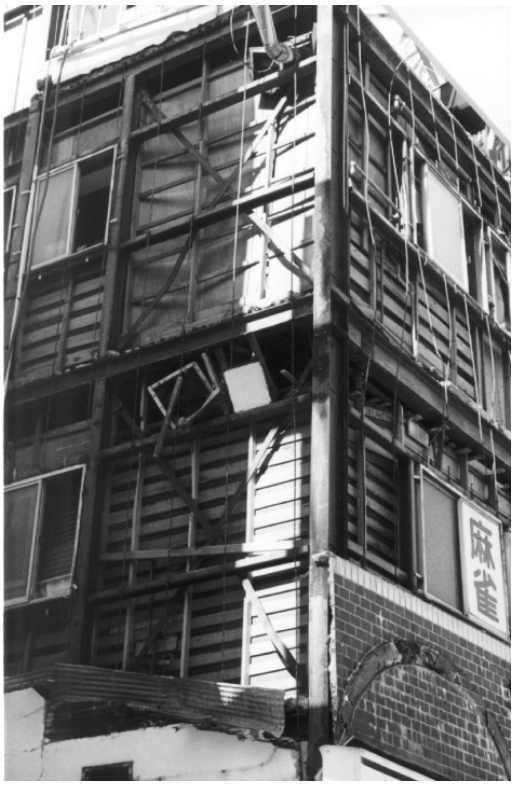

Figure C6A.4: Braced frame with light tension bracing showing damage but no collapse from the 1995 Kobe earthquake

C6A.2.4 Splices in columns

These typically involved riveted (pre-1950) or bolted (post-1950) steel sections, with the rivets or bolts transferring tension across the splice and compression being transferred by direct bearing.

Figures C6A.1 and C6A.2 show plated box columns connected by riveted angles. Figure C6A.3 shows a bolted UC splice detail in the column, a forerunner of the bolted column splice details of HERA Report R4-100 (Hyland, 1999). Such bolted splices generally perform well.

C6A.3 Braced Frames

For the pre-1976 buildings covered by this document, braced frames incorporating steel bracing involved concentrically braced framing (CBF): either X-braced CBFs or V-braced CBFs.

Figure C6A.4 shows an X-braced CBF with relatively light bracing, while Figure C6A.5 shows a V-braced CBF. While both examples are from Kobe, Japan they have similar details to early New Zealand buildings.

Figure C6A.5: V-braced CBF showing damage but no collapse from the 1995 Kobe earthquake

Appendix C6B: Historical Steel Grades and Characteristic/Nominal Strengths

C6B.1 United Kingdom

The characteristic (lower)/nominal material properties of historical UK steelwork are given in Tables C6B.1 to C6B.4. Geometric properties of UK sections can be obtained from publications such as that by Bates (1991).

Table C6B.1: Characteristic/nominal properties of mild structural steels from the UK (Bates, 1991 and Bussell, 1997)

Period	Plate thickness (mm)	Yield strength (MPa)	Tensile strength (MPa)	Ultimate strain (mm/mm)
<1906	All	-*	432	-
1906-48*	All	-*	432	0.2
1948-68	≤19	247	432	0.16
	t >19	232	432	0.16

Note:
*A nominal yield strength of 210 MPa may be used for steel manufactured before 1948 in the UK.

Table C6B.2: Characteristic/nominal properties of mild structural steels from the UK manufactured to BS 4360:1968 (1968-86)

Grade	Plate thickness, t (mm)	Yield strength (MPa)	Tensile strength (MPa)	Ultimate strain (mm/mm)
40 A, B & C	$t \leq 16$	232	402	0.22
	$16 < t \leq 38$	224	402	0.22
40 D & E	$t \leq 16$	263	402	0.22
	$16 < t \leq 38$	247	402	0.22
43 A, B & C	$t \leq 16$	247	432	0.20
	$16 < t \leq 38$	239	432	0.20
43 D & E	$t \leq 16$	278	432	0.20
	$16 < t \leq 38$	270	432	0.20

Table C6B.3: Characteristic/nominal properties of high tensile structural steels from the UK (Bussell, 1997 and Bates, 1991)

Period	Plate thickness, t (mm)	Yield strength (MPa)	Tensile strength (MPa)	Ultimate strain (mm/mm)
1934-65 (BS 548)	$t \leq 32$	355	571	0.14
1943-62	$t \leq 19$	324	541	0.14
(BS 968)	$t > 19$	293	510	0.14
1962-68	$t \leq 16$	355	494	0.15
(BS 968)	16<t≤32	340	494	0.15

Table C6B.4: Characteristic/nominal properties of high tensile structural steels from the UK manufactured to BS 4360:1968 (1968-86)

Grade	Plate thickness, t (mm)	Yield strength (MPa)	Tensile strength (MPa)	Ultimate strain (mm/mm)
50 A, B, C, & D	$t \leq 16$	355	494	0.18
	$16 < t \leq 38$	347	494	0.18
55 C & E	$t \leq 16$	448	556	0.17
	$16 < t \leq 25$	432	556	0.17
	$25 < t \leq 38$	417	556	0.17

C6B.2 Australia

The characteristic (lower)/nominal properties of steelwork provided in Australian standard specifications before the introduction of joint AS/NZ standards in 1996 are given in Tables C6B.5 and C6B.6.

Table C6B.5: Characteristic/nominal strengths of mild structural steels from Australia (Kotwal, 2000)

Period	Grade	Plate thickness, t (mm)	Yield strength (MPa)	Tensile strength (MPa)	Standard
1928-56	A1	All	-*	432	AS A1-1928 (Sections)
1928-37	A1	All	-*	432	AS A1-1928 (Plates)
1937-55	D	All	216	432	AS A33-1937 (Plates)
	E	All	193	386	
	F	All	162	324	
1955-65	D	≤19	236	432	AS A33-1955 (Plates)
	D	>19	228	432	
	E	All	193	386	
	F	All	162	324	
1956-65	A1	≤19	236	432	AS A1-1956 (Sections)
		>19	228	432	
1965-71		$t \leq 19$	247	417	AS A149-1965 (Plates & sections)
		$19 < t \leq 38$	232	417	
		$t > 38$	228	417	
1965-71	A	$t \leq 19$	232	394	AS A135-1965 (Notch ductile steel - plates) (Toughness test requirement introduced)
		$t > 19$	220	394	
	B	$t \leq 19$	247	425	
		$t > 19$	236	425	

Period	Grade	Plate thickness, t (mm)	Yield strength (MPa)	Tensile strength (MPa)	Standard
1966-71	A151	t < 16	355	478	AS A151-1966
		16 < t ≤ 32	348	478	
		t > 32	339	478	
1966-73	20	> 6.4	178	309	AS A157-1966 (Plates)
	24	> 6.4	208	371	
1971-80	250, 250L0	t ≤ 12.5	262	414	AS A186-1971 & AS A187-1971 & AS 1204-1972 (Sections & flat bars)
		< 12.5 < t ≤ 38	248	414	
	350, 350L0	t ≤ 12.5	359	483	
		12.5 < t ≤ 38	345	483	
	WR350	All	345	483	
1971-80	250	t ≤ 9.5	276	414	AS A186-1971 & AS A187-1971 & AS 1204-1972 (Plates)
		9.5 < t ≤ 12.5	262	414	
		12.5 < t ≤ 19	248	414	
		19 < t ≤ 38	232	414	
	300	t ≤ 12.5	310	448	
		t > 12.5	296	448	
	350	t ≤ 12.5	365	483	
		t > 12.5	345	483	
	400	t ≤ 12.5	414	517	
	500	t ≤ 9.5	483	552	
	WR350	All	345	483	
	WR400	All	414	517	
	WR500	All	483	552	
1973-80	180	>6	180	310	AS 1405-1973 (Plates)
	210	>6	210	370	
1980-90	200	All	200	300	AS 1204-1980 (Sections, flat bars and plates)
	A revision of AS 1204-1972 and AS 1405-1973. Grades 180 and 210 plates were replaced by new grade 200 plates. Grades 300,400 and 500 plates were removed. The rest remained the same as in AS 1204-1972.				
1980-90	WR350	All	345	483	AS 1205-1980 (Sections, flat bars and plates)
1990-96	200	t ≤ 12	200	300	AS 3678-1990 (Plates)
	250	t ≤ 8	280	410	
		8 < t ≤ 12	260	410	

Period	Grade	Plate thickness, t (mm)	Yield strength (MPa)	Tensile strength (MPa)	Standard
		$12 < t \leq 50$	250	410	
	300	$t \leq 8$	320	430	
		$8 < t \leq 12$	310	430	
		$12 < t \leq 20$	300	430	
		$12 < t \leq 150$	280	430	
	350	$t \leq 12$	360	450	
		$12 < t \leq 20$	350	450	
		$20 < t \leq 80$	340	450	
	400	$t \leq 12$	400	480	
		$12 < t \leq 20$	380	480	
		$20 < t \leq 50$	360	480	
	WR350	$t \leq 50$	340	450	
1990-96	250*	$t \leq 12$	260	410	AS 3679-1990 (Sections & flat bars)
		$< 12 < t \leq 40$	250	410	
	350*	$t \leq 12$	360	480	
		$t < 12 \leq 40$	340	480	
	WR350	All	340	480	
	BHP Australia replaced most of their Grade 250 sections with new Grade 300 sections in 1994, while BHP New Zealand replaced their Grade 350 sections with new Grade 300 sections				

Note:
*A nominal yield strength of 210 MPa may be used for steel manufactured before 1937 in Australia.

Table C6B.6: Characteristic/nominal strengths of hollow structural steels from Australia

Period	Grade	Yield strength (MPa)	Tensile strength (MPa)	Standard
1973-81	200	210	-	AS 1163-1973
	250	250	-	
	350	360	-	
1981-91	C200 and H200	200	320	AS 1163-1981
	C250 and H250	250	350	
	H350	350	450	
1981-88	C350	350	450	AS 1163-1981
1988-91	C350	350	430	AS 1163-1981 (Amd 2)
1991-09	C250	250	320	AS 1163-1991
	C350	350	430	
	C450	450	500	

C6B.3 Australia/New Zealand

The first joint Australian and New Zealand structural steel specifications were introduced in 1996. Characteristic (lower)/nominal strengths outlined in these joint specifications are given in Tables C6B.7 and C6B.8.

Table C6B.7: Characteristic/nominal strengths of mild structural steels to AS/NZS 3678 and AS/NZS 3679

Period	Grade	Plate thickness (mm)	Yield strength (MPa)	Tensile strength (MPa)	Standard
1996-now	Same as 3678-1990, but a new grade 450 is added.				AS/NZS 3678:1996 AS/NZS 3678:2011
	450	$t \le 20$	450	520	
		$20 < t \le 32$	420	500	
		$32 < t \le 50$	400	500	
1996-2010	250	$t \le 11$	260	410	AS/NZS 3679:1996 (Plates)
		$11 < t \le 40$	250	410	
	300	$t \le 11$	320	440	
		$11 < t \le 17$	300	440	
		$17 < t \le 40$	280	440	
	350	$t \le 11$	360	480	
		$11 < t \le 40$	340	480	
	400	$t \le 17$	400	520	
		$t > 17$	380	520	
2010-now	Grade 250 and 400 sections removed. A new S0 grade introduced. The rest remained the same as AS/NZS 3679:1996.				AS/NZS 3679:2010 (Plates)

Table C6B.8: Nominal strengths of hollow structural steels to AS/NZS 1163:2009

Period	Grade	Yield strength (MPa)	Tensile strength (MPa)	Standard
2009-now	C250	250	320	AS/NZS 1163:2009
	C350	350	430	
	C450	450	500	

C6B.4 USA and Continental Europe

Material and geometric properties of historical continental sections can be obtained from publications such as those by Bates (1991) and SB4.6 (2007).

Structural steelwork imported from the USA before the 1960s is likely to have a lower yield strength than that imported from the UK (refer to Table C6B.9). Geometric properties of US sections can be obtained from publications such as that by Ferris (1954).

Table C6B.9: Characteristic/nominal strengths for steels manufactured in the USA for buildings, based on Ferris (1954) and ASCE 41-13 (2014)

Period	Yield strength (MPa)	Tensile strength (MPa)
<1900	165	248
1901–08	207	414
1909–23	193	379
1924–31	207	379
1932–60	228	417

C6B.5 Steels of Unknown Origin

When the origins of structural steelwork cannot be confirmed, the default nominal strengths in Table C6B.10 should be used.

Table C6B.10: Nominal strengths for structural steels of unknown origin

Time period	Yield strength (MPa)	Tensile strength (MPa)
Pre-1948	210	-
1948–Now	230	-

PART C

Moment Resisting Frames with Infill Panels C7

Document Status

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This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

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1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

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Please go to www.EQ-Assess.org.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.EQ-Assess.org.nz

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C7. Moment Resisting Frames with Infill Panels

C7.1 General

C7.1.1 Scope and outline of this section

This section sets out the guidelines for the seismic assessment of structural steel or reinforced concrete moment resisting frames with masonry infill panels, with or without openings. Infill panels can consist of:

- unreinforced clay brick masonry
- hollow clay brick masonry (which can be filled or unfilled, reinforced or unreinforced), or
- concrete block masonry (which can be solid or hollow; unfilled, partially filled or fully filled; and reinforced or unreinforced).

These guidelines are valid for walls in good condition with negligible mortar joint cracking or brick splitting other than some minor damage due to settlement or similar factors which, by observation, are unlikely to be detrimental to their performance during an earthquake.

The assessment procedures presented cover in-plane effects on the frame elements and the panel and also the assessment of face-loaded capacities.

Note:

Frames with infill panels have been used in New Zealand since the early 1920s. Masonry infill panels modify the seismic response of the adjacent frame elements in terms of stiffness, the nature of the applied loads and local ductility demands.

Many of the behaviour issues associated with frames with infill panels arise from discontinuities of infill, resulting in soft storeys or non-uniform distribution of storey stiffness. This in turn leads to a high concentration of seismic loading to be transferred amongst the associated elements. Consequently, if infill panels are present in a building, it is important that their influence on its seismic performance is explicitly considered.

C7.1.2 Useful publications

Useful information on materials, inspection and assessment of infill frames is contained in:

ASCE 41-13 (2014). *Seismic evaluation of existing buildings,* American Society of Civil Engineers and Structural Engineering Institute, Reston, Virginia, USA.

FEMA 306 (1998). *Evaluation of earthquake damaged concrete and masonry wall buildings – Basic Procedures Manual*, Applied Technology Council, Redwood City, California.

Flanagan, R.D. and Bennett, R.M. (1999). *Arching of masonry infilled frames: comparison of analytical methods*, ASCE Practice Periodical on Structural Design and Construction, 4(3), 105-110.

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MSJC (2011). TMS 402-11: *Building code requirements for masonry structures*, Masonry Joint Standards Committee, Reston, Virginia.

Stavridis, A. (2009). *Analytical and experimental study of seismic performance of reinforced concrete frames with masonry walls*, PHD Dissertation, University of California, San Diego.

Turgay, T., Durmus, M.C., Binici, B. and Ozcebe, G. (2014). *Evaluation of the predictive models for stiffness, strength and deformation capacity of RC frames with masonry infill walls*, ASCE Journal of Structural Engineering, 06014003, 1-9.

C7.1.3 Definitions and acronyms

Action	Set of concentrated or distributed forces acting on a structure (direct action), or deformation imposed on a structure or constrained within it (indirect action). The term 'load' is also often used to describe direct actions.
Beam	A member subjected primarily to loads producing flexure and shear
Column	A member subjected to loads producing flexure shear and axial actions
Dead load	The weight of the building materials that make up a building, including its structure, enclosure and architectural finishes. The dead load is supported by the structure (walls, floors and roof).
Ductile/ductility	Describes the ability of a structure to sustain its load-carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
Face-loaded walls	Walls subjected to out-of-plane inertial forces. Also see Out-of-plane load.
Gravity load	The load applied in a vertical direction, including the weight of building materials (dead load), environmental loads such as snow, and building contents (live load)
Infill panel	A panel of masonry bounded by beams and columns and constructed partially or fully within the plane of a steel or reinforced concrete frame
Infill wall	See infill panel
In-plane load	Load acting along the wall length
Irregular building	A building that has an irregularity that could potentially affect the way in which it responds to earthquake shaking. A building that has a sudden change in its plan shape is considered to have a horizontal irregularity. A building that changes shape up its height (such as one with setbacks or overhangs) or that is missing significant load-bearing elements is considered to have a vertical irregularity. Structural irregularity is as defined in NZS 1170.5:2004.
Lateral load	Load acting in the horizontal direction, which can be due to wind or earthquake effects
Load	See Action
Masonry	Any construction in units of clay, stone or concrete laid to a bond and joined together with mortar
Moment resisting frame (MRF)	A building frame system in which lateral loads are resisted by shear and flexure in members and joints of the frame
Mortar	The cement/lime/sand mix in which masonry units are bedded
Out-of-plane load	Load acting at right angles to the wall surface. Walls subjected to out-of-plane shaking are referred to as face-loaded walls.
Soft storey	A level (storey) in a multi-storey building which is weaker than the levels above
Wythe	A continuous vertical section of masonry one unit in thickness. A wythe may be independent of, or interlocked with, the adjoining wythe(s).

C7.1.4 Notation, symbols and abbreviations

Unless otherwise stated, the notation in this section has the following meanings:

Symbol	Meaning
a	Width of equivalent diagonal compression strut
A_{op}	Area of the opening in an infill panel
A_{wtot}	Gross area of an equivalent infill panel with no openings
C	Residual strength ratio
$C_{\text{i}}(T_{\text{p}})$	Part spectral shape coefficient from NZS 1170.5:2004. Refer to Section C3.
d	Effective member depth to the centroid of the tension reinforcing steel
d'	Effective member depth to the centroid of the compression reinforcing steel
E_{bb}	Probable moduli of elasticity of the bounding beam
E_{bc}	Probable moduli of elasticity of the bounding column
E_{f}	Probable modulus of elasticity of frame material
E_{m}	Probable modulus of elasticity of infill material
f'_{m}	Probable masonry compressive strength
F_{ph}	Seismic out-of-plane demands on masonry infill walls
f_{y}	Probable yield strength of reinforcement
h_{col}	Column height between centre lines of beam
h_{inf}	Height of infill panel
I_{bb}	Moment of inertia in the plane of the infill for the bounding beam
I_{bc}	Moment of inertia in the plane of the infill for the bounding column
j_{d}	Internal lever arm
K	Empirical constant
K_{ini}	Uncracked stiffness of perforated infill panel
$K_{\text{ini}}^{\text{solid}}$	Stiffness of an equivalent infill frame with a solid panel
l_{beff}	Effective beam length
l_{ceff}	Effective column length
L_{inf}	Length of infill panel
$M_{\text{o}}^{\text{beam}}$	Overstrength moment capacity of the beam
$M_{\text{o}}^{\text{col}}$	Overstrength moment capacity of the column
P	Axial load on the infill panel
q_{prob}	Probable uniformly distributed lateral load capacity
$q_{\text{prob}}^{\text{solid}}$	Probable uniformly distributed lateral load capacity of an equivalent infill panel with no openings
r_{inf}	Diagonal length of infill panel

t	Thickness or net thickness of infill panel depending on application. Refer particular equations
T_{p}	Period of a part from NZS 1170.5:2004
V_{beam}	Shear demand on reinforced concrete beam
V_{cc}	Probable corner crushing strength of the infill panel
V_{col}	Shear demand on reinforced concrete column
V_{fre}	Probable expected storey shear strength of the bare frame
V_{in}	Probable infill in-plane shear strength
V_{prob}	Probable expected in-plane strength of solid infill panel
V_{s}	Shear resistance provided by the steel shear reinforcement
α	Corner-to-corner crack angle measured to the axis of the member
α_{arch}	Horizontal arching coefficient
α_{b}	Coefficient - refer to Equation C7.25
α_{c}	Coefficient - refer to Equation C7.24
β	Ratio of the frame to infill strength
β_{arch}	Vertical arching coefficient
γ	In-plane cracking capacity reduction coefficient
Δ_{prob}	Probable deformation (drift deflection) capacity of masonry infill
θ	Angle whose tangent is the infill height-to-length aspect ratio
θ_{b}	Infill strut angle for determining reinforced concrete beam actions
θ_{c}	Infill strut angle for determining reinforced concrete column actions
μ_{p}	Ductility of the part in accordance with NZS 1170.5:2004
ρ_{w}	Volumetric ratio of the infill panel reinforcement

C7.2 Typical Building Practices in New Zealand

Building construction comprising reinforced concrete frames with masonry infill was most common in New Zealand between the early 1920s and the mid-1960s. The style most commonly adopted at the time was for masonry infill panels along the building length transverse to the street frontage to have few or no openings, while the street frontage and rear infill walls had extensive openings (refer to Figure C7.1).

Figure C7.1: Examples of reinforced concrete frames with clay brick masonry infill (Kevin Walsh and Laura Putri)

Masonry infill walls built before the 1950s were generally made of unreinforced clay brick masonry (typically dimensioned 220 mm x 110 mm x 74 mm) mortared directly to the infill frames on all four sides (i.e. with no seismic gap) (Kam et al., 2011). These unreinforced clay brick masonry walls typically (but not always) consisted of two single wythes with a central internal air cavity and were often plastered or painted (refer to Figure C7.2).

Figure C7.2: Typical clay brick cavity infill wall construction with plaster façade and single brick removed, exposing the air cavity separating two single wythes of clay brick (Kevin Walsh)

Hollow clay brick masonry infill walls were also used in New Zealand during this period but were less common (refer to Figure C7.3).

Figure C7.3: Example of a reinforced concrete frame with terracotta masonry infill (Kevin Walsh)

Seismic gaps between the infill walls and bounding frames became more common with the adoption of the 1965 Model Building Bylaw (NZSS 1900:1965). At this point, lightly reinforced concrete block masonry (typically dimensioned 190 mm × 190 mm x 390 mm) became more popular for use as infill (refer to Figure C7.4).

Figure C7.4: Examples of reinforced concrete frames with concrete block masonry infill (Laura Putri)

Masonry infill within reinforced concrete frames became less popular with the adoption of the 1970s loading and design standards (Kam et al., 2011). However, concrete block masonry infill within steel frames continued to be used into more modern times, particularly in large storage and industrial buildings (refer to Figure C7.5).

Figure C7.5: Examples of steel frame industrial building with concrete block infill (David Biggs)

Concrete block masonry infill walls can also be found in relatively modern reinforced concrete frame structures (i.e. built from the 1980s onwards). In these structures block infill walls are frequently used as fire rated walls, often adjacent to site boundaries. However, while these infill walls were typically separated from the adjacent columns with seismic gaps, in many instances – particularly in buildings designed before the introduction of the loadings standard NZS 4203:1992 – the width of the seismic gap will not be sufficient to accommodate significant frame deflections (refer to Section C2).

C7.3 Factors Affecting Seismic Performance of Masonry Infill Panels and Buildings Containing Them

The seismic behaviour of moment resisting frames with masonry infill is complex. If the gaps between the infill panel and the frame cannot accommodate the seismic deformations, the elements will interact. The infill panels can add considerable strength and stiffness to the system when they are behaving essentially elastically, and this can significantly alter the seismic response of a building.

For buildings located on corner sites the presence of infill panels on the non-street boundaries can result in an undesirable torsional structural response during an earthquake which may not have been anticipated in the original building design and may significantly alter the seismic demands on other elements in the building (e.g. bare frames on the street frontages).

Gaps between the infill panel and the frame can arise from:

- the construction process not providing a tight infill
- shrinkage, or
- deliberate allowance for anticipated building drifts when the building was originally designed.

If there is only a nominal gap between the frame and the infill panel (as occurs from shrinkage) the components will initially act in a fully composite fashion, as a structural wall with boundary elements. As lateral deformations increase, the behaviour becomes more complex as a result of the frame attempting to deform in a shear mode. Separation occurs between the frame and the panel at the corners on the tensional diagonal, and a diagonal compression strut develops on the compression diagonal (refer to Figure C7.6). Localised contact occurs between the frame and the panel (Paulay and Priestley, 1992).

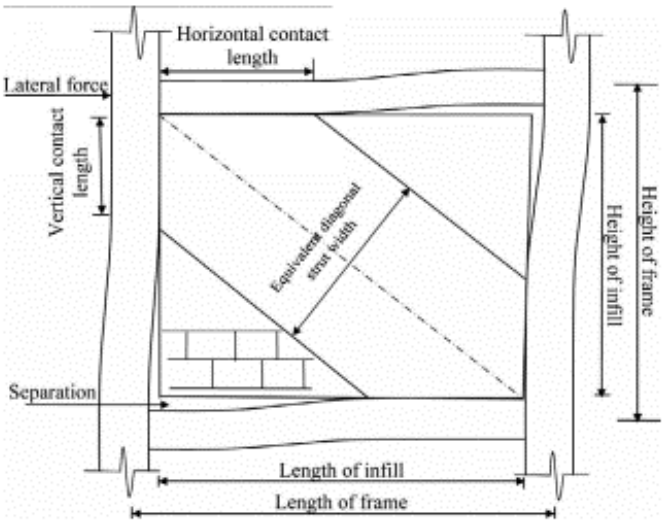


Figure C7.6: Infill frame behaviour when subject to seismic loading (Halder et al., 2013)

A number of different failure modes are possible for masonry infill frames. These include:

- tension or compression failure of the frame elements
- shear failure of the masonry infill panel
- corner crushing compression failure of the infill panel
- flexural or shear failure of the frame elements
- out-of-plane failure of the infill panel, and
- tensile failure of beam to column connections due to compressive prying action from the infill panel.

The out-of-plane performance of the infill panel itself can be significantly enhanced if the infill is tight within the frame. However, the degree of restraint provided will depend on the integrity of the mortar packing between the infill and the frame (which should always be confirmed before any reliance is placed on it).

Each of these failure modes needs to be considered when assessing the building's earthquake rating, as described below. In many situations mixed failure modes may occur. A common mixed failure mode is shear failure of the infill panel which then initiates a flexural or shear failure of the frame elements. Another commonly observed mixed failure mode is reduction of out-of-plane capacity of the infill panel due to infill panel cracking associated with in-plane demands. This is because the in-plane cracking reduces the ability of the panel to resist out-of-plane loads by arching action.

The dynamic behaviour of buildings with infill frames can change significantly during earthquakes as a result of damage sustained by the infill panels. For example, out-of-plane damage to infill panels along one side of a building could result in a torsional response which might be detrimental to the global performance of the building.

C7.4 Observed Seismic Behaviour of Masonry Infill Panels and Buildings Containing Them

Extensive damage to, and collapse of, masonry infill panels was observed during the 2009 L'Aquila earthquake sequence in central Italy. Many masonry infill panels within reinforced concrete framed buildings failed primarily due to out-of-plane mechanisms that developed because of inadequate or absent connections between the inner and outer wythes of masonry (Braga et al., 2011) and potentially poor packing between the infill and the frame (refer to Figure C7.7). Individual wythes (typically the outer wythe) often collapsed separately from their counterparts due to the high slenderness ratios associated with their non-composite response (Verderame et al., 2009).

While infill panels contributed initially to the strength and stiffness of the frame systems, in several cases soft-storey mechanisms formed during aftershocks as a result of the collapse of infill walls at the same floor level in preceding earthquakes (Augenti and Parisi, 2010).

Figure C7.7: Masonry infill frame damage observed following the 2009 L'Aquila earthquakes (Win Clarke)

By comparison, few buildings constructed with masonry infill walls experienced collapse in Christchurch during the September 2010 or February 2011 earthquakes. However, partial height masonry infills caused short column effects and corresponding reinforced concrete column shear cracking in one building, leading to partial collapse of the frame in a June 2011 aftershock (Kam et al., 2011, and also refer to Figure C7.8). Flexural-shear damage to masonry infill walls from in-plane loading was observed in some cases, while out-of-plane collapse of masonry infill was observed in at least one case.

Figure C7.8: Masonry infill frame damage observed following the 2010–2011 Canterbury earthquakes (Kam et al., 2011)

C7.5 Material Properties

C7.5.1 Clay brick masonry

Material properties for assessing the capacity of clay brick masonry infill walls can be determined in accordance with Section C8, except that the Young's modulus of clay brick masonry should be taken as:

$$E_{\mathrm{m}} = 700\, f'_{\mathrm{m}} \qquad \text{…C7.1}$$

where:

f'_{m} = probable masonry compressive strength (MPa).

Alternatively Young's modulus of clay brick masonry can be determined by field testing in accordance with Appendix C8A.

C7.5.2 Concrete block masonry

Young's modulus of concrete block masonry should be taken as:

$$E_{\mathrm{m}} = 900\, f'_{\mathrm{m}} \qquad \text{…C7.2}$$

where:

f'_{m} = probable masonry compressive strength (MPa).

C7.5.3 Reinforced concrete

Material properties for assessing the capacity of reinforced concrete frame elements can be determined in accordance with Section C5.

C7.5.4 Structural steel

Material properties for assessing the capacity of structural steel components can be determined in accordance with Section C6.

C7.6 Assessment of Masonry Infill for Out-of-Plane Actions

C7.6.1 Probable out-of-plane strength

C7.6.1.1 General

The predominant out-of-plane resisting mechanism for masonry infills is horizontal and vertical arching action. The out-of-plane resistance of masonry infill, as calculated in the equation below, is based upon an arching model of the infill in the bounding frame. Therefore, it neglects the contribution of any reinforcement that may be present in the infill in determining the out-of-plane flexural strength of participating infill.

The out-of-plane probable strength of an infill wall may be taken as:

$$q_{\text{prob}} = 730\gamma\,(f'_{\text{m}})^{0.75}\,t^2\left(\frac{\alpha_{\text{arch}}}{L_{\text{inf}}^{2.5}} + \frac{\beta_{\text{arch}}}{h_{\text{inf}}^{2.5}}\right) \quad \text{…C7.3}$$

where:

q_{prob} = probable uniformly distributed lateral load capacity (kPa)
f'_{m} = probable masonry compressive strength (MPa)
h_{inf} = clear height of infill panel (mm)
t = thickness of infill panel (mm), not to exceed $\frac{1}{8}h_{inf}$
L_{inf} = length of infill panel (mm)
α_{arch} = horizontal arching coefficient
β_{arch} = vertical arching coefficient
γ = in-plane cracking capacity reduction coefficient.

Horizontal and vertical arching coefficients can be calculated as:

$$\alpha_{\text{arch}} = \frac{1}{h_{\text{inf}}}\left(E_{\text{bc}}\,I_{\text{bc}}\,h_{\text{inf}}^2\right)^{0.25} \leq 50 \quad \text{…C7.4}$$

$$\beta_{\text{arch}} = \frac{1}{L_{\text{inf}}}\left(E_{\text{bb}}\,I_{\text{bb}}\,L_{\text{inf}}^2\right)^{0.25} \leq 50 \quad \text{…C7.5}$$

where:

E_{bc} = probable moduli of elasticity of the bounding column (MPa)
E_{bb} = probable moduli of elasticity of the bounding beam (MPa)
I_{bc} = moment of inertia in the plane of the infill for the bounding column (mm^4)
I_{bb} = moment of inertia in the plane of the infill for the bounding beam (mm^4).

In-plane cracking capacity reduction coefficient can be calculated as:

$$\gamma = 1.1\left(1 - \frac{h_{\text{inf}}}{55t}\right) \leq 1.0 \quad \text{…C7.6}$$

Note:

The equation for estimating probable out-of-plane strength of masonry infill has been derived based on the work of Dawe and Seah (1989) and verified against a large experimental data set compiled and analysed by Flanagan and Bennett (1999) from seven different test programs. These included clay brick infills in concrete frames, clay tile infills in steel frames, clay brick infills in steel frames, and concrete masonry infills in steel frames. The experimental tests involved infills with height-to-thickness ratios ranging from 6.8 to 35.3, which represent the limits for use of the recommended method.

Equation C7.3 includes a capacity reduction coefficient to account for the reduction in out-of-plane strength due to prior in-plane cracking derived from a limited number of tests completed by Angel et al. (1994) on masonry infill with reinforced concrete frame elements.

When columns of different cross-sectional properties are used on either side of the infill, average properties should be used to calculate the capacity. When beams of different cross-sectional properties are used above and below the infill, average properties should be used to calculate this capacity. In the case of a single storey frame, the cross-sectional properties of the bounding beam above the infill should be used to calculate this capacity.

Allowances should be made for the effects of cracking on the cross-sectional properties of reinforced concrete frame elements when they are present. Recommended procedures in NZS 3101:2006 Concrete Structures Standard can be used to determine cross-sectional properties of reinforced concrete frame elements.

When a side gap greater than $0.02t$ is present, α_{arch} should be taken as zero. When a top gap greater than $0.02t$ is present, β_{arch} should be taken as zero.

Columns and beams with infill on both sides in the plane of the infill are likely to be flexurally rigid due to opposing arching actions from either side. Hence, values for α_{arch} and $\beta_{\text{arch}} < 50$ should generally only be applied at building corners, in top storeys, and in frame bays adjacent to portal openings.

It should be noted that q_{prob} is the resistance due to arching action of the panel as it deflects. At low loads, lateral restraint preventing the panel from moving out of the frame is necessary to enable arching action to begin.

The probable capacity of infills containing openings can be obtained using Equation C7.7:

$$q_{\text{prob}} = q_{\text{prob}}^{\text{solid}} \left(1 - \frac{A_{\text{op}}}{A_{\text{wtot}}}\right) \qquad \text{…C7.7}$$

where:

q_{prob} = probable uniformly distributed lateral load capacity
$q_{\text{prob}}^{\text{solid}}$ = probable uniformly distributed lateral load capacity of an equivalent infill panel with no openings
A_{op} = area of the opening in the infill panel
A_{wtot} = gross area of an equivalent infill panel with no openings.

Note:

Equation C7.7 has been derived from the study reported by Mays et al. (1998), who considered yield line theory and finite element modelling for out-of-plane loads applied to concrete panels. The engineer should be aware that this equation has had limited experimental validation. In particular, the equation may not be applicable for infill panels with large openings ($A_{\text{op}}/A_{\text{wtot}} > 0.2$) when, as a consequence of the penetration size, two-way arching may not be possible.

Alternatively, a more conservative result is obtained by neglecting arching effects and using the procedures presented in Section C8 for the out-of-plane response of unreinforced masonry walls without bounding frames.

Generally, the stiffness and strength of the boundary frame will be sufficient to enable arching action to occur when the infill is subjected to out-of-plane actions, assuming the absence of side and/or top gaps between the infill and boundary frame. Once the capacity of the infill has been established taking into account horizontal or vertical arching action as appropriate, the stiffness and strength of the boundary frame should be assessed to ensure that the thrusts associated with arching action can be adequately supported and that the assumption of an approximately rigid boundary frame is valid. As no rigorous procedure has yet been developed for this assessment of the boundary frame, engineering judgement is needed based upon the specific details of the building being considered.

C7.6.1.2 Effect of reinforcement

If the infill panel is reinforced, the probable out-of-plane strength can be calculated using conventional reinforced masonry flexural theory in accordance with NZS 4230:2004 using probable material strengths and strength reduction factors equal to 1.0. In this instance beneficial effects of horizontal and vertical arching should be ignored.

C7.6.2 Out-of-plane demands

Seismic out-of-plane demands on masonry infill walls, F_{ph}, can be determined assuming the infill is a secondary structural element in accordance with Section C3.

Note:

When calculating out-of-plane seismic demands, the NZS 1170.5:2004 part spectral shape coefficient, $C_{\text{i}}(T_{\text{p}})$, can be taken conservatively as equal to 2.0. This is because the fundamental period of vibration for most masonry infill walls, T_{p}, will typically be less than 0.75 sec. Similarly, the NZS 1170.5:2004 parts' ductility demand, μ_{p}, can be taken as equal to 1.0 for unreinforced infills and equal to 1.25 for reinforced infills.

C7.7 Assessment of Masonry Infill for In-Plane Actions

Note:

The material in this section has largely been sourced from FEMA 306 (1998) and ASCE 41-13 (2014) with some updates to include recent research developments.

C7.7.1 Modelling of infill panels

C7.7.1.1 General

The calculation of masonry infill in-plane stiffness and strength based on nonlinear finite element analysis of a composite frame substructure with infill panels that account for the presence of openings, post-yield cracking, and cyclic degradation of masonry is permitted. Due to the complexity of the structural system resulting from the frame-infill interaction, finite element models should be validated by considering published or project-specific experimental data from cyclic quasi-static or dynamic tests.

Alternatively, the use of simplified numerical models with diagonal struts to simulate the effect of the infill detailed in Section C7.7.1.2 can be used to model infilled frames.

Note:

Refer to Stavridis (2009) and Al-Chaar et al. (2008) for additional information on the development and calibration of finite element models.

Even when significant gaps exist between infill panels and frame members, interaction can still occur between the elements. When the gap closes, the strength and stiffness of the infill frame will increase markedly. A simplified bounded approach can be used to evaluate the performance of the system. For the case when a gap still exists, the strength and stiffness of the system can be taken to be that of the bare frame. Once the gap has closed the combined strength and stiffness of the infill panel and the frame can be used.

Alternatively, compression-only nonlinear gap elements could be used to model the interaction explicitly.

The in-plane lateral stiffness of an infilled frame system is not the same as the sum of the frame and infill stiffnesses because of the interaction of the infill with the surrounding frame. Experiments have shown that, when subjected to seismic forces, small lateral deformations of the frame result in compressive contact stresses developing between the frame and the infill, with associated separation of the infill at the two diagonally opposed corners (ASCE 41-13, 2014, and refer also to Figure C7.6).

The location and orientation of the diagonal compression strut cannot be clearly defined and different geometries have been proposed:

- with struts forming along the diagonal of the frame located concentrically (refer to Figure C7.9)
- eccentrically (refer to Figure C7.10)

- at an angle of 45 degrees for infill frames with aspect ratios greater than 1.5 (refer to Figure C7.11)
- with a combination of struts to account for openings in penetrated infills (refer to Figure C7.12), or
- with a single eccentric strut for partial height infills (refer to Figure C7.13).

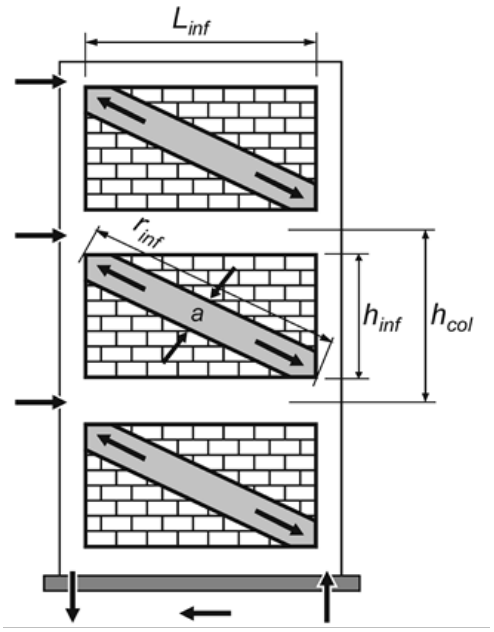


Figure C7.9: Compression strut analogy—concentric struts (ASCE 41-13, 2014)

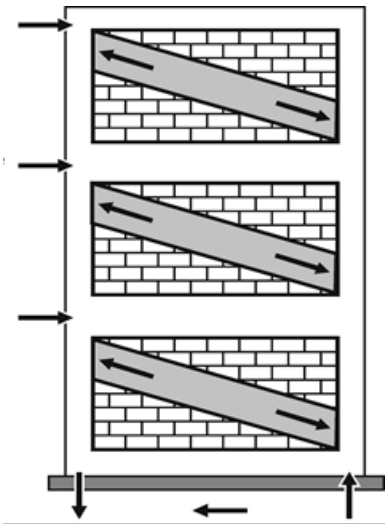


Figure C7.10: Compression strut analogy—eccentric struts (ASCE 41-13, 2014)

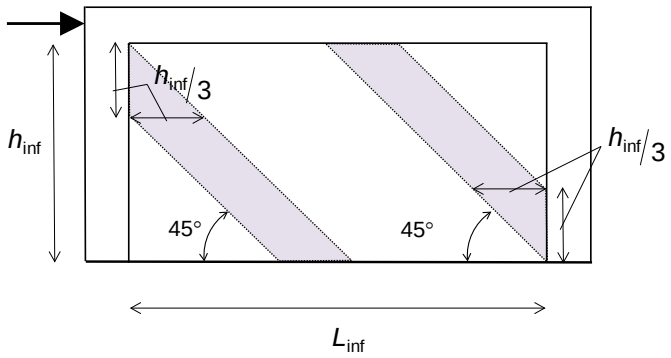


Figure C7.11: Compression strut analogy – struts at 45 degrees acting at the top of the left (windward) column and the bottom of the right (leeward) column for high aspect ratio infill panels (Stavridis, 2009)

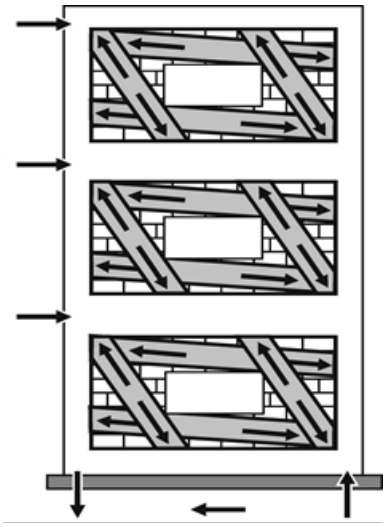


Figure C7.12: Compression strut analogy – penetrated infills (ASCE 41-13, 2014)

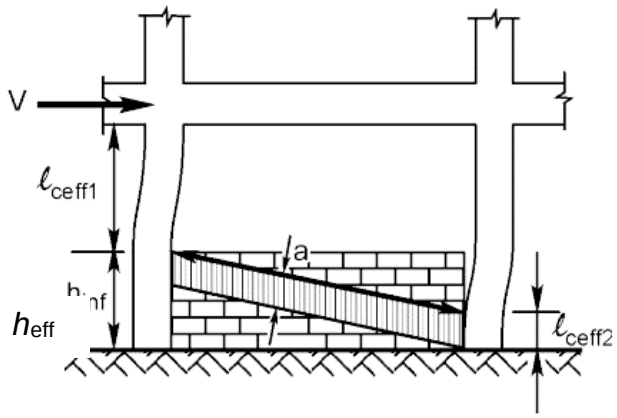


Figure C7.13: Compression strut analogy – partial infills

Because theoretical work and experimental data for determining multiple strut placement and strut properties are not sufficient to establish reliable guidelines for all possible infill configurations, the selection of the strut locations, widths and orientations needs judgement on a case-by-case basis. The engineer should be aware that if analytical models with frame elements are constructed to simulate the behaviour of infilled frames under seismic forces, the results can be significantly affected by the selected strut locations.

C7.7.1.2 Solid infill panels

The probable elastic in-plane stiffness of a solid masonry infill panel prior to cracking can be represented with an equivalent diagonal compression strut of width a, given by Equation C7.8 (Turgay et al., 2014). The equivalent strut should have the same thickness and modulus of elasticity as the infill panel it represents.

$$a = 0.18(\lambda_1 h_{\text{col}})^{-0.25} r_{\text{inf}} \quad \text{…C7.8}$$

where:

$$\lambda_1 = \left[\frac{E_{\text{m}} t sin2\theta}{4E_{\text{f}} I_{\text{bc}} h_{\text{inf}}}\right]^{\frac{1}{4}}$$

and:

h_{col} = column height between centre lines of beam (mm)
h_{inf} = height of infill panel (mm)
E_{f} = probable modulus of elasticity of frame material (MPa)
E_{m} = probable modulus of elasticity of infill material (MPa)
I_{bc} = moment of inertia in the plane of the infill for the bounding column (mm^4)
L_{inf} = length of infill panel (mm)
r_{inf} = diagonal length of infill panel (mm)
t = thickness of infill panel (mm)
θ = angle whose tangent is the infill height-to-length aspect ratio (radians) given by the following:

$$\theta = \tan^{-1}\left[\frac{h_{\text{inf}}}{L_{\text{inf}}}\right] \quad \text{…C7.9}$$

Unless positive anchorage capable of transmitting in-plane forces from the frame members to all masonry wythes is provided on all sides of the walls, only the masonry wythes in full contact with the frame elements should be considered when computing in-plane stiffness.

Stiffness of cracked unreinforced masonry infill panels can be represented with equivalent struts. The strut properties should be determined from analyses that consider the nonlinear behaviour of the infilled frame system after the masonry is cracked.

C7.7.1.3 Penetrated infill panels

Note:

Experiments on perforated infill panels have shown that, under seismic forces, two sets of cracks develop at small lateral deformations and initiate the nonlinear behaviour. The first set is along the frame-infill boundary and the second set consists of cracks that initiate at the corners of openings and radiate into the infill at an angle close to 45 degrees. The stress field is clearly affected by the presence of the openings. However, the exact mechanism is still not clear.

A possible representation of these stress fields with multiple compression struts, as shown in Figure C7.12, has been proposed by Hamburger (1993). However, as theoretical work and experimental data for determining multiple strut placement and strut properties are not sufficient to establish reliable guidelines, assessment methods are as recommended below.

The in-plane stiffness of infills with openings should be assessed using rational strut and tie models using material properties given in other sections of this document, e.g. for concrete (in Section C5) and masonry (in Section C8). An alternative simplified approach for estimating the uncracked stiffness of perforated infill panel, K_{ini}, based on the stiffness of a frame with a solid panel is (ASCE 41-13, 2014):

$$K_{\text{ini}} = \left(1 - 2\frac{A_{\text{op}}}{A_{\text{wtot}}}\right) K_{\text{ini}}^{\text{solid}} \quad \ldots\text{C7.10}$$

where:

A_{op} = area of the opening in the infill panel
A_{wtot} = gross area of an equivalent infill panel with no openings
$K_{\text{ini}}^{\text{solid}}$ = stiffness of an equivalent infill frame with a solid panel.

The in-plane stiffness of an equivalent infill frame with a solid panel, $K_{\text{ini}}^{\text{solid}}$, can be determined using the procedure detailed in Section C7.7.1.2.

C7.7.2 Probable in-plane strength

C7.7.2.1 General

Expected in-plane probable strength of a solid infill panel, V_{prob}, should be the lesser of the probable shear strength, V_{in}, and the probable corner crushing strength, V_{cc}, of the infill panel.

The compressive force in solid infill panels can be estimated assuming the development of one diagonal strut for aspect ratios smaller than 1.5 and two diagonal struts for larger aspect ratios (Stavridis, 2009). In the latter case, the force is distributed between the diagonal struts along 45 degree angles that initiate near the top of the windward column and the bottom of the leeward column (refer to Figure C7.11).

C7.7.2.2 Shear strength

For solid infill panels the probable infill shear strength, V_{in}, should be taken as the lower of the values given by Equations C7.11, C7.12 and C7.13 (Turgay et al., 2014):

$$V_{\text{in}} \leq 0.33\sqrt{f'_{\text{m}}}tL_{\text{inf}} \quad \ldots\text{C7.11}$$

$$\leq 0.83tL_{\text{inf}} \quad \ldots\text{C7.12}$$

$$\leq 0.41tL_{\text{inf}} + 0.45P \quad \ldots\text{C7.13}$$

where:

f'_{m} = probable masonry compressive strength (MPa)
L_{inf} = length of infill panel (mm)
P = axial load on the infill (N)
t = net thickness of infill panel (mm).

The net thickness of the infill panel, *t*, is the minimum thickness of the cross-sectional area of the panel. For a solid or fully grouted infill panel this is the total wall thickness. For unfilled or partially filled panels this is the net thickness of the masonry units.

Axial loads on the infill panel will be those due to gravity and the bounding action of the frame elements. When a gap is present at the top of the infill panel the former will typically be due to the self-weight of the panel and will therefore generally be small.

For the case when no gap is present at the top of the infill panel, the infill may also support tributary floor loads. In addition, when the bounding frame is of reinforced concrete construction creep effects can result in a transfer of gravity loads from the columns to the infill panels.

Axial loads due to the bounding action of the frame can be estimated from the following equation (FEMA 306, 1998):

$$P = \left(\Delta/h_{\text{inf}}\right)^2 \theta_{\text{e}}^2 t L_{\text{inf}} E_{\text{m}} \qquad \text{…C7.14}$$

where:

E_{m} = probable modulus of elasticity of infill material (MPa)
L_{inf} = length of infill panel (mm)
h_{inf} = height of infill panel (mm)
P = axial load on the infill (N)
t_{inf} = thickness of infill panel (mm)
$t_{\text{net,inf}}$ = net thickness of infill panel (mm)
Δ/h_{inf} = inter-storey drift angle (radians).
Δ = lateral deformation (deflection) of top of infill panel relative to base (mm)

Note:

Equations C7.11 to C7.13 are detailed in a paper by Turgay et al. (2014) for determining expected infill shear strength. These are identical to the equations described in section B.3.4.3 of TMS 402-11 (MSJC, 2011) except that the 1.5 denominator has been omitted. Analytical work by Turgay et al. (2014) demonstrated that the alternate expressions provide more reliable estimates of the probable infill shear strength when compared with ASCE 41-06 (2006) and TMS 402-11 (MSJC, 2011).

Review of the research completed by Turgay et al. (2014) suggests that Equation C7.13 will not typically govern the expected shear strength of infill panels. This is consistent with the work completed by Haldar et al. (2013) and Semnani et al. (2014).

C7.7.2.3 Corner crushing strength

The probable corner crushing strength, V_{cc}, of masonry infills should be taken as (Flanagan and Bennett, 2001):

$$V_{\text{cc}} = K t f'_{\text{m}} \qquad \text{…C7.15}$$

where:

f'_{m} = probable masonry compressive strength (MPa)
K = empirical constant (mm)
t = net thickness of infill panel (mm).

Using the recommendation from Flanagan and Bennett (2001) the empirical constant, K, can be taken as 250 mm for steel and concrete frames with solid clay brick, clay tile and concrete masonry infill. This value provides a better estimate of the probable corner crushing strength when compared with TMS 402-11 (MSJC, 2011).

C7.7.2.4 Effect of reinforcement

If the infill panel is reinforced, the probable infill shear strength will be increased. The additional shear capacity associated with the infill panel reinforcement, V_{s}, can be taken as (FEMA 306, 1998):

$$V_{\mathrm{s}} = \rho_{\mathrm{w}} f_{\mathrm{y}} t L_{\mathrm{inf}} \qquad \ldots \text{C7.16}$$

where:

ρ_{w} = volumetric ratio of the infill panel reinforcement
f_{y} = probable yield strength of the infill panel reinforcement (MPa)
L_{inf} = length of infill panel (mm)
t = thickness of infill panel (mm).

Volumetric ratio of the infill panel reinforcement, ρ_{w}, used in Equation C7.16 can be taken as the minimum associated with the horizontal or vertical panel reinforcement.

C7.7.3 Generalised strength-deformation relationships

Figure C7.14 illustrates the generalised strength-deformation relationship for masonry infill panels where V_{E} is the probable shear strength of the infill panel. Probable drift capacities (%) for masonry infill panels are given in Table C7.1 below.

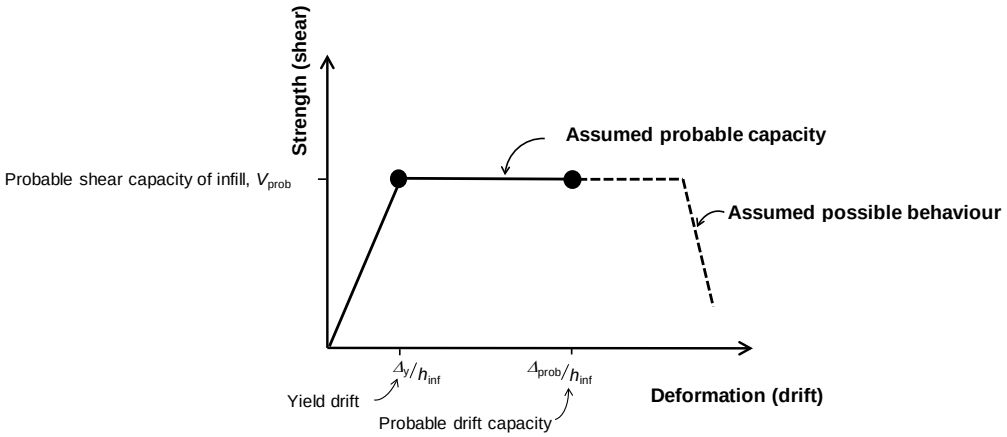


Figure C7.14: Generalised strength-deformation relationship for masonry infill panels

Table C7.1: Probable deformation (drift) capacity of infill panels

$\beta = \frac{V_{fre}}{V_{prob}}$	$\frac{L_{inf}}{h_{inf}}$	Residual strength ratio *C*	Probable drift capacities (%)
$1.0 \leq \beta < 1.3$	0.5	N/A	0.70
	1.0	N/A	0.55
	2.0	N/A	0.40
$\beta \geq 1.3$	0.5	N/A	1.00
	1.0	N/A	0.80
	2.0	N/A	0.60

Note:

1. Interpolation can be used between table values.

When establishing the probable drift capacity of an infill panel, the ratio of the frame to infill strength, β, should be determined considering the expected lateral strength of each element. V_{fre} is the expected storey probable shear strength of the bare frame, taken as the probable shear capacity of the column calculated in accordance with Section C7.8.7. V_{prob} is the probable in-plane infill strength calculated in accordance with Section C7.7.2.

Note:

The generalised deformation values in Table C7.1 have been sourced from ASCE 41-13 (2014). However, the ASCE 41-13 (2014) deformation limits have been divided by 1.5 to ensure that an appropriate margin is provided against collapse of the masonry infill panel.

Probable capacities for structural steel or reinforced concrete frame members that surround the infill panels should be as recommended in Sections C5 and C6 of this document except as modified in Section C7.8 below.

C7.8 Influence of Infill Panels on Frame Members

C7.8.1 General

The flexural and shear strength assessment of any structural steel or reinforced concrete frames that surround infill panels should be based on Sections C5 and C6 of this document, including consideration of related seismic demands on beam-column joints, except as modified below. It is emphasised that the presence of infills modifies and magnifies the shear demands on the frame members by shortening the distance between in-span plastic hinges.

Experience from field observations and experimental work suggests that beams are less susceptible to damage when compared with columns.

As the demands on the bounding members are dependent on the strut size some iteration may be required in the calculations outlined below.

C7.8.2 In-plane shear demands on columns adjacent to solid infill panels

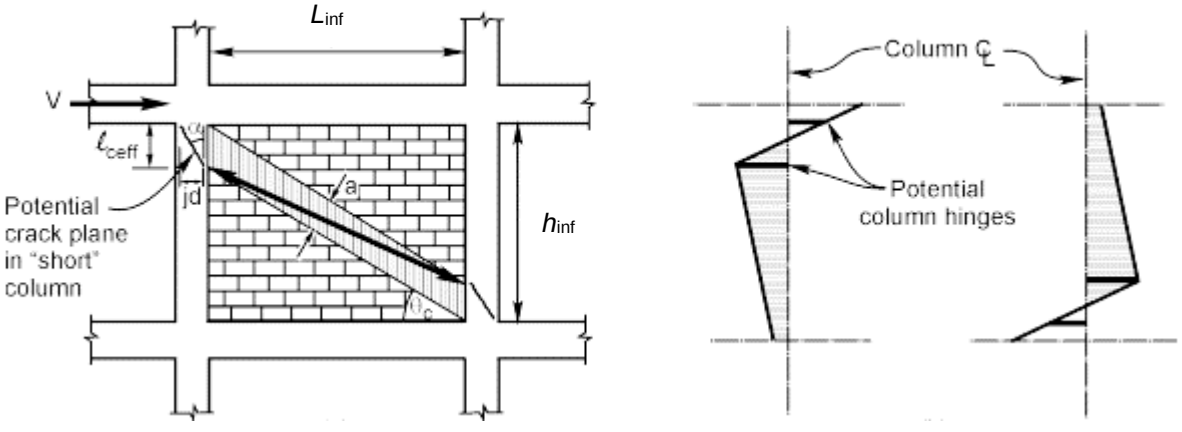
The maximum expected flexural and shear demands on columns adjacent to solid infill panels can be estimated by application of the horizontal component of the expected strut force at a distance of l_{ceff} from the top or bottom of the panel as illustrated in Figure C7.15 below. The effective length, l_{ceff}, may be found from:

$$l_{\text{ceff}} = \frac{a}{\cos\theta_{\text{c}}} \qquad \text{…C7.17}$$

where:

a = the equivalent strut width calculated in Section C7.7.1 above and tan θ_{c} can be found by solving the following equation:

$$\tan\theta_{\text{c}} = \left. h_{\text{inf}} - \left(\frac{a}{\cos\theta_{\text{c}}}\right) \right/ L_{\text{inf}} \qquad \text{…C7.18}$$



(a) Strut placement **(b) Moment demands on the columns**

Figure C7.15: Estimating infill strut demands on columns

An upper bound maximum column shear demand when plastic hinges form in the column can be estimated as:

$$V_{\text{col}} = \frac{2M_{\text{o}}^{\text{col}}}{l_{\text{ceff}}} \quad \text{…C7.19}$$

where:

$M_{\text{o}}^{\text{col}}$ = overstrength moment capacity of the column.

C7.8.3 In-plane shear demands on columns adjacent to partial height infill panels

The maximum expected flexural and shear demands on columns adjacent to partial height infill panels can be estimated by application of the horizontal component of the expected strut force at a distance of l_{ceff1} from the top of the panel and l_{ceff2} from the bottom of the panel as illustrated in Figure C7.13.

The effective length of the "leeward" column, l_{ceff2}, and associated shear demands can be determined using the procedure detailed in the previous section. For the "windward" column (the left column in Figure C7.13) the effective length, l_{ceff1}, can be estimated as:

$$l_{\text{ceff1}} = h_{\text{col}} - h_{\text{inf}} \quad \text{…C7.20}$$

C7.8.4 In-plane shear demands on beams adjacent to solid infill panels

The maximum expected flexural and shear demands on beams adjacent to solid infill panels can be estimated by application of the vertical component of the expected strut force at a distance of l_{beff} from each end of the panel as illustrated in Figure C7.16 below. The effective beam length, l_{beff}, may be found from:

$$l_{\text{beff}} = \frac{a}{\sin\theta_{\text{b}}} \quad \text{…C7.21}$$

where:

a = the equivalent strut width calculated in Section C7.7.1 above and $\tan\theta_{\text{b}}$ can be found by solving the following equation:

$$\tan\theta_{\text{b}} = h_{\text{inf}} \Big/ \left(L_{\text{inf}} - \left(\frac{a}{\sin\theta_{\text{b}}}\right)\right) \quad \text{…C7.22}$$

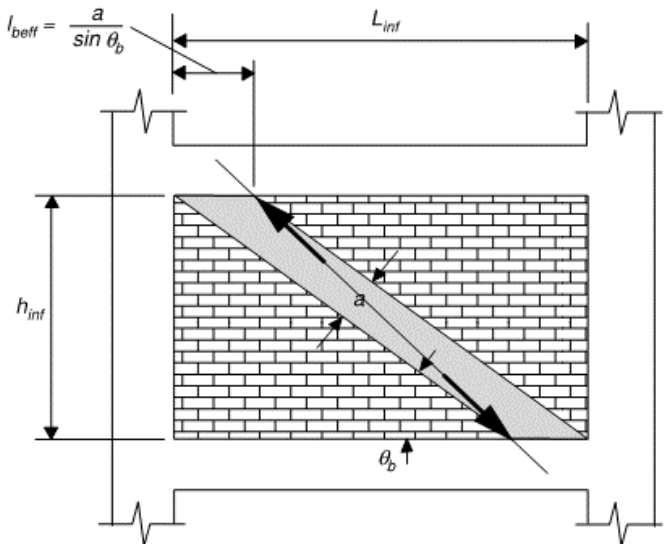


Figure C7.16: Estimating infill strut demands on beams (FEMA 306, 1998)

An upper bound maximum beam shear demand when plastic hinges form in the beam, including effects of tributary slab steel if present, with a reduced length equal to l_{beff} can be estimated as:

$$V_{\text{beam}} = \frac{2M_{\text{o}}^{\text{beam}}}{l_{\text{beff}}} \quad \text{…C7.23}$$

where:

$M_{\text{o}}^{\text{beam}}$ = is the overstrength moment capacity of the beam.

Note:

Experience from field observations and experimental work suggests that when a beam is confined by infill panels above and below, and when no gap is present between the top of the wall and the bottom of the beam, in most typical situations the in-plane shear demands on the beam will be limited and this failure mechanism is unlikely to occur.

C7.8.5 In-plane shear demands on beams and columns adjacent to perforated infill panels

Because theoretical work and experimental data for determining multiple strut placement and strut properties are not sufficient to establish reliable guidelines for all possible infill configurations, the determination of maximum expected flexural and shear demands on beams and columns with perforated infills requires judgement on a case-by-case basis. Procedures detailed in Sections C7.7.1.2 and C7.7.1.3 can be adapted to suit the alternate strut locations discussed in Section C7.7.1.3.

C7.8.6 In-plane tension demands on beam to column connections

The horizontal component of compression force from the infill diagonal compression strut will impose tension forces in the beam to column connection between a beam and an exterior column. This tension force and associated deformation may be sufficient to degrade the vertical load carrying capacity of the beam to column connection.

An assessment of the horizontal component of tension force through the connection needs to be made and compared with the tension capacity of the beam to column connection, to determine if such a failure mode is likely.

C7.8.7 Modified shear capacity

C7.8.7.1 Structural steel frame members

The probable shear capacity of steel frame members can be determined in accordance with Section C6 of this document.

For steel frames with solid webs shear failure of the frame members may not be a critical mode of failure due to the ductility of solid steel webs in shear. The shear capacity of laced and battened encased steel columns is more problematical and dependent on the type and capacity of the ties between the column members.

C7.8.7.2 Reinforced concrete frame members

For shear-critical reinforced concrete frame members, a corner-to-corner crack angle is expected to form between the hinges. For columns and beams the potential crack angle can be calculated using Equations C7.24 and C7.25 respectively:

$$\alpha_{\mathrm{c}} = \tan^{-1}\frac{j_{\mathrm{d}}}{l_{\mathrm{ceff}}}; 20° < \alpha_{\mathrm{c}} < 45° \qquad \text{…C7.24}$$

where:

l_{ceff} = effective column length
j_{d} = internal lever arm within the column member which, in lieu of a more precise analysis, may be taken as 80% of the overall member depth.

Similarly, the potential corner-to-corner crack angle forming in a beam can be estimated as:

$$\alpha_{\mathrm{b}} = \tan^{-1}\frac{d-d'}{l_{\mathrm{beff}}}; 20° < \alpha_{\mathrm{c}} < 45° \qquad \text{…C7.25}$$

where:

l_{beff} = effective beam length
d = effective depth to the centroid of the tension reinforcing steel
d' = effective depth to the centroid of the compression reinforcing steel.

The probable shear capacity of the bare frame members can be determined in accordance with Section C5. The corner-to-corner crack angles calculated above can be used when determining shear contribution, V_{s}, provided by the steel shear reinforcement.

C7.8.8 Bond slip of lap-splice connections in reinforced concrete frame members

Lap-splice connections of column longitudinal reinforcing steel are often present at the base of reinforced concrete columns. Their seismic behaviour can be determined in accordance with Section C5.

Note:

Providing that the lap length is sufficient to develop the expected yield strength of the reinforcing steel, the moment capacity of the section can be attained. However, post elastic deformations quickly degrade the bond-strength capacity, and within one inelastic cycle of loading the lap splice should be assumed to have become ineffective (FEMA 306, 1998).

C7.9 Improving the Seismic Performance of Moment Resisting Frames with Masonry Infill Panels

C7.9.1 Improving out-of-plane performance

Techniques for improving the out-of-plane seismic performance of moment resisting frames with masonry infill panels include:

- providing supplementary vertical mullions
- strengthening the infill wall using reinforced concrete overlays
- strengthening the infill wall using fibre reinforced polymer (FRP) overlays or near surface mounted FRP strips (note that FRP strips will be required on both sides)
- strengthening the infill wall using engineered cementitious composite (ECC) shotcrete overlays
- removing the infill wall.

C7.9.2 Improving in-plane performance

Techniques for improving the in-plane seismic performance of moment resisting frames with masonry infill panels include:

- strengthening the infill wall using reinforced concrete overlays
- strengthening the infill wall using FRP overlays or near surface mounted FRP strips
- strengthening the infill wall using ECC shotcrete overlays
- filling excessively sized infill wall openings with appropriate materials
- providing additional transverse reinforcement to deficient frame members to increase shear capacity, confinement and/or lap-splice continuity
- jacketing deficient frame elements to increase flexural capacity
- providing supplementary lateral load resisting systems to, for example, new reinforced concrete shear walls or structural steel braced frames
- removing the infill wall or isolating the infill wall from the frame elements
- strengthening the tension capacity of beams to external columns in infilled steel frames where required.

When providing supplementary lateral load resisting systems to improve the global performance of the structure (one of the suggestions above) deformation compatibility effects need to be considered. The new supplementary lateral load resisting elements should be detailed to have sufficient strength and stiffness so the seismic demands on the existing infill frame are limited to the required levels.

In many circumstances, this will mean that new structural braced frames or reinforced concrete shear walls are required: in other words, new moment resisting frames are unlikely to be stiff enough.

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PART C

Unreinforced Masonry Buildings C8

Document Status

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1	July 2017	Initial release

This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

This document may be downloaded from www.EQ-Assess.org.nz in parts:

1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

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Please go to www.EQ-Assess.org.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.EQ-Assess.org.nz

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C8. Unreinforced Masonry Buildings

C8.1 General

C8.1.1 Background

This section draws on key observations from the 2010/11 Canterbury earthquake sequence and on the significant quantity of research conducted in recent years at the University of Auckland, University of Canterbury and further afield. New sections include revised information on materials characterisation, a new method for diaphragm assessment, a new approach to the treatment of in-plane pier capacity based on failure modes, and the introduction of spandrel models.

This section was first released in 2015 as a revision to Section 10 of the unreinforced masonry (URM) section in the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" ("the 2006 guidelines", NZSEE, 2006). Only minor editorial changes including the addressing of errata items have been made to that version.

URM construction can be vulnerable to earthquake shaking because of its high mass, lack of integrity between elements and lack of deformation capability. The most hazardous features of URM buildings are inadequately restrained elements at height (such as façades, chimneys, parapets and gable-end walls), face-loaded walls, and their connections to diaphragms and return walls. These can present a significant risk to occupants as well as people within a relatively wide zone from the building.

Assessing the performance of these buildings can be complex as potential failure mechanisms are different from those occurring in other building types. Performance tends to be limited to out-of-plane wall behaviour, relative movement of different elements attached to flexible diaphragms, and tying of parts. This conflicts with the more typical idealisation of a building acting as one unified mass, but is essential to understand in order to assess these structures reliably.

The seismic capacity of URM bearing wall buildings is also difficult to quantify and may result in margins against collapse that are small for the following reasons:

- URM walls and piers may have limited nonlinear deformation capability depending on their configuration, material characteristics, vertical stresses and potential failure modes.
- They rely on friction and overburden from supported loads and wall weights.
- They often have highly variable material properties.
- Their strength and stiffness degrade with each additional cycle of greater displacement of inelastic response to shaking. Therefore, they are vulnerable to incremental damage, especially in larger-magnitude, longer-duration earthquakes with multiple aftershocks.

Unlike other construction materials covered by these guidelines URM has not been permitted to contribute to the building lateral load resisting system in new buildings since 1964. Therefore, there is no standard for new URM buildings which could be used to compare to the standard achieved for an existing building. New building standard (*NBS*) and *%NBS* as

it relates to URM buildings is therefore assumed to be defined by the requirements set out in this section.

If buildings have undergone damage in an earthquake, much of the cyclic capacity may have already been used by the main event. Assessment of these buildings after an earthquake should consider this damaged state. As a result, their seismic capacity could be significantly lower than in their undamaged or repaired state. This is the important rationale for interim shoring for URM buildings (refer to Figure C8.1) to mitigate further damage as an important part of building conservation. These techniques typically provide tying (rather than strengthening) to prevent further dilation of rocking or sliding planes, and to relieve stresses at areas of high concentration.

Note:

These guidelines recommend considering selective strengthening of URM buildings as a first step before proceeding to a detailed assessment, particularly in high seismicity areas. Improvement of diaphragm to wall connections, for example, will almost certainly be required to provide the building with any meaningful capacity as the as-built details will provide almost no support.

Using sound engineering judgement when assessing URM buildings is also important or the engineer may end up with an economically non-viable solution, with the result that demolition may appear to be the only option.

Figure C8.1: Temporary securing of a mildly damaged solid masonry URM building (Dunning Thornton/Heartwood Community)

C8.1.2 Scope

This section sets out guidelines for assessing:

- **unreinforced solid clay brick masonry buildings;** constructed of rectangular units in mortar, laid in single or multi-wythe walls, and in forms of bond such as common bond, English bond, running bond and Flemish bond.

These guidelines are valid for:

- **walls in good condition**; with negligible mortar joint cracking, brick splitting, settlement or similar factors
- **walls under face load attached to rigid or flexible diaphragms**
- **brick veneers under face loading**
- **stone masonry where the stones are layered.**

They can also be applied, with some additional requirements, to:

- unreinforced stone masonry that is well coursed and laid in running bond
- hollow or solid block masonry
- hollow clay brick and concrete block masonry (refer to Section C7 for assessment of brick or block infill masonry walls in framed construction)
- rubble stone masonry: the failure modes of these structures may be other than those covered here, including the possibility of delamination
- cobble stone masonry: assessment of face-loaded capacity is not covered by these guidelines.

Not in scope

This section does not cover:

- earthquake-damaged masonry buildings
- reinforced partially filled and fully filled block masonry.

Note:

Although the strengthening of URM buildings is outside the scope of this section, brief comments on this topic have been included in Section C8.12.

C8.1.3 Basis of this section

This section is largely based on experimental and analytic studies undertaken at the University of Auckland, University of Canterbury and in Australia, and on the research undertaken by Magenes and Calvi (1997) and Blaikie (1999, 2002). It also draws on ASCE 41-13 (2014).

Most of the default stress values have been adopted from tests undertaken at the University of Auckland (Lumantarna et al., 2014a; Lumantarna et al., 2014b) and from other sources including FEMA 306 (1998), ASCE 41-13 (2014), Kitching (1999) and Foss (2001).

Procedures for assessing face-loaded walls spanning vertically in one direction are based on displacement response that includes strongly nonlinear effects. These procedures have been verified by research (Blaikie, 2001, 2002) using numerical integration time history analyses

and by laboratory testing that included testing on shake tables. This research extended the preliminary conclusions reached in Blaikie and Spurr (1993). Other research has been conducted elsewhere, some of which is listed in studies including Yokel and Dikkers (1971), Fattal (1976), Hendry (1973, 1981) Haseltine (1977), West (1977), Sinha (1978), ABK Consultants (1981), Kariotis (1986), Drysdale (1988), Lam (1995) and Mendola (1995). More recent research has been conducted by Derakhshan et al (2013a, 2013b, 2014a, 2014b).

Other useful information on materials, inspection and assessments is contained in FEMA 306 (1998) and ASCE 41-13 (2014).

C8.1.4 How to use this section

This section is set out as follows.

Understanding URM buildings (Sections C8.2 to C8.4)

These sections provide important context on the characteristics of URM buildings, typical building practices in New Zealand, and observed behaviour in earthquakes. As URM is a non-engineered construction, and given the recent learnings about its seismic performance, the engineer should review this information carefully before proceeding to the assessment.

Assessing URM buildings (Sections C8.5 to C8.11)

These sections explain how to approach the assessment depending on what is being asked and the type of building that is being assessed. Given the nature of URM construction and the number of previous strengthening techniques used on these buildings, on-site investigation is particularly important. These sections provide a checklist of what to look for on-site as well as probable material properties, before setting out the detailed assessment methods.

Improving URM buildings (Section C8.12)

Although formally outside scope, this section includes some brief comments on improving seismic performance of existing URM buildings. This is an introduction only to a broad field of techniques which is under continual development and research.

C8.1.5 Definitions and acronyms

Action	Set of concentrated or distributed forces acting on a structure (direct action), or deformation imposed on a structure or constrained within it (indirect action). The term 'load' is also often used to describe direct actions.
Adhesion	Bond between masonry unit and mortar
Basic building	Building of up to two storeys in height with flexible diaphragms where there is little expected interaction between parallel lines of seismic resistance
BCA	Building Consent Authority
Beam	A member subjected primarily to loads producing flexure and shear
Bearing wall	A wall that carries (vertical) gravity loads due to floor and roof weight
Bed joint	The horizontal layer of mortar on which a brick or stone is laid
Bond	The pattern in which masonry units are laid
Brittle	A brittle material or structure is one that fails or breaks suddenly once its probable strength capacity has been reached. A brittle structure has very little tendency to deform before it fails, and it very quickly loses lateral load carrying capacity once failure is initiated.
Cavity wall	A cavity wall consists of two 'skins' separated by a hollow space (cavity). The skins are commonly both masonry, such as brick or concrete block, or one could be concrete. The cavity is constructed to provide ventilation and moisture control in the wall.
Cohesion	Bond between mortar and brick
Collar joint	A vertical longitudinal space between wythes of masonry or between an outer masonry wythe and another backup system. This space is often specified to be filled solid with mortar or grout, but sometimes collar-joint treatment is left unspecified.
Course	A course refers to a row of masonry units stacked on top of one another
Critical structural weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be potential CSWs.
Cross wall	An interior wall that extends from the floor to the underside of the floor above or to the ceiling, securely fastened to each and capable of resisting lateral forces
Dead load	The weight of the building materials that make up a building, including its structure, enclosure and architectural finishes. The dead load is supported by the structure (walls, floors and roof).
Design strength	The nominal strength multiplied by the appropriate strength reduction factor
Diaphragm	A horizontal structural element (usually suspended floor or ceiling or a braced roof structure) that is connected to the vertical elements around it and distributes earthquake lateral forces to vertical elements, such as walls, of the lateral force-resisting system. Diaphragms can be classified as flexible or rigid.
Dimension	When used alone to describe masonry units, means nominal dimension
Ductile/ductility	Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
Earthquake-Prone Building (EQP)	A legally defined category which describes a building that has been assessed as likely to have its ultimate limit state capacity exceeded in moderate earthquake shaking (which is defined in the regulations as being one third of the size of the shaking that a new building would be designed for on that site). A building having seismic capacity less than 34%*NBS*.

Earthquake Risk Building (ERB)	A building that falls below the threshold for acceptable seismic risk, as recommended by NZSEE (i.e. <67*%NBS* or two thirds new building standard)
Face-loaded walls	Walls subjected to out-of-plane inertial forces. Also see Out-of-plane load.
Flexible diaphragm	A diaphragm which for practical purposes is considered so flexible that it is unable to transfer the earthquake loads to shear walls even if the floors/roof are well connected to the walls. Floors and roofs constructed of timber, and/or steel bracing in a URM building fall in this category.
Gravity load	The load applied in a vertical direction, including the weight of building materials (dead load), environmental loads such as snow, and building contents (live load)
Gross area	The total cross-sectional area of a section through a member bounded by its external perimeter faces without reduction for the area of cells and re-entrant spaces
In-plane load	Load acting along the wall length
In-plane wall	Wall loaded along its length. Also referred as in-plane loaded wall.
Irregular building	A building that has an irregularity that could potentially affect the way in which it responds to earthquake shaking. A building that has a sudden change in its plan shape is considered to have a horizontal irregularity. A building that changes shape up its height (such as one with setbacks or overhangs) or that is missing significant load-bearing elements is considered to have a vertical irregularity. Structural irregularity is as defined in NZS 1170.5:2004.
Lateral load	Load acting in the horizontal direction, which can be due to wind or earthquake effects
Leaf	See Wythe
Load	See Action
Load path	A path through which vertical or seismic forces travel from the point of their origin to the foundation and, ultimately, to the supporting soil
Low-strength masonry	Masonry laid in weak mortar; such as weak cement/sand or lime/sand mortar
Masonry	Any construction in units of clay, stone or concrete laid to a bond and joined together with mortar
Masonry unit	A preformed unit intended for use in masonry construction, e.g. brick, concrete block
Mortar	The cement/lime/sand mix in which masonry units are bedded
Mullion	A vertical member, of stone, metal or wood, between the lights of a window, the panels in wainscoting, or the like
Net area	The gross cross-sectional area of the wall less the area of un-grouted areas or penetrations
Out-of-plane load	Load acting at right angles to the wall surface. Walls subjected to out-of-plane shaking are referred to as face-loaded walls.
Partition	A non-loadbearing wall which is separated from the primary lateral structure
Party wall	A party wall (occasionally party-wall or parting wall) is a dividing wall between two adjoining structures providing support for either or both
Pier	A portion of wall between doors, windows or similar structures
Pointing (masonry)	Troweling mortar into a masonry joint after the masonry units have been laid. Higher quality mortar is used than for the brickwork.
Primary element	An element which is part of the primary lateral structure

Probable strength	The expected or estimated mean strength of a member/element, calculated using the section dimensions as detailed and the probable material strengths as defined in these guidelines
Regular building	A building that is not an irregular building
Required strength	The strength of a member/element required to resist combinations of actions for ultimate limit states as specified in AS/NZS 1170.0:2002
Return wall	A short wall usually perpendicular to, and connected to a wall orientated in the direction of loading to increase its structural stability
Rigid diaphragm	A suspended floor, roof or ceiling structure that is able to provide effective transfer of lateral loads to walls. Floors or roofs made from reinforced concrete, such as reinforced concrete slabs, fall into this category.
Running or stretcher bond	The unit set out when the units of each course overlap the units in the preceding course by between 25% and 75% of the length of the units
Seismic hazard	The potential for damage caused by earthquakes. The level of hazard depends on the magnitude of probable earthquakes, the type of fault, the distance from faults associated with those earthquakes, and the type of soil at the site.
Seismic system	That portion of the structure which is considered to provide the earthquake resistance to the entire structure
Shear wall	A wall which resists lateral loads along its primary axis (also known as an in-plane wall)
SLaMA	Simple Lateral Mechanism Analysis (refer to Section C2)
Special study	A procedure for justifying a departure from these guidelines or determining information not covered by them. Special studies are outside the scope of these guidelines.
Stack bond	The unit set out when the units of each course do not overlap the units of the preceding course by the amount specified for running or stretcher bond
Structural element	Combinations of structural members that can be considered to work together; e.g. the piers and spandrels in a penetrated wall, or beams and columns in a moment resisting frame
Through stone	A long stone (header unit) that connects two wythes together in a stone masonry wall. It is also known as bond stone. Contrary to its name, a through stone can also be a concrete block, a wood element, or steel bars with hooked ends embedded in concrete that perform the same function.
Transom	A transverse horizontal structural element of wood, steel, stone or concrete
Transverse wall	See Cross wall
Unreinforced masonry (URM) wall	A wall comprising masonry units connected together with mortar and containing no steel, timber, cane or other reinforcement.
Veneer	See Wythe
Wall	A vertical element which because of its position and shape contributes to the rigidity and strength of a structure
Wall tie	The tie in a cavity wall, used to tie the internal and external walls (or wythes), constructed of wires, steel bars or straps
Wythe	A continuous vertical section of masonry one unit in thickness. A wythe may be independent of, or interlocked with, the adjoining wythe(s). A single wythe is also referred to as a veneer or leaf.

C8.1.6 Notation, symbols and abbreviations

Symbol	Meaning
A	Angular deflection (rotation) of the top and bottom parts of a wall panel relative to a line through the top and bottom restraints, radian. The angle is in radians. It is measured as if there were no inter-storey deflection.
A_{gross}	Gross plan area of diaphragm
$\ddot{A}_{\text{max}}$	Max acceleration
A_{n}	Area of net mortared/grouted section of the wall web, mm^2
A_{n}	Net plan area of masonry wall, mm^2
A_{net}	Net plan area of diaphragm excluding any penetration, m^2
a	Parameter given by equation
B	Depth of diaphragm, m
b	Parameter given by equation
c	Masonry bed-joint probable cohesion, N/mm^2. The ability of the mortar to work in conjunction with the bricks. This is related to moisture absorption in the bricks. It depends less on the absorption qualities of individual brick types and is not greatly influenced by keying of the brick surface (e.g. holes, lattices or patterning). Cohesion is relevant to the primary decision of whether to use cracked or un-cracked masonry properties for the analyses.
$C(0)$	Elastic site hazard spectrum for horizontal loading at fundamental period of 0 sec
$C(T_1)$	Elastic site hazard spectrum for horizontal loading
$C(T_{\text{d}})$	Seismic coefficient at required height at period T_{d}
$C_{\text{h}}(0)$	Spectral shape factor for relevant soil determined from Clause 3.1.1, NZS 1170.5:2004, g
$C_{\text{h}}(T_1)$	Spectral shape factor for relevant site subsoil type and period T_1 as determined from Section 3, NZS 1170.5:2004, g
$C_{\text{hc}}(T_{\text{p}})$	Spectral shape factor for site subsoil type *C* and period T_{p} as determined from Section 3, NZS 1170.5:2004, g
C_{Hi}	Floor height coefficient for level i as defined in NZS 1170.5:2004
$C_{\text{i}}(T_{\text{p}})$	Part spectral shape factor
C_{m}	Value of the seismic coefficient, applied uniformly to the entire panel, that would cause a mechanism to just form, g
$C_{\text{p}}(0.75)$	Seismic coefficient for parts at 0.75 sec. Value of the seismic coefficient that would cause a mechanism to just form, g
$C_{\text{p}}(T_{\text{p}})$	Design response coefficient for parts as defined by Section 8, NZS 1170.5:2004, g
D	Dimensional (e.g. two dimensional or three dimensional)
D_{ph}	Displacement response (demand) for a wall panel subject to an earthquake shaking as specified by Equation C8.18, mm
e	Eccentricity
E_{m}	Young's modulus of masonry, MPa, kN/m^2

Symbol	Meaning
e_b	Eccentricity of the pivot at the bottom of the panel measured from the centroid of W_b, mm
e_o	Eccentricity of the mid height pivot measured from the centroid of W_b, mm
e_p	Eccentricity of *P* measured from the centroid of W_t, mm
e_t	Eccentricity of the mid height pivot measured from the centroid of W_t, mm
F	Applied load on timber lintel
F_i	Equivalent static horizontal force at the level of the diaphragm (level i)
f'_b	Probable compressive strength of bricks measured on the flat side, MPa
f'_j	Normalised mortar compressive strength, MPa
f'_j	Probable mortar compressive strength, MPa
f'_{ji}	Measured irregular mortar compressive strength, MPa
f'_m	Probable masonry compressive strength, MPa
f'_r	Modulus of rupture of bricks, MPa
f'_t	Probable tensile strength of masonry, MPa
$f_{t,eff}$,	Equivalent tensile strength of masonry spandrel, MPa
f_a	Axial compression stress on masonry due to gravity load, MPa
f_{bt}	Probable brick tensile strength, MPa. May be taken as 85% of the stress derived from splitting tests or as 50% of the stress derived from bending tests.
f_{dt}	Probable diagonal tensile strength of masonry, MPa
f_{hm}	Probable compression strength of the masonry in the horizontal direction ($0.5f'_m$), MPa
g	Acceleration due to gravity, m/sec^2
G'_d	Reduced diaphragm shear stiffness, kN/m
$G'_{d,eff}$	Effective diaphragm shear stiffness, kN/m
G_d	Shear stiffness of straight sheathed diaphragm, kN/m
G_m	Shear modulus of masonry, MPa
h	Free height of a cantilever wall from its point of restraint or height of wall in between restraints in case of a simply-supported face-loaded wall. The clear height can be taken at the centre-to-centre height between lines of horizontal restraint. In the case of concrete floors, the clear distance between floors will apply.
h_{eff}	Height of wall or pier between resultant forces
h_i	Average of the heights of point of support
h_i	Height of attachment of the part
h_i	Height of level i above the base of the building
H_l	Height of wall below diaphragm, m

Symbol	Meaning
h_{n}	Height from the base to the uppermost seismic weight or mass of the primary structure
h_{sp}	Height of spandrel excluding depth of timber lintel if present
h_{tot}	Total height of spandrel
H_{u}	Height of wall above diaphragm, m
I_{g}	Moment of inertia for the gross section representing uncracked behaviour
I_{xx}	Mass moment of inertia about x-x axis, kgm^2
I_{yy}	Mass moment of inertia about y-y axis, kgm^2
J	Rotational inertia of the wall panel and attached masses, kgm^2
J_{anc}	Rotational inertia of ancillary masses, kgm^2
J_{bo}	Rotational inertia of the bottom part of the panel about its centroid, kgm^2
J_{bo}	Polar moment of inertia about centroid, kgm^2
J_{to}	Rotational inertia of the top part of the panel about its centroid, kgm^2
k	In-plane stiffness of walls and piers, N/mm
K_{R}	Seismic force reduction factor for in-plane seismic force
L	Span of diaphragm, m
l	Length of header
l_{sp}	Clear length of spandrel between adjacent wall piers
L_{w}	Length of wall
M	Moment capacity of the panel
$M_1,\ M_{\mathrm{i}},\ M_{\mathrm{n}}$	Moment imposed on wall/pier elements
m	Mass, kg
m_{i}	Seismic mass at the level of the diaphragm (level i)
$N(T_1, D)$	Near fault factor determined from Clause 3.1.6, NZS 1170.5:2004
n	Number of recesses
$N_1, N_{\mathrm{i}}, N_{\mathrm{n}}$	Axial loads on pier elements
P	Superimposed and dead load at top of wall/pier
P	Load applied to the top of panel acting through the pivot at the top of the wall
p	Depth of mortar recess, mm
$P-\Delta$	P-delta
p_{p}	Mean axial stress due to superimposed and dead load in the adjacent wall piers
p_{sp}	Axial stress in the spandrel
P_{w}	Self-weight of wall and pier
Q	Live load
$Q_1, Q_{\mathrm{i}}, Q_{\mathrm{n}}$	Shear in pier element

Symbol	Meaning
R	Return period factor, R_{u} determined from Clause 3.1.5, NZS 1170.5:2004
r_{a}	Rise of arch (refer to Figure C8.70)
r_{i}	Radius of intrados (lower side) of arch (refer to Figure C8.70)
r_{o}	Radius of extrados (upper side) of arch (refer to Figure C8.70)
R_{P}	Risk factor for parts as defined in NZS 1170.5:2004
R_{u}	Return period factor for ultimate limit state as defined in NZS 1170.5:2004
S_{i}	Sway potential index
S_{p}	Structural performance factor in accordance with NZS 1170.5:2004
t	Depth of header
t	Effective thickness, which may vary with position, mm
T_1	Fundamental period of the building, sec
T_{d}	Fundamental period of diaphragm, sec
t_{gross}	Overall thickness of wall, which may vary with position, mm
t_{l}	Effective thickness of walls below the diaphragm, m
t_{nom}	Nominal thickness of wall excluding pointing, which may vary with position, mm
T_{p}	Effective period of parts, sec
t_{u}	Effective thickness of walls above the diaphragm, m
V	Probable shear strength capacity
V_{b}	Horizontal base shear
V_{dpc}	Probable capacity of a slip plane for no slip
V_{dt}	Probable in-plane diagonal tensile strength capacity of pier and wall
V_{fl}	Shear induced in spandrel due to peak flexural strength of spandrel
$V_{\mathrm{fl,r}}$	Shear induced in spandrel due to residual flexural strength of spandrel
$(V_{\mathrm{prob}})_{\mathrm{global,base}}$	Probable base shear capacity of building
$(V_{\mathrm{prob}})_{\mathrm{line,i}}$	Probable shear capacity of wall along line i
$(V_{\mathrm{prob}})_{\mathrm{wall1}}$	Probable shear capacity of wall 1
V_{r}	Probable in-plane rocking strength capacity of pier and wall
V_{s}	Probable in-plane bed-joint shear strength capacity of pier and wall
V_{s1}	Probable peak shear strength of spandrel
V_{s2}	Probable peak shear strength of spandrel
$V_{\mathrm{s,r}}$	Probable residual spandrel shear strength capacity or probable residual wall sliding shear strength capacity
V_{tc}	Probable in-plane toe crushing strength capacity of pier and wall
$V_{\mathrm{tc,r}}$	Residual in-plane toe crushing strength capacity of pier and wall
W	Weight of the wall and pier

Symbol	Meaning
W_b	Weight of the bottom part of the panel
W_i	Seismic weight at level i
W_t	Weight of the top part of the panel
W_{trib}	Uniformly distributed tributary weight
y_b	Height of the centroid of W_b from the pivot at the bottom of the panel
y_t	Height from the centroid of W_t to the pivot at the top of the panel
Z	Hazard factor as defined in NZS 1170.5:2004
α_a	Arch half angle of embrace
α_{ht}	t/l ratio correction factor
α_{tl}	t/l ratio correction factor
α_w	Diaphragm stiffness modification factor taking into account boundary walls
β	Factor to correct nonlinear stress distribution
β_i	The ratio of the applied shear at level i to the shear at the base of the line under consideration
β_s	Spandrel aspect ratio
β_{sp}	Width of spandrel
γ	Participation factor for rocking system relating the deflection at the mid height hinge to that obtained from the spectrum for a simple oscillator of the same effective period and damping
Δ	Horizontal displacement, mm
Δ_d	Horizontal displacement of diaphragm
Δ_i	Deflection that would cause instability of a face-loaded wall under forces W_b, W_t and P only
Δ_m	An assumed maximum useful deflection = $0.6\Delta_i$ and $0.3\Delta_i$ for simply-supported and cantilever walls respectively used for calculating deflection response capacity
Δ_t	An assumed maximum useful deflection = $0.6\Delta_m$ and $0.8\Delta_m$ for simply-supported and cantilever walls respectively used for calculating fundamental period of face-loaded rocking wall
$\Delta_{tc,r}$	Deformation at the onset of toe crushing
Δ_y	Yield displacement
θ	Chord rotation of spandrel measured parallel to the displaced wall, relative to pier, radian
θ_y	Yield rotation of the spandrel
μ	Structural ductility factor in accordance with NZS 1170.5:2004
μ_{dpc}	DPC coefficient of friction
μ_f	Probable coefficient of friction of masonry
μ_p	Ductility of part (wall)
ρ	Density (mass per unit volume)
ξ_{sys}	Equivalent viscous damping of the system

Symbol	Meaning
$\Sigma V^*_{\text{u,Pier}}$	Sum of the 100%*NBS* shear force demands on the piers above and below the joint calculated using K_{R} = 1.0
$\Sigma V_{\text{n,Pier}}$	Sum of the piers' capacities above and below the joint
$\Sigma V^*_{\text{u,Spandrel}}$	Sum of the 100%*NBS* shear force demands on the spandrels to the left and right of the joint calculated using K_{R} = 1.0
$\Sigma V_{\text{n,Spandrel}}$	Sum of the spandrel capacities to the left and right of the joint
ϕ	Strength or capacity reduction factor
Ψ	The maximum inter-storey slope which may need to be measured between diaphragms which includes both inter-storey and diaphragm deflection, radian

C8.2 Typical URM Building Practices in New Zealand

C8.2.1 General

Most of New Zealand's URM buildings were built during a relatively narrow window of time; between the late 1870s and 1940s (Russell and Ingham, 2010a). As a result, construction methods are relatively uniform with only a few variations reflecting the origins of the stonemasons and the customary stones ("hard rock" or "soft rock") they used for laying. However, these buildings vary substantially in their structural configuration and layout.

C8.2.2 Building forms

The range of typical URM buildings is set out in Table C8.1 together with some common characteristics for each type. Note that:

- most of the smaller buildings are cellular in nature, combining internal masonry or timber walls with the perimeter masonry façade to provide an overall rigid unit
- many smaller commercial URM buildings have fairly open street façades at ground level and high bottom storeys
- larger buildings tend to have punched wall frames (refer to Figure C8.2) and open plan areas where floors and roofs are supported by timber, cast iron or steel posts
- large, complex buildings such as churches are particularly vulnerable to earthquake shaking as they tend to have irregular plans, tall storey heights, offset roofs, few partitions and many windows.

In these guidelines smaller buildings (i.e. less than or equal to two storeys in height), including small churches and halls, are categorised as *basic buildings* to distinguish them from more complex buildings. Basic buildings are only those with flexible diaphragms where there is little interaction between parallel lines of seismic resistance. Simplified approaches, particularly associated with determining material property and analysis, and ignoring the effects of torsion or transfer of load in plan between floors, are possible when assessing buildings with these characteristics. These are covered in the appropriate sections below.

The interaction of buildings constructed with common boundary or party walls is discussed in more detail in Section C8.5.4.

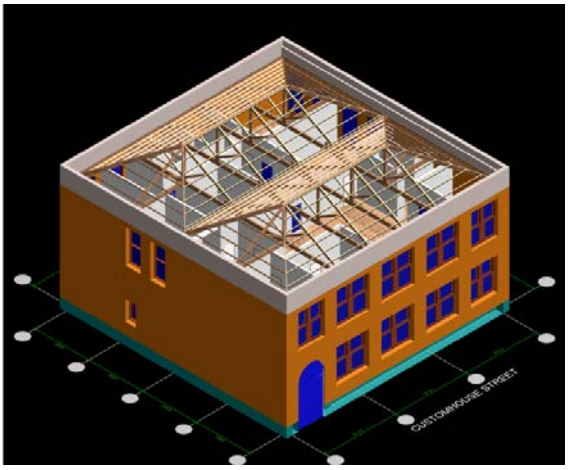
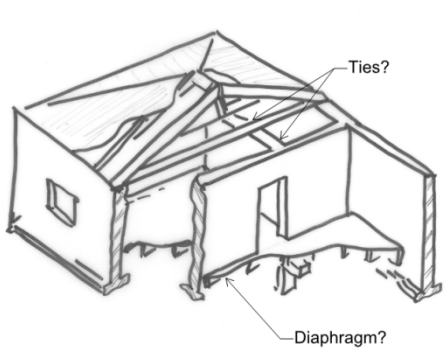
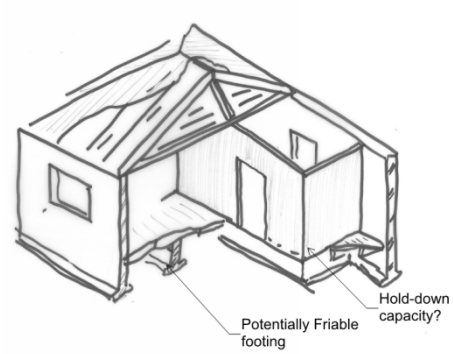
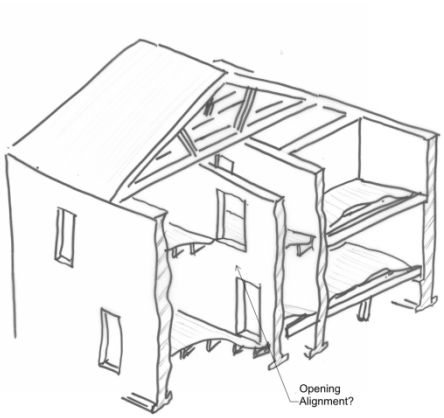
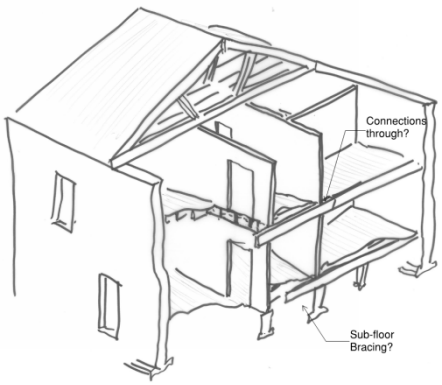
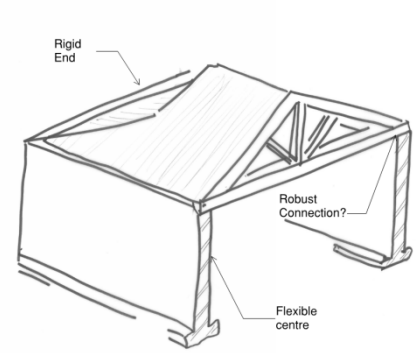
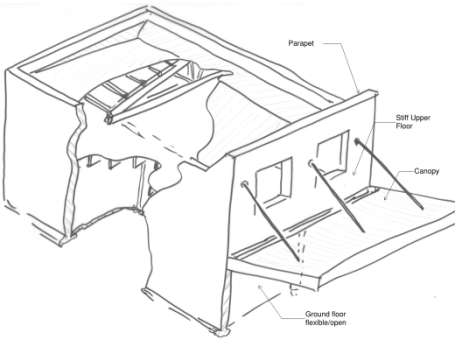
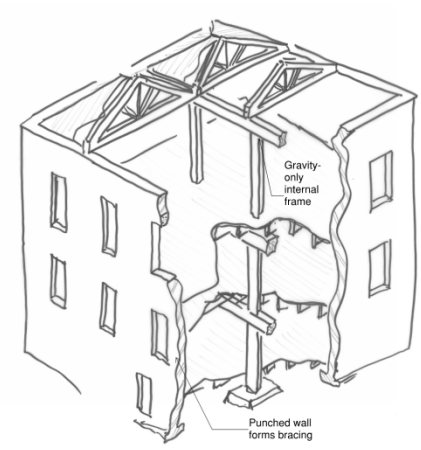
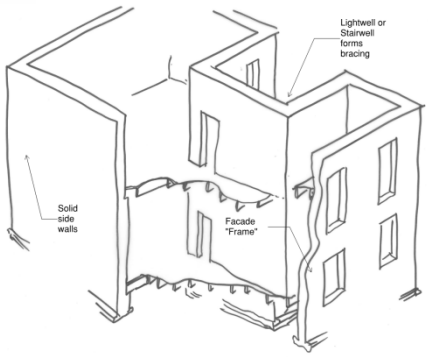
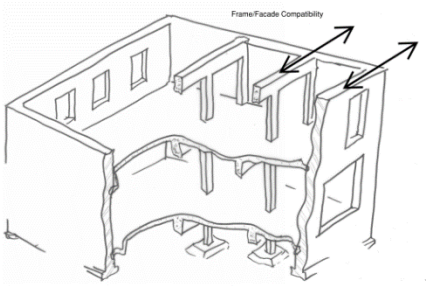
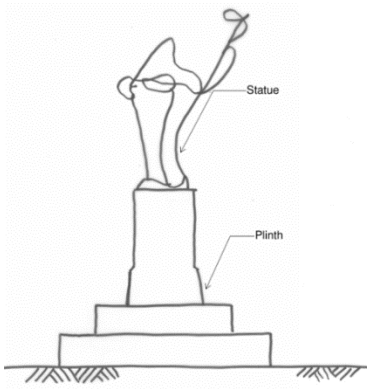
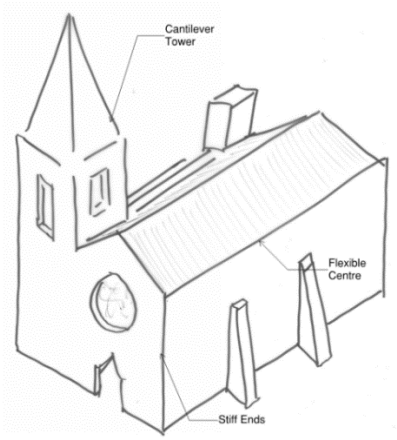


Figure C8.2: URM building with punched wall

Table C8.1: Building forms

Form	Illustration	Particular issues
1 storey cellular: **Masonry internal walls** *Bracing predominantly from in-plane walls cantilevering from ground level*		• Bonding at wall intersections • Plan regularity – diaphragm demand if irregular • Relative stiffness/strength from varying wall lengths • Subfloor height and level of fixity • Ground floor diaphragm/bracing
1 storey cellular: **Timber internal walls** *Bracing predominantly from walls loaded in-plane cantilevering from ground level*		• Connection to masonry at intersections • Stiffness compatibility with masonry – wall geometry • Stiffness compatibility with masonry – materiality (plaster/lath, fibrous plaster) • Flexibility of strapping/lining with respect to masonry • Timber wall foundation bracing capacity
>1 storey cellular: **Masonry internal walls** *Bracing predominantly from walls loaded in-plane with interaction over doorways and between floors*		As for 1 Storey plus: • Wall coupling over doorways • Change in wall thickness at first floor
>1 storey cellular: **Timber internal walls** *Bracing predominantly from walls loaded in-plane with interaction over doorways and between floors*		As for 1 Storey plus: • Hold-down of upper walls to lower walls • Hold-down and bracing of lower walls to piles

Form	Illustration	Particular issues
1 storey open: *Bracing predominantly from walls loaded out-of-plane cantilevering from ground level*		• End walls and differential stiffness • Ground conditions and foundations critical • Wall connection with ground floor slab if present
>1 storey open: *Bracing predominantly from walls loaded out-of-plane cantilevering from ground level, with contributions from end walls* Most common town centre commercial structures		• Diaphragm stiffness • Diaphragm strength • Ancillary structures forming bracing • Contribution of shop front beams/frame • Plan regularity
Multi-storey open *Bracing predominantly from perimeter walls loaded in-plane*		• Wall-to-diaphragm connection demands high for out-of-plane wall loads • Diaphragm stiffness important for out-of-plane wall analysis • Diaphragm strength demands often high • Holes in diaphragms • Punched walls in-plane analysis can be complex
Multi-storey with internal structures *Bracing from combination of internal walls and perimeter walls loaded in-plane*		• Wall-to-diaphragm connection demands high • Compatibility between flexible internal and stiff external structures • Punched walls in-plane analysis can be complex

Form	Illustration	Particular issues
Multi-storey frame/wall *Bracing from combination of internal walls and perimeter walls loaded in-plane*		• Often heavyweight floors: stiff but strength difficult to ascertain • Internal frame stiffness vs perimeter punched wall stiffness • High shear demands on in-plane connection to perimeter elements
Monumental – single form *Bracing predominantly from cantilever action, single degree of freedom* Statues, towers, chimneys and the like		• Often rocking governed – can be beneficial • Foundation stability critical • Combination of materials forming masonry unit • Damping
Monumental – multiple forms *Multiple degrees of freedom with different stiffnesses/periods* Most churches and larger civic structures		• Highly complex interaction between elements • Special study • Peer review recommended

C8.2.3 Foundations

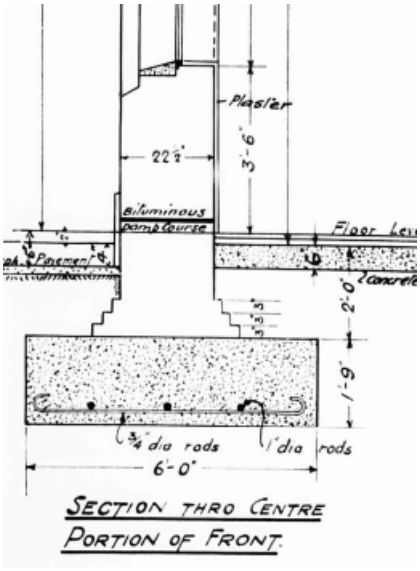
Foundations for URM buildings were typically shallow strip footings (refer to Figure C8.3(a)), including under openings in punched walls or facades. Bricks were typically placed transverse to the wall to give a half-to-one brick-thickening, although larger multi-stepped thickenings were used in large structures. The bricks were typically protected from direct contact with the ground with a layer of concrete. In smaller buildings, this was often thin and unreinforced.

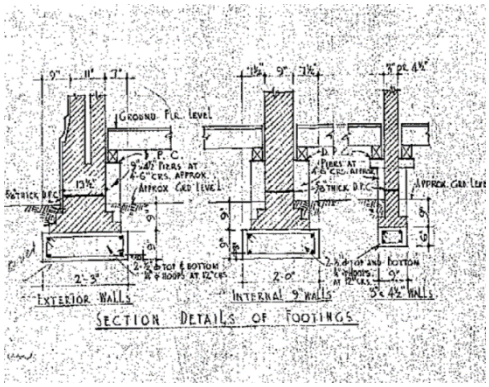
Deeper concrete strips (refer to Figure C8.3(b)) for larger buildings were often nominally reinforced with plain reinforcing bars, flats, or train/tram rails. In extremely poor ground or where the foundation formed a sea wall or wharf, these reinforced concrete strips generally spanned between driven timber or sometimes between steel or precast piles. The design was often rudimentary, with the depth of the concrete at least half that of the span regardless of reinforcement.

As the widening of the foundation was often nominal, some settlement was common in poorer ground either during or after construction. Settlement during construction could often be "built in" so would not be visible.

Larger industrial buildings with timber, steel or cast iron posts were often founded on large, isolated pads. As these were sized for the "live" actions, they are often lightly loaded so are an excellent indicator of settlement.

(a) Typical foundation details





(b) A cross section of URM building foundation

Figure C8.3: URM building foundations

C8.2.4 Wall construction

Solid and cavity walls were common types of construction:

- Solid walls were generally used for industrial buildings and buildings on the outskirts of town, and for party walls and walls either not visible or in lower storeys.
- Cavity walls were used in buildings to control moisture ingress. They also allow the use of higher quality bricks where a better architectural finish was required on the exterior.

In cavity walls, the exterior masonry wythes act as an architectural finish (which can give a misleading impression of these walls' structural thickness). It was also common to provide an outer wythe that was continuous over the full height of the wall plus an inner one-brick-thick wythe for the top storey and two or more wythes for lower storeys (refer to Figure C8.4). Construction quality was usually better for visible walls and veneers than in hidden areas or at the rear of buildings.

Figure C8.4: Change in cross section of brick wall (Holmes Consulting Group)

Often a cavity wall, which was originally on the exterior of the building, has become an interior wall following subsequent alteration. This will be recognisable by a wall thickness that is not a wythe multiple.

C8.2.4.1 Wall thickness

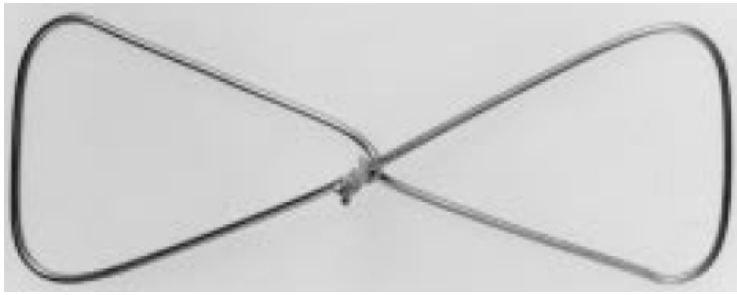
The commonly used nominal thicknesses of brick walls in New Zealand are 230 mm (9", two wythes), 350 mm (14", three wythes) and 450 mm (18", four wythes). This is in addition to any outer veneer of 110 mm (4½", one wythe).

C8.2.4.2 Cavity ties

In cavity walls, outer wythes were usually tied to the inner wythe or main structural wall with #8 ties, sometimes with a kink in the middle, or with flat pieces of tin generally at spacings of 900 mm horizontally and every fifth or sixth course vertically (refer to Figure C8.5). Cast steel, wrought steel or mild steel toggles were sometimes used at similar spacings.

(a) Common wire ties

(b) Double hook ties



(c) Butterfly ties

(d) V-drip flat fishtailed wall ties

Figure C8.5: Commonly observed wall ties (Dizhur)

C8.2.4.3 Masonry bond and cross sections

A number of different bond patterns have been used for URM buildings, as described below. The bond pattern is an important feature of URM buildings: it determines how the masonry units in a wall are connected and has a significant effect on both the wall strength and how its components act together as a complete structural member/element.

Stretcher units, or stretchers, are bricks laid in the plane of the wall. Header units, or headers are bricks laid across the wall joining the masonry wythes together.

In cross section, a wall three units thick is a three wythe wall. To act as one, each wythe should be adequately connected to the adjoining wythe with headers at appropriate intervals.

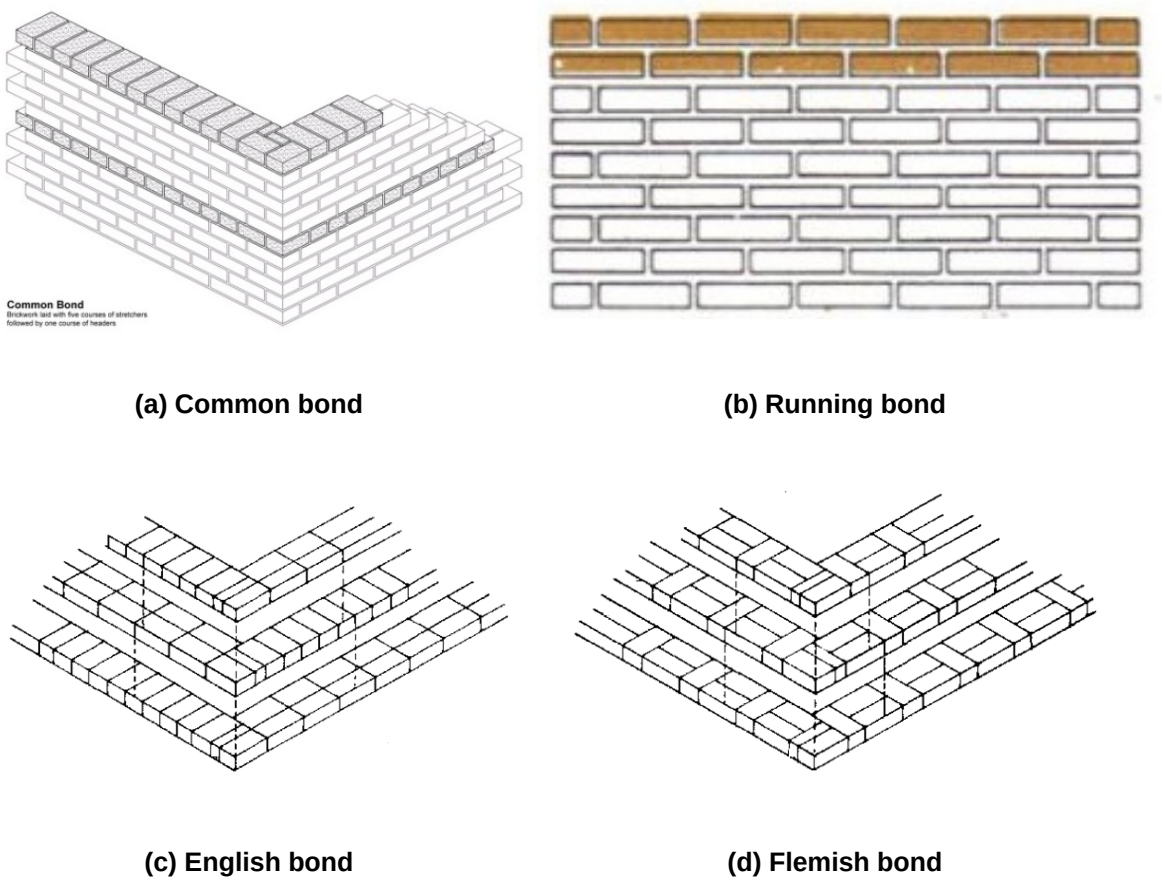
Note that sometimes fake headers are incorporated into a wythe that do not cover two adjoining wythes. These can disguise the presence of a cavity wall where there is a cavity void between the inner and outer wythes.

Clay brick masonry

Most New Zealand URM buildings were constructed with either common bond, which is the most frequently occurring bond pattern, or English bond, which is often found on the bottom (ground) storey.

Common bond is sometimes referred to as American bond or English garden wall bond. It has layers of stretchers, and headers every three to six courses (refer to Figure C8.6(a)). These headers can be at different levels in different buildings, and sometimes even within the same building. For example, the headers may be every second course at the bottom of the ground storey but every fourth course near the top of the third storey. Header courses may be irregular and made to fit in at ends of walls and around drainpipes with half widths and other cut bricks.

English bond has alternating header and stretcher courses (refer to Figure C8.6(c)).



(a) Common bond **(b) Running bond**

(c) English bond **(d) Flemish bond**

Figure C8.6: Different types of brick masonry bonds

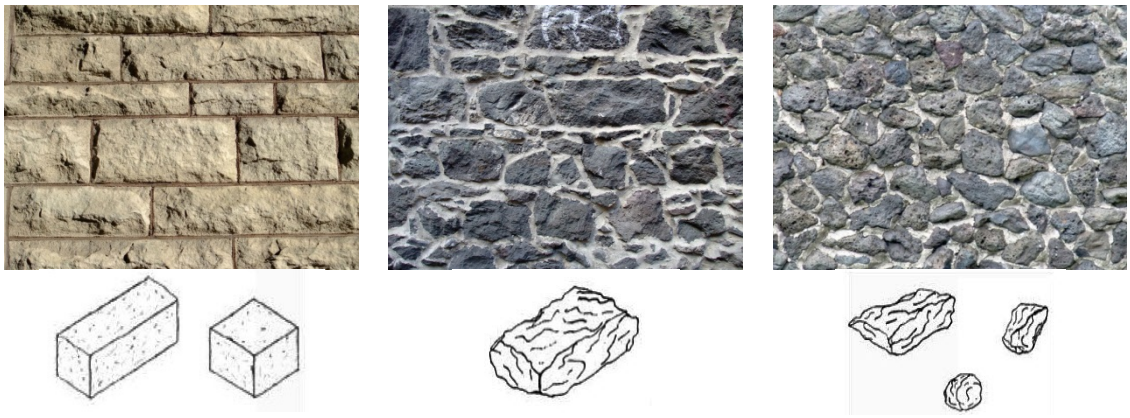
Other bond patterns used in New Zealand include Running bond (refer to Figure C8.6(b)) and Flemish bond (refer to Figure C8.6(d)). Running bond (stretcher courses only) often indicates the presence of a cavity wall. Flemish bond (alternating headers and stretchers in every course) is the least common bond pattern and is generally found between openings on an upper storey; for example, on piers between windows.

Stone masonry

Stone masonry buildings in New Zealand are mainly built with igneous rocks such as basalt and scoria, or sedimentary rocks such as limestone. Greywacke, which is closely related to schist, is also used in some parts of the country. Trachyte, dolerite, and combinations of these are also used.

Wall texture

Wall texture describes the disposition of the stone courses and vertical joints. There are three different categories (refer to Figure C8.7): ashlar (squared stone); rubble (broken stone); and cobble stones (field stone), which is less common.

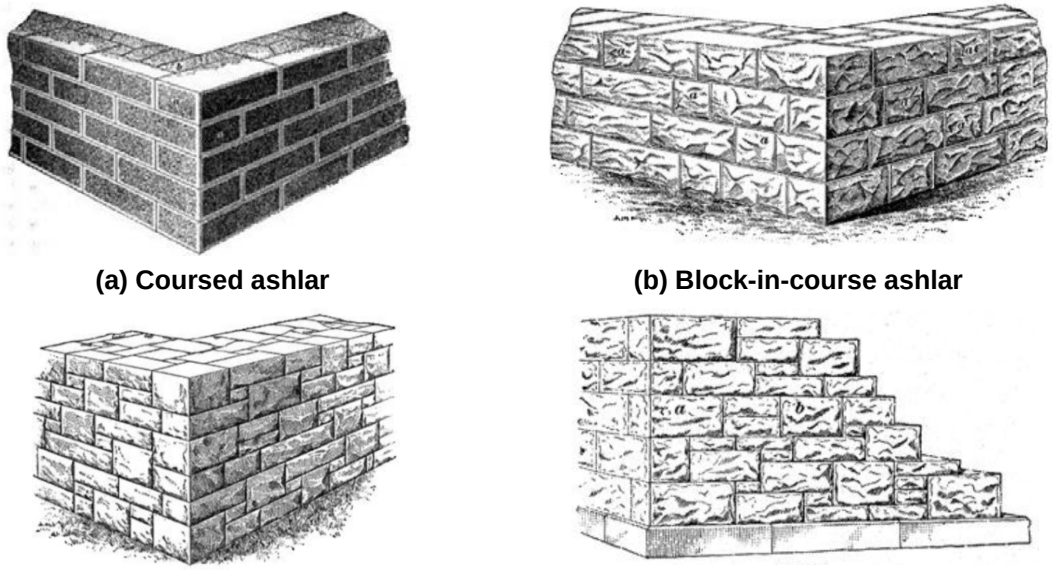


(a) Ashlar (squared stone) **(b) Rubble (broken stone)** **(c) Cobble stones (field stone)**

Figure C8.7: Classification of stone units (Giaretton)

Ashlar (dressed or undressed) is stonework cut on four sides so that the adjoining sides will be at right angles to each other (refer to Figure C8.7(a)). Ashlar is usually laid as either coursed ashlar, which is in regular courses with continuous joints (refer to Figure C8.8(a)), or block-in-course ashlar (refer to Figure C8.8(b)). It may also appear as broken courses (which describes the broken continuity of the bed and head joints) of either random-course ashlar (refer to Figure C8.8(c)), or broken ashlar (refer to Figure C8.8(d)).

All ashlar should have straight and horizontal bed joints, and the vertical joints should be kept plumb. This type of stone can also be found in coursed rubble; in which case it may be considered as a hybrid between rubble and ashlar stonework.



(a) Coursed ashlar **(b) Block-in-course ashlar**

(c) Random-course ashlar **(d) Broken ashlar**

Figure C8.8: Schematic of different forms of Ashlar bond (Lowndes, 1994)

Rubble stonework consists of stones in which the adjoining sides are not required to be at right angles (refer to Figure C8.7(b)). This form of masonry was often used for rough masonry such as foundations and backing, and frequently consists of common, roughly dressed field stone.

Wall cross section

It is usually not possible to establish the cross section characteristics of a stone masonry wall from the bond pattern. More detailed inspection is required to identify any connections between the wythes; to determine what material the core is composed of; and to locate any voids, a cavity, or the presence of other elements such as steel ties. All of these contribute to determining the wall's structural properties.

(a) Dressed stone in outer leaves and "rubble" fill

(b) Stone facing and brickwork backing

(c) Stone facing and concrete core

Figure C8.9: Stone masonry cross sections in New Zealand. Representative cases observed in Christchurch after the Canterbury earthquakes (Giaretton)

Concrete block masonry

Although solid concrete masonry was used in New Zealand from the 1880s, hollow concrete block masonry was not used widely until the late 1950s. Masonry was usually constructed in running bond, but stacked bond was sometimes used for architectural effect.

From the 1960s onwards, masonry was usually constructed with one wythe 190 mm thick, although this was sometimes 140 mm thick. Cavity construction, involving two wythes with a cavity between, was mostly used for residential or commercial office construction but occasionally for industrial buildings. The external wythe was usually 90 mm thick and the interior wythe was either 90 mm or 140 mm. Cavity construction was often used for infills, with a bounding frame of either concrete or encased steelwork.

To begin with, reinforcement in concrete masonry was usually quite sparse, with vertical bars tending to be placed at window and door openings and wall ends, corners and intersections, and horizontal bars at sill and heads and the tops of walls or at floor levels. Early on, it was common to fill just the reinforced cells. Later, when the depressed web open-ended bond beam blocks became more available, more closely spaced vertical reinforcement became more practicable. When the depressed web open-ended bond beam blocks (style 20.16) became available without excessive distortion from drying shrinkage, these tended to replace the standard hollow blocks for construction of the whole wall (with specials at ends, lintels and the like).

Wire reinforcement formed into a ladder structure ("Bloklok" or a similar proprietary product) was common in cavity construction. Two wires ran in the mortar in bed joints, joined across the cavity by another wire at regular centres and acting as cavity ties.

C8.2.5 Constituent materials

C8.2.5.1 Bricks

New Zealand brick sizes are based on imperial size. The most common nominal size of clay bricks used in masonry buildings is 230 mm x 110 mm x 70 mm (9"x 4½"x 3½').

C8.2.5.2 Mortar

Mortar is usually soft due to factors including inferior initial construction, ageing, weathering and leaching (refer to Figure C8.10). Both the type and proportions of mortar constituents varied significantly throughout the country. Until early last century, lime-sand mortar was common but cement-lime-sand mortar and cement-sand mortar were also used.

Note:

While the lime in lime mortars will continue to absorb moisture and "reset", over time it will leach and this leads to deterioration of the mortar.

Figure C8.10: Soft mortar. Note the delaminated mortar from bricks in the background. (Ingham and Griffith, 2011)

C8.2.5.3 Timber

Totara, rimu, matai (black pine) and kahikatea (white pine) were the most commonly used timber species in URM buildings.

C8.2.5.4 Concrete block

From the beginning, hollow concrete blocks were manufactured by the Besser process, where lean mix concrete was compacted into moulds using vibration. Concrete strength was usually 30 MPa or greater.

C8.2.6 Floor/roof diaphragms

Floors of URM buildings were usually made from timber and sometimes from reinforced concrete slabs.

C8.2.6.1 Timber floors

Timber floor diaphragms are usually constructed of 19-25 mm thick tongue and groove (T&G) membrane nailed to timber joists that are supported by timber or steel beams. Matai, rimu and oregon were commonly used for the floor diaphragm membrane. These timbers may have hardened from a century of drying and be "locked up" from long use. The diaphragm may also have been damaged by insect infestation or decay from moisture ingress. As well as the timber characteristics, the response of these diaphragms during an earthquake is dictated by the behaviour of the nail joints. It should be recognised that the nails in use a century ago were much softer than those used today. Resistance comes primarily from friction between the boards, complemented by "vierendeel" action from the pairs of nails in a board. A further complication is that the response of timber diaphragms is different for each direction, recognising that joists and boards span in different directions. Hence, diaphragm in-plane stiffness and strength should be assessed for earthquake loading oriented both parallel and perpendicular to the orientation of the joists.

C8.2.6.2 Reinforced concrete slabs

Reinforced concrete slabs were usually monolithic to brick walls and form a rigid diaphragm. While they may have been reinforced with bars, as is commonly the case for modern construction, these bars were often round or of a roughness pattern that provides significantly less bond than expected today. As a result, the presence of termination details (such as hooks, thickenings or threads/nuts) will have a marked effect on the load carrying capacity. Other types of reinforcement included expanded metal lath (refer to Figure C8.11) and even train rails.

Figure C8.11: Concrete slab with expanded metal lath reinforcement. Corrosion of the lath from carbonation of the concrete over time has caused the concrete to spall.

Portland cement gradually became available throughout New Zealand from the 1890s to the late 1920s, which was the time of much URM construction. Non-Portland cement concretes (often called “Clinker” concretes, as they were produced from only a single firing of lime products) are significantly weaker and should be assessed with caution. Similarly, as concrete was a relatively expensive material during these times, voids or ribs were often formed in slabs using hollow ceramic tiles.

Note:

Take care when making assumptions relating to the concrete strength. Intrusive investigation is essential to understand the makeup of the original slab construction and its constituents properly if forces greater than nominal are to be transferred.

C8.2.6.3 Roofs

The roof structure is usually provided with straight sarking (refer to Figure C8.12) or diagonal sarking (refer to Figure C8.13) nailed to purlins supported by timber trusses. Straight sarking has similar action to flooring, but boards are often square edges so they do not have the stiffness and strength of the high-friction tongue and groove connection. Diagonal sarking is naturally stiffer and stronger than rectangular sarking because the boards provide the diagonal “truss” members between the rafters and purlins. However, its ductility and displacement capacity will be less than for rectangular sarking as movements will cause direct shearing of the fixings along the lines of the boards.

Note:

Refer to Section C8.8.3 for the capacities of these types of systems. This is also covered in more detail in Section C9.

(a) Typical horizontal roof sarking

(b) Roof diaphragm with vertical sarking

Figure C8.12: Typical timber diaphragms – straight sarking

Figure C8.13: Typical timber diaphragms – diagonal sarking

The strength of both floor and roof diaphragms is complemented by the ceiling sheathing material. Common types of ceilings that provide structural capacity are timber lath-and-plaster, fibrous plaster, steel lath-and-plaster, and pressed metal. More modern additions of plywood boards and plasterboard may have also occurred over time.

C8.2.7 Diaphragm seating and connections

URM buildings are characterised by absent or weak connections between various structural components.

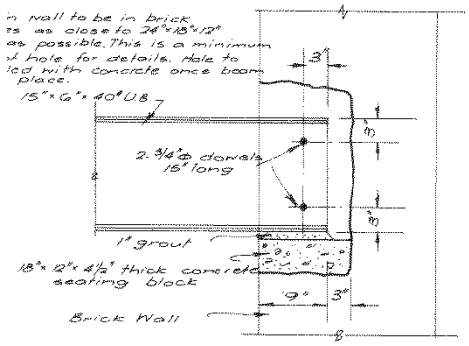
Often, walls parallel to the joists and rafters are not tied to the floors and roof respectively (refer to Figure C8.14), except in a few cases depending on the design architect. Wall-diaphragm anchor plates, sometimes referred to as rosettes or washers, have been used to secure diaphragms to walls since the late 19th century (refer to Figure C8.15). If these are present in a building, they may have been installed during the original construction or at any time since as a remediation measure.

Figure C8.14: A lack of connection of the walls parallel to joist and rafters with diaphragms and return walls leading to collapse of wall under face load

Figure C8.15: 1896 image showing anchor plate connections installed in early URM construction (National Library of New Zealand)

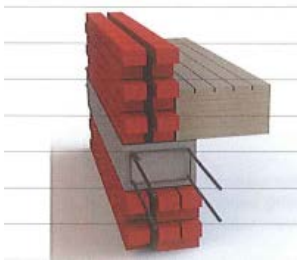
Even where walls are carrying beams, joists or rafters, they are not always secured to these elements. Connections made of steel straps tying the beams, joists or rafters to walls have been observed (refer to Figures C8.16 and C8.17), sometimes with a fish-tail cast into concrete pockets.

Another common feature is a gap on either side of the timber joists and beams to avoid moisture transfer from brickwork to timber. With such connections, horizontal shear cannot be transferred from walls to joists. However, if the joists are set tightly in the pocket they can be effective in horizontal shear transfer between the wall and floor structure.

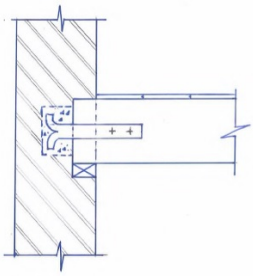


(a) Steel beam to wall pocketed connection

(b) Floor joist to wall connection. Note presence of steel strap (Matt Williams)



(c) Floor seating arrangement



(d) Fish-tail connection between wall and joist

Figure C8.16: Typical connection between masonry walls and joist

(a) Wall to roof truss connection (Miyamoto International)

(b) Roof seating arrangement and parapet wall (Dymtro Dizhur)

(c) Wall to roof truss connection. Note truss is seated on a concrete padstone (Miyamoto International)

Figure C8.17: Typical wall to roof connections

C8.2.8 Wall to wall connections

In most cases, there are no mechanical connections provided to tie orthogonal walls together. Concrete bands may be provided but may not be tied together at corners as it is possible that they were built by different teams at different stages. If they are jointed, it may just be with intermittent steel ties, or bricks pocketed into the abutting walls which have very little tie or shear capacity.

C8.2.9 Damp-proof course (DPC)

Most traditional buildings incorporate a damp-proof course (DPC) in the masonry between foundations and ground floor level. This can be made from galvanised metal, lead, slate, thick bitumen or bitumen fabric.

The DPC layer usually forms a slip plane (refer to Figure C8.18(a)) which is weaker than the surrounding masonry for sliding. It also forms a horizontal discontinuity which can affect bond for face loading or hold-down of walls for in-plane loading. Sliding on the DPC layer has been recorded, as shown in Figure C8.18(b).

Consideration of the DPC layer is an important part of establishing the capacity of the wall: refer to Section C8.8.6 for details.

(a) DPC below timber – Chest Hospital, Wellington

(b) Bitumen DPC and sliding evident after the 2013 Cook Strait earthquakes

Figure C8.18: Common DPC materials

C8.2.10 Built-in timber

Most traditional URM buildings incorporate built-in timbers (refer to Figure C8.19) for:

- fixing of linings, skirting, cornices and dado/picture rails
- plates supporting intermediate floor joists
- forming header connections between wall layers, and
- top plates for affixing rafters or trusses.

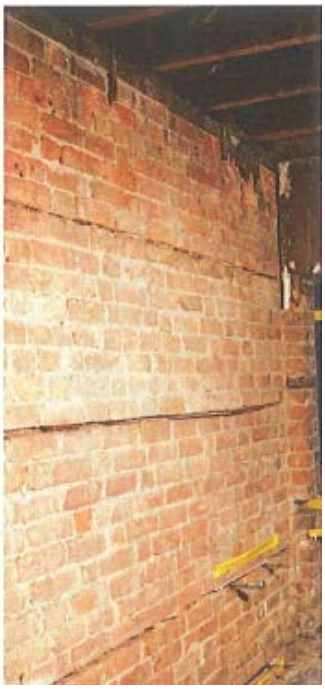


Figure C8.19: 12 mm timber built into every eighth course for fixing linings

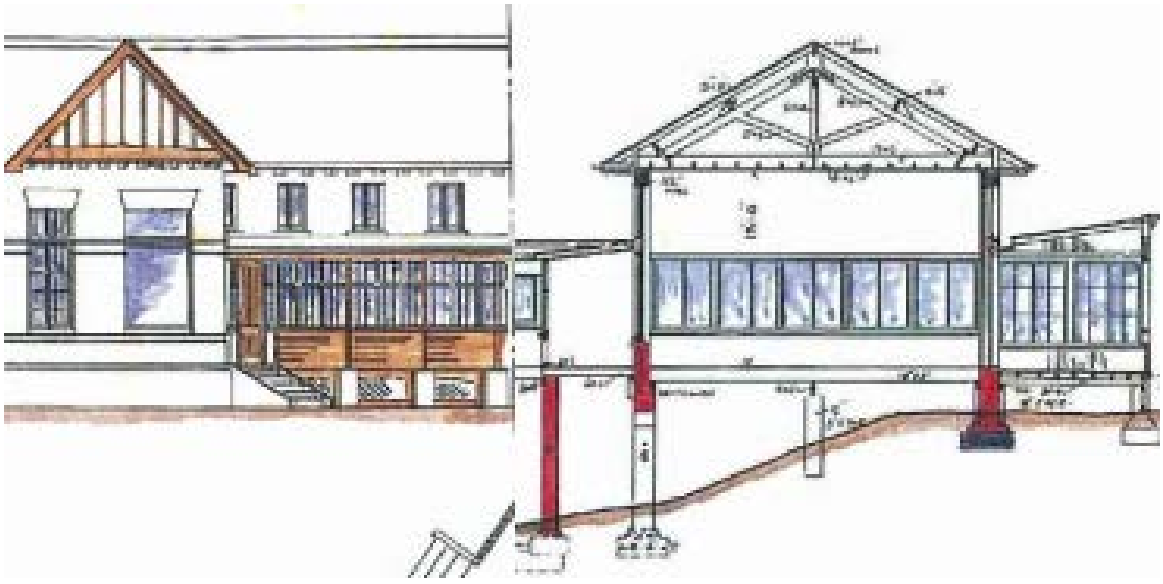
Degradation of these items is common, which causes localised stresses or bowing of walls. This will typically be more severe on the south side of buildings or nearer the ground level. Timber also shrinks, particularly perpendicular to the grain, and such timbers are often not in full contact with the surrounding masonry. In the case of continuous timber plates, engagement with the masonry is often limited to localised timber blocks notched into the walls.

C8.2.11 Bond beams

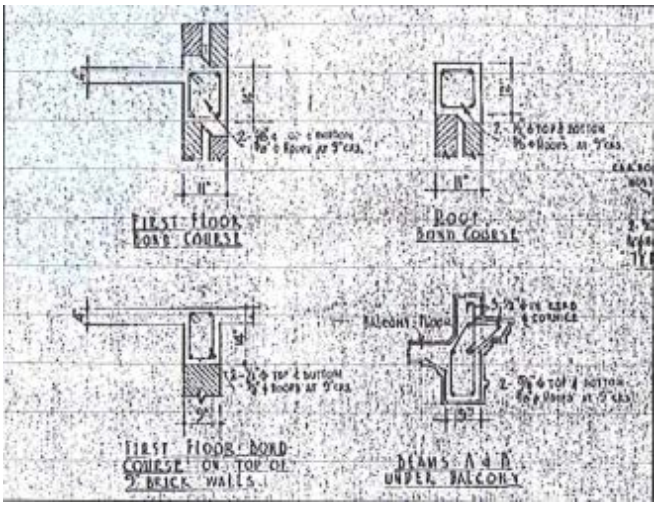
Bond beams or perimeter tie beams (refer to Figure C8.20) were typically constructed of reinforced concrete, plain concrete or timber. They can provide significant benefits to the performance of masonry buildings, including:

- providing a larger, often stronger substrate for the attachment of fixings and thereby providing better load distribution
- distributing diaphragm loads along the length of a wall
- tying wythes together in cavity construction (refer to Figure C8.20), provided that the bond beam is laid over both wythes
- providing coupling between wall panels for in-plane loads
- providing longitudinal tying to spandrel beams, and
- providing out-of-plane stability to face-loaded walls.

Depending on the age of the structure, there may be poor/no hook or termination details in reinforced concrete bond beams, so concentrated loads near the ends of such bond beams should be avoided. Stirrup reinforcement in these beams is often nominal – if present at all – so care should be taken when shear loads are being applied to these elements.



(a) Bond beam in cavity wall also forming lintel – Chest Hospital, Wellington (Dunning Thornton)



(b) Typical lintel detail (Dizhur)

Figure C8.20: Bond beams

The presence of a concrete band provides no surety that reinforcement is present. Figure C8.21 shows a concrete capping beam that is obviously not reinforced.

The reinforcement in the beam may also have degraded or may soon degrade if carbonation/chloride attack has penetrated into the concrete to the depth of reinforcing. When severe, this will split the concrete.

Figure C8.21: The wide cracks through bond beams indicate a lack of reinforcement in the beam (Dizhur)

C8.2.12 Bed-joint reinforcement

Bed-joint reinforcement (course reinforcement) varies in type and application. It can include:

- single wires or pairs of wires laid in mortar courses to augment in-plane performance
- single wires or pairs of wires laid in mortar courses to act as lintels or ties to soldier courses
- prefabricated/welded lattices laid in multi-wythe walls to ensure bond
- prefabricated/welded lattices laid across cavity walls to form cavity ties
- cast iron oversize cavity ties laid in multi-wythe walls to ensure bond, and
- chicken mesh.

Bed-joint reinforcement is often small in size relative to a fairly massive wall. It adds robustness but usually does not add significant structural strength.

This type of reinforcement is not usually apparent from a visual inspection. However, the requirement for bed-joint reinforcement was often noted in the original masonry specifications and has been observed in brick buildings.

C8.2.13 Lintels

Lintels commonly comprise:

- reinforced concrete beams over the full width of the wall
- reinforced concrete beams behind a decorative facing course, with this facing course supported on cavity ties or a steel angle
- steel angles
- steel flats (shorter spans)
- timber piece
- soldier course arches or flat arches, and
- stone lintels.

Arches or flat arches add a permanent outward thrust to a building which can destabilise walls in plane. This thrust along with any other forces should be resisted by ties in the building.

Reinforced concrete beams can contribute to in-plane pier/wall behaviour as they effectively reinforce the spandrel. However, they concentrate bearing loads at their supports and, if such frames dilate, can be points of overloading or destabilisation. They are also useful components for attachments for diaphragms (if the window heads are sufficiently high) as they provide a robust, blocky element to connect to.

C8.2.14 Secondary structure and critical non-structural items

Parapets are commonly placed on top of the perimeter walls. They are usually positioned off centre from the wall beneath, and capping stones or other ornamental features are then attached to the street side. Roof flashings are often chased into the brickwork on the external face just above roof level, creating a potential weak point in the masonry where rocking can occur.

Note:

Parapets, chimneys, pediments, cornices and signage (refer to Figure C8.22) on street frontages present a significant hazard to the public. The Ministry of Business, Innovation and Employment has issued a determination (2012/043) clarifying that external hazards such as these must be included in the seismic assessment rating of a building.

Heavy partition walls are potentially critical non-structural (or secondary structural) items which are usually not tied to the ceiling diaphragm and can pose a serious threat to life safety.

Figure C8.22: Secondary elements (Miyamoto International)

C8.2.15 Seismic strengthening methods used to date

Many URM buildings have been strengthened over the years either because of legislative requirements (e.g. earthquake-prone building legislation) or post-earthquake reconstruction (e.g. following the 1942 Wairarapa earthquake).

A number of strengthening techniques have been used (Ismail, 2012). The main principles were to tie unrestrained elements, such as chimneys and parapets, to the main load-bearing structure and to tie various building elements together so the building could act globally as a box with the intention that the available lateral capacity of the building could be fully mobilised even though it may not always have been increased.

Note:

Before 2004, seismic strengthening requirements for URM buildings were very low. In addition, in most strengthening projects the material properties were not verified by testing, anchors were mostly untested, and they were installed without documented quality assurance procedures.

Assessment of previously retrofitted buildings requires an understanding of the retrofit measures that historically have been carried out and the likely effect these would have on the seismic performance.

Techniques used historically for strengthening different structural mechanisms include:

- **chimneys:** internal post-tensioning and steel tube reinforcement, concrete filling, external strapping and bracing, removal and replacement
- **parapets:** vertical steel mullions, raking braces, steel capping, post-tensioning, internal bonded reinforcement, near surface mounted (NSM) composite strips
- **face-loaded walls:** vertical steel or timber mullions, horizontal transoms, post-tensioning, internal bonded reinforcement, composite fibre overlay, NSM composite strips, reinforced concrete or cementitious overlay, grout saturation/injection, horizontal and vertical reinforced concrete bands.
- **wall-diaphragm connections:** steel angle or timber joist/ribbon plate with either grouted bars or bolts/external plate, blocking between joists notched into masonry,

external pinning to timber beam end or to concrete beam or floor, through rods with external plates, new isolated padstones, new bond beams

- **diaphragm strengthening:** plywood overlay floor or roof sarking, plywood ceiling, plywood/light gauge steel composite, plasterboard ceiling, thin concrete overlay/topping, elastic cross bracing, semi-ductile cross bracing (e.g. Proving ring), replacement floor over/below with new diaphragm
- **in-plane wall strengthening/new primary strengthening elements:** sprayed concrete overlay, vertical post-tensioning, internal horizontal reinforcement or external horizontal post-tensioning, bed-joint reinforcement, composite reinforced concrete boundary or local reinforcement elements, composite fibre reinforced (FRP) boundary or local reinforcement elements, nominally ductile concrete walls or punched wall/frame or reinforced concrete masonry walls, nominally ductile steel concentric or cross bracing, limited ductility steel moment frame or concrete frame or concrete walls or timber walls, ductile eccentrically braced frame/K-frames, ductile concrete coupled or rocking walls, or tie to new adjacent (new) structure, structural plaster
- **reinforcement at wall intersections in plan:** removal and rebuilding of bricks with inter-bonding, bed-joint ties, drilled and grouted ties, metalwork reinforcing internal corner, grouting of crack
- **foundation strengthening:** mass underpinning, grout injection, concentric/balanced re-piling, eccentric re-piling with foundation beams, mini piling/ground anchors
- **façade wythe ties:** helical steel mechanical engagement – small diameter, steel mechanical engagement – medium diameter, epoxied steel rods/gauze sleeve, epoxied composite/non-metallic rods, brick header strengthening
- **canopies:** reinforcement or recast of existing hanger embedment, new steel/cast iron posts, new cantilevered beams, deck reinforcement to mitigate overhead hazard, conversion to accessible balcony, base isolation.

Figures C8.23 to C8.27 illustrate some of these techniques. Also refer Table C8.2 in Section C8.6.11, which lists common strengthening techniques and particular features or issues to check for each method.

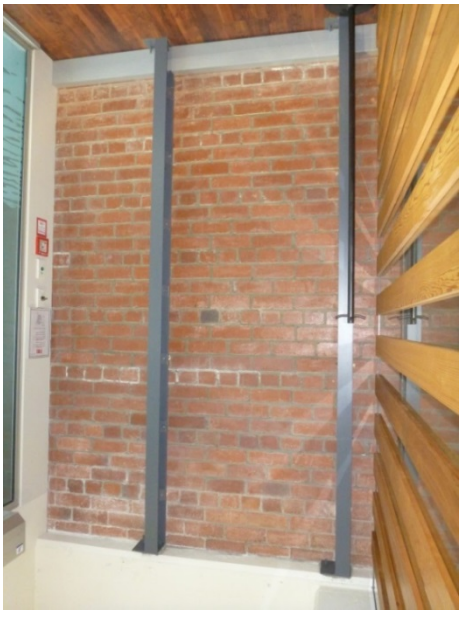
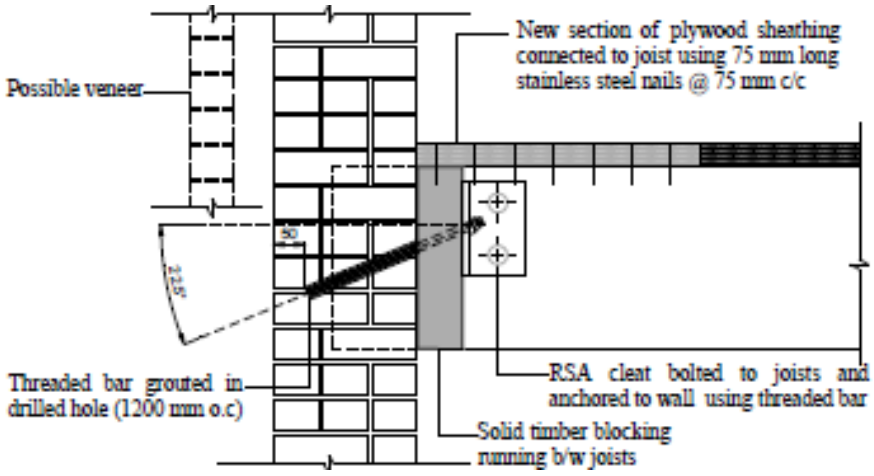
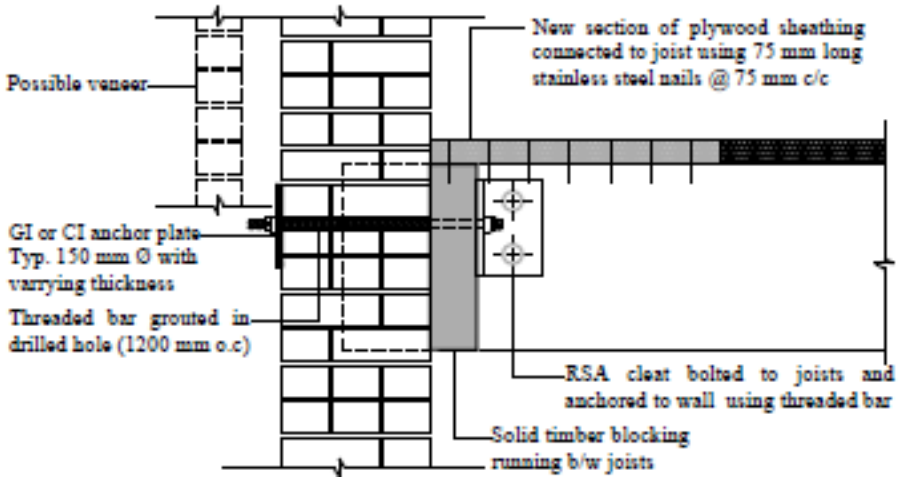


Figure C8.23: Bracing of wall against face load (Dunning Thornton)



(a) Bent adhesive anchor



(b) Through anchor with end plate (plate anchor)

Figure C8.24: Wall-diaphragm connections (Ismail, 2012)

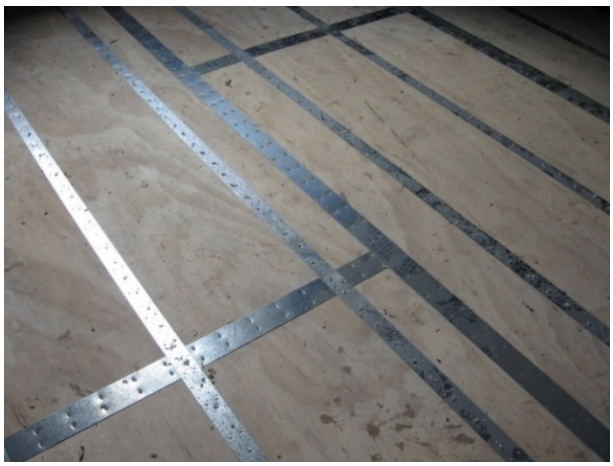
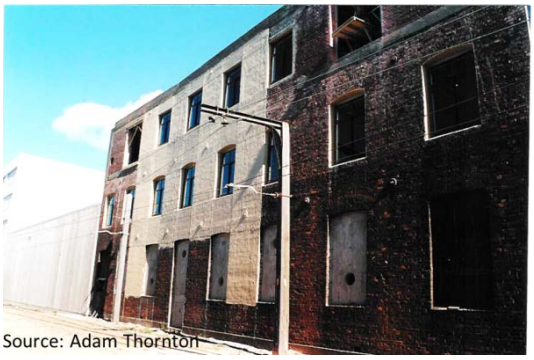


Figure C8.25: New plywood diaphragm (Holmes Consulting Group)

(a) Concentric steel frame (Beca)

(b) Steel frame (Dizhur)



(c) FRP overlay

(d) Steel frame (Dunning Thornton)

Figure C8.26: Improving in-plane capacity of URM walls

Strengthening of parapets is often carried out using racking braces, with one end tied to the timber roof structure (refer to Figure C8.27). However, issues with this method include a lack of vertical tie-down to counter the vertical force component of brace and ground shaking, or the flexibility of the roof amplifying shaking of the parapet.

Note:

When strengthening parapets, it is essential to make a robust connection down to the wall below and back into the structure. The danger of non-robust strengthening is that the parapet still fails, but collapses in larger, more dangerous pieces.

Figure C8.27: Parapet bracing. Note a lack of vertical tie-up of the parapet. (Dizhur)

C8.3 Observed Seismic Behaviour of URM Buildings

C8.3.1 General

When assessing and retrofitting existing URM buildings it is important to understand the potential seismic deficiencies and failure hierarchy of these buildings and their components.

The most hazardous of these deficiencies are inadequately restrained elements located at height, such as street-facing façades, unrestrained parapets, chimneys, ornaments and gable end walls. These are usually the first elements to fail in an earthquake and are a risk to people in a zone extending well outside the building perimeter.

The next most critical elements are face-loaded walls and their connections to diaphragms and return walls. Even though their failure may not lead to the building's catastrophic collapse, they could pose a severe threat to life safety.

However, when building members/elements are tied together and out-of-plane failure of walls is prevented, the building will act as a complete entity and in-plane elements will come under lateral force action.

Failures of URM buildings (summarised in Figure C8.28) can be broadly categorised as:

- **local failures** – these include the toppling of parapets, walls not carrying joists or beams under face load, and materials falling from damaged in-plane walls. These local failures could cause significant life-safety hazards, although buildings may still survive these failures.
- **global failures** – these include failure modes leading to total collapse of a building due to such factors as loss of load path and deficient configuration.

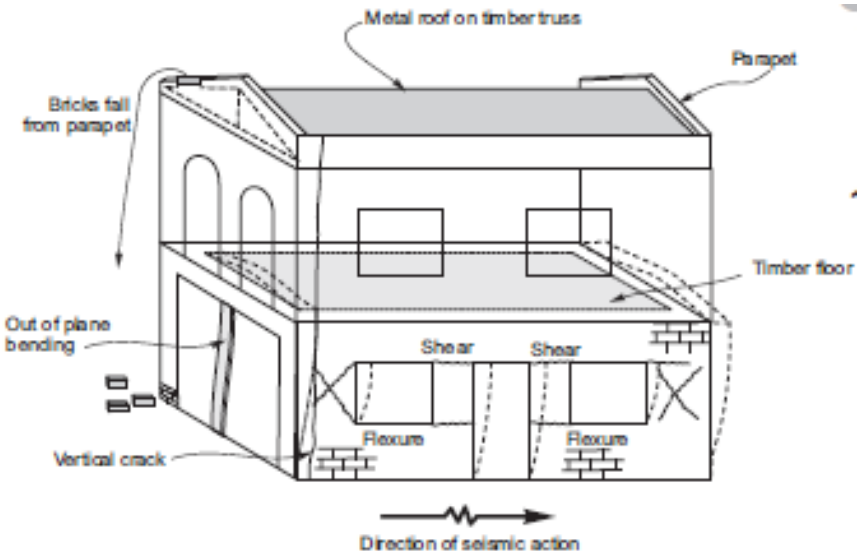


Figure C8.28: Failure modes of URM buildings

In URM buildings, in-plane demands on walls decrease up the height of the walls. In-plane capacity also decreases with height as the vertical load decreases. In contrast, out-of-plane demands are greatest at the upper level of walls (refer to Figure C8.29), but out-of-plane capacity is lowest in these areas due to a lack of vertical load on them. Hence, the toppling of walls starts from the top unless these are tied to the diaphragm.

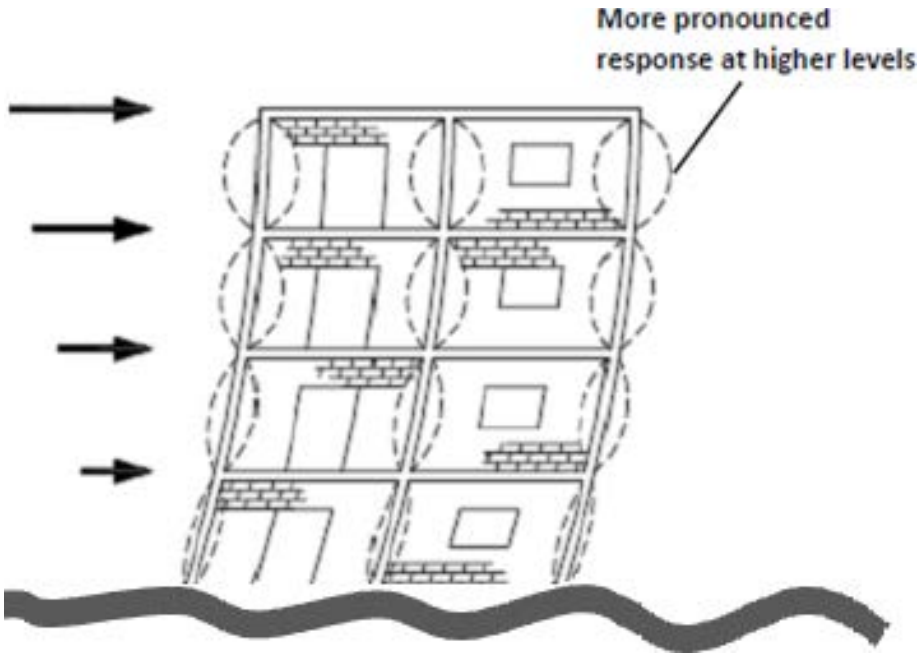


Figure C8.29: Out-of-plane vibration of masonry walls are most pronounced at the top floor level (adapted from Tomazevic, 1999)

C8.3.2 Building configuration

Building configuration tends to dictate the nature of URM failures. Cellular type buildings act as stiff structures, attracting high accelerations and therefore force-governed failure of their parts. Collapse of walls under face load as they try to span vertically and horizontally between floors and abutting walls respectively tends to be independent for each cell, depending on the angle of loading and the wall configuration.

Buildings where the span or flexibility of the diaphragm is an order of magnitude more than the walls tend to have more displacement-related failures. Walls and parapet collapse initiates from the mid-span of the diaphragm where movements are greatest (but accelerations are not necessarily as high).

Taller buildings may exhibit less damage at low levels than shorter buildings (refer to Figure C8.30), as the confinement of the masonry from the weight above provides significant strength. In larger buildings, the weaker elements (usually spandrels) fail first from bottom up (as shown later in this section, in Figure C8.46). This results in period lengthening of the structure and reduces the ability to transmit forces up the building.

As with all structures, the behaviour of URM buildings with a more regular configuration is generally more predictable. Buildings with irregular plan configurations, such as those on street corners (especially with an acute angle corner), suffer high displacements on their outer points. Shop fronts similarly experience high drifts, but these are often masked by "buttressing" from adjacent buildings in a "row" effect. This effect also disguises a vertical irregularity in which stiff façades tend to move as a solid element above the flexible open shop front.

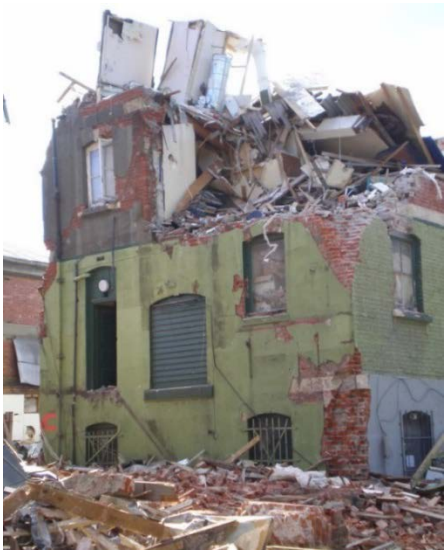


Figure C8.30: Reduction of damage towards base of building as axial load increases (Dunning Thornton)

C8.3.3 Diaphragms

The timber diaphragms commonly used in URM buildings are generally flexible, which may result in large diaphragm displacements during an earthquake. These will impose large displacement demand on the adjoining face-loaded walls, which could lead them to fail (refer to Figure C8.31).

Figure C8.31: Out-of-plane wall failure due to excessive roof diaphragm movement (Dizhur et al., 2011)

Figure C8.32 shows a photograph of delamination of plaster due to interaction between wall and ceiling due to shear transfer.

Figure C8.32: Lath and plaster ceiling. Note that stresses where shears are transmitted to the wall have caused the plaster to delaminate from the timber lath.

In some cases, diaphragm and shear-wall accelerations can increase with the flexibility of the diaphragm (Tena-Colunga and Abrams, 1996).

C8.3.4 Connections

C8.3.4.1 General

The following types of damage to wall-diaphragm connections have been postulated (Campbell et al., 2012) – the first four were actually observed during the 2010/11 Canterbury earthquake sequence:

- punching shear failure of masonry
- yield or rupture of connector rod
- rupture at join between connector rod and joist plate
- splitting of joist or stringer
- failure of fixing at joist
- splitting or fracture of anchor plate
- yield or rupture at threaded nut.

C8.3.4.2 Wall to wall connections

Connections between the face-loaded and return walls will open (i.e. there is return wall separation) after a few initial cycles of shaking (refer to Figure C8.33) because of stiffness incompatibility between stiff in-plane and flexible face-loaded walls and a natural dilation of a wall and pier assembly working in plane. This leads to loss of flange effect and softening of the building, resulting in a change in dynamic characteristics of the walls and piers. The integrity of connection between wall at junctions and corners depends on bonding between orthogonal walls.

While return wall separation can cause significant damage to the building fabric it does not necessarily constitute significant structural damage. This is provided the wall elements have adequate out-of-plane capacity to span vertically and there are enough wall diaphragm ties.

(a) Vertical cracks (Dizhur)

(b) Corner vertical splitting where walls are poorly keyed in together

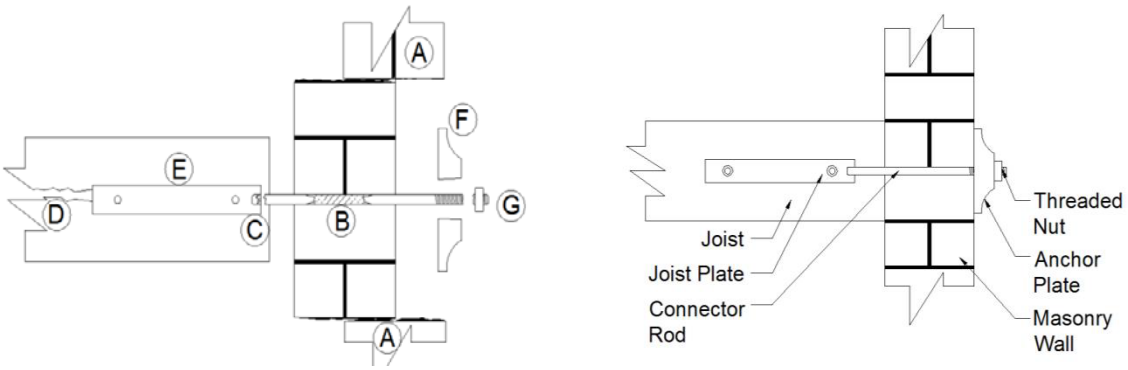
Figure C8.33: Damage to in-plane and face-loaded wall junctions

C8.3.4.3 Wall to floor/wall to roof connections

Failure of rosettes, rupture of anchor bars and punching shear failure of the wall was commonly observed following the 2010/11 Canterbury earthquake sequence (refer to Figure C8.34). This failure mode is characterised by failure of the mortar bed and head joints in a manner that traces a failure surface around the perimeter of the anchor plate. For multi-wythe walls the head joints will not be in alignment and, as for a concrete punching shear failure, it is possible that the failure surface on the interior surface of the wall may cover a broader area.

Figure C8.34: Plate anchor on verge of punching shear failure (Dizhur et al., 2011)

Testing at the University of Auckland (Campbell et al., 2012) has shown that anchor plates may exhibit a variety of different failure modes (refer to Figures C8.35 and C8.36 for examples) so their condition should be considered carefully.



(a) Location of failure modes **(b) Components of the connection assembly**

Figure C8.35: Wall-diaphragm anchor plate failure modes (Campbell et al., 2012)

(a) Sample 1-02: Failure where previously necked **(b) Sample 2-01: Brittle failure of anchor plate** **(c) Sample 2-02: Brittle failure where connector rod was fixed to joist plate**

(d) Sample 3: Failure where previously necked **(e) Sample 4: Failure at threaded region** **(f) Sample 6: Failure at threaded region**

Figure C8.36: Observed failure modes from tensile test series (Campbell et al., 2012)

Adhesive anchorages have been a popular form of anchorage for many years. These typically involve a threaded rod being chemically set into a drilled hole using either grout or epoxy adhesive. Unfortunately, there have been numerous observations of failed adhesive anchorages following the 2010/11 Canterbury earthquake sequence (refer to Figure C8.37). Reasons for this include:

- their use in regions expected to be loaded in flexural tension during an earthquake (such as on the rear surface of a parapet that may topple forward onto the street) – the brick

work was likely to crack in the vicinity of the anchorages and cause them to fail, even if the adhesive had been placed effectively

- incorrect installation – examples included cases of insufficient or absent adhesive, where the drilled hole had not been sufficiently cleared of brick dust from the drilling operation so there was inadequate bond to the brick surface, or where the inserted anchorage was of insufficient length
- anchors that were adequately set into a brick but the secured brick had failed in bed-joint shear around its perimeter. As a result, only the individual brick was left connected to the anchorage, while the remainder of the brickwork had failed.

Figure C8.37: Failed adhesive brick anchors (Dizhur et al., 2013)

C8.3.5 Walls subjected to face loads

Out-of-plane wall collapse under face load is one of the major causes of destruction of masonry buildings, particularly when a timber floor and roof are supported by these walls. The seismic performance of the URM face-loaded walls depends on the type of diaphragm, performance of wall-diaphragm connections and the wall-wall connection. Figure C8.38 illustrates the response of face-loaded walls to the type of diaphragm and wall-diaphragm connections.

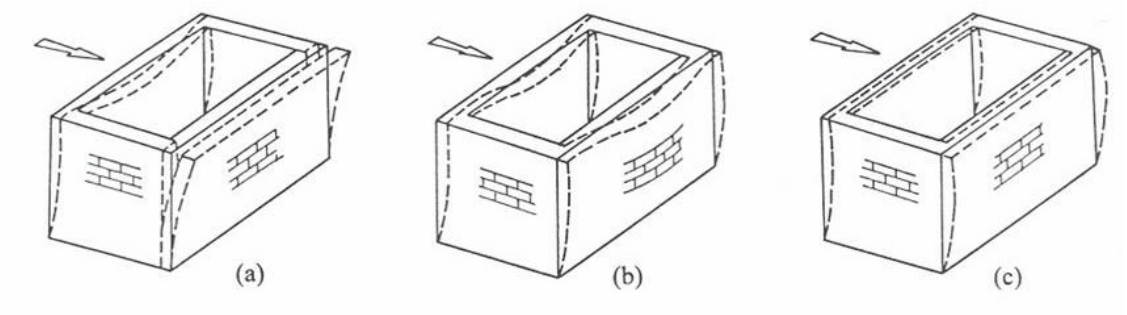


Figure C8.38: Effect of types of diaphragm on face-loaded walls – a) inferior wall-to-wall connection and no diaphragm, b) good wall-to-wall connection and ring beam with flexible diaphragm, c) good wall-to-wall connection and rigid diaphragm

Figures C8.39 and C8.40 show images of damage to masonry buildings due to collapse of walls under face load.

Figure C8.39: Out-of-plane instability of wall under face load due to a lack of ties between the face-loaded wall and the rest of the structure (Sharpe)

Gable end walls sit at the top of walls at the end of buildings with pitched roofs. If this triangular portion of the wall is not adequately attached to the roof or ceiling, it will rock as a free cantilever (similar to a chimney or parapet) so is vulnerable to collapse. This is one of the common types of out-of-plane failure of gable walls (refer to Figure C8.40).

Figure C8.40: Collapse of gable wall. Note a secured gable end that survived earthquake loading and a companion failed gable end that was not secured. (Ingham and Griffith, 2011)

Cavity wall construction can be particularly vulnerable to face-loading. Severe structural damage and major collapse of URM buildings with this type of construction was observed during the 2010/2011 Canterbury earthquake inspections (refer to Figure C8.41) and their performance was significantly worse than solid URM construction in resisting earthquake forces.

Figure C8.41: Failure of URM cavity walls (Dizhur)

The veneers of cavity wall construction also have the potential to topple during earthquake shaking (refer to Figure C8.41). Toppling is typically attributed to the walls' high slenderness ratio, deteriorated condition of the ties, overly flexible ties, pull-out of ties from the mortar bed joints due to weak mortar (refer to Figure C8.5), or a total absence of ties.

In multi-storey buildings the out-of-plane collapse of walls is more pronounced at the top floor level. This is due to the lack of overburden load on the walls and amplification of the earthquake shaking there (refer to Figure C8.29).

C8.3.6 Walls subjected to in-plane loads

Damage to URM walls due to in-plane seismic effects (in the direction of the wall length) is less significant than damage due to out-of-plane seismic effects. In addition, the stocky elements in URM (walls, piers and spandrels) usually make these structures more forgiving of distress in individual elements than the skeletal structures of modern framed buildings; principally, because the spectral displacements are small compared to the member dimensions. Nevertheless, some failure modes are less acceptable than others.

In general, the preferred failure modes are rocking or sliding of walls or individual piers. These modes have the capacity to sustain high levels of resistance during large inelastic straining. For example, sliding displacements at the base of a wall can be tolerated because the wall is unlikely to become unstable due to the shear displacements.

Masonry walls are either unpenetrated or penetrated. A penetrated wall consists of piers between openings plus a portion below openings (sill masonry) and above openings (spandrel masonry). When subjected to in-plane earthquake shaking, masonry walls and piers may demonstrate diagonal tension cracking, rocking, toe crushing, sliding shear, or a combination of these. Similarly, the spandrels may demonstrate diagonal tension cracking,

unit cracking or joint sliding. Figure C8.42 shows the potential failure mechanisms for unpenetrated and penetrated walls.

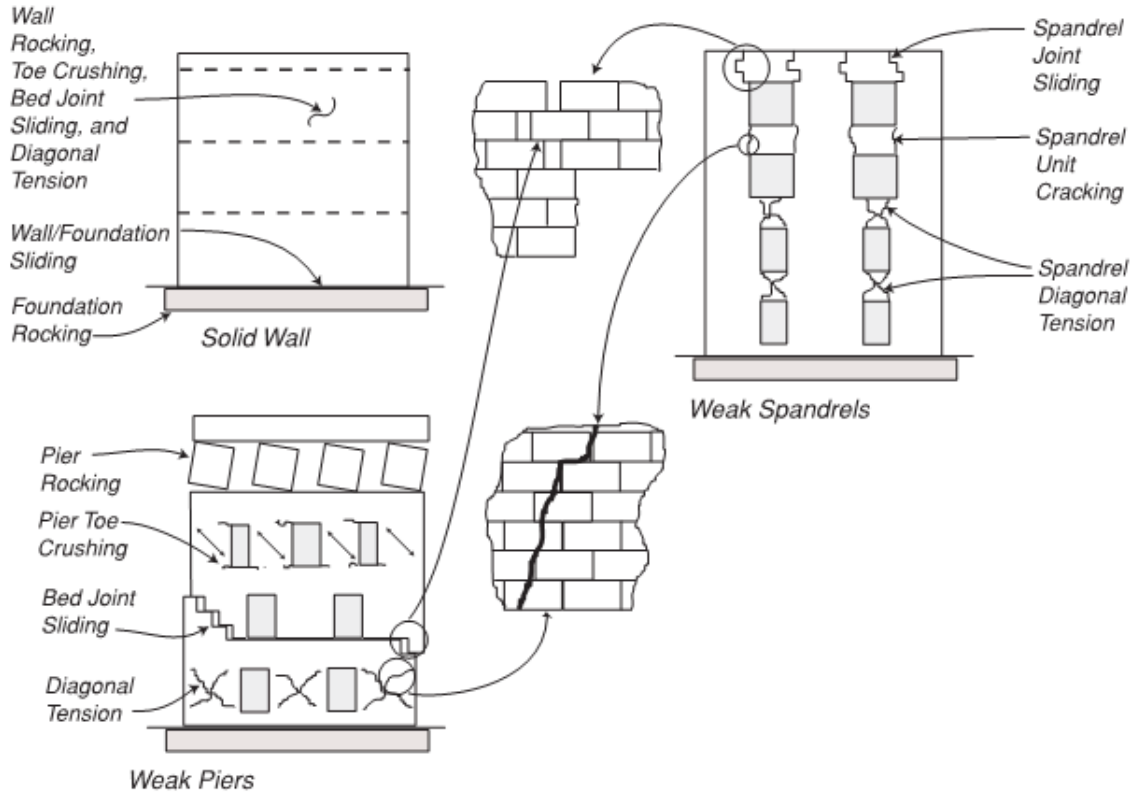


Figure C8.42: In-plane failure modes of URM wall (FEMA 306, 1998)

Rocking of URM piers may result in the crushing of pier end zones and, under sustained cyclic loading, bricks could delaminate if the mortar is weak. An example of this is shown in Figure C8.43, where the damage to the building is characterised by the rotation of entire piers.

Figure C8.43: Rocking and delamination of bricks of a one-storey unreinforced brick masonry building with reinforced concrete roof slab (Bothara and Hiçyılmaz, 2008)

Sliding shear can occur along a distinctly defined mortar course (refer to Figure C8.44(a)) or over a limited length of several adjacent courses, with the length that slides increasing with height (refer to Figure C8.44(b)). This can often be mistaken for diagonal tension failure, which is less common in walls with moderate to low axial forces.

(a) Sliding shear failure along a defined plane at first floor level (Dunning Thornton)

(b) Stair-step crack sliding, in walls with low axial loads (Bothara)

Figure C8.44: Sliding shear failure in a brick masonry building

Alternatively, masonry piers subjected to shear forces can experience diagonal tension cracking, also known as X-cracking (refer to Figure C8.45). Diagonal cracks develop when tensile stresses in the pier exceed the masonry tensile strength, which is inherently very low. This type of damage is typically observed in long and squat piers and on the bottom storey of buildings, where gravity loads are relatively large and the mortar is excessively strong.

(a) Diagonal tension cracks to a brick pier. Note splitting of bricks (Dizhur)

(b) Diagonal tension cracks to brick masonry. Note splitting of bricks, indicative of mortar stronger than bricks (Russell, 2010)

Figure C8.45: Diagonal tension cracking

In the penetrated walls, where spandrels are weaker than piers, the spandrel may suffer catastrophic damage (refer to Figure C8.46). This could turn squat piers into tall piers, resulting in a reduction in the overall wall capacity and an increase in expected deflections. The increase in deflection will increase the fundamental period of the building and reduce the demands which may be a mitigating effect. In any event, the consequences of failure of the spandrels and the resulting effect on life safety needs to be considered.

As noted in Section C8.2.9, sliding on the DPC layer has also been observed (refer to Figure C8.18 in that section).

Figure C8.46: Failure of spandrels. Also note rocking of upper piers and corner cracking of the parapet. (Dizhur)

C8.3.7 Secondary members/elements

The instability of parapets and chimneys is caused by these elements acting as rocking cantilevers which can topple when sufficiently accelerated (refer to Figure C8.47). Braced chimneys and parapets also failed during the 2010/11 Canterbury earthquake sequence (Ismail, 2012). Possible reasons include:

- bracing to the roof caused coupling with the vertical response modes of the roof trusses where the roof structure was flexible
- ties tying the parapets to the wall below the diaphragm level did not exist or were deficient
- strengthening standards were low (until 2004 the general requirement was to strengthen URM buildings to two thirds of NZSS 1900.8:1965)
- spacing between lateral support points was too large
- high vertical accelerations
- lack of deformation compatibility between support points (refer to Figure C8.47(b)).

(a) Out-of-plane instability of parapet (Beca) **(b) Chimney at onset of falling (Dizhur)**

Figure C8.47: Secondary members/elements

Canopies can be both beneficial and detrimental in relation to life safety (refer to Figure C8.48):

- They are often hung off the face of the buildings so columns supporting their outer edge do not obstruct the footpath or roadway. When subject to vertical loads, these diagonal hangers act to pry the outer layers of brick off the face of the building at the connection point.
- However, if they are sufficiently robust in their decking and fixings or if they are propped, they can provide overhead protection by taking at least the first impact of any falling objects.

Figure C8.48: Face-load failure of URM façade exacerbated by outward loadings from downward force on canopy. Note the adjacent propped canopy did not collapse. (Dunning Thornton)

C8.3.8 Pounding

This failure mechanism only occurs in row-type construction (refer to Section C8.5.4) where there is insufficient space between adjacent buildings so they pound into each other when vibrating laterally during an earthquake. Many examples of pounding damage to URM buildings were observed following the 2010/11 Canterbury earthquake sequence (refer to Figure C8.49).

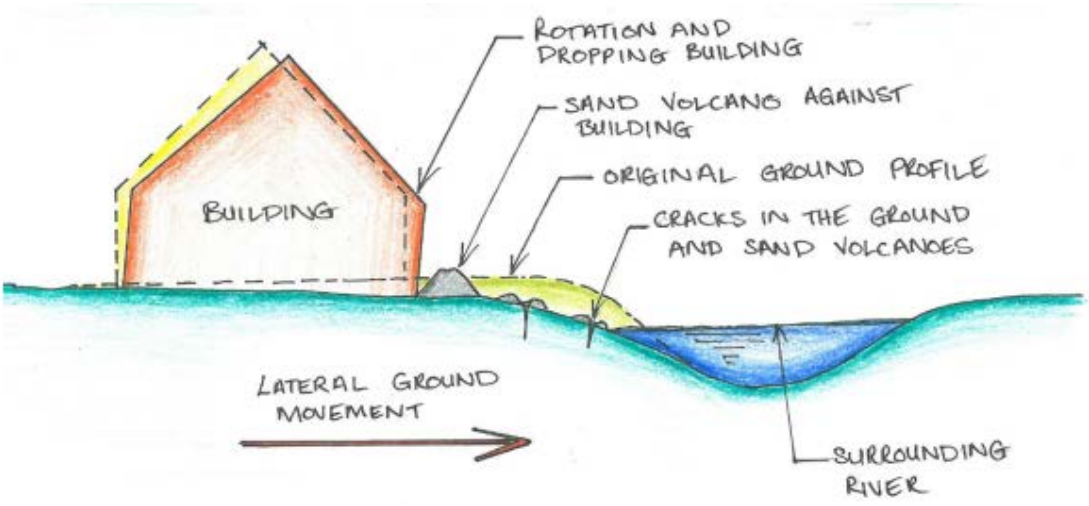
Figure C8.49: Pounding failure (Cole)

The magnitude of pounding depends upon the floor alignment between adjacent buildings, the difference in stiffness between the buildings, the pounding surface, floor weights, and clearance of structural separation between adjacent buildings if separation is provided.

C8.3.9 Foundations and geotechnical failure

Foundation damage that can be seen by inspection is commonly from lateral spreading and differential settlement. URM buildings typically have no tying capacity at foundation level, so they split at the weakest point along a wall. “Failure” is often an extremely large displacement (refer to Figure C8.50). However, given the slower and non-cyclic nature of lateral spreading, this is less likely to induce actual collapse until extreme displacements are reached.

(a) Large diagonal cracks and lateral movement of the access ramp caused by ground movement



(b) Settlement and lateral spread towards river

Figure C8.50: Earthquake-induced geotechnical damage to URM buildings (Neill et al., 2014)

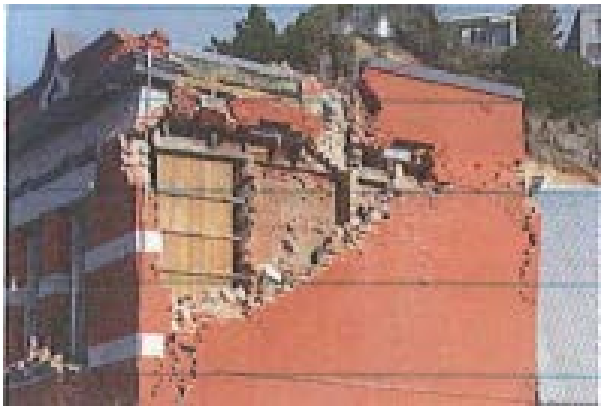
C8.4 Factors Affecting Seismic Performance of URM Buildings

C8.4.1 Number of cycles and duration of shaking

The strength and stiffness of URM degrades rapidly with an increasing number of cycles and the duration of ground shaking (refer to Figure C8.51). In general, a number of cycles of moderate acceleration sustained over time can be much more difficult for an URM building to withstand than a single, much larger peak acceleration (FEMA 454, 2006). Similarly, damage from higher acceleration, shorter period ground shaking from shallow earthquakes could be considerably greater than from deep earthquakes. This could affect stiffer URM buildings far more than flexible frame and timber structures.

(a) Post-September 2010 event – minor visible damage

(b) Post-February 2011 event – wall section on verge of failure



(c) Post-June 2011 event – wall collapse

Figure C8.51: Progressive damage and effect of shaking duration – 2010/11 Canterbury earthquake sequence (Dizhur)

Note:

The assessment of damaged buildings is outside the scope of these guidelines, and therefore progressive deterioration after the main event is not considered. It is assumed that the building will have been appropriately stabilised if this had been required after the main event.

C8.4.2 Other key factors

C8.4.2.1 General

Other key factors affecting the seismic performance of URM buildings include:

- building form
- unrestrained components
- connections
- wall slenderness
- diaphragm deficiency
- in-plane walls
- foundations
- redundancy
- quality of construction and alterations, and
- maintenance.

C8.4.2.2 Building form

A structurally irregular building suffers more damage than a regular building because of the concentration of both force and displacement demands on certain elements. An example of this is buildings along urban streets where the façades facing the street can be highly penetrated, with relatively narrow piers between openings, and the bottom storey could be totally open. This configuration could impose significant torsional demand and soft/weak storey mechanism. This can result in increased displacement demand and may lead to collapse.

C8.4.2.3 Unrestrained components

Instability of parapets and chimneys is caused by their low bending strength and high imposed accelerations. When subject to seismic actions, they rock on their supports at the roof line and can topple over when sufficiently accelerated by an earthquake.

C8.4.2.4 Connections

URM buildings can show significant resilience to seismic shaking as long as the building and its components can maintain their integrity. The wall-diaphragm anchors serve to reduce the vertical slenderness of a wall and also to make the building elements work together as a whole, rather than as independent parts. However, one of the most significant deficiencies in URM buildings in New Zealand is the lack of adequate connections; particularly those between walls and diaphragms.

C8.4.2.5 Wall slenderness

Unreinforced face-loaded masonry walls are weak in out-of-plane bending so are susceptible to out-of-plane failures. The earthquake vulnerability of a URM wall to out-of-plane bending is predominantly dictated by its slenderness (the ratio between thicknesses to span of wall). Cavity walls are especially vulnerable as the steel ties connecting the exterior wythes to the backing wall can be weakened by corrosion.

C8.4.2.6 Diaphragm deficiency

Diaphragms act as a lid to a box and are essential for tying the walls together and ensuring that lateral loads are transferred to the lateral load resisting elements. If diaphragms are too flexible, their ability to do this is compromised. Excessive diaphragm displacement imposes large displacement demand on walls, particularly on face-loaded walls, which could result in wall collapse.

C8.4.2.7 In-plane walls

These walls provide global strength and stiffness against earthquake load. Their seismic performance is defined by: the slenderness of walls and piers; vertical load; size and location of penetrations; relative strength between mortar and masonry units; and presence of bond beams, built-in timber and DPC.

C8.4.2.8 Foundations

Foundation flexibility and deformation affect the local and global earthquake response of URM buildings. However, foundations tend to be quite tolerant to deformations and building failure is rarely caused by ground settlement unless the ground underneath the building liquefies or suffers lateral spreading. Foundation effects or soil-structure interaction tend to reduce the force demand on the primary lateral-force-resisting elements, such as stiff in-plane loaded walls. At the same time, ground deformation can pose an additional rotational demand on the bottom storey wall under face load. The base fixity of the wall needs to be considered carefully as do the conditions at the wall base that have accumulated over the building's life (such as undermining by broken drains, clay heave or alteration of the surrounding soil or levels), and if these have changed with earthquake-induced liquefaction.

Existing high bearing pressures require careful consideration with respect to possible liquefaction-induced settlements. Settlement of long solid walls is often not a critical consideration for a URM building as the upper floors and roof frame into the walls with pin connections. However, careful consideration of the induced damage to any perpendicular/abutting walls is essential. For taller walls, ratcheting down with cyclic in-plane actions may be a consideration (refer Section C4). With little or no reinforcement in the footings (or ground slabs if present), there will be little resistance to lateral spreading or ground lurch, so vulnerability to these induced displacements should be assessed.

C8.4.2.9 Redundancy

The redundancy of a building refers to the alternative load paths able to add to resistance. The ability to redistribute demands through a secondary load path is an important consideration, as a building with low redundancy will be susceptible to total collapse if only one of its structural elements fails.

C8.4.2.10 Quality of construction and alterations

URM buildings in New Zealand represent an old building stock which has gone through many changes of occupancy. As a result, there may have been a number of structural modifications at different times which may not have been well considered, such as opening new penetrations in walls and diaphragms, removing existing components and adding new components. Such alterations will affect seismic performance.

C8.4.2.11 Maintenance

Older buildings that have been insufficiently maintained will have reduced material strength due to weathering (refer to Figure C8.52), corrosion of cavity ties (refer to Figure C8.53), rotting of timber and other processes that weaken masonry, connection capability, timber and reinforced concrete members. Similarly, water penetration in lime-based masonry will lead to leaching of lime from the mortar.

Figure C8.52: Severely degraded bricks and mortar due to moisture ingress (Ingham and Griffith, 2011)

The metallic cavity ties used in the original construction of URM cavity walls typically have no corrosion protection so are prone to severe deterioration (refer to Figure C8.53).

Figure C8.53: Metal cavity ties in rusted condition (Dizhur et al., 2011)

C8.5 Assessment Approach

C8.5.1 General

The assessment of a URM building requires an understanding of the likely behaviour of a number of building components and how these are likely to interact with each other.

The nature of the construction of this type of building means that each one is unique in terms of construction, quality of the original workmanship and current condition.

Therefore, it is important that the engineer has an appreciation of how the building was constructed, its current condition, the observed behaviour of similar buildings in previous earthquakes and a holistic view of the factors likely to affect its seismic performance. These issues are discussed in Sections C8.2, C8.3 and C8.4, which are considered to be essential reading prior to progressing through the assessment processes outlined in this section.

It is a general recommendation of these guidelines that the capacity of a building should be considered independently from the demands (imposed inertial loads and displacements) placed on it, bringing both together only in the final step of the assessment process. This is no different for URM buildings and is the basis behind the recommended assessment processes outlined below.

Past observations in earthquakes indicate that some components of URM buildings are particularly vulnerable to earthquake shaking and a hierarchy in vulnerability can be identified that can be useful in guiding the assessment process. Figure C8.54 shows a capacity "chain" for a typical URM building, with component vulnerability decreasing from left to right on the chain. The capacity of the building will be limited by the capacity of the weakest link in the chain, and the ability of each component to fully develop its capacity will typically be dependent on the performance of components to the left of it on the chain. This suggests that the assessment of component capacities should also proceed from left to right in Figure C8.54.

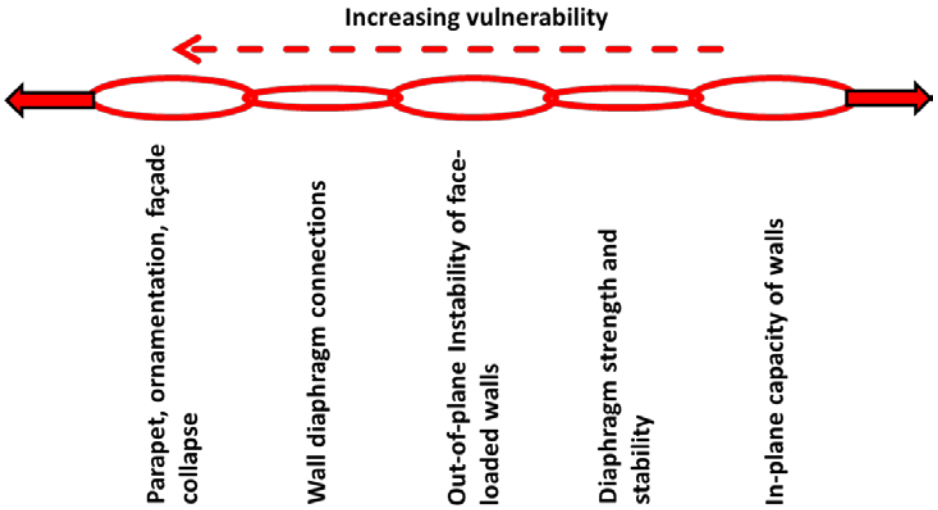


Figure C8.54: The capacity "chain" and hierarchy of URM building component vulnerability

While the critical structural weakness in a structural system will often be readily apparent (e.g. lack of any positive ties from brick walls to floors/roof) it will generally be necessary to evaluate the capacity of each link in the chain to fully inform on the components that require retrofit and the likely cost of this.

URM buildings come in different configurations, sizes and complexity. While complex buildings may require a first principles approach to the assessment of element capacity and internal actions within elements, simplifications are possible for more basic structures. Guidance is provided for both the detailed complete solutions and basic solutions for common simple buildings.

In Section C8.5.2 the assessment process, as it applies to URM buildings, is discussed with particular emphasis on how the approach might be varied depending on the complexity of the building. The assessment approach will also be influenced by any previous strengthening (refer to Section C8.5.3), and its location (including when it is a row building (refer to Section C8.5.4)).

C8.5.2 Assessment process

Key steps involved in the assessment of URM buildings are shown in Figure C8.55 and described below.

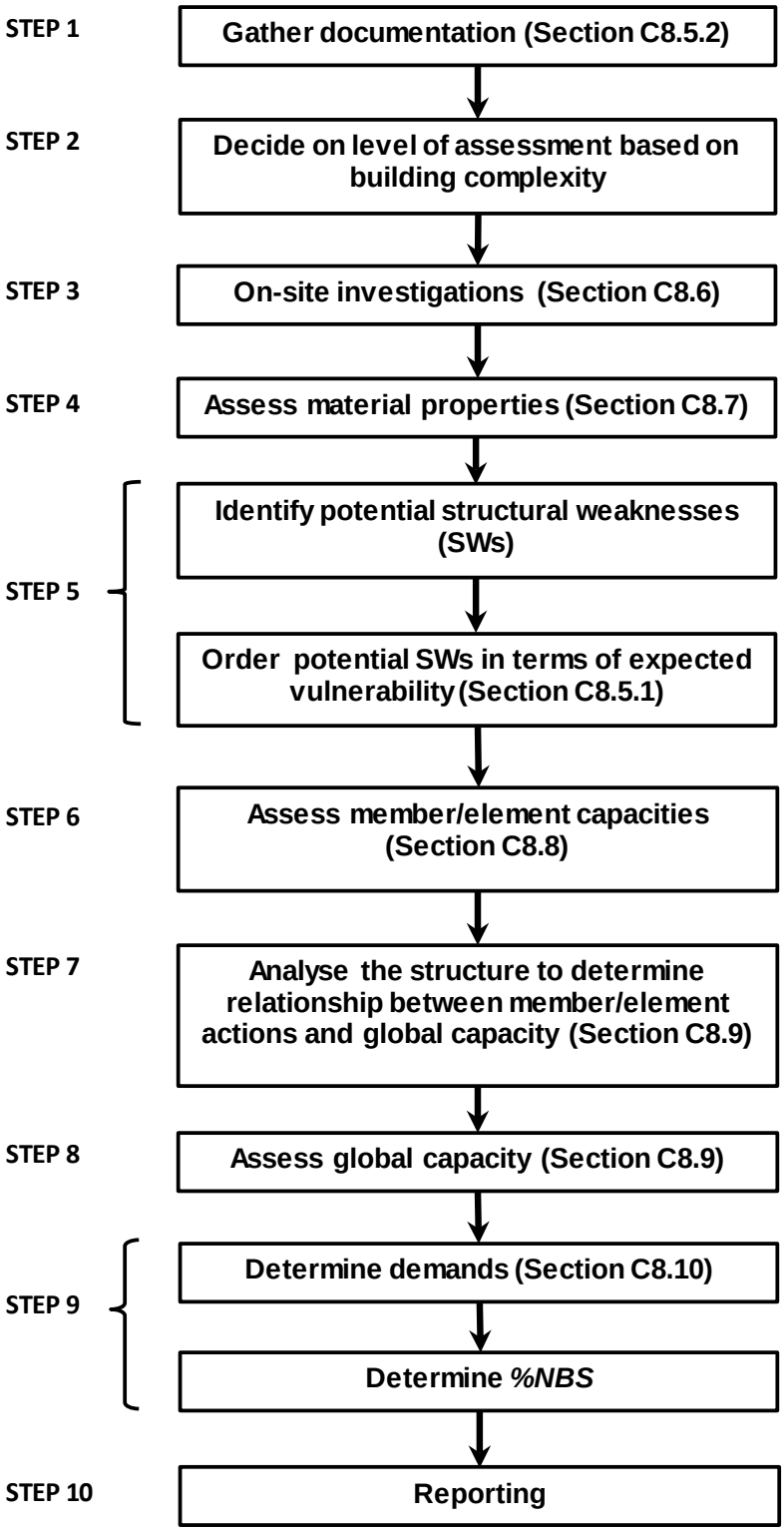


Figure C8.55: Assessment process for URM buildings

Step 1 Gather documentation

Collect relevant information and documents about the building including drawings, design feature reports, calculations and specifications, and any historical material test results and inspection reports (if available).

If the building has been previously altered or strengthened, collect all available drawings, calculations and specifications of this work.

Study this information before proceeding with the on-site investigation.

Step 2 Consider building complexity

Determine an assessment strategy based on an initial appraisal of the complexity of the building. This can be reviewed as the assessment progresses.

Although all aspects will need to be considered for all buildings, simplifications can be made for basic buildings e.g. one or two storey commercial, rectangular in plan. For these buildings the default material strengths are expected to be adequate without further consideration so that on-site testing, other than scratch testing of the bed joints to ascertain mortar type and quality, is not considered necessary. Foundation rotations are also not expected to have a significant effect so can be ignored.

Concentration of effort should be on assessing the score for face-loaded walls, connections from the walls to the diaphragms and the diaphragms (lateral deflection between supported walls). The score for the walls in plane will depend on the ability (stiffness) of the diaphragm to transfer the shears but the calculations required are likely to be simple irrespective of whether the diaphragms are rigid (concrete) or flexible (timber, steel braced). Behaviour can be assumed to be linear-elastic (i.e. ignore any nonlinear behaviour).

Complexity is likely to be increased if a building has previously been retrofitted. Not all issues with the building will necessarily have been addressed in historical retrofits. Stiffness compatibility issues will often not have been considered or fully addressed.

Step 3 Investigate on-site

Refer to Section C8.6.

Evaluate how well the documentation describes the “as constructed” and, where appropriate, the “as strengthened” building.

Carry out a condition assessment of the existing building.

Complete any on-site retrieval of samples and test these.

Identify any site conditions that could potentially affect the building performance (refer to Section C4).

Step 4 Assign material properties

Start by using the probable material properties that are provided in Section C8.7, or establish actual probable values through intrusive testing (the engineer may come back to this step depending on the outcome of the assessment).

Recognise that for basic buildings obtaining building-specific material strengths through testing may not be necessary to complete an assessment.

Step 5 Identify potential structural weaknesses and relative vulnerability

First, identify all of the various components in the building, and then identify potential structural weaknesses (SWs) related to these.

The identification of potential SWs in this type of building requires a good understanding of the issues discussed in Sections C8.2, C8.3 and C8.4.

Early recognition of SWs and their relative vulnerability and interdependence is likely to reduce assessment costs and focus the assessment effort.

Prior experience is considered essential when identifying the SWs in complex buildings.

Separate the various members/elements into those that are part of the primary lateral load resisting system and those that are not (secondary structural). Some elements may be categorised as having both a primary lateral load resisting function (e.g. in-plane walls and shear connections to diaphragms) and a secondary structural function (e.g. face-loaded walls and supporting connections).

The relative vulnerability of various elements in typical URM buildings is likely to be as follows (refer also Figure C8.54):

- **Inadequately restrained elements located at height:** e.g. street-facing façades, unrestrained parapets, chimneys, ornaments and gable end walls. Collapse of these elements may not lead to building collapse but they are potential life safety hazards and therefore their performance must be reflected in the overall building score.
- **Inadequate connection between face-loaded walls and floors/roof:** little or no connection capacity will mean that the walls will not be laterally supported when the inertial wall forces are in a direction away from the building. It can then be easily concluded that the walls and/or connections will be unlikely to score above 34*%NBS*, except perhaps in low-seismic regions. If observations indicate reasonable diaphragm action from the floors and/or roof, adequate connections will mean that the out-of-plane capacity of the face-loaded walls may now become the limiting aspect.
- **Out-of-plane instability of face-loaded walls:** if the wall capacity is sufficient to meet the requirements set out for face-loaded walls, then the capacity of the diaphragms becomes important as the diaphragms are required to transfer the seismic loads from the face-loaded walls into the in-plane walls.
- **The in-plane capacity of walls:** these are usually the least vulnerable elements.

Step 6 Assess element capacities

Calculate the seismic capacities from the most to the least vulnerable element in turn. There may be little point in expending effort on refining existing capacities only to find that the capacity is significantly influenced by a more vulnerable item that will require addressing to meet earthquake-prone requirements or target performance levels. Connections from brick walls to floors/roof diaphragms are an example of this. Lack of ties in moderate to high seismic areas will invariably result in an earthquake-prone status for the masonry wall and therefore it may be more appropriate and useful to assess the wall as < 34*%NBS* and also calculate a capacity assuming ties are in place. This will inform on the likely effect of retrofit measures.

An element may consist of a number of individual members. For example, the capacity of a penetrated wall (an element) loaded in-plane will need to consider the likely behaviour of each of the piers and the spandrel regions between and above and below the openings respectively (the elements). For some elements the capacity will be a function of the capacity of individual members and the way in which the members act together. Therefore, establishing the capacity of an element may require structural analysis of the element to determine the manner in which actions in the members develop.

For each member/element assess whether or not exceeding its capacity (this may be more easily conceptualised as failure for these purposes) would lead to a significant life safety hazard, (refer to Part A and Section C1 for discussion of what constitutes a significant life safety hazard). If it is determined that it will not, then that member/element can be neglected in the assessment of the expected seismic performance of the structure. The same decisions may need to be made regarding the performance of members within an element.

Step 7 Analyse the global structure

In general, the complexity and extent of the analysis should reflect the complexity of the building.

Start with analyses of low sophistication, progressing to greater sophistication only as necessary.

An analysis of the primary lateral load resisting structure will be required to determine the relationship between the global capacity and the individual member/element actions.

The analysis undertaken will need to recognise that the capacity of members/elements will not be limited to consideration of elastic behaviour. Elastic linear analysis will likely be the easiest to carry out but the engineer must recognise that restricting to elastic behaviour will likely lead to a conservatively low assessment score.

The analysis will need to consider the likely impacts of plan eccentricities (mass, stiffness and/or strength).

Step 8 Assess global capacity

From the structural analyses determine the global capacity of the building. This will be the capacity of the building as a whole determined at the point that the most critical member/element of the primary lateral load resisting system reaches its determined capacity.

It may also be useful to determine the global capacity assuming successive critical members/elements are addressed (retrofitted). This will inform on the extent of retrofit that would be required to achieve a target score. A member/element will not be critical if its failure does not lead to a significant life safety hazard.

Step 9 Determine the demands and *%NBS*

Determine the global demand for the building from Section C3 and assess the global *%NBS* (global capacity/global demand x 100).

Assess the demands on secondary structural items and parts of the building and assess *%NBS* for each (capacity/demand x 100).

List the *%NBS* values in a table.

The critical structural weakness (CSW) will be the item in the table with the lowest *%NBS* score and that *%NBS* becomes the score for the building.

Review the items in the *%NBS* table to confirm that all relate to elements, the failure of which would lead to a significant life safety hazard. If not, revise the assessment to remove the non-significant life safety elements from consideration.

Step 10 Reporting

Refer to Part A and Section C1.

C8.5.3 Assessment of strengthened buildings

Seismic assessment of URM buildings that previously have been strengthened is similar to that undertaken for un-strengthened structures except that the performance of previously installed strengthening members/elements has to be taken into account. (Table C8.2 in Section C8.6 provides a detailed list of strengthening techniques used in URM buildings and associated features.)

Issues requiring consideration include the capacity of the installed elements, diaphragm continuity, and deformation compatibility between the original and installed strengthening elements.

C8.5.3.1 Wall-to-diaphragm anchors

The effectiveness of existing wall-to-diaphragm anchors needs to be verified. Examples of poorly performing anchors that are known to have been used in previous strengthening projects include:

- **Shallow embedment grouted anchors:** anchors installed with low embedment depths (i.e. less than half the wall thickness) were observed to perform poorly under face loads (Moon et al., 2011).
- **Grouted plain round bar anchors:** plain round bars have a low bond strength compared with threaded bar or deformed reinforcing bar anchors.
- **Mechanical expansion anchors:** mechanical anchors do not generally perform well in URM due to the low tensile capacity of masonry and the limited embedment depths that can be achieved with available mechanical anchors.

The default connector strengths detailed in Section C8.8.4 can be used for existing wall to diaphragm anchors that are in good condition and are known to have been installed and tested in accordance with the requirements of Appendix C8A.

Existing non-headed wall anchors of unknown construction should be proof tested in accordance with the test procedures detailed in Appendix C8A.

Existing headed wall anchors should be tested if there is evidence of significant corrosion or if anchor capacities greater than the default values detailed in Section C8.8.4 are required.

Existing wall-to-diaphragm anchor connections that rely on cross-grain bending of boundary joists should be reviewed. Cross-grain bending will occur in the boundary joist when face-loaded walls pull away from supporting floor diaphragms for the case when wall anchor

brackets are not provided (refer Figure C8.56). Timber has low cross-grain bending capacity and, in many instances, has been found to be inadequate to resist the necessary seismic loads in past earthquakes (ICBO, 2000). Capacity is greatly improved if the ribbon board or solid blocking is well connected to the joists. Where the connection is to a boundary joist, the presence of solid blocking between one or more pairs of joists should be checked, with adequate connection to the joists.

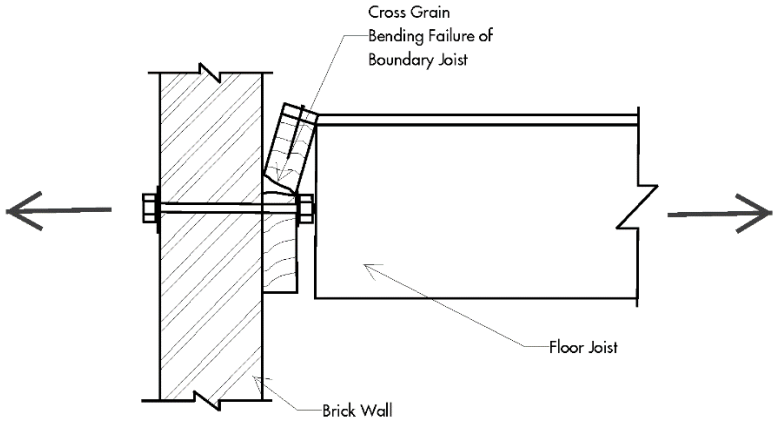


Figure C8.56: Out-of-plane loading cross grain bending failure mechanism (Oliver, 2010)

C8.5.3.2 Diaphragm continuity

Detailing of existing strengthened diaphragms should be reviewed to ensure that reliable load paths exist to transfer the inertia loads from the face-loaded URM walls into the body of the diaphragm.

Existing nailed plywood sheathing joints should not be relied upon to transfer tension forces unless adequate detailing is provided at the joint locations (ICBO, 2000). The sub-diaphragm design methodology can be used to assess existing diaphragm strengthening continuity (Oliver, 2010), with checks then made to assess if those discontinuous diaphragms that arise when continuity is not realised or is lost can continue to fulfil the role of structural diaphragms, even if originally not intended to be discontinuous.

C8.5.3.3 Deformation compatibility

Flexible lateral load resisting systems, such as structural steel or reinforced concrete moment resisting frames, have been used to strengthen URM buildings (refer to Figure C8.26(a)).

When assessing the effect of strengthening measures such as this, deformation compatibility between the stiff URM structure and the more flexible lateral load resisting system needs to be considered.

An understanding of the nonlinear strength-deformation relationship for each strengthening element will be required so that this can be compared with the relationships determined for the URM elements and other structural systems that may be present.

Often it will not be possible to mobilise the full capacity of a flexible strengthening element before the deformation capacity of the URM is exceeded. If so, one option available is to delete the URM from the primary seismic resisting system (assuming there is confidence that a significant life safety hazard does not arise from the failure of the masonry) and reassess the capacity.

C8.5.4 Assessment of URM row buildings

URM row buildings are buildings of similar structural form arranged side by side with insufficient seismic gaps to their neighbours, often with common boundary (party) walls: i.e. there is interaction between the individual buildings during a seismic shaking such that they cannot be considered in isolation. Buildings interconnected across boundaries should be considered as one building for the purposes of assessment, (refer to Part A).

Note:

The guidance below has been inferred from observed building damage only.

The effect of seismic shaking on row buildings is complex but also one of the least researched topics, particularly for URM buildings. It requires a special study which is outside the scope of these guidelines.

The effects of seismic shaking due to a lack of seismic gap can be both favourable (for the building within the row) and unfavourable (for the buildings on the ends of the row) provided the buildings are similar. Both of these effects should be accounted for when assessing the building's overall seismic performance. The building or structure within a row could become an end building if adjacent buildings are demolished.

Favourable effects include the potential for the whole block of row buildings to act as one unit and share seismic loads, and buttressing of central buildings by adjacent buildings in a row or an isolated building.

Unfavourable effects include pounding (knee effect and impact) on vertical load-bearing elements; the loss of which could potentially lead to loss of the gravity load path.

Buildings at the ends of rows suffer from two significant additional effects. First, they can be subject to the inertia/pounding effects of not just the adjacent building but some accumulation of effects along the row. Second and more importantly, forces tend to be almost unidirectional, pushing the end buildings off the row. This ratcheting effect is particularly detrimental to masonry structures where strains/crack widths accumulate much more quickly than when elements are able to complete a full return cycle. Therefore, the standard procedures for the assessment of buildings at the ends of rows should be used with care and consideration for these effects.

Note:

These guidelines recommend that all row effects on a particular building from the overall structure are described as part of its analysis and the vulnerabilities recorded. A "building" may be being assessed as if it is on one title, but the building from a structural connectivity point of view may extend for the whole block. The connectivity of the parts should be brought to the Building Consent Authority's (BCA's) attention throughout the assessment or retrofit consent process. Strengthening one "building" as part of a row will reduce the hazard in that section, but the seismic capacity of the overall building may still remain low due to the capacities in the remainder of the structure. The legal and compliance effects of row buildings should be discussed and agreed with owners and BCAs as part of any assessment process.

C8.5.4.1 General performance

The performance of row buildings depends primarily on the alignment (or otherwise) of:

- floor diaphragms
- façades
- primary transverse bracing elements, when situated against the boundary, and
- common walls.

The extent of misalignment of floors increases the bending effect on structures that are common to both buildings. When the extent of misalignment is greater than the depth of the wall, shear failure can also be induced.

Often, even if floors are misaligned, the façades are in the same plane (this is common in URM buildings). As a large proportion of the mass of the building is in the façade, it will not participate in the pounding action between the misaligned floors.

The effect of pounding damage to masonry buildings is generally less than for a frame or rigid diaphragm building as it tends to be more localised. Because of the high stiffness and often low height of these buildings, the impact forces are high frequency and associated with small displacements, and therefore carry less energy. Façades and other walls in the same alignment pound in their strong direction. Pounding between parallel walls where the pounding energy is dispersed over a large area will have a smaller effect than localised punching.

In addition to the above, most URM buildings have timber floors which have little mass to cause pounding. Similarly, with flexible diaphragms the impact energy is absorbed over a larger displacement. However, it is important to consider that URM is a brittle material and is sensitive to impact. Therefore, the engineer should consider whether the damage caused is likely to lead to loss of significant vertical load-carrying elements.

C8.5.4.2 Building interconnection

If row buildings share common walls but are not reliably tied together they are considered as one building with interconnected structures (refer to Part A). However, the length of dependable seating of the floors, or roof elements on the common wall will need to be assessed against the relative displacement of each building section.

If they are tied, note that the performance of elements that provide tying between the buildings (and similarly retrofit ties) can be classified into three types: rigid, elastic unbonded, and ductile. Rigid and elastic unbonded elements transfer force without dissipation of energy. For elastic unbonded elements, if there is sufficient stretch to allow the relative movement of the two structures their different stiffnesses will interact and will interrupt each other's resonances. Some force will also be lost through pounding as the elements return together. Where floors align, the ties may take the form of simple rods or beams. Where floors misalign, these rods/beams will be coupled to a vertical column element which will (elastically) transfer the floor force across the offset.

C8.6 On-site Investigations

C8.6.1 General

The engineer will need to conduct a detailed building inspection in order to assess existing building strength and before preparing any strengthening proposal.

This on-site investigation should cover the whole building, paying particular attention to the rear of the building and any hidden areas. It should include, but not be limited to, the following aspects.

C8.6.2 Form and configuration

Verify or establish the form and configuration of the building and its various components, including load paths between members, elements, and systems. As URM buildings may have had many changes of occupancy, there may be significant differences between available documentation and the actual building. Record this if so.

Note the number of storeys, building dimensions and year of construction. The notes of building dimensions should include opening locations and their dimensions, and should identify any discontinuities in the structural system.

Note the structural system and material description, including vertical lateral force-resisting system, basement and foundation system.

Also note any architectural features that may affect earthquake performance, including unrestrained items such as parapets or chimneys.

Note adjacent buildings and any potential for pounding and falling hazards. (Also refer to Section C8.5.4 for specific implications for row buildings.)

C8.6.3 Diaphragm and connections

Note the diaphragm types. For timber diaphragms, investigate the timber type, joist and beam spacing, and their connections, membrane and cladding type.

Note the presence of floor and roof diagonal bracing systems and the dimensions of these elements.

Examine wall-diaphragm connections and anchorage types (mechanical, adhesive and plate) to identify details and condition. Removal of floor or ceiling tiles may be required to investigate connections and anchorage types. Record the condition of these connections, any variation in connection types and other features such as any alterations or deterioration.

Note:

If adhesive anchors are used, these warrant careful investigation. In some cases, a visual inspection will not be sufficient and an on-site testing programme should be considered.

A dribble of epoxy on the wall can indicate that the anchor hole was filled properly. However, it may also indicate that there are voids between segments of adhesive along the length of the anchor; or that the anchor was inserted, taken out and reinserted.

For pocket type connections, check if the joists/rafters/beams are tightly packed by masonry on both sides or if there is a gap on both sides of the joists/rafters/beams.

When inspecting the diaphragm, note the location and size of the penetration accommodating stair or elevator access. Studies have shown that when penetrations are less than 10% of the diaphragm area it is appropriate to reduce in-plane diaphragm stiffness and strength in proportion to the reduction in diaphragm area. However, for larger diaphragm penetrations a special study should be undertaken to establish their influence on diaphragm response.

Note if the diaphragm has previously been re-nailed at every nail joint using modern nails placed by a nail gun or if it has been varnished.

The assessment should also consider the quality of the fixings from any sheathing to the supporting structure to transfer the loads and prevent buckling of the diaphragm. Plaster, especially if cementitious, will act to protect the fixings. However, rusting of nails and screws can cause splitting of timber which can drastically reduce the strength of a sarking board of the supporting framing. These guidelines encourage careful examination for rusting or signs of leaks, especially in roof cavities if these are accessible.

C8.6.4 Load-bearing walls

Record the walls' general condition including any deterioration of materials, damage from past earthquakes, or alterations and additions that could affect earthquake performance.

For multi-wythe construction, record the number of wythes, the distance between wythes, placement of inter-wythe ties, and the condition and attachment of wythes. Note that cavity walls will appear thicker than the actual structural wall.

Record the bond type of the masonry, including the presence and distribution of headers. If possible, confirm that the bond bricks (headers) are not fake and cover more than one wythe. Check if the collar joint is filled.

Check any unusual characteristics, such as a mix of walling units or unusual crack patterns.

Record the type and condition of the mortar and mortar joints (for example, any weathering, erosion or hardness of the mortar) and the condition of any pointing or repointing, including cracks and internal voids. It is important to establish the mortar strength relative to the bricks as stronger mortar can lead to a brittle mode of failure. Investigation of existing damage to masonry walls can reveal their relative strength. Damage to bricks indicates a stronger mortar and weaker brick.

Note:

Visual inspection and simple scratching of the bricks and mortar may be sufficient to investigate the quality of masonry constituents. To be fully effective, the visual inspection should include both faces of the masonry.

Note that the mortar used for pointing is usually far better than the actual main body of the mortar, so scrape the point to full depth to investigate this.

The extent to which detailed testing of the materials should be considered will depend on the importance of the building and the likely sensitivity of the material properties to the assessment result.

Check any damp areas and the rear part of the building to investigate the quality and deterioration of the masonry and its constituents.

Note any horizontal cracks in bed joints, vertical cracks in head joints and masonry units, or diagonal cracks near openings.

Record the presence of bond beams and their locations, and covered walls. Signs of cracking or decay should be investigated and, where appropriate, include chemical testing. Refer to Section C5 for further information on concrete testing.

Examine and record any rotting and insect infestation of timber. Investigate timber in contact with masonry, particularly in damp areas.

Record the presence of any DPC layers.

Identify any vertical member/elements that are not straight. Bulging or undulations in walls should be observed. Note any separation of exterior wythes, out-of-plumb walls, and leaning parapets or chimneys. Check URM party walls and partitions and investigate whether these are tied to the structural system.

If opening up is permitted, include areas with built-in timbers (described in Section C8.2.10) so allowance can be made during the analysis. This analysis should allow for the brick capacity only, with no beneficial support from the timber unless specific investigations can prove otherwise. Existing bowing of walls and a lack of vertical load path where timber plates have shrunk can severely reduce face load capacity.

C8.6.5 Non-loadbearing walls

Record the material and construction details of the non-loadbearing walls. These walls may stiffen the floor diaphragm and brace the main loading walls. Their weight could be a significant portion of the total weight.

Check any unusual wall plaster construction.

C8.6.6 Concrete

Take care when making assumptions relating to the concrete strength and detailing. Intrusive investigation is essential to understand the makeup of the original construction and its constituents properly if any greater than nominal forces are to be transferred.

C8.6.7 Foundations

Note the type, material and structure of the foundation system.

Check if the bricks are in contact with the soil. Degradation can occur depending on the extent to which the bricks were fired when originally produced, and/or if the soil is damp.

C8.6.8 Geotechnical and geological hazards

Carefully investigate any foundation settlement or deterioration due to vegetation. In particular, check around drains and slopes.

Note any geological site hazards such as susceptibility to liquefaction and conditions for slope failure and surface fault rupture. Look for past signs of ground movement.

C8.6.9 Secondary elements

Record the details of secondary elements such as parapets, ornamentation, gable walls, lift wells, heavy equipment, canopies and chimneys. Include details of their dimensions and location. Also check for the presence of capping stones or other ornamental features as these create additional mass and eccentricity.

In particular, check if parapets are positioned off-centre to the wall beneath. Inspect parapets to estimate the location of the rocking pivot.

C8.6.10 Seismic separation

Investigate seismic separation with adjacent buildings. (Note that an apparent presence of a structural separation is not necessarily an indication that pounding will not occur unless the entire length of the separation is clear of any obstructions between the two buildings (Cole et al., 2011).

C8.6.11 Previous strengthening

Verify any strengthening systems that have been used against available drawings and documentation. Record any variations and deterioration observed. Check as-built accuracy and note the type of anchors used, their size and location. Use Table C8.2 to check for particular issues that can arise with different strengthening techniques: record any relevant observations. Also refer to Section C8.5.3 for additional considerations for strengthened buildings, including deformation compatibility between the original and installed strengthening elements.

Table C8.2: Historical techniques used for URM buildings and common features

Structural mechanism	Technique	Comments/issues
Chimneys	Internal post-tensioning	Requires well-mapped, understood and not degraded vertical load-path
	Internal steel tube reinforcement	Wrap-around/tie reinforcement to connect to tube important
	Concrete filling	Adds mass Adhesion to surrounding brick often insufficient to tie
	External strapping	Inward collapse needs to be checked, especially if mortar degraded on inside Geometry often means external frames step outward: changes in angle need full resolution not to apply stress concentrations to masonry

Structural mechanism	Technique	Comments/issues
	External bracing	Raking braces should have all vertical components of load resolved at each end Compatibility of stiff braced chimney with a flexible diaphragm must be checked
	Removal and replacement with lightweight	Heritage and weathering implications
Parapets (durability and weathering of particular concern)	Vertical steel mullions	Robust attachment to upper levels of brick with little wall/weight above critical Weathering through roof
	Raking braces	Robust attachment to upper levels of brick with little wall/weight above critical Interaction with roof modes can destabilise Vertical tie-down required to raking braces
	Steel capping spanning between abutting frames or walls	Anchorage depth down into mass of parapet to clamp down loose upper bricks
	Internal post-tensioning	Anchorage depth down into mass of parapet to clamp down loose upper bricks
	External post-tensioning	Anchorage depth down into mass of parapet to clamp down loose upper bricks
	Internal bonded reinforcement	Anchorage depth down into mass of parapet to clamp down loose upper bricks
	Near Surface Mounted (NSM) composite strips	Parapet responds differently to different directions of load UV degradation
Face-loaded walls	Vertical steel mullions (refer to Figure C8.23)	Stiffness vs out-of-plane rocking/displacement capability important Regularity/robustness of attachment to wall is important
	Vertical timber mullions	Stiffness vs out-of-plane rocking/displacement capability important Regularity/robustness of attachment to wall is important
	Horizontal transoms spanning between abutting frames or walls	Stiffness and attachment requirements need to consider wall above which gives clamping action to masonry at level of attachment
	Internal post-tensioning	Durability Anchorage level and fixity Level of pre-stress to allow rocking without brittle crushing
	External post-tensioning	As above
	Internal bonded reinforcement	Maximum quantity to ensure ductile failure Anchorage beyond cracking points, and consider short un-bonded lengths
	Composite fibre overlay	Preparation to give planar surface very involved
	Near Surface Mounted (NSM) composite strips	Wall responds differently to different directions of load Bond important if in-plane capacity is not to be weakened
	Reinforced concrete overlay	Wall responds differently to different directions of load

Structural mechanism	Technique	Comments/issues
	Reinforced cementitious overlay	Wall responds differently to different directions of load Ductility of reinforcement important for deflection capacity
	Grout saturation/injection	Elastic improvement only: more suitable for low seismic zones and very weak materials
Connection of walls to diaphragms	Steel angle with grouted bars (refer to Figure C8.24(a))	Bar anchorage Diaphragm/bar eccentricity must be resolved
	Steel angle with bolts/external plate (refer to Figure C8.24(b))	Diaphragm/bar eccentricity must be resolved
	Timber joist/ribbon plate with grouted bars	Bar anchorage Diaphragm/bolt eccentricity causes bending of timber across grain - a potential point of weakness
	Timber joist/ribbon plate with bolts/external plate	Diaphragm/bolt eccentricity causes bending of timber across grain - a potential point of weakness
	Blocking between joists notched into masonry	Joist weak axis bending must be checked Tightness of fit of joists into pockets Degradation of joists
	External pinning to timber beam end	Quality assurance/buildability of epoxy in timber Concentrated localised load Development in masonry (external plate preferred for high loads)
	External pinning to concrete beam or floor	Development in masonry (external plate preferred for high loads) Concrete floor type (hollow pots, clinker concrete)
	Through rods with external plates	Elastic elongation Concentrated localised load
	New isolated padstones	Tightness of fit Resolution of eccentricity between masonry bearing and diaphragm connection
	New bond beams	High degree of intervention
Diaphragm strengthening	Plywood overlay floor or roof sparking (refer to Figure C8.25)	Flexibility Requires continuous chord members and primary resistance elements
	Plywood ceiling	As above, plus existing ceiling battening/fixings may not be robust or may be decayed
	Plywood/light gauge steel composite	Stiffer but less ductile than ply-only Eccentricities between thin plate and connections must be resolved
	Plasterboard ceiling	As ply ceiling but less ductile Prevention of future modification/removal
	Thin concrete overlay/topping	Thickness for adequate reinforcement Additional mass Ductility capacity of non-traditional reinforcement Buckling restraint/bond to existing structure

Structural mechanism	Technique	Comments/issues
	Elastic cross bracing	Stiffness relative to wall out-of-plane capacity Edge distribution members and chords critical Concentration of loads at connections
	Semi-ductile cross bracing (e.g. Proving ring)	As elastic Energy absorption benefit not easily quantified without sophisticated analysis
	Replacement floor over/below with new diaphragm	Design as new structure
In-plane wall strengthening **New primary strengthening elements** (refer to Figure C8.26)	Sprayed concrete overlay	Restraint to existing floor/roof structure Out-of-plane capacity of wall Ductility capacity if used very dependent on aspect ratio Chords Foundation capacity needs to be checked (uplift/rocking)
	Internal vertical post-tensioning	Ensure pre-stress limited to ensure no brittle failure See out-of-plane issues also
	External vertical post-tensioning	Ensure pre-stress limited to ensure no brittle failure See out-of-plane issues also
	Internal horizontal reinforcement	Coring/drilling difficult Stressing horizontally requires good vertical (perpendicular) mortar placement and quality
	External horizontal post-tensioning	Stressing horizontally requires good vertical (perpendicular) mortar placement and quality
	Bed-joint reinforcement	Workmanship critical Low quantities of reinforcement only possible
	Composite reinforced concrete boundary or local reinforcement elements	Development at ends/nodes Bond to existing
	Composite FRP boundary or local reinforcement elements	As above plus stiffness compatibility with existing
	Nominally ductile concrete walls or punched wall/frame	High foundation loads result
	Nominally ductile reinforced concrete masonry walls	Stiffness compatibility considering geometry (including foundation movement) important
	Nominally ductile steel concentric or cross bracing	Stiffness compatibility assessment critical considering element flexibility, plan position and diaphragm stiffness Drag beams usually required
	Limited ductility steel moment frame	Flexibility/stiffness compatibility very important
	Limited ductility concrete frame	Flexibility/stiffness compatibility important

Structural mechanism	Technique	Comments/issues
	Limited ductility concrete walls	Assess effectiveness of ductility, including foundation movements Ensure compatibility with any elements cast against Drag beams often required
	Limited ductility timber walls	Flexibility/stiffness compatibility very important Drag beams often required
	Ductile EBF/K-frames	Element ductility demand vs building ductility assessment important Drag beams usually required
	Ductile concrete coupled or rocking walls	Element ductility demand vs building ductility assessment important Ensure compatibility with any elements cast against drag beams often required
	Tie to new adjacent (new) structure	Elastic elongation and robustness of ties to be considered Higher level of strengthening likely to be required
Reinforcement at wall intersections in plan	Removal and rebuilding of bricks with inter-bonding	Shear connection only with capacity reduced considering adhesion and tightness of fit Disturbance of bond to adjacent bricks
	Bed-joint ties	Small reinforcement only practical but can be well distributed Care with resolving resultant thrust at any bends
	Drilled and grouted ties	Tension only: consider shear capacity Depth to develop capacity typically large Compatibility with face-load spanning of wall
	Metalwork reinforcing internal corner	Attachment to masonry Small end-distance in abutting wall can mean negligible tension capacity
	Grouting of crack	Shear friction only: tension mechanism also required Stabilises any dilation but does not allow recovery
Foundation strengthening	Mass underpinning	Creates hard point in softer/swellable soils Even support critical
	Grout injection	Creates hard point in softer/swellable soils Difficult to quantify accurately
	Concentric/balanced re-piling	Localised “needles” through walls must provide sufficient bearing for masonry
	Eccentric re-piling with foundation beams	Stiffness of found beams important to not rotate walls out-of-plane
	Mini piling/ground anchors	Cyclic bond less than static bond Testing – only static practical Vulnerable to bucking if liquefaction
	Pile type: vertical stiffness and pre-loading	Pre-loading dictates load position Pre-loading important if new foundations less stiff than existing Dynamic distribution between new and old likely different than static

Structural mechanism	Technique	Comments/issues
		Effects of liquefaction must be considered: may create limiting upper bound to strengthening level
Façade wythe ties	Helical steel mechanical engagement – small diameter	Low tension capacity, especially if cracked
	Steel mechanical engagement – medium diameter	Some vierendeel action between wythes Durability
	Epoxied steel rods/gauze sleeve	Some vierendeel action between wythes
	Epoxied composite/non-metallic rods	Stiffness
	Brick header strengthening	Additional new headers still brittle: can become overstressed under thermal/seasonal or foundation loadings in combination
Canopies	Reinforce or recast existing hanger embedment	Degradation of steel Depth of embedment to ensure sufficient mass of bricks to prevent pull-out
	New steel/cast iron posts	Propping of canopy can mitigate hazard from masonry falling to pavement Props in addition to hangers are not so critical with regard to traffic damage
	New cantilevered beams	Co-ordination with clerestory/bressumer beam Backspan reaction on floor
	Deck reinforcement to mitigate overhead hazard	Sacrificial/crushable layer to mitigate pavement hazard
	Conversion to accessible balcony	Likely to achieve all of the above objectives for canopies and also has natural robustness as designed for additional live load. Hazard still exists for balcony occupants
Base isolation		A lack of sufficient gap around the building Vertically re-founding the building

C8.7 Material Properties and Weights

C8.7.1 General

This section provides default probable material properties for clay brick masonry and other associated materials.

These values can be used for assessment of URM buildings in the absence of a comprehensive testing programme (refer to Appendix C8A for details). However, to arrive at any reliable judgement, some on-site testing such as scratching, etc. as discussed in this section is recommended.

Note:

Before proceeding to on-site intrusive testing it is important to sensibly understand what information will be collected from any investigation, how this information would be used, and what value it would add to the reliability of the assessment. Sensitivity analyses can be used to determine the influence of any material parameter on the assessment outcome and therefore whether testing to refine that material parameter beyond the default values given in this section is warranted.

When assessing the material characteristics of the building, survey the entire building to ensure that the adopted material properties are representative. It may be appropriate to assign different material properties to different masonry walls depending on variations in age, weathered condition or other aspects.

C8.7.2 Clay bricks and mortars

Recommended probable default material properties for clay bricks and lime/cement mortar, correlated against hardness, are given in Tables C8.3 and C8.4. The descriptions in these tables are based on the use of a simple scratch test but there are a variety of similar, simple on-site tests the engineer can use.

To ensure that the test is representative of the structural capability of the materials, remove any weathered or remediated surface material prior to assessing the hardness characteristics. This requirement is particularly important for establishing mortar material properties where the surface mortar is either weathered or previously remediated and may not be representative of the mortar at depth. One recommended technique to establish whether the mortar condition is uniform across the wall thickness is to drill into the mortar joint and inspect the condition of the extracted mortar dust as the drill bit progresses through the joint.

Table C8.3: Probable strength parameters for clay bricks (Almesfer et al., 2014)

Brick hardness	Brick description	Probable brick compressive strength, f'_b (MPa)	Probable brick tensile strength, f_{bt} (MPa)
Soft	Scratches with aluminium pick	14	1.7
Medium	Scratches with 10 cent copper coin	26	3.1
Hard	Does not scratch with above tools	35	4.2

Table C8.4: Probable strength parameters for lime/cement mortar (Almesfer et al., 2014)

Mortar hardness	Mortar description	Probable mortar compressive strength, f'_j (MPa)	Probable Cohesion, c (MPa)	Probable coefficient of Friction, μ_f^{Ψ}
Very soft	Raked out by finger pressure	0-1	0.1	0.3
Soft	Scratches easily with fingernails	1-2	0.3	
Medium	Scratches with fingernails	2-5	0.5	0.6
Hard	Scratches using aluminium pick	To be established from testing	0.7	0.8
Very hard†	Does not scratch with above tools	To be established from testing		

Note:

† When very hard mortar is present it can be expected that walls subjected to in-plane loads and failing in diagonal shear will form diagonal cracks passing through the bricks rather than a stair-stepped crack pattern through the mortar head and bed joints. Such a failure mode is non-ductile. Very hard mortar typically contains cement.

Ψ Values higher than 0.6 may be considered with care/investigation depending upon the nature/roughness of the brick material and the thickness of the mortar with respect to the brick roughness.

Values for adhesion may be taken as half the cohesion values provided in Table C8.4.

In cases where the probable modulus of rupture of clay bricks cannot be established from testing, the following value may be used (Almesfer et al., 2014):

$$f'_r(\text{MPa}) = 0.12 f'_b \qquad \ldots\text{C8.1}$$

C8.7.3 Compressive strength of masonry

In cases where the compressive strength of masonry cannot be established from the testing of extracted masonry prisms, the probable masonry compressive strength, f'_m, can be established using Equation C8.2 (Lumantarna et al., 2014b). Table C8.5 presents probable compressive strength values of clay brick masonry based on this equation using the brick and mortar probable compressive strength values from Tables C8.3 and C8.4.

$$f'_m(\text{MPa}) = \begin{cases} 0.75 {f'_b}^{0.75} \text{x} \, {f'_j}^{0.3} & \text{for } f'_j \geq 1 \text{ MPa} \\ 0.75 {f'_b}^{0.75} & \text{for } f'_j < 1 \text{ MPa} \end{cases} \qquad \ldots\text{C8.2}$$

Table C8.5: Probable compressive strength of clay brick masonry, f'_m

Mortar strength, f'_j (MPa)	Probable brick compressive strength, f'_b (MPa)		
	14	26	35
0	5.4	8.6	10.8
1	5.4	8.6	10.8
2	6.7	10.6	13.3
5	8.8	14.0	17.5
8	10.1	16.1	20.1

C8.7.4 Tensile strength of masonry

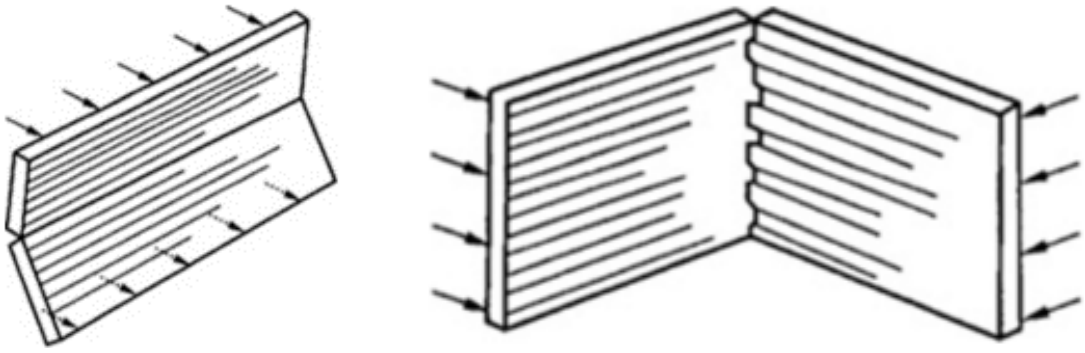
The tensile strength of masonry in both horizontal and vertical directions, including any cement rendering and plaster, should be assumed to be zero for walls that can be subjected to face load, except when the requirements given in Section C8.8.5.2 for elastic analysis are satisfied for vertical spanning face-loaded walls.

When assessing the tensile strength of spandrels refer to Section C8.8.6.3.

Note:

When the requirements of Section C8.8.5.2 are met values of 0.2 and 0.4 MPa would seem appropriate for f'_{t} when the failure plane is parallel and perpendicular to the bed joints respectively (refer to Figure C8.57). Where there is a high likelihood of low adhesion between the masonry units and the mortar (e.g. when lime mortar has leached), zero tensile strength of masonry should be assumed.

These values should be used for assessing the probable capacity of elements/members whenever tension develops in the masonry.



(a) Plane of failure parallel to bed joint

(b) Plane of failure perpendicular to bed joint

Figure C8.57: Tensile failure planes

C8.7.5 Diagonal tensile strength of masonry

Where specific material testing is not undertaken to determine probable masonry diagonal tension strength, this may be taken as:

$$f_{\mathrm{dt}}(\mathrm{MPa}) = 0.5c + f_{\mathrm{a}}\mu_{\mathrm{f}} \qquad \ldots\text{C8.3}$$

where:

c	=	masonry bed-joint cohesion
μ_{f}	=	masonry co-efficient of friction
f_{a}	=	axial compression stress due to gravity loads calculated at the mid height of the wall/pier (MPa).

C8.7.6 Modulus of elasticity and shear modulus of masonry

The masonry modulus of elasticity, E_{m}, can be calculated by using the masonry probable compressive strength in accordance with Equation C8.4 (Lumantarna et al., 2014b). Note that this value of modulus of elasticity has been established as a chord modulus of elasticity between $0.05f'_{\mathrm{m}}$ and $0.7f'_{\mathrm{m}}$ in order to represent the elastic stiffness appropriate up to maximum strength.

Young's modulus of clay brick masonry can be taken as:

$$E_{\mathrm{m}}(MPa) = 300f'_{\mathrm{m}} \quad \ldots\text{C8.4}$$

Shear modulus of clay brick masonry can be taken as (ASCE 41-13, 2014):

$$G_{\mathrm{m}}(MPa) = 0.4\,E_{\mathrm{m}} \quad \ldots\text{C8.5}$$

C8.7.7 Timber diaphragm material properties

Refer to Section C9 for timber diaphragm material properties.

C8.7.8 Material unit weights

The engineer can use the unit weights given in Table C8.6 as default values if more reliable measurements are not available.

Table C8.6: Unit weights

Material	Unit weight (kN/m^3)
Brick masonry	18
Oamaru stone masonry	16
Timber	5-6

C8.8 Assessment of Member/Element Capacity

C8.8.1 General

This section covers the assessment of the capacity of the various members and elements that make up a masonry building.

In the displacement-based procedure for face-loaded walls that is presented, the assessment of the demand is an integral part of the procedure.

C8.8.2 Strength reduction factors

The assessment procedures in these guidelines are based on probable strengths and, therefore, the strength reduction factor, ϕ, should be set equal to 1.0. The probable strength equations and recommended default probable capacities in this section assume ϕ equals 1.0.

C8.8.3 Diaphragms

C8.8.3.1 General

Diaphragms in URM buildings fulfil two principal functions. They provide support to the walls oriented perpendicular to the direction of loading. Also, if they are stiff enough, they have the potential to allow shears to be transferred between walls in any level, to resist the storey shear and the torsion due to any plan eccentricities.

The relative lateral stiffness of the diaphragms to the walls providing lateral support is often quite low due to the high stiffness of the walls, particularly for diaphragms constructed of timber or steel bracing.

Flexibility in a diaphragm, if too high, can reduce its ability to provide adequate support to walls and thus affect the response of these walls, or render its ability to transfer storey shears to minimal levels, although this will not generally be an issue if recognised and appropriately allowed for in the global analysis of the building. Therefore, considering the effects of diaphragm flexibility is essential for proper understanding of both in-plane and out-of-plane response of the walls.

When assessing the capacity of diaphragms it is necessary to consider both their probable strength and deformation capacities.

The probable strength capacity should be determined in accordance with the requirements in these guidelines that relate to the particular construction material of the diaphragm.

The deformation capacity will be that for which the strength capacity can be sustained.

The deformation capacity is also limited to that which it is expected will result in detrimental behaviour of supported walls or of the building as a whole.

The diaphragm deformations should be included when determining the inter-storey deflections for checking overall building deformations against the NZS 1170.5:2004 limit of 2.5%.

In the sections below recommendations are provided for diaphragm deformation limits to ensure adequate support for face-loaded walls and for flexible (timber) and rigid diaphragms. Rigid diaphragms would typically need to be constructed of concrete to achieve the necessary relative stiffness with the walls.

C8.8.3.2 Diaphragm deformation limits to provide adequate support to face-loaded walls

In order to ensure that the face-loaded walls are adequately supported, the maximum diaphragm in-plane displacement measured with respect to the diaphragm support walls should not exceed 50% of the thickness of the supported (face-loaded) walls (refer to Figure C8.58). For cavity construction with adequate cavity ties installed, the inner masonry wythe is usually the load-bearing wythe and this criterion will require the maximum acceptable diaphragm displacement to be limited to 50% of the thickness of the inner wythe.

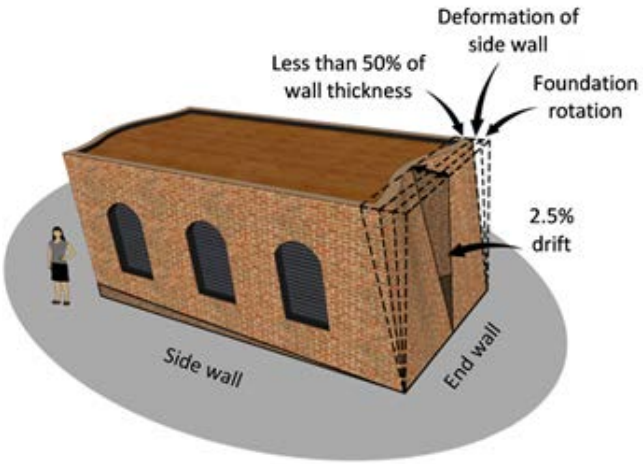


Figure C8.58: Mid-span diaphragm displacement limit for URM building on a flexible foundation

C8.8.3.3 Timber diaphragms

General

Most URM buildings in New Zealand have flexible timber floor and ceiling diaphragms. Their in-plane deformation response is strongly influenced by the characteristics of the nail connections (Wilson et al., 2013a) and their global response is most adequately replicated as a shear beam (Wilson et al., 2013b). Responses can be separated into directions either parallel or perpendicular to the orientation of the joists (Wilson et al., 2013c), as illustrated in Figure C8.59. They are significantly influenced by the presence of any floor or ceiling overlay, the degradation of the diaphragm due to aspects such as moisture or insect damage, and any prior remediation such as re-nailing or varnishing (Giongo et al., 2013). If the diaphragms have had epoxy coatings that have penetrated into the joints between the flooring, this has been observed to result in substantial stiffening. Therefore, these guidelines recommend undertaking a sensitivity analysis, recognising that the effective diaphragm stiffness could be more than given here by an order of magnitude or greater.

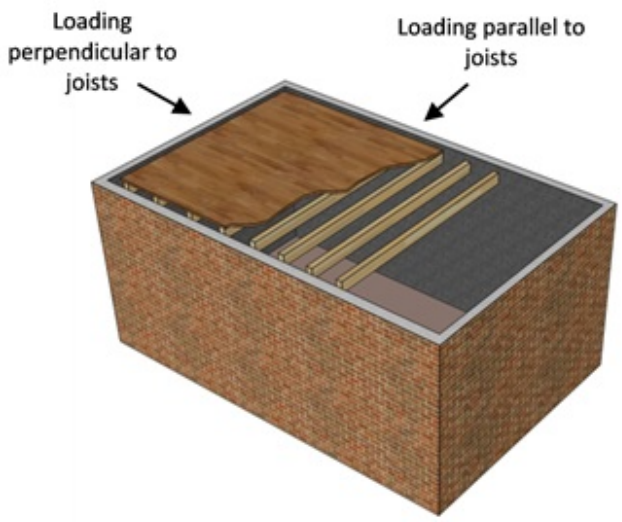


Figure C8.59: Orthogonal diaphragm response due to joist orientation

It is assumed here that the diaphragm is adequately secured to all perimeter walls via pocketing and/or anchorages to ensure that diaphragm deformation occurs rather than global sliding of the diaphragm on a ledge. It is also assumed that the URM boundary walls deform out-of-plane in collaboration with deformation of the flexible timber diaphragm. For non-rectangular diaphragms, use the mean dimensions of the two opposing edges of the diaphragm to establish the appropriate dimensions of an equivalent rectangular diaphragm.

Note:

Timber roofs of URM buildings were often built with both a roof and ceiling lining. As a result, roof diaphragms are likely to be significantly stiffer than the mid height floor diaphragms if there are no ceilings on the mid-floors. Diagonal sarking in the roof diaphragm will also further increase its relative stiffness compared to the floor diaphragms.

If the diaphragm being assessed has an overlay or underlay (e.g. of plywood or pressed metal sheeting), consult the stiffness and strength criteria for improved diaphragms. The engineer will still need to consider stiffness and ductility compatibility between the two. For example, it is likely that a stiff, brittle timber lath-and-plaster ceiling will delaminate before any straight sarking in the roof above can be fully mobilised.

While the flooring, sarking and sheathing provide a shear load path across the diaphragm, it is necessary to consider the connections to the surrounding walls (refer to Section C8.8.4) and any drag or chord members. A solid URM wall may be able to act as a chord as it has sufficient in-plane capacity to transfer the chord loads directly to the ground. However, a punched URM wall with lintels only over the openings will have little tension capacity and may be the critical element in the assessment. Timber trusses and purlins, by their nature, only occur in finite lengths: their connections/splices designed for gravity loads may have little tie capacity.

Probable strength capacity

The probable strength capacity of a timber diaphragm should be assessed in accordance with Section C9 of these guidelines.

Probable deformation capacity

Deformations in timber diaphragms should be assessed using the effective diaphragm stiffness defined below.

The probable deformation capacity should be taken as the lower of the following, assessed for each direction:

- L/33 for loading oriented perpendicular to the joists or L/53 for loading oriented parallel to the joists
- deformation limit to provide adequate support to face-loaded walls. Refer Section C8.8.3.2.
- deformation required to meet global inter-storey drift limit of 2.5% in accordance with NZS 1170.5:2004. Refer Section C8.8.3.1.

Effective diaphragm stiffness

To determine the effective stiffness of a timber diaphragm, first assess the condition of the diaphragm using the information in Table C8.7.

Table C8.7: Diaphragm condition assessment criteria (Giongo et al., 2014)

Condition rating	Condition description
Poor	Considerable borer; floorboard separation greater than 3 mm; water damage evident; nail rust extensive; significant timber degradation surrounding nails; floorboard joist connection appears loose and able to wobble
Fair	Little or no borer; less than 3 mm of floorboard separation; little or no signs of past water damage; some nail rust but integrity still fair; floorboard-to-joist connection has some but little movement; small degree of timber wear surrounding nails
Good	Timber free of borer; little separation of floorboards; no signs of past water damage; little or no nail rust; floorboard-to-joist connection tight, coherent and unable to wobble

Next, select the diaphragm stiffness using Table C8.8 and accounting for both loading orientations.

Note:

While other diaphragm characteristics such as timber species, floor board width and thickness, and joist spacing and depth are known to influence diaphragm stiffness, their effects on stiffness can be neglected for the purposes of this assessment.

Pre-testing has indicated that re-nailing vintage timber floors using modern nail guns can provide a 20% increase in stiffness.

Table C8.8: Shear stiffness values† for straight sheathed vintage flexible timber floor diaphragms (Giongo et al., 2014)

Direction of loading	Joist continuity	Condition rating	Shear stiffness†, G_d (kN/m)
Parallel to joists	Continuous or discontinuous joists	Good	350
		Fair	285
		Poor	225
Perpendicular to joists††	Continuous joists, or discontinuous joists with reliable mechanical anchorage	Good	265
		Fair	215
		Poor	170
	Discontinuous joists without reliable mechanical anchorage	Good	210
		Fair	170
		Poor	135

Note:

† Values may be amplified by 20% when the diaphragm has been renailed using modern nails and nail guns

†† Values should be interpolated when there is mixed continuity of joists or to account for continuous sheathing at joist splice

For diaphragms constructed using other than straight sheathing, multiply the diaphragm stiffness by the values given in Table C8.9. If roof linings and ceiling linings are both assumed to be effective in providing stiffness, add their contributions.

Table C8.9: Stiffness multipliers for other forms of flexible timber diaphragms (derived from ASCE 41-13, 2014)

Type of diaphragm sheathing		Multipliers to account for other sheathing types
Single straight sheathing		x 1.0
Double straight sheathing	Chorded	x 7.5
	Unchorded	x 3.5
Single diagonal sheathing	Chorded	x 4.0
	Unchorded	x 2.0
Double diagonal sheathing or straight sheathing above diagonal sheathing	Chorded	x 9.0
	Unchorded	x 4.5

For typically-sized diaphragm penetrations (usually less than 10% of gross area) the reduced diaphragm shear stiffness, G'_d, is given by Equation C8.6:

$$G'_d(kN/m) = \frac{A_{net}}{A_{gross}} G_d \quad \text{…C8.6}$$

where A_{net} and A_{gross} refer to the net and the gross diaphragm plan area respectively (in square metres).

For non-typical sizes of diaphragm penetration, a special study should be undertaken to determine the influence of diaphragm penetration on diaphragm stiffness and strength.

The effective diaphragm stiffness should be modified further to account for stiffness of the URM boundary walls deforming in collaboration with the flexible timber diaphragm.

Hence:

$$G'_{\mathrm{d,eff}}(\mathrm{kN/m}) = \alpha_{\mathrm{w}} G'_{\mathrm{d}} \qquad \ldots\text{C8.7}$$

where α_{w} may be determined using any rational procedure to account for the stiffness and incompatibility of deformation modes arising from collaborative deformation of the URM walls displacing out-of-plane as fixed end flexure beams and the diaphragm deforming as a shear beam.

In lieu of a special study, prior elastic analysis has suggested that Equation C8.8 provides adequate values for α_{w}:

$$\alpha_{\mathrm{w}} \cong 1 + \left({t_{\ell}^3}/{H_{\ell}^3} + {t_{\mathrm{u}}^3}/{H_{\mathrm{u}}^3} \right) {L^2 E_{\mathrm{m}}}/{B G'_{\mathrm{d}}} \qquad \ldots\text{C8.8}$$

where:

t_{ℓ}	=	effective thickness of walls below the diaphragm, m
t_{u}	=	effective thickness of walls above the diaphragm, m
H_{ℓ}	=	height of wall below diaphragm, m
H_{u}	=	height of wall above diaphragm, m
E_{m}	=	Young's modulus of masonry, MPa
B	=	depth of diaphragm, m
L	=	span of diaphragm perpendicular to loading, m.
G'_{d}	=	reduced diaphragm shear stiffness, kN/m

Refer to Figure C8.60 for definition of the above terms.

For scenarios where the URM end walls are likely to provide no supplementary stiffness to the diaphragm, $\alpha_{\mathrm{w}} = 1.0$ should be adopted.

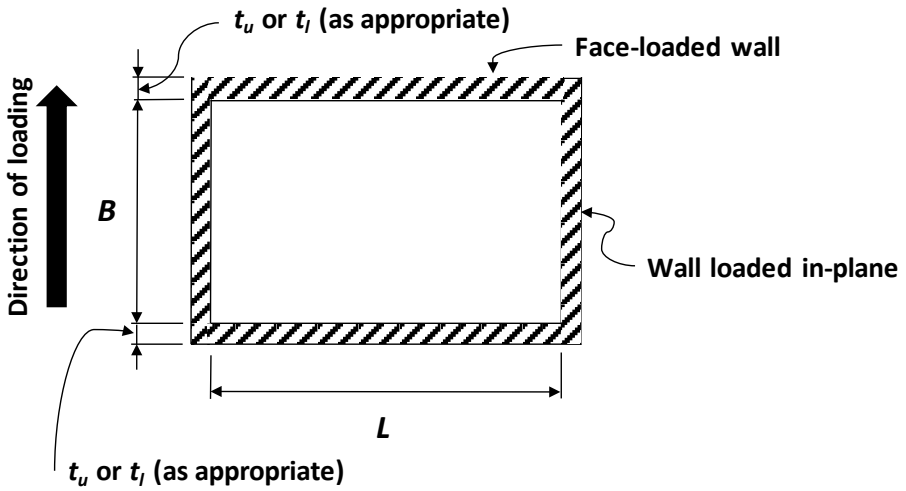


Figure C8.60: Schematics showing dimensions of diaphragm

C8.8.3.4 Rigid diaphragms

When considering rigid diaphragms, the engineer can use a "strut-and-tie" method. However, investigate the presence of termination details (hooks, thickenings, threads/nuts) carefully as their ability to transfer the loads at the strut-and-tie nodes is likely to govern the diaphragm capacity.

Rigid diaphragms can be assumed to have minimal effect on the response of out-of-plane walls.

C8.8.4 Connections

C8.8.4.1 General

The probable capacity of diaphragm to wall connections is taken as the lowest probable capacity of the failure modes listed below:

- punching shear failure of masonry
- yield or rupture of connector rod in tension or shear
- rupture at join between connector rod and joist plate
- splitting of joist or stringer
- failure of fixing at joist
- splitting or fracture of anchor plate
- yield or rupture at threaded nut.

Suggested default probable capacities for embedded and plate bearing anchors are provided below. Guidance on specific assessment of capacities is also provided.

C8.8.4.2 Embedded anchors

The engineer can use the probable capacities provided in Tables C8.10 and C8.11 in lieu of specific testing provided that:

- the capacity should not be taken greater than the probable capacities of the anchor itself or the anchor to grout or grout to brick bond
- when the embedment length is less than four bolt diameters or 50 mm, the pull-out strength should be taken as zero
- the minimum edge distance to allow full shear strength to be assumed should be 12 diameters
- shear strength of anchors with edge distances equal to or less than 25 mm should be taken as zero.

Linear interpolation of shear strength for edge distances between these bounds is permitted (ASCE 41-13, 2014).

Simultaneous application of shear and tension loads need not be considered when using the values from Tables C8.10 and C8.11.

Table C8.10: Default anchor probable shear strength capacities for anchors into masonry units only[1]

Anchorage type	Rod size	Probable shear strength capacity[2] (kN)
Bolts/steel rods fixed through and bearing against a timber member[1,2]	M12	8.5
	M16	15
	M20	18.5
Bolts/steel rods fixed through a steel member (washer) having a thickness of 6 mm or greater	M16	20

Note:

1. Anchors into mortar bed joints will have significantly lower shear capacities
2. Timber member to be at least 50 mm thick and MSG8 grade or better
3. For adhesive connectors embedment should be at least 200 mm into solid masonry

The values in Table C8.11 are based on the pull-out of a region of brick, assuming cohesion or adhesion strength of the mortar on the faces of the bricks perpendicular to the application of the load factored by 0.5 and friction on the top and/or bottom faces (refer to Figure C8.61), depending on the height of wall above the embedment as follows:

- 0 m (i.e. at the top of the wall) – adhesion only on the bottom and side faces.
- ≥0.3 m but < 3 m – adhesion on the top, bottom and side faces, friction on the top and bottom faces.
- ≥3 m – cohesion on the top, bottom and side faces, friction on the top and bottom faces.

A factor of 0.5 has been included in these values to reflect the general reliability of mechanisms involving cohesion/adhesion and friction.

Table C8.11: Default anchor probable tension pull-out capacities for 0 m, ≥0.3 m and ≥ 3 m of wall above the embedment[2]

Mortar hardness	Single-wythe wall (kN)			Embedment 160 mm[1] into two-wythe wall (kN)			Embedment 250 mm[1] into three-wythe wall (kN)		
	0	≥0.3 m[(3)]	≥3 m	0	≥0.3 m[(3)]	≥3 m	0	≥0.3 m[(3)]	≥3 m
Very soft	0.3	0.5	1	1	1.5	4	1.5	3	8
Soft	1	1.5	3	2.5	4	9	5	8	18
Medium	1.5	2.5	6	4	6.5	15	8	14	31
Hard	2.5	3.5	8	6	9	21	11	19	43
Very hard	>2.5[(4)]	>4[(4)]	>8[(4)]	>6[(4)]	>10[(4)]	>21	>11[(4)]	>20[(4)]	>43[(4)]

Notes:

1. Representative value only: assumes drilling within 50 mm of far face of wall.
2. Simultaneous application of tension and shear loading need not be considered.
3. These values are intended to be used until there is ≥3 m of wall above the embedment.
4. Values for very hard mortar may be substantiated by calculation but can be assumed to be at least those shown.

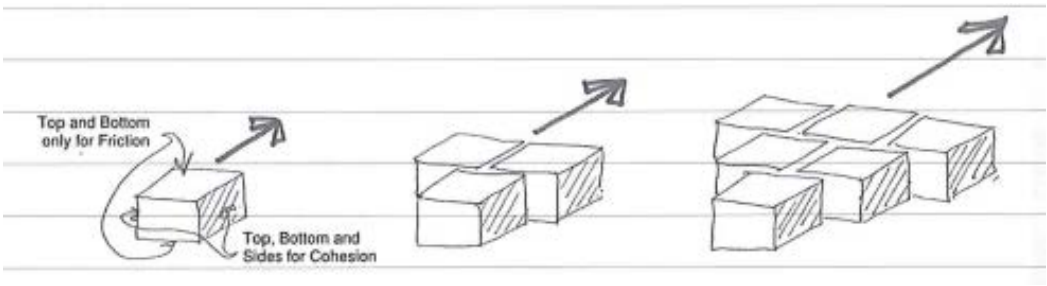


Figure C8.61: Basis for embedded anchor capacity estimation

The designer should select a bar diameter and tested epoxy system that will develop the required bond directly to the bricks and grout system as appropriate. Alternatively, cement mortars can be used but the capacity should be substantiated by site pull-out tests, using the grouting and cleanout methodology proposed by relevant standards/specifications.

For coarse thread screws, use the manufacturer's data for the direct bond to bricks, taking account of the brick compressive strength and ensuring that fixings are into whole bricks rather than mortar courses.

When assessing the capacity of straight or bent adhesive anchors, refer to the product specification and the methodology prescribed by the anchor manufacturer.

For inclined embedded anchors, the horizontal force capacity should be reduced to the horizontal vector component, and checks made for an adequate load path for the vertical component. If the inclination is less than 22.5 degrees these effects can be considered insignificant and the full capacity of the anchor can be assumed.

C8.8.4.3 Plate anchors

For plate anchors, postulate the potential failure surface to estimate its capacity.

A wall punching shear model is shown in Figure C8.62.

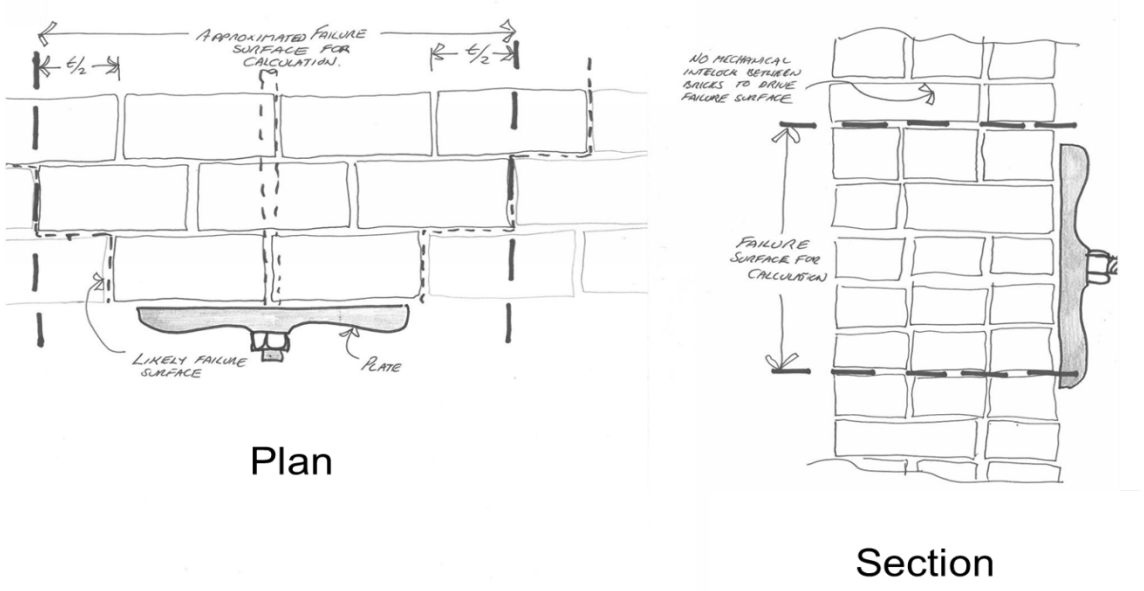


Figure C8.62: Failure surfaces for plate anchors

C8.8.4.4 Capacity of wall between connections

Where the lateral spacing of connections used to resist the wall anchorage force is greater than four times the wall thickness, measured along the length of the wall, check the section of wall spanning between the anchors to resist the local out-of-plane bending caused by the lateral force (FEMA P-750, 2009). This check might be undertaken allowing for arching in the masonry; for example, through the compressive membrane forces that develop when a conical "yield line" pattern develops in the brick around the anchor.

For most applications involving bearing plates, it should be sufficiently accurate to assume a cylinder with a cross section the same shape as the bearing plate but lying outside it all round by half the thickness of the wall. Cohesion may be considered to be acting on the sides of this cylinder.

C8.8.5 Wall elements under face load

C8.8.5.1 General

This section provides both force-based (assuming elastic behaviour) and displacement-based inelastic methods for assessing face-loaded walls. The force-based methods utilising the direct tensile capacity of the masonry are only appropriate if all of the criteria listed in Section C8.8.5.2 are met.

Note:

The procedures in some earlier versions of these guidelines (such as the 1995 "Red Book" (NZSEE, 1995)) that were based on the concept of equating total energy (strain energy of deformation plus potential energy due to shifts of weights) of the rocking wall to that for an elastic oscillator have since been shown to be deficient. These procedures give inconsistent results and are potentially unsafe; particularly where walls are physically hinged at floor levels (i.e. when they are supported on a torsionally flexible beam with no wall underneath) or made of stiff (high modulus of elasticity) masonry.

This update uses the same formulations as the 2006 guidelines (NZSEE, 2006) but accommodates some of the more significant recent research findings. These are based on work carried out at the University of Auckland and University of Adelaide (Derakhshan et al., 2013a and 2013b; Derakhshan et al., 2014a and 2014b). However, these guidelines do not include all of the detailed procedures set out in this research (Derakhshan et al., 2014a) as there were some simplifying assumptions that made these procedures less suitable for thicker walls.

Procedures given for assessing face-loaded walls spanning one-way horizontally, or two-way horizontally and vertically, are based on response assuming only weak nonlinear effects (i.e. assumption of elastic or nominally elastic response). These are based on less rigorous research and are not as well developed as procedures for walls spanning vertically. Caution is therefore required when using these recommendations.

Further research has been carried out in this area and more comprehensive procedures are likely to be included in the next update of these guidelines.

For walls spanning vertically in one direction between a floor and another floor or the roof, or vertically cantilevered (as in partitions and parapets), assure the lateral restraint of the floors and the roof for all such walls. If this restraint cannot be assured, the methods presented here for one-way vertically spanning walls cannot be used. However, it might still be possible to assess such walls by analysing them as spanning horizontally between other walls, columns or other elements, or as two-way assemblages.

Multi-wythe walls can be considered as one integral unit for face-loading if:

- all wythes are interconnected with header courses at least every fourth course and regularly along the length of the wall, or
- testing or special study has confirmed that the wythes are capable of acting as integral units.

Otherwise, consider each wythe as acting independently.

Header courses are typically provided every four to six courses in common bond. This would normally suffice for walls loaded out-of-plane (but note the caution raised above). These header courses would normally pass through the whole wall, with bricks lapping in the interior as required. For example, in triple brick walls the header course on the inside will be either one brick higher or lower than the header course on the outside to allow lapping over the central wythe.

If the above criterion is not met, investigate the sufficiency of the available header course by assuming a vertical shear acting on the centreline of the lower wall equal to $P + W_{\mathrm{t}} + 0.5W_{\mathrm{b}}$. This shear needs to be resisted by header bricks crossing the centreline. For this purpose, the engineer can assume each header brick contributes a shear resistance of $2f_{\mathrm{r}}bt^2/l$, where b, t and l are the breadth, depth and length of the header and f_{r} is its modulus of rupture of brick in bending.

If a wythe is not integral with the main structural wall, assume the wythe wall piggybacks the backing wall. If both wythes are one brick (110 mm) thick, the engineer can assume they carry their own load independently for out-of-plane checks.

Non-structural masonry (usually single-wythe partitions, acoustic linings or fire linings) should be considered as a mass within the building and the risks for face-load collapse evaluated.

Internal walls with floors on both sides can be assumed to be supported at floor levels but checks on the diaphragms (strength and deformation) and perpendicular walls will still be required.

Walls should be assessed in every storey and for both directions of response (inwards and outwards). Set the rating of the wall at the least value found, as failure in any one storey for either direction of loading will lead to progressive failure of the whole wall.

C8.8.5.2 Vertical spanning walls

General

When using an elastic analysis to determine the capacity of a wall section, the direct tensile strength of the masonry should be ignored unless:

- the capacity so determined is halved and the available ductility is assumed to be 1
- an inspection of the wall reveals no signs of cracking at that section, and
- the in-plane calculations indicate cracking of the brickwork is not expected.

If a displacement-based approach is adopted, the maximum out-of-plane displacement should be limited to 0.6 times the instability displacement for simply supported walls and 0.3 times the instability displacement for cantilever walls; e.g. parapets.

In the case of walls supported against face load, deflection of the supports will need to meet minimum requirements to ensure the walls can respond as assumed. In these guidelines, limits on the deflection of diaphragms are considered a diaphragm capacity issue and are defined in Section C8.8.3.2. These deflection limits should also apply to any other supports to face-loaded walls; for example, the support that may be provided by steel portal or steel bracing retrofits.

Elastic analysis

A simple bending analysis may be performed for the seismic assessment of face-loaded walls using Equation C8.9 provided that the criteria given in Section C8.8.5.2 are met. Equation C8.9 is applicable for a unit wall length.

$$M = \frac{t_{\text{nom}}^2}{6}\left(f_{\text{t}}' + \frac{P}{A_{\text{n}}}\right) \qquad \text{…C8.9}$$

where:

P = load applied to top of panel (N)
A_{n} = net plan area of masonry (mm^2)
M = moment capacity of the panel (Nmm)
t_{nom} = nominal thickness of wall excluding pointing (mm)

$$t_{\text{nom}} = t_{\text{gross}} - np \qquad \text{…C8.10}$$

where:

f_{t}' = probable tensile strength
p = depth of mortar recess (in mm) as shown in Figure C8.63
t_{gross} = overall thickness of wall (in mm)
n = number of recesses.

n = 2 if recesses are provided on both sides; n = 1 otherwise.

If the recess is less than 6 mm, it can be ignored.

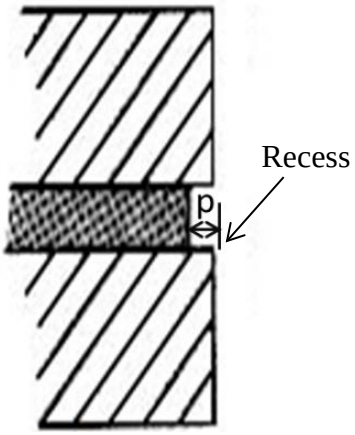


Figure C8.63: Pointing with recess

The imposed moment may be assumed critical at:

- mid height of walls restrained at the top and bottom, or
- at the base of cantilever walls.

The direct tensile strength, f'_t, should be ignored in capacity calculations unless there is no sign of pre-cracking in the wall at the section being considered and cracking of the brickwork in the region of the section is not expected for loading in-plane and the strength capacity calculated assuming tensile strength has been factored by 0.5. The ductile capability of the should be taken as 1.

Inelastic displacement-based analysis for walls spanning vertically between supports

Follow the steps below to assess the displacement response capability and displacement demand in order to determine the adequacy of the walls.

Note:

Appendix C8B provides some guidance on methods for determining key parameters. Refer to Figure C8B.1 for the notation employed.

Some approximations have been provided which can be used (these are listed after these steps) if wall panels are uniform within a storey (approximately rectangular in vertical and horizontal section and without openings).

Charts are provided in Appendix C8C that allow assessment of *%NBS* for regular walls (vertically spanning and vertical cantilever) in terms of height to thickness ratio of the wall, gravity load on the wall, and parameters defining the demand on the wall.

The wall panel is assumed to form hinge lines at the points where effective horizontal restraint is assumed to be applied. The centre of compression on each of these hinge lines is assumed to form a pivot point. The height between these pivot points is the effective panel height h (in mm). At mid height between these pivots, height $h/2$ from either, a third pivot point is assumed to form.

The recommended steps for assessment of walls following the displacement-based method are discussed below:

Step 1

Divide the wall panel into two parts: a top part bounded by the upper pivot and the mid height between the top and bottom pivots; and a bottom part bounded by the mid height pivot and the bottom pivot.

Note:

This division into two parts is based on the assumption that a significant crack will form at the mid height of the wall, where an effective hinge will form. The two parts are then assumed to remain effectively rigid. While this assumption is not always correct, the errors introduced by the resulting approximations are not significant.

One example is that significant deformation occurs in the upper part of top-storey walls. In particular, where the tensile strength of the mortar is small the third hinge will not necessarily form at the mid height.

Step 2

Calculate the weight of the wall parts: W_{b} (in N) of the bottom part and W_{t} (in N) of the top part, and the weight acting at the top of the storey, P (in N).

Note:

The weight of the wall should include any render and linings, but these should not be included in t_{nom} or t (in mm) unless the renderings are integral with the wall. The weight acting on the top of the wall should include all roofs, floors (including partitions and ceilings and the seismic live load) and other features that are tributary to the wall.

Step 3

From the nominal thickness of the wall, t_{nom}, calculate the effective thickness, t.

Note:

The effective thickness is the actual thickness minus the depth of the equivalent rectangular stress block. The reduction in thickness is intended to reflect that the walls will not rock about their edge but about the centre of the compressive stress block.

The depth of the equivalent rectangular stress block should be calculated with caution, as the depth determined for static loads may increase under earthquake excitation. Appendix C8B suggests a reasonable value based on experiments, $t = t_{\mathrm{nom}}$ (0.975-0.025 P/W). The thickness calculated by this formula may be assumed to apply to any type of mortar, provided it is cohesive. For weaker (and softer) mortars, greater damping will compensate for any error in the calculated t.

Step 4

Assess the maximum distance, e_{p}, from the centroid of the top part of the wall to the line of action of P. Refer to Figure C8B.1 for definition of e_{b}, e_{t} and e_{o}. Usually, the eccentricities e_{b} and e_{p} will each vary between 0 and $t/2$ (where t is the effective thickness of the wall). Exceptionally they may be negative; i.e. where P promotes instability due to its placement.

When considering the restraint available from walls on foundations assume the foundation is the same width as the wall and use the following values for e_b:

0	if the factor of safety for bearing under the foundation, for dead load only (FOS), is equal to 1
$t/3$	if FOS = 3 (commonly the case)
$t/4$	if FOS = 2.

Note:

Figure C8B.2 shows the positive directions for the eccentricities for the assumed direction of rotation (angle A at the bottom of the wall is positive for anti-clockwise rotation).

The walls do not need to be rigidly attached or continuous with a very stiff section of wall beyond to qualify for an assumption of full flexural restraint.

Care should be taken not to assign the full value of eccentricity at the bottom of the wall if the foundations are indifferent and may themselves rock at moments less than those causing rocking in the wall. In this case, the wall might be considered to extend down to the supporting soil where a cautious appraisal should then establish the eccentricity. The eccentricity is then related to the centroid of the lower block in the usual way.

Step 5

Calculate the mid height deflection, Δ_i, that would cause instability under static conditions. The following formula may be used to calculate this deflection.

$$\Delta_i = \frac{bh}{2a} \quad \text{…C8.11}$$

where:

$$b = W_b e_b + W_t(e_o + e_b + e_t) + P\left(e_o + e_b + e_t + e_p\right) - \Psi(W_b y_b + W_t y_t) \quad \text{…C8.12}$$

and:

$$a = W_b y_b + W_t(h - y_t) + Ph \quad \text{…C8.13}$$

Note:

The deflection that would cause instability in the walls is most directly determined from virtual work expressions, as noted in Appendix C8B.

Step 6

Assign the maximum usable deflection, Δ_m (in mm), as 0.6 Δ_i.

Note:

The lower value of the deflection for calculation of instability limits reflects that response predictions become difficult as the theoretical limit is approached. In particular, the response becomes overly dependent on the characteristics of the earthquake, and minor perturbances lead quickly to instability and collapse.

Step 7

Calculate the period of the wall, T_p, as four times the duration for the wall to return from a displaced position measured by Δ_t (in mm) to the vertical. The value of Δ_t is less than Δ_m. Research indicates that $\Delta_t = 0.6\Delta_m = 0.36\Delta_i$ for the calculation of an effective period for use in an analysis using a linear response spectrum provides a close approximation to the results of more detailed methods. The period may be calculated from the following equation:

$$T_p = 4.07\sqrt{\frac{J}{a}} \qquad \text{…C8.14}$$

where J is the rotational inertia of the masses associated with W_b, W_t and P and any ancillary masses, and is given by the following equation:

$$J = J_{bo} + J_{to} + \frac{1}{g}\left\{W_b[e_b^2 + y_b^2] + W_t[(e_o + e_b + e_t)^2 + y_t^2] + P\left[\left(e_o + e_b + e_t + e_p\right)^2\right]\right\} + J_{anc} \qquad \text{…C8.15}$$

where J_{bo} and J_{to} are mass moment of inertia of the bottom and top parts about their centroids, and J_{anc} is the inertia of any ancillary masses, such as veneers, that are not integral with the wall but that contribute to the inertia.

When treating cavity walls, make the following provisions:

- When the veneer is much thinner than the main wythe, the veneer can be treated as an appendage. For inelastic analysis, the veneers can be accounted through J_{anc}.
- If both wythes are one brick (110 mm) thick, then these could be treated as independent walls. Allocate appropriate proportion of overburden on them and solve the problem in the usual way.
- Where an accurate solution is the objective, solve the general problem with the kinematic constraint that the two walls deflect the same.

Note:

The equations are derived in Appendix C8B. The method in this Appendix can be used to assess less common configurations as necessary.

Step 8

Calculate the design response coefficient $C_p(T_p)$ in accordance with Section 8 of NZS 1170.5:2004 taking $\mu_p = 1$ and substituting $C_i(T_p)$:

$$C_i(T_p) = C_{hc}(T_p) \qquad \text{…C8.16}$$

where:

$C_{hc}(T_p)$ = the spectral shape factor ordinate, $C_h(T_p)$, from NZS 1170.5:2004 for Ground Class C and period T_p provided that, solely for the purpose of calculating $C_{hc}(T_p)$, T_p need not be taken less than 0.5 sec.

When calculating C_{Hi} from NZS 1170.5:2004 for walls spanning vertically and held at the top, h_i should be taken as the average of the heights of the points of support (typically these will be at the heights of the diaphragms). In the case of vertical cantilevers, h_i should be

measured to the point from which the wall is assumed to cantilever. If the wall is sitting on the ground and is laterally supported above, h_i may be taken as half of the height to the point of support.

If the wall is sitting on the ground and is not otherwise attached to the building it should be treated as an independent structure, not as a part. This will involve use of the appropriate ground spectrum for the site.

Note:

The above substitution for $C_i(T_p)$ has been necessary because the use of the tri-linear function given in NZS 1170.5:2004 (Equations 8.4(1), 8.4(2) and 8.4(3)) does not allow appropriate conversion from force to displacement demands. The revised $C_i(T_p)$ converts to the following, with the numerical numbers available from NZS 1170.5:2004 Table 3.1.

$C_i(T_p)$ = 2.0 for $T_p < 0.5$ sec

= $2.0(0.5/T_p)^{0.75}$ for $0.5 < T_p < 1.5$ sec

= $1.32/T_p$ for $1.5 < T_p < 3$ sec

= $3.96/T_p^2$ for $T_p > 3$ sec

Only 5% damping should be applied. Experiments show that expected levels of damping from impact are not realised: the mating surfaces at hinge lines tend to simply fold onto each other rather than impact.

Step 9

Calculate γ, the participation factor for the rocking system. This factor may be taken as:

$$\gamma = \frac{(W_b y_b + W_t y_t)h}{2Jg} \quad \text{…C8.17}$$

Note:

The participation factor relates the response deflection at the mid height of the wall to the response deflection for a simple oscillator of the same period and damping.

Step 10

From $C_p(T_p)$, T_p, R_p *and* γ calculate the displacement response, D_{ph} (in mm) as:

$$D_{ph} = \gamma(T_p/2\pi)^2 C_p(T_p).R_p.g \quad \text{...C8.18}$$

where:

$C_p(T_p)$ = the design response coefficient for face-loaded walls (refer to Step 8 above, and for more details refer to Section C8.10.3)

T_p = period of face-loaded wall, sec

R_p = the part risk factor as given by Table 8.1, NZS 1170.5:2004

$C_p(T_p).R_p \leq 3.6$.

Note that with T_p expressed in seconds, the multiplied terms $(T_p/2\pi)^2 \times C_p(T_p) \times g$ may be closely approximated in metres by:

$$(T_p/2\pi)^2 \times C_p(T_p) \times g = \text{MIN}(T_p/3, 1) \quad \text{…C8.19}$$

Step 11

Calculate

$$\%NBS = 100 \times \Delta_{\mathrm{m}} / D_{\mathrm{ph}} = 60\left(\Delta_{\mathrm{i}} / D_{\mathrm{ph}}\right) \quad \ldots \text{C8.20}$$

Note:

The 0.6 factor applied to Δ_{i} reflects that response becomes very dependent on the characteristics of the earthquake for deflections larger than $0.6\Delta_{\mathrm{i}}$.

The 2006 guidelines allowed a 20% increase in *%NBS* calculated by the above expression. However that is not justified now that different displacements are used for capacity and for the period and the subsequent calculation of demand.

The following Steps 12 to 14 are only required for anchorage design.

Step 12

Calculate the horizontal accelerations that would just force the rocking mechanism to form. The acceleration may be assumed to be constant over the height of the panel, reflecting that it is associated more with acceleration imposed by the supports than with accelerations associated with the wall deflecting away from the line of the supports. Express the acceleration as a coefficient, C_{m}, by dividing by g.

Note:

Again, virtual work proves the most direct means for calculating the acceleration. Appendix C8B shows how and derives the following expression for C_{m}, in which the ancillary masses are assumed part of W_{b} and W_{t}.

$$C_{\mathrm{m}} = \frac{b}{(W_{\mathrm{b}} y_{\mathrm{b}} + W_{\mathrm{t}} y_{\mathrm{t}})} \quad \ldots \text{C8.21}$$

Note:

To account for the initial enhancement of the capacity of the rocking mechanism due to tensile strength of mortar and possible rendering, we recommend that C_{m} be cautiously assessed when mortar and rendering are present or in the case where the wall is intended to be retrofitted. The value of C_{m} may also be too large to use for the design of connections. Accordingly, it is recommended that C_{m} need not be taken greater than the maximum part coefficient determined from Section 8 of NZS 1170.5:2004 setting R_{p} and μ_{p} = 1.0.

Step 13

Calculate $C_{\mathrm{p}}(0.75)$, which is the value of $C_{\mathrm{p}}(T_{\mathrm{p}})$ for a part with a short period from NZS 1170.5:2004, and define a seismic coefficient for the connections which is the lower of $\mathrm{C_m}$, $C_{\mathrm{p}}(0.75)$ or 3.6.

Note:

$C_{\mathrm{p}}(0.75)$ is the short period ordinate of the design response coefficient for parts from NZS 1170.5:2004, and 3.6 g is the maximum value of $C_{\mathrm{p}}(T_{\mathrm{p}})$ required to be considered by NZS 1170.5:2004 when R_{p} and μ_{p} = 1.0.

Step 14

Calculate the required support reactions using the contributing weight of the walls above and below the connection (for typical configurations this will be the sum of W_b and W_t for the walls above and below the support accordingly) and the seismic coefficient determined in Step 13.

Step 15

Calculate

$$\%NBS = \frac{\text{Capacity of connection from Section C8.8.4 x 100}}{\text{Required support reaction from Step 14}} \quad \ldots C8.22$$

Note:

If supports to face-loaded walls are being retrofitted, we recommend that the support connections are made stronger than the wall(s) and not less than required using a seismic coefficient of C_p (0.75); i.e. do not take advantage of a lower C_m value.

Simplifications for regular walls

The following approximations can be used if wall panels are uniform within a storey (approximately rectangular in vertical and horizontal section and without openings) and the inter-storey deflection does not exceed 1% of the storey height. The results are summarised in Table C8.12.

The steps below relate to the steps for the general procedure set out above.

Step 1 Divide the wall as before.

Step 2 Calculate the weight of the wall, W (in N), and the weight applied at the top of the storey, P (in N).

Step 3 Calculate the effective thickness as before, noting that it will be constant.

Step 4 Calculate the eccentricities, e_b, e_t and e_p. Each of these may usually be taken as either $t/2$ or 0.

Step 5 Calculate the instability deflection, Δ_i from the formulae in Table C8.12 for the particular case.

Step 6 Assign the maximum usable deflection, Δ_m, for capacity as 60% of the instability deflection.

Step 7 Calculate the period, which may be taken as $4.07\sqrt{(J/a)}$, where J and a are given in Table C8.12. Alternatively, where the wall is fairly thin (*h/t* is large), the period may be approximated as:

$$T_p = \sqrt{\frac{0.28h}{(1+2P/W)}} \quad \ldots C8.23$$

in which h is expressed in metres.

Step 8 Calculate $C_p(T_p)$ following Equation C8.16.

Step 9 Calculate the participation factor as for the general method, with the numerator of the expression expanded to give $\gamma = Wh^2/8Jg$. This may be taken at the maximum value of 1.5 or may be assessed by using the simplified expression for J shown in Table C8.12.

Step 10 Calculate D_{ph} from $C_p(T_p)$, T_p *and* γ in the same manner as for the general method.

Step 11 Calculate *%NBS* in the same manner as for the general method.

Note:

Charts are provided in Appendix C8C that allow the *%NBS* to be calculated directly for various boundary conditions for regular walls spanning vertically, given h/t_{Gross} for the wall, gravity load on the wall and factors defining the demand.

Table C8.12: Static instability deflection for uniform walls – various boundary conditions

Boundary condition number	**0**	**1**	**2**	**3**
e_p	0	0	$t/2$	$t/2$
e_b	0	$t/2$	0	$t/2$
b	$(W/2+P)t$	$(W+3P/2)t$	$(W/2+3P/2)t$	$(W+2P)t$
a	$(W/2+P)h$	$(W/2+P)h$	$(W/2+P)h$	$(W/2+P)h$
$\Delta_i = bh/(2a)$	$t/2$	$\frac{(2W+3P)t}{(2W+4P)}$	$\frac{(W+3P)t}{(2W+4P)}$	t
J	$\{(W/12)[h^2+7t^2]+Pt^2\}/g$	$\{\left(\frac{W}{12}\right)[h^2+16t^2]+9Pt^2/4\}/g$	$\{\left(\frac{W}{12}\right)[h^2+7t^2]+9Pt^2/4\}/g$	$\{(W/12)[h^2+16t^2]+4Pt^2\}/g$
C_m	$(2+4P/W)t/h$	$(4+6P/W)t/h$	$(2+6P/W)t/h$	$4(1+2P/W)t/h$

Note:

1. The boundary conditions of the piers shown above are for clockwise potential rocking.
2. The top eccentricity, e_t, is not related to a boundary condition, so is not included in the table. The top eccentricity, e_t, is the horizontal distance from the central pivot point to the centre of mass of the top block which is not related to a boundary condition.
3. The eccentricities shown in the sketches are for the positive sense. Where the top eccentricity is in the other sense e_p should be entered as a negative number.

Vertical cantilevers

Parameters for assessing vertical cantilevers, such as partitions and parapets, are derived in Appendix C8B. Please consult this appendix for general cases.

For parapets of uniform rectangular cross section, the following approximations can be used. These steps relate to the steps set out earlier for the general procedure for walls spanning between vertical diaphragms.

Step 1 There is no need to divide the parapet. Only one pivot is assumed to form: at the base.

Step 2 The weight of the parapet is W (in N). P (in N) is zero.

Step 3 The effective thickness is t (in mm) = $0.98t_{\text{nom}}$.

Step 4 Only e_{b} is relevant and it is equal to $t/2$. However, if the wall is supported on the ground, refer to Step 4 of the general procedure for walls spanning vertically between diaphragms

Step 5 The instability deflection measured at the top of the parapet $\Delta_{\text{i}} = t$.

Step 6 The maximum usable deflection measured at the top of the parapet $\Delta_{\text{m}} = 0.3\Delta_i = 0.3t$.

Step 7 The period may be calculated from the assumption that $\Delta_{\text{t}} = 0.8\Delta_{\text{m}} = 0.24\Delta_{\text{i}}$.

$$T_{\text{p}} = \sqrt{0.65h\left[1 + \left(\frac{t}{h}\right)^2\right]} \quad \text{…C8.24}$$

in which h, the height of the parapet above the base pivot, and t, the thickness of the wall, are expressed in metres. The formulation is valid for $P = 0, e_{\text{b}} = t/2$, $y_b = h/2$ and approximating $t = t_{\text{nom}}$.

Step 8 Calculate $C_{\text{p}}(T_{\text{p}})$ (refer to Step 8 of the general procedure for walls spanning vertically between diaphragms).

Step 9 Calculate $\gamma = 1.5/[1 + (t/h)^2] \leq 1.5$ …C8.25

Step 10 Calculate D_{ph} from $C_{\text{p}}(T_{\text{p}})$, T_{p} and γ and as before.

Step 11 Calculate *%NBS* as for the general procedure for walls spanning between a floor and an upper floor or roof, from;

$$\%NBS = 100\,\Delta_{\text{m}}/D_{\text{ph}} = 30\,\Delta_{\text{i}}/D_{\text{ph}} = 30\,t/D_{\text{ph}}. \quad \text{...C8.26}$$

Note:

The following Steps 12 to 14 are only required for anchorage design.

Step 12 Calculate $C_{\text{m}} = t/h$ …C8.27

Step 13 Calculate $C_{\text{p}}(0.75)$ which is the value of $C_{\text{p}}(T_{\text{p}})$ for a part with a short period from NZS 1170.5:2004, and define a seismic coefficient for the connections which is the lower of C_{m}, C_{p} (0.75) and 3.6.

Step 14 Calculate the base shear from W, C_{m} and $C_{\mathrm{p}}(0.75)$. This base shear adds to the reaction at the roof level restraint.

Note:

Charts are provided in Appendix C8C that allow the *%NBS* to be calculated directly for various boundary conditions for regular walls cantilevering vertically, given h/t_{Gross} for the wall, gravity load on the wall and factors defining the demand.

Gables

Figure C8.64(a) shows a gable that is:

- free along the vertical edge
- simply supported along the top edge (at roof level), and
- continuous at the bottom edge (ceiling or attic floor level).

This somewhat unusual case is useful in establishing parameters for more complex cases. The following parameters can be derived from this gable:

$$a = \frac{h}{6}(2W + 3P) \qquad \ldots\text{C8.28}$$

$$J = \frac{W}{24g}(32t^2 + h^2) + \frac{9Pt^2}{4g} \qquad \ldots\text{C8.29}$$

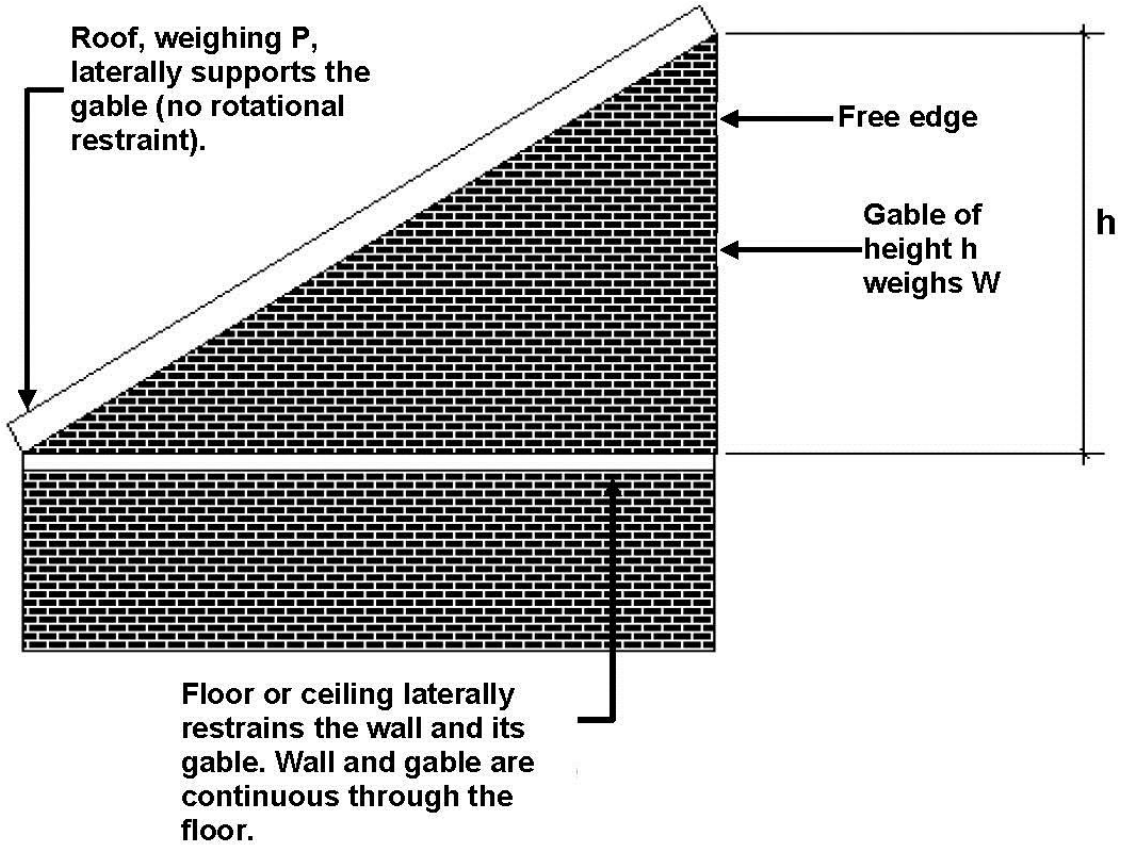
Note:

In the above equations, W and P are total weights, not weights per unit length. Also note that the participation factor now has a maximum value of 2.0 ($t << h, P = 0$).

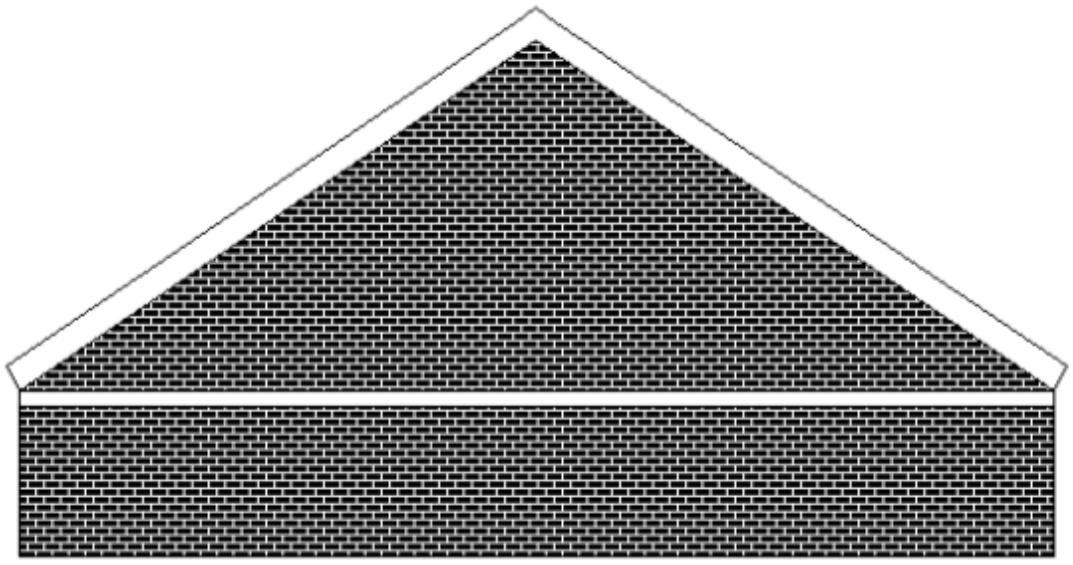
These results can be used for the gable in Figure C8.64(b) to provide a cautious assessment that does not recognise all of the factors that could potentially enhance the performance of such gables, such as the beneficial effects of membrane action.

Note:

There are several factors that enhance performance in gables like those shown in Figure C8.64(a), all of which relate to the occurrence of significant membrane action. Guidance on this aspect will be provided in future versions of this document when the necessary research (including testing) has been undertaken. (Please also refer to the following section on walls spanning horizontally and vertically.)



(a) Basic gable wall for defining parameters



(b) Typical gable for which results from (a) can be applied

Figure C8.64: Gable configurations

C8.8.5.3 Horizontal and vertical-horizontal spanning panels

Past earthquakes have shown that URM walls can act as a two-way spanning panel showing yield line patterns (refer to Figure C8.65) similar to those that occur in a two-way spanning slab if the walls are attached to the supports on four sides. However, a special study is recommended if two-way spanning is to be assumed. This study should take into account different elastic properties, displacement compatibility, and any detrimental effects resulting from the expected behaviour of the wall in the orthogonal direction.

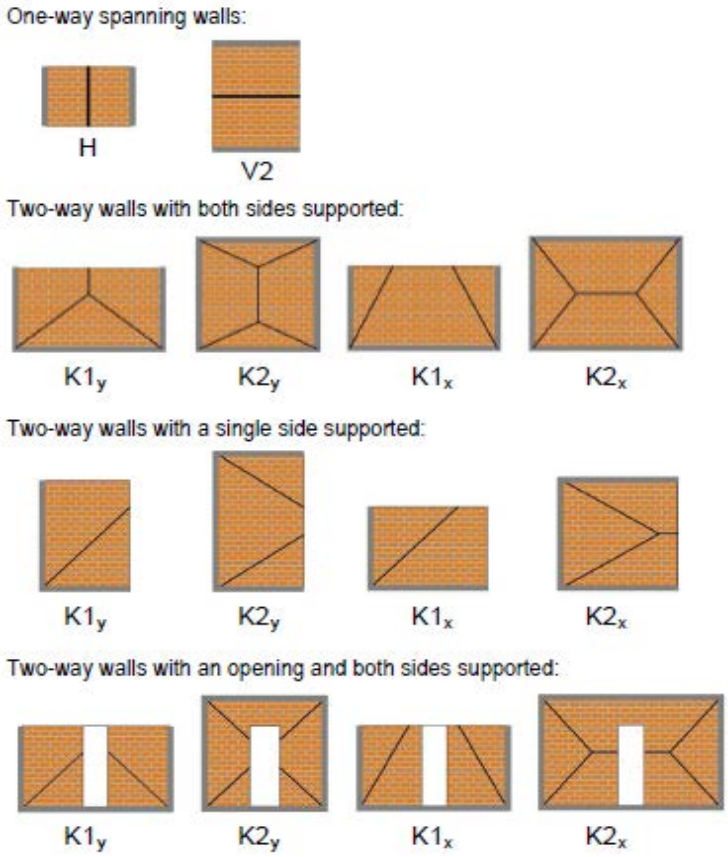


Figure C8.65: Idealised cracking patterns for masonry walls

Note:

Computationally intensive analytical methodologies such as finite element analysis have been shown to predict the out-of-plane strength of two-way spanning URM walls with good reliability. However, their reliance on knowing the precise values of material properties, the high computational effort and the high analytical skill required of the user makes them unsuitable for everyday design use.

The approach prescribed by the Australian masonry code AS 3700:2011 (AS, 2011) for ultimate strength design of two-way spanning walls is the so-called virtual work method, developed by Lawrence and Marshall (1996). This is a form of rigid plastic analysis which assumes that, at the point of ultimate strength, the load resistance of the wall is obtained from contributions of moment capacities along vertical and diagonal crack lines in two-way bending mechanisms (refer to Figure C8.65). Comparisons of strength predictions with a large experimental data set have been shown to be largely favourable in the sources mentioned before, in spite of numerous shortcomings of the moment capacity expressions used within the method which are still currently prescribed in AS 3700:2011 (AS, 2011).

More recently, Willis et al. (2004) and Griffith et al. (2007) have developed alternative expressions for calculating the moment capacities which incorporate significant improvements over the AS 3700 expressions as they are based on more rational mechanical models, account for the beneficial effects of vertical compression, and are dimensionally consistent. Furthermore, Willis et al. (2004) demonstrated that the expressions perform favourably in predicting the ultimate load capacity when implemented into the virtual work approach.

The currently available research is not sufficient for assessing two-way panels in a typical design office environment. However, significant progress has been made into the behaviour of walls of this kind, e.g. Vaculik (2012), and this is likely to be translated this into procedures suitable for design office use and routine assessment in time for the next update of these guidelines.

C8.8.6 Walls under in-plane load

C8.8.6.1 General

The capacity of wall members/elements will typically be limited by their horizontal shear capacity.

Wall members/elements under in-plane load can be broadly categorised into two main groups: walls without penetration and walls with penetrations.

The capacity of a wall element without penetrations should be assessed as outlined in Section C8.8.6.2.

The recommended approach to assessing the capacity of a wall element with penetrations is as follows:

Step 1: Divide the wall element into individual “members” consisting of the “piers” between the penetrations and “spandrel” members above and below the penetrations.

Step 2: Carry out a plane frame lateral load analysis of the wall to determine the relationship between the earthquake lateral load and the actions in the piers (including axial load) and the spandrels.

Step 3: Determine the capacity of the piers in a similar manner to walls in accordance with Section C8.8.6.2. This will be a function of axial load on the pier (tension and/or compression).

Step 4: Determine the capacity of the spandrels in accordance with Section C8.8.6.3.

Step 5: Determine if the capacity of the penetrated wall is governed by spandrel or pier capacity. This will need to be evaluated for each spandrel to pier connection, and the effect of the potential axial load in the piers will need to be considered. A sway index as defined in Section C8.8.6.4 can be used to do this.

Step 6: Based on the sway index, determine if the capacity of each pier element is governed by the pier itself or the abutting spandrel element.

Step 7: Carry out an analysis of the wall element to determine its capacity based on the capacity of the individual piers (including the effects of axial load) acting in series (refer to Section C8.8.6.4).

Note:

For basic buildings, when assessing the capacity of the wall element the effect of the spandrels may be ignored and the piers assumed to extend over the full height of the wall as cantilevers. This will avoid the need to assess the effect of lateral load induced axial loads, but larger displacements may be predicted.

The degree to which a wall on a single line, but extending over several storeys, should be broken down into individual members will depend on the method of analysis used to establish the building's global capacity. This is discussed further in Section C8.9. Typically it is expected that it will be necessary to assess the capacity for each wall line between each storey in the building.

C8.8.6.2 In-plane capacity of URM walls and pier elements

The in-plane strength capacity of URM walls and pier elements should be taken as the lower of the assessed diagonal tensile, toe crushing, in-plane rocking or bed-joint sliding strength capacities as determined below. This then becomes the mode of behaviour and the basis for the calculation of the deformation capacity. Where DPC layers are present these may also limit the shear that can be resisted.

For the purposes of assessing the wall or pier capacities for each mechanism the yield displacement, Δ_y, may be taken as the sum of the flexural and shear in-plane displacements (making allowance for cracking, i.e. the effective modulus of elasticity and shear modulus, etc., as recommended in Section C8.7.6) when the element is subjected to a lateral shear consistent with achieving the shear strength for that mechanism as given below. Refer also to Section C8.9.4.5.

Diagonal tensile capacity

This is one of the most important checks to be carried out.

The maximum diagonal tensile strength of a wall, pier or spandrel without flanges (or where the engineer has decided to ignore them) can be calculated using Equation C8.30 (ASCE 41-13, 2014). Otherwise, refer to the relevant reference to account for the effect of flanges.

$$V_{\mathrm{dt}} = f_{\mathrm{dt}} A_{\mathrm{n}} \beta \sqrt{1 + \frac{f_{\mathrm{a}}}{f_{\mathrm{dt}}}} \qquad \ldots \text{C8.30}$$

where:

β = factor to correct nonlinear stress distribution (refer to Table C8.13)
A_{n} = area of net mortared/grouted section of the wall web, mm^2
f_{dt} = masonry diagonal tension strength (refer to Equation C8.3), MPa
f_{a} = axial compression stress due to gravity loads calculated at mid height of the wall/pier, MPa.

Table C8.13: Shear stress factor, β, for Equation C8.30

Criterion			β
Slender piers, where $h_{\text{eff}}/l > 1.5$			0.67
Squat piers, where $h_{\text{eff}}/l < 1.0$			1.00
Note: Linear interpolation is permitted for intermediate values of h_{eff}/l			

Refer to Figure C8.66 for the definition of h_{eff}.

This failure mode occurs when the diagonal tensile strength of a wall or pier is exceeded by the principal stresses. It is one of the undesirable failure modes as it causes a rapid degradation in strength and stiffness after the formation of cracking, ultimately leading to loss of load path. For this reason a deformation limit of Δ_y for this failure mode is recommended.

This failure mode is more common where axial stresses are high, piers are squatter and the tensile strength of masonry is low.

Diagonal tension failure leads to formation of an inclined diagonal crack that commonly follows the path of bed and head joints through the masonry, because of the lower strength of mortar compared to brick. However, cracking through brick is also possible if the mortar is stronger. In New Zealand masonry, the crack pattern typically follows the mortar joint.

For conditions where axial stresses on walls or piers are relatively low and the mortar strengths are also low compared to the splitting strengths of the masonry units, diagonal tension actions may be judged not to occur prior to bed-joint sliding. However, there is no available research to help determine a specific threshold of axial stress and relative brick and mortar strengths that differentiates whether cracking occurs through the units or through the mortar joints (ASCE 41-13, 2014).

Toe crushing capacity

The probable toe crushing strength, V_{tc}, of a wall, pier or spandrel can be calculated using Equation C8.31 if no flanges are present or if the engineer has decided to ignore them. If flanges are to be accounted for, refer to the relevant reference.

$$V_{\text{tc}} = (\alpha\, P + 0.5\, P_{\text{w}}) \left(\frac{L_{\text{w}}}{h_{\text{eff}}}\right) \left(1 - \frac{f_{\text{a}}}{0.7 f'_{\text{m}}}\right) \qquad \text{…C8.31}$$

where:

α = factor equal to 0.5 for fixed-free cantilever wall/pier, or equal to 1.0 for fixed-fixed wall/pier
P = superimposed and dead load at top of the wall/pier
P_{w} = self-weight of wall/pier
L_{w} = length of the wall/pier, mm
h_{eff} = height to resultant of seismic force (refer to Figure C8.66), mm
f_{a} = axial compression stress due to gravity loads at the base of the wall/pier, MPa
f'_{m} = masonry compression strength, MPa (refer to Section C8.7.3).

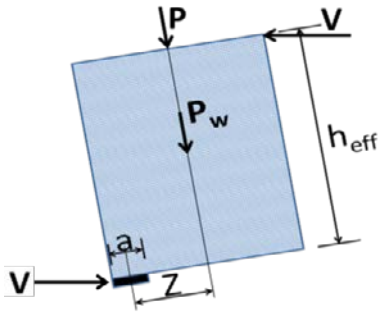


Figure C8.66: A rocking pier

A deformation limit of Δ_y or ϕ_y is recommended for this failure mode for walls/piers and spandrels respectively.

A toe crushing failure mode is not an expected failure mode of low-rise New Zealand walls or piers during in-plane loading. However, it still needs to be assessed; particularly when the walls have been retrofitted with un-bonded post-tensioning or a seismic intervention that inhibits the diagonal tension failure mode.

Rocking capacity

Rocking failure is one of the stable modes of failure. Experimental investigations undertaken by Knox (2012), Anthoine et al. (1995), Costley and Abrams (1996), Franklin et al. (2001), Magenes and Calvi (1995), Moon et al. (2006), Bruneau and Paquette (2004), Xu and Abrams (1992), and Bothara et al. (2010) have confirmed that URM elements exhibiting rocking behaviour have substantial deformation capacity past initial cracking but also exhibit very low levels of hysteretic damping.

A generalised relationship between strength and deformation for the rocking mechanism is shown in Figure C8.67.

The maximum probable rocking strength of a wall (considered over one level) or pier, V_r, can be calculated using Equation C8.32.

$$V_r = 0.9\,(\alpha P + 0.5P_w)\,\frac{L_w}{h_{eff}} \qquad \text{…C8.32}$$

where:

V_r = strength of wall or wall pier based on rocking
α = factor equal to 0.5 for fixed-free cantilever wall, or equal to 1.0 for fixed-fixed wall pier
P = superimposed and dead load at the top of the wall/pier under consideration
P_w = self-weight of the wall/pier
L_w = length of wall or pier, mm
h_{eff} = height to resultant of seismic force (refer to Figure C8.66), mm.

When assessing the capacity of walls without openings for the full height of the building, Equation C8.32 will need to be adjusted to account for the different location of the lateral force. This can be assumed to be applied at two thirds of the height of the building from the point of fixity.

Nonlinear response of rocking URM piers is generally characterised by a negative post-yield slope due to P-delta effects but will be limited by toe crushing, as the effective bearing area at the toe of the rocking pier reduces to zero under increasing lateral displacement (refer to Figure C8.67). This latent toe crushing differs from that discussed above as it typically occurs at larger rotations and lower shears.

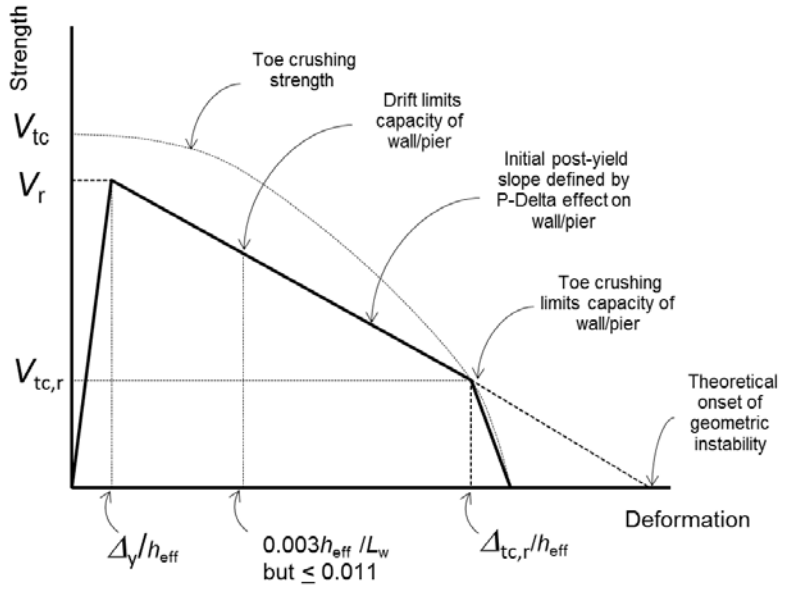


Figure C8.67: Generalised force-deformation relationship for rocking of unreinforced masonry walls or piers (ASCE 41-13, 2014)

Deformation associated with the onset of toe crushing, $\Delta_{tc,r}/h_{eff}$, should be calculated using a moment-curvature or similar analytical approach and a maximum usable strain at the compression fibre of 0.0035. The axial compressive stress on the toe due to gravity loads should be based on an equivalent compression zone of the effective net section of the rocking pier that is in bearing.

Under rare conditions, the geometric stability of the rocking pier due to P-delta effects may govern the ultimate deformation capacity. In the absence of substantiating test results, assume elastic unloading hysteretic characteristics for rocking URM in-plane walls and wall piers.

Note:

It is recommended that the capacity of a rocking wall/pier is limited to that consistent with a wall/pier lateral drift equal to the lower of $0.003h_{eff}/L_w$ or 0.011. The lateral performance of a rocking wall is considered to be less reliable and not to provide the level of resilience considered appropriate when the deflections exceed these values. Wall/pier elements that are not part of the seismic resisting system and which have a thickness greater than 350 mm (3 wythes) are expected be able to provide reliable vertical load carrying capacity at higher deflections approaching twice the limits given above. These greater limits can also be used for all wall/pier elements when cyclic stiffness and strength degradation are included in the analysis method used. Such an analysis will automatically include redistribution of the lateral loads between elements when this is necessary.

Assumption of fixity or cantilever action depends on the stiffness and overall integrity of the spandrels above and below the rocking pier and on how effectively spandrels can transmit vertical shears and bending. Conversely, wall spandrels that are weak relative to adjacent piers may not provide fixity at the tops and bottoms of piers and may result in piers acting as cantilevers. In general, deep spandrels could provide fixed-fixed boundary conditions.

Note that if the self-weight of the pier is large and boundary conditions are fixed-fixed, Equation C8.32 may overestimate the rocking capacity.

This behaviour mode is common where axial stresses are low, walls or piers are slender (height to length ratio > 2) and mortar strength are relatively better.

Bed-joint sliding shear capacity

Bed-joint sliding failure is one of the stable modes of failure. Investigations undertaken by various researchers have confirmed that URM elements exhibiting bed-joint sliding behaviour have substantial deformation capacity past initial cracking.

The recommended generalised force-deformation relationship for URM walls and wall piers governed by bed-joint sliding or sliding stair-stepped failure modes is illustrated in Figure C8.68. A simplified form of the ASCE 41-13 (2014) force-deformation relationship has been adopted.

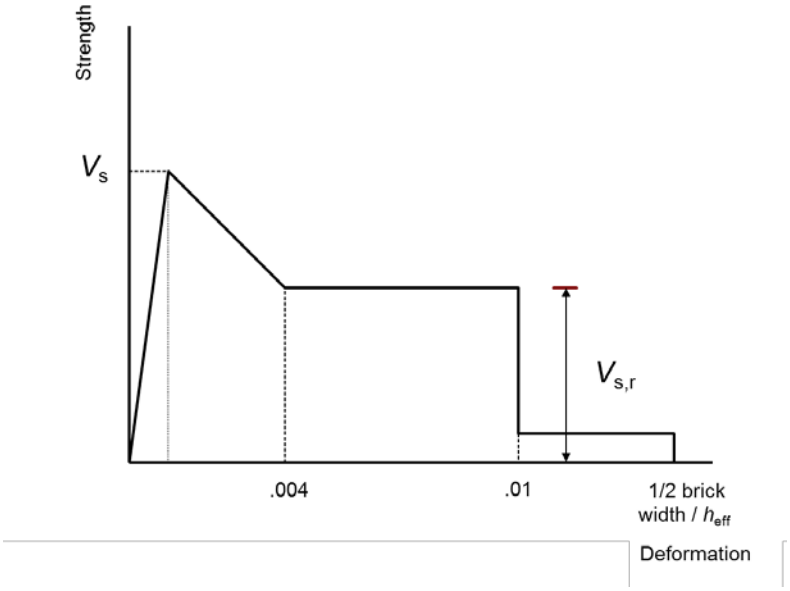


Figure C8.68: Generalised force-deformation relationship for unreinforced masonry walls or piers governed by bed-joint sliding or stair-stepped sliding

The maximum probable sliding shear strength, V_s, can be found from Equation C8.33.

$$V_s = 0.7(t_{nom} L_w c + \mu_f (P + P_w)) \quad \text{…C8.33}$$

where:

μ_f = masonry coefficient of friction
P = superimposed and dead load at top of the wall/pier
P_w = self-weight of wall/pier above the sliding plane being considered.

The 0.7 factor is to reflect the overall reliability of the sliding mechanism calculation.

The capacity for bed-joint sliding in masonry elements is a function of bond and frictional resistance. Therefore, Equation C8.33 includes both factors. However, with increasing cracking, the bond component is progressively degraded until only the frictional component remains. The probable residual wall sliding shear capacity, $V_{s,r}$, is therefore found from Equation C8.33 setting the cohesion, c, equal to 0.

Note:

It is recommended that the bed-joint sliding capacity of a rocking wall/pier is limited to a lateral drift of 0.003. The lateral performance of a wall/pier is considered to be unreliable and not able to provide the level of resilience considered appropriate when the deflections exceed this value. Wall/pier elements that are not part of the seismic resisting system are expected to be able to provide reliable vertical load carrying capacity at higher drifts, approaching 0.0075. These greater limits can also be used for all wall/pier elements when cyclic stiffness and strength degradation are included in the analysis method used. Such an analysis will automatically include redistribution of the lateral loads between elements when this is necessary.

Slip plane sliding

A DPC layer, if present, will be a potential slip plane, which may limit the capacity of a wall.

The probable shear capacity of a slip plane for no slip can be found from Equation C8.34:

$$V_{dpc} = \mu_{dpc} (P + P_w) \quad \text{…C8.34}$$

where:

μ_{dpc} = DPC coefficient of friction. Typical values are 0.2-0.5 for bituminous DPC, 0.4 for lead, and higher (most likely governed by the mortar itself) for slate DPC.

Other terms are as previously defined.

Note:

Where sliding of a DPC layer is found to be critical, testing of the material in its current/in-situ state may be warranted. Alternatively, parametric checks, where the effects of low/high friction values are assessed, may show that the DPC layer is not critical in the overall performance.

Sliding on a DPC slip plane does not necessarily define the deformation capacity of this behaviour mode.

Evaluating the extent of sliding may be calculated using the Newmark sliding block (Newmark, 1965) or other methods. However, exercise caution around the sensitivity to different types of shaking and degradation of the masonry above/below the sliding plane. Where sliding is used in the assessment to give a beneficial effect, this should be subject to peer review.

Effect of wall and pier flanges

It is common practice to ignore the effects of flanges on the walls or piers while assessing the in-plane capacity of walls and piers. However, experimental research undertaken by Costley and Abrams (1996), Bruneau and Paquette (2004), Moon et al. (2006), Yi et al. (2008), and Russell and Ingham (2010b) has shown that flanges have the potential to influence the response of in-plane walls. Flanged walls can have considerably higher strength and stiffness than those without flanges. The assessment could be particularly non-conservative where estimated rocking, sliding shear, or stair-step cracking strength (which are stable modes of failure) is close to the diagonal tensile strength of pier and walls. The recommended approach is to assess how much flange is required for diagonal tension to be the critical behaviour mode and, based on this, determine if further investigation is required.

Note:

One of the preconditions for taking into account the effect of the flanges is that they should remain integral with the in-plane piers and walls during the seismic shaking. Therefore, the integrity of the connections must be ascertained before ignoring or including them.

If flanges are taken into account, it is common to assume that the lengths of flanges acting in compression are the lesser of six times the thicknesses of the in-plane walls or the actual lengths of the flanges. It is also common to assume that equivalent lengths of tension flanges (to resist global or element overturning) are based on likely crack patterns relating to uplift in flange walls (Yi et al., 2008). Other approaches that either model or consider different flange lengths qualitatively may result in a variety of crack patterns and corresponding sequences of actions.

C8.8.6.3 URM spandrel capacity

General

The recommended generalised force-deformation relationship for URM spandrels is illustrated in Figure C8.69. The recommended generalised force-deformation relationship is based on experimental work undertaken by Beyer and Dazio (2012a and 2012b), Knox (2012), Graziotti et al. (2012), and Graziotti et al. (2014) and as recommended by Cattari et al. (2014).

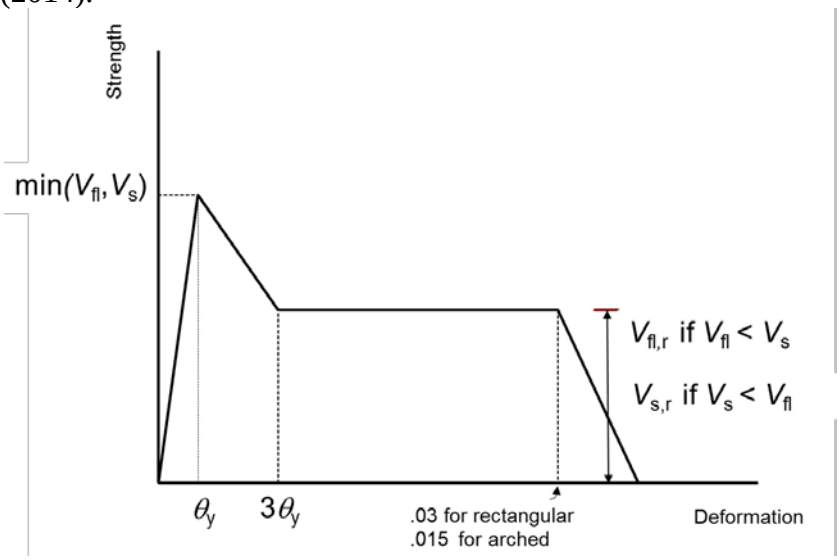


Figure C8.69: Generalised force-deformation relationship for unreinforced masonry spandrels

The probable in-plane shear capacity of a URM spandrel should not be taken greater than that implied by the probable spandrel flexural strength.

θ is the chord rotation of the spandrel, relative to the piers.

Note:

It is considered prudent to limit the deformation capacity of a spandrel panel to a panel drift of $3\theta_y$ (Beyer and Mangalathu, 2014) if its capacity is to be relied on as part of the seismic resisting system. Panel chord rotation capacities beyond 0.02 or 0.01 for rectangular and arched spandrels respectively, for panels that are not assumed to be part of the lateral seismic resisting system, are not recommended as the performance of the spandrel (i.e. its ability to remain in place) could become unreliable at rotations beyond these limits. These greater limits can also be used for all spandrel elements when cyclic stiffness and strength degradation are included in the analysis method used. Such an analysis will automatically include redistribution of the lateral loads between elements when this is necessary. Therefore, the need to distinguish, in advance, between elements of the lateral and non-lateral load resisting systems is not required.

Two generic types of spandrel have been identified: rectangular and those with shallow arches. Recommendations for the various capacity parameters for these two cases are given in the following sections.

Investigations are continuing on appropriate parameters for deep arched spandrels. In the interim, until more specific guidance is available, it is recommended that deep arched spandrels are considered as equivalent rectangular spandrels with a depth that extends to one third of the depth of the arch below the arch apex.

The geometrical definitions used in the following sections are shown on Figure C8.70.

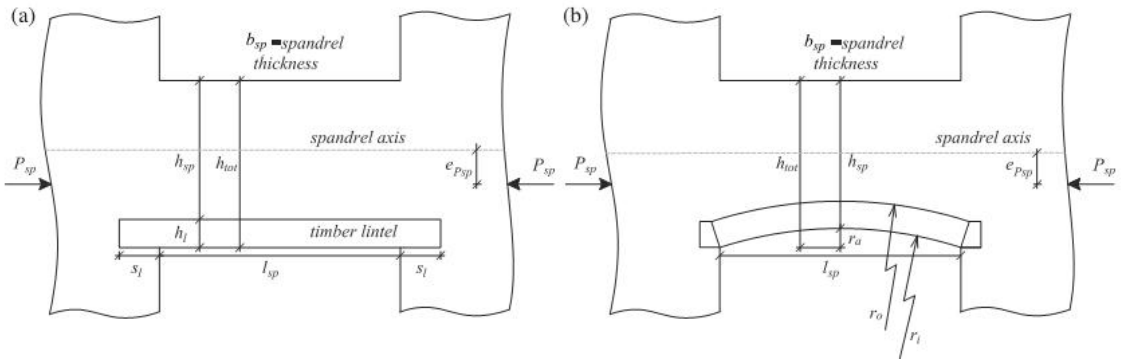


Figure C8.70: Geometry of spandrels with (a) timber lintel and (b) shallow masonry arch (Beyer, 2012)

Rectangular spandrels

The expected in-plane strength of URM spandrels with and without timber lintels can be determined following the procedures detailed below.

Note:

There is limited experimental information on the performance of URM spandrels with lintels made from materials other than timber. However, URM spandrels with steel lintels are expected to perform in a similar manner to those with timber lintels.

When reinforced concrete lintels are present the capacity of the spandrel can be calculated neglecting the contribution of the URM.

Shear due to flexural behaviour

The shear developed in a rectangular URM spandrel limited by the probable flexural strength of a spandrel, V_{fl}, can be estimated using Equation C8.35 (Beyer, 2012). Timber lintels do not make a significant contribution to the probable flexural capacity of the spandrels so can be ignored.

$$V_{\text{fl}} = \left(f_{\text{t,eff}} + p_{\text{sp}}\right)\frac{h_{\text{sp}}^2 b_{\text{sp}}}{3l_{\text{sp}}} \qquad \text{…C8.35}$$

where:

$f_{\text{t,eff}}$ = equivalent probable tensile strength of masonry spandrel
p_{sp} = axial stress in the spandrel
h_{sp} = height of spandrel excluding depth of timber lintel if present
b_{sp} = width of spandrel
l_{sp} = clear length of spandrel between adjacent wall piers.

Unless the spandrel is prestressed or provided with continuous bond beam above the opening, the axial stress in the spandrel can be assumed to be negligible when determining the peak flexural capacity.

The equivalent probable tensile strength of a masonry spandrel, $f_{\text{t,eff}}$, can be estimated using Equation C8.36:

$$f_{\text{t,eff}} = \alpha_{\text{s}}\left(c + 0.5\mu_{\text{f}}p_{\text{p}}\right) + \frac{c}{2\mu_{\text{f}}} \qquad \text{…C8.36}$$

where:

p_{p} = mean axial stress due to superimposed and dead load in the adjacent wall piers
μ_{f} = masonry coefficient of friction
c = masonry bed-joint cohesion
α_{s} = bond pattern factor taken as the ratio of horizontal crack length vs sum of the vertical crack length.

For spandrels constructed using 230 mm x 110 mm x 70 mm bricks α_s can be estimated as follows:

Running bond: $\alpha_s = 1.4$
Common bond: $\alpha_s = 1.2$
English bond: $\alpha_s = 0.7$
Stack bond: $\alpha_s = 0.0$.

Shear due to residual flexural behaviour

The shear developed in a rectangular URM spandrel due to the probable residual flexural strength of the spandrel, $V_{fl,r}$, can be estimated using Equation C8.37 (Beyer, 2012). Timber lintels do not often make a significant contribution to the residual flexural capacity of URM spandrels so they can be ignored.

$$V_{fl,r} = \frac{p_{sp}\, h_{sp}^2 b_{sp}}{l_{sp}} \left(1 - \frac{p_{sp}}{0.85 f_{hm}}\right) \quad \text{...C8.37}$$

where:

p_{sp} = axial stress in the spandrel
f_{hm} = compression strength of the masonry in the horizontal direction ($0.5f'_m$).

Axial stresses are generated in spandrel elements due to the restraint of geometric elongation. Results from experimental research indicate that negligible geometric elongation can be expected when peak spandrel strengths are developed (Beyer, 2012; Graziotti et al., 2012), as this is at relatively small spandrel rotations. As a result, there is little geometric elongation. Significant geometric elongation can occur once peak spandrel strengths have been exceeded, and significant spandrel cracking occurs within the spandrel, as higher rotations are sustained in the element. An upper bound estimate of the axial stress in a restrained spandrel, p_{sp}, can be determined using Equation C8.38 (Beyer, 2014):

$$p_{sp} = (1 + \beta_s) f_{dt} \frac{l_{sp}}{\sqrt{l_{sp}^2 + h_{sp}^2}} \quad \text{...C8.38}$$

where:

f_{dt} = masonry probable diagonal tension strength
β_s = spandrel aspect ratio (l_{sp}/h_{sp}).

Equation C8.38 calculates the limiting axial stress generated in a spandrel associated with diagonal tension failure of the spandrel. The equation assumes the spandrel has sufficient axial restraint to resist the axial forces generated by geometric elongation.

In most typical situations the engineer can assume that spandrels comprising the interior bays of multi-bay pierced URM walls will have sufficient axial restraint such that diagonal tension failure of the spandrels could occur.

Spandrels comprising the outer bays of multi-bay pierced URM walls typically have significantly lower levels of axial restraint. In this case the axial restraint may be insufficient to develop a diagonal tension failure in the spandrels. Sources of axial restraint that may be

available include horizontal post-tensioning, diaphragm tie elements with sufficient anchorage into the outer pier, or substantial outer piers with sufficient strength and stiffness to resist the generated axial forces. For the latter to be effective the pier would need to have enough capacity to resist the applied loads as a cantilever.

It is anticipated that there will be negligible axial restraint in the outer bays of many typical unstrengthened URM buildings. In this case the engineer can assume the axial stress in the spandrel is nil when calculating the residual flexural strength.

Probable shear strength

The probable shear strength of a rectangular URM spandrel, V_s, can be estimated using either Equation C8.39 (Beyer, 2012) or Equation C8.40 (Turnsek and Čačovič, 1970) as outlined below. Timber lintels do not make a significant contribution to the peak shear capacity of URM spandrels so can be ignored.

$$V_s = \frac{2}{3}\left(c + \mu_f p_{sp}\right) h_{sp} b_{sp} \quad \text{...C8.39}$$

$$V_s = f_{dt}\beta_{sp}\left(\sqrt{1 + \frac{p_{sp}}{f_{dt}}}\right) h_{sp} b_{sp} \quad \text{...C8.40}$$

where:

f_{dt} = probable masonry diagonal tension strength
β_{sp} = factor to correct the nonlinear stress distribution in the spandrel from Table C8.14.

Table C8.14: Shear stress factor, β_{sp}, for Equation C8.30

Criterion	β_{sp}
Slender spandrels, where $l_{sp}/h_{sp} > 1.5$	0.67
Squat spandrels, where $l_{sp}/h_{sp} < 1.0$	1.00
Note: Linear interpolation is permitted for intermediate values of l_{sp}/h_{sp}	

Unless the spandrel is prestressed the engineer can assume the axial stress in the spandrel is negligible when determining the shear capacity. Equation C8.39 is the shear strength associated with the formation of cracks through head and bed joints over almost the entire height of the spandrel: use this equation when the mortar is weaker than the brick. If the mortar is stronger than the brick and fracture of the bricks is likely to occur, use Equation C8.40.

Residual shear strength

Once shear cracking has occurred the URM spandrel can no longer transfer in-plane shear demands. When present, timber lintels acting as beams (simply supported at one end and fixed at the other) can transfer the vertical component of the spandrel load, F, to the adjacent pier (refer to Figure C8.71).

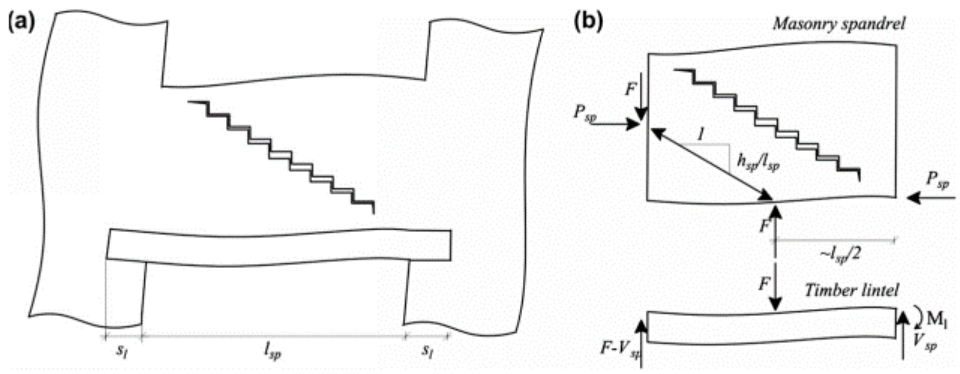


Figure C8.71: Shear mechanism of URM spandrels with timber lintels (Beyer, 2012)

Residual shear strength of cracked rectangular URM spandrels with timber lintels can be estimated as the minimum of Equation C8.41 or the capacity of the timber lintel to resist the applied load (Beyer, 2012). When no timber lintel is present the residual shear capacity of URM spandrels is negligible and can be assumed to be nil.

$$V_{\mathrm{s,r}} = \frac{11}{16} p_{\mathrm{sp}} \frac{h_{\mathrm{sp}}^2 b_{\mathrm{sp}}}{l_{\mathrm{sp}}} \quad \text{...C8.41}$$

The applied load, F, to be resisted by the timber lintel can be calculated as:

$$F = p_{\mathrm{sp}} \frac{h_{\mathrm{sp}}^2 b_{\mathrm{sp}}}{l_{\mathrm{sp}}} \quad \text{...C8.42}$$

Spandrel axial stresses, p_{sp}, can be calculated in accordance with the procedures outlined above. Confirm the ability of the timber lintel to sustain the applied load.

Spandrels with a shallow arch

Shear due to flexural behaviour

The shear developed in an URM spandrel due to the probable flexural capacity of a spandrel with a shallow arch, V_{fl}, can be estimated using Equation C8.43 (Beyer 2012):

$$V_{\mathrm{fl}} = h_{\mathrm{sp}} b_{\mathrm{sp}} \left(f_{\mathrm{t,eff}} \frac{h_{\mathrm{sp}}}{3 l_{\mathrm{sp}}} + p_{\mathrm{sp}} \tan \alpha_{\mathrm{a}} \right) \quad \text{...C8.43}$$

where α_{a} is the arch half angle of embrace computed as:

$$\alpha_{\mathrm{a}} = \tan^{-1} \left(\frac{l_{\mathrm{sp}}}{2(r_{\mathrm{i}} - r_{\mathrm{a}})} \right) \quad \text{...C8.44}$$

where dimensions r_{i}, r_{a} and l_{sp} are defined in Figure C8.70. The arch is considered shallow if the half angle of embrace, α_{a}, satisfies Equation C8.45 where r_{o} is also defined in Figure C8.70.

$$\cos \alpha_{\mathrm{a}} \geq \frac{r_{\mathrm{i}}}{r_{\mathrm{o}}} \quad \text{...C8.45}$$

Unless the spandrel is prestressed the engineer can assume the axial stress in the spandrel is negligible when determining the peak flexural capacity.

Shear due to residual flexural behaviour

The shear developed in an URM spandrel due to the residual flexural capacity, $V_{\mathrm{fl,r}}$, of a spandrel with a shallow arch can be estimated using Equation C8.46 (Beyer 2012) and by referring to Figure C8.70.

$$V_{\mathrm{fl,r}} = \frac{p_{\mathrm{sp}}\, h_{\mathrm{sp}} h_{\mathrm{tot}} b_{\mathrm{sp}}}{l_{\mathrm{sp}}}\left(1 - \frac{p_{\mathrm{sp}}}{0.85 f_{\mathrm{hm}}}\right) \quad \ldots\text{C8.46}$$

where the dimension h_{tot} is defined in Figure C8.70. Spandrel axial stresses, p_{sp}, can be calculated with the procedures set out in the previous section.

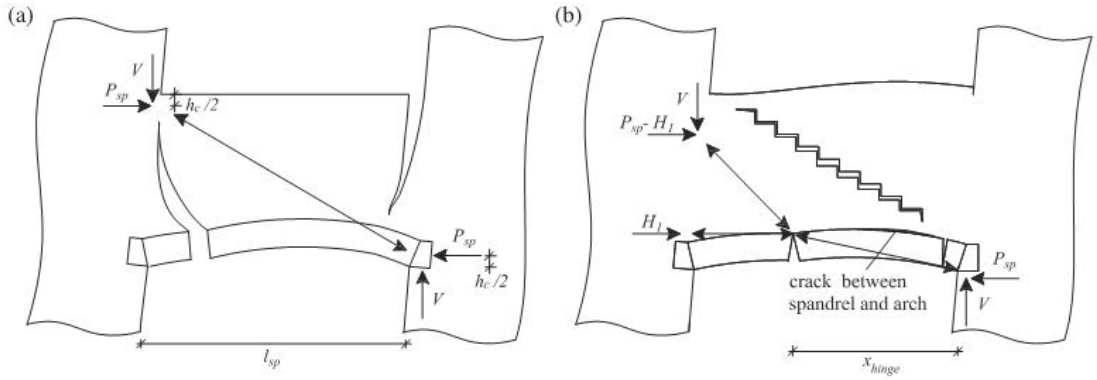


Figure C8.72: Spandrel with shallow arch. Assumed load transfer mechanism after (a) flexural and (b) shear cracking. (Beyer, 2012)

Probable shear strength

The probable shear strength of a URM spandrel with a shallow arch, V_{s}, can be estimated using Equation C8.47 or Equation C8.48 (Beyer, 2012) as outlined below:

$$V_{\mathrm{s}} = h_{\mathrm{sp}} b_{\mathrm{sp}} \left[\frac{2}{3}\left(c + \mu_{\mathrm{f}} p_{\mathrm{sp}}\right) + p_{\mathrm{sp}} \tan \alpha_{\mathrm{a}}\right] \quad \ldots\text{C8.47}$$

$$V_{\mathrm{s}} = \quad h_{\mathrm{sp}} b_{\mathrm{sp}} \left(f_{\mathrm{dt}} b_{\mathrm{sp}} \sqrt{1 + \frac{p_{\mathrm{sp}}}{f_{\mathrm{dt}}}} + p_{\mathrm{sp}} \tan \alpha_{\mathrm{a}}\right) \quad \ldots\text{C8.48}$$

Unless the spandrel is prestressed the estimator can assume the axial stress in the spandrel is negligible when determining the peak shear capacity. Equation C8.47 is the peak shear strength associated with the formation of cracks through head and bed joints over almost the entire height of the spandrel: it applies when the mortar is weaker than the brick. Use Equation C8.48 if the mortar is stronger than the brick and fracture of the bricks will occur.

Residual shear strength

Once shear cracking has occurred the URM spandrel itself can no longer transfer in-plane shear demands (refer to Figure C8.72). The probable residual capacity of the lintel is therefore equivalent to the probable shear capacity of the arch which can be computed as follows (Beyer, 2012):

$$V_{\mathrm{s,r}} = h_{\mathrm{sp}} b_{\mathrm{sp}} p_{\mathrm{sp}} \tan \alpha_{\mathrm{a}} \quad \ldots\text{C8.49}$$

Spandrel axial stresses, p_{sp}, can be calculated in accordance with the procedures provided in the previous section.

C8.8.6.4 Analysis methods for penetrated walls

This section provides an overview of analysis methods that can be used to assess the capacity of a penetrated wall made up of members and of elements. Recommendations made regarding modelling assumptions for global analyses in Section C8.9.4 also apply to analyses of URM members/elements.

Analysis of in-plane loaded URM walls and perforated walls can be carried out using the simplified "pier only" model shown in Figure C8.73 (Tomazevic, 1999). This analysis procedure assumes that the spandrels are infinitely stiff and strong, and therefore that the wall piers will govern the seismic response of the building. This simplified procedure may lead to non-conservative assessments for those structures which contain weak spandrels, or for structures assessed on the assumption that piers of dissimilar width rock simultaneously with shears calculated pro rata on the rocking resistance.

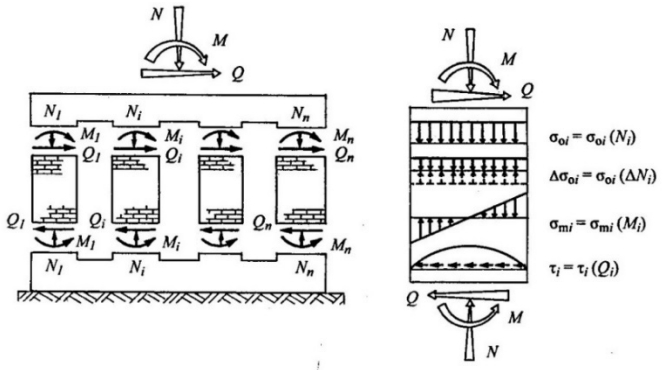


Figure C8.73: Forces and stresses in in-plane piers (Tomazevic, 1999)

Linear and nonlinear equivalent frame models as shown in Figure C8.74 (Magenes, 2006) can be used to analyse the in-plane response of perforated URM walls. Work by Knox has extended the equivalent frame model to include weak spandrel behaviour (Knox, 2012).

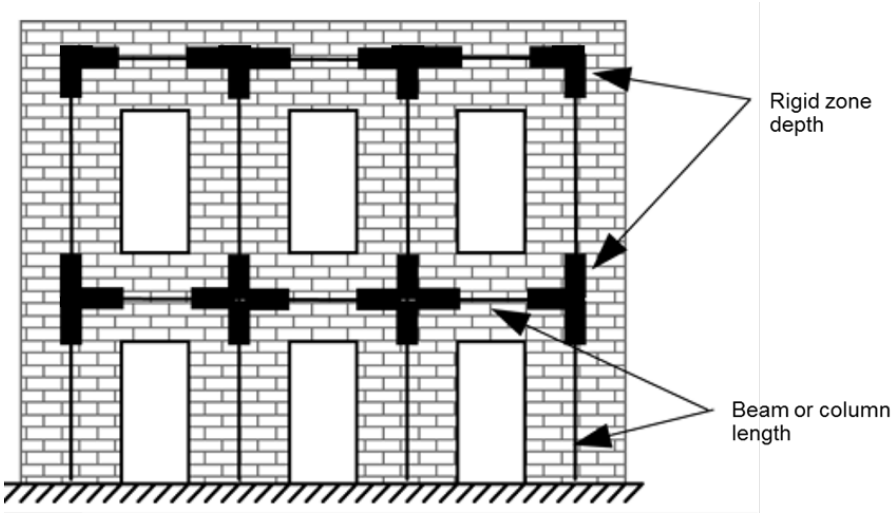


Figure C8.74: Equivalent frame

To investigate whether perforated wall behaviour is governed by spandrel or pier capacity a sway potential index, S_i, can be defined for each spandrel-pier joint by comparing the demand: capacity ratios for the piers and spandrels at each joint:

$$S_i = \frac{\frac{\Sigma V^*_{u,Pier}}{\Sigma V_{n,Pier}}}{\frac{\Sigma V^*_{u,Spandrel}}{\Sigma V_{n,Spandrel}}} \quad \ldots C8.50$$

where:

$\Sigma V^*_{u,Pier}$ = sum of the 100%*NBS* shear force demands on the piers above and below the joint calculated using $K_R = 1.0$

$\Sigma V_{n,Pier}$ = sum of the piers' capacities above and below the joint

$\Sigma V^*_{u,Spandrel}$ = sum of the 100%*NBS* shear force demands on the spandrels to the left and right of the joint calculated using $K_R = 1.0$

$\Sigma V_{n,Spandrel}$ = sum of the spandrel capacities to the left and right of the joint.

When $S_i > 1.0$ a weak pier – strong spandrel mechanism may be expected to form, and when $S_i < 1.0$ a strong pier – weak spandrel mechanism may be expected to form.

Nonlinear analysis of URM piers and spandrels can be carried out using 2D plane stress elements or solid 3D elements. This method has the advantage that the stress and strains developed in the URM members/elements can be assessed directly and deformation compatibility is maintained. Compression-only gap elements can be included in the analysis model to account for pier rocking (Knox, 2012).

For URM walls with openings of differing sizes and relatively weaker piers compared to stronger spandrels, Moon et al (2004) have recommended that the effective height of each rocking pier is represented as the height over which a diagonal compression strut is most likely to develop in the pier at the steepest possible angle that would offer the least lateral resistance (refer to Figure C8.75). As a result, effective heights for some rocking piers adjacent to unequal size openings will vary depending upon the direction of loading. The angles to the piers generally depend on bed and head joint dimensions and stair-step cracking along mortar joints. If the diaphragms are rigid or reinforced concrete bands are provided, the effective height of the piers may be limited to the bottom of the diaphragm or the concrete band, as appropriate.

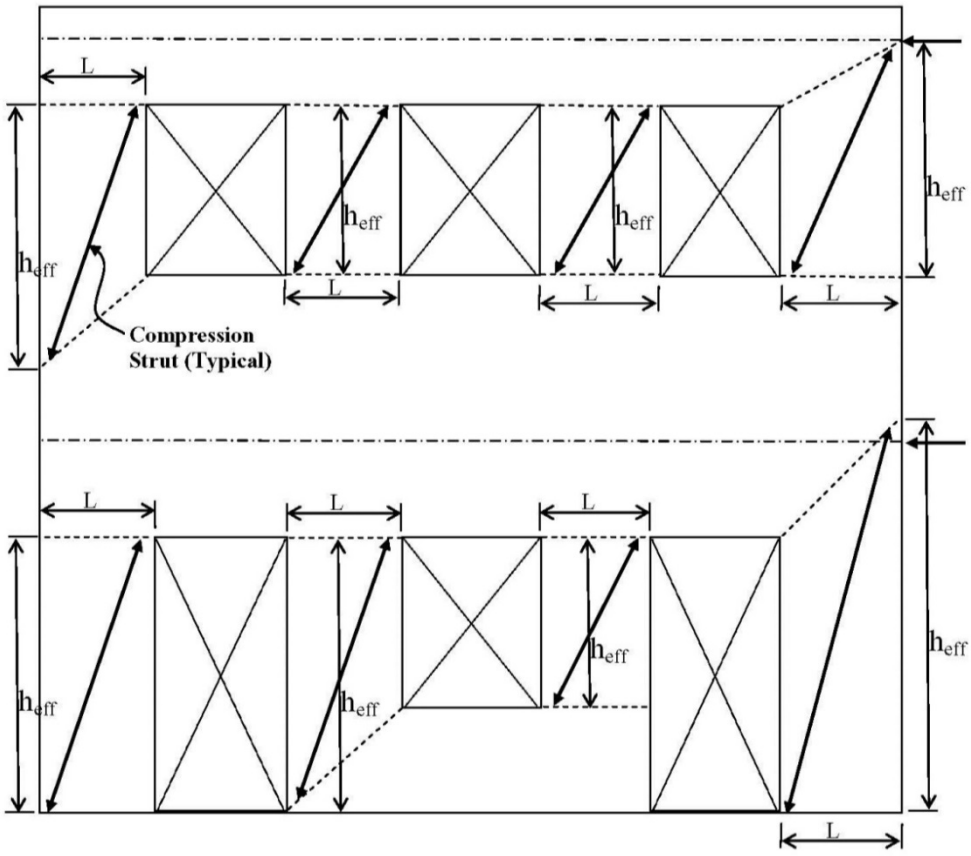


Figure C8.75: URM rocking pier effective heights based on development of diagonal compression struts that vary with direction of seismic force (ASCE 41-13, 2014)

The capacity of a penetrated wall element at a particular level can also be determined from the capacity (strength and deformation) of the individual wall/pier elements assuming that displacement compatibility must be maintained along the element and using the force deformation relationships defined above for the governing mode of behaviour of each element. This can also be extended to multiple levels if required, and the capacity of the whole wall determined if the engineer has some knowledge of the lateral load distribution with height. This can be considered a variant of the Simple Lateral Mechanism Analysis (SLaMA) approach described elsewhere in these guidelines.

C8.8.7 Other items of a secondary nature

Items of a secondary nature such as canopies and architectural features should be assessed for parts and components loads.

C8.9 Assessment of Global Capacity

C8.9.1 General

The global capacity of the building is the strength and deformation capacity of the building taken as a whole, ignoring the performance of secondary elements. For this purpose face-loaded masonry walls are considered to be secondary elements unless the wall is providing primary support to the building or building part; e.g. by cantilever action of the wall. Diaphragms distributing lateral shears between lateral load resisting elements (as opposed to providing support to face-loaded walls) are considered to be primary structure and therefore the capacity of these load paths through diaphragms and through connections from walls to diaphragms needs to be considered when assessing the global capacity.

The global capacity of the building is likely to be significantly influenced by the relative in-plane stiffness of the diaphragms compared with the in-plane lateral stiffness of the masonry walls. Timber and cross-braced steel diaphragms will typically be "flexible" in this sense and this allows simplifications to be made in the assessment of global capacity, as outlined below. Assuming high diaphragm stiffness where this is not assured can lead to erroneous assessment results; e.g. non-conservative assessments of diaphragm accelerations and inaccurate estimates of load distribution between lateral load resisting elements (Oliver, 2010). Flexible diaphragms can be explicitly modelled in 3D analysis models using linear or nonlinear 2D plane stress or shell elements, but care is required and the additional complexity will rarely be warranted for basic buildings. Well-proportioned concrete floor and roof slabs in small buildings may be assumed to be rigid.

Consideration of the nonlinear capacity of masonry members/elements is encouraged as it often leads to a higher global capacity than if the member/element capacities are limited to yield (elastic) levels. Consideration of nonlinear behaviour requires a displacement-based assessment approach. In many situations this is reasonably easy to implement and is recommended for the greater understanding of building seismic behaviour that it often provides.

When more than one lateral load mechanism is present, or when there are elements with varying strengths and stiffness, a displacement-based approach is considered essential to ensure displacement compatibility is achieved and the global capacity is not overstated. This is often the case for masonry buildings, particularly those that have been previously retrofitted with flexible and assumed ductile (low strength) systems.

When assessing the global capacity it will be necessary to complete an analysis of the building structure to assess the relationship between the individual member/element capacities and the global demands. Simple hand methods of analysis are encouraged in preference to overly sophisticated methods which may imply unrealistic transfers in shear between members/elements that will be difficult to achieve in practice and may go unrecognised in the assessment. When sophisticated analyses are used, it is recommended that simpler methods are also used to provide order of magnitude verification.

The objective of global capacity assessment is to find the highest globally applied load/displacement that is consistent with reaching the strength/deformation capacity in the most critical member/element. The recommended approach for URM buildings is described in Figure C8.76. The global strength capacity can be referred to in terms of base shear

capacity. The deformation capacity will be the lateral displacement at h_{eff} for the building consistent with the base shear capacity accounting for nonlinear behaviour as appropriate.

This section provides guidance on the assessment of the global capacity for both basic and complex buildings. It also provides guidance on methods of analysis and modelling parameters.

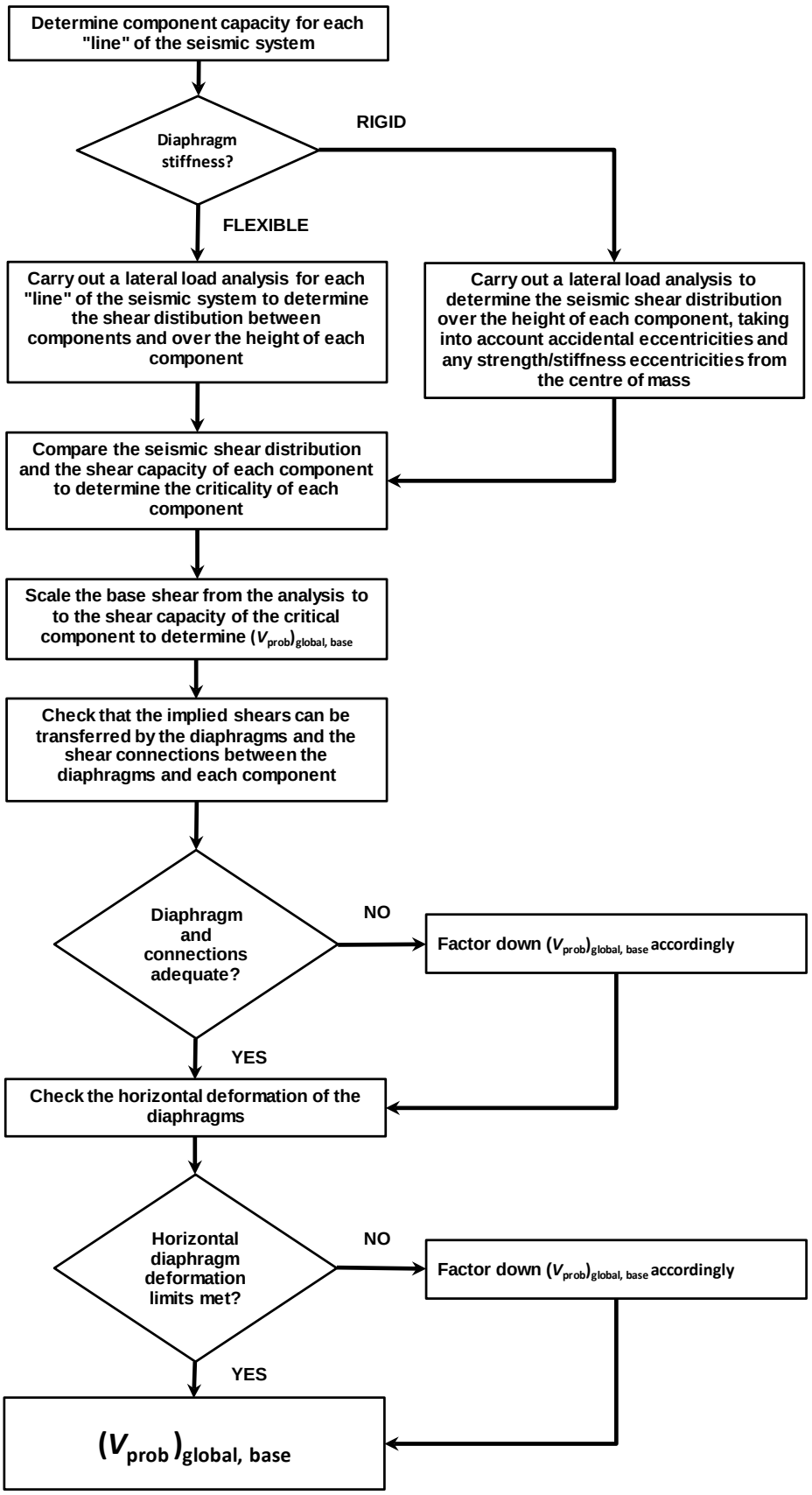


Figure C8.76: Global capacity assessment approach for URM buildings

C8.9.2 Global capacity of basic buildings

Determining the global capacity of basic URM buildings can be a simple exercise. Consider, for example, the single storey buildings shown in Figure C8.77. If the roof diaphragm is flexible the global capacity in each direction will be the lowest element capacity on any system line in that direction when there are only two system lines. When there are more than two system lines then the global capacity in a direction will be the capacity of the line in that direction which has the lowest value of V_{prob}/tributary mass, where V_{prob} in this context is the sum of the element probable capacities along the particular line of the seismic system.

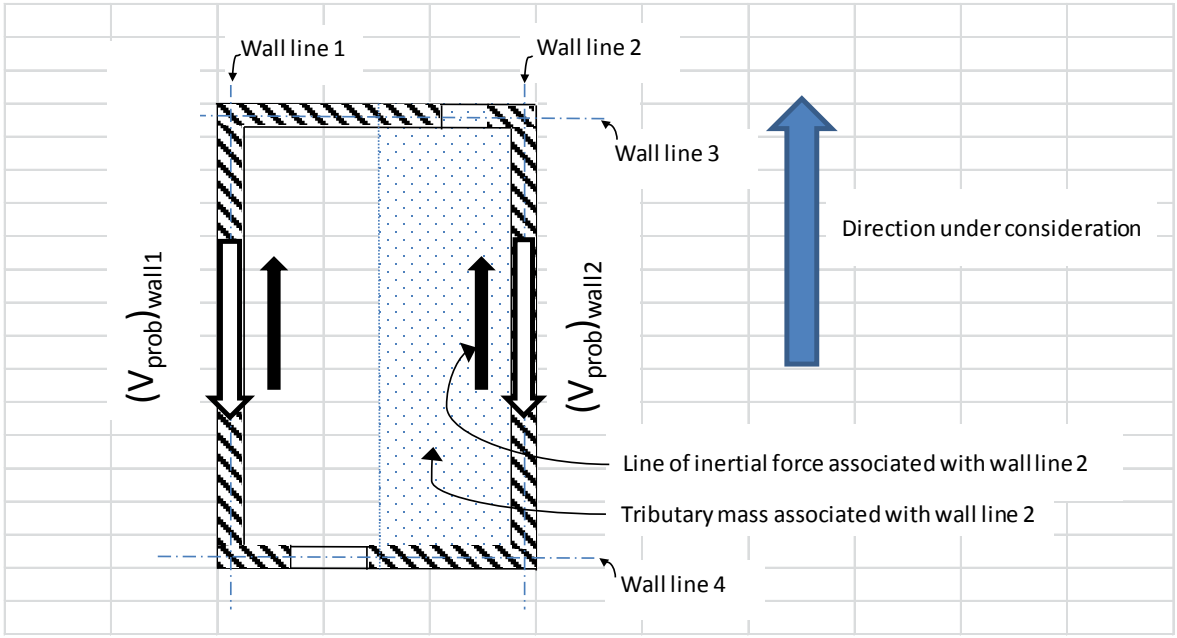


Figure C8.77: Relationship between demand and capacity for a basic building with a flexible diaphragm

For such buildings there would be little to gain from consideration of the nonlinear behaviour of the elements when determining the global capacity. However, an understanding of the nonlinear capability, without jeopardising the vertical load carrying capacity, will provide confidence that the building has resilience. If the demand is to be calculated in accordance with Section C8.10.2.2, nonlinear behaviour is assumed if K_{R} is greater than 1.

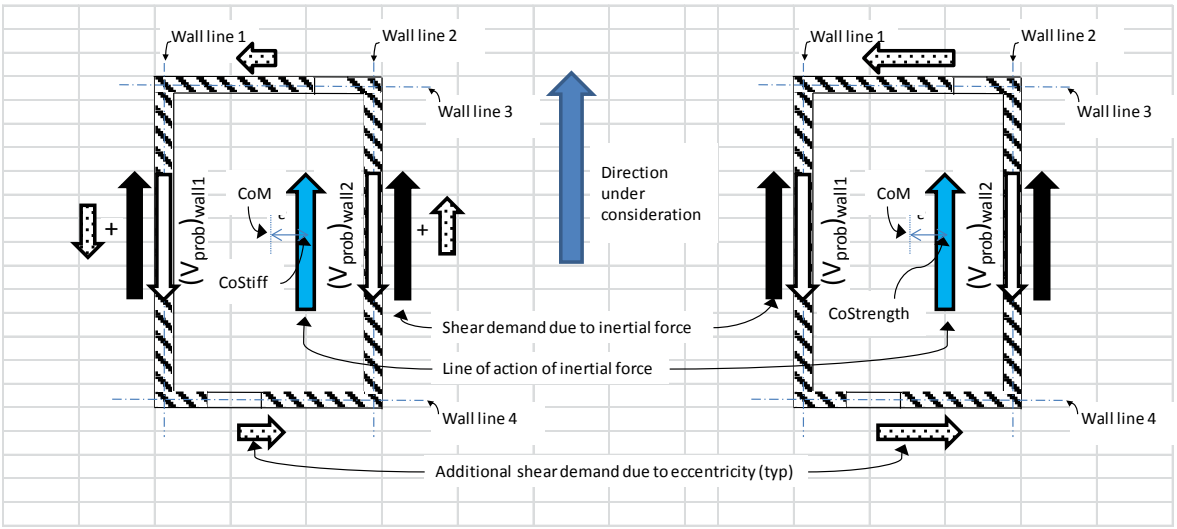
Some small buildings with flexible diaphragms will not have identifiable or effective lateral load paths to provide lateral resistance to all parts of the building. An example of this is the open front commercial building where the sole means of lateral support might be cantilever action of the ends of the side walls, the capacity of which will be highly dependent on the restraint available from the wall foundation, and likely to be negligible.

Basic buildings of two or three storeys with flexible diaphragms can be considered in a similar fashion, after first completing a simple analysis to determine the variation in shear over the height of each line of the seismic system. The global capacity of such buildings will be limited to the capacity of the line where $(V_{\text{prob}})_{\text{line,i}}/\beta_{\text{i}}$ is the lowest. $(V_{\text{prob}})_{\text{line,i}}$ is the sum of the element capacities along a line of the seismic system at level i and β_{i} is the ratio of the applied shear at level i to the shear at the base of the line under consideration. For most basic buildings β_{i} will be the same for all lines of the seismic system.

The presence of rigid diaphragms in basic buildings introduces an additional level of complexity into the building analysis. However, this analysis can still be kept quite simple for many buildings.

For buildings with rigid diaphragms it will be necessary to consider the effect of the demand and resistance eccentricities (accidental displacement of the seismic floor mass and the location of the centre of stiffness or strength as appropriate). Refer to Figure C8.78. If the lines of the seismic system in the direction being considered have some nonlinear capability it is considered acceptable to resist the torque resulting from the eccentricities solely by the couple available from the lines of the seismic system perpendicular to the direction of loading. This will lead to a higher global capacity in many buildings than would otherwise be the case. If this approach is to be followed it would be more appropriate to consider the centre of strength rather than the centre of stiffness when evaluating the eccentricities.

NZS 1170.5:2004 requires that buildings not incorporating capacity design are subjected to a lateral action set comprising 100% of the specified earthquake actions in one direction plus 30% of the specified earthquake actions in the orthogonal direction. The 30% actions perpendicular to the direction under consideration are not shown in Figure C8.78 for clarity and, suitably distributed, would need to be added to the shears to be checked for the perpendicular walls. These are unlikely to be critical for basic buildings. If the diaphragm is flexible, concurrency of the lateral actions should be ignored.



a) Linear elastic **b) With nonlinear capability on wall lines 1 and 2**

Figure C8.78: Relationship between demand and capacity for a basic building with rigid diaphragms

In the above discussion it has been assumed that the diaphragms are stiff enough to provide the required support to the face-loaded walls orientated perpendicular to the direction of loading. Diaphragms are considered as primary structural elements for the transfer of these actions and their ability to do so may affect the global capacity of the building in that component direction. Limits have been suggested in Section C8.8.3.2 for the maximum diaphragm deflections to ensure adequate wall support. These limits are likely to be exceeded in flexible diaphragms, even in small basic buildings, and should be checked. If the limits are exceeded, the global capacity of the building in that direction will need to be reduced accordingly.

C8.9.3 Global capacity of complex buildings

Many complex URM buildings will be able to be assessed adapting the recommendations outlined above for basic buildings. However, the assessment of complex buildings will often require a first-principles approach and a good understanding of the past performance of such buildings.

The overall objective discussed in Section C8.9.1 remains. However, the more complex the building the more likely it will be necessary to utilise more complicated analysis techniques simply to keep track of element actions and applied inertial forces. It is recommended that simple techniques be used in all cases to identify the primary load paths and to verify the order of magnitude of the outputs.

Use of linear-elastic analysis techniques and limiting member/element capacities to elastic behaviour may significantly underestimate the global capacity of complex buildings. However, nonlinear considerations can completely alter the mechanisms that can occur.

Aspects that are likely to require specific consideration in the assessment of complex buildings include:

- foundation stiffness
- diaphragm stiffness
- nonlinear behaviour of multi-storey, penetrated walls and development of sway mechanisms
- potential soft storeys
- non-horizontal diaphragms.

C8.9.4 Global analysis

C8.9.4.1 Selection of analysis methods

Four analysis methods are generally considered:

- equivalent static analysis (linear static)
- modal response analysis (linear dynamic)
- nonlinear pushover (nonlinear static)
- nonlinear time history (nonlinear dynamic).

Linear analysis techniques supplemented with simple nonlinear techniques (e.g. adapted SLaMA) are likely to be appropriate for all but the most complex of New Zealand's URM buildings.

Nonlinear analysis techniques are appropriate for buildings which contain irregularities and when higher levels of nonlinear behaviour are anticipated. If nonlinear pushover analysis procedures are used, include appropriate allowances in the analysis for anticipated cyclic strength and stiffness degradation.

Nonlinear time history analyses can be used to analyse most URM buildings. They are able to account explicitly for cyclic strength and stiffness degradation. These analyses are complex. They should not be undertaken lightly and then only by those that have experience in the processes involved. A full appreciation of the reliability of the input parameters and

the likely sensitivity of the outputs to these is required. Refer to relevant references for nonlinear acceptance criteria.

Note:

Nonlinear modelling of URM walls is feasible, but experience to date suggests that analytical results will not always provide reliable estimates of performance because of the variability in actual material strength and condition. Any analytical modelling should include several analyses to test sensitivity to material variation, modelling method and earthquake motion.

Special care is required with the application of damping, especially when considering a mix of low and high period modes. The resulting force reduction from damping for the mode considered should be investigated by a special study for finite element analysis. For assessing URM buildings, Caughey damping rather than Raleigh damping should be considered.

C8.9.4.2 Mathematical modelling

Mathematical models used for linear analysis techniques should include the elastic, un-cracked in-plane stiffness of the primary lateral load resisting elements. Consider both shear and flexural deformations.

If using nonlinear analysis techniques, the mathematical model should directly incorporate the nonlinear load-deformation characteristics of individual in-plane elements (i.e. backbone curves). Include cyclic degradation of strength and stiffness in the member modelling when appropriate. Recommended nonlinear analysis parameters for non-brittle URM failure modes are given in Section C8.8.6.2.

C8.9.4.3 Fundamental period

The mass of URM buildings is normally dominated by the mass of the masonry. However, stiffness will depend on the relative flexibility of the walls, the floor diaphragms and the ground (foundation rotation). While the period of these structures can be quite difficult to calculate with precision and there are several modes of vibration to consider, it will often fall within the plateau section of the spectra, so precision is not required. For larger buildings (tall or long), especially those with long flexible diaphragms, special consideration of these effects may be required.

In the case of large buildings, it may not be sufficient to consider all parts of the building loaded at the same time and having the same time period. Commonly used methods include sub-structuring: i.e. subdividing the structure into sections, each including its elements and all mass tributary to it. Each section is then analysed separately and checked for compatibility with neighbouring sections along the margins between the sections. These sections should typically be no more than one third of the building width or more than 30 m.

Note:

The effective period of individual sections of URM buildings will often still be short and, if this is the case, this final step will not be required.

C8.9.4.4 Seismic mass

URM buildings are essentially systems with mass distributed over the height, with barely 10-20% of the seismic mass contributed by floors and roof. This is especially the case for buildings with timber floors and lightweight roofs. In this context, the concept of a lumped mass system is problematic. However, unless a more sophisticated analysis has been undertaken to capture the effect of distributed mass systems, an assessment based on masses lumped at diaphragm levels is acceptable as loads from the face-loaded walls would be transferred to the in-plane walls through the diaphragm.

However, for shear checks at the base of the in-plane walls and piers of any storey, the seismic demand should include accumulated floor level forces from the upper storeys and the seismic force due to the total mass of the in-plane wall above the level being considered. This is in contrast to assessments of concrete construction, where the mass of the lower half of the bottom storey is ignored when estimating the active mass for the base shear.

C8.9.4.5 Stiffness of URM walls and wall piers subject to in-plane actions

The stiffness of in-plane URM walls subjected to seismic loads should be determined considering flexural, shear and axial deformations. The masonry should be considered to be a homogeneous material for stiffness computations with an expected elastic modulus in compression, E_{m}, as discussed in earlier sections.

For elastic analysis, the stiffness of an in-plane URM wall and pier should be considered to be linear and proportional with the geometrical properties of the un-cracked section, excluding any wythe that does not meet the criteria given in Section C8.2.4.3.

Laboratory tests of solid shear walls have shown that behaviour can be depicted at low force levels using conventional principles of mechanics for homogeneous materials. In such cases, the lateral in-plane stiffness of a solid cantilevered wall, k, can be calculated using Equation C8.51:

$$k = \frac{1}{h_{\mathrm{eff}}^3 / 3E_{\mathrm{m}}I_{\mathrm{g}} + h_{\mathrm{eff}} / A_{\mathrm{n}}G_{\mathrm{m}}} \qquad \text{…C8.51}$$

where:

h_{eff} = wall height, mm
A_{n} = net plan area of wall, mm^2
I_{g} = moment of inertia for the gross section representing uncracked behaviour, mm^4
E_{m} = masonry elastic modulus, MPa
G_{m} = masonry shear modulus, MPa.

The lateral in-plane stiffness of a pier between openings with full restraint against rotation at its top and bottom can be calculated using Equation C8.52:

$$k = \frac{1}{h_{\mathrm{eff}}^3 / 12E_{\mathrm{m}}I_{\mathrm{g}} + h_{\mathrm{eff}} / A_{\mathrm{n}}G_{\mathrm{m}}} \qquad \text{…C8.52}$$

Note that a completely fixed condition is often not present in actual buildings.
Equation C8.52 could be used to estimate spandrel stiffness.

C8.10 Assessment of Earthquake Force and Displacement Demands

C8.10.1 General

This section sets out the procedures for estimating both force and displacement demands on URM buildings and their parts.

Section C3 describes how the earthquake demands are to be assessed.

For the purposes of defining seismic demands, the structural system which carries seismic load and provides lateral resistance to the global building should be considered the primary seismic resisting system (primary structure). The members/elements which do not participate in the overall lateral resistance of the structure and which rely on the primary structure for strength and/or stability should be assumed to be parts and components. Parts and components need to be assessed for any imposed deformations from the primary seismic resisting system.

Therefore all in-plane walls and diaphragms are classified as primary lateral structure. Everything else, such as face-loaded walls and parapets, and ornamentation, are considered to be secondary structure, and where appropriate, critical non-structural items.

C8.10.2 Primary lateral structure

C8.10.2.1 General

Determine the horizontal demands on the primary lateral structure, in accordance with Section C3 taking $\mu = 1$, $S_p = 1$ and $\xi_{sys} = 15\%$. Although μ is set at 1 it is intended that the benefits of any nonlinear deformations from the assessment of the capacity are also taken.

Note:

The use of 15% damping accounts for a number of factors including low likelihood of resonance between elements in a building or the building as a whole, and additional damping from flexible diaphragms, radiation damping and localised damage. Where these phenomena are not present in a mechanism, lower levels of damping may need to be considered. For example, the monument shown in Table C8.1 ("monumental - single form"), when the base width is small compared with the height, is likely to exhibit clear single-degree-of-freedom rocking with minimal interference by other mechanisms and so lower levels of damping values, even less than 5%, may need to be considered.

It should be noted that if building response is likely to be governed by diagonal tension failure, damping should be limited to 5%, unless capacity of those wall or pier is ignored from total capacity, but consequences of loss of gravity load support from these walls/piers does not cause instability to any of the structure above.

When required, a triangular distribution of earthquake load demands over the height of the structure may be assumed, without allowance for additional demand at the top of the structure.

Note:

The additional force required to be distributed to the top of the structure using the equivalent static horizontal force distribution determined from NZS 1170.5:2004 is considered to be too conservative for stiff URM buildings where higher mode effects are likely to be insignificant.

C8.10.2.2 Basic buildings

For basic buildings, as defined in Section C8.2.2, a force-based assessment of in-plane demands for walls/piers and spandrels, for each line of resistance, may be determined using a horizontal demand seismic coefficient, $C(T_1)$, given by Equation C8.53 where a load reduction factor, K_{R}, has been used in lieu of the ratio of the structural performance factor and structural ductility factor given in NZS 1170.5:2004.

C8.53

$$C(T_1) = C_{\mathrm{h}}(T_1)\, Z\, R_{\mathrm{u}}\, N(T_1, D)/K_{\mathrm{R}} \quad \ldots$$

where:

$C_{\mathrm{h}}(T_1)$ = the spectral shape factor determined from Clause 3.1.2, NZS 1170.5:2004 for the first mode period of the walls/piers making up the line of resistance, T_1, g.

Lines of resistance in basic buildings will typically have a short period, within the plateau region of the spectral shape factor plot, which means the calculation of the period can often be avoided.

Z = the hazard factor determined from Clause 3.1.4, NZS 1170.5:2004

R_{u} = the return period factor, R_{u} determined from Clause 3.1.5, NZS 1170.5:2004

$N(T_1, D)$ = the near fault factor determined from Clause 3.1.6, NZS 1170.5:2004

K_{R} = the seismic force reduction factor determined from Table C8.15 for each line of resistance.

Note:

The horizontal design co efficient for basic buildings from Equation C8.53 is based on the damping allowance of 5% defined by NZS 1170.5:2004 rather than the general allowance of 15% given in Section C8.10.2.1. This is because the performance enhancement effects that justify the use of the higher damping in the general case are allowed for separately and explicitly in the K_{R} factor.

As a defined characteristic of basic buildings is flexible diaphragms each line of resistance can be individually assessed, ignoring (where reasonable) the stiffness or failure modes of an adjacent line.

Table C8.15: Recommended force reduction factors for linear static method

Seismic performance/controlling parameters	Force reduction factor, K_R	Notes
Pier rocking, bed-joint sliding, stair-step failure modes	3	Failure dominated by strong brick-weak mortar
Pier toe failure modes	1.5	
Pier diagonal tension failure modes (dominated by brick splitting)	1.0	Failure dominated by weak brick-strong mortar
Spandrel failure modes	1.0	The spandrels need not be assessed as outlined below.

Note:

The concept of a ductility factor (deflection at ultimate load divided by the elastic deflection) can be meaningless for most URM buildings. The introduction of K_R primarily reflects an increase in the damping available and therefore reduced elastic response rather than ductile capability assessed by traditional means. Therefore the displacements calculated from the application of $C(T_1)$ are the expected displacements and should not be further modified by K_R.

These force reduction factors apply in addition to relief from period shift (if any).

Redistribution of seismic demands between individual elements of up to 50% within a line of resistance is permitted when $K_R = 3.0$ applies, provided that the effects of redistribution are accounted for in the analysis.

Engineering judgement should be used when an element's mechanisms are close in capacity as to the consequence of the assessment being inaccurate: if diagonal tension failure were to occur and cause either a significant reduction in capacity for the building, or cause a loss of gravity support to an area of the building, the more conservative $K_R = 1.0$ should be adopted. The designer must keep in mind the highly variable nature of the material and the approximations made in the estimates of material strengths.

When there are mixed behaviour modes among the walls/piers in a line of resistance, the engineer must take the mechanism with the lowest K_R factor to define the K_R factor for that line as a whole. Alternatively, the capacity of any piers for which K_R is less than the value that has been adopted for the line of resistance can be ignored; but only if the consequences of loss of gravity load support from these walls/piers does not cause instability to any of the structure above.

If there are mixed failure modes among the walls and piers in a line of resistance, the displacement compatibility between these piers and walls should be evaluated.

For the case of perforated walls when a strong pier – weak spandrel mechanism governs the wall behaviour $K_R = 1.0$ shall be adopted for the wall line as a whole and the capacity of the line of resistance is governed by the spandrel capacity. Alternatively, the capacities of the spandrels can be ignored and the higher K_R factors detailed in Table C8.15 used for the remaining (taller) simple rocking pier members provided the consequences of loss/collapse of the ignored spandrels are considered. When a weak pier-strong spandrel mechanism governs, a "pier only" analysis can be used and the higher than unity K_R factor given in Table C8.15 can be adopted.

C8.10.3 Secondary and critical non-structural items

Refer to Section 8 of NZS 1170.5:2004 for determination of seismic demands on secondary and critical non-structural items.

For face-loaded walls, assessed using the forced-based or displacement-based method in Section C8.8.5, the demands are included within the method. Note that for the displacement-based approach, the Part Spectral Shape Coefficient, $C_i(T_p)$, defined in NZS 1170.5:2004 has been replaced with a formulation that better converts into a displacement spectrum for this purpose.

C8.10.4 Vertical demands

Vertical ground motions in close proximity to earthquake sources can be substantial. However, opinion is divided on how significant vertical accelerations are on the performance of URM buildings.

While vertical ground accelerations could potentially reduce the gravity and compression forces in the walls, reducing their stability and reducing the pull-out strength of ties installed to restrain them back to the diaphragms, there is evidence to suggest that there is typically a time delay between the maximum vertical accelerations and the maximum horizontal accelerations, meaning that they are unlikely act together at full intensity.

In advance of further investigations on this subject, it is considered reasonable to ignore vertical accelerations when assessing the stability of masonry walls and the capacity of embedded anchors.

When vertical accelerations are considered the demands may be determined from NZS 1170.5:2004.

C8.10.5 Flexible diaphragms

C8.10.5.1 General

Masonry walls loaded in-plane are typically relatively rigid structural elements. Consequently, the dominant mode of response for buildings containing flexible diaphragms is likely to be the response of the diaphragms themselves, due to inertial forces from diaphragm self-weight and the connected URM boundary walls responding out-of-plane.

Note:

Flexible diaphragms in the context of URM buildings and these guidelines are those constructed of timber or which are steel braced.

Concrete diaphragms can be assumed to be rigid. A concrete diaphragm with large penetrations could be relatively flexible compared with the supporting walls. ASCE 41-13 (2014) provides a procedure for checking the relative stiffness should this be of concern.

Seismic demands on flexible diaphragms in URM buildings which are braced by URM walls should, therefore, be based on the period of the diaphragm and a horizontal seismic coefficient assuming that the diaphragm is supported at ground level (i.e. no amplification

to reflect its height in the building). The seismic coefficient to be used is therefore $C(T)$ from NZS 1170.5:2004, where T is the first horizontal mode period of the diaphragm. If the diaphragm is able to behave in a ductile fashion (e.g. steel bracing with connection capacities exceeding the overstrength capacity of the brace) μ of up to 3 may be assumed. Otherwise, μ should be taken as 1. The value of S_{p} should be in accordance with the ductile capability of the diaphragm.

If the diaphragm is braced by flexible lateral load resisting elements (i.e. non-URM or short URM walls), the seismic demands can be determined using a seismic coefficient equal to $F_{\mathrm{i}}/m_{\mathrm{i}}$, with a lower limit of $C(0)$ where F_{i} is the equivalent static horizontal force determined from NZS 1170.5:2004 at the level of the diaphragm (assuming 5% damping) and m_{i} is the seismic mass at that level. This is the pseudo-Equivalent Static Analysis (pESA) method outlined in Section C2.

C8.10.5.2 Timber diaphragms

The diaphragm in-plane mid-span lateral displacement demand, Δ_{d}, is given by Equation C8.54.

$$\Delta_{\mathrm{d}}\ (\mathrm{m}) = \frac{3}{16}\frac{C(T_{\mathrm{d}})W_{\mathrm{trib}}L}{BG'_{\mathrm{d,eff}}} \qquad \text{...C8.54}$$

where:

$C(T_{\mathrm{d}})$ = seismic coefficient at required height for period, T_{d}, determined in accordance with Section C8.10.5.1
W_{trib} = uniformly distributed tributary weight, kN
L = span of diaphragm, m
B = depth of diaphragm, m
$G'_{\mathrm{d,eff}}$ = effective shear stiffness of diaphragm, refer to Equation C8.55, kN/m
T_{d} = lateral first mode period of the diaphragm determined in accordance with Equation C8.55, sec.

The period, T_{d}, of a timber diaphragm, based on the deformation profile of a shear beam excited in an approximately parabolic distribution, is given by Equation C8.55 (Wilson et al., 2013c).

$$T_{\mathrm{d}}(sec) = 0.7 \times \sqrt{\frac{W_{\mathrm{trib}}L}{G'_{\mathrm{d,eff}}B}} \qquad \text{…C8.55}$$

where:

W_{trib} = total tributary weight acting on the diaphragm, being the sum of the weight of the tributary face-loaded walls both half-storey below and above the diaphragm being considered (i.e. the product of the tributary height, thickness and density of the out-of-plane URM walls tributary to the diaphragm accounting for wall penetrations) and diaphragm self-weight plus live load ($\psi_{\mathrm{E}}\, x\, Q_{\mathrm{i}}$ as per NZS 1170.5:2004 Section 4.2).

Other terms are as defined for Equation C8.54.

0.7 has units of $1/\sqrt{g}$.

C8.10.6 Rigid diaphragms

Rigid diaphragms are primary structure and the demands are determined in accordance with NZS 1170.5:2004 as outlined in Section C8.10.2. If required, floor acceleration demands should be assessed as indicated in Section C8.10.5.1 however, damping should be limited to 5% for flexible diaphragms supported by flexible lateral load resisting systems.

C8.10.7 Connections providing support to face-loaded walls

The demands on connections providing support to face-loaded masonry walls shall be calculated in accordance with Steps 12, 13 and 14 in Section C8.8.5.2.

Assume that the demand is uniformly distributed across all anchorages located at the specific wall-diaphragm interface. Repeat the exercise for the orthogonal loading direction, reversing loading regimes for a given anchorage.

C8.10.8 Connections transferring diaphragm shear loads

Wall-diaphragm connections required to transfer shears from diaphragms to walls (loaded in-plane) should be considered to be primary structure and therefore the demands are evaluated in accordance with Section C8.10.2. The demand may be assumed to be uniformly distributed along the wall to diaphragm connection.

Unless capacity design principles are applied, the demands should be assessed assuming $\mu = S_{\mathrm{p}} = 1$.

C8.11 Assessment of *%NBS*

The assessment of the *%NBS* earthquake rating for the building should be in accordance with Section C1.

C8.12 Improving Seismic Performance of URM Buildings

The overarching problem is that New Zealand's URM building stock is simply not designed for earthquake loads and lacks a basic degree of connection between structural elements to allow all parts of the building to act together (Goodwin et al., 2011).

The basic approach to improving the seismic performance of URM buildings is to:

- secure all unrestrained parts that represent falling hazards to the public (e.g. chimneys, parapets and ornaments)
- improve the wall-diaphragm connections or provide alternative load paths; improve the diaphragm; and improve the performance of the face-loaded walls (gables, facades and other walls) by improving the configuration of the building and in-plane walls
- strengthen specific structural elements, and
- consider adding new structural components to provide extra support for the building.

When developing strengthening options, note that differing levels of seismic hazard will mean that a solution advised in a high seismic area could be too conservative in a low seismic area. Also note that even though a building may have more than 34*%NBS* seismic capacity, if that is limited by a brittle mode of failure and/or the failure mode could trigger a sequence of failure of other elements, the risk of failure of the limiting element should be carefully assessed and mitigated.

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Appendix C8A: On-site Testing

C8A.1 General

While the seismic response of URM buildings is significantly influenced by characteristics such as boundary conditions and the behaviour of inter-element connections, on-site testing of material properties improves the reliability of the seismic assessments and the numerical models that describe the seismic behaviour of URM buildings, and it may lead to less conservative retrofit designs. However, the non-homogenous nature of masonry combined with the age of URM buildings make it difficult to reliably predict the material properties of masonry walls.

It is recommended that field sampling or field testing of URM elements is conducted. Field sampling refers to the extraction of samples from an existing building for subsequent testing offsite, while field testing refers to testing for material properties in situ. The following sections describe a set of techniques that can be used to determine masonry material properties.

Before proceeding to on-site testing, it is important to sensibly understand what information will be collected from the investigation, how that would be used and what value the information will add to reliability of the assessment. Before deciding an investigation programme, sensitivity analyses should be undertaken to determine what assessment parameters are more important and likely to influence the assessment result and whether the default parameter values given are likely to be appropriate/sufficient.

Only rarely should on-site testing be considered necessary for basic buildings.

C8A.2 Masonry Assemblage (Prism) Material Properties

If masonry assemblage (prism) samples are to be extracted for laboratory testing they should be single leaf and at least three bricks high. If they are two leafs thick or more, cut them into single leaf samples. If rendering plaster is present, remove this from both sides of the samples. Cap the prepared samples using gypsum plaster to ensure uniform stress distribution.

Test individual brick units and mortar samples as per Section C8A.3 when sampling of larger assemblages is not permitted or practical. Masonry properties can then be predicted using the obtained brick and mortar properties as set out in Section C8.7.

C8A.2.1 Masonry compressive strength

Determine masonry compressive strength in accordance with ASTM C 1314-03b (ASTM, 2003c). Figure C8A.1 shows a typical prism sample before testing. Aluminium frames are attached to the sample ends and a displacement gauge spans between the frames to measure the sample displacement.

ASTM C 1314-03b (ASTM, 2003c) also enables the engineer to determine the masonry modulus of elasticity (further detailed in Section C8A.2.2).

Figure C8A.1: Example of extracted sample with test rig attached for the prism compression test

C8A.2.2 Masonry modulus of elasticity

Laboratory calibrated displacement measurement

Laboratory calibrated displacement measurement devices may be attached to the masonry prisms during the compression tests detailed in Section C8A.2.1. Incorporate a minimum of two measurement devices to record displacements at opposing sample faces. Their gauge lengths should cover the distance from the middle of the top brick to the middle of the bottom brick. Use the recorded measurement to derive the masonry stress-strain relationship and subsequently the masonry modulus of elasticity, E_{m}. The stress and strain values considered in the calculation of E_{m} are those between 0.05 and 0.70 times the masonry compressive strength (f'_{m}).

In situ deformability test incorporating flat jacks

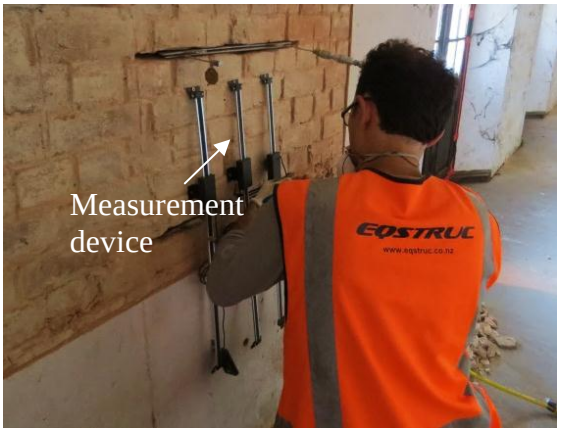
Flat jack testing is a versatile and effective technique that provides useful information on the mechanical properties of historical constructions. In situ measurements of masonry modulus of elasticity should be performed in accordance with the ASTM C 1197-04 (ASTM, 2004) in situ deformability test.

Note:

Extensive studies have been conducted to confirm the reliability of this test, including the work by Noland et al. (1991), Gregorczyk and Lourenço (2000); Parivallal et al. (2011); and Simões et al. (2012).

The in situ deformability test is moderately destructive as it requires the removal of horizontal mortar joints (bed-joint) for the insertion of the two flat jacks (refer to Figure C8A.2(a)). The horizontal slots are separated by at least five courses of brickwork, but the separation distance should not exceed 1.5 times the flat jack length. A pressure controlled hydraulic pump is used to inflate the flat jacks, applying vertical confinement pressure to the masonry between the two jacks. To monitor displacement, typically three measurement devices are attached between the two flat jacks (refer to Figure C8A.2(b)). These flat jacks need to be calibrated, following ASTM C 1197-04 (ASTM, 2004).

(a) Cutting mortar bed-joints and insertion of flat jacks into clay brick masonry

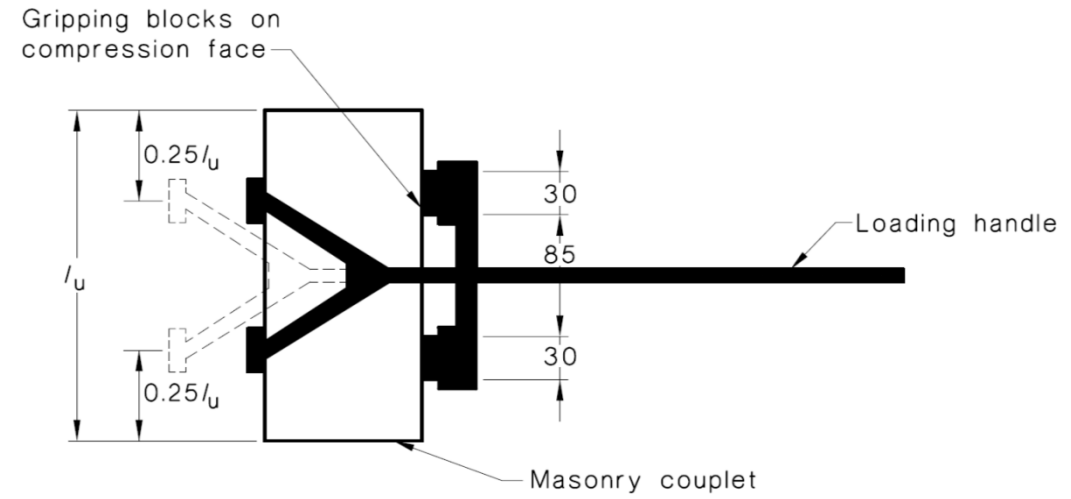


(b) In situ deformability test set-up under preparation in clay brick masonry

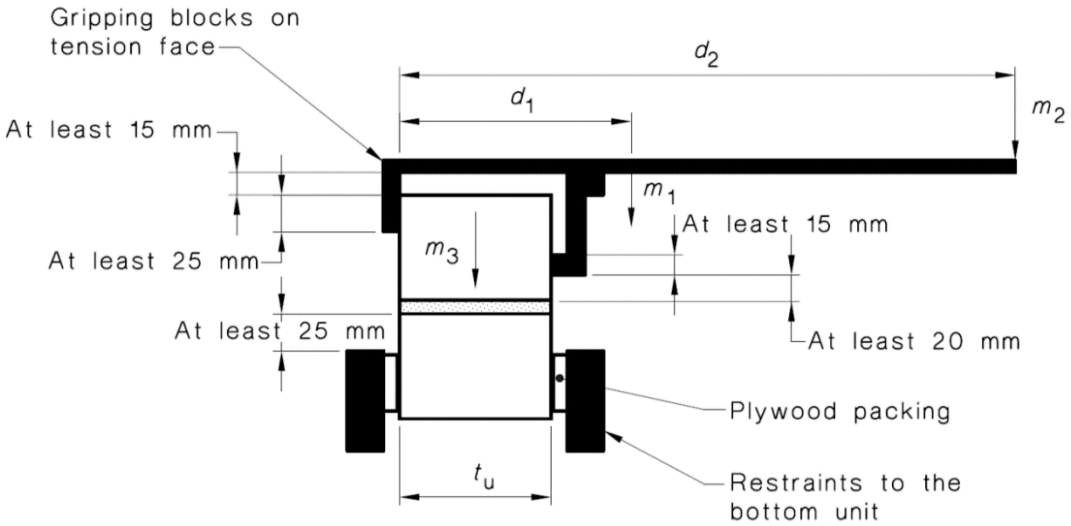
Figure C8A.2: In situ deformability test preparation (EQ STRUC Ltd)

C8A.2.3 Masonry flexural bond strength

Extract masonry prisms two bricks high and a single brick wide, and subject these to the flexural bond test of AS 3700-2001 (Australian Standards, 2001). Remove any rendering plaster from the sides of the sample before performing this test. Cut any samples that are two leafs thick or more into single leaf masonry prism samples. Alternatively, the engineer may conduct the flexural bond test in situ if this is more practical.



(a) Plan view



(b) Elevation view

Figure C8A.3: Flexural bond test set-up (AS 3700-2001)

C8A.2.4 Masonry bed-joint shear strength

Conduct the ASTM C 1531-03 (ASTM, 2003b) in situ bed-joint shear test to determine masonry bed-joint properties. This type of test is moderately destructive as it requires the removal of at least one brick on one side of the test specimen to allow for insertion of a hydraulic jack, as well as the removal of a vertical mortar joint on the opposite side to allow horizontal bed-joint movement to occur. The hydraulic jack is then loaded, using a pressure controlled hydraulic pump, until visible bed-joint sliding failure occurred. The bed-joint shear strength can then be derived from the peak pressure records.

Alternatively, extract three brick high masonry prisms for laboratory testing following the triplet shear test BS EN 1052-3 (BSI, 2002). This test should be conducted while applying axial compression loads of approximately 0.2 MPa, 0.4 MPa and 0.6 MPa. At least three masonry prism samples should be tested at each level of axial compression. Remove any rendering plaster from both sides of the sample before testing. Cut any masonry samples that are two leafs thick or more into single leaf samples. Bed-joint shear tests performed in the laboratory and in situ are shown in Figure C8A.4.

(a) Laboratory shear triplet test

(b) In situ shear test without flat jacks (EQ STRUC Ltd)

Figure C8A.4: In situ and laboratory bed- joint shear test

The in-situ bed-joint shear test is limited to tests of the masonry face leaf. When the masonry unit is pushed in a direction parallel to the bed joint, shear resistance is provided across not only the bed-joint shear planes but also the collar joint shear plane. Because seismic shear is not transferred across the collar joint in a multi-leaf masonry wall, the estimated shear resistance of the collar joint must be deducted from the test values. This reduction is achieved by including a 0.75 reduction factor in Equation C8.33, which is the ratio of the areas of the top and bottom bed joints to the sum of the areas of the bed and collar joints for a typical clay masonry unit.

The term P in Equation C8.33 represents the axial overburden acting on the bed joints. This value multiplied by the bed-joint coefficient of friction, (μ_f), allows estimation of the frictional component contributing to the recorded bed-joint stress. Due to the typically large variation of results obtained from individual bed-joint shear strength tests, the equation conservatively assumes $\mu_f = 1.0$ for the purposes of determining cohesion, c. Therefore, for simplicity, the μ_f term has been omitted from the equation.

C8A.3 Constituent Material Properties

C8A.3.1 Brick compressive strength

Extract individual brick units for the ASTM C 67-03a (ASTM, 2003a) half brick compression test. Cut these brick units into halves and cap them using gypsum plaster before compression testing (refer to Figure C8A.5). Note that it is possible to obtain half brick units from the residual samples of the Modulus of Rupture test described in Section C8A.3.2.

Figure C8A.5: Brick and mortar sample and compression test set-up (EQ STRUC Ltd)

C8A.3.2 Brick modulus of rupture

Extract individual brick units from the building and subject these to the Modulus of Rupture (MoR) test ASTM C 67-03a (ASTM, 2003a). The tested brick specimens from the MoR test may be subjected to the half brick compression test ASTM C 67-03a (ASTM, 2003a) in order to obtain a direct relationship between the brick MoR and compressive strength, f'_{b}. Previous experimental investigation has confirmed that the brick unit MoR can be approximated to equal $0.12f'_{\mathrm{b}}$.

C8A.3.3 Mortar compressive strength

Extract irregular mortar samples for laboratory testing. As it is common for URM walls to have eroded mortar joints that were later repaired using stronger mortar, take care when selecting the location for mortar sample extraction to ensure that these samples are representative.

The method to determine mortar compressive strength is detailed in ASTM C 109-08 (ASTM, 2008). This method involves testing 50 mm cube mortar samples, which generally are not attainable in existing buildings as most mortar joints are only 10 to 18 mm thick. Therefore, cut the irregular mortar samples into approximately cubical sizes with two parallel sides (top and bottom). The height of the mortar samples should exceed 15 mm in order to satisfactorily maintain the proportion between sample size and the maximum aggregate size. Cap the prepared samples using gypsum plaster to ensure a uniform stress distribution and testing in compression (Valek and Veiga, 2005). Refer to Figure C8A.6 for examples.

Measure the height to minimum lateral dimension (h/t) ratio of the mortar samples and use this to determine the mortar compressive strength correction factors. Divide the compression test result by the corresponding correction factors in Equation C8A.1. The average corrected strength is equal to the average mortar compressive strength, f'_{j}.

$$f'_{\mathrm{j}} = \alpha_{\mathrm{tl}}\alpha_{\mathrm{ht}}f'_{\mathrm{ji}} \qquad \text{…C8A.1}$$

where:

f'_{j} = normalised mortar compressive strength
α_{tl} = t/l ratio correction factor
α_{ht} = t/l ratio correction factor
f'_{ji} = measured irregular mortar compressive strength.

Equation C8A.1 normalises the measured compressive strength of irregular mortar samples to the compressive strength of a 50 mm cube mortar. Factors α_{tl} and α_{ht} are calculated as per Equations C8A.2 and C8A.3 (where $M.F$ should be calculated as per Equation C8A.4) respectively. Factor α_{tl} is required in order to normalise the sample t/l ratio to 1.0, while factor α_{ht} is required in order to normalise the sample h/t ratio to 1.0, corresponding to a cubic mortar sample that is comparable to a 50 mm cube. These factors were derived based on the study detailed in Lumantarna (2012).

$$\alpha_{\mathrm{tl}} = 0.42\frac{t}{l} + 0.58 \qquad \text{…C8A.2}$$

$$\alpha_{\mathrm{ht}} = \frac{1}{M.F} \qquad \text{…C8A.3}$$

$$M.F = 2.4\left(\frac{h}{t}\right)^2 - 5.7\left(\frac{h}{t}\right) + 4.3 \qquad \text{…C8A.4}$$

When conducting tests on laboratory manufactured samples make 50 mm mortar cubes, leave these to cure under room temperature (±20 °C) for 28 days, and test them in compression following the mortar cube compression test ASTM C 109-08 (ASTM, 2008).

(a) Example of typical extracted mortar samples | **(b) Example of typical mortar sample preparations** | **(c) Example of typical test set-up**

Figure C8A.6: Determination of mortar compression strength (EQ STRUC Ltd)

C8A.4 Proof Testing of Anchor Connections

An epoxied or grouted anchorage system is a typical method of connecting the floor and roof diaphragms of the building to masonry walls. Reliable anchor pull-out and shear strength is important for assessment or design of anchors and the specification of anchor spacing. Standard installation procedures of embedded anchors involve drilling the masonry wall, cleaning the drilled hole, and epoxying or grouting threaded steel bars to the specified embedment depth, typically 50 mm less than the wall thickness. Two-part epoxy or high strength grouts are typically used with surface preparation conducted in accordance with the manufacturer's specifications.

On-site quality control and proof testing should be undertaken on at least 15% of all installed adhesive anchors, of which 5% should be tested prior to the installation of more than 20% of all anchors. Testing is required to confirm workmanship (particularly the mixing of epoxy and cleaning of holes) and anchor capacity against load requirements. If more than 10% of the tested anchors fail below a test load of 75% of the nominated probable capacity, discount the failed anchors from the total number of anchors tested as part of the quality assurance test. Test additional anchors to meet the 15% threshold requirements. Failures that cannot be attributed to workmanship issues are likely to be indicative of an overestimation of the available capacity and a reassessment of the available probable capacity is likely to be required.

C8A.4.1 Anchors loaded in tension

Once the adhesive is cured (typically over 24 hours), the steel anchors can be loaded in tension using a hydraulic jack until ultimate carrying capacity is reached (ASTM, 2003) or when the load exceeds two times the specified load. The typical test set-up is shown in Figure C8A.7. A 600 mm clear span of reaction frame allows testing of up to 300 mm embedment depth without exerting any confining pressures onto the test area, as the reaction frame supports are outside the general zone of influence. On completing the test, the anchor stud is typically cut flush with the wall surface.

(a) Typical anchor pull test set-up

(b) Close up of the typical test set-up with an alternative test frame

Figure C8A.7: Typical anchor pull-out test set-up (EQ STRUC Ltd)

C8A.4.2 Anchors loaded in shear

The test set-up that could be adopted for in situ testing of anchors loaded in shear is shown in Figure C8A.8. Monotonic shear loading can be applied by using a single acting hydraulic actuator, with the external diameter of the actuator selected to be as small as possible. The bracket arrangements should minimise the tension loads in the anchors. The aim is to determine the shear capacity in the absence of tension.

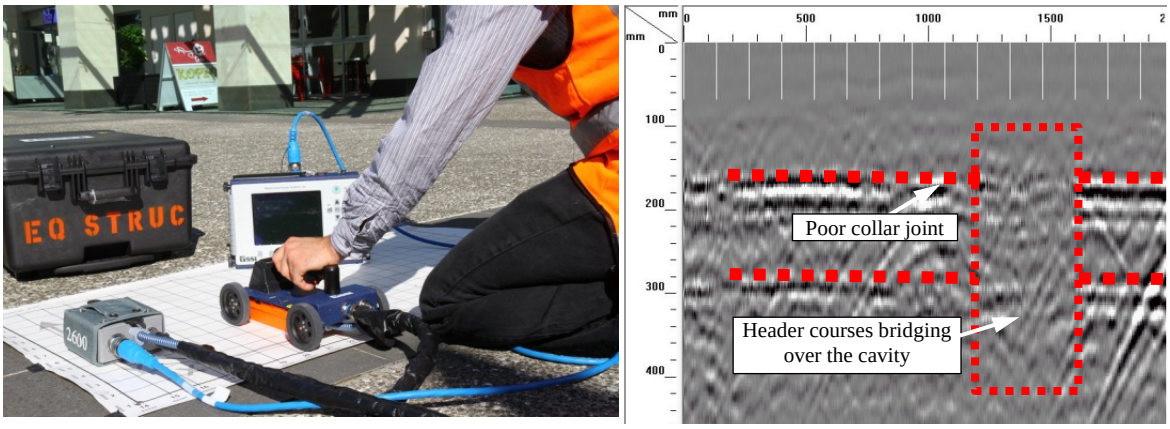
(a) Typical anchor shear tests set-up (push cycle)

(b) Typical anchor shear tests set-up (pull cycle)

Figure C8A.8: Shear tests set-up used (EQ STRUC Ltd)

C8A.5 Investigation of Collar Joints and Wall Cavities

Investigation of collar joints quality and wall cavities can be undertaken using a Ground Penetrating Radar (GPR) structural scanner (refer to Figure C8A.9(a)). The scanner is capable of accurately determining the member thickness, metallic objects, voids and other information. An example of the information provided by GPR scanning is presented in Figure C8A.9(b).



(a) GPR scanner

(b) Typical results output

Figure C8A.9: Example of non-invasive scanning using Ground Penetrating Radar (GPR) scanner technology (EQ STRUC Ltd)

C8A.6 Cavity Tie Examination

The main focus of the cavity tie examination is to identify the condition and frequency of the cavity ties embedded between the leaves of the cavity URM walls. A borescope inspection camera can be used to inspect the air cavity through a void left from a removed brick or an air vent (refer to Figure C8A.10).

(a) Borescope inspection camera **(b) Typical example of cavity observations**

Figure C8A.10: Borescope inspection camera (EQ STRUC Ltd)

Appendix C8B: Derivation of Instability Deflection and Fundamental Period for Face-Loaded Masonry Walls

C8B.1 General Considerations and Approximations

There are many variations that need to be taken into account when considering a general formulation for URM walls that might fail out-of-plane. These include the following:

- Walls will not usually be of a constant thickness in a building, or even within a storey.
- Walls will have embellishments, appendages and ornamentation that may lead to eccentricity of masses with respect to supports.
- Walls may have openings for windows or doors.
- Support conditions will vary.
- Existing buildings may be rather flexible, leading to possibly large inter-storey displacements that may adversely affect the performance of face-loaded walls.

The following approximations can be used to simplify the analysis while still accounting for some key factors.

1 Deformations due to distortions (straining) in the wall can be ignored. Assume deflections to be entirely due to rigid body motion.

> **Note:**
>
> This is equivalent to saying that the change in potential energy from a disturbance of the wall from its initial position is mostly due to the movement of the masses of the elements comprising the wall and the movements of the masses tributary to the wall. Strain energy contributes less to the change in potential energy.

2 Assume that potential rocking occurs at the support lines (e.g. at roof or floor levels) and, for walls that are supported at the top and bottom of a storey, at the mid height. The mid height rocking position divides the wall into two parts of equal height: a bottom part (subscript b) and a top part (subscript t). The masses of each part are not necessarily equal.

> **Note:**
>
> It is implicit within this assumption and (1) above that the two parts of the wall remain undistorted when the wall deflects. For walls constructed of softer mortars or walls with little vertical pre-stress from storeys above, this is not actually what occurs: the wall takes up a curved shape, particularly in the upper part. Nevertheless, errors occurring from the use of the stated assumptions have been found to be small and the engineer will still obtain acceptably accurate results.

3 Assume the thickness to be small relative to the height of the wall. Assume the slope, A, of both halves of the wall to be small; in the sense that $cos(A) \approx 1$ and $sin(A) \approx A$.

Note:

The approximations for slope are likely to be sufficiently accurate for reasonably thin walls. For thick walls where the height to thickness ratio is smaller, the formulations in this appendix are likely to provide less accurate results and force-based approaches provide an alternative.

4 Inter-storey slopes due to deflection of the building are assumed to be small.

Note:

Approximate corrections for this effect are noted in the method.

5 In dynamic analyses, the moment of inertia is assumed constant and equal to that applying when the wall is in its undisturbed position, whatever the axes of rotation.

Note:

The moment of inertia is dependent on the axes of rotation. During excitation, these axes continually change position. Assuming that the inertia is constant is reasonable within the context of the other approximations employed.

6 Damping is assumed at the default value in NZS 1170.5:2004, which is 5% of critical.

Note:

For the aspect ratio of walls of interest, additional effective damping due to loss of energy on impact is small. Furthermore, it has been found that the surfaces at rocking (or hinge) lines tend to fold onto each other rather than experience the full impact that is theoretically possible, reducing the amount of equivalent damping that might be expected. However, for in-plane analysis of buildings constructed largely of URM, adopting a damping ratio that is significantly greater than 5% is appropriate.

7 Assume that all walls in storeys above and below the wall under study move “in phase” with the subject wall.

Note:

Analytical studies have found this to be the case. One reason for this is that the effective stiffness of a wall as it moves close to its limit deflection (e.g. as measured by its period) becomes very low, affecting its resistance to further deflection caused by accelerations transmitted to the walls through the supports. This assumption means that upper walls, for example, will tend to restrain the subject wall by exerting restraining moments.

C8B.2 Vertically Spanning Walls

C8B.2.1 General formulation

Figures C8B.1 and C8B.2 show the configuration of a wall panel within a storey at two stages of deflection. The wall is intended to be quite general. Simplifications to the general solutions for walls that are simpler (e.g. of uniform thickness) are made in a later section.

Figure C8B.1 shows the configuration at incipient rocking. Figure C8B.2 shows the configuration after significant rocking has occurred, with the wall having rotated through an angle A and with mid height deflection, Δ, where $\Delta = Ah/2$.

In Figure C8B.1 the dimensions e_b and e_t relate to the mass centroids of the upper and lower parts of the panel. The dimension e_p relates to the position of the line of action of weights from upper storeys (walls, floors and roofs) relative to the centroid of the upper part of the panel. The arrows on the associated dimensioning lines indicate the positive direction of these dimensions for the assumed direction of motion (angle A at the bottom of the wall is positive in the anti-clockwise sense). Under some circumstances the signs of the eccentricities may be negative; for example for e_p when an upper storey wall is much thinner than the upper storey wall represented here, particularly where the thickness steps on one face. When the lines of axial force from diaphragm and walls from above are different, the resultant force should be calculated.

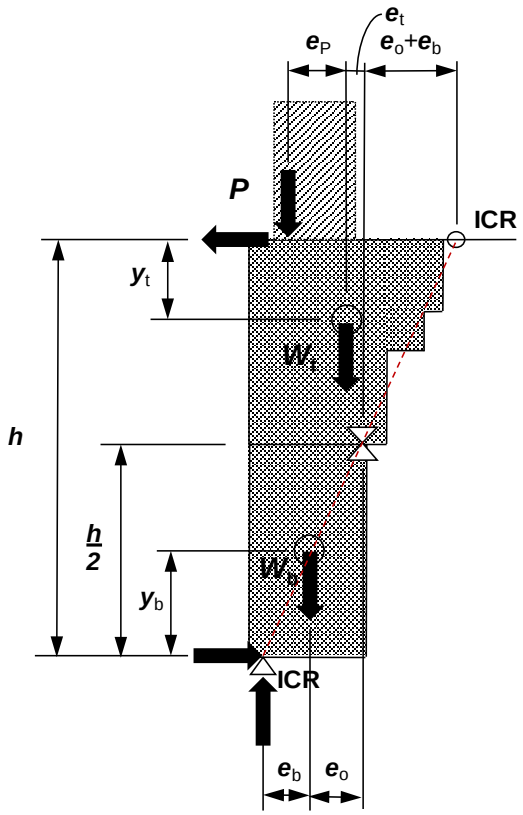


Figure C8B.1: Configuration at incipient rocking

The instantaneous centres of rotation (ICR) are also marked on these figures. These are useful in deriving virtual work expressions.

C8B.2.2 Limiting deflection for static instability

The equation of equilibrium can be written directly by referring to Figure C8B.2 and using virtual work expressions. For static conditions this is given by:

$$W_{\mathrm{b}}(e_{\mathrm{b}} - Ay_{\mathrm{b}}) + W_{\mathrm{t}}\big(e_{\mathrm{o}} + e_{\mathrm{b}} + e_{\mathrm{t}} - A(h - y_{\mathrm{t}})\big) + P\big(e_{\mathrm{o}} + e_{\mathrm{b}} + e_{\mathrm{t}} + e_{\mathrm{p}} - Ah\big) - \Psi(W_{\mathrm{b}}y_{\mathrm{b}} + W_{\mathrm{t}}y_{\mathrm{t}}) = 0 \quad \ldots\text{C8B.1}$$

The final term represents the effect of any inter-storey drift. In the derivation presented, the total deformation has been assumed to be that resulting from the summation of the drift and the rocking wall.

Writing:

$$a = W_{\mathrm{b}}y_{\mathrm{b}} + W_{\mathrm{t}}(h - y_{\mathrm{t}}) + Ph \quad \ldots\text{C8B.2}$$

and:

$$b = W_{\mathrm{b}}e_{\mathrm{b}} + W_{\mathrm{t}}(e_{\mathrm{o}} + e_{\mathrm{b}} + e_{\mathrm{t}}) + P\big(e_{\mathrm{o}} + e_{\mathrm{b}} + e_{\mathrm{t}} + e_{\mathrm{p}}\big) - \Psi(W_{\mathrm{b}}y_{\mathrm{b}} + W_{\mathrm{t}}y_{\mathrm{t}}) \quad \ldots\text{C8B.3}$$

and collecting terms in A, the equation of equilibrium is rewritten as:

$$-aA + b = 0 \quad \ldots\text{C8B.4}$$

from which:

$$A = \frac{b}{a} \quad \ldots\text{C8B.5}$$

when the wall becomes unstable.

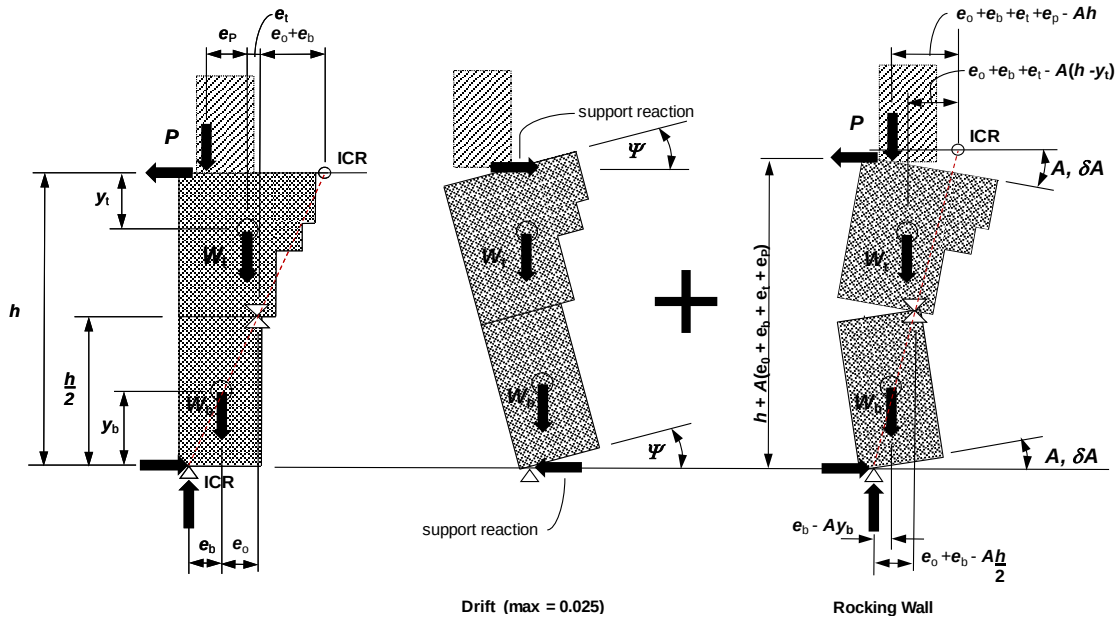


Figure C8B.2: Configuration when rotations have become significant and there is inter-storey drift

Therefore, the critical value of the deflection at mid height of the panel, at which the panel will be unstable, is:

$$\Delta_{\mathrm{i}} = A\frac{h}{2} = \frac{bh}{2a} \qquad \ldots\text{C8B.6}$$

It is assumed that Δ_{m}, a fraction of this deflection, is the maximum useful deflection. Experimental and analytic studies indicate that this fraction might be assumed to be about 0.6. At larger displacements than $0.6\Delta_{\mathrm{i}}$, analysis reveals an undue sensitivity to earthquake spectral content and a wide scatter in results.

C8B.2.3 Equation of motion for free vibration

When conditions are not static, the virtual work expression on the left-hand side in the equation above is unchanged but the zero on the right-hand side of the equation is replaced by mass x acceleration, in accordance with Newton's law. This gives:

$$-aA + b = -J\ddot{A} \qquad \ldots\text{C8B.7}$$

This uses the usual notation for acceleration (a double dot to denote the second derivative with respect to time; in this case indicating angular acceleration), and J as the rotational inertia.

The rotational inertia can be written directly from Figures C8B.1 and C8B.2, noting that the centroids undergo accelerations vertically and horizontally as well as rotationally, and these accelerations relate to the angular acceleration in the same way as the displacements relate to the angular displacement. While the rotational inertia is dependent on the displacements, the effects of this variation are ignored. Therefore, the rotational inertia is taken as that when no displacement has occurred. This gives the following expression for rotational inertia:

$$J = J_{\mathrm{bo}} + J_{\mathrm{to}} + \frac{1}{\mathrm{g}}\Big\{W_{\mathrm{b}}[e_{\mathrm{b}}^2 + y_{\mathrm{b}}^2] + W_{\mathrm{t}}[(e_{\mathrm{o}} + e_{\mathrm{b}} + e_{\mathrm{t}})^2 + y_{\mathrm{t}}^2] + P\Big[\big(e_{\mathrm{o}} + e_{\mathrm{b}} + e_{\mathrm{t}} + e_{\mathrm{p}}\big)^2\Big]\Big\} + J_{\mathrm{anc}} \qquad \ldots\text{C8B.8}$$

where J_{bo} and J_{to} are the mass moments of inertia of the bottom and top parts respectively about their centroids, and J_{anc} is the inertia of any ancillary masses, such as veneers, that are not integral with the wall but contribute to its inertia.

For a wall with unit length, held at the top and bottom, and rocking crack at mid height, with a density of ρ per unit volume, the mass moment of inertia about the horizontal axis through the centroid is given by:

$$I_{\mathrm{xx}}(\mathrm{kgm^2}) = \rho\, t_{\mathrm{Gross}} \left(\frac{h}{2}\right)^3 \Big/ 12 \qquad \ldots\text{C8B.9}$$

The corresponding mass moment of inertia about the vertical axis through the centroid is:

$$I_{\mathrm{yy}}\ (\mathrm{kgm^2}) = \rho \left(\frac{h}{2}\right) t_{\mathrm{Gross}}^3 \Big/ 12 \qquad \ldots\text{C8B.10}$$

The polar moment of inertia through the centroid is the sum of these, or:

$$J_{\text{bo}}(\text{ kgm}^2) = J_{\text{to}} = I_{\text{xx}} + I_{\text{yy}} = \rho t_{\text{Gross}}\left(h/2\right)\left[t_{\text{Gross}}^2 + \left(h/2\right)^2\right]/12$$
$$= \frac{m}{2}\frac{\left[t_{\text{Gross}}^2+\left(h/2\right)^2\right]}{12} = \frac{W}{2g}\frac{\left[t_{\text{Gross}}^2+\left(h/2\right)^2\right]}{12} \quad \text{…C8B.11}$$

where m is the mass (kg), W (N) is the weight of the whole wall panel and g is the acceleration of gravity.

Note that in this equation the expressions in square brackets are the squares of the radii from the instantaneous centres of rotation to the mass centroids, where the locations of the instantaneous centres of rotation are those when there is no displacement. Some CAD programs have functions that will assist in determining the inertia about an arbitrary point (or locus), such as about the ICR shown in Figure C8B.2.

Collecting terms and normalising the equation so that the coefficient of the acceleration term is unity gives the following differential equation of free vibration:

$$\ddot{A} - \frac{a}{J}A = -\frac{b}{j} \quad \text{…C8B.12}$$

C8B.2.4 Period of free vibration

The solution of the equation for free vibration derived in the previous section is:

$$A = C_1 sinh\left(\sqrt{\frac{a}{J}}\tau\right) + C_2 cosh\left(\sqrt{\frac{a}{J}}\tau\right) + \frac{b}{a} \quad \text{…C8B.13}$$

The time, τ, is taken as zero when the wall has its maximum rotation, $A(= \Delta/2h)$. Using this condition and the condition that the rotational velocity is zero when the time $\tau = 0$, the solution becomes:

$$A = \left(\frac{2\Delta}{h} - \frac{b}{a}\right) cosh\left(\sqrt{\frac{a}{J}}\tau\right) + \frac{b}{a} \quad \text{….C8B.14}$$

Take the period of the "part", T_{p}, as four times the duration for the wall to move from its position at maximum deflection to the vertical. Then the period is given by:

$$T_{\text{p}} = 4\sqrt{\frac{J}{a}} cosh^{-1}\left(\frac{b/a}{b/a - 2\Delta/h}\right) \quad \text{…C8B.15}$$

This can be simplified further by substituting the term for Δ_{i} found from the static analysis and putting the value of Δ used for the calculation of period as Δ_{t} to give:

$$T_{\text{p}} = 4\sqrt{\frac{J}{a}} cosh^{-1}\left(\frac{1}{1 - \Delta_{\text{t}}/\Delta_{\text{i}}}\right) \quad \text{…C8B.16}$$

By accepting that the deflection ratio of interest is 0.6 (i.e. $^{\Delta_m}/_{\Delta_i} = 0.6$), then this becomes:

$$T_p = 6.27\sqrt{\frac{J}{a}} \quad \text{…C8B.17}$$

as in the 2006 guidelines. However, research (Derakhshan et al. (2014a)) indicates that the resulting period and responding displacement demand is too large if a spectrum derived from linear elastic assumptions is used. Rather, this research suggests that an effective period calculated from an assumed displacement of 60% of the assumed displacement capacity should be used. Therefore, the period is based on $\Delta_t = 0.36\Delta_i$ so that:

$$T_p = 4.07\sqrt{\frac{J}{a}} \quad \text{…C8B.18}$$

C8B.2.5 Maximum acceleration

The acceleration required to start rocking of the wall occurs when the wall is in its initial (undisturbed) state. This can be determined from the virtual work equations by assuming that $A = 0$. Accordingly:

$$\ddot{A}_{max} = \frac{b}{J} \quad \text{…C8B.19}$$

However, a more cautious appraisal assumes that the acceleration is influenced primarily by the instantaneous acceleration of the supports, transmitted to the wall masses, without relief by wall rocking. Accordingly:

$$C_m = \frac{b}{(W_b y_b + W_t y_t)} \quad \text{…C8B.20}$$

where C_m is the acceleration *coefficient* to just initiate rocking.

C8B.2.6 Participation factor

The participation factor can be determined in the usual way by normalising the original form of the differential equation for free vibration, modified by adding the ground acceleration term. For the original form of the equation, the ground acceleration term is added to the right hand side. Written in terms of a unit rotation, this term is $(W_b y_b + W_t y_t)$ times the ground acceleration. The equation is normalised by dividing through by J, and then multiplied by h/2 to convert it to one involving displacement instead of rotation. The participation factor is then the coefficient of the ground acceleration. That is:

$$\gamma = \frac{(W_b y_b + W_t y_t)h}{2Jg} \quad \text{…C8B.21}$$

C8B.2.7 Simplifications for regular walls

Simplifications can be made where the thickness of a wall within a storey is constant, there are no openings, and there are no ancillary masses. Further approximations can then be applied:

- The weight of each part (top and bottom) is half the total weight, W.
- $y_b = y_t = h/4$

- The moment of inertia of the whole wall is further approximated by assuming that all e are very small relative to the height (or, for the same result, by ignoring the shift of the ICR from the mid-line of the wall), giving $J = Wh^2/12g$. Alternatively, use the simplified expressions for J given in Table C8B.1.

Approximate displacements for static instability

Table C8B.1 gives values for a and b and the resulting mid height deflection to cause static instability when e_b and/or e_p are either zero or half of the effective thickness of the wall, t. In this table e_o and e_t are both assumed equal half the effective wall thickness. While these values of the eccentricities are reasonably common, they are not the only values that will occur in practice.

The effective thickness may be assumed as follows:

$$t = \left(0.975 - 0.025\frac{P}{W}\right) t_\mathrm{nom} \qquad \text{…C8B.22}$$

where t_nom is the nominal thickness of the wall.

Experiments show that this is a reasonable approximation, even for walls with soft mortar. In that case there is greater damping and that reduces response, which compensates for errors in the expression for effective thickness.

Approximate expression for period of vibration

Noting that:

$$a = \left(\frac{W}{2} + P\right) h \qquad \text{…C8B.23}$$

and using the approximation for J relevant to a wall with large aspect ratio, the expression for the period is given by:

$$T_\mathrm{p} = 4.07\sqrt{\frac{2Wh}{12g(W+2P)}} \qquad \text{…C8B.24}$$

where it should be noted that the period is independent of the restraint conditions at the top and bottom of the wall (i.e. independent of both e_b and e_p).

If the height is expressed in metres, this expression simplifies to:

$$T_\mathrm{p} = \sqrt{\frac{0.28h}{(1+2P/W)}} \qquad \text{…C8B.25}$$

It should be appreciated that periods may be rather long.

This approximation errs on the low side, which leads to an underestimate of displacement demand and therefore to slightly incautious results. The fuller formulation is therefore preferred.

Participation factor

Suitable approximations can be made for the participation factor. This could be taken at the maximum value of 1.5. Alternatively, the numerator can be simplified as provided in the following expression, and the simplified value of J shown in Table C8B.1 can be used.

Maximum acceleration

By making the same simplifications as above, the maximum acceleration is given by:

$$\ddot{A}_{\max} = \frac{b}{J} = \frac{12bg}{Wh^2} \qquad \ldots C8B.26$$

Or, more cautiously, the *acceleration coefficient*, C_m, is given in Table C8B.1 for the common cases regularly encountered.

C8B.2.8 Adjustments required when inter-storey displacement is large

Using the common limit on Ψ of 0.025, and substituting for $W_b = W_t = W/2$ and $y_b = y_t = h/4$, δb is found to be $Wh/160$. Taking $h/t = 25$, in the absence of any surcharge, the percentage reduction in the instability deflection for each case shown in Table C8B.1 is 31% for Cases 0 and 2, and 16% for Cases 1 and 3. These are not insignificant, and these affects should be assessed especially in buildings with flexible principal framing such as steel moment resisting frames.

Table C8B.1: Static instability deflection for uniform walls, various boundary conditions

Boundary condition number	**0**	**1**	**2**	**3**
e_p	0	0	$t/2$	$t/2$
e_b	0	$t/2$	0	$t/2$
b	$(W/2+P)t$	$(W+3P/2)t$	$(W/2+3P/2)t$	$(W+2P)t$
a	$(W/2+P)h$	$(W/2+P)h$	$(W/2+P)h$	$(W/2+P)h$
$\Delta_i = bh/(2a)$	$t/2$	$\frac{(2W+3P)t}{(2W+4P)}$	$\frac{(W+3P)t}{(2W+4P)}$	t
J	$\{(W/12)[h^2+7t^2]+Pt^2\}/g$	$\{\left(\frac{W}{12}\right)[h^2+16t^2]+9Pt^2/4\}/g$	$\{\left(\frac{W}{12}\right)[h^2+7t^2]+9Pt^2/4\}/g$	$\{(W/12)[h^2+16t^2]+4Pt^2\}/g$
C_m	$(2+4P/W)t/h$	$(4+6P/W)t/h$	$(2+6P/W)t/h$	$4(1+2P/W)t/h$

Note:

1. The boundary conditions of the piers shown above are for clockwise potential rocking.
2. The top eccentricity, e_t, is not related to a boundary condition, so is not included in the table. The top eccentricity, e_t, is the horizontal distance from the central pivot point to the centre of mass of the top block which is not related to a boundary condition.
3. The eccentricities shown in the sketches are for the positive sense. Where the top eccentricity is in the other sense e_p should be entered as a negative number.

C8B.3 Vertical Cantilevers

C8B.3.1 General formulation

Figure C8B.3 shows a general arrangement of a cantilever. The wall illustrated has an overburden load at the top, but this load will commonly be zero, as in a parapet. Where a load does exist it is important to realise that the mass associated with that load can move horizontally. As a result the inertia of the wall is affected by the overburden to a greater extent than if the wall was supported horizontally at the top. If the top load is supported on the wall in such a way that its point of application can change, as is the case if it is through a continuous beam or slab that crosses the wall, there will be an eccentricity of the point of application of P.

Sometimes several walls will be linked; for example, when a series of face-loaded walls provide the lateral resistance to a single storey building. This case can be solved by methods derived from the general formulation, but express formulations for it are not provided here.

For the single wall illustrated, it is assumed that P is applied eccentric to the centre of the wall at the top and that point of application remains constant. It is straightforward to obtain the following parameters:

$$a = Wy_{\mathrm{b}} + Ph \qquad \text{...C8B.27}$$

$$b = We_{\mathrm{b}} + P(e_{\mathrm{b}} + e_{\mathrm{p}}) \qquad \text{...C8B.28}$$

$$J = \frac{W}{12g}(h^2 + t_{\mathrm{nom}}^2) + \frac{W}{g}(y_{\mathrm{b}}^2 + e_{\mathrm{b}}^2) + \frac{P}{g}\left[h^2 + \left(e_{\mathrm{b}} + e_{\mathrm{p}}\right)^2\right] \qquad \text{…C8B.29}$$

Note that in these equations e_{p} is taken as positive in the sense shown in Figure C8B.3.

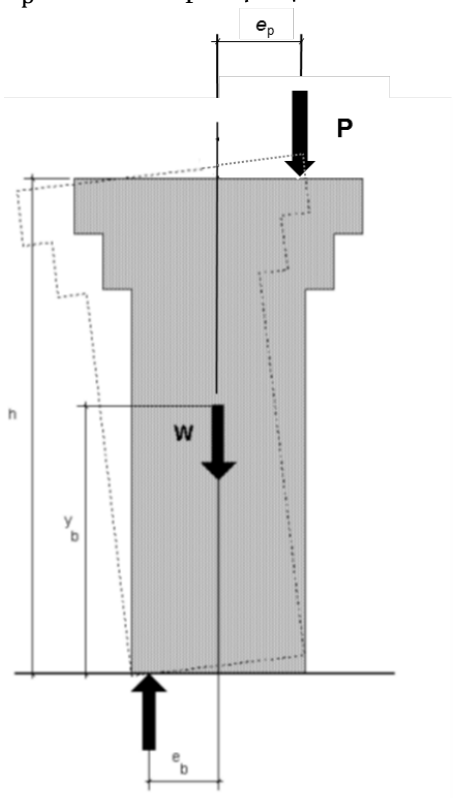


Figure C8B.3: Single cantilever

C8B.3.2 Limiting deflection for static instability

When the wall just becomes unstable, the relationship for A remains the same as before but the deflection is Ah. Thus, the limiting deflection is given by:

$$\Delta_{\mathrm{i}} = Ah = \frac{bh}{a} = \frac{[We_{\mathrm{b}} + P(e_{\mathrm{b}} + e_{\mathrm{p}})]h}{Wy_{\mathrm{b}} + Ph} \quad \text{...C8B.30}$$

For the case where $P = 0$ and $y_{\mathrm{b}} = h/2$ this reduces to $\Delta_{\mathrm{i}} = 2e_{\mathrm{b}} = t$.

C8B.3.3 Period of vibration

If $\Delta_{\mathrm{t}} = 0.36\Delta_{\mathrm{i}}$ as for the simple case, the general expression for period would remain valid. However, cantilevers are much more susceptible to instability under real earthquake stimulation than wall panels that are supported both top and bottom. Therefore, the maximum useable displacement for calculation of capacity, Δ_{m}, is reduced from $0.6\Delta_{\mathrm{i}}$ to $0.3\Delta_{\mathrm{i}}$ and the displacement for calculation of period changes from $0.6\Delta_{\mathrm{m}}$ to $0.8\Delta_{\mathrm{m}} = 0.24\Delta_{\mathrm{i}}$ so that:

$$T_{\mathrm{p}} = 3.1\sqrt{\frac{J}{a}} \quad \text{…C8B.31}$$

where $P = 0$, $e_{\mathrm{b}} = t/2$, $y_{\mathrm{b}} = h/2$, approximating $t = t_{\mathrm{nom}}$ and expressing h in metres, the period of vibration is given by:

$$T_{\mathrm{p}} = \sqrt{0.65h\left[1 + \left({}^{t}/_{h}\right)^{2}\right]} \quad \text{…C8B.32}$$

Note that P, whether eccentric or not, will not affect the static instability displacement, and therefore neither the displacement demand (by affecting the period) nor the displacement capacity.

C8B.3.4 Participation factor

The expression for the participation factor remains unaffected; that is, $\gamma = Wh^2/(2Jg)$. This may be simplified for uniform walls with $P = 0$ (no added load at the top) by inserting the specific expression for J. This gives:

$$\gamma = \frac{3}{2\left(1 + \left({}^{t}/_{h}\right)^{2}\right)} \quad \text{…C8B.33}$$

C8B.3.5 Maximum acceleration

Using the same simplifications as above:

$$C = \frac{t}{h} \quad \text{…C8B.34}$$

C8B.4 Cavity Walls

The following procedure is suggested for evaluating the score for a cavity wall. It is assumed that there is a common height, h, and that the total load applied to the top of the combined panel, P, is appropriately allocated into each wythe.

Step 1 Separately for each wythe work out J, a and b. It would be possible to follow this right through to a general conclusion for *%NBS* for each wythe but that would only be of interest, not a useable solution.

Step 2 Find the combined J, a and b by adding the individual values for each wythe determined from Step 1.

Step 3 Using the combined a and b find the static instability displacement $\Delta_{\mathrm{i}} = (b/a)$ x $(h/2)$. The maximum usable deflection is $0.6\ \Delta_{\mathrm{i}}$. The displacement used for the calculation of period is $\Delta_{\mathrm{t}} = 0.36\ \Delta_{\mathrm{i}}$.

Step 4 Find the participation factor γ. This is $Wh^2/8Jg$, but with W being the combined weight of both wythes and J being the polar moment of inertia for the combined system as derived in Step 2.

Step 5 Using J and Δ_{t} derive the period, T_{p}, and the displacement demand, D_{ph}, using the appropriate equations from Section C8.8.5.

Step 6 Using the demand D_{ph} determined from Step 5 and the reliable capacity Δ_{m} determined from Step 3, determine the score as:

$$\%NBS = 100\ \Delta_{\mathrm{m}}/D_{\mathrm{ph}} = 60\ \Delta_{\mathrm{i}}/D_{\mathrm{ph}}. \qquad \ldots\text{C8B.35}$$

Appendix C8C: Charts for Assessment of Out-of-Plane Walls

C8C.1 General

This appendix presents simplified ready-to-use charts for estimation of *%NBS* for face-loaded URM walls with uniform thickness. The charts have been developed for walls with various slenderness ratios (wall height/thickness) vs Basic Performance Ratio (BPR). The BPR can be converted to *%NBS* after dividing it by the product of the appropriate spectral shape factor ($C_{\mathrm{h}}(0)$), required to evaluate $C(0)$ for parts, return period factor (R), hazard factor (Z), near-fault factor ($N(T,D)$), and part risk factor (R_{p}) which have been assigned unit values for developing the charts. The charts are presented for various boundary conditions and ratio of load on the wall to self-weight of the wall.

Refer to Section C8 and Appendix C8B for symbols and sign conventions.

This appendix includes charts for the following cases:

- one-way vertically spanning walls laterally supported both at the bottom and the top with no inter-storey drift
- one-way vertically spanning walls laterally supported at the top and the bottom with inter-storey drift of 0.025
- vertical cantilever walls.

The following section presents how these charts should be used.

C8C.2 One-way Vertically Spanning Face-Loaded Walls

Charts for one-way vertically spanning face-loaded walls are presented in Figures C8C.1(a)-(f), C8C.2(a)-(f) and C8C.3(a)-(f) for 110 mm, 230 mm and 350 mm thick walls respectively for inter-storey drift of 0.00. Similarly, charts for an inter-storey drift of 0.025 are presented in Figures C8C.4(a)-(f), C8C.5(a)-(f) and C8C.6(a)-(f) for 110 mm, 230 mm and 350 mm thick walls respectively. The charts have been developed for $e_{\mathrm{t}} = e_{\mathrm{o}} = t/2$ and various values for e_{p}.

Follow the following steps for estimation of *%NBS* for a vertically spanning face-loaded wall:

- Identify thickness, t_{Gross}, and height, h, of the wall.
- Calculate slenderness ratio of the wall (h/t_{Gross}).
- Calculate the total self-weight, W, of the wall.
- Calculate vertical load, P, on the wall. This should include all the dead load and appropriate live loads on the wall from above.
- Calculate P/W.
- Calculate eccentricities (e_{b} and e_{p}). e_{b} could be $t/2$ or 0, whereas e_{p} could be $\pm t/2$ or 0. To assign appropriate values, check the base boundary condition and location of P on the wall. Calculation of effective thickness, t, is not required.

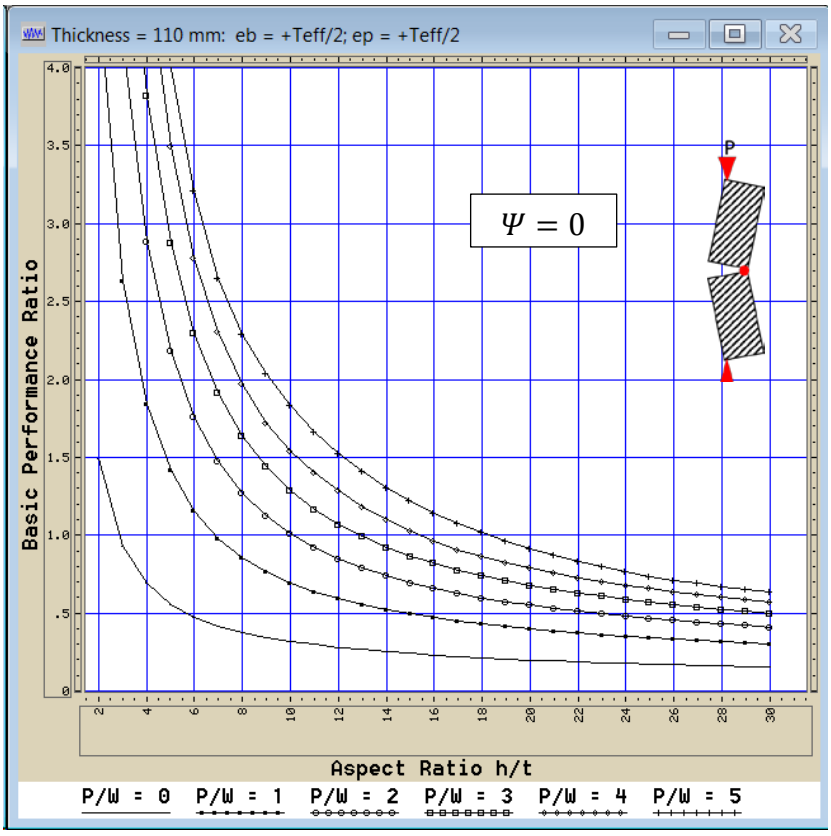
- Refer to the appropriate charts (for appropriate e_{b} and e_{p}, P/W and inter-storey drift).
- Estimate Basic Performance Ratio (BPR) from the charts. Linear interpolation between plots may be used as necessary for inter-storey drifts between 0 and 0.025.
- Refer to NZS 1170.5:2004 for $C_{\mathrm{h}}(0)$ required to evaluate $C(0)$ for parts, $R, Z, N(T, D)$, C_{Hi} and R_{p}. For estimation of C_{Hi}, h_{i} is height of the mid height of the wall from the ground.
- $\%NBS = \frac{Basic\ Performance\ Ratio\ from\ charts\ for\ h/t}{C_{\mathrm{h}}(0)RZN(T,D)C_{\mathrm{Hi}}R_{\mathrm{P}}}$

C8C.3 Vertical Cantilevers

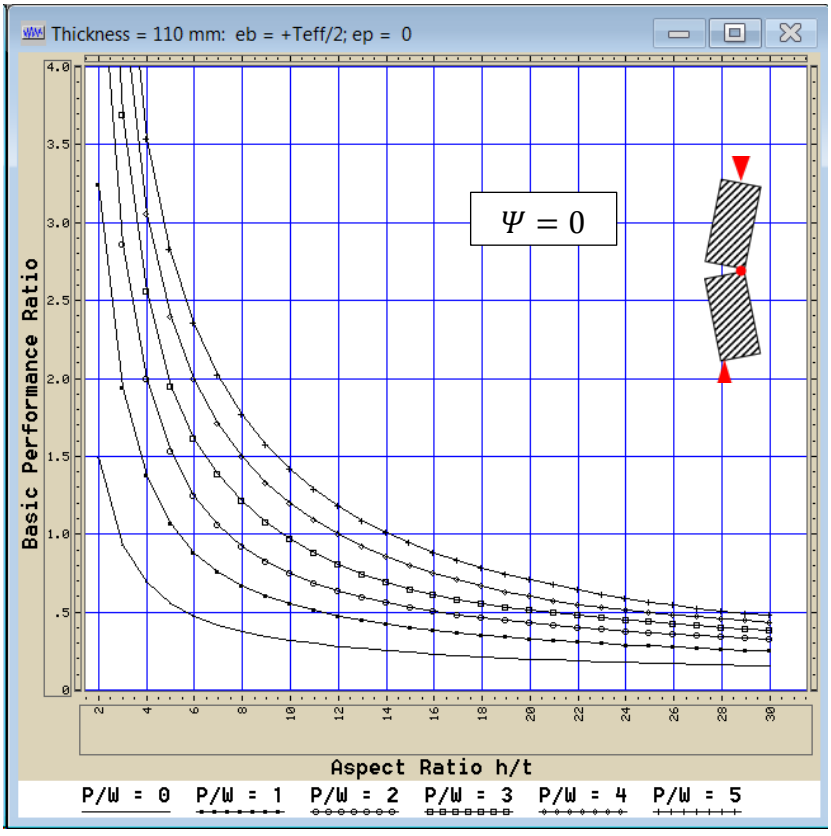
Charts for one-way cantilever walls are presented in Figures C8C.7(a)-(c), C8C.8(a)-(c) and C8C.9(a)-(c) for 110 mm, 230 mm and 350 mm thick walls respectively.

Follow the following steps for estimation of *%NBS* of a face-loaded cantilever wall:

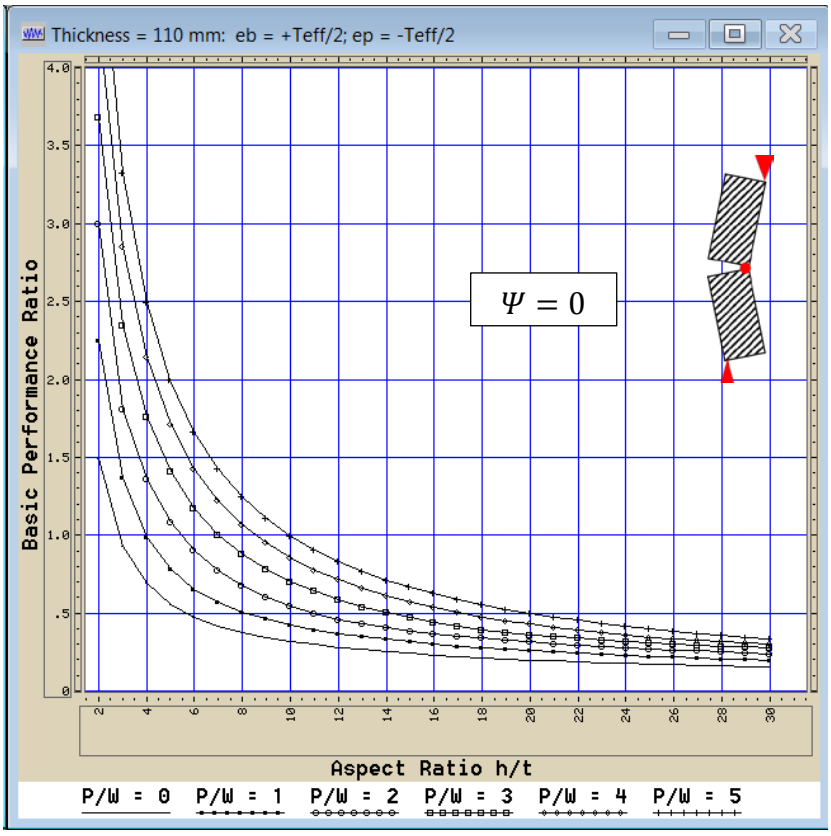
- Identify thickness, t_{Gross}, and height, h, of the wall.
- Calculate slenderness ratio of the wall (h/t_{Gross}).
- Calculate total self-weight, W, of the wall above the level of cantilevering plane.
- Calculate vertical load, P, on the wall, if any. This should include all the dead load and appropriate live loads on the wall from above.
- Calculate P/W.
- Calculate eccentricity, e_{p}, for loading P. e_{p} could be $\pm t/2$ or 0, which depends upon location of P on the wall. Calculation of effective thickness, *t*, is not required.
- Refer to the appropriate charts (for appropriate e_{p} and P/W).
- Estimate Basic Performance Ratio (BPR) from the charts. Interpolation between plots may be used as necessary.
- Refer NZS 1170.5:2004 for $C_{\mathrm{h}}(0)$ required to evaluate $C(0)$ for parts, $R, Z, N(T, D)$, C_{Hi} and R_{P}. For estimation of C_{Hi}, h_{i} shall be taken as height of the base of the cantilever wall.
- $\%NBS = \frac{Basic\ Performance\ Ratio\ from\ charts\ for\ h/t}{C_{\mathrm{h}}(0)RZN(T,D)C_{\mathrm{Hi}}R_{\mathrm{P}}}$



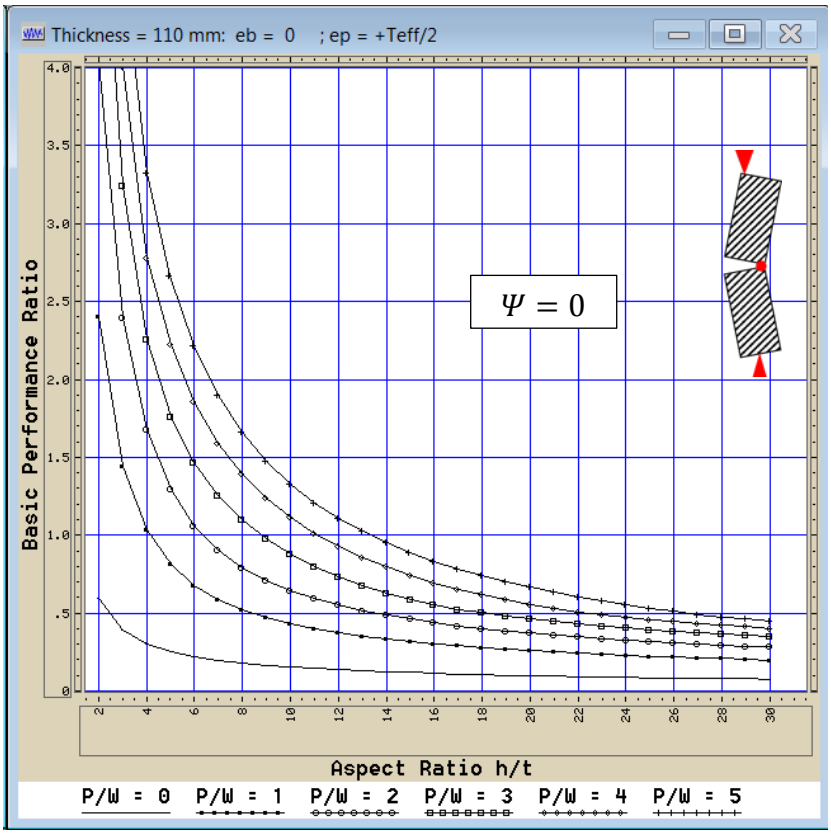
(a) For $e_b = +t/2$ and $e_p = +t/2$ ($t_{Gross} = 110$ mm)



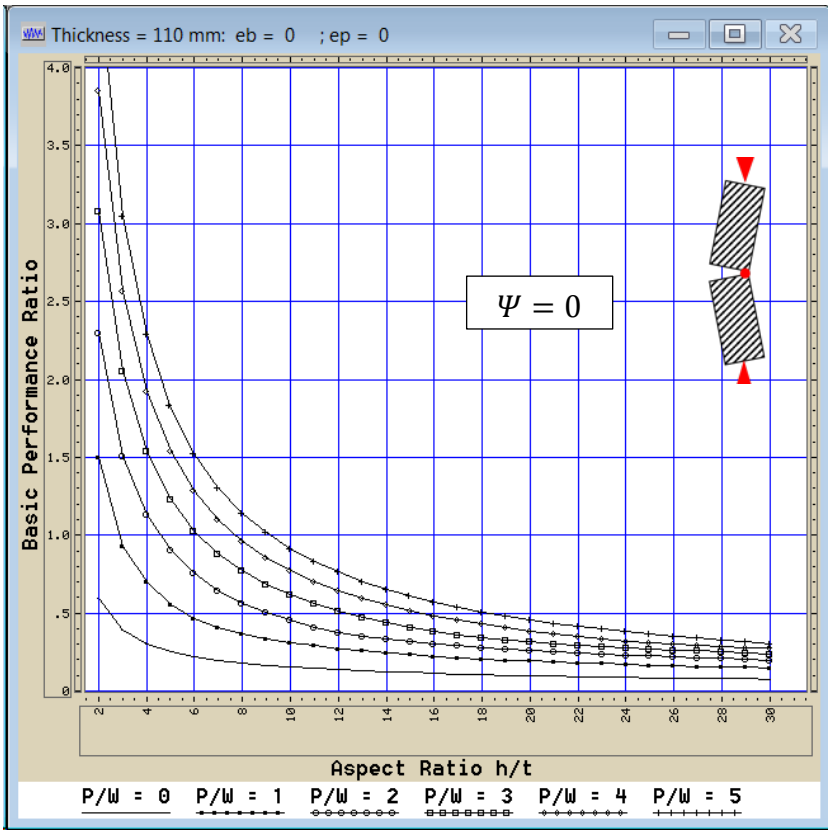
(b) For $e_b = +t/2$ and $e_p = 0$ ($t_{Gross} = 110$ mm)



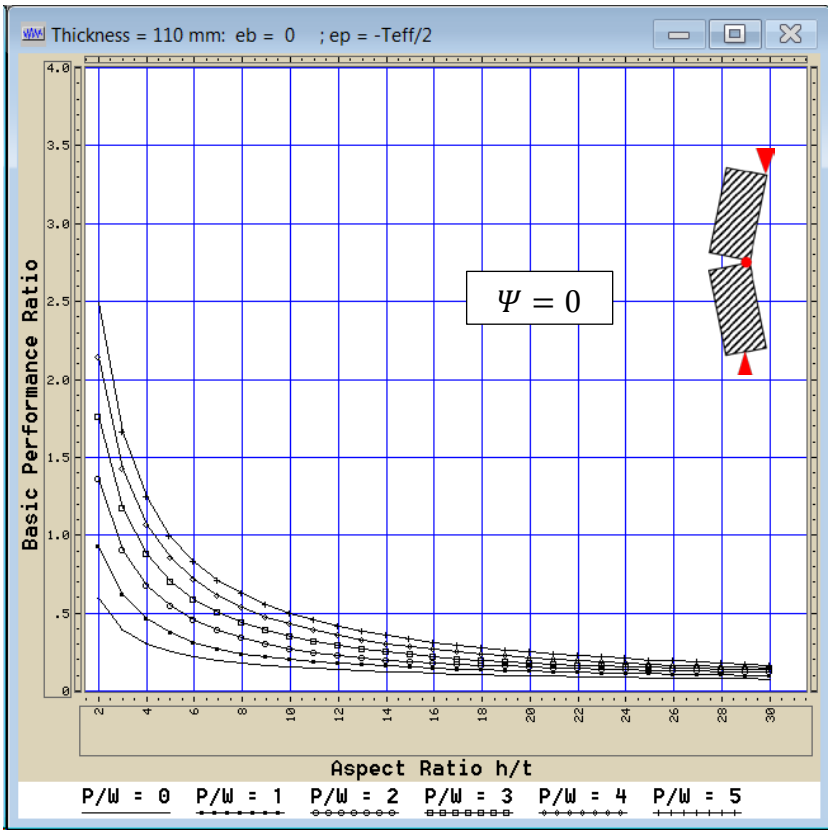
(c) For $e_b = +t/2$ and $e_p = -t/2$ ($t_{Gross} = 110$ mm)



(d) For $e_b = 0$ and $e_p = t/2$ ($t_{Gross} = 110$ mm)

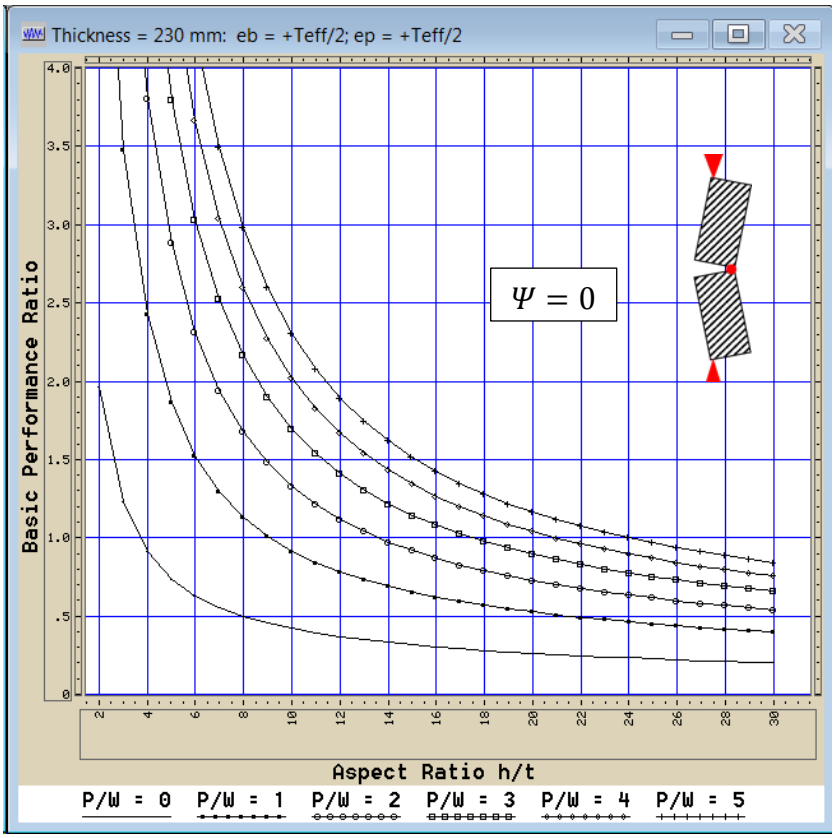


(e) For $e_b = 0$ and $e_p = 0$ ($t_{Gross} = 110$ mm)

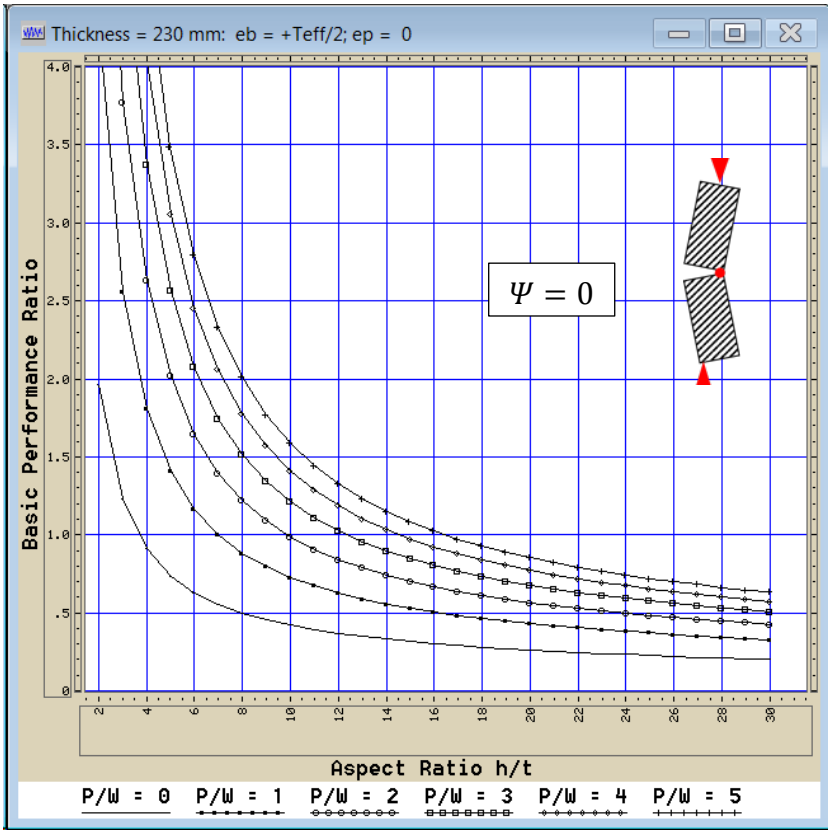


(f) For $e_b = 0$ and $e_p = -t/2$ ($t_{Gross} = 110$ mm)

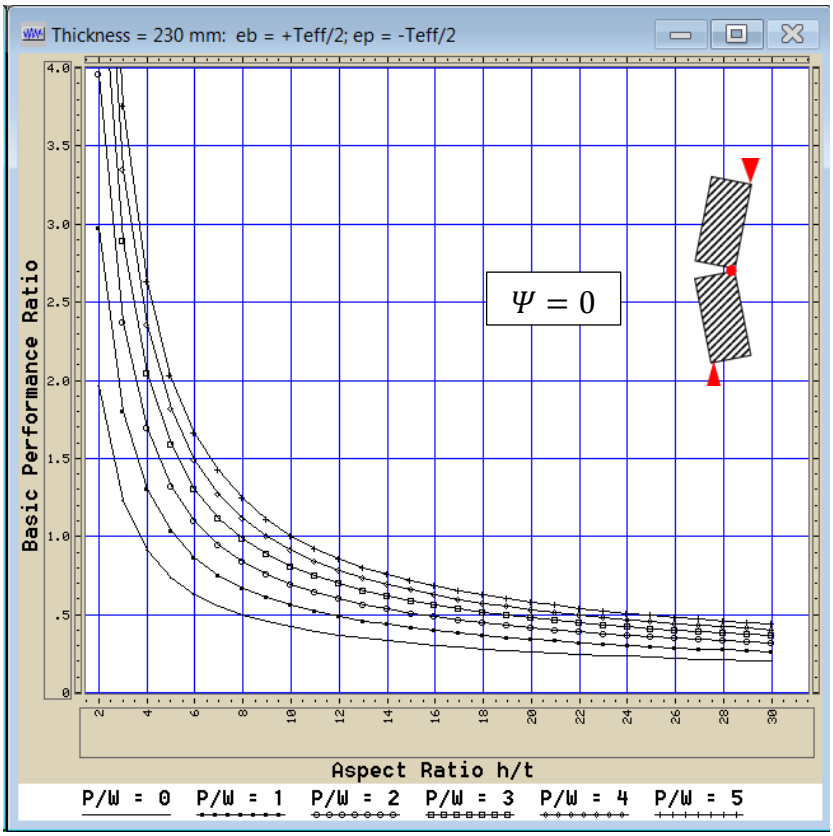
Figure C8C.1: 110 mm thick one-way vertically spanning face-loaded walls ($\Psi = 0$)



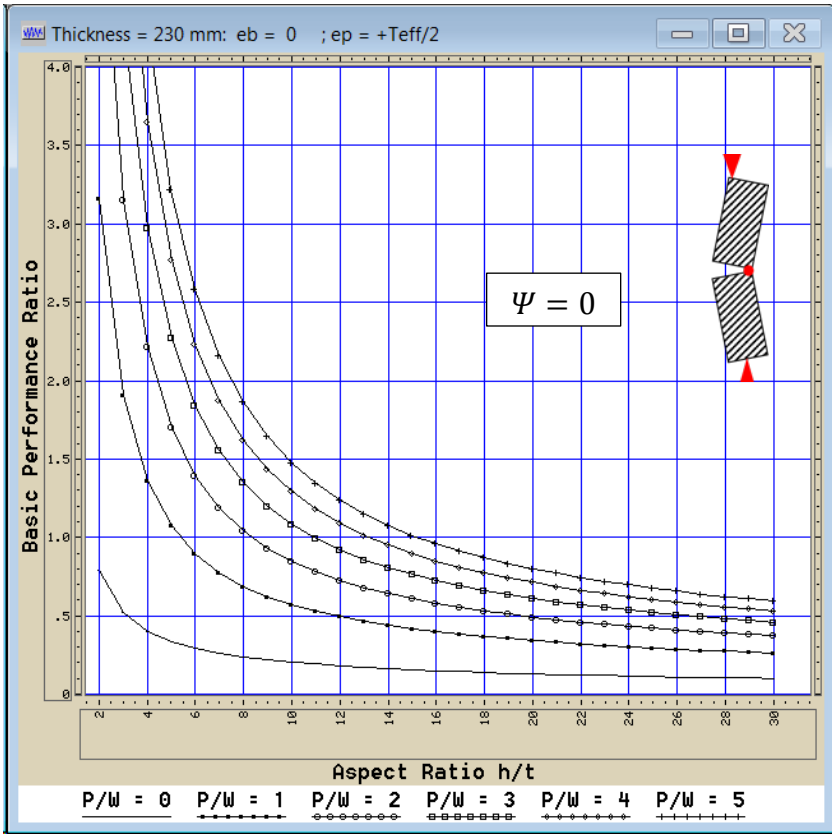
(a) For $e_b = +t/2$ and $e_p = +t/2$ ($t_{Gross} = 230$ mm)



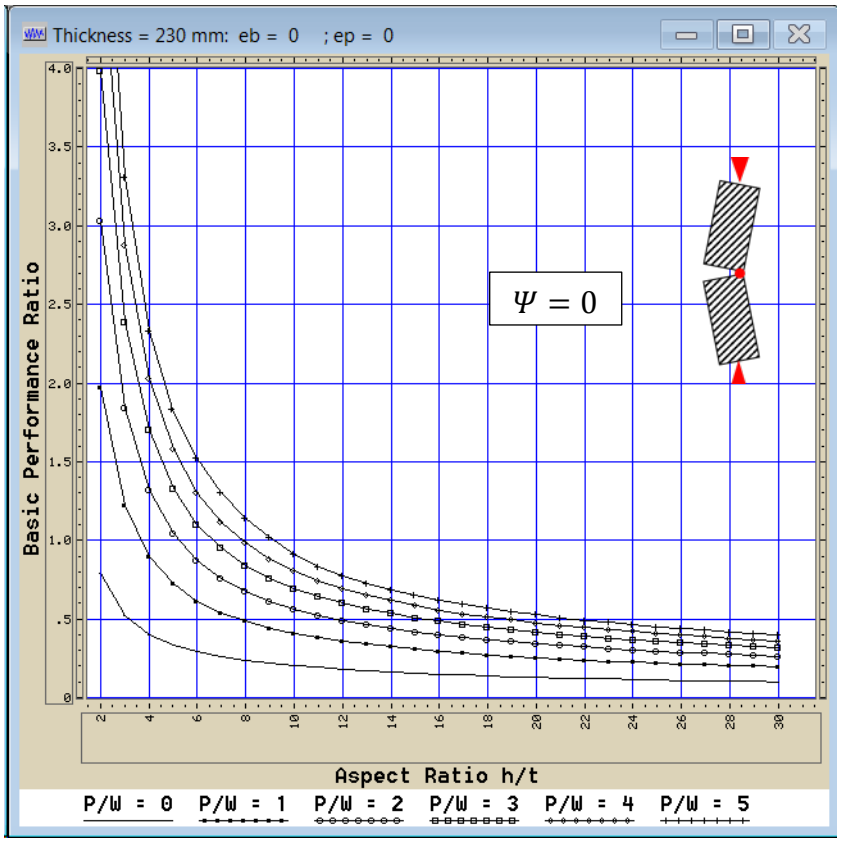
(b) For $e_b = +t/2$ and $e_p = 0$ ($t_{Gross} = 230$ mm)



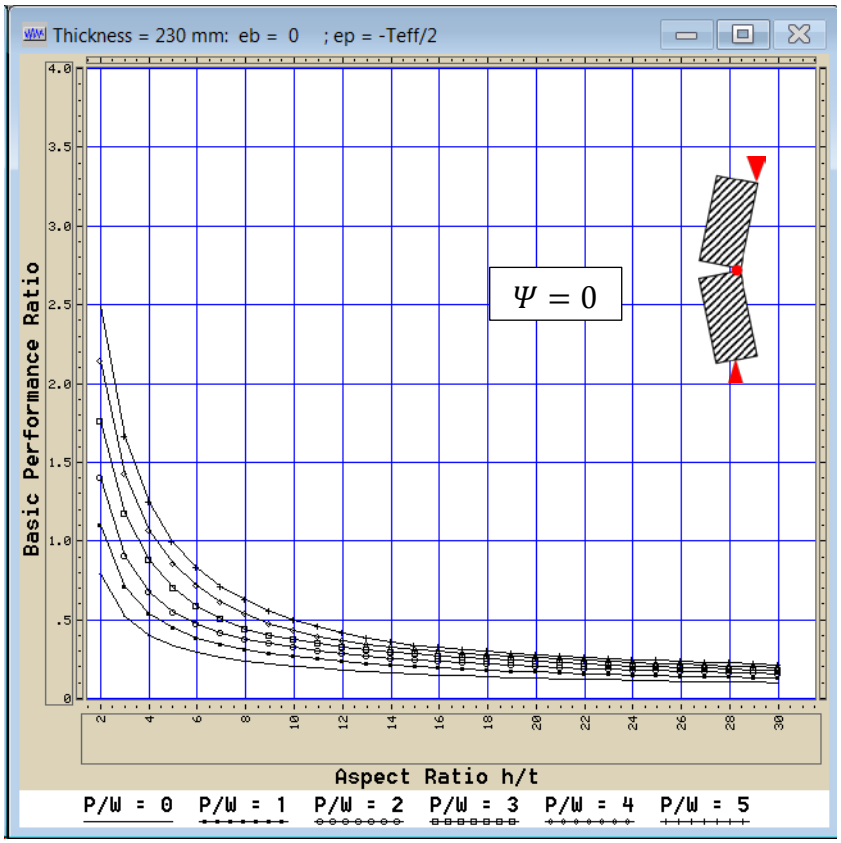
(c) For $e_b = +t/2$ and $e_p = -t/2$ ($t_{Gross} = 230$ mm)



(d) For $e_b = 0$ and $e_p = t/2$ ($t_{Gross} = 230$ mm)

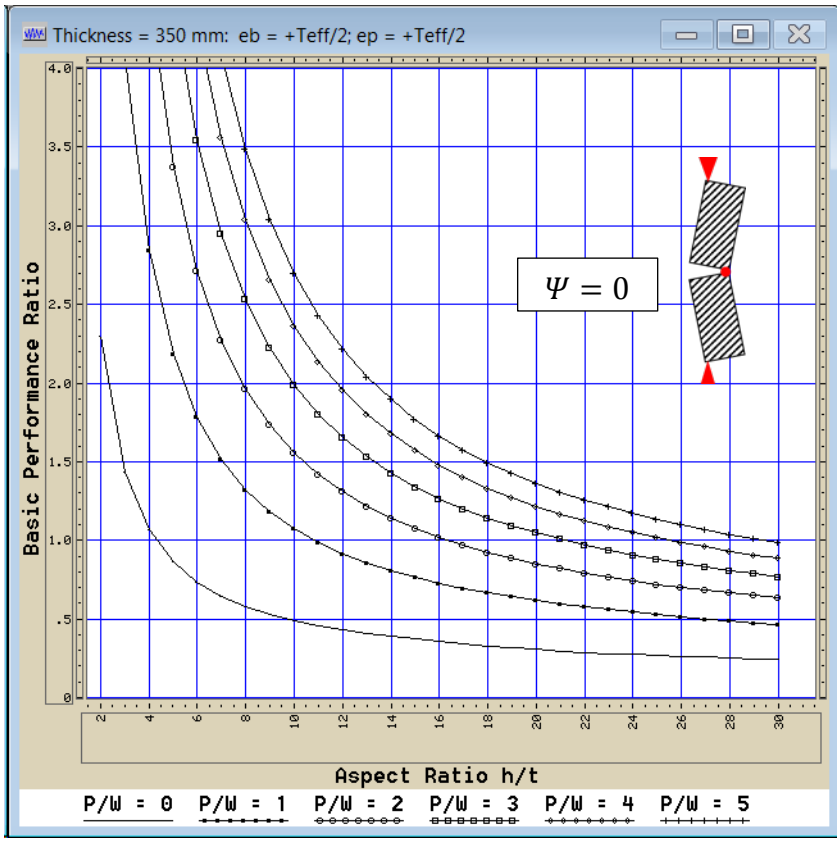


(e) For $e_b = 0$ and $e_p = 0$ ($t_{Gross} = 230$ mm)

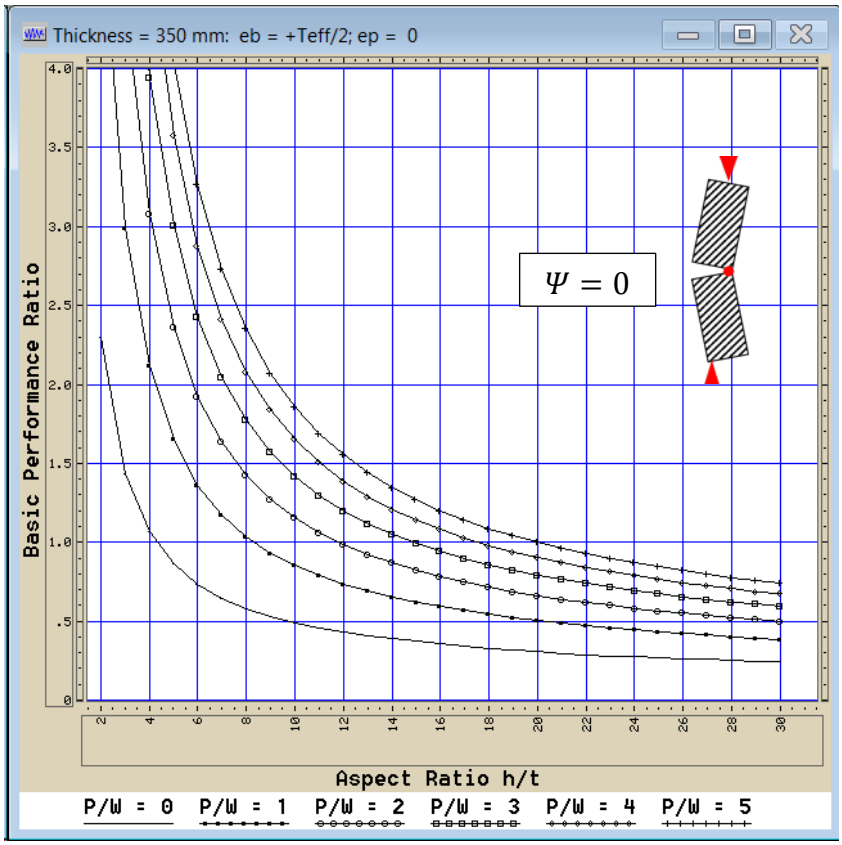


(f) For $e_b = 0$ and $e_p = -t/2$ ($t_{Gross} = 230$ mm)

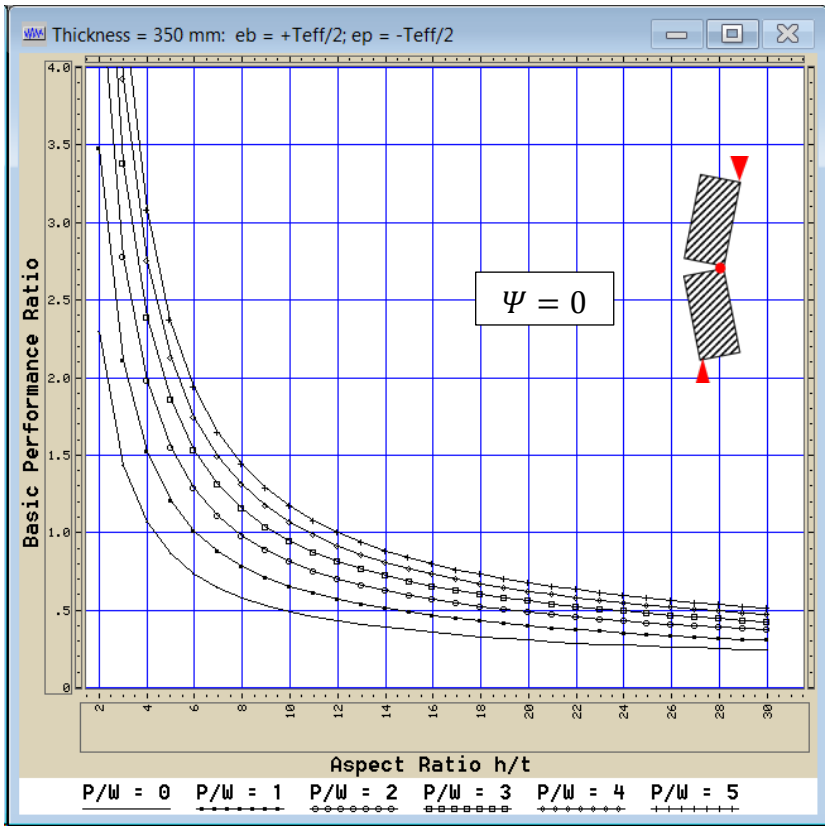
Figure C8C.2: 230 mm thick one-way vertically spanning face-loaded walls ($\Psi = 0$)



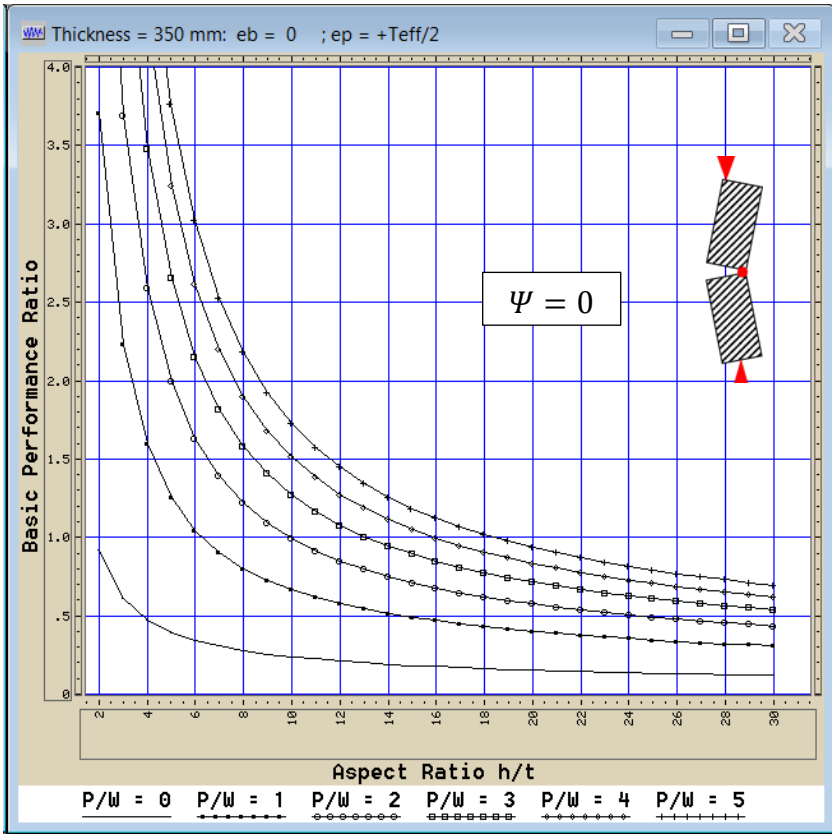
(a) For $e_b = +t/2$ and $e_p = +t/2$ ($t_{Gross} = 350$ mm)



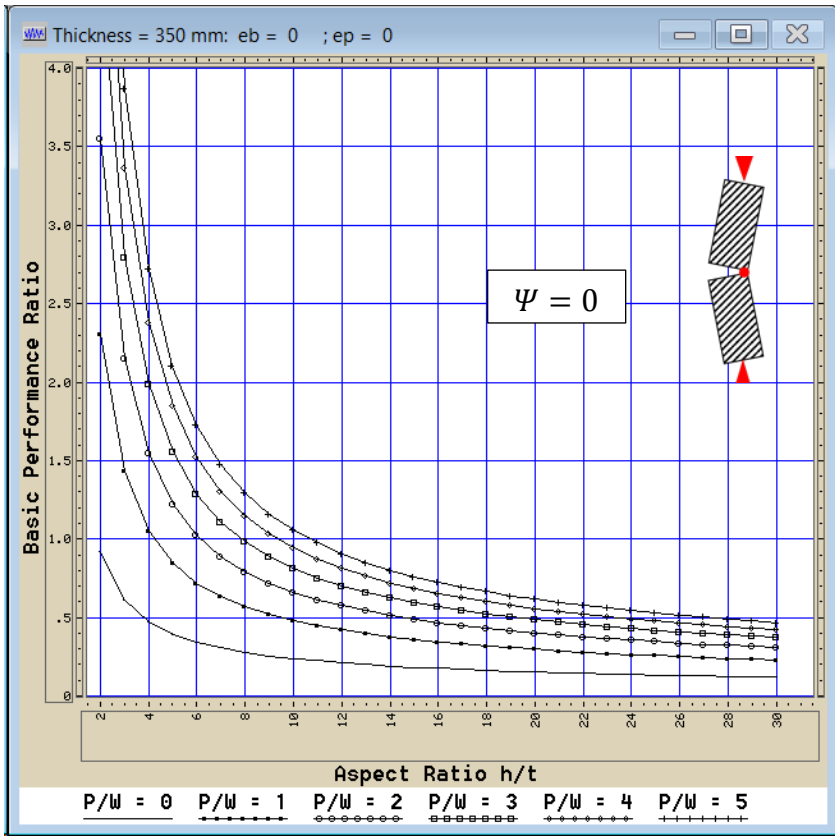
(b) For $e_b = +t/2$ and $e_p = 0$ ($t_{Gross} = 350$ mm)



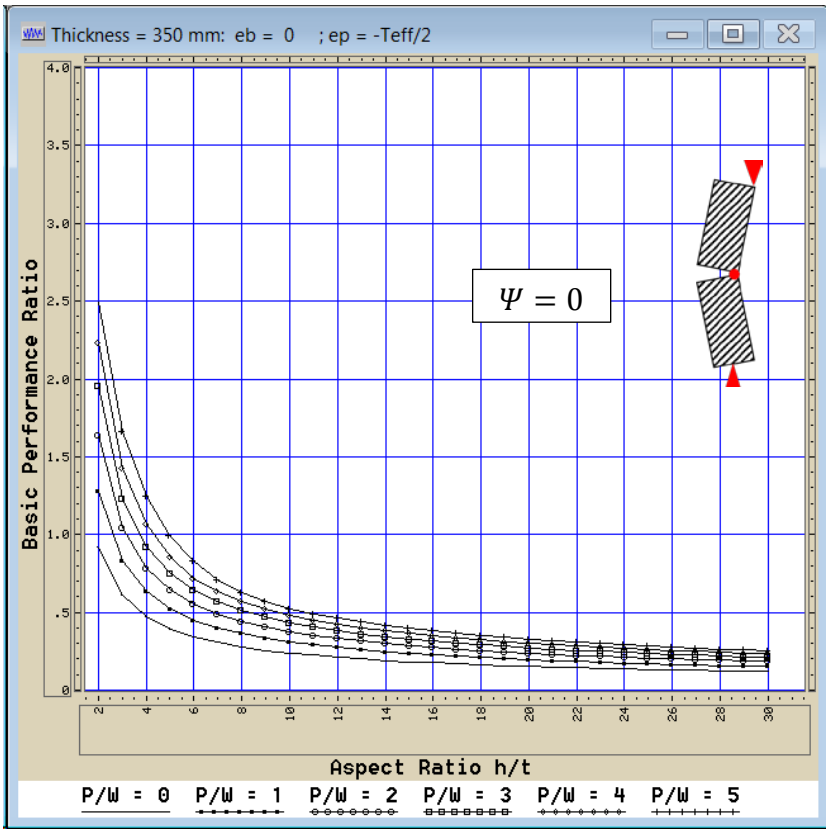
(c) For $e_{\mathrm{b}} = +t/2$ and $e_{\mathrm{p}} = -t/2$ ($t_{\mathrm{Gross}} = 350$ mm)



(d) For $e_{\mathrm{b}} = 0$ and $e_{\mathrm{p}} = t/2$ ($t_{\mathrm{Gross}} = 350$ mm)

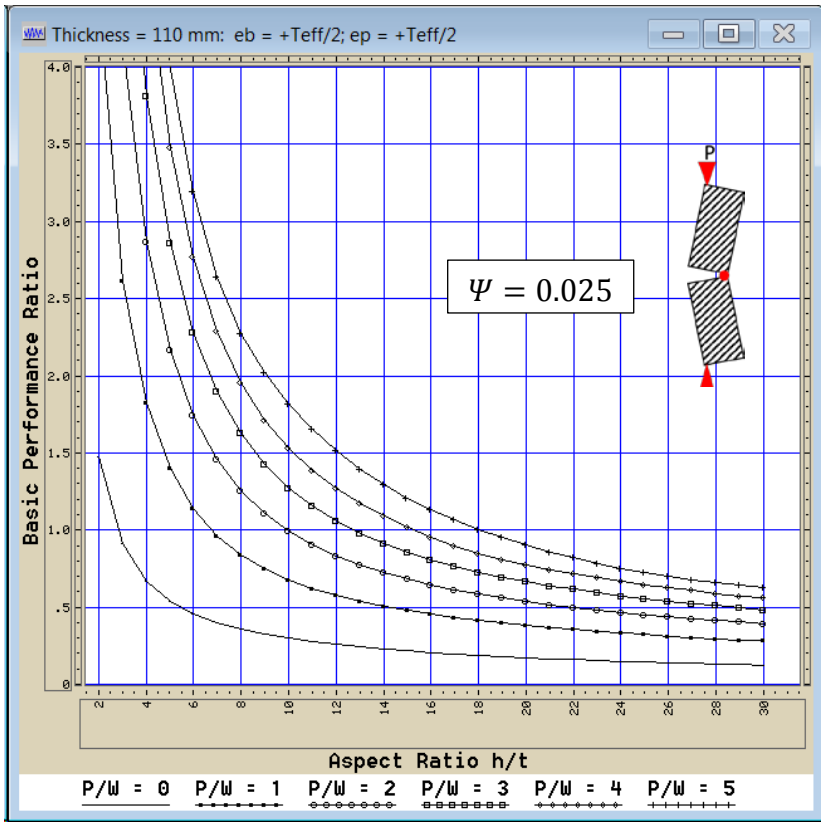


(e) For $e_b = 0$ and $e_p = 0$ ($t_{Gross} = 350$ mm)

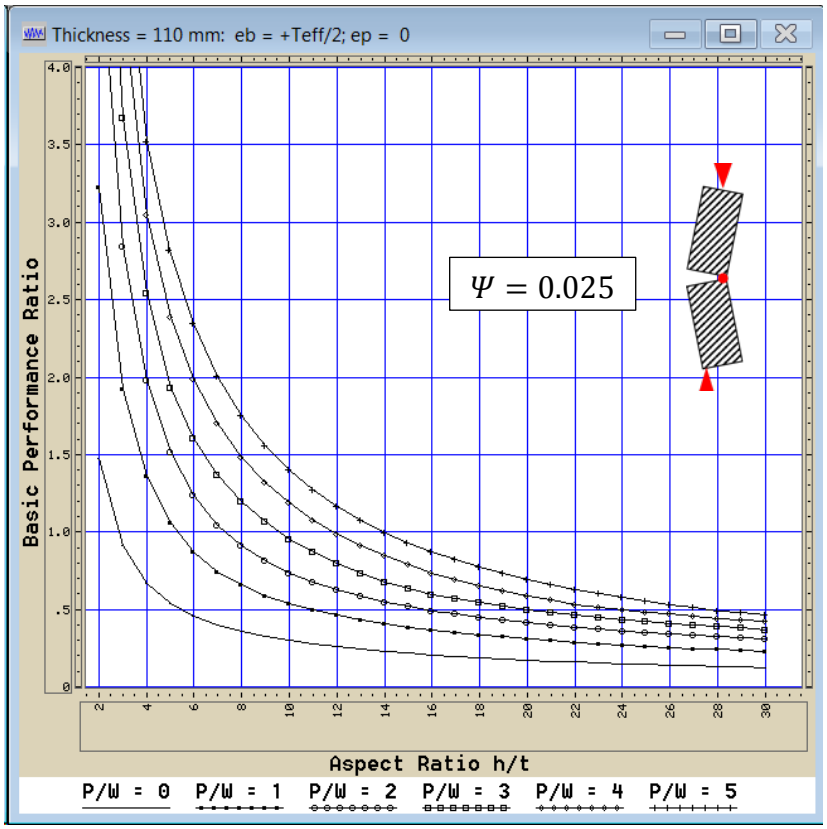


(f) For $e_b = 0$ and $e_p = -t/2$ ($t_{Gross} = 350$ mm)

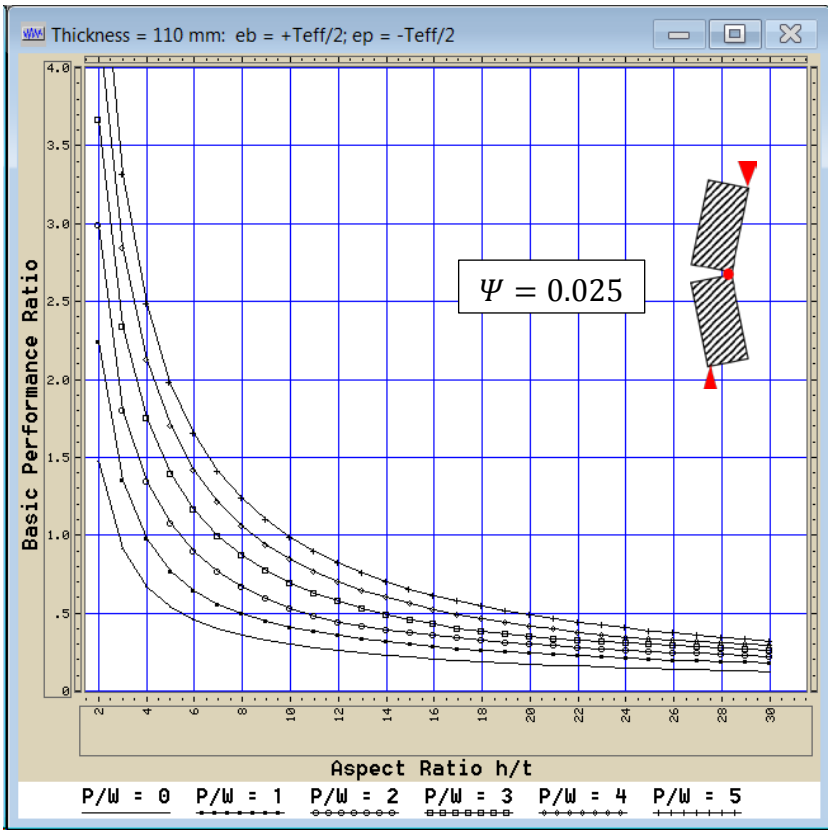
Figure C8C.3: 350 mm thick one-way vertically spanning face-loaded walls ($\Psi = 0$)



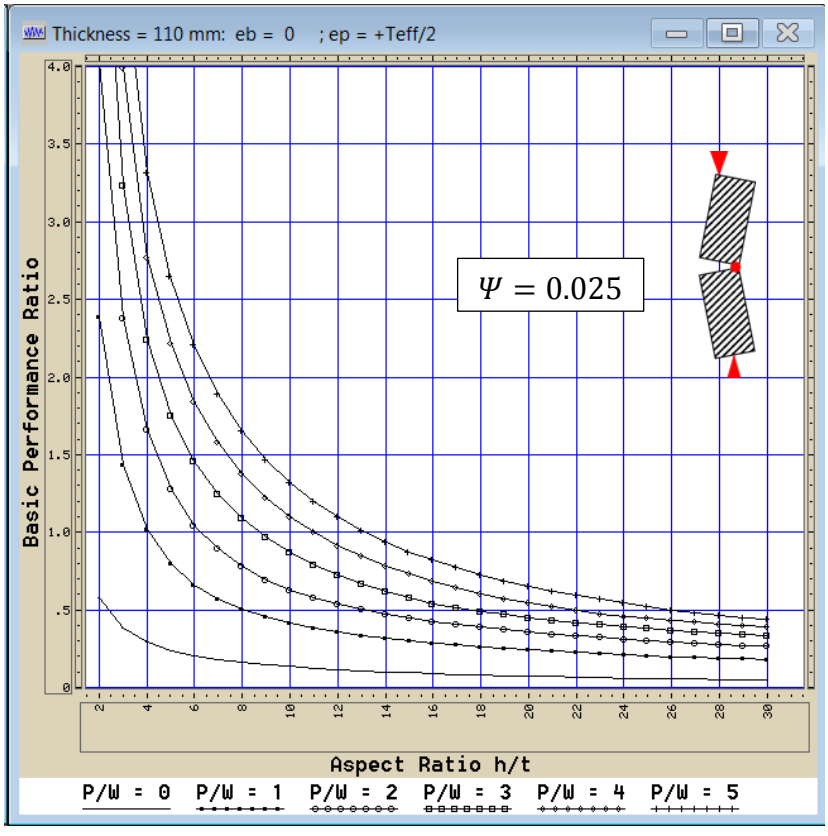
(a) For $e_b = +t/2$ and $e_p = +t/2$ ($t_{Gross} = 110$ mm)



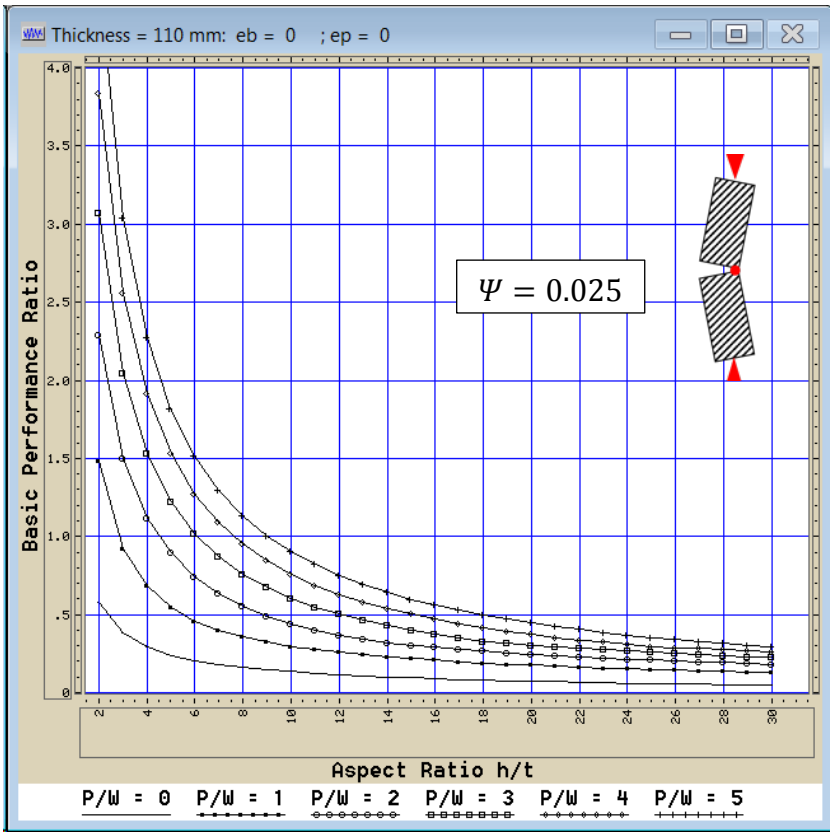
(b) For $e_b = +t/2$ and $e_p = 0$ ($t_{Gross} = 110$ mm)



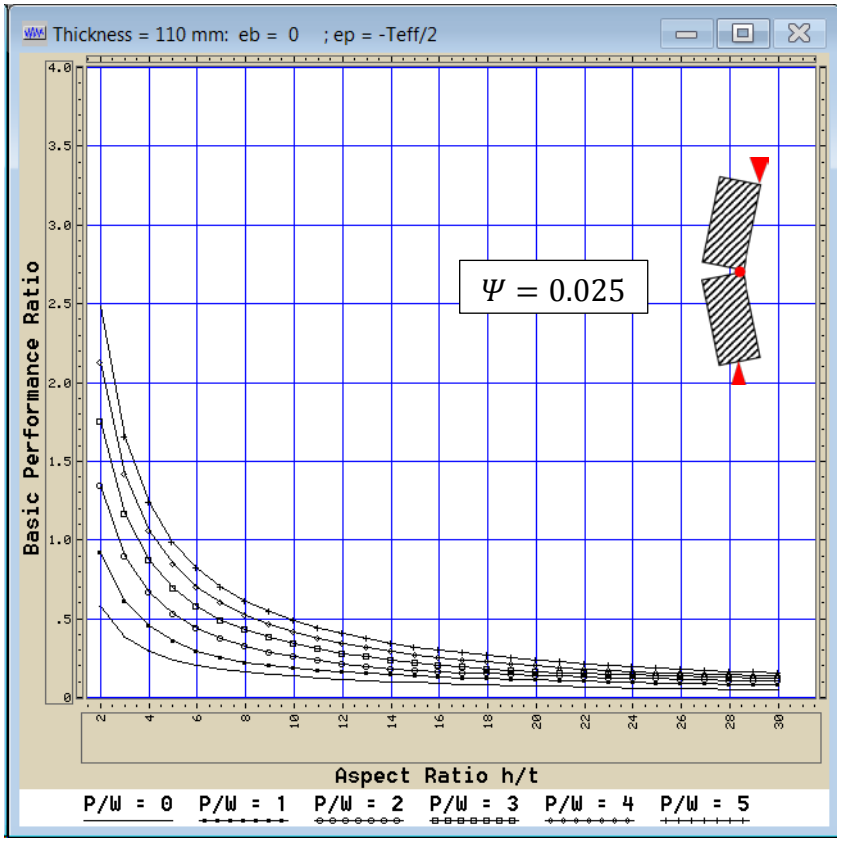
(c) For $e_b = +t/2$ and $e_p = -t/2$ ($t_{Gross} = 110$ mm)



(d) For $e_b = 0$ and $e_p = t/2$ ($t_{Gross} = 110$ mm)

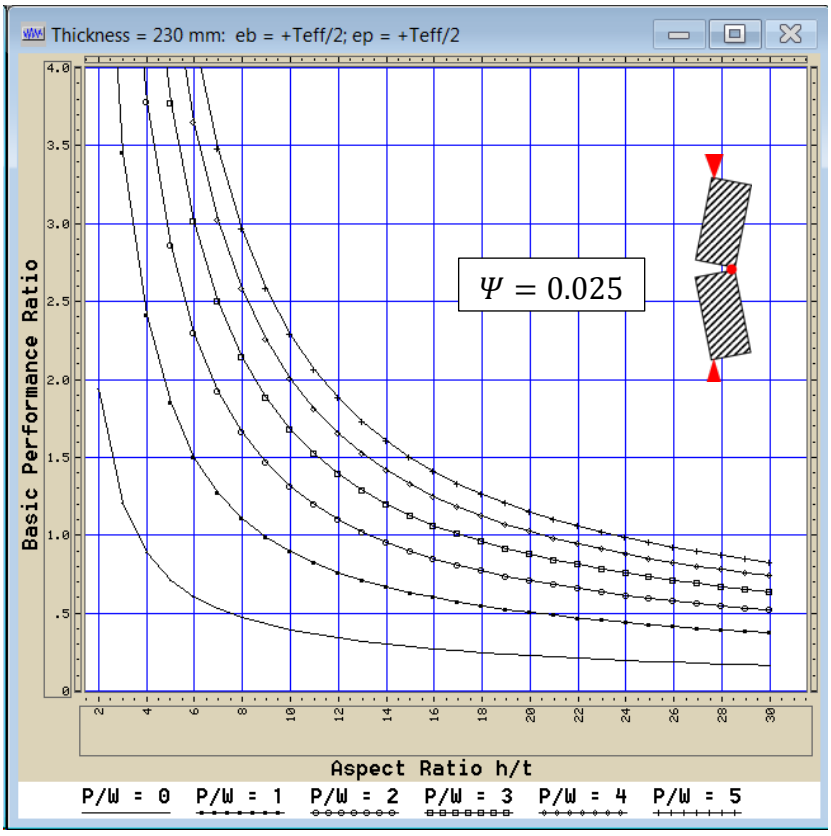


(e) For $e_b = 0$ and $e_p = 0$ ($t_{Gross} = 110$ mm)

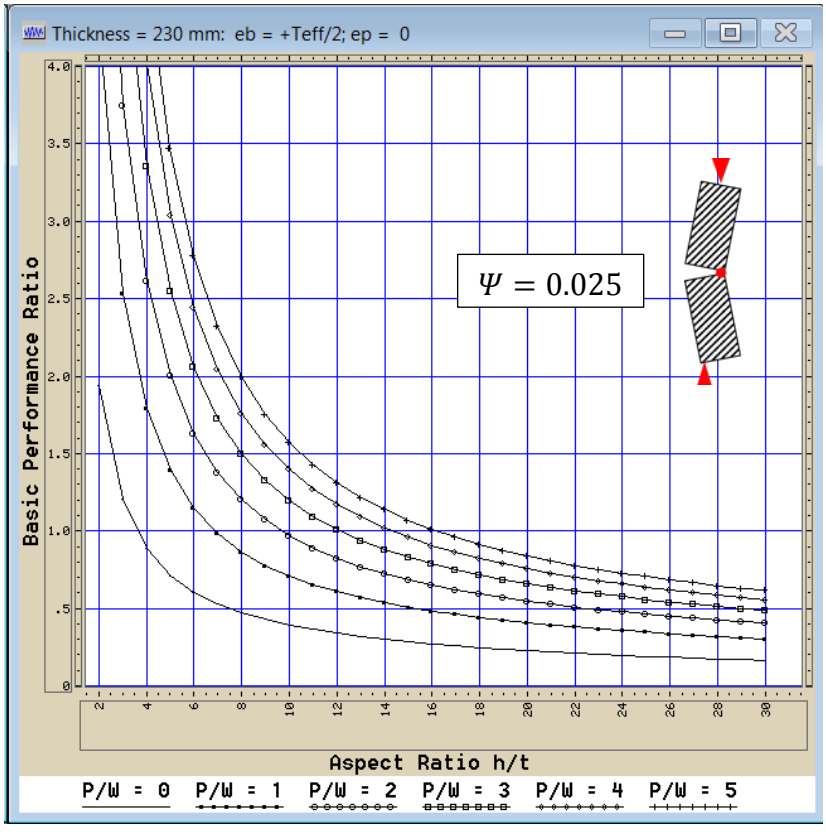


(f) For $e_b = 0$ and $e_p = -t/2$ ($t_{Gross} = 110$ mm)

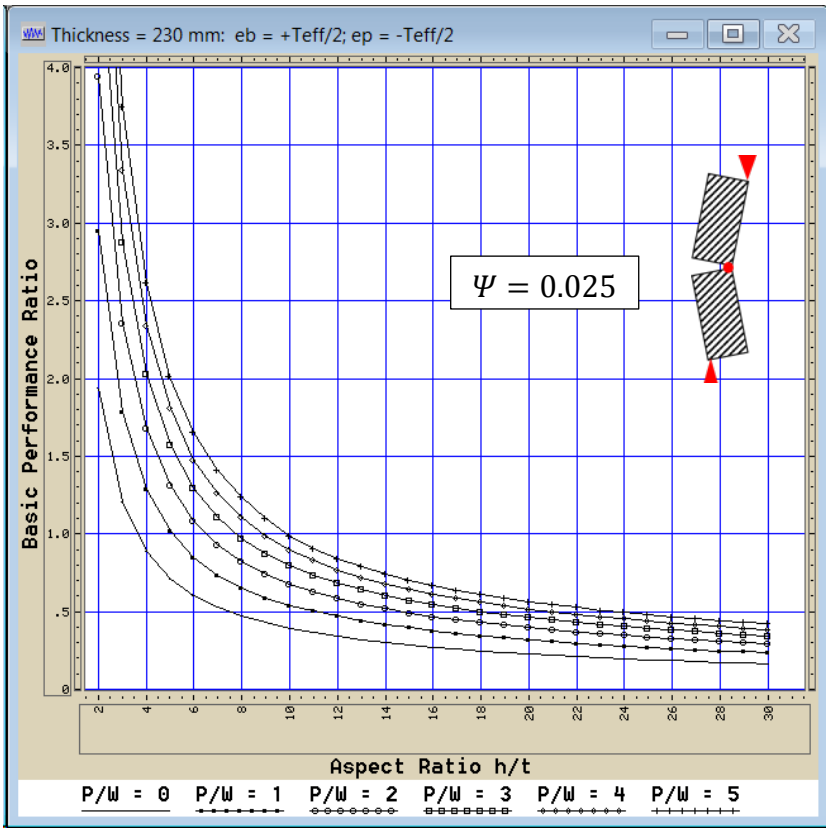
Figure C8C.4: 110 mm thick one-way vertically spanning face-loaded walls ($\Psi = 0.025$)



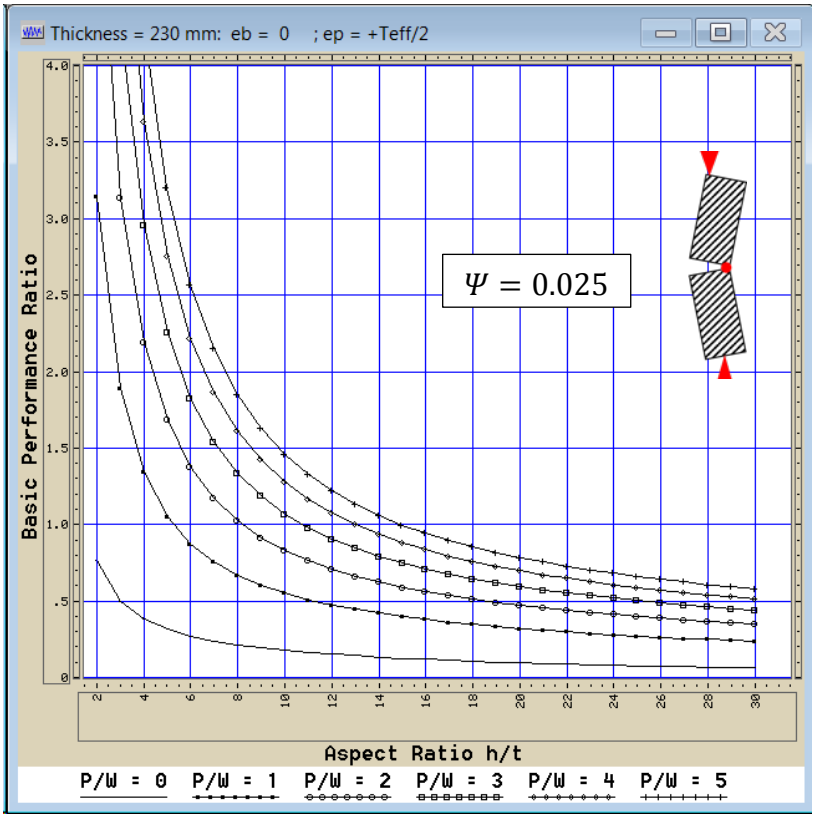
(a) For $e_b = +t/2$ and $e_p = +t/2$ ($t_{Gross} = 230$ mm)



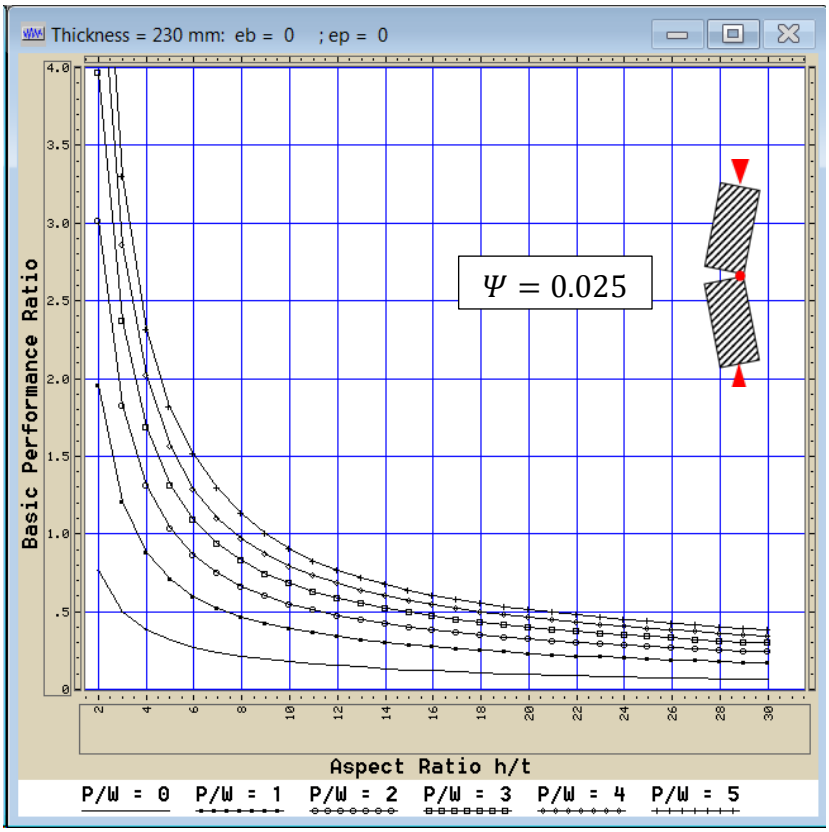
(b) For $e_b = +t/2$ and $e_p = 0$ ($t_{Gross} = 230$ mm)



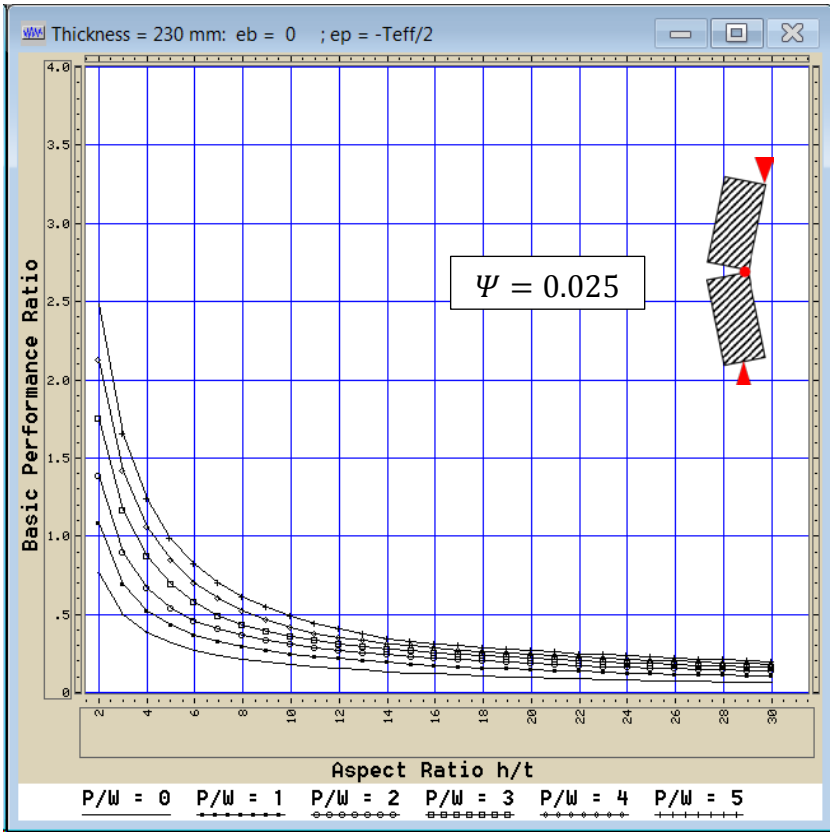
(c) For $e_b = +t/2$ and $e_p = -t/2$ ($t_{Gross} = 230$ mm)



(d) For $e_b = 0$ and $e_p = t/2$ ($t_{Gross} = 230$ mm)

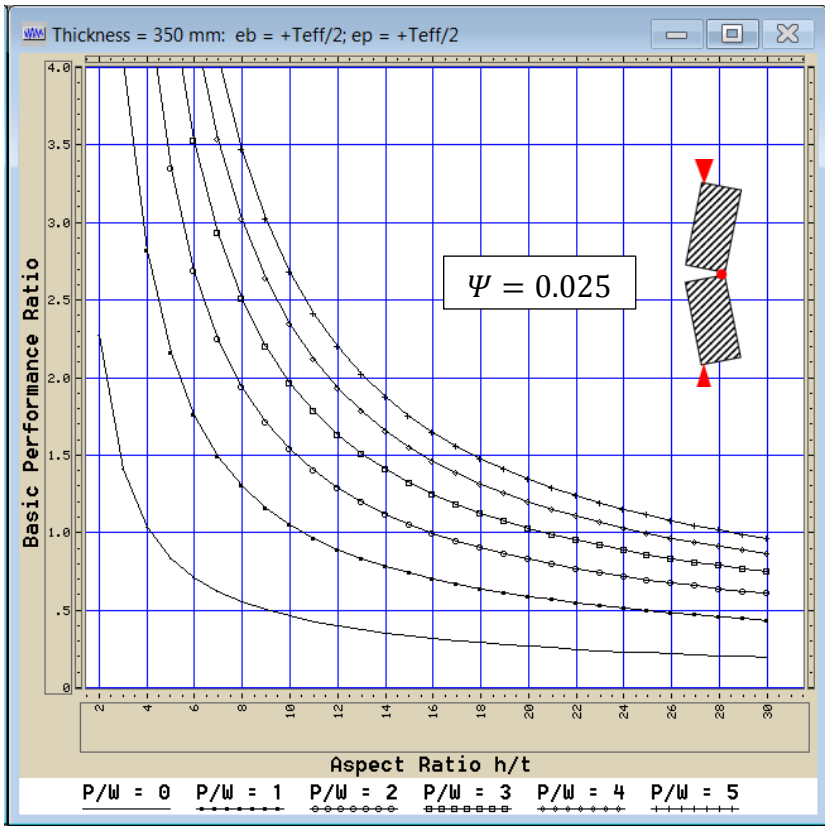


(e) For $e_b = 0$ and $e_p = 0$ ($t_{Gross} = 230$ mm)

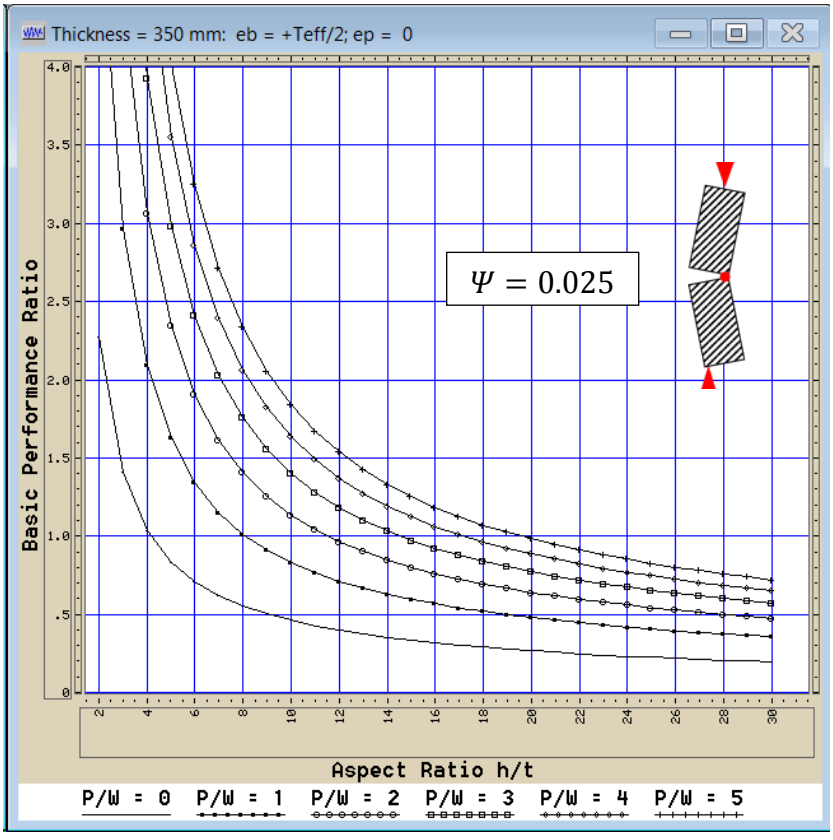


(f) For $e_b = 0$ and $e_p = -t/2$ ($t_{Gross} = 230$ mm)

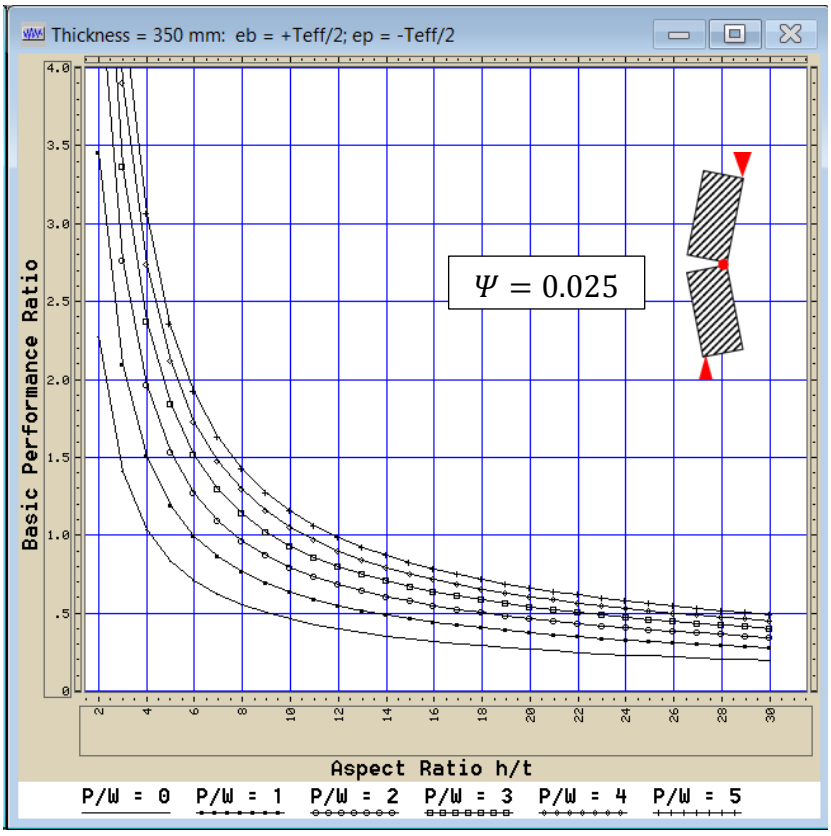
Figure C8C.5: 230 mm thick one-way vertically spanning face-loaded walls ($\Psi = 0.025$)



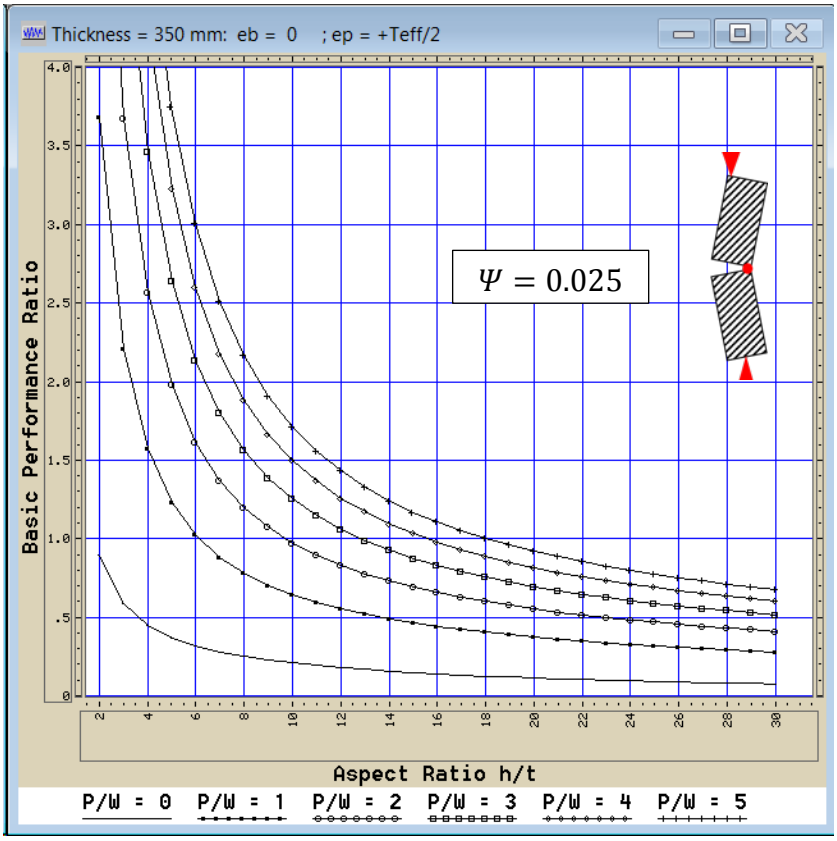
a) For $e_b = +t/2$ and $e_p = +t/2$ ($t_{Gross} = 350$ mm)



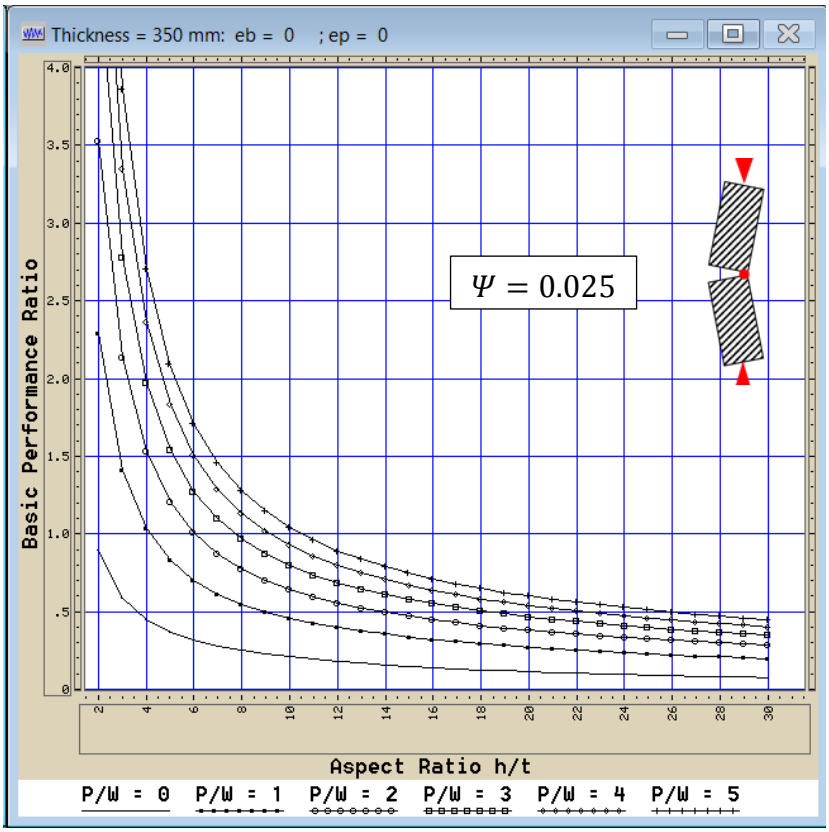
b) For $e_b = +t/2$ and $e_p = 0$ ($t_{Gross} = 350$ mm)



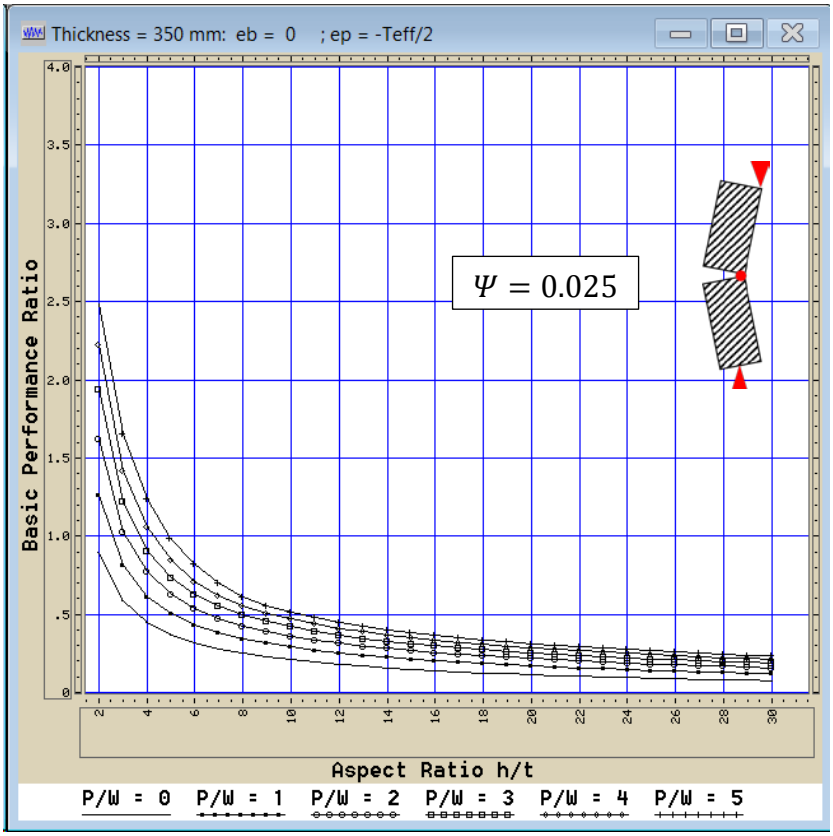
c) For $e_b = +t/2$ and $e_p = -t/2$ ($t_{Gross} = 350$ mm)



d) For $e_b = 0$ and $e_p = t/2$ ($t_{Gross} = 350$ mm)

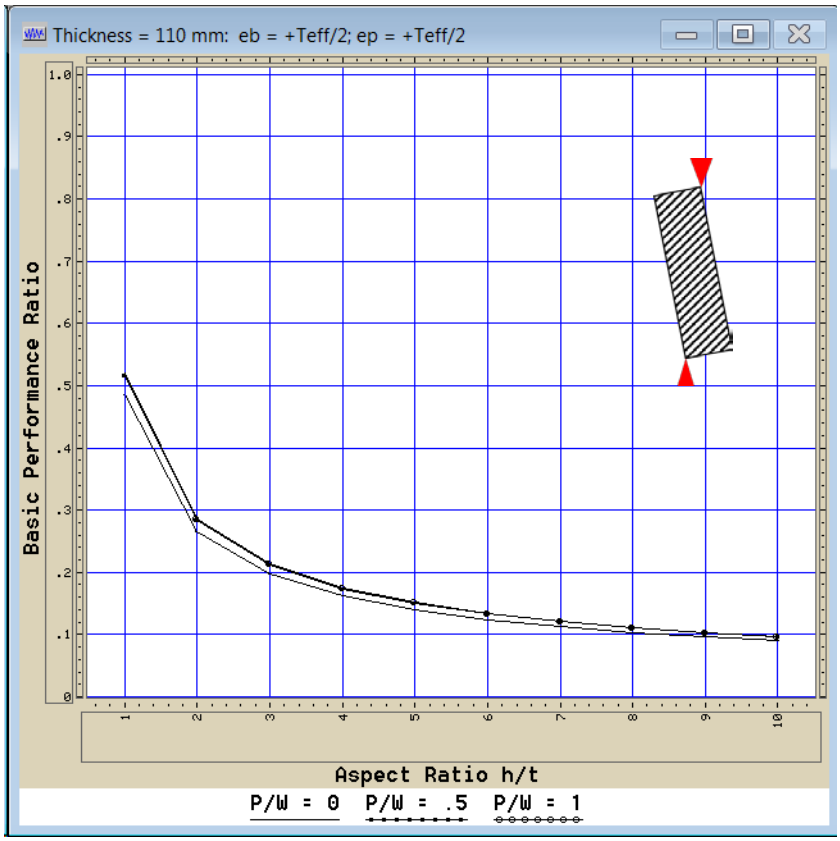


e) For $e_b = 0$ and $e_p = 0$ ($t_{Gross} = 350$ mm)

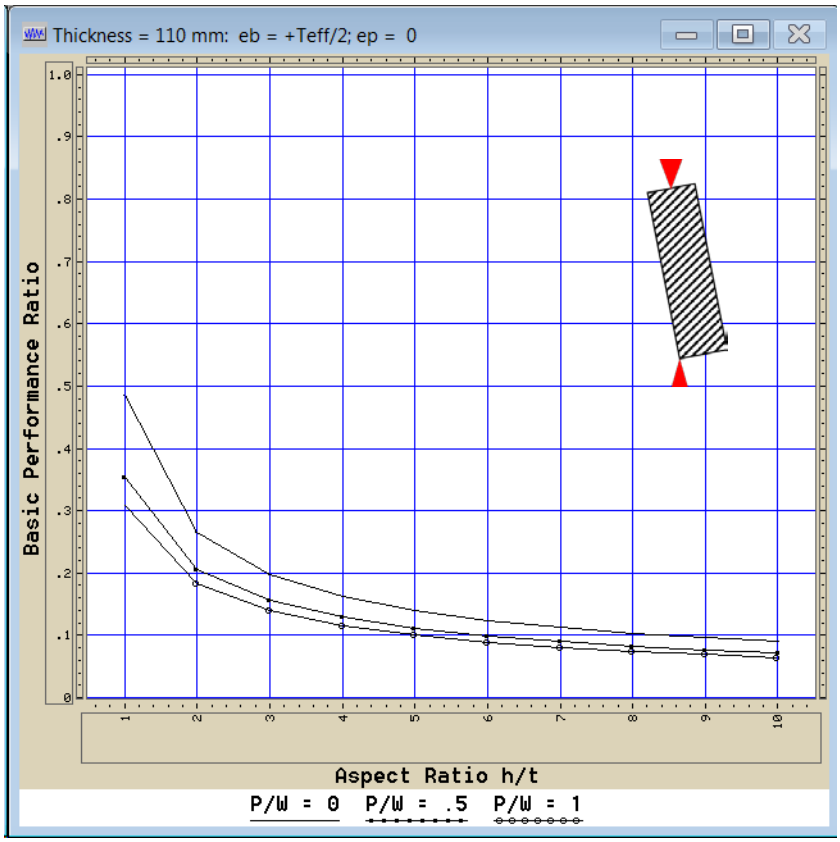


f) For $e_b = 0$ and $e_p = -t/2$ ($t_{Gross} = 350$ mm)

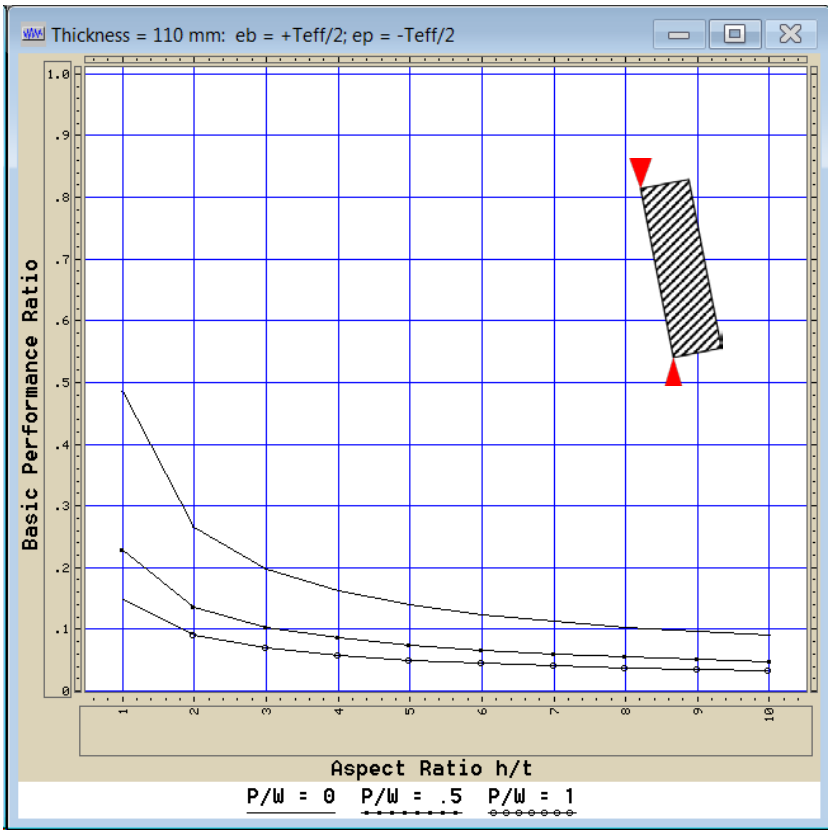
Figure C8C.6: 350 mm thick one-way vertically spanning face-loaded walls ($\Psi = 0.025$)



a) For $e_b = +t/2$ and $e_p = +t/2$ ($t_{Gross} = 110$ mm)

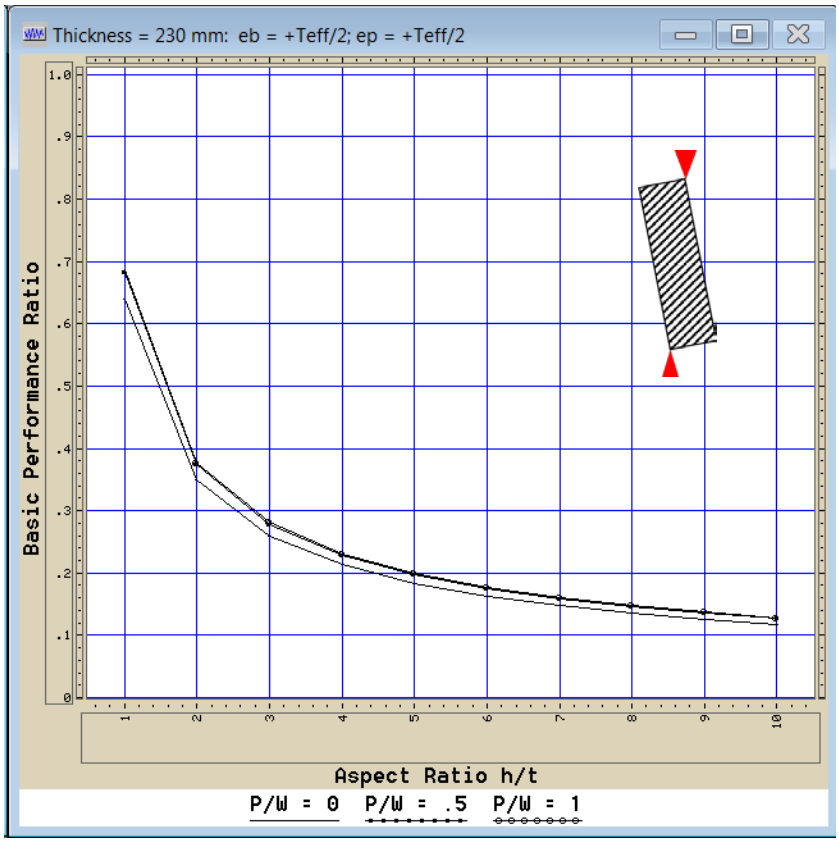


b) For $e_b = +t/2$ and $e_p = 0$ ($t_{Gross} = 110$ mm)

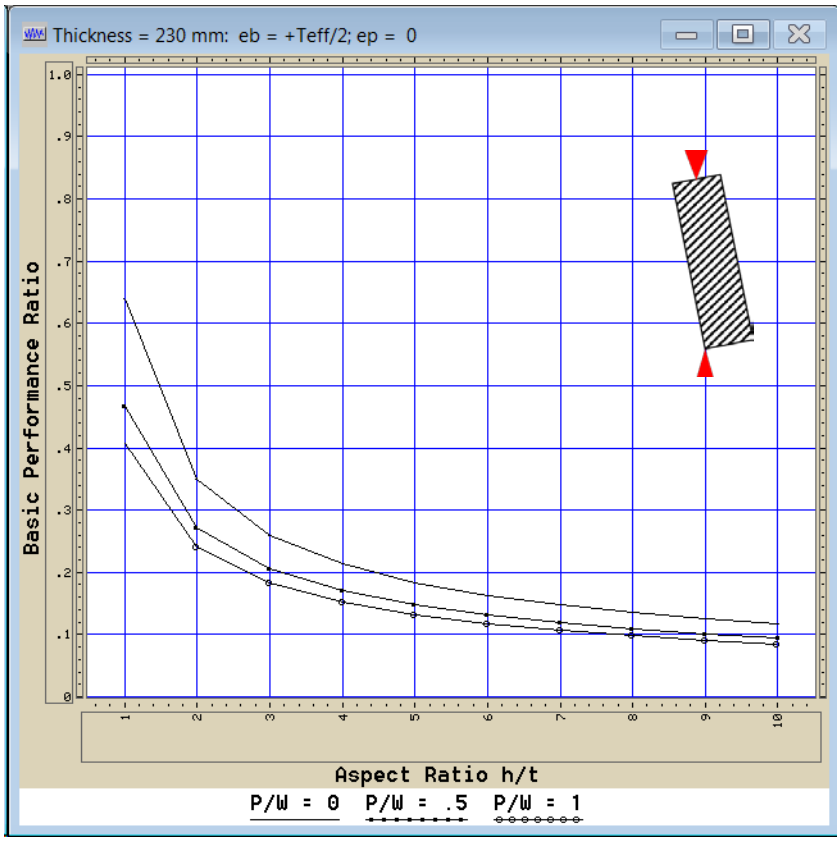


c) For $e_b = +t/2$ and $e_p = -t/2$ ($t_{Gross} = 110$ mm)

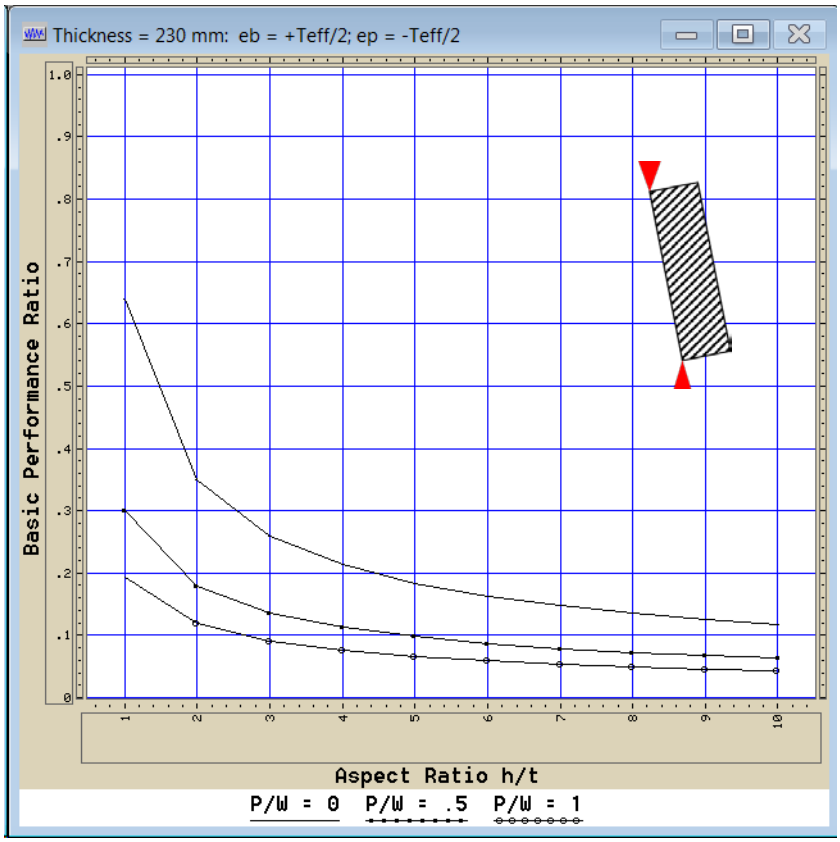
Figure C8C.7: 110 mm thick cantilever wall



a) For $e_b = +t/2$ and $e_p = +t/2$ ($t_{Gross} = 230$ mm)

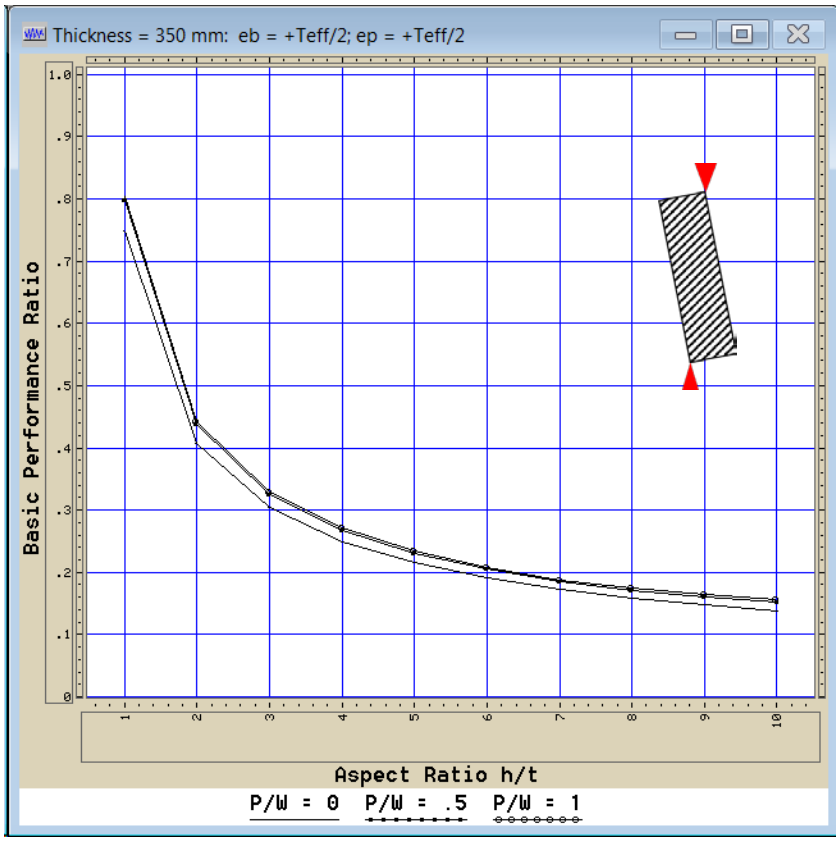


b) For $e_{\mathrm{b}} = +t/2$ and $e_{\mathrm{p}} = 0$ ($t_{\mathrm{Gross}} = 230$ mm)

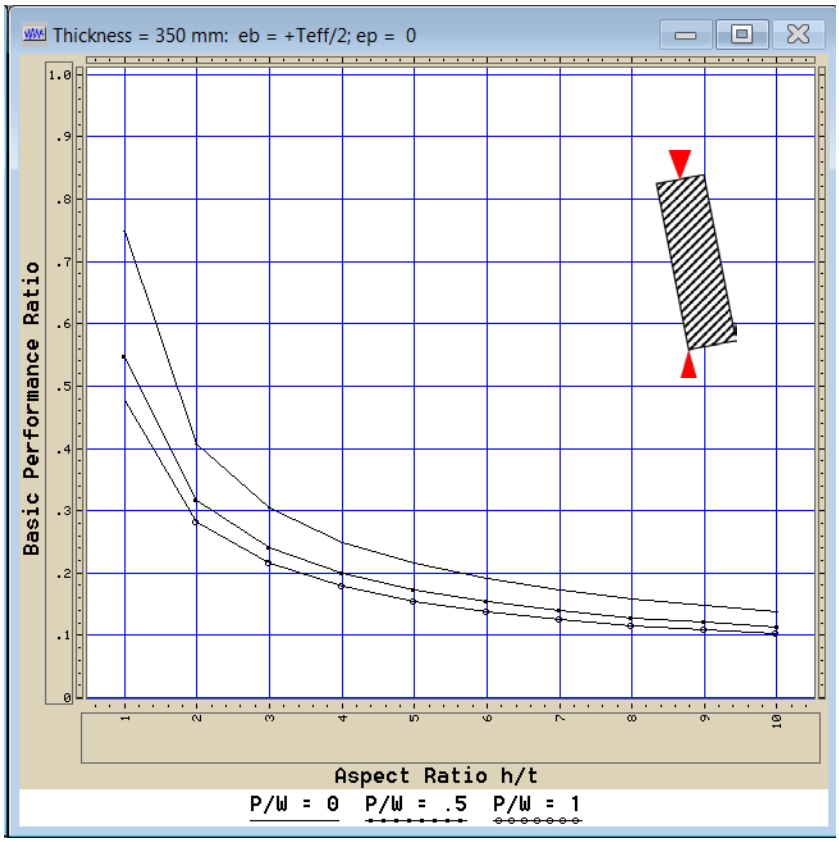


c) For $e_{\mathrm{b}} = +t/2$ and $e_{\mathrm{p}} = -t/2$ ($t_{\mathrm{Gross}} = 230$ mm)

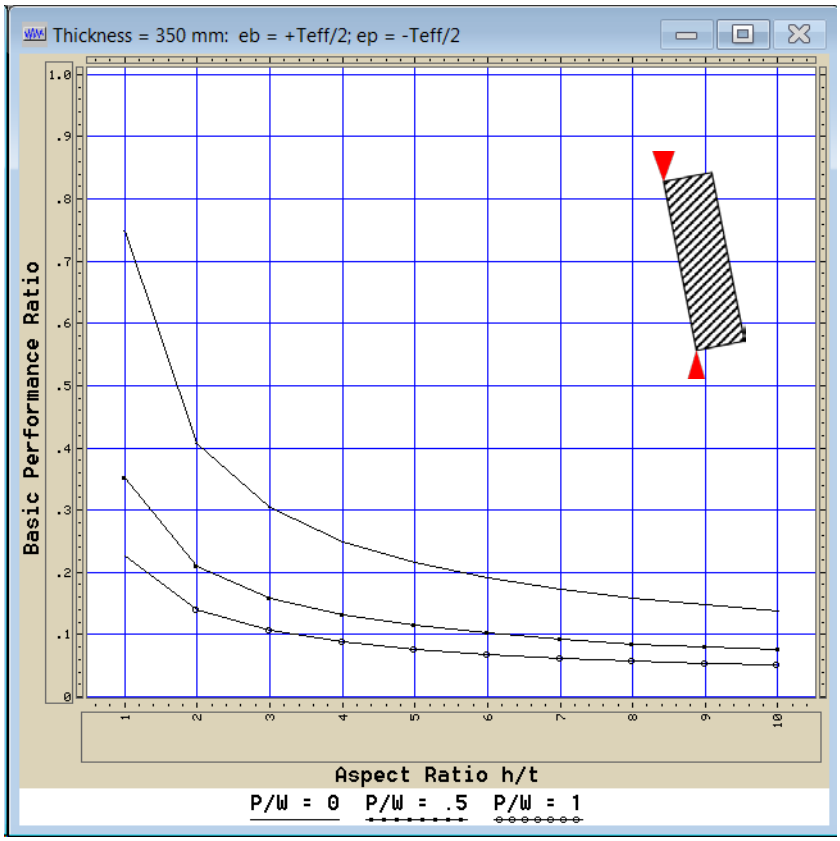
Figure C8C.8: 230 mm thick cantilever wall



a) For $e_b = +t/2$ and $e_p = +t/2$ ($t_{Gross} = 350$ mm)



b) For $e_b = +t/2$ and $e_p = 0$ ($t_{Gross} = 350$ mm)



c) For $e_b = +t/2$ and $e_p = t/2$ ($t_{Gross} = 350$ mm)

Figure C8C.9: 350 mm thick cantilever wall

PART C

Timber Buildings C9

Document Status

Version	Date	Purpose/ Amendment Description
1	July 2017	Initial release

This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

This document may be downloaded from www.EQ-Assess.org.nz in parts:

1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

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Please go to www.EQ-Assess.org.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.EQ-Assess.org.nz

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C9. Timber Buildings

C9.1 General

C9.1.1 Scope and outline of this section

This section provides guidance for the Detailed Seismic Assessment (DSA) of timber buildings to enable a consistent approach with the other materials addressed in these guidelines. In particular, it should assist by providing information on common forms of timber construction and estimation of the relevant member/element/system capacities. It builds on the section "Detailed Assessment of Timber Structures" in the previous version of these guidelines (NZSEE, 2006).

This section includes guidance for assessing:

- timber framed buildings where the timber framing in conjunction with lightweight materials provides bracing resistance to lateral loads, and
- engineered timber buildings that incorporate elements such as timber portals.

When assessing buildings that are constructed primarily from other materials (such as unreinforced masonry (URM) or concrete) but include components such as timber diaphragms (refer to Section C9.6.3) which may influence their seismic behaviour, this section should be read in conjunction with the relevant material sections (e.g. Section C8 Unreinforced Masonry Buildings and Section C5 Concrete Buildings).

Note:

Timber framed buildings and engineered timber buildings meeting modern design standards are not expected to be earthquake prone unless a particularly vulnerable aspect is present and, even then, this would need to be one which would lead to a significant life safety hazard in the event of failure.

A DSA of a timber building is typically performed after an Initial Seismic Assessment (ISA) has been undertaken in accordance with Part B of these guidelines. It should be noted that an ISA can identify high risk building components such as URM brick walls, heavy roofs, chimneys and poor foundation systems that can adversely affect the performance of a timber building. The mitigation or replacement of these undesirable features can increase the expected building performance, potentially making it unnecessary to undertake a detailed analysis/assessment.

C9.1.2 Definitions and acronyms

Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable strength capacity has been reached. A brittle structure has little tendency to deform inelastically before it fractures.
Capacity design	A design process for new buildings that identifies zones where post elastic response is acceptable and details these accordingly. All other parts of the primary structure are then designed to ensure other undesirable inelastic response mechanisms are suppressed.
Cross laminated timber (CLT)	Engineered wood made from multiple layers of boards placed cross-wise to adjacent layers for increased rigidity and strength
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines
Diaphragm	A horizontal structural element (usually a suspended floor or ceiling or a braced roof structure) that is strongly connected to the walls around it and distributes earthquake lateral forces to vertical elements, such as walls, of the lateral force-resisting system. Diaphragms can be classified as flexible or rigid.
Ductile/ductility	Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
European Yield Model (EYM)	Method for assessing connection design
Glulam	Glued laminated timber. A structural timber product made from layers of timber bonded with structural adhesives.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.
Irregular building	A building that has an irregularity that could potentially affect the way in which it responds to earthquake shaking. A building that has a sudden change in its plan shape is considered to have a horizontal irregularity. A building that changes shape up its height (such as one with setbacks or overhangs) or that is missing significant load-bearing elements is considered to have a vertical irregularity. Structural irregularity is as defined in NZS 1170.5:2004.
Laminated veneer lumber (LVL)	Engineered wood composite made from rotary peeled veneers, glued with a durable adhesive and laid up with parallel grain orientation to form long continuous sections
Lateral load	Load acting in the horizontal direction, which can be due to wind or earthquake effects
Load path	A path through which vertical or seismic forces travel from the point of their origin to the foundation and, ultimately, to the supporting soil
Oriented strand board (OSB)	Engineered wood particle board formed by adding adhesives and then compressing layers of wood strands (flakes) in specific orientations
Plywood	Layered panel product comprising veneers of solid wood bonded to adjacent layers, with grain direction orientated at right angles
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be part of the primary gravity structure.

Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects this is determined using probable material strengths. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Sarking	Typically in New Zealand, timber construction board material fixed to timber framing to provide a diaphragm. Provides a surface to which other materials can be applied.
Shear wall	A wall which resists lateral loads along its primary axis (also known as an in-plane wall)
Sheathing	The board, lining or panel material used in floor, wall and roof assemblies
Simple Lateral Mechanism Analysis (SLaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (pushover) relationship for the building as a whole
Stressed skin panels	Structural flat plates which rely on composite action for resistance to out-of-plane loads. Flexural strength is provided by the skins and shear resistance is provided by the filling of webs between the skins.

C9.1.3 Notation, symbols and abbreviations

Symbol	Meaning
%NBS	Percentage of new building standard as assessed by application of these guidelines
A	Sectional area of one chord (mm^2)
a	Aspect ratio of shear walls: • 0 when relative movement along sheet edges is prevented • 1 when transverse sheathing panels are used • 2 when 2.4 x 1.2 m panels are orientated with the 2.4 m length parallel with the diaphragm chords (= 0.5 alternative orientation)
A_{p}	Sectional area of the plate (mm^2)
B	Depth of diaphragm (mm)
B	Distance between diaphragm or shear wall chord members (mm)
B	Length of the wall
b	Width of sheathing board (mm)
E	Elastic modulus of the chord members (MPa)
e_{n}	Nail slip resulting from the shear force V (mm)
F_{b}	Probable bending strength of a board (N/mm^2)
F_{c}	Probable strength of a sheathing board in compression parallel to the grain (MPa)
F_{n}	Probable nail strength (N)
F_{prob}	Probable (strength) capacity of a diaphragm
G	Shear modulus of the sheathing (MPa)
H	Height of the wall or storey under consideration (mm)
k_{i}	Modification factor from NZS 3603:1993
L	Span of a diaphragm (mm)

l	Spacing between joints or studs (mm)
M	Moment couple formed by a pair of nails
m	Number of sheathing panels along the length of the edge chord
N	Total number of nails across the width of a diaphragm, or number of nails fixing a board to a plate as appropriate
$P-delta$	Structural actions induced as a consequence of the gravity loads being displaced horizontally due to horizontal effects
s	Nail spacing (mm)
S_{p}	Structural performance factor in accordance with NZS 1170.5:2004
t	Thickness of the sheathing board (mm)
V	Shear force in storey under consideration (N)
v'	Probable interboard friction capacity for timber floors
$(V_{\mathrm{prob}})_{\mathrm{i}}$	Probable horizontal shear in Newtons
V_{prob}	Probable shear strength
v_{prob}	Probable shear force in Newtons per metre length of wall
W	Lateral load applied to a horizontal diaphragm (N)
w	A universally distributed load causing bending in the plate members of a diaphragm
z	Section modulus of the sheathing board = $\frac{b^2t}{6}$, where t is the thickness of the board (mm^3)
Δ_1	Diaphragm flexural deformation considering chords acting as a moment resisting couple (mm)
Δ_2	Diaphragm shear deformation resulting from beam action of the diaphragm (mm)
Δ_3	Deformation due to nail slip for horizontal diaphragm (mm)
Δ_4	Deformation due to support connection relaxation (mm)
Δ_5	Wall shear deformation (mm)
Δ_6	Deformation due to nail slip (mm)
Δ_7	Deformation due to flexure as a cantilever (may be ignored for single storey shear walls with H/B ratios less than 1.0)
δ_{c}	Vertical downward movement (mm) at the base of the compression end of the wall (this may be due to compression perpendicular to the grain deformation in the bottom plate)
Δ_{h}	Mid span deflection of a horizontal diaphragm
δ_{t}	Vertical upward movement (mm) at the base of the tension end of the wall (this may be due to deformations in a nailed fastener and the members to which it is anchored)
Δ_{w}	Horizontal inter-storey deflection in one storey of a shear wall
θ	Flexural rotation at base of storey under consideration (radians)
μ	Structural ductility factor in accordance with NZS 1170.5:2004
ϕ	Strength reduction factor

C9.2 New Zealand Construction Practices

C9.2.1 Timber building types

Timber is a readily available building material in New Zealand. Since the earliest European settlement it has been used widely for many different building types including residential, office, industrial and public buildings.

The two main categories of timber buildings are:

- timber framed structures such as those designed using non-specific design guides and standards, and
- engineered buildings such as halls, commercial and industrial buildings.

Some examples are shown in Figure C9.1.

Timber is also used in other building types (refer to Section C9.2.4).

Figure C9.1: Examples of timber buildings

C9.2.2 Timber framed structures

C9.2.2.1 Frames and bracing

Timber framed structures employ what is commonly referred to as stick framing: small section timber such as 90 mm x 45 mm (historically 4” x 2”). These elements are combined to create wall frames with timber studs, and top and bottom plates. In older structures (prior to the introduction of NZS 3604:1978), bracing was commonly provided by the addition of let-in diagonal braces (typically 6” x 1” or 4” x 1”) or cut-in diagonal braces (typically 4” x 2” or 3” x 2”) between the studs. Occasionally, older buildings with timber framed walls rely on an internal lathe and plaster lining to provide the bracing rather than employing diagonal members. For a period between the use of lathe and plaster and of sheet linings, around the early 1900s to 1920s, wide horizontal boards approximately 25 mm were often used as a backing for scrim and then wallpaper was applied over this.

Modern timber framed walls are likely to be reliant on their lining materials for providing bracing resistance. Linings include plasterboard, plywood, oriented strand board (OSB), particle board and sometimes fibre cement board.

C9.2.2.2 Floors

Floors of timber framed structures typically consist of multiple horizontal joist members between 400 mm and 600 mm apart. The span limits for sawn timber members tend to restrict the size of rooms in the building. More modern buildings may use engineered products for the floor system, such as: engineered wood joists (I joists), laminated veneer lumber (LVL) joists, nail plated parallel flange truss joists, solid glulam panels, or cross laminated timber (CLT) panels. In these cases, the spans are likely to be greater than for sawn timber. The joists of ground floors are typically seated on timber bearers on piles. Upper floor joists are typically seated on the top plates of the walls or a ribbon plate side fixed to wall framing.

Older floors are generally constructed using tongue and groove strip timber members up to approximately 200 mm wide, fixed with two nails at each joist crossing. Some old commercial structures may have a “mill floor” which is a solid panel consisting of timber planks on edge and nail-fixed together. More modern floors are typically constructed with sheet materials such as particle board, plywood or fibre cement board products, fixed with nails or screws around sheet perimeters. Other fixings are generally used in the in-field area of the sheet at larger spacings to the intermediate joists.

C9.2.2.3 Roofs and ceilings

Older style roof framing includes rafters spanning between eaves and ridges (often supported at intermediate points by a propped under-purlin), overlaid with purlins to which the roofing products are attached. Sometimes, solid or hit-and-miss strip timber sarking is present as an alternative to purlins. Ceiling linings can be supported on ceiling joists (or the underside of floor joists in upper floors), which also span between walls. Ceiling joists are typically of a smaller depth than floor joists and intermediate support is often provided from above.

Modern roof construction typically consists of timber roof trusses with pressed metal tooth plate connectors spanning from outside wall to outside wall.

In both cases, bracing is provided in the roof space (either in the plane of the roof or in the roof space down to the top plates of internal walls), particularly if the roof has gable ends. Because of their shape, hip roofs generally support themselves against lateral loads and other bracing is not usually necessary.

C9.2.3 Engineered timber buildings

While engineered timber buildings can take many forms, their main characteristic is that they generally use larger member sizes, such as heavy posts and beams, to achieve greater spans. Systems include portal frames with moment resisting knee joints and bolted timber trusses. The portals may be constructed from glue laminated timber, round wood or LVL with glued, bolted or splice plated knee joints (e.g. steel and griplam nails or plywood with dowel type connectors, e.g. nails, screws, drift pins, bolts).

When heavy bolted timber trusses are used in conjunction with heavy sawn timber columns, a diagonal timber brace is often included at the connection between the truss and the column to provide a moment connection, thus creating portal-like action. In the orthogonal direction either steel angle or rod bracing is employed, or light timber framed walls with timber sheet material is used to resist the lateral loads.

C9.2.4 Timber in other building types

Timber has been used extensively in buildings primarily constructed of other materials for floor joists, roof framing, flooring and sarking under roofs etc. If assessing such a building this section may need to be read in conjunction with the specific sections detailing the assessment of these other building types (e.g. Section C5 Concrete Buildings and Section C8 Unreinforced Masonry Buildings).

C9.3 Observed Behaviour of Timber Buildings in Earthquakes

C9.3.1 General performance

In general, low-rise timber framed buildings have performed extremely well with regard to life safety in large earthquakes.

Rainer and Karacabeyli (2000) carried out a survey of observed damage to timber framed buildings caused by eight significant earthquakes in the USA, Canada, New Zealand and Japan. Their study concluded that two-storey timber framed buildings largely met the life safety criterion required by design standards. The fatalities recorded in timber framed buildings were predominantly in larger (three-storey to four-storey) buildings or as a result of external hazards such as landslides. When subject to peak ground accelerations in excess of 0.6 g some Californian two-storey timber buildings exhibited soft-storey behaviour and suffered partial collapse; while in Kobe, Japan, minimal damage was observed.

In the 1987 earthquake centred at Edgecumbe in the Bay of Plenty, fewer than 50 of the nearly 7000 houses in the affected region suffered substantial damage and none collapsed (Pender and Robertson, 1987). The majority of buildings in this region were of 1ight timber frame construction and about two thirds were constructed during the period 1950 to 1979, prior to the introduction of NZS 3604:1978. Most of the significant damage occurred in building foundations. However, hundreds of houses suffered some lesser degree of damage including sliding off their foundations, damage to brick veneers, chimney collapse, and failure of foundation posts (pole frame structures) and roof struts.

The Christchurch earthquake of 22 February 2011 provided substantial evidence on building performance given that the majority of houses in the Canterbury region are light timber framed buildings. Buchanan et al. (2011a) summarised the observed damage to timber framed housing due to this earthquake, noting that single-storey and two-storey light timber framed buildings performed extremely well for life safety. The only recorded fatalities in timber framed residential buildings were attributed to rockfall.

The performance of engineered timber buildings was also reviewed by Buchanan et al. (2011b). The authors noted that these buildings generally performed well both for life safety and serviceability, with most buildings ready for occupation a short time after the event. Most of the damage that occurred resulted from lateral spreading, settlement resulting from liquefaction, and unusually high levels of horizontal and vertical ground acceleration.

Other observations included that structural and non-structural damage was common but, in general, the structural integrity of these buildings was maintained. A small number of soft-storey failures were observed in older two-storey timber framed houses, but these typically did not result in collapse. These failures were often due to minimal bracing in the lower floors, potentially as a result of alterations.

Significant damage was observed to the internal wall linings of some timber buildings, particularly those with an asymmetric layout and large window openings. Damage to and collapse of brick veneers, unreinforced chimneys and heavy roof tiles was common in areas subject to high peak ground accelerations.

Concrete slab foundations generally performed well unless subject to liquefaction induced settlement or lateral spreading; although failure of the connection to a foundation wall or edge thickening was common where the slabs were unreinforced.

Foundations with short concrete piles and concrete perimeter walls generally performed very well, particularly when the perimeter walls were reinforced. Similar to slab foundations, damage was observed to pile foundations when subject to liquefaction induced settlement or lateral spreading.

Note:

The general good performance of timber buildings in earthquakes is considered to be due, at least in part, to their relatively low supported mass and ability to deform considerably (via deformation in the connections) without loss of gravity load support. This means that while serviceability may not necessarily be achieved, it is unlikely that buildings of this type will create a significant life safety hazard even during severe earthquake shaking. Care should be taken, however, when there are elements within timber structures that either increase the mass (e.g. heavy wall partitions) or indicate a potentially vulnerable mechanism is present that would concentrate nonlinear behaviour (e.g. a cantilever or poorly cross braced sub-floor structure).

C9.3.2 Performance of timber framed school buildings in the Canterbury earthquakes

A large number of school buildings in Christchurch are constructed from timber. These include both classroom-type buildings of one or two storeys and large span buildings such as halls and gymnasia. These school buildings were reviewed extensively following the Canterbury earthquake sequence of 2010/11 (Opus International Consultants, 2015), providing a platform for reviewing the performance of timber buildings generally.

Despite high levels of peak ground acceleration during these earthquakes at a range of school sites across Canterbury, no school structures collapsed and no serious injuries or fatalities were recorded. Therefore, timber school buildings performed well in the Canterbury earthquakes from a life safety perspective, confirming previous expectations. However, significant damage was caused by lateral spreading and liquefaction.

Further, detailed engineering evaluations of these schools after the earthquakes have shown that timber buildings performed better than conventional methods of theoretical structural analysis would suggest. This is because timber buildings (with the exception of portal framed structures typically used in warehouses, halls and gymnasia) generally have many additional load paths that are not easily quantifiable but are able to carry and redistribute loads and deform significantly in a seismic event.

Note:

The results of two separate full scale tests commissioned by the Ministry of Education and one by Housing New Zealand in 2013 (refer to Appendix C9A) confirmed observations from the Canterbury earthquake sequence of the resilience of timber framed buildings. These tests confirmed the view held by many structural engineers that timber framed buildings constructed before the establishment of modern seismic codes have an inherent

lateral resistance and deformation capacity beyond that which can be readily calculated. Timber framed buildings meeting modern seismic code requirements are expected to have earthquake resilience that meets or exceeds current minimum building code requirements for life safety.

Previous work by the Ministry of Education following the 1998 National Structural and Glazing Survey included the replacement of most heavy roofs on its school buildings. This action undoubtedly improved the performance of the lightweight building stock and has reduced the risk of serious damage during seismic events.

C9.4 Assessment Approach

C9.4.1 General assumptions and considerations

This section outlines the assessment approach for both timber framed structures and engineered timber buildings. The approach and extent of analysis will vary with the building's complexity and the degree of certainty regarding element capacities, and should be in accordance with the objectives outlined in Section C1 and procedures in Section C2.

Also refer to Sections C1 and C2 for guidance on:

- documentation that should be sourced to undertake the assessment, and inspection requirements to verify the design is in accordance with the design documentation
- what, if any, intrusive testing should be considered, and
- the general assessment and analysis procedures that should be considered.

As noted in Section C9.1.1:

- Timber framed buildings and engineered timber buildings meeting modern design standards are not expected to be earthquake prone unless a particularly vulnerable aspect is present and, even then, this would need to be one which would lead to a significant life safety hazard in the event of failure.
- An ISA typically performed before any DSA can identify high risk building elements such as URM brick walls, heavy roofs, chimneys and poor foundation systems that can adversely affect the performance of a timber building. The mitigation or replacement of these can increase the expected building performance, potentially making it unnecessary to undertake a detailed analysis/assessment.

Note:

Analysis of the results from full-scale testing of timber buildings (Brunsdon, et al. (2014); Connor-Woodley (2015); and BRANZ (2015)) has indicated that the global seismic performance of these buildings is expected to be very good (when considered against life safety objectives) and far greater than the results of structural calculations may suggest.

As a result, a revised structural performance factor, $S_{p,}$ of 0.5 (a lower bound of 0.7 has typically been used) is recommended when completing a DSA for timber buildings as outlined later in this section.

The following general assumptions and considerations should be used in the assessment of timber buildings:

- The assessing engineer should have access to relevant design standards including NZS 1170.5:2004, NZS 3603:1993, NZS 3604:2011 and AS 1720.1:2010.
- The engineer should identify the critical and controlling load paths, the strength hierarchy, and likely mechanisms of the system to assist with determining the available ductility capacity using a rational analysis (where possible).
- An inelastic analysis is not considered necessary for the majority of timber buildings due to the flexibility of the diaphragms and ability to redistribute lateral load between timber elements of different stiffness on different bracing lines. However, an appreciation of the deformation capacity of timber elements is considered essential when these are being used in conjunction with elements of other material types and timber systems with significantly differing deformation capacities.

- The provision that 8% of the horizontal seismic base shear is applied at the eaves/roof level should only be considered if a heavy roof is present or a heavy wall is required to be propped at roof level.
- Assessment is based on probable capacities which are taken as the nominal capacities defined in NZS 3603:1993; i.e. taking the strength reduction factor, ϕ, equal to 1, but using probable material strengths.
- The probable material strengths and member/element capacities set out in Sections C9.5 and C9.6 respectively can be used as default values in the absence of better available data that is specific to the building. The element capacities in Section C9.6 assume that the load path into and out of each member is complete and sufficient to transfer the required demands. This should be confirmed.
- Traditional sawn timber has a wide range of strength properties, with the characteristic (lower fifth percentile) strength used in the design of new timber structures. In structures with many contributing timber members the assessment of the collective capacity should be assessed using the appropriate modification factors, k_{i} , from NZS 3603:1993.
- Where they are not visible and there is no drawing record, walls should be assumed to have no diagonal braces unless otherwise confirmed by site investigation.
- The specified lateral seismic deflection limits specified in NZS 1170.5:2004 are not expected to be relevant for typical timber buildings when the focus is on life safety. This is because the mass supported is typically low and considerable deflections can generally be sustained. However, this should not be considered a blanket relaxation as in some cases (e.g. when there is a large supported mass (roof or wall)) careful appraisal will be required before the deflection limits can be ignored.
- Portal framed structures are typically governed by deflection limits and non-seismic actions, so stiffness rather than strength will likely have governed section size. Joint strength and deformation capacity is then likely to be critical. The capacity of joints with dowel type fasteners is usually limited by timber bearing/crushing, which is a ductile mechanism.

For buildings constructed primarily of other materials (e.g. concrete or URM) but with timber elements, such as floors or roofs, that could affect their seismic performance, it is important to determine the state of the connection between the floors and the supporting walls and/or the sarking to the roof. This will have a direct bearing on whether or not the floors and/or roof can act as a diaphragm in distributing the seismic floor loads to the walls and whether the walls are tied together. Therefore, the state of the wall/diaphragm connection may determine the possible load paths for transferring seismic actions down to the foundations.

C9.4.2 Force-based approach

A force-based assessment approach is generally considered sufficient for most simple low-rise timber framed buildings.

Note:

A displacement-based assessment approach is considered essential when timber elements of significantly varying deformation capacity are being used in combination, or when timber elements are being used in conjunction with elements of other materials.

For a force-based assessment of a timber building it is generally acceptable to use a structural ductility factor where a ductile mechanism can be identified and the factor can be justified. If the mechanism(s) cannot be identified with certainty, the mechanism should be assumed to be brittle and the structural ductility factor limited to 1.25.

Note:

In older timber buildings and some modern buildings a capacity design may not have been undertaken, so brittle failure mechanisms may be present.

For timber framed buildings no more than two storeys high and with regular layouts, the bracing design provisions of NZS 3604:2011 can be adopted. This option should only be adopted if the distribution and spacing of bracing walls is generally in accordance with NZS 3604:2011. As bracing demands given in NZS 3604:2011 are derived from $\mu = 3.5$ and $S_p = 0.70$, these demands should be scaled accordingly for other values of μ and S_p.

For engineered buildings, multi-unit buildings and complex layouts, earthquake demands should be calculated in accordance with Section C3 with the amended provision that the 8% allowance applied at eaves/roof level should only be considered if a heavy roof is present or upper support of a heavy wall is required.

A structural performance factor of $S_p = 0.5$ is recommended for the assessment of timber buildings.

Note:

The structural performance factor takes account of a number of effects including structural redundancy, additional energy dissipation, the likely short duration of peak load, and higher material strengths and connection capacities. The value $S_p = 0.5$ is considered reasonable based on observed behaviour in earthquakes and on the destructive testing of timber framed buildings.

A force-based assessment will require determining the probable flexural, shear, axial and bracing capacities of the members, elements and connections using the information in Sections C9.5 and C9.6 and other references as necessary. In doing so, the potential failure mechanisms should be identified and considered when assigning the available ductility in the system.

It is emphasised that failures in timber connections can be brittle. Reference can be made to the European Yield Model (EYM) and Brittle Failure methods (EN 1995-1-1:2004 and Quenneville, 2009) or other similar methods to determine the failure mode for connections.

The global earthquake rating should be determined in accordance with Section C1 using the probable strength capacity of the global structure and the global base shear demand determined from Section C3.

C9.4.3 Displacement-based approach

A displacement-based assessment approach, taking account of nonlinear behaviour, is recommended, as limiting the capacity to first yield is likely to underestimate the capacity in many structures. It is also essential when attempting to combine together the contribution of systems of various nonlinear deformation capability and/or of different materials.

All assessment procedures outlined in Section C1 have completing a SLaMA as the first step. This requires the engineer to have a good understanding of the deformation capacity of the various systems to ensure displacement compatibility issues, particularly when the deformations are in the nonlinear range, are addressed.

Note:

To assist with using a displacement-based approach, BRANZ (BRANZ, 2013; BRANZ, 2015) has tested a variety of wall systems commonly used in New Zealand timber construction to better understand their lateral load-resisting behaviour. These tests have provided probable strength and deformation capacities and stiffnesses for a range of bracing systems. Examples are included in Appendix C9B.

The global earthquake score should be determined in accordance with the procedures outlined in Section C2.

C9.4.4 Other issues

C9.4.4.1 General

Engineers should consider any particular vulnerabilities or weaknesses within the structure and use their engineering judgement to consider the effects of these.

Some likely issues include: horizontal irregularity, vertical irregularity, heavy roofs and masonry veneer claddings, building condition, foundations and slope considerations, geotechnical hazards, and stairs. These are discussed below, together with suggestions about how to alter the recommended ductility and structural performance factors accordingly.

C9.4.4.2 Horizontal irregularity

Where horizontal irregularities exist, the engineer should consider the torsional behaviour of the building; in particular, the diaphragm performance with reference to Section C9.6.3.

C9.4.4.3 Vertical irregularity

Vertical strength/stiffness irregularities, such as soft storeys, can occur in two and three storey, multi-unit residential buildings and also in buildings with garages on the ground floor. If a soft-storey mechanism is likely, the engineer should pay particular attention to the connections between bracing elements and the magnitude and consequences of P-delta actions. As a result, a reduced structural ductility factor may need to be assumed (for a force-based procedure) to take account of the likelihood of a shake-down scenario.

C9.4.4.4 Heavy roofs and masonry veneer claddings

The presence of a heavy roof and/or masonry veneer cladding may increase the dynamic response of a building due to the additional high mass. The engineer should ensure that the mass is appropriately accounted for in the analysis and that valid load paths exist to transfer the inertial loads from source into the structure.

C9.4.4.5 Building condition

Alterations, post-construction, are common in timber buildings but are not always visible. The engineer should undertake an appropriate level of inspection to provide confidence that any alterations, such as the removal of walls, have been identified. This may involve intrusive works in roof spaces, wall cavities and sub-floors.

Building damage, deterioration, corrosion of structural elements and the effects of biological decay (such as borer infestation and wood rot) should be considered and the capacities downgraded accordingly.

C9.4.4.6 Foundations and slope considerations

If a building is constructed on concrete perimeter walls and the sub-floor height is 0.8 m or less, it is considered reasonable to assess the building considering the ground floor as the base of the building (i.e. as if it were constructed on a slab-on-grade). If the sub-floor height is greater than 0.8 m, the ground floor mass should be included, but the mass of foundation perimeter walls should not be included to calculate the equivalent static forces to be applied at the upper levels of the building.

Inadequate or poor connection of the floor framing to the piles is common. Buildings which have a sub-floor height of 600 mm or less are unlikely to present a life safety hazard if they come off their foundations (although significant damage may result). Therefore, the capacity of the sub-floor in these buildings should not govern the *%NBS* earthquake rating for the building.

The capacity of bolted connections in foundations should be calculated using the provisions of NZS 3603:1993, NZS 3604:2011, or EN 1995-1-1:2004 and Quenneville (2009).

When assessing the foundations for timber framed buildings, the value of S_p to be used for the calculation of the earthquake score for the foundation should be that appropriate for the foundation system.

If a building is constructed on a slope greater than 1 in 8, as shown in Figure C9.2, this may require a review of the sub-floor bracing design and construction. If there are substantial foundation cross-bracing elements present or if the building is supported by a reinforced concrete or reinforced concrete block retaining wall that is not showing any signs of movement, then typically no further sub-floor assessment is considered necessary. If these elements are not present then further assessment should be undertaken. It is also recommended that the centre of rigidity for the subfloor system is checked in relation to the location of the centre of mass to check for potential torsional effects.

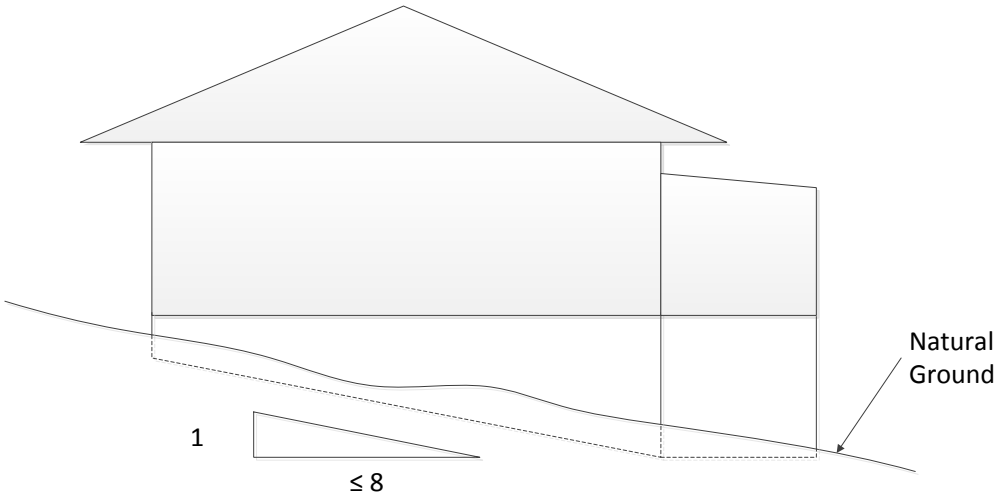

Figure C9.2: Definition of sloping ground

C9.4.4.7 Geotechnical hazards

As settlement of timber framed buildings caused by liquefaction is unlikely to lead to a significant life safety hazard, the effects of liquefaction should not govern the *%NBS* earthquake rating. However, other geotechnical hazards such as slope failure that could lead to a significant loss of foundation support may be critical.

Refer to Section C4 to assess potential geotechnical hazards that may be relevant to a particular site.

C9.4.4.8 Stairs

Internal stairs constructed of timber are unlikely to lead to a significant life safety hazard due to loss of egress but may contribute to an irregularity in structure stiffness.

External stairways, depending on their construction type, may be more vulnerable than internal stairs and should be checked.

C9.5 Material Properties

C9.5.1 General

Probable or expected values for the material properties should be used when assessing an existing timber building.

C9.5.2 Material strengths

An assessment of the probable strengths of existing materials may be made from the results of tests. If no test results are available, the engineer should either conduct suitable tests or assess conservative values of strength by comparison with the properties of similar timbers to those given in NZS 3603:1993, the Timber Design Guide (NZTIF, 2007) or other recognised sources such as technical literature from manufacturers for products such as glulam and LVL.

For timber structures built before 2000 the engineer may take probable material strength values as the characteristic strengths given in NZS 3603:1993 and reproduced in Table C9.1. (Note that the values in this table vary from the values given in Amendment 4 to this standard.)

For timber structures built from 2000 onwards the probable material strengths for Radiata pine and Douglas fir may be taken as the characteristic strengths given in Amendment 4 to NZS 3603:1993. (Note that the timber in almost all buildings constructed during this period is either Radiata pine or Douglas fir.)

Note:

The characteristic strength for tension parallel to the grain was reduced for a number of the species in 1996, after new testing. However, the reduction in stresses for Radiata pine and Douglas fir are the only ones included here because very few, if any, of the other species would have been used in building construction after that date.

C9.5.3 Modification factors

The modification factors (k_i) given in NZS 3603:1993 should be used where appropriate.

Table C9.1: Probable material strengths for visually graded timber (MPa) (characteristic strengths from NZS 3603:1993)

Species	Grade	Bending	Compression parallel	Tension parallel	Shear in beams	Compression perpendicular	Modulus of elasticity (GPa)
1. Moisture condition – Dry (m/c = 16% or less)							
Radiata pine	No. 1 framing	17.7	20.9	10.6[+]	3.8	8.9	8.0
Radiata pine	Engineering	24.5	24.2	12.2	3.8	8.9	10.0
Douglas fir	No. 1 framing	17.7	22.1	10.6[+]	3.0	8.9	8.0
Douglas fir	Engineering	22.4	25.4	11.2	3.0	8.9	9.9
Larch	No. 1 framing	22.7	27.1	13.6	3.5	8.9	9.6
Rimu	Building	19.8	20.1	11.8	3.8	10.9	9.5
Kahikatea	Building	14.5	19.5	8.6	3.0	5.9	6.8
Silver beech	Building	23.6	24.8	14.2	3.5	7.1	9.3
Red beech	Building	28.0	30.4	16.8	5.3	12.4	13.4
Hard beech	Building	29.5	26.6	17.7	5.0	14.2	13.6
2. Moisture condition – Wet (m/c = 25% or greater)							
Radiata pine	No. 1 framing	14.8	12.7	8.9[++]	2.4	5.3	6.5
Radiata pine	Engineering	20.1	15.0	10.0	2.4	5.3	8.1
Douglas fir	No. 1 framing	14.8	14.5	8.9[++]	2.4	4.7	6.5
Douglas fir	Engineering	20.1	17.1	10.0	2.4	4.7	8.0
Larch	No. 1 framing	15.0	17.4	8.9	2.7	5.6	7.7
Rimu	Building	15.0	14.5	8.9	2.7	6.8	8.3
Kahikatea	Building	13.9	14.2	8.3	2.4	4.4	6.0
Silver beech	Building	20.7	19.2	12.4	2.7	3.8	7.5
Red beech	Building	25.1	18.3	15.0	3.8	7.7	11.3
Hard beech	Building	28.3	24.2	17.1	4.4	10.6	12.1

Notes:

[+] Reduced to 8.8 MPa in 1996

[++] Reduced to 7.4 MPa in 1996

C9.6 Element Capacities

C9.6.1 General

The structural systems of timber structures are typically made up of multiple members/ elements which collectively define the strength and deformation capacity of the system as a whole. Behaviour of the elements (including shear walls, diaphragms, beams, columns, and braces) is dictated by physical properties such as: area; material grade; thickness, depth and slenderness ratios; lateral torsional buckling resistance; and connection details. Connected members include sheet products, planks, linear bracing, stiffeners, chords, sills, and struts.

The actual physical dimensions of individual timber members/elements that are being relied for load transfer should be measured rather than relying on nominal sizing; e.g. nominal 100 mm x 50 mm stud dimensions are generally less due to choice of cutting dimensions and later machining and/or drying shrinkage. Modifications to member capacities can be caused by notching, holes, and in some situations splits and cracks. The presence of decay or deformation should be noted and allowed for.

The connections are an important aspect of timber systems and often determine the deformation capacity as a whole. The type, size, spacing and condition of fixings such as nails will often be critical when determining the capacity and, although it will be difficult and impractical to confirm every fixing, checks should be made to confirm the general arrangements and condition.

The physical properties of the various components are needed in order to characterise building performance properly for a DSA. The starting point for establishing the properties should be the available construction documents. Accordingly, the engineer should carry out a preliminary review of these documents to identify primary vertical (gravity) and lateral load-carrying elements and systems, and their critical members and connections.

Next, conduct site inspections to verify conditions and make sure that building alterations have not changed the original design. In the absence of a complete set of building drawings, inspect the building thoroughly to identify these members, elements, and systems, as described in Section C1. If reliable record drawings do not exist, an as-built set of building plans may need to be produced. This may necessitate removal of linings to observe critical structural connections.

The intention of these guidelines is that the earthquake scores/ratings are based on probable capacities of elements. The probable strength capacity is assessed using probable material strengths as outlined in Section C9.5, and taking a strength reduction factor, ϕ, of 1. The probable deformation capacity of timber elements is likely to exceed other practical constraints.

C9.6.2 Timber shear walls

C9.6.2.1 General

The important failure modes for wood and light frame shear walls are sheathing failure, connection failure, tie-down failure, and excessive deflection. The probable strength capacity of wood and light timber frame shear walls should be taken as the nominal capacity of the shear wall assembly from NZS 3603:1993, i.e. using a strength reduction factor, ϕ,

equal to 1 but using probable material properties. For assemblies for which specific bracing test information is available (typically following test procedures such as the P21 test used in New Zealand to determine wind and earthquake ratings of bracing elements in timber framed structures built since 1980) the derived bracing ratings from those tests may be used.

Note:

The behaviour of wood and light frame shear walls is complex and influenced by many factors, the most significant of which is the wall sheathing. Wall linings can be divided into many categories (e.g. brittle, elastic, strong, weak, good at dissipating energy, and those poor at dissipating energy). In many existing timber framed buildings the wall linings were not expected to act as bracing (e.g. lath and plaster linings). Engineers should verify the presence of diagonal bracing behind such linings if possible. Other factors that can influence the behaviour of shear walls include the fixing pattern and the hold-down connections.

Some older shear walls are designed based on values from monotonic load tests and historically accepted values. The allowable shear per unit length used for design was assumed to be the same for long walls, narrow walls, walls with stiff tie-downs, and walls with flexible tie-downs. Only recently have shear wall assemblies – framing, covering, anchorage – been tested using cyclic loading procedures.

If different walls are lined with dissimilar materials along the same line of lateral-force resistance, the analysis should be based on the resistance of the individual elements maintaining displacement compatibility.

For overturning calculations on shear wall elements, stability should be evaluated in accordance with AS/NZS 1170.0:2002. Net tension due to overturning should be resisted by uplift connections at the ends of the element unless a rocking system can be justified.

It is important to consider the effects of openings in shear walls. This is because the presence of anything other than a small opening in a shear wall will cause a reduction in the stiffness and yield capacity due to a reduced length of wall available to resist lateral forces. Special analysis techniques are required to assess the effects of openings. The presence of chord members around the openings, with linings well fixed to them, will reduce the loss in overall stiffness and limit damage in the area of the openings. Equally, the effect on behaviour when these members are not present should be carefully considered.

C9.6.2.2 Types of timber shear walls

Transverse sheathing

Transverse sheathing or board lining consists of boards up to 25 mm thick and usually 100-200 mm wide, nailed in a single layer at right angles to the studs.

These walls tend to be overlaid with scrim material and wallpaper in residential construction. The sheathing resists the shear force caused by lateral loading. The perimeter members carry axial loading from the gravity loads and the lateral loading, whereas the intermediate studs are not loaded axially by the lateral loading but nevertheless provide support to the sheathing and enable the interconnection of sheathing elements.

The moment resistance provided by the nail couples at each stud crossing is the lateral load-resisting mechanism. The resisting mechanism of the couplet is less effective with narrower boards but there are more couplets for the same wall height, meaning the wall result is similar for all board widths. Nail slip is the dominant cause of lateral deflection in shear walls of common dimensions. Flexural strains in the chord members and shear distortion in the sheathing itself may also contribute to the total deflection capacity.

Single diagonal sheathing

The shear force applied to the shear wall is carried by tension or compression in the 45º diagonal sheathing and is transferred to the perimeter members by the nails.

This form of shear wall is likely to be found on external walls of warehouses, large school buildings and hall type structures between the column supports of portal frames or braced trusses.

Double diagonal sheathing

Two layers of sheathing on the same side of the framing significantly improves the shear characteristic of a shear wall. When double diagonal sheathing is used with one layer diagonally opposed to the other, one layer acts in tension and the other in compression, and the shear is assumed to be shared. Thus, the two layers act as a shear membrane.

Panel sheathing

This consists of wood structural panels (such as plywood or oriented strand board), gypsum plasterboard, or fibre cement board that is placed on framing members and nailed in place. Different grades and thicknesses of panels may have been used on one or both sides of the wall depending on requirements for gravity load support, shear capacity, and fire protection. Edges at the ends of the structural panels are usually supported by the framing members. Edges at the sides of the panels could have been blocked or unblocked.

Fixing patterns and fixing size can vary greatly. Spacing is commonly in the range of 75-150 mm on centre at the supported and blocked edges of the panels, and 250-300 mm on centre at the panel interior. In older construction, the fixings were usually nails. In more modern construction using gypsum plasterboard and some fibre cement board products, the fixings may be screws.

C9.6.2.3 Strength and stiffness of timber shear walls

An assessment of the probable strength of timber shear walls should be based on an assessment of the probable strengths of the materials making up the particular shear wall. Depending on the wall type, the formulae given in Appendix C9C can be used to determine the shear wall probable strength. In the absence of test results, the probable strength values contained in Table C9.2 may be used in lieu of more detailed calculations.

The deflection at the notional yield can be calculated using the formulae in Appendix C9D. For many shear walls the major component affecting the stiffness is the nail slip. It is acceptable to base the stiffness initially on the nail slip component of deformation unless the nail spacing is sufficiently close to induce large forces in the cladding.

Table C9.2: Probable strength values for existing timber framed wall bracing systems (based on 2.4 m wall height)

Bracing type	Probable strength values
150 x 25 mm let-in brace at 45º	2.0 kN
150 x 25 mm let-in brace at 45° and sheet material* one face	2.5 kN
150 x 25 mm let-in brace at 45° and sheet material* both faces	3.7 kN
90 x 45 mm fitted brace both ways at 45º	2.0 kN
90 x 45 mm fitted brace both ways at 45º and sheet material* one face	2.5 kN
90 x 45 mm fitted brace both ways at 45º and sheet material* both faces	3.7 kN
90 x 45 mm dog leg brace (600 mm wall length)	0.75 kN
Timber framed stud walls with wood or metal lath and plaster	1.5 kN/m each side
Timber framed stud walls with diagonal braces and wood or metal lath and plaster	2.8 kN/m
Gypsum plasterboard one side, and fixed at 300 mm centres (no diagonal timber braces included)	1.0 kN/m
Gypsum plasterboard one side, and fixed at 150 mm centres (no diagonal timber braces included)	2.5 kN/m
Gypsum plasterboard two sides, and fixed at 300 mm centres (no diagonal timber braces included)	2.0 kN/m
Gypsum plasterboard two sides, and fixed at 150 mm centres (no diagonal timber braces included)	3.0 kN/m
Match lining on one or both faces (no diagonal timber braces included)	1.25 kN/m
3.2 mm tempered hardboard fixed with clouts at 200 mm centres	3.0 kN/m
Horizontal board sheathing	1.0 kN/m
Horizontally oriented corrugated steel sheets	2.0 kN/m
Vertically oriented corrugated steel sheets	1.50 kN/m
140 x 20 mm bevel back weatherboard	0.30 kN/m

Note:

*Sheet material is defined as having a density of not less than 450 kg/m^3. It may be a wood-based material not less than 4.5 mm thick or a gypsum-based material not less than 8 mm thick, both fixed to framing members not closer than 10 mm from sheet edges.

When determining the probable wall bracing capacity using the values in Table C9.2 the capacity of each bracing element should be calculated by multiplying by the length of the bracing element and adjusting for height in accordance with the following equation:

$$\frac{2.4}{\text{element height in metres}}$$

This equation is applicable for framing with sheet bracing products attached (and therefore it is not applicable for bracing systems such as horizontal sarking). Elements less than 2.4 m in height should be rated as if they are 2.4 m high. Walls of varying height should have their bracing capacity adjusted using the average height.

Where bracing units are used in place of force units (e.g. kNs), a conversion of 1 kN = 20 bracing units should be used.

Consideration should also be given to the aspect ratio of the wall element; i.e. its overall height to length ratio. If published indicative bracing ratings are being relied on, it should be ensured that the length of the element is applicable for the published value. This is because failure mechanisms can change with aspect ratio, resulting in altered ratings per unit length. For narrow elements (height: length ratio > 2) consideration should be given to reducing the published capacity. It is suggested that a linear reduction of strength is applied from 1 times the published data for ratios of 2:1 to zero for ratios equal and greater than 3.5:1.

Note:

The bracing units apply to the capacity of an individual wall panel. Any weak links or issues with the stiffness of the diaphragms which may limit or determine the extent to which individual panels are able to contribute to the overall building capacity should be identified.

C9.6.3 Roof and floor diaphragms

C9.6.3.1 General

The probable strength of timber diaphragms should be taken as the probable capacity of the diaphragm assembly determined from a rational assessment of the individual elements. The effects of openings in timber diaphragms also need to be considered. The presence, or lack, of chords and collectors will affect the load carrying capacity of the diaphragm. Connections between diaphragms and other components including shear walls, drag struts, collectors, cross ties, and out-of-plane anchors also need to be considered.

The behavior of horizontal wood diaphragms is influenced by the type of sheathing, size and spacing of fasteners, existence of perimeter chord or flange members, and the ratio of span length to width of the diaphragm. The presence of anything other than small openings in diaphragms will cause a reduction in the stiffness and capacity of the diaphragm due to a reduced length of diaphragm available to resist lateral forces. Special analysis techniques and detailing are required at the openings.

The presence or addition of trimming members around the openings will reduce the loss in stiffness of the diaphragm and limit damage in the area of the openings. The presence of chords at the perimeter of a diaphragm will significantly reduce diaphragm deflections due to bending, and will increase the stiffness of the diaphragm over that of an unchorded diaphragm. However, the increase in stiffness due to chords in a single straight sheathed diaphragm is minimal due to the flexible nature of these diaphragms.

Note:

The actions on the individual elements of a diaphragm will depend on the relative stiffness of the diaphragm compared with the lateral stiffness of the connected vertical elements. The relative stiffness will change if the vertical elements are loaded into the nonlinear range, at which point a timber diaphragm could be considered as rigid. The analysis of diaphragms is discussed further in Section C2 and for URM buildings in Section C8.

C9.6.3.2 Types of timber diaphragms

Transverse sheathing

This type of diaphragm consists of 25 mm thick boards, usually 100-200 mm wide, nailed in a single layer at right angles to the cross members such as joists in a floor or rafters in a roof. In a floor, the boards are usually tongue and groove in order to improve the interconnection between the boards and thus improve the vertical load sharing ability of the system. In a roof, the boards are often square edged with no interaction between boards.

Note that sometimes the boards may be spaced with gaps between the boards as wide as the width of the boards. In such cases the diaphragm action will be less because of the smaller number of nail couplets per unit area.

The sheathing serves the dual purpose of supporting gravity loads and resisting shear forces in the diaphragm. Most often, the sheathing will have been nailed with 60 mm or 75 mm long, 3.15 mm diameter jolt head nails, with two or more nails per sheathing board at each support. Shear forces perpendicular to the direction of the sheathing are resisted by the nail couple and some major axis bending of the sheathing boards. Shear forces parallel to the direction of the sheathing are transferred through the nails in the supporting joists or framing members below the sheathing joints, which then work in weak axis bending.

Single diagonal sheathing

This consists of sheathing boards of 25 mm thickness and 100-200 mm wide, nailed in a single layer at a 45º angle to the cross members. This type of sheathing was generally only used in roof planes. It was common for the diagonal boards in some areas of the roof to be running at right angles to other areas in order to provide compression struts for loading in opposing directions.

This sheathing supports gravity loads and resists shear forces in the diaphragm. Commonly, the sheathing was nailed with 60 mm or 75 mm long, 3.15 mm diameter jolt head nails, with two or more nails per board at each support. The shear capacity of the diaphragm is dependent on the size, number and spacing of the nails at each sheathing board. This type of diaphragm has greater strength and stiffness than transverse sheathing.

Panel sheathing

Panel sheathing consists of wood, gypsum plasterboard or fibre cement structural panels (such as plywood or particle board) placed on framing members and nailed in place. Different grades and thicknesses of structural panels are commonly used, depending on requirements for gravity load support and shear capacity. Edges at the ends of the structural panels are usually supported by the framing members. Edges at the sides of the panels may be blocked or unblocked.

Fixing patterns and fixing size can vary greatly. Spacing of fixings is commonly in the range of 75 mm to 150 mm at the supported and blocked edges of the panels, and 250 mm to 300 mm at the panel interior. In older construction, the fixings are generally nails. In more modern construction using gypsum plasterboard and some fibre cement board products, the fixings may be screws.

C9.6.3.3 Strength and stiffness of timber diaphragms

The assessment of probable strength for timber diaphragms should be based on an assessment of the materials making up the particular diaphragm and their individual probable strengths. Depending on the type of timber diaphragm, the formulae given in Appendix C9E can be used to determine its probable strength. In the absence of test results, the maximum values contained in Table C9.3 may be used in lieu of more detailed calculations.

Indicative values for diaphragm shear stiffness are also provided in Table C9.3. Diaphragm deflections can also be calculated using the formulae in Appendix C9F. For many diaphragms, the major aspect affecting the stiffness is the nail slip. It is acceptable to start by basing the stiffness on the nail slip component of deformation.

Softboard linings are considered to provide insufficient diaphragm action and any contribution to strength or stiffness should be ignored.

Table C9.3: Probable stiffness and strength values for existing horizontal diaphragms

Diaphragm type		Probable shear stiffness	Probable strength values
A1	Roofs with straight sheathing (sarking) and roofing applied directly to the sheathing – loading parallel to rafters	250 kN/m	4.0 kN/m
A2	Roofs with straight sheathing (sarking) and roofing applied directly to the sheathing – loading perpendicular to rafters	180 kN/m	3.0 kN/m
B	Roofs with diagonal sheathing and roofing applied directly to the sheathing	700 kN/m	10.5 kN/m
C1	Floors with straight tongue and groove sheathing – loading parallel to joists	285 kN/m*	4 kN/m
C2	Floors with straight tongue and groove sheathing – loading perpendicular to joists	215 kN/m*	3 kN/m
D	Floors and roofs with sheathing and existing gypsum plasterboard or fibre cement sheets re-nailed to the joists or rafters	4000 kN/m	Add 1.5 kN/m to the values for Items A1, A2, C1 and C2
E	Gypsum plasterboard ceilings fixed at 150 mm centres to the underside of roof framing (edges blocked) – loading parallel to rafters	7000 kN/m	6 kN/m

Note:
* Fair condition assumed

C9.6.4 Timber portal frames

Because there is a wide range of materials, connections and spans used for timber portal frames, it is not practical to provide a comprehensive table of probable capacities for these. Instead, establish the probable strength of a timber portal frame by using either generic material properties from Section C9.5 for solid timber sections or proprietary information from manufacturers, if known. Generic glue laminated timber properties may be taken from AS/NZS 1328.2:1998. The probable strength of spliced joints may be estimated using NZS 3603:1993 and assuming the probable strengths are the nominal strength ($\phi = 1$), having regard to the connectors used and the splicing products.

C9.6.5 Timber trusses

Trusses in older buildings may use nailed plywood gussets at joints (smaller spans) or multiple member chord and web members with bolted connections (larger spans). The bolted connections may also be strengthened by the addition of split ring or shear plate connectors, but this will be difficult to establish. If the joints include connectors, proprietary strength information may be used if this is available.

C9.6.6 Connections

The method of connecting the various elements of the structural system is critical to its performance. The type and character of the connections should be determined by a review of the plans and a field verification of the conditions. The connection between a timber diaphragm and the supporting structure is of prime importance in determining whether or not the two parts of the structure can act together. Except for light timber framed buildings, the form of connections is such that the flexural strength at first yield and their post-elastic stiffness can be determined by rational assessment.

In general, the determination of the capacity of connections should be undertaken in accordance with the provisions of NZS 3603:1993 assuming that the probable strength is the nominal strength ($\phi = 1$). If relevant, more detailed analysis of connection failure mechanisms can be determined using European Yield Methods. Refer to EN 1995-1-1:2004 and Quenneville (2009).

In URM buildings the connections of timber elements to the masonry are often nominal and generally should not be relied upon for engineering purposes. Further, the performance of such connections is influenced by the level of deterioration that may have taken place in both the masonry and the timber members, and by any corrosion of the bolts themselves. When assessing such connections, also refer to Section C8 Unreinforced Masonry Buildings.

Note:

Section C8 and Beattie (1999) contain further information about the likely performance of timber diaphragm to masonry wall connections.

C9.7 Improving the Seismic Performance of Timber Buildings

The process of conducting a DSA may identify structural weaknesses in the building that could be mitigated to improve its seismic performance.

For timber framed buildings, typical methods for improving seismic performance include:

- removing heavy elements such as concrete tile roofs, masonry veneer or chimneys
- replacing lining materials for existing wall bracing and diaphragm elements
- re-nailing or re-screwing existing structural wall linings
- adding supplementary bracing in the form of structural frames
- improving hold-down connections
- improving foundations; e.g. by adding additional cross bracing to existing foundation piles or anchor piles, and by improving the connections between the foundations and the superstructure.

For engineered timber framed buildings, methods of improving seismic performance include:

- enhancing connections at the joints in portal frame systems; e.g. by adding additional plates to the knee and apex joints
- fixing additional material to timber members to increase capacity
- enhancing foundation connections.

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Appendix C9A: Full Scale Tests on Timber Framed Buildings

C9A.1 Tests on Classroom Blocks

In 2013, the Ministry of Education commissioned testing and invasive investigations of standard classroom blocks of timber framed construction to gather further evidence of the performance of these buildings. This included full scale destructive tests of two types of classroom block, an "Avalon" and a "Dominion" block. The findings from these investigations are summarised below and described in more detail by Brunsdon et al. (2014).

The first test involved two classrooms that formed part of a four-classroom Avalon block at South End School, Carterton, Wairarapa. Avalon timber framed blocks were commonly constructed in the late 1950s and early 1960s. They feature a front wall that is essentially fully glazed, with no recognisable structural bracing panels. The classroom ceiling features a high-level vertical glazed (or "clerestory") section; again, with no identifiable form of bracing.

The destructive test confirmed the general engineering expectation that timber framed buildings with older glazed facades have a strength and resilience significantly in excess of their calculated capacity. Test results indicated that failure of the glazing in the longitudinal direction occurred at more than five times the nominal calculated probable capacity of the building. A margin of three to four times was achieved in the associated test of a transverse wall.

The second test was undertaken on a Dominion block at Hammersley Park School, Christchurch. Dominion blocks were built in the 1950s and are timber framed buildings with brick veneer cladding to the walls, weather boards at gable ends and light weight corrugated steel cladding to the roof. The block selected for testing was constructed as a multi-classroom block.

Two adjacent classrooms at the western end were tested in the longitudinal direction. A single classroom at the eastern end was tested in the transverse direction. This destructive test also confirmed the general engineering expectation that timber framed buildings with older glazed facades have a strength and deformation capacity significantly in excess of their calculated capacity. Test results indicate that failure in the longitudinal direction occurred at more than eight times the nominal calculated probable lateral strength capacity of the building. A margin of two and a half to three times was achieved in the associated test of a transverse wall (refer to Figure C9A.1).

Figure C9A.1: Transverse test of the Dominion block showing high levels of drift

C9A.2 Tests on Housing Unit

In 2013, Housing New Zealand commissioned BRANZ Ltd to undertake a full scale test of a two-storey timber framed housing unit in Upper Hutt. This housing unit was constructed in the 1950s and consisted of four units separated by reinforced blockwork party walls. It had a significant number of wall openings at the ground floor which meant that a very short length of plasterboard lined walls was available to resist lateral load. The findings from these investigations are noted below and described in more detail by Connor-Woodley (2015).

The housing unit was tested in both the longitudinal and transverse directions. Similarly to the Ministry of Education tests, the results indicated significant capacity: in this case, a strength of over five times the calculated strength and a significant deformation capacity without creating a significant life safety hazard (refer to Figures C9A.2 and C9A.3).

Figure C9A.2: Longitudinal test of Housing New Zealand unit showing significant racking of ground floor walls

Figure C9A.3: Typical internal damage to plasterboard lined walls during test of Housing New Zealand unit

Appendix C9B: Strength vs Deformation Capacity Relationships for Generic Bracing Elements

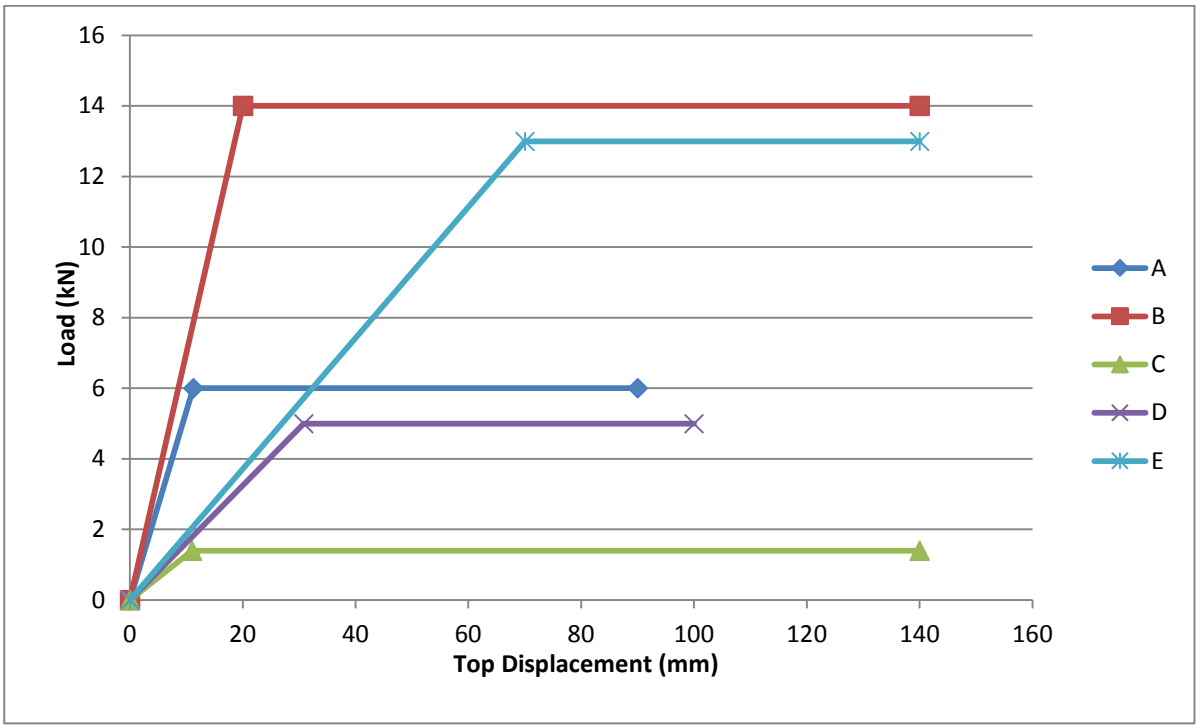

Figure C9B.1: Capacity relationships for timber framed walls with typical sheathing materials, for heights as noted below

A: 5.5 m high panels with 12 mm particleboard up to a height of 3.7 m and 4.5 mm hardboard between 3.7 and 5.5 m

12 mm particleboard fixed with 40 mm x 1.5 mm jolt head nails at 300 mm maximum centres

Hardboard fixed with 25 mm x 1.6 mm jolt head nails at 300 mm maximum centres.

B: 5.5 m high panels with 12 mm particleboard up to a height of 3.7 m and 4.5 mm hardboard between 3.7 and 5.5 m

12 mm particleboard re-nailed with 50 mm x 2.5 mm flat head galvanised nails at 300 mm maximum centres

Hardboard fixed with 30 mm x 2.5 mm flat head nails at 300 mm maximum centres.

C: Panels up to a height of 5.5 m of 200 x 25 rusticated weatherboards (nett coverage per board 155 mm)

Weatherboards nailed with 60 mm x 2.8 mm jolt head galvanised nails, minimum one per board/stud crossing.

D: Panels up to a height of 3.6 m of full height 12 mm particleboard for the interior face and full height rusticated weatherboards for the exterior face

12 mm particleboard nailed with 40 mm x 1.6 mm jolt head nails at 300 mm maximum centres

Rusticated weatherboards nailed with 60 mm x 2.8 mm jolt head galvanised nails, minimum one per board/stud crossing.

E: Panels up to a height of 3.6 m of full height 12 mm particleboard for the interior face and full height rusticated weatherboards for the exterior face

12 mm particleboard re-nailed with 50 mm x 2.5 mm flat head nails at 300 mm maximum centres

Rusticated weatherboards nailed with 60 mm x 2.8 mm jolt head galvanised nails, minimum one per board/stud crossing.

Note:

These relationships have been derived from BRANZ tests (Study Report SR 305 (2013)) for 5.5 m high panels comprising 140 x 45 mm studs at 600 mm centres and nogs at 1200 mm centres between steel portal legs which were 4.4 m apart. Any resistance provided by the steel portals bending about their weak axis is not included and there was no contribution from any steel link beams between the portals.

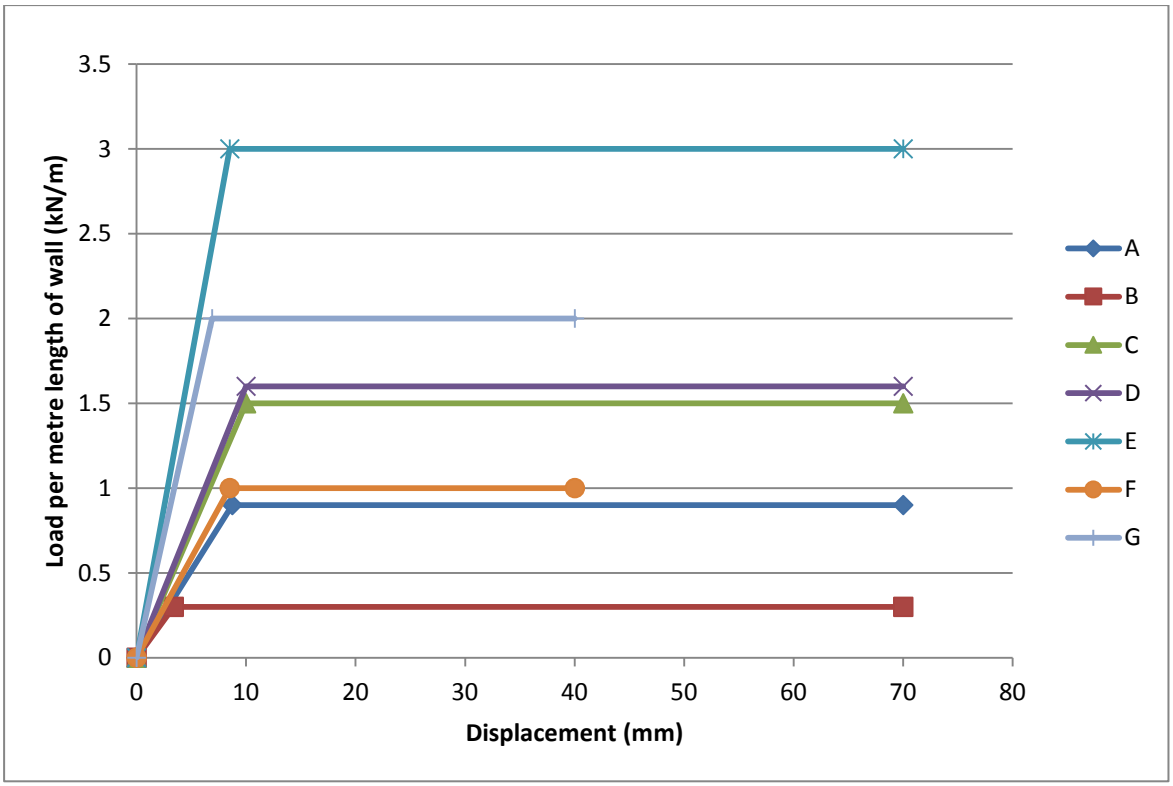

Figure C9B.2: Capacity relationships for 2.4 m high timber framed walls with sheathing materials as noted below

A: Horizontal 200 mm board sheathing

1.8 m wide panel

Three studs at 600 mm centres

200 mm wide horizontal boards on one face of frame (minimal gap between).

B: Bevel back weatherboard sheathing

3 m wide panel

Five studs at 600 mm centres

Bevel back weatherboards on one side of frame, fixed with one nail at 600 m centres.

C: Vertically oriented corrugated iron sheathing

2.4 m wide panel

Studs at 600 mm centres

Nogs at 800 mm centres

Vertical corrugated iron fixed through every third peak generally to plates and nogs.

D: Horizontally oriented corrugated iron sheathing

3.0 m wide panel

Studs at 600 mm centres

Horizontal corrugated iron fixed through every third peak generally to studs.

E: 3.2 mm hardboard sheet with clouts

1.2 m long panel

Studs at 600 mm centres

Hardboard on one side of frame, fixed with clouts at 200 mm centres.

F: Single side 10 mm plasterboard

1.2 m wide panel

Studs at 600 mm centres

Plasterboard on one side of frame, fixed with 30 mm long FH galvanized nails at 300 mm centres.

G: Double side 10 mm plasterboard

1.2 m wide panel

Studs at 600 mm centres

Plasterboard on both sides of frame, fixed with 30 mm long FH galvanised nails at 300 mm centres.

Note:

Results derived from BRANZ tests (Study Report SR 305 (2013)).

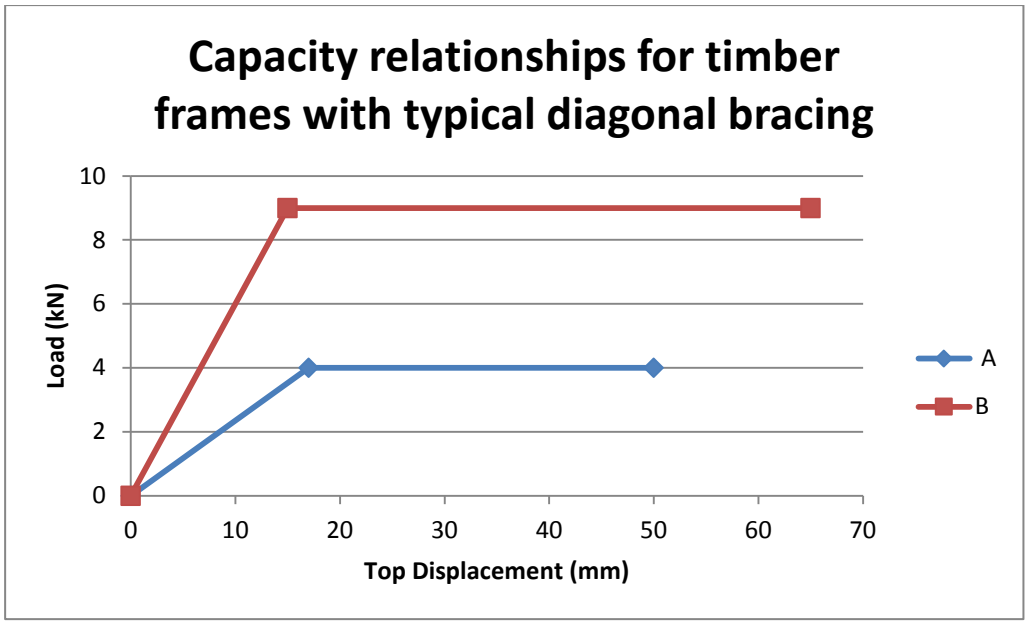

Figure C9B.3: Capacity relationships for 2.4 m high timber frames with diagonal bracing as noted below

A: Opposing 90 mm x 45 mm diagonal braces at 45º cut between studs
Studs at 600 mm centres.

B: Two consecutive opposing 150 mm x 25 mm let-in diagonal braces at 45º
Studs at 600 mm centres.

Note:

Relationships derived from BRANZ tests (Study Report SR 305 (2013)).

Appendix C9C: Timber Shear Wall Strength

C9C.1 Transverse Sheathing

The probable strength of transversely sheathed shear walls depends on the resisting moment furnished by nail couples at each stud crossing. If the nail couple $M = F_{\mathrm{n}}.s$, then the probable shear force in Newtons per metre length of wall, v_{prob}, that can be resisted is:

$$v_{\mathrm{prob}} = \frac{F_{\mathrm{n}}}{l}.\frac{s}{b} \qquad \ldots\text{C9C.1}$$

and the probable shear strength, V_{prob}, of the wall in Newtons is:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{n}} sB}{bl} \qquad \ldots\text{C9C.2}$$

where:

F_{n} = probable nail strength (N)
s = nail spacing (mm)
l = spacing between studs (m)
b = width of sheathing board (mm)
B = length of the wall (m).

Friction between the board edges can be assumed to increase the probable strength of the wall by the addition of a term Bv', where

v' = 74 N/m for 25 mm sawn boards
= 148 N/m for 50 mm sawn boards
= 222 N/m for tongue and groove boards.

The probable in-plane strength of the sheathing in Newtons is given by the expression:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{b}} zB}{bl} \qquad \ldots\text{C9C.3}$$

where:

F_{b} = the characteristic bending stress of the board (N/mm^2)
z = section modulus of the sheathing board = $\frac{b^2 t}{6}$, where t is the thickness of the sheathing board (mm).

C9C.2 Single Diagonal Sheathing

The probable horizontal shear in Newtons, $\left(V_{\mathrm{prob}}\right)_{\mathrm{i}}$, carried by each board is:

$$\left(V_{\mathrm{prob}}\right)_{\mathrm{i}} = \frac{1}{\sqrt{2}} N F_{\mathrm{n}} \qquad \ldots\text{C9C.4}$$

giving a total probable strength in kilonewtons of:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{n}} NB}{2b} \qquad \ldots\text{C9C.5}$$

Since the axial force in the sheathing is the same on both sides of any intermediate stiffener, no load is transferred into the stiffeners from the sheathing. However, the perimeter members are subjected to both axial loads and bending and must be assessed for the combined stresses (see NZS 3603:1993). The bending in the plate members is caused by a universally distributed load, w in N/mm, of:

$$w = \frac{NF_{\mathrm{n}}}{b} \qquad \ldots\text{C9C.6}$$

The probable in-plane strength of the sheathing boards, in Newtons, is given by:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{c}} b t}{2} \qquad \ldots\text{C9C.7}$$

where:

N = the number of nails fixing the board to the plate
t = thickness of the sheathing board (mm)
F_{c} = characteristic stress in the sheathing board in compression parallel to the grain (N/mm^2).

Other symbols are as defined in Section C9C.1.

C9C.3 Double Diagonal Sheathing

Based on the strengths of the nail pairs at the end of each sheathing board, and assuming that all nails extend into the top and bottom plates, the probable strength of the shear wall is given by:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{n}} N B}{b} \qquad \ldots\text{C9C.8}$$

The probable in-plane strength in kilonewtons of the sheathing boards over the wall length is given by the expression:

$$V_{\mathrm{prob}} = F_{\mathrm{c}} B t \qquad \ldots\text{C9C.9}$$

The probable capacity of the chords in Newtons is given by:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{c}} B A}{H} \qquad \ldots\text{C9C.10}$$

while the probable capacity of the plates in Newtons is given by:

$$V_{\mathrm{prob}} = F_{\mathrm{c}} A_{\mathrm{p}} \qquad \ldots\text{C9C.11}$$

where:

A = cross sectional area of the chord (mm^2)
A_{p} = cross sectional area of the plate (mm^2)
H = the height of the wall (m).

Other symbols are as defined in Sections C9C.1 and C9C.2.

C9C.4 Panel Sheathing

The probable strength values in Table C9.1 should be used in assessing the strength of these elements, unless specific tests are carried out.

Appendix C9D: Timber Shear Wall Deflections

The horizontal inter-storey deflection in one storey of a shear wall, Δ_w, can be calculated from:

$$\Delta_w = \Delta_4 + \Delta_5 + \Delta_6 + \Delta_7 \quad \text{…C9D.1}$$

where:

Δ_4 = deflection due to support connection relaxation (mm)
Δ_5 = wall shear deflection (mm)
Δ_6 = deflection due to nail slip (mm)
Δ_7 = deflection due to flexure as a cantilever (mm) (may be ignored for single storey shear walls).

For transverse sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \quad \text{…C9D.2}$$

$$\Delta_5 = 0 \quad \text{…C9D.3}$$

$$\Delta_6 = 2\frac{H}{s}e_n \quad \text{…C9D.4}$$

$$\Delta_7 = H\theta \quad \text{…C9D.5}$$

For single diagonal sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \quad \text{…C9D.6}$$

$$\Delta_5 = \frac{VH}{GBt} \quad \text{…C9D.7}$$

$$\Delta_6 = 2\sqrt{2}e_n \quad \text{for the case where H ≤ B, OR} \quad \text{…C9D.8}$$

$$= 2\sqrt{2}\frac{H}{B}e_n \quad \text{for the case where H > B}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \quad \text{…C9D.9}$$

For double diagonal sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \quad \text{…C9D.10}$$

$$\Delta_5 = \frac{VH}{GBt} \quad \text{…C9D.11}$$

$$\Delta_6 = \sqrt{2}e_n \quad \text{for the case where H ≤ B, OR} \quad \text{….C9D.12}$$

$$= \sqrt{2}\frac{H}{B}e_n \quad \text{for the case where H > B} \quad \text{…C9D.13}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \quad \text{…C9D.14}$$

For panel sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \quad \text{…C9D.15}$$

$$\Delta_5 = \frac{VH}{GBt} \quad \text{…C9D.16}$$

$$\Delta_6 = 2(1 + a)me_n \quad \text{…C9D.17}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \quad \text{…C9D.18}$$

where:

a = aspect ratio of each sheathing panel:
- = 0 when relative movement along sheet edges is prevented
- = 1 when transverse sheathing panels are used
- = 2 when 2.4 x 1.2 m panels are orientated with the 2.4 m length parallel with the diaphragm chords (i.e. vertical) (= 0.5 alternative orientation)

A = sectional area of one chord (i.e. end stud) (mm^2)

B = distance between shear wall chord members (mm)
= length of the wall

e_n = nail slip resulting from the shear force V (mm)

E = elastic modulus of the chord members (MPa)

G = shear modulus of the sheathing (MPa)

H = height of the storey under consideration (mm)

m = number of sheathing panels along the length of the edge chord

s = spacing of the nail couples in a board (mm)

t = thickness of the sheathing (mm)

V = shear force in storey under consideration (N)

θ = flexural rotation at base of storey under consideration (radians)

δ_c = vertical downward movement (mm) at the base of the compression end of the wall (this may be due to compression perpendicular to the grain deformation in the bottom plate)

δ_t = vertical upward movement (mm) at the base of the tension end of the wall (this may be due to deformations in a nailed fastener and the members to which it is anchored).

Appendix C9E: Timber Diaphragm Strength

C9E.1 Transverse Sheathing

The probable strength of transversely sheathed diaphragms (i.e. diaphragms where the sheathing runs perpendicular to the diaphragm span) depends on the resisting moment furnished by nail couples at each joist/rafter crossing. If the nail couple $M = F_{\mathrm{n}}.s$, then the shear force in Newtons per metre length at the support, v, that can be resisted is:

$$v_{\mathrm{prob}} = \frac{F_{\mathrm{n}}}{l}.\frac{s}{b} \qquad \ldots\text{C9E.1}$$

and the total probable strength capacity of the diaphragm in Newtons based on nail capacity is:

$$F_{\mathrm{prob}} = \frac{2F_{\mathrm{n}}sB}{bl} \qquad \ldots\text{C9E.2}$$

Friction between the board edges can increase the probable capacity of the diaphragm by the addition of a term, $2Bv$', where:

v' = 74 N/m for 25 mm sawn boards
= 148 N/m for 50 mm sawn boards
= 222 N/m for tongue and groove boards.

The probable in-plane strength capacity of the diaphragm based on the strength of the sheathing is given by the expression:

$$F_{\mathrm{prob}} = \frac{2F_{\mathrm{b}}zB}{bl} \qquad \ldots\text{C9E.3}$$

where:

F_{n} = nominal nail strength (N)
F_{b} = the probable bending stress of the sheathing board, N/mm^2
s = nail spacing (mm)
l = spacing between joists (m)
b = width of sheathing board (mm)
B = depth of diaphragm (m)
z = section modulus of the sheathing board = $\frac{b^2t}{6}$, where t is the thickness of the board (mm^3).

C9E.2 Single Diagonal Sheathing

The probable strength of the diaphragm depends on the probable shear capacity of the total number of nails into the edge member across the width of the diaphragm. The probable shear capacity of the diaphragm in kilonewtons is:

$$V_{\mathrm{prob}} = \frac{F_{\mathrm{n}}NB}{b} \qquad \ldots\text{C9E.4}$$

where:

N = total number of nails into the edge member

The probable in-plane shear strength of the diaphragm based on the strength of the sheathing in kilonewtons is given by the expression:

$$V_{\text{prob}} = F_{\text{c}} B t \qquad \ldots\text{C9E.5}$$

where:

F_{c} = characteristic stress in the sheathing board in compression parallel to the grain (N/mm^2)

t = thickness of the sheathing board (mm).

Other symbols are as defined in Section C9E.1.

The probable strength of the chord members needs to be assessed for combined bending and axial stresses (refer to NZS 3603:1993).

C9E.3 Panel Sheathing

The probable strength values in Table C9.1 should be used in assessing the strength of these elements, unless specific tests are carried out.

Appendix C9F: Timber Diaphragm Deflections

The mid span deflection of a horizontal diaphragm, Δ_h, can be calculated from:

$$\Delta_h = \Delta_1 + \Delta_2 + \Delta_3 \quad \text{…C9F.1}$$

where:

Δ_1 = diaphragm flexural deflection considering chords acting as a moment resisting couple (mm)

Δ_2 = diaphragm shear deflection resulting from beam action of the diaphragm (mm)

Δ_3 = deflection due to nail slip for horizontal diaphragm (mm).

For transverse sheathing:

$$\Delta_1 = 0 \quad \text{…C9F.2}$$

$$\Delta_2 = 0 \quad \text{…C9F.3}$$

$$\Delta_3 = \frac{Le_n}{2s} \quad \text{…C9F.4}$$

For single diagonal sheathing:

$$\Delta_1 = \frac{5WL^3}{192EAB^2} \quad \text{…C9F.5}$$

$$\Delta_2 = \frac{WL}{4EBt} \quad \text{…C9F.6}$$

$$\Delta_3 = \frac{(1+a)me_n}{2} \quad \text{…C9F.7}$$

For panel sheathing:

$$\Delta_1 = \frac{5WL^3}{192EAB^2} \quad \text{…C9F.8}$$

$$\Delta_2 = \frac{WL}{8GBt} \quad \text{…C9F.9}$$

$$\Delta_3 = \frac{(1+a)me_n}{2} \quad \text{…C9F.10}$$

where:

a = aspect ratio of each sheathing panel:
- = 0 when relative movement along sheet edges is prevented
- = 1 when transverse sheathing panels are used
- = 2 when 2.4 m x 1.2 m panels are orientated with the 2.4 m length parallel with the diaphragm chords (= 0.5 alternative orientation)

A = sectional area of one chord (mm^2)

B = distance between diaphragm chord members (mm)

e_n = nail slip resulting from the shear force V (mm)

E = elastic modulus of the chord members (MPa)
G = shear modulus of the sheathing (MPa)
L = span of a horizontal diaphragm (mm)
m = number of sheathing panels or boards along the length of the edge chord
s = nail couplet spacing (mm)
t = thickness of the sheathing (mm)
W = lateral load applied to a horizontal diaphragm (N).

Note:

The assumption made in the equations above is that the diaphragm remains essentially elastic. The deformation estimate is therefore the nominal yield displacement.

PART C

Secondary Structural and Non-Structural Elements C10

Document Status

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This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

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1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

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Please go to www.EQ-Assess.org.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.EQ-Assess.org.nz

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C10. Secondary Structural and Non-structural Elements

C10.1 General

C10.1.1 Scope and outline of this section

This section provides guidance for engineers carrying out Detailed Seismic Assessments (DSA) of building elements that are not part of the primary lateral or gravity structure within a building or section of a building. Such elements are referred to as secondary structural and non-structural (SSNS) elements and systems. These guidelines aim to provide a consistent approach to assessing these elements and systems, with a focus on scoring them on the basis of both their capacity and whether or not they are expected to be a significant life safety hazard or have the potential to damage adjacent property should their capacity be exceeded.

Note:

To be a life safety hazard, or to cause damage to other property, a building element must be able to fall to the extent that it is able to create the hazard. This could be direct or through impact with other building elements. To be classified as significant, a number of people need to be exposed to the danger.

These concepts are discussed further in Part A of these guidelines, which defines the types of building element that are expected to pose a significant life safety hazard and whether or not they would be expected to be considered in establishing the earthquake rating for the building as a whole.

An assembly of SSNS elements is often referred to as a system (e.g. a suspended ceiling system), but is considered as a single building element within these guidelines.

Damage to some types of building element can have a significant economic impact, particularly in terms of repair and business continuity costs, even in relatively small earthquakes. While these aspects are not life safety hazards, they may require specific consideration for other types of assessment brief.

During the Christchurch earthquake of 22 February 2011, a significant number of the injuries that people suffered within buildings were attributed to building contents. Assessment reports should also comment on the vulnerability of the building contents.

C10.1.2 Definitions and acronyms

Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable strength capacity has been reached. A brittle structure has little tendency to deform inelastically before it fractures.
Building element	Any structural or non-structural component and assembly incorporated into or associated with a building. Included are fixtures, services, drains, permanent mechanical installations for access, glazing, partitions, ceilings and temporary supports (from the New Zealand Building Code).
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines
Ductile/ductility	Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
HSNO	Hazardous substances and new organisms
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.
Lateral load	Load acting in the horizontal direction, which can be due to wind or earthquake effects
Load path	A path through which vertical or seismic forces travel from the point of their origin to the foundation and, ultimately, to the supporting soil
Non-structural element	An element within the building that is not considered to be part of either the primary or secondary structure
Primary gravity structure	Portion of the main building structural system identified as carrying the gravity loads through to the ground. Also required to carry vertical earthquake induced accelerations through to the ground. May also be part of the primary lateral structure.
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be part of the primary gravity structure.
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects, this is determined using probable material strengths. For geotechnical issues, the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Secondary structural element	A structural element that is not part of the primary structure
Secondary structure	Portion of the structure that is not part of either the primary lateral or primary gravity structure but nevertheless is required to transfer inertial and gravity loads for which assessment/design by a structural engineer would be expected. Included are precast concrete panels, curtain wall framing systems, stairs and supports for significant building services items.
Significant life safety hazard	A hazard resulting from the loss of gravity load support of a member/element of the primary or secondary structure, or of the supporting ground, or of non-structural items that would reasonably affect a number of people. When shelter under normally expected furniture is available and suitable, mitigation of the hazard below a significant status is assumed.
SSNS	Secondary structural and non-structural
Unreinforced masonry (URM)	A member or element comprising masonry units connected together with mortar and containing no steel, timber, cane or other reinforcement

Veneer	A skin or leaf of URM (typically a single wythe) that is reliant on intermediate support from other elements to resist face loads
Wythe	A continuous vertical section of masonry one unit in thickness. A wythe may be independent of, or interlocked with, adjoining wythe(s). A single wythe is also referred to as a veneer or leaf.

C10.1.3 Notation, symbols and abbreviations

Symbol	Meaning
%NBS	Percentage of new building standard as assessed by application of these guidelines
b_g	Glass panel width
$C_i(T_p)$	Spectral shape factor for a part in accordance with NZS 1170.5:2004
c	Clearance between glass panels and framing
h_{cw}	Curtain wall fixing height
h_g	Glass panel height
Q	Element performance factor
S_p	Structural performance factor of a building in accordance with NZS 1170.5:2004
T_p	Fundamental period of vibration for a part in accordance with NZS 1170.5:2004
$\delta_{capacity}$	Deformation capacity of an SSNS element and its connections
δ_{demand}	Deformation demand on an SSNS element and its connections
δ_i	Lateral deformation of building floor level i at the location of an SSNS element
Δ_i	Lateral deformation of building floor level i at its centre of mass
δ_{panel}	Deformation capacity of an individual glass panel
μ_p	Ductility of a part in accordance with NZS 1170.5
ϕ	Strength reduction factor

C10.2 Observed Behaviour of Secondary Structural and Non-structural Elements in Past Earthquakes

C10.2.1 General performance

SSNS elements are often extensively damaged in moderate to large earthquakes. For example, a study of the 66,000 buildings that suffered damage in the 1994 Northridge earthquake revealed that three quarters of these sustained damage to building elements when there was no damage to their primary structure (Charleson, 2008).

Historically, fatality rates attributed to SSNS element failures have tended to be significantly lower than those attributed to failures of the primary structure. The low rate for SSNS failures is mainly because they fell rather than because they triggered a disproportionate collapse. However, injuries and damage to other property were relatively common, particularly heavy items which were particularly hazardous.

The consequences of the failure of a secondary structural element are illustrated in Figure C10.1, which shows the final and original locations of a precast concrete cladding panel.

Figure C10.1: Failure of a precast concrete panel (Baird et al., 2011)

C10.2.2 Performance in the Canterbury earthquakes

There was extensive damage to SSNS building elements during the 2010-11 Canterbury earthquake sequence. Damage was commonly observed for ceilings and in-ceiling services, as were failures of masonry veneer, stairs, precast cladding panels (SESOC, 2013) and plant support in some buildings.

There were a limited number of fatalities solely attributed to SSNS failures in non-residential buildings. However, SSNS were the cause of the majority of injuries caused directly by earthquake damage (Yeow et al., 2017).

Some examples of non-structural damage as a result of these earthquakes are shown in Figure C10.2.

Figure C10.2: Ceiling and other damage (Dhakal, 2010 and McGuigan)

C10.3 Historical Treatment and Factors Affecting Seismic Performance

The structural design of many potentially hazardous SSNS elements has historically received less consideration from a seismic perspective than the design of the primary structural elements. These building elements were generally designed after the primary structure – often, during the construction phase – and with little consideration of how the building in which they would be installed could influence their behaviour during an earthquake.

Many existing SNSS elements were proprietary systems and their design and construction were unlikely to have been overseen by a structural engineer.

There has also been a limited and variable degree of compliance with standards for building services (e.g. NZS 4219 Seismic Performance of Engineering Systems in Buildings). Engineering assessments should therefore always assume non-compliance with these standards until proven otherwise.

The capacity of elements and their connections to the primary structure is usually governed by their ability to accommodate:

- deformations within the structure that are generated by the earthquake
- internal inertial forces generated by the earthquake, and
- impact from any other building elements during the earthquake.

Connections can affect the performance of a building element and will often limit the earthquake score of the whole element, particularly where there is little redundancy or capacity to accommodate deformations.

Note:

Connections for precast concrete panels, curtain wall framing systems and stairs often have insufficient capacity to accommodate the seismic deformations of the primary lateral structural system.

Inertial forces are more likely to cause tall, slender building elements to fail in flexure or topple; shorter elements to rock, overturn or slide; and squat elements to rock or slide.

Failure of an inadequately restrained element (e.g. a suspended ceiling system) can lead to a consequential failure of another element (e.g. a fire sprinkler system).

C10.4 Assessing Secondary Structural and Non-structural Elements

C10.4.1 Approach and objectives

This section outlines the detailed assessment approach that, along with the extent of the assessment, will vary with the complexity of the element or system it forms, and the degree of certainty about element capacities.

The assessment should use the objectives outlined in Section C1 and the procedures outlined in that section and Section C2. In particular, refer to Sections C1 and C2 for guidance on:

- documentation that should be sourced to undertake the assessment
- inspections required to verify that the construction and installation are in accordance with the design documentation
- what, if any, intrusive testing should be considered, and
- the general assessment and analysis procedures that should be considered.

An Initial Seismic Assessment (ISA) should usually be performed before a DSA because this can identify high risk building elements such as heavy chimneys and unreinforced masonry (URM) parapets that could significantly reduce the earthquake rating for a building. Mitigation or replacement of these elements can increase the expected building performance and may avoid the need for a DSA.

Note:

Some elements (e.g. curtain wall framing) may require specialist advice to carry out the assessment. However, when this happens, the engineer carrying out the assessment should establish a holistic understanding of how the building, the elements and their connections interact based on this advice.

Engineers should consider particular vulnerabilities or weaknesses within building elements and use engineering judgement to consider the effects these would have on their behaviour during an earthquake.

C10.4.2 Inspection and investigation

During site inspections, engineers are expected to identify the systems, sub-systems, elements, members and connections, as described in Section C1. They should then use an appropriate level of inspection to determine the condition of the building elements and their connections, as these may have deteriorated since their original installation. This may require intrusive work in roof spaces, ceilings and wall cavities to investigate and record the relevant details. Element and connection damage, decay and corrosion should be recorded so that capacities are downgraded to account for this.

The assessment should also consider the effects of any alterations, particularly for older buildings. These buildings have often gone through multiple changes of occupancy. As a result, there may be a number of additions and modifications to building elements that may not have been well considered.

Some buildings will have large numbers of some types of building element. The inspection and investigation programme for these buildings will need to identify the more critical locations, where the elements are likely to be subjected to the highest demands (forces and deformations) and/or likely to be in the worst condition (e.g. due to corrosion etc.). A statistically valid sample of those more critical elements needs to be inspected, with the inspection quantity chosen to best represent the life safety hazard posed by that type of element within the building.

Note:

A condition assessment is recommended as part of a DSA for all secondary and non-structural elements, particularly those in older buildings. The focus should be on inspecting and assessing the condition of the following elements and their connections to the primary structure:

- URM parapets/chimneys
- precast concrete panel and curtain wall framing
- external heavy plant and HSNO vessel support frames that are attached to a building
- canopies, balconies, billboards and other appendages that are attached to a building.

C10.4.3 General assumptions and considerations

Assessments of SSNS building elements should:

- determine which elements are expected to either pose a significant life safety hazard or damage adjacent property, either directly or indirectly, if they are able to fall. Refer to Part A.
- consider whether a force based approach, a deformation-based approach or aspects of both approaches are appropriate for each element.
- consider how a force based approach used in one direction would influence or be influenced by a deformation-based approach in the other
- determine demand forces and deformations in accordance with Section C3.

Note:

The demands for these elements are likely to be those specified in Section 8 of NZS 1170.5:2004.

- carefully consider the appropriate part/component categories (P1 to P6) within tables 8.1 and C8.1 of NZS 1170.5:2004. More than one part/component category may apply to a particular building element, but only those relating to life safety are intended to be considered.
- consider whether an element could fall off supports such as ledges that are too short to accommodate the deformations of the primary lateral structure
- consider whether an element is or could become an unintended integral part of the primary structure that can both change the behaviour of the structure and overload the connections to the element, or cause it to buckle or crush

Note:

Heavy, non-loadbearing infill walls can have inadequate clearance between them and the structural fames that surround them. Refer to Section C7 for guidelines on assessing their effect on reinforced concrete frames.

SSNS elements such as stairs and precast panels that are often connected to more than one level of the building can also do this when they have insufficient capacity to accommodate the deformations of the primary lateral structure.

The effect these have on the primary structure, including the impact on plan regularity, also needs to be considered.

- consider how the orientation of the element relative to the primary structure will affect both the demand and the capacity
- identify the critical and controlling load paths, the strength hierarchy, and likely mechanisms of the system to assist with determining the available ductility capacity using a rational analysis (where possible)
- base the assessment on probable capacities (i.e. the strength reduction factor should be set at 1.0 and probable material strengths should be used), and
- assess element capacities using Section C10.6 assuming that the load path into and out of each element is complete and sufficient to transfer the required demands. This assumption should be confirmed.

A force based assessment approach (Section C10.4.4) may be sufficient for SSNS elements that are only connected to the primary structure at one horizontal elevation or floor level.

A deformation-based assessment approach (Section C10.4.5) is usually needed when:

- a relatively rigid SSNS building element is connected to the primary structure at more than one horizontal elevation, or
- there are multiple sub-systems of different configurations and materials.

Elements that are flexible in one direction may be able to accommodate deformations of the structure in that direction, so may only need to be assessed using a force based approach.

The capacity of some connections to the primary structure (Section C10.5) and SSNS elements may need to be assessed for simultaneously applied deformations and forces, particularly when the elements are heavy.

C10.4.4 Force-based assessment

Element demand forces should be determined in accordance with Section C3.

For a force-based assessment the engineer carrying out the assessment will need to determine the probable strength capacity of each member within the SSNS element using:

- material-specific guidance in Sections C1 to C9
- connection-specific guidance in Section C10.5
- element-specific guidance in Section C10.6, and
- other references as necessary.

In doing so, the potential failure mechanisms should be identified and, where relevant, deformation capacities and fixing capacities determined by rational analysis.

C10.4.5 Deformation-based assessment

Lateral displacements of the primary structure will normally be used to assess the impacts on SSNS building elements and to determine how earthquake actions could be accommodated. Structural analysis (refer to Section C2) will normally be used to predict inter-storey displacements at the location each element is attached to the primary structure.

Note:

The assessment needs to include all sources of lateral deflection (e.g. soil flexibility, ductility (where appropriate), torsion and P-delta) and other deformations (e.g. frame elongation and vertical deformations), including those that may not have been considered when the building was originally designed.

The earthquake score for an SSNS element is in terms of the deformation capacity of the element and its connections and the deformation demand imposed by the building:

$$\%NBS = \frac{Q \times \delta_{\text{capacity}}}{\delta_{\text{demand}}} \times 100\% \qquad \ldots\text{C10.1}$$

where:

δ_{demand}	=	deformation demand for the element and its connections
δ_{capacity}	=	deformation capacity of the element and its connections
Q	=	element performance factor; $Q = 1.0$ for most SSNS elements.

The structural analysis predicting lateral displacement, Δ_{i}, at the centre of mass at each level within the building needs to transform this displacement to the displacement, δ_{i}, at the location of each SSNS building element. Other forms of deformation (e.g. frame elongation and vertical deformations) will need to be added to deformations calculated from those displacements.

The deformation capacity of an SSNS element requires judgement because the structural elements within modern buildings are detailed to prevent building collapse at deformations well beyond ultimate limit state defined shaking levels.

Some elements, such as stairways supported on ledges, have a higher risk of falling and need $Q < 1$ to reflect that. Guidance is provided by Section 8 of NZS 1170.5:2004, which requires ledges providing gravity support for building elements to accommodate ultimate limit state deformations multiplied by a factor of $2/S_{\text{p}}$, where S_{p} is the structural performance factor. The Q for elements supported on ledges would therefore be $Q = S_p/2$.

C10.5 Connections to the Primary Structure

C10.5.1 General considerations

The connections used to attach SSNS building elements to the primary structure are an important consideration and often determine the capacity of these elements. Connections typically have three independent parts that need to be assessed: a fixing into the body of the element, a connector or body of the connection, and a fixing to the structure.

It is important to consider how the seismic actions are distributed to each of the element's connections. This may be more complex when the axes of the element are not aligned with those for the primary structure.

Aspects of connections that the engineer carrying out the assessment should consider include the following:

- the offset between the element's centre of mass and each connection
- the stiffness (or flexibility) of the element relative to that of the structure to which it is affixed
- how inertial forces from the element are distributed between its connections
- continuity of the load path for internal forces through to its connections
- whether there could be unintended consequences due to eccentricity either within the connection or between the connection and the structure, and
- how relative horizontal and vertical movements of different fixing points into the structure are accommodated (e.g. points either side of a seismic gap as well as on two levels).

Note:

These guidelines focus on building elements that pose a significant life safety hazard or are likely to cause damage to other property should they fall. A portion of their connections or fixings may be able to fail without allowing the building element to fall, either because of their location (e.g. those at the bottom of a hanging panel) or because they have some redundancy. However, it will also be necessary to confirm if such a failure could lead to premature failure of other connections or cause other building elements to fall.

C10.5.2 Inspecting connections

Issues that have been observed when inspecting SSNS building element connections and that need careful attention during inspections include:

- corrosion, particularly of hidden fixings
- mechanical damage caused during construction or service
- substitution with another type of connection during installation
- insufficient strength and/or ductility and/or deformation capacity
- fasteners within regions of primary structure that are likely to suffer significant deformation, e.g. an active link zone within an eccentrically braced frame or plastic hinge regions within concrete beams
- the use of welds near areas of high stress concentration

- poor workmanship or design, particularly for anchors
- higher capacity connections that have no mechanical interlock with the element (or structural) reinforcement, and
- connections that rely on shallow embedded or drilled-in anchors.

Holes and slots in connections often have insufficient clearance to accommodate the cumulative effects of fabrication tolerances, installation tolerances and deformations between two levels or sections of the primary structure. Additional issues that affect connections with slotted holes include:

- slots that are too short to accommodate the deformation demands
- bolts that are too close to one end of the slot when the element is installed, and
- possible constraints to the free movement of bolts as washers are missing (which can lead to binding) or the bolts or nuts are more than snug tight.

Note:

Unless the bolts in a slotted connection can be left loose, they may not be able to provide the degree of separation necessary to prevent damage to the SSNS building element or avoid the SSNS element from participating as part of the primary lateral structure. Bushes can be used in the slots to provide confidence that the slot will behave as intended without binding if the bolts have been accidentally overtightened.

The type, size, spacing and condition of fixings such as bolts, rivets and screws will often be critical when determining their capacity. Although it may be difficult and impractical to confirm every fixing, checks should be made to confirm the general arrangements and condition. Consideration could be given to destructive or proof testing of one or more fixings if there is doubt about what is hidden beneath the surface of concrete or these are otherwise inaccessible.

Refer to the appropriate material sections (Sections C5 to C9) for additional guidance on inspecting the:

- material the element is formed from and any fixings into it
- material used for the body of the connection, and
- fixing attaching the connection body to the structure.

C10.5.3 Force-based connection assessments

For force-based assessment of SSNS element connections, the part ductility factor should be taken as $\mu_p = 1.0$ unless it can be demonstrated that a higher factor is appropriate.

The probable connection capacity should be taken as the minimum of the capacity of the fixings into the component, the fixings into the structure and the connector strength. The earthquake score can be evaluated as the probable connection capacity divided by the critical connection demand.

The probable connection capacity should be determined in accordance with either the relevant material sections in Part C of these guidelines or other recognised sources such as

technical literature from manufacturers. The probable capacity of fixings cast into concrete can be calculated in accordance with Section 17.5 in NZS 3101:2006.

Note:

The additional requirements for fixings in Section 17.5 of NZS 3101:2006 should be considered when assessing the probable connection capacity.

The capacity may be affected by structural deformations in another direction.

Forces are only likely to be uniformly distributed between connectors when the body of the element and the structure are both rigid relative to the connector stiffness. A tributary mass method may be more appropriate for heavy elements, with appropriate adjustments where there is little or no connector redundancy.

The probable seismic capacity of adhesive anchor systems is not covered by NZS 3101:2006 and may be assessed using ACI 355.4:2011 Qualification of Post-Installed Adhesive Anchors in Concrete.

C10.5.4 Deformation-based connection assessments

For deformation-based assessment of SSNS element connections, consideration should first be given to the consequences of failure. If the connection is the weakest link within a relatively rigid element and is therefore likely to fail when this element is subjected to the types of seismic deformation demands defined in Section C10.4.5, the earthquake score is evaluated using the probable deformation capacity of the connections.

Note:

The deformation capacity of a connection for a rigid building element will often be limited by the observed clearance or length of slot as noted in Section C10.5.2.

Flexible building elements that are able to accommodate some or all of the structural deformations may require a combination of force and deformation-based assessment.

C10.6 Specific Element Capacities

C10.6.1 General considerations

SSNS elements are often assemblies of members that collectively define the strength and deformation capacity of the element. Behaviour of the elements (including precast panels, stairs, ceilings, and bracing) is dictated by physical properties such as their area, material grade, thickness, depth/slenderness ratios, lateral torsional buckling resistance, and connection details. Connected members include sheet products, light gauge steel, bracing, stiffeners, struts, and frames.

The probable capacity of SSNS elements should be determined using the probable material strengths as outlined in Sections C5 to C9 depending on the type of material. No strength reduction factor needs to be applied (i.e. use a strength reduction factor of $\phi = 1.0$) and probable material strengths can be used.

The inspected physical dimensions of individual members/elements that are being relied on for load transfer should be used. Modifications to member capacities can be caused by notching and holes. The presence of decay or deformation should be noted and allowed for.

C10.6.2 URM parapets/chimneys

The seismic responses and earthquake scores of URM parapets and chimneys will often be governed by their aspect ratios, condition, construction details and any existing restraint details.

C10.6.2.1 Inspection

Common issues that have been identified in seismic assessments of these elements and that need careful attention during inspections include:

- insufficient or no restraint to prevent out-of-plane movement/toppling
- roof flashings that are often chased into masonry just above roof level, creating a potential weak point where rocking can occur
- out-of-plumb parapets or chimneys that have a reduced capacity to resist toppling in the out-of-plumb direction
- architectural features such as concrete strips or beyond-vertical facings that can create potential weak points or impose overturning forces
- poor condition of restraint members or the structure they are attached to, and
- elements that are able to fall onto and cause damage to adjacent property.

C10.6.2.2 Assessment

The probable capacities of URM parapets and URM chimneys should be calculated in accordance with the “vertical cantilevers” subsection within Section C8 of these guidelines.

The earthquake score of a rectangular URM chimney is the lower of the scores for each of the two principal axis directions.

The assessment needs to consider the potential life safety hazard and the potential for damage to adjacent property should a parapet or chimney fall.

Note:

Chimneys falling onto iron roofs, or through tiles into ceiling spaces with a mitigation measure such as a plywood deck, may not result in a significant life safety hazard and therefore may not need to have an earthquake score. Similarly, a chimney falling onto an iron roof and sliding off will only be a significant life safety hazard if the rubble can fall onto an area where a number of people could be at risk such as paths and entryways.

Assessments of URM parapets and URM chimneys with existing bracing members need to consider the following issues that were identified following the Canterbury earthquakes:

- any interaction between the response modes of the URM element and the support structure that the other end of the bracing member connects to
- the degree of deformation compatibility between bracing support points when a URM parapet/chimney is braced by more than one bracing member
- the strength of the bracing member, its fixings, and the structure it is fixed to, and
- early pull-out of adhesive anchors due to poor workmanship, poor design, or their being installed in URM flexural tension zones.

C10.6.3 Masonry veneers

Earthquake scores for masonry veneers are often governed by the locations and condition of their internal ties. Common issues that are identified in seismic assessments of masonry veneers and that should be considered by the engineer include:

- insufficient numbers of ties to restrain out-of-plane actions
- ties that can be rendered ineffective by inadequate anchorage, weak mortar or corrosion
- the capacity of veneers tied to a main wythe in cavity wall construction can depend on the capacity of the main wythe, and
- insufficient clearance to prevent in-plane loading from the primary structure.

Note:

The location of a masonry veneer may determine how it is to be considered in terms of a significant life safety hazard or its potential to damage other property if it were to fall. There is a greater life safety hazard when a veneer is situated above an egress path from the building or public thoroughfare. Other areas around the perimeter of the building are less critical and may not need to be considered.

C10.6.3.1 Inspection

Identify the high-hazard locations and determine the thickness of any veneers. A good indication of the thickness is often obtained by examining a corner intersection of the veneers. The interlocking bricks should have a brick end visible in every second course, which will identify the thickness. If it is impossible to identify the veneer thickness in this manner, it may be necessary to drill a small hole through the veneer to physically measure its thickness.

Tie locations can be identified using a metal detector from the outside surface of the veneer. This investigation will also provide information on the spacing of the ties. Once located, a hole can be drilled through the veneer and an endoscope inserted to establish the type and

condition of the ties. Note that this requires specialist equipment and operators. An individual brick can also be removed (for later replacement) to gain access to the cavity.

The thickness of the veneer and the presence, condition and locations of ties are used to decide whether or not further assessment is required.

C10.6.3.2 Assessment

Brick masonry veneers and requirements for ties shall be assessed in accordance with Section C8 with the demands determined in accordance with Section C3.

Supporting structure shall be assessed in accordance with the requirements for the appropriate material (refer to Section C5 to C9).

C10.6.4 Heavy non-loadbearing partition walls

Heavy non-loadbearing partition walls usually consist of:

- unreinforced clay brick masonry
- hollow clay brick masonry (which can be filled or unfilled, reinforced or unreinforced)
- unreinforced concrete block masonry (which can be solid or hollow, unfilled, partially filled or fully filled), or
- reinforced concrete block masonry (which can be partially filled or fully filled).

Common issues that have been identified in seismic assessments of heavy non-loadbearing partition walls include:

- insufficient or missing restraint at the tops of the walls that is required to prevent out-of-plane movement, forcing the walls to act as cantilevers, and
- insufficient gaps between ends of walls and the main structure to allow for inter-storey drift, which can result in damage to the walls and/or the main structure that may not have otherwise occurred.

Earthquake scores for these elements will often be governed by their aspect ratios and connections.

The probable out-of-plane capacity of URM non-loadbearing partition walls should be calculated in accordance with Section C8 of these guidelines as “wall elements under face load”.

The probable in-plane capacity of URM non-loadbearing partition walls should be calculated in accordance with Section C8 of these guidelines as “walls under in-plane load”.

C10.6.5 Precast concrete panels

Precast concrete panels for the purposes of this section include all tilt-up and cladding/façade variants. Where these are an integral part of the primary structure, even if this is unintentional, they should be considered as primary structure.

Earthquake scores for precast concrete panels will often be governed by their connections, which should be inspected and assessed using the guidance in Section C10.5.

The assessment of precast concrete panels normally requires a combination of a force-based approach and a deformation-based approach.

C10.6.5.1 Inspection

Inspections of precast panels should aim to identify the following high-risk features:

- panels that do not have sufficient clearance with adjacent panels to accommodate inter-storey drift
- panels attached to two floor levels using slotted connections
- panels that may collide at building corners
- L-shaped panels that wrap around building corners, and
- panels that are supported by cantilevered concrete slabs, or with large eccentricities to the supporting structural element.

Note:

Following the February 22, 2011 Christchurch earthquake, several precast panel connections that were slotted to accommodate movement were observed to have been installed incorrectly, e.g. with washers welded in place or the bolt at one end of the slot, as shown in Figure C10.3 (Baird et al., 2011). These slotted connections were not able to accommodate movement as intended.

Figure C10.3: Precast connections with no remaining sliding movement allowance to accommodate inter-storey drift (Baird et al., 2011)

C10.6.5.2 Force-based assessment

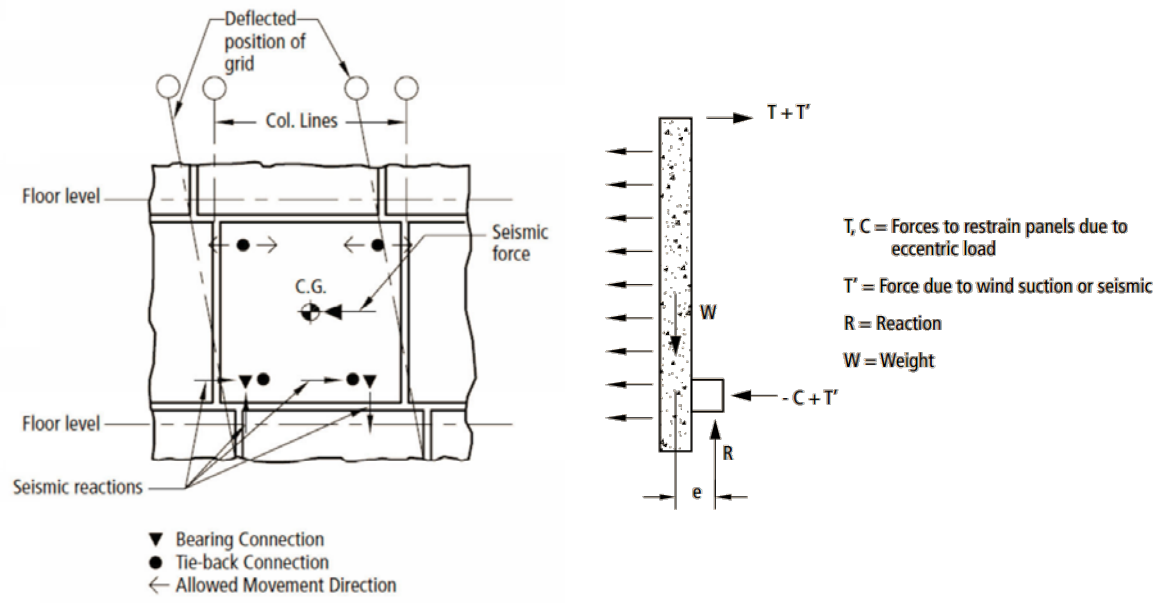
A force-based approach is required to assess the in-plane and out-of-plane demands and capacities of precast panels.

Note:

When calculating seismic demands, the NZS 1170.5 part spectral shape factor can be conservatively taken as $C_{\mathrm{i}}(T_{\mathrm{p}}) = 2.0$. The fundamental period of vibration for most precast panels, T_{p}, will be less than 0.75 seconds. Similarly, the NZS 1170.5 ductility of the part should be taken as $\mu_{\mathrm{p}} = 1.0$ unless the specific detailing of the connections can be verified as per Section C10.5.2.

It is important to consider how the seismic demands are distributed to each of the precast panel connections. For example, in-plane seismic demands for a single-storey panel are typically resisted by a pair of fixings at the base of the panel. These connections have to resist the total horizontal in-plane demands and the additional vertical demands induced by over-turning in the panel, as shown in Figure C10.4(a).

Out-of-plane seismic demands can often be considered as a uniformly distributed load that is resisted by all panel connections or that may be distributed using tributary areas. Additional out-of-plane demands will also be generated by the eccentricity between the panel and supporting primary structure, as shown in Figure C10.4(b).



(a) In-plane demands from deformation of primary structure relative to a precast panel

(b) Out-of-plane demands, including eccentric support by the primary structure

Figure C10.4: Simultaneous seismic demands on a precast concrete panel (PCI, 2007)

Section C10.5.3 provides guidance on evaluating the seismic demands and capacities for precast panel connections. If required, the out-of-plane capacity of the panel can be assessed in accordance with the BRANZ Design Guide – *Slender Precast Concrete Panels with Low Axial Load* (Beattie, 2007).

Note:

The capacity of the connection is typically more critical than the out-of-plane capacity of the panel. The out-of-plane capacity of the panel is more critical when the panel is very slender, such when the height-to-thickness ratio is greater than 60 (Beattie, 2007), or when reinforced using non-ductile mesh.

C10.6.5.3 Deformation-based assessment

A deformation-based approach is also required to assess the deformation capability of the precast panel and its connections. The lateral displacements of the primary structure can be used to determine the horizontal deformation demands for the panel and its connections.

The horizontal deformation demands are the drift between fixing points of the panel. For full storey height panels, this will be equivalent to the inter-storey deformation evaluated as

described in Section C10.4.5. These demands should include any additional deformations, such as frame elongation and vertical deformations.

The high in-plane stiffness of precast panels normally requires the connections to accommodate all of the relative deformations between the primary structure and the panel during earthquake shaking, as shown in Figure C10.4.

A slotted or flexible connection is often used in modern structures to accommodate this relative deformation. The earthquake score can be evaluated as the lateral deformation capacity of the connections divided by the lateral deformation demands.

Older structures are not commonly detailed to accommodate these deformations; however, the panels and their connections may be able to deform and accommodate some lateral deformation. In such cases, loads will be introduced into the panel and the engineer should then consider the capacity the panel and connections have to transfer these loads when assessing the earthquake score. Potential failure mechanisms include failure of the connections (bolts shearing off, brackets yielding, and fixings pulling out) or shear/flexural damage to the panel.

Earthquake scores for precast panels should reflect the high-risk features identified during their inspection (refer to Section C10.1.1 earlier).

C10.6.6 Stairs and ramps

Stairs and ramps are part of the gravity structure. They are not generally elements within the primary lateral structure, but ramps may be designed to be an integral part of it.

Both stairs and ramps are required to support gravity loads and may be the only means of escape from the building following an earthquake. Collapse of one flight of stairs can lead to a pancaking effect as this progressively overloads each flight below it (as was observed in the Forsythe Barr building in the 22 February 2011 Christchurch earthquake). This cuts off the means of escape as well as being a significant life safety hazard.

C10.6.6.1 Inspection

The earthquake scores for stairs and ramps will often be governed by their connections. Therefore, the first step in an inspection is to identify how the stair or ramp element is connected to the primary structure and how it is likely to respond to movement of the upper floor relative to the lower floor. Straight stairs may be attached at both ends but are often free to slide longitudinally at one end (usually the bottom), or at an intermediate landing.

Identify the support provided for any mid-height landings – particularly for return stairs that are often supported along one edge of the landing – and how the landing and flights will be affected by lateral movements. Landings in straight stairs may only be supported by the flights.

Note:

There may be different degrees or types of connectivity in the longitudinal direction (along the flight) and transverse direction.

There may be unintentional connection (e.g. when a gap becomes plugged or closed) or disconnection (e.g. when a ledge has insufficient width) as the primary structure deforms.

Stairs with inadequate allowance for some primary structure deformations can become an unintended strut between floors and should be considered as an element within the primary lateral structure when assessing their earthquake score. Their potential effect on the primary structure, including the impact on plan regularity, also needs to be considered.

Straight stairs were shown to be more likely to pose a significant life safety hazard than return stairs in the Christchurch earthquake. However, damaged landings in return stairs may affect building evacuation.

Common issues that have been identified in seismic assessments of stairs and that need careful inspection include:

- slotted holes, sliding gaps, and ledge bearing dimensions, which are often insufficient to accommodate the cumulative effects of fabrication tolerances, installation tolerances and primary structure deformations
- ledge widths that can be reduced by spalling along the edge
- sliding gaps at the base of precast concrete stair units, which often become filled over time and could impair their function or lead to premature stair failure due to axial compression (refer to Figure C10.5), and
- mid-height landings and flights of return stairs that may warp (vertically) to accommodate horizontal deformations of one floor level relative to the other.

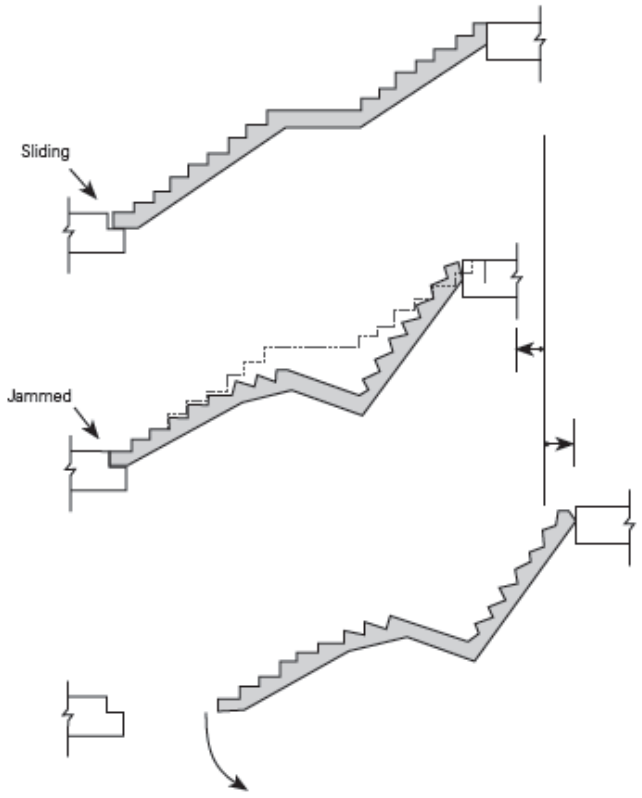


Figure C10.5: Straight stair failure as the upper floor moves left (middle) and then right (bottom) relative to the lower floor (Beca, 2011 and Bull, 2011)

Additional guidance is provided by DBH (2011), SESOC (2013) and FEMA E-74 (2012).

C10.6.6.2 Force-based assessment

Force-based assessment is generally required to verify that the connection capacity transverse to the ramp or stair flight(s) is capable of restraining the ramp or stairs. The transverse strength of the ramp or stair flights is unlikely to be of concern because the loading will be in plane.

C10.6.6.3 Deformation-based assessment

The deformation demands for ramps and stairs are calculated using Section C10.4.5.

The deformation capacity depends on the geometry of the ramp or stairs and how they are connected (or not) to the structure at the two floor levels. Sliding joints have three states: sliding (or free); jammed; and unseated (or falling), as illustrated for straight stairs in Figure C10.5. The undesirable jammed state induces similar compression forces in the element to those that would occur if it was connected at both ends. However, it may not pose the same life safety risk as the unseated or falling state. Stairs connected at both ends have a fourth state whereby the flight becomes a tie and is stretched.

Deformations that are transverse to straight stairs are less likely to pose a life safety hazard because there will usually be a portion of the seating supporting the element. However, for wider ramps, that could even be wide in comparison to their length or span, and could become unseated over sufficient length to jam as soon as the deformations reverse.

The earthquake score for sliding joints should be taken as the lesser of the following:

- the seating length (less reductions due to spalling, etc.), divided by the
- the available gap, divided by the required seating length from NZS 1170.5:2004, and
- the compressive capacity of the element divided by the difference between the deformation demand and the available gap (see below).

Steel stair flights may be capable of accommodating larger inter-storey deformations than reinforced concrete flights using slotted hole connections. However, if the bolts reach the ends of the slots, parts of the stair may yield or deform in a manner that does not result in collapse. If the slot length is insufficient to accommodate the expected inter-storey drift and there is the potential for ductile deformation of the steel elements, the bolt fixings should be checked to ensure that they are not weaker than the steel elements.

When a ramp or stairway is attached at both ends of the flight or is jammed, compressive loads will be induced in the stairs. Their earthquake scores will depend on their capacity to resist these loads (in which case they will influence the behaviour of the main structure) or, if they are not strong enough, on how they will fail. Potential failure mechanisms include failure of the end connections (shearing of bolts, yielding of rebar, fracture of timber connections), buckling or compressive failure of the stair flight, and shear failure of the stair flight.

Yielding steel may prevent collapse of a reinforced concrete flight that is being stretched, provided the steel is well anchored into both the floor and the flight. However, compression forces that develop when the motion reverses could lead to shear failure at one or both ends of the flight, and possibly even collapse. Understanding the detailing of reinforced concrete stairs is important for judging the likely earthquake behaviour.

Steel framed structures may have consequences that are similar to those for reinforced concrete structures. Again, the analysis of the main structure behaviour will influence the stair behaviour.

Stairs in timber structures that are fixed at both ends are more likely to influence the behaviour of the primary structure because the diagonal stair linking the floors may have a similar stiffness to the primary lateral structure. Large timber and steel stair stringers are likely to transfer significant forces without overloading the stair. However, the connections between the stair and the floor may fail, either by extracting the fixings from the floor or through local failure of the timber at the connection (refer to Figure C10.6). This could lead to collapse of the flight in an opening situation or the top of the flight could rise above the floor level, as shown in this figure.

Figure C10.6: Stair that has sheared connections to the floor on a timber framed structure and ridden up above the floor (BRANZ)

C10.6.7 Heavy plant, storage racking, and hazardous substances and new organisms (HSNO) vessels

Earthquake scores for heavy plant, storage racking and HSNO vessels will often be governed by the capacity of braced frames and the capacity of connections to the main structure.

Heavy plant includes air conditioning/handling units, pumps and chillers, although this list is not exhaustive. Minimum mass limits for heavy plant, depending on the location of the plant item and its contents, are given in Section 8 of NZS 1170.5:2004. Boilers have additional considerations that are addressed in the specific guidance for HSNO vessels below (Section C10.6.7.4).

Storage racking has a wide range of applications, ranging from low level shop racking (typically less than 2 m high for the public to pick items from stock) to 10 m or more in

height for warehouse storage applications. This guidance is for high level storage racking (>2 m high) because lower racking generally only poses a low risk to life safety.

HSNO vessels are tanks or containers with substances that would pose a significant health hazard if the vessel or its restraints failed or the tank ruptured and the contents were spilled. The contents of some pressure vessels (e.g. boilers) are stored at high temperature and pose significant scalding and explosion hazards.

Note:

HSNO vessels may or may not be marked as containers of hazardous substances, depending on the type and volume of the substance. There is guidance on labelling in the *Approved Code of Practice for the Management of Substances Hazardous to Health in the Place of Work* (Department of Labour, 1997) and the *Stationary Container Systems Performance Standard* (Worksafe New Zealand, 2015).

For pressure vessels, AS/NZS 1200:2015 *Pressure Equipment* refers to NZS 1170.5:2004 and to IPENZ Practice Note 19 *Seismic Resistance of Pressure Equipment* (IPENZ, 2016) for the seismic design of boilers, pressure vessels and pressure piping.

C10.6.7.1 Inspection

Common issues that are identified in seismic assessments of heavy plant, storage racking and HSNO vessels that require consideration when inspecting them include:

- insufficient bracing and inadequately proportioned frame members
- insufficient connection capacity, particularly to the primary structure, and
- insufficient allowance for differential movement between plant and associated piping.

Note:

The contents of an unmarked vessel should be assumed to be hazardous unless proven otherwise.

C10.6.7.2 Assessing heavy plant

Heavy plant items may provide significant driving mass for the primary structure but do not otherwise influence the structure response. Plant items with a mass exceeding 20% of the combined mass of the plant item and the structure and a lowest translational period greater than 0.2 seconds are expected to affect the behaviour of the primary structure. Such situations require special consideration during the assessment of the primary structure and the dynamic characteristics of the item. This should be undertaken in accordance with the rest of Part C of these guidelines.

Likely failure mechanisms and their consequences with respect to life safety need to be identified for heavy plant items. The consequences of failure can be variable. The location will often determine its potential life safety hazard along with the consequences of its failure. For example, a container discharging a hazardous substance into an isolated location may pose minimal life safety risk but could still pose a serious health risk.

For those items that are considered to pose a significant life safety risk, the engineer should ascertain the method(s) of support to determine whether the plant item will be influenced by

force or deformation. Floor mounted and ceiling supported plant are likely to be most influenced by lateral forces. However, ceiling mounted equipment may also require deformation considerations and the effects of impact against the main structure or other components. Floor mounted plant may also be connected to pipework and electrical trunking which is attached to the level above. Consideration should be given to the interaction between the plant item and these items, keeping in mind the potential to cause a significant life safety hazard.

Heavy plant items within a building are categorised as "P2 and P3" in accordance with NZS 1170.5:2004 Table 8.1. However, these items may also be categorised as:

- P5 when in importance level 4 (IL4) buildings or they are required to remain operational for the building to be occupied, or
- P6 if their failure could cause disproportionate damage (e.g. water loss above perishable goods).

It should be noted that neither the P5 nor the P6 cases are related to life safety as considered in these guidelines, notwithstanding that damage to perishable goods may become a health risk if they cannot be removed quickly.

The engineer should determine the demands on the plant item restraint system in accordance with Section C3.

The earthquake score can be evaluated as the probable capacity of the restraint system, including the connections divided by the demand on the restraint system.

C10.6.7.3 Assessing storage racking

High level racking systems (those over 2 m) are considered to be structures in their own right and are therefore subject to the requirements of the New Zealand Building Code (NZBC). However, there are many storage racks between 2 m and 4 m high in commercial premises that were unlikely to have been designed and installed to meet the NZBC.

Racks are normally only installed at the ground floor level of a structure to provide forklift access for loading and unloading. Therefore, no site ground motion amplification will be necessary.

However, some racking systems are installed on a suspended floor slab (e.g. supermarket racking in a store with parking underneath). In these cases, the engineer needs to consider the effect of the supporting structure on the rack performance when assessing the probable rack capacity. The strength of the supporting structure should also be considered to ensure that the seismic loads introduced to the floor can be adequately resisted (refer to Section C5).

The majority of racking systems will be proprietary products and, if it is possible to ascertain the brand of the rack, an approach to the supplying company is recommended to obtain design information. These companies may also be able to supply the design capacity using in-house design software.

High level storage racks are quite different from building structures in that they use simple principles in their vertical and lateral load-resisting systems and they generally carry large loads; normally much larger than the weight of the rack itself. Therefore, it is important to

identify the characteristics of the loads stored on the racks. There is also little redundancy available in the system, so the failure of one element can lead to a domino effect on the rest of the rack.

Racks have an advantage over a building structure in that, as they are lightweight compared to their contents, their performance can be improved by strategically re-arranging the contents so the heavier items are near the floor and lighter items above. Even when there is no opportunity to re-arrange the stock to reduce demands on the rack, there is still an opportunity to partially unload it and set appropriate loading limits.

Note:

The engineer needs to be sure that any content stacking arrangements or loading limits can be managed on an ongoing basis if these are to reduce the demands on the racks and avoid life safety (or any other) consequences.

When assessing the seismic capacity of the racking system, the load-resisting mechanisms should be checked first. In the down-aisle direction, the critical factor will usually be the bending capacities of the connections between the uprights and the beams, because these members are normally stronger than these connections. In the cross-aisle direction, lateral load resistance is provided by braced transverse frames. Racks have minimal reliable diaphragm action at the beam (or shelf) levels, which is provided by friction forces between stored pallets and the beams.

The seismic demand should be assessed in accordance with NZS 1170.5:2004. For the cross-aisle direction, a maximum ductility factor of 1.25 should be used, unless cyclic test results are available for representative frames. In the down-aisle direction a ductility factor of 1.25 should also be used unless cyclic tests on connections demonstrate ductile behaviour. While not so critical in the down-aisle direction, the probable capacities and ductilities of the cross-aisle frame baseplates and anchor bolts should be estimated in order to determine the cross-aisle capacity. Anchor bolts are seldom cast-in-place so their capacity will generally be less than that of a cast-in anchor. In the absence of information on the anchor bolts, one of these should be removed to determine its properties. An assessment of the anchor capacity may be made using NZS 3101:2006 section 17.

The earthquake score can be evaluated as the probable capacity of the racking system and its connections to the main structure in both the down-aisle and cross-aisle directions divided by the critical demand on the rack and its restraint system. The structure to which the rack is attached should also be considered, and the minimum earthquake score will be the least of these values.

C10.6.7.4 Assessing HSNO vessels

HSNO vessels should be assessed using either a force-based or deformation-based assessment as appropriate.

Unmarked vessel contents should be assumed to be hazardous unless proven otherwise, and assigned a Part category of at least P2 or P3 in accordance with NZS 1170.5:2004 Table 8.1. Refer also to the notes beneath that table for further direction on inclusions and exclusions.

Forced-based assessment of HSNO vessels

HSNO vessels will normally be mounted on the floor of a building structure unless they are light enough or have a small enough volume to be mounted on a wall. In the absence of specific information on the contents and their degree of hazard, the engineer should judge the likely effects of failure given the position of the vessel and his/her assessment of the contents, and then use NZS 4219:2009 or NZS 1170.5:2004 to determine the demands on the vessel and its restraint system.

The earthquake score can be evaluated as the lowest of the probable vessel or restraint capacity, including the probable capacity of the connections, divided by the critical demand on the vessel or its restraints, respectively.

Deformation-based assessment of HSNO vessels

HSNO vessels will generally not be stand-alone items that are only connected to the structure through the restraint system. Supply and/or distribution pipes may also be connected to the vessel. The failure of these may be as catastrophic as failure of the vessel itself.

If the only pipework is on the same floor as the HSNO vessel, the restraints provided for this pipework should be assessed for their capacity against critical demands. The potential for stiffness incompatibility between the vessel and pipework restraints and the effect on the integrity of the system should be assessed. Flexible joints may be already in place or all connections may be rigid.

If the pipework is restrained at a floor level above the one on which the vessel is mounted, there will be a degree of inter-storey drift that should be accommodated.

The maximum demand drift may be calculated using Section C10.4.5. A judgement will be required as to the ability of the pipework and its connection to the vessel to accommodate this demand.

For pipework that spans between the vessel and the floor above, the earthquake score can be evaluated as the probable deformation capacity of the pipework divided by the demand deformation.

C10.6.8 Curtain wall framing systems

Earthquake scores for curtain wall framing will often be governed by the ability of the system to accommodate the lateral deformations of the primary structure. A force-based assessment is only required for heavy curtain wall framing systems; i.e. those that weigh more than 70 kg/m^2.

C10.6.8.1 Inspection

Common issues that have been identified in seismic assessments of curtain wall framing include:

- limited or no ability to accommodate inter-storey deformations, and
- in the event of failure, that significant areas of the glazing or curtain wall may fall.

Mitigation measures may be provided to limit the life safety risk should a curtain wall or its large or hazardous components fall. Where these are provided, they should be inspected first because further inspection and assessment may not be required.

Examples of mitigation measures include situations where the curtain wall is above an area that is inaccessible and low hazard glazing has been used; for example toughened glass (also known as tempered glass), laminated glass, or wire mesh glass (also known as wired glass).

Note:

Toughened (or tempered) glass greatly reduces the seismic hazard because the glass breaks into small dull fragments instead of large hazardous shards. Toughened glass can sometimes be identified by a small label etched in the corner of the glass. Look for words such as “TEMPGLASS”, “TEMPERED”, “TOUGHENED” or “SAFETY”. If no label is present, toughened glass can also be identified by looking at a light source on the other side of the glass through polarised lenses. If the glass is toughened it will reveal lines or spots in the glass that are not normally visible.

Laminated glass has a film layer that does not allow for the glass to separate when broken. This reduces the risk of the glass falling out of the curtain wall and also prevents people or objects from falling through the glass. Laminated glass is commonly used where there is the possibility of impact by a person, such as next to a walkway. Laminated glass is more difficult to identify as it is less likely to have a label. It is possible to identify laminated glass by looking for multiple reflections when an object is placed next to the glass. Glass that is not laminated shows only two reflections from the two surfaces of the glass.

C10.6.8.2 Force-based assessment

The seismic demands on connections for these systems can be determined using Section C10.5.3.

Note:

The double-skin systems shown in Figure C10.7 are examples of curtain wall framing that would require a force-based assessment.

As most residential and commercial curtain wall systems are lighter than 50 kg/m^2, a force-based assessment is not often required.

Figure C10.7: Double skin curtain wall system

Note:

Wind actions typically govern the design of the connections and componentry of curtain wall systems because these systems are relatively lightweight. A force-based assessment will generally demonstrate that curtain wall systems are adequate to resist seismic actions and hence there is no need for further assessment. Observations of curtain wall framing damage during the 2010-2011 Canterbury earthquakes demonstrated that most failures were the result of deformation demands, rather than overloading of connections (Baird et al, 2011).

When calculating seismic demands, the NZS 1170.5:2004 part spectral shape factor, $C_{\mathrm{i}}(T_{\mathrm{p}})$, can be taken conservatively as equal to 2.0. Similarly, the NZS 1170.5 part ductility demand, μ_{p}, should be taken as equal to 1.0 unless it can be verified that the connections are specifically detailed to provide ductility as per Section C10.4.4.

Curtain wall connections are normally regularly spaced, so it is reasonable to consider that seismic demands will be distributed evenly between connections in both the in-plane and out-of-plane directions. Seismic demands should therefore be calculated using the tributary weight of the curtain wall multiplied by the parts coefficient.

Curtain wall connections typically comprise three components: fixing to the curtain wall transom or mullion, the connector body, and the fixing to structure. The probable connection capacity should be taken as the lowest of these three components. The earthquake score can be evaluated as the probable connection capacity divided by the critical connection demand.

The probable capacity of connections should be determined in accordance with the relevant sections within Part C of these guidelines or other recognised sources such as technical literature from manufacturers.

C10.6.8.3 Deformation-based assessment

If the engineer determines that the curtain wall itself presents a significant life safety risk if it fell from the building, then a deformation-based assessment is required. Deformation demands are evaluated using Section C2.

The deformation demands upon curtain wall framing are typically accommodated by clearances within the system between the stiff glass panes, and the more flexible extruded aluminium transoms and mullions, as shown in Figure C10.8. In higher seismic zones, or in more flexible buildings, more clearance is required than can be accommodated by the glass to frame clearance alone, so seismic mullions/transoms are sometimes provided. These have separate extruded aluminium sections that can move relative to each and accommodate the seismic movement.

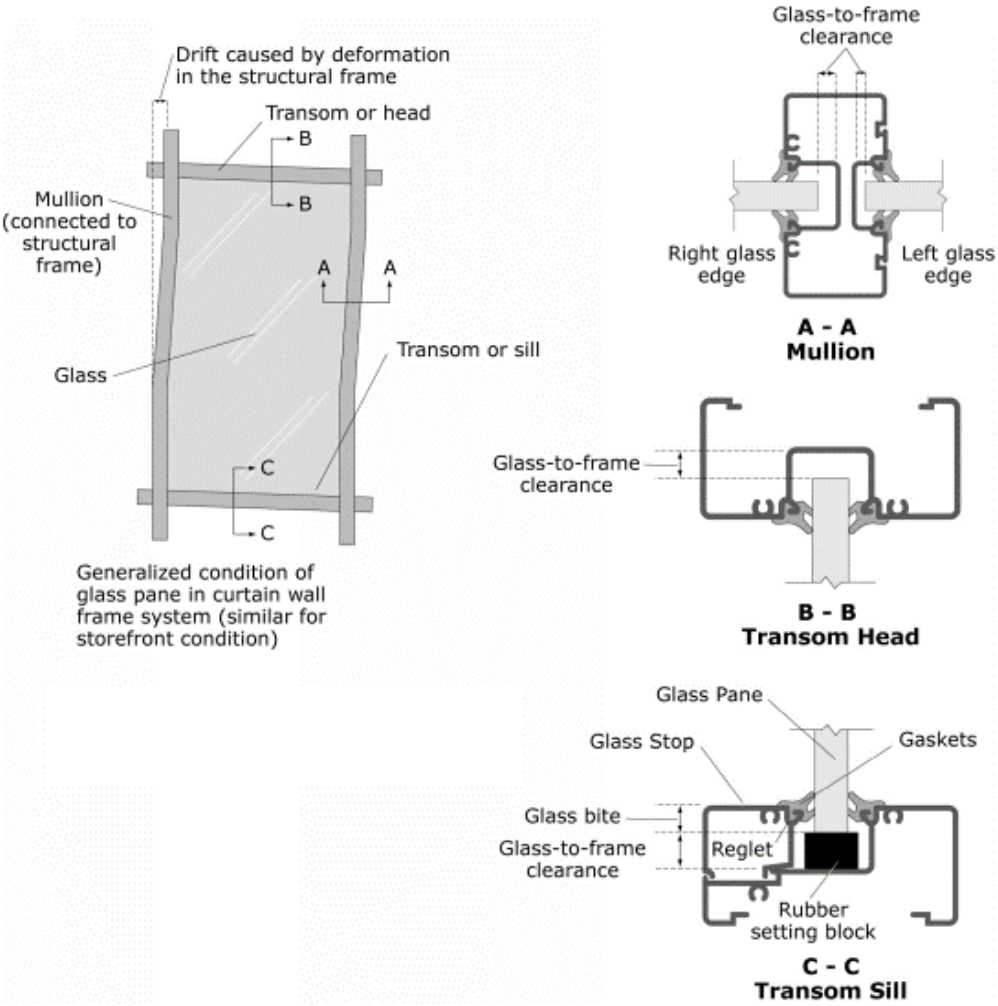


Figure C10.8: Typical curtain wall indicating glass-to-frame clearance (FEMA E-74, 2012)

The deformation demands for the individual glass panel are given by Equation C10.2223 below. The curtain wall fixing height is normally the inter-storey height. However, care is needed where curtain wall systems are fixed to precast concrete spandrel panels and the inter-storey drift is concentrated over less than the full storey height.

$$\delta_{\text{capacity}} = \frac{h_{\text{cw}}}{h_{\text{g}}} \times \delta_{\text{panel}} \qquad \ldots\text{C10.2}$$

where:

δ_{panel} =deformation capacity of an individual glass panel
h_{g} =glass panel height
h_{cw} =curtain wall fixing height.

Most of the deformation capacity in an individual panel is provided by the glass to frame clearance, or the movement capacity of mullions/transoms, which should be determined from drawings if possible. If no information is available and it is not possible to determine these by inspection of the glazing system, use the typical construction clearance of 10 mm between the glass and frame. The deformation capacity is calculated from the clearances using Equation C10.3334, which includes allowance for the glass being seated on setting blocks.

$$\delta_{\text{panel}} = c\left(2 + \frac{h_{\text{g}}}{b_{\text{g}}}\right) \quad \text{…C10.3}$$

where:

c =clearance between sides and top of glass panel and framing
b_{g} =glass panel width.

Experimental testing has demonstrated that curtain wall systems are typically able to withstand deformations greater than the initial glass to frame contact before glazing falls out (Behr, 2009; King and Lim, 1991; and Wright, 1989). This is due to cracking of the glass, localised crushing at the contact location and deformation of the framing.

The risk of curtain wall failure is adjusted using the element performance factor, Q, with the coefficients given in Table C10.1.

Table C10.1: Element performance factor, Q, for curtain walls (adapted from NZS 4223.1:2008 and Behr 2009)

Curtain Wall Description	Q factor
Modern commercial curtain wall system	4
Pre-1980s curtain wall or residential curtain wall system	2
Frameless glazing or rigid framing	1.25

C10.6.9 Ceilings

Earthquake scores for suspended ceilings will often be governed by the spacing of bracing elements and the capacity of connections, particularly those to the primary structure.

C10.6.9.1 Inspection

Common issues that are identified in seismic assessments of suspended ceilings include:

- insufficient bracing to the primary structure above (this does not always apply to perimeter fixed suspended ceiling systems)
- insufficient capacity of connections, particularly those to the primary structure
- interconnected partition walls with inadequate separation to primary structure causing unintended restraint within ceiling systems

- insufficient clearance to in-ceiling services, and
- heavy and unrestrained ceiling tiles.

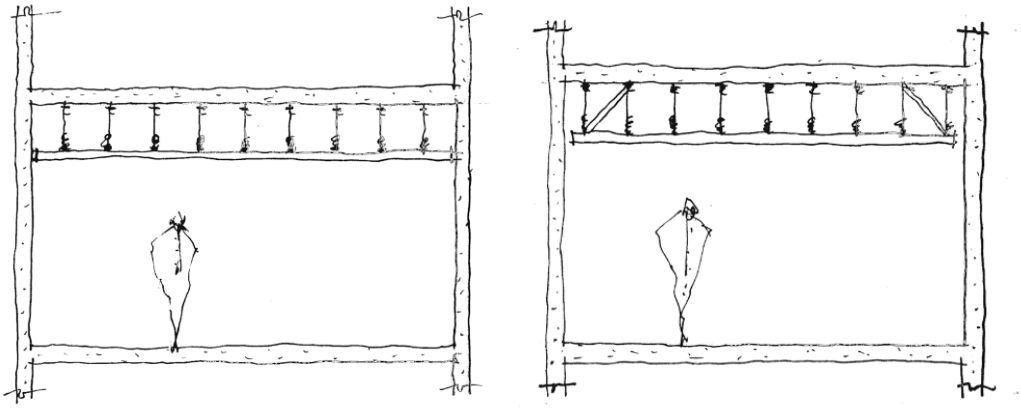
Common issues that are identified in seismic assessments of monolithic ceilings include the insufficient capacity of members and connections, particularly to the primary structure.

The assessment of ceilings should first consider whether the failure of the ceiling would create a significant life safety hazard if it were to fall, and that there is no practical mitigation available to those who would be affected by the ceiling falling (e.g. furniture under which to shelter). The assessment should aim to identify whether the following high-risk attributes are present:

- large, uninterrupted regions of ceiling with no (or very little) seismic restraint
- heavy ceiling tiles (e.g. vinyl-faced or plasterboard tiles weighing over 7.5 kg)
- partition walls fixed to the ceiling system
- building services supported directly by the ceiling
- very large plenum heights or ceiling heights
- braces installed at angles greater than 45 degrees
- no vertical stiffener present at diagonal braces.

C10.6.9.2 Force-based assessment

If a detailed assessment of the ceiling is deemed necessary, then a force-based approach is required to assess the capacity of ceiling systems. The assessment should identify what seismic restraint is provided (if any) in order to determine the component that will govern the ceiling capacity. Suspended ceilings are typically either perimeter fixed or back-braced, as shown in Figure C10.9.



(a) Perimeter fixed suspended ceiling **(b) Floating ceiling with back-braces**

Figure C10.9: Types of suspended ceiling (BRANZ, 2015)

The following steps outline the process for force-based assessment of a suspended ceiling:

Step 1

Inspect the ceiling to identify how it is restrained (perimeter fixed or back-braced) and to determine the maximum main and cross tee rail lengths in the ceiling system. Refer to

Figure C10.9 for typical ceiling restraint arrangements and to Figure C10.10 for typical back-bracing details.

Step 2

Determine the seismic weight of the ceiling per unit area, including the suspended services supported by the ceiling. In the absence of any material data, the seismic weight of the ceiling should be based on the weights provided in Table C10.2.

Step 3

Determine seismic demands per unit area of ceiling, assuming the ceiling is a part in accordance with Section C3.

Step 4

Determine seismic demands upon both the main and cross tees using the seismic demands per unit area from Step 3 and the spacing of the respective tee rails.

Step 5

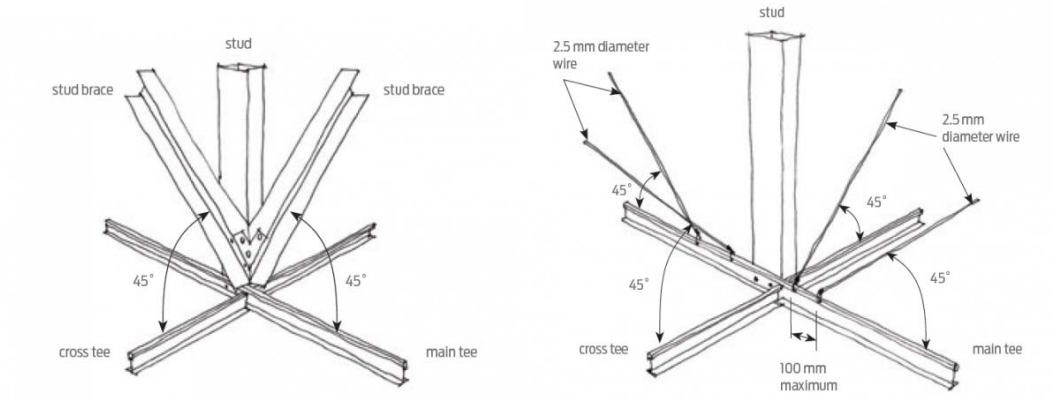
Calculate the theoretical maximum tee rail lengths by dividing the tee rail capacities by the seismic demands obtained in Step 4. In the absence of any product data, the tee tail capacities should be based on those provided in Table C10.3.

Step 6

Calculate the earthquake score by dividing the maximum theoretical tee rail lengths obtained in Step 5 by the actual lengths of tee rail identified in Step 1. If back-bracing is present, the distance between braces should be used in lieu of the actual tee rail lengths.

Step 7

If back-braces are present it is necessary to check the braces have adequate capacity for the area of ceiling they are restraining. In the absence of any product data, capacities of the components and their connections should be based on those provided in Table C10.3. For a back-braced system the earthquake score is calculated by dividing the brace capacity by the seismic demand of the area of ceiling being restrained.



(a) Strut bracing **(b) Wire bracing**

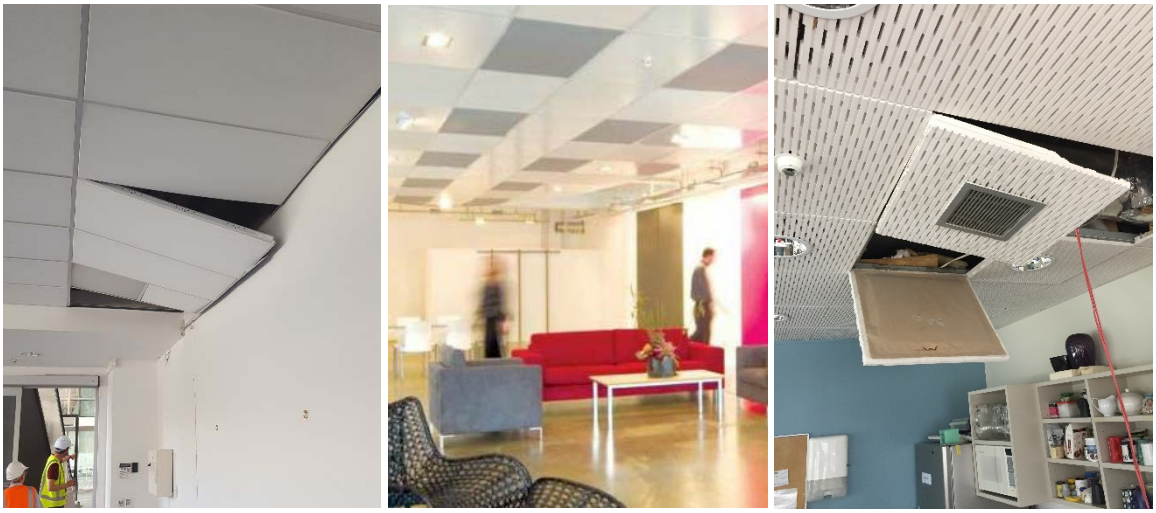
Figure C10.10: Back-braced suspended ceiling system (BRANZ, 2015)

Table C10.2: Typical ceiling weights

Ceiling type	Description	Weight (kg/m^2)
Light-weight suspended	Mineral fibre ceiling tiles, supporting typical suspended services	0.10
Medium-weight suspended	Vinyl-faced ceiling tiles, supporting typical suspended services	0.15
Heavy-weight suspended	Plaster ceiling tiles, or light-medium weight ceiling supporting heavy suspended services	0.20
Monolithic	Plasterboard lined steel stud, supporting typical suspended services	0.15

Table C10.3: Typical suspended ceiling component capacities (Armstrong, 2013)

Ceiling component	Capacity (kN)
Main tee	1.0
Cross tee	0.6
2.5 mm diameter wire bracing (at 45 degrees)	1.0



(a) Light-weight mineral fibre ceiling tile **(b) Medium-weight vinyl-faced ceiling tile** **© Heavy-weight plaster ceiling tile**

Figure C10.11: Illustrations of light, medium and heavy weight ceilings (BRANZ)

Note:

When calculating seismic demands, the NZS 1170.5:2004 part ductility demand, μ_p, can be taken as equal to 2.0 for suspended ceilings and equal to 3.0 for fixed ceilings, as per Table C8.2 in NZS 1170.5:2004.

The seismic weight of the suspended ceiling should include an allowance for a distributed service load of not less than 3 kg/m^2. This weight is included in Table C10.2 above.

In an Importance Level 4 building such as a hospital, it is recommended that under-braced suspended ceilings should be identified because the space beneath them would need to be evacuated, even though their failure would not pose a significant life safety hazard.

C10.6.9.3 Deformation-based assessment

Deformation-based assessments are only required when the suspended ceiling system is likely to impact the tops of non-structural walls that are only attached to the lower floor. This assessment will require consideration of:

- the deformation demand (refer to Section C10.4.5)
- the horizontal gap between the ceiling system and the wall
- the deformability and strength of the wall, and
- how any impacting would affect the ceiling system.

C10.6.10 Canopies, balconies, billboards and other appendages

The earthquake scores of canopies, balconies, billboards and appendages that are attached to buildings will often be governed by the capacity of their connections to the primary structure. Common issues that are identified in seismic assessments of these elements include:

- insufficient capacity of connections to the primary structure, and
- corrosion of hidden fixings.

Note:

Ratcheting should be considered on non-vertical cantilevering elements where this may affect the earthquake score of the element.

The assessment should begin by determining whether a force-based assessment, a deformation-based assessment, or both are applicable. Procedures for both assessment procedures are given below.

C10.6.10.1 Force-based assessment

The following steps outline the force-based assessment procedure for canopies, balconies, billboards and other appendages attached to buildings:

Step 1

Inspect the element to identify the properties of its structural members and how these members are connected to the building.

Step 2

Determine the seismic weight of the element.

Step 3

Determine seismic demands on the members, assuming the element is a part in accordance with Section C3.

Step 4

Calculate the theoretical maximum element capacities.

Step 5

Determine the earthquake score of each member and connection by dividing the capacity by the seismic demand

Step 6

Take the earthquake score for the element to be the lowest of the Step 5 scores.

C10.6.10.2 Deformation-based assessment

A deformation-based approach is required when it is necessary to assess the deformation capacity of the element and its connections. The lateral displacements of the primary structure derived from structural analysis can be used to determine the horizontal deformation demands on the element and its connections. The horizontal deformation demands should be taken as the drift between fixing points of the element. For full storey items, this will be equivalent to the inter-storey drift.

The deformations of the primary structure at the element connection points will need to be accommodated in the element, its connections, or both. Slotted or flexible connections are often provided in modern structures to accommodate relative deformations. The earthquake score can be evaluated as the total lateral deformation capacity of the element and connections, divided by the lateral demands calculated.

Note:

Where a slotted connection has been used, it should be checked to ensure that the slot can accommodate movement as intended.

In older structures, it is uncommon for connection movement to be accommodated. However, the element and its connections may be able to deform to accommodate some lateral deformation. In such cases, loads will be introduced into the element and the capacity the element and connections have to resist these loads is used to calculating the earthquake score. Potential failure mechanisms include failure of the connections (shearing of bolts, yielding of brackets, and pull-out of fixings) and shear/flexural damage to the element.

C10.7 Improving Seismic Performance

The process of conducting a DSA may identify potential seismic performance issues in building elements and their connections. These can be mitigated to improve their seismic performance.

Typical methods for improving seismic performance include:

- removing heavy elements such as URM parapets, URM chimneys, ceilings and heavy non-loadbearing partition walls
- improving cladding panel connections
- relocating heavy plant from high level installations to a basement or ground level
- replacing corroded bracing elements/connections
- adding supplementary bracing
- inserting joints into building elements that span across primary structure seismic joints
- improving stair bearing and sliding details
- internal post-tensioning, steel tube reinforcement, or concrete filling of URM chimneys
- adding supplementary connections and/or improving existing connections, and
- providing tethers to restrict the fall distance.

Note:

Tethers and other means of preventing a falling element becoming a significant life safety hazard need to be assessed on their ability to reliably arrest the element's fall. Refer to NZS 1170.5:2004 for guidance.

Some additional methods for improving the seismic performance of elements and their connections can be found in FEMA E-74, ASCE 41-06 (2006) and ASCE 41-13 (2014).

Note:

Some typical URM façade and URM parapet securing concepts have been published (MBIE 2017b). Designers may find these securing concepts useful when designing strengthening measures to improve the seismic performance of these elements.

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