



Low Damage Seismic Design

Volume 3: Technical Guidance
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RESOURCES



**Te Kāwanatanga
o Aotearoa**
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It is expected that this release of the LDS Guidelines will be updated as the field evolves and research, such as that identified in the Industry Research Needs appendix, is conducted and as user feedback is received.

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Executive Summary

Low Damage Seismic Design (LDSD) is a building design philosophy that achieves better than New Zealand Building Code minimum requirements. A key goal of LDSD is to deliver buildings that are less likely to be damaged and thereby limit disruption and losses in future earthquakes. This guidance document draws upon knowledge that has been developed both in New Zealand and internationally which can be used by project managers, design consultants, contractors and facilities management to design, construct and maintain seismically resilient buildings.

This document is Volume Three of a three-volume LDSD Guidance Series:

- Low Damage Seismic Design: Benefits, Options, and Getting Started (Volume One).
- Low Damage Seismic Design: Performance Framework (Volume Two).
- Low Damage Seismic Design: Technical Guidance (Volume Three).

Low Damage Seismic Design: Benefits, Options, and Getting Started (Volume One) is intended to help building owners and tenants decide if LDSD is right for their project. The document introduces the philosophy and benefits of LDSD.

Low Damage Seismic Design: Performance Framework (Volume Two) has been written for building owners, developers, tenants, project managers and design consultants. The document defines the recommended performance framework for LDSD projects and defines the LDSD outcome objectives and related performance goals.

Low Damage Seismic Design: Technical Guidance (Volume Three – this document) has been written for project managers, design consultants, contractors, and facilities management. The purpose of this document is to inform designers how to achieve LDSD to the agreed outcome objectives and post event performance goals defined in Volume Two. This document provides technical design criteria which can be used by design teams to demonstrate a building design meets the requirements for LDSD. Compliance with the New Zealand Building Code is also required.

LDSD emphasises thoughtful early decisions in design, including site selection, structural regularity, and compatibility of structural and non-structural components. Stiff, regular buildings on good ground have been shown to perform well in earthquakes when secondary and non-structural elements are adequately restrained. Importantly, LDSD does not always require novel seismic devices; conventional systems, when thoughtfully applied, can often meet LDSD performance goals.

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1 Introduction

Low Damage Seismic Design (LSDS) is a building design philosophy that achieves better than New Zealand Building Code (NZBC) minimum requirements. A key goal of LSDS is to deliver buildings that are less likely to be damaged and thereby limit disruption and losses in future earthquakes. This guidance document draws upon knowledge that has been developed both in New Zealand and internationally which can be used by project managers, design consultants, contractors and facilities management to design, construct and maintain seismically resilient buildings.

The adoption of LSDS on any project is voluntary. If building owners, developers and tenants choose not to adopt LSDS; many of the recommendations contained in this guidance document around regularity, design coordination and a documented briefing process between designers and clients are considered of value as good design practice.

1.1 Overview of Guidance Documents

This document is Volume Three of a three-volume LSDS Guidance Series:

- Low Damage Seismic Design: Benefits, Options, and Getting Started (Volume One).
- Low Damage Seismic Design: Performance Framework (Volume Two).
- **Low Damage Seismic Design: Technical Guidance (Volume Three).**

Low Damage Seismic Design: Benefits, Options, and Getting Started (Volume One) is intended to help building owners and tenants decide if LSDS is right for their project. The document introduces the philosophy and benefits of LSDS. It contains an explanation of key concepts and terms involved with LSDS and outlines the value of an LSDS approach to building design for building developers and owners. It also provides advice on how to start an LSDS project.

Low Damage Seismic Design: Performance Framework (Volume Two) has been written for building owners, developers, tenants, project managers and design consultants. The document defines the recommended performance framework for LSDS projects and how LSDS projects fit within the New Zealand building regulatory system. The performance framework also defines the LSDS outcome objectives and related performance goals.

Low Damage Seismic Design: Technical Guidance (Volume Three – this document) has been written for project managers, design consultants, contractors, and facilities management. It provides designers with a methodology to achieve the LSDS outcome objectives and post event performance goals.

1.2 Purpose and Scope

The purpose of this document is to inform designers how to achieve LSDS to the agreed Outcome Objectives and post event Performance Goals defined in Volume Two. This document provides technical design criteria which can be used by design teams to demonstrate a building design meets the requirements for LSDS. Compliance with the NZBC is also required.

The scope of this document is primarily aimed at new buildings; however, the concepts are also applicable to existing buildings.

1.3 Organisation and Content

This document is organised as follows:

- Section 1, *Introduction*, introduces LDSD and summarises some of the sustainability and downstream social benefits.
- Section 2, *How to Achieve LDSD*, provides an overview of the recommended process for achieving LDSD.
- Section 3, *Design and Briefing Process*, describes a recommended design and briefing processes clients and design teams can follow when delivering a LDSD project.
- Section 4, *Seismic Hazard*, describes how seismic hazards are to be determined, including minimum seismic demands to be used for the design of building components and for geotechnical assessments. This chapter also considers hazards associated with fault rupture, seismic slope instability and tsunamis.
- Section 5, *Design Requirements for LDSD Category Level 1 Buildings*, contains technical design criteria design teams can use to demonstrate a building design meets the requirements for LDSD Category Level 1 Buildings.
- Section 6, *Design Requirements for LDSD Category Level 2 and 3 Buildings* contains technical design criteria for the design of building components which can be used by design teams to demonstrate a building design meets the requirements for LDSD Category Level 2 and 3 Buildings.
- Section 7, *Procurement, Construction and Post Construction*, contains additional information to help ensure building designs are appropriately procured, constructed and maintained. This chapter also provides advice on building monitoring and post-event resilience and recovery.
- Appendices, include additional resources on the following:
 - i. Appendix A, *Definitions and Acronyms*.
 - ii. Appendix B, *Notation*.
 - iii. Appendix C, *Direct Performance Assessment*.
 - iv. Appendix D, *LDSD Primary Structural System Option Summaries*.
 - v. Appendix E, *Industry Research Needs*.
 - vi. Appendix F, *Downstream Social Benefits*.
 - vii. Appendix G, *US Functional Recovery Provisions*.
 - viii. Appendix H, *Factors Affecting Cost and Reliability of Seismic Qualification of Proprietary Services Equipment*.
 - ix. Appendix J, *Facade Testing Standards Information*.

1.4 Low Damage Seismic Design Overview

1.4.1 Background

The 2010-2011 Canterbury earthquake sequence caused significant damage to many modern buildings in Christchurch (Kam, Pampanin, & Elwood, 2011). More recently, the 2016 M_w 7.8 Kaikōura earthquake caused significant damage to numerous buildings in Wellington, despite the epicentre being some 200 km away to the southwest (Cubrinovski, et al., 2020). In the immediate aftermath of the 2016 Kaikōura earthquake about 11% (1,80,000 m²) of the city's office space was closed for assessment and many office buildings had to be demolished (Elwood, Filippova, Noy, & Paz, 2020).

In light of such events, industry has recognised that a life safety focus is insufficient to meet community expectations and communication about the likely performance of buildings in earthquakes should be improved (Kam, Pampanin, & Elwood, 2011, and NZSEE, 2024). Recognizing this, Engineering NZ and Structural Engineering Society New Zealand (SESOC), in partnership with the New Zealand Ministry of Business, Innovation and Employment (MBIE), established a LDSD project in 2019. This project built on ideas and concepts for LDSD that were circulating in New Zealand at the time (Hare, Oliver, & Galloway, 2012).

This three-volume LDSD Guidance Series, developed in partnership with MBIE, SESOC and the Natural Hazards Commission Toka Tū Ake (NHC), represents the culmination of the original 2019 LDSD project.

1.4.2 What is Low Damage Seismic Design?

LDSD is the careful design of seismically resilient buildings that are less likely to be damaged in future earthquakes. This is achieved primarily through good design practice by a well-coordinated design team. The advantage of LDSD is there is likely to be less disruption and losses to building owners and occupants in future earthquakes. Good design practice means conscious, thoughtful, decisions are made about the selection of building sites, typologies and components. These decisions are documented and clearly articulated across the owner, designers and contractors.

LDSD is the process to design buildings to have improved functionality and a low probability of damage after earthquakes.

Having early engagement with the client, design team, and in some cases early contractor involvement, to discuss issues such as structural regularity and system choice, expected building movements and compatibility with systems and finishes is essential to ensuring LDSD is successfully implemented on projects.

Observations of building performance in past earthquakes (Shiga, 1968), (Baird & Ferner, 2017), (Pettinga, Sarrafzadeh, & Elwood, 2019), (Lagos, et al., 2020) and (NHC, 2024) have consistently demonstrated well detailed, stiff, regular buildings on good ground perform well in earthquakes provided secondary and non-structural elements are adequately restrained. Limiting building displacements is an effective means to mitigate damage to the primary structure and *deformation sensitive* secondary and non-structural elements such as partition walls and façade components. While this approach does not necessarily mitigate damage to *acceleration sensitive* components, these observations confirm that, except for very high levels of seismic ground shaking, satisfactory performance of acceleration sensitive components can generally be expected if they are appropriately restrained.

Low Damage Seismic Design: Performance Framework (Volume Two), describes three different LDSD categories are provided for in the LDSD guidelines - Level 1, Level 2 and Level 3.

LDSD Category Level 1 does not have any specific damage control performance goals. Instead, it relies on 'thoughtful' choices and good design practice such as:

- Regularity of primary structure.
- More restrictive ULS drift limits.
- Capacity design requirements for primary structure.
- Consideration of latest seismic hazard knowledge as defined in TS 1170.5 (SNZ, 2025).
- Adoption of the BIP Code of Practice (BIP, 2025) for the design of non-structural elements.
- Construction monitoring requirements for both structural and non-structural elements.

LDSD Category Level 2 and 3 buildings have specific damage control performance goals. These goals correspond to limiting the Expected Annual Loss (EAL) of buildings and building elements; and limiting the probability, in a 10-year period of an earthquake causing building damage that exceeds 5% of the building replacement cost.

It is important to note in many cases it will not be necessary to include novel anti-seismic devices into structural systems (i.e. seismic isolators or supplemental dampers) to achieve LDSD performance goals. LDSD performance goals can often be achieved using conventional structural systems, applied in a thoughtful way, in regular, well-coordinated buildings. An exception to this is buildings located in areas of very high seismicity (i.e. Wellington or Wairarapa) where novel anti-seismic devices might be needed to meet the higher performance requirements for LDSD Category Level 3. Appendix D contains further advice on structural systems that could be considered for LDSD projects.

1.4.3 Drift Sensitive vs Acceleration Sensitive Components

A key LDSD concept that all members of the project team should be familiar with is the difference between drift versus acceleration sensitive components.

Drift sensitive components are those components that are most adversely affected by the change of shape of a building when it moves out of plumb during an earthquake (refer Figure 1-1). An example of a drift sensitive component is a partition wall which is built plumb and level with large sheet materials that can suffer damage when they are pushed out of square.

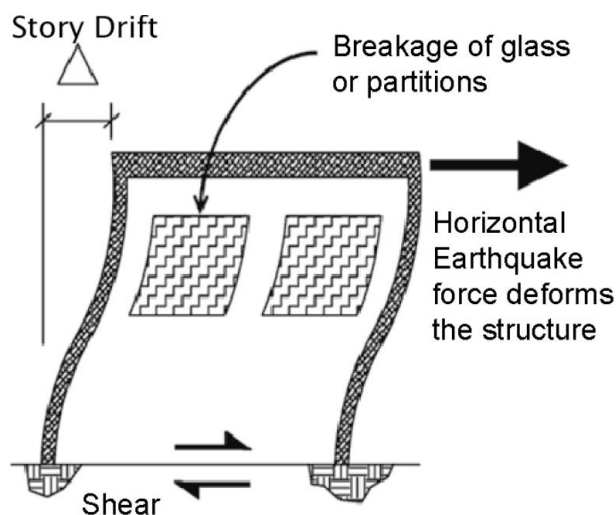


Figure 1-1 Damage to drift sensitive components due to building deformation (FEMA, 2011)

Acceleration sensitive components are those building elements that are most adversely affected by inertia forces resulting from horizontal and vertical floor accelerations induced in a building during an earthquake. An example of an acceleration sensitive component is a pipe or cable ladder that, by virtue of the momentum of its own

weight, results in horizontal forces on the component which may cause damage to itself, its connection to the building or other items close to the component.

This document promotes the approach of limiting building drifts as the primary means to limit damage to building components during earthquakes. An outcome of designing stiff buildings to limit building drift is that these buildings will typically experience higher floor accelerations when compared with equivalent more flexible buildings. Acceleration sensitive components will therefore experience larger seismic inertia forces, and this may require larger, or more braces and anchors to restrain them. Stiff buildings might also require more seismically resilient plant and equipment, with higher capacities to withstand the larger inertia forces without damage or loss of functionality. In higher seismic zones, seismic qualification requirements for NSEs might also be triggered by stiffer buildings (refer Section 6.6.2.14).

1.4.4 How does this relate to %NBS Ratings?

The percentage of New Building Standard (%NBS) is a rating index used to characterise the expected seismic response of an existing building to earthquake shaking. It is used to identify existing buildings that represent a higher seismic risk than a similar new building, built to the minimum life safety requirements of the New Zealand Building Code (or New Building Standard) (NZSEE, 2017).

The %NBS rating was specifically developed to support the implementation of the earthquake-prone building legislation. The legislation seeks to quantify the seismic performance of existing buildings in relation to an equivalent new building, and a simple metric was needed to classify buildings.

However, %NBS ratings do not provide a good measure of how vulnerable a building is to sustaining damage during an earthquake. Nor do they provide an accurate indicator of the length of time that may be required to return a building back to an operational state following an earthquake. %NBS ratings only consider building performance in relation to protecting people's lives.

1.5 Related Considerations

1.5.1 Embodied Carbon

Intuitively designing stiffer buildings to reduce the likelihood of damage occurring in future earthquakes, and thereby avoiding further carbon emissions tied to the repair and or replacement as a result of this damage (i.e. seismic carbon), might be considered a logical solution to reduce whole-of-life carbon emissions. However, these future savings are only realised if an earthquake occurs, and even then, the quantity and impact of these emissions are uncertain. In contrast, the carbon we expend in construction today is a certainty. Reducing carbon emissions in the present decade over possible emissions in the future has been argued as our best bet to mitigate the effects of climate change (ARUP, 2024), and this should therefore be considered when assessing the additional upfront carbon costs required for LDSD.

In striving for improved seismic resilience, we should be careful to not create excessively up-front carbon intensive buildings but rather look to carefully assess and balance guaranteed up-front carbon impacts with any potential reductions in future seismically linked carbon emissions.

Project teams should explicitly consider and quantify the impact different material selections and structural system options have on the embodied carbon cost of projects (both upfront carbon and whole-of-life carbon emissions) throughout the design process. The carbon intensity of our main materials is expected to reduce over the coming decades, so arguably they are at their highest today. Consequently, design teams should limit the use of carbon intensive materials when they can, and when required, prioritise their use to those building components which have the most impact on improving the seismic performance of the building and thereby reduce seismic carbon risks.

More research to understand the relationship between design characteristics and seismic carbon risk across a range of building forms, structural systems, and hazard zones is in progress . Until the outcomes of this research become available project teams should evaluate the short- and long-term carbon impacts associated with significant design decisions and look for opportunities to reduce up-front carbon costs. Further information on designing with low carbon as a driver can be found in SESOC Low Carbon Top-Tips Guide (SESOC, 2024).

1.5.2 Downstream Social Benefits

LDSD buildings provide several direct and indirect benefits and opportunities. The benefits will resonate differently depending on the use of the building and the building owner's long-term interest in the property. Key benefits for building developers, owners and their tenants are:

- Increased certainty of performance.
- Increased property value.
- Reduced losses.
- Improved insurability.
- Reduced stress.
- Supporting community resilience.

For further information on downstream social benefits of LDSD refer to Appendix F and Volume One.

1.6 US Functional Recovery Provisions

The United States Federal Emergency Management Agency (FEMA) is developing similar functional recovery guidance for increasing the seismic resilience of buildings. A summary of the new US functional recovery provisions and how they relate to LDSD has been included in Appendix G as it may be of interest to New Zealand designers.

2 How to Achieve LDSD

2.1 LDSD Performance Framework

Volume 2 defines the LDSD Performance Framework. This framework is represented by outcome objectives, performance goals and physical states and is summarised below. Refer to Volume 2 for more detailed information about the LDSD Performance Framework.

2.1.1 LDSD Outcome Objectives

Outcome objectives for LDSD buildings are:

- *Life Safety*, a low probability of loss of life or significant injuries.
- *Damage Control (Building Asset Protection)*, a low probability of damage leading to significant economic loss.
- *Improved Functionality*, by reducing the probability of damage, the time to return to functionality is likely to be significantly lower than a minimum code compliant building.

The Building Code addresses the life safety outcome objective through verification methods or acceptable solutions, by means of an Ultimate Limit State (ULS). The damage outcome objective is the primary focus of LDSD and is addressed through the introduction of a Damage Control Limit State (DCLS).

The Improved Functionality outcome objective is articulated through Operational States. As is noted in Volume 2, in significant earthquakes, disruption and return to function can often be controlled by aspects beyond the building site, such as cordons, utility outages, and labour shortages.

2.1.2 Damage Control Performance Goals

Three LDSD building categories are provided for in Volume 1 and 2: Level 1, Level 2 and Level 3. LDSD Category Level 2 and 3 buildings require consideration of a new DCLS. In contrast, LDSD Category Level 1 buildings do not require consideration of the new DCLS. While LDSD Category Level 1 buildings do not have specified damage control performance goals, these buildings have additional design considerations which are expected to significantly improve their seismic performance when compared to code minimum building designs.

Performance goals proposed in relation to damage control for LDSD Category Level 2 and 3 buildings correspond to limiting the Expected Annual Loss (EAL) of buildings and building elements and limiting the probability of building damage exceeding 5% of the building replacement cost in an earthquake in a 10-year period. The limiting values targeted for LDSD Category Level 2 and 3 buildings are detailed in Volume 2.

2.1.3 Improved Functionality Performance Goal

Three Operational State Categories are defined in Volume 2:

- *OS-I Shelter in Place*, a building for which post-earthquake functionality has been significantly reduced. Building services may be damaged and unavailable until necessary repairs are completed. The structure has not sustained safety-critical structural damage, and the building is able to withstand another major earthquake.
- *OS-II Partial Functionality*, a building for which post-earthquake structural and non-structural damage is limited to the extent that the basic intended functions of the building's pre-earthquake use are maintained or can be restored within an acceptable time (usually measured in days rather than weeks).

- *OS-III Full Functionality*, a building for which post-earthquake structural and non-structural damage is limited to the extent that the intended functions of the building's pre-earthquake use are maintained or can be restored within an acceptable time (usually measured in minutes to hours rather than days).

Reducing the time to return a building to an operational state is enhanced by reducing the probability of damage. LDSD Category Level 2 and 3 buildings are considered to have a significantly higher likelihood of achieving Operational State II - Partial Functionality following a significant earthquake than an equivalent, minimum code compliant building. However, it is acknowledged that wider impacts beyond the site, or damage to critical secondary or non-structural elements in larger earthquakes may lead to increased reoccupation timeframes.

Critical non-structural elements and systems necessary for an operational state should be identified by the project team. A mitigation strategy for each of these critical elements and systems should be established to enable the specified operational state to be returned within the acceptable timeframe.

2.1.4 Physical States

Volume 2 defines physical states that are intended to be the link between the LDSD performance goals and the technical design criteria which are defined in this document. The physical states describe the physical condition and extent of tolerable damage to building components corresponding to the DCLS level of ground shaking.

2.2 Design Process Overview

The recommended LDSD design process is summarised in Figure 2-1 below. As can be seen in Figure 2-1 it is recommended project teams identify if LDSD is a project goal during the project establishment phase. This is because LDSD can have a significant impact on site selection, building form and the design of structural and non-structural elements. Delaying the decision to adopt LDSD on a project can have significant cost and programme implications.



Figure 2-1 Recommended LDSD design process

Section 3 of this document contains detail on a recommended LDS Design and Briefing Process. There are several important steps and considerations to ensure good project outcomes. These include:

- Early client discussions to prompt appropriate consideration of LDS.
- The importance of site selection.
- Requirements for consultants and their scopes of service.
- Briefing process leading to the LDS Brief and how this is to be complied with, including the key design considerations at each milestone and during construction.

In order for a building to be considered a LDS building there are a number of design and construction requirements, and also some processes, that need to be followed to give clients, tenants and purchasers confidence that LDS has been achieved. These are defined in Table 2-1 below. Note that for small scale or simple projects some of these requirements may be simplified or combined, with the agreement of the LDS Reviewer.

Table 2-1 Demonstrating achievement of LDSD

LDSD Item	LDSD Category ¹	
	L1	L2 & L3
1. Project Establishment		
Initial Briefing process carried out with the client leading to an agreed High-Level Brief (3.1.3.1)		
Get appropriate consultant advice for site selection if the site is not fixed (3.1.3.2)		
Carry out assessment of site geotechnical state, incl. fault proximity (3.1.3.2)		
Ensure LDSD is included in all consultant commissioning scopes (3.1.4.2)		
Appoint Non-structural Seismic Designer (3.1.4.3)		
Identify LDSD Lead for the project (3.1.4.2)		
Appoint Independent LDSD Reviewer (3.1.4.4)		
Consider Early Contractor Involvement (ECI) for specialist seismic technologies (7.1)		
2. Concept Design		
LDSD Brief developed with client and LDSD Category Level agreed on (3.2.1)		
Test multiple strategies for foundations and structure (3.2.2)		
Multi discipline LDSD workshops (also refer NSE COP)		
Building form is simple and regular (3.2.2.1)		
Building Movement Strategy Report commenced (3.2.2.3)		
Major plant locations have been considered for LDSD aspects (3.2.2.1)		
LDSD Milestone Review (3.6.3.1)		
3. Preliminary Design		
Design Criteria agreed (3.3.1)		
Specific LDSD Section in consultants' Design Features Reports (DFRs)		
Specific LDSD Consultant Coordination workshops		
Building Movement Strategy Report updated		
Early Contractor Involvement including for proprietary seismic devices where applicable		
LDSD Milestone Review (3.6.3.1)		
4. Developed/Detailed Design		
Specific LDSD Section in consultants' DFRs		
Specific LDSD Consultant Coordination workshops		
Agreement on Design Methodology for each Building Component (2.3 or 2.4)		
Updated Building Movement Strategy Report		
Specific consideration of inspection and repair		
LDSD Milestone Review (3.6.3.1)		
Detailed LDSD Peer Review (3.6.3.2)		
5. Procurement		
Tender documents include specific information for LDSD (7.2)		
Selection process includes LDSD attributes	N/A	
LDSD Milestone Review (3.6.3.1)		
6. Construction		
Contractor team includes a NSE Seismic Coordinator (7.2)		
Contractor and Subcontractors demonstrate compliance with LDSD requirements (7.3)	N/A	
Consultant observation team review compliance with LDSD requirements (7.3)		
O&M Manuals include LDSD Information (7.4)		
LDSD Milestone Review (3.6.3.1)		

LDSD Item	LDSD Category ¹	
	L1	L2 & L3
7. Post Construction		
Building Tenant Design Guide (7.4)		
Building Re-Occupation Strategy Report (7.6.1)		
LDSD Milestone Review (3.6.3.1)		

Notes:

1. Orange indicates a requirement and grey indicates a recommendation

Table 2-1 should be used by the Independent LDSD Reviewer when completing the Milestone LDSD Reviewers and the Detailed LDSD Review (refer Section 3.6.3.2).

2.3 Design Methodology for LDSD Category Level 1 Buildings

LDSD Category Level 1 buildings do not have specified damage control performance goals defined in Volume 2. Instead, design requirements for LDSD Category Level 1 buildings have been established to ensure these buildings behave in a predictable, desirable manner when subject to earthquake shaking by applying fundamentally good engineering practice. Chapter 5 details the recommended design methodology for LDSD Category Level 1 buildings.

2.4 Design Methodology for LDSD Category Level 2 and 3 Buildings

Three verification pathways are available to demonstrate a building meets the outcome objectives and post event performance goals for LDSD Category Level 2 and 3 Buildings:

1. Prescriptive Design Method;
2. Direct Design Method; and
3. Direct Assessment Method

These verification pathways are summarised below. For a more detailed description refer Volume 2.

2.4.1 Prescriptive Design Method

The Prescriptive Design Method is limited to buildings which meet certain regularity restrictions or building components or systems where expected performance is reasonably reliable, and the design method is covered by the Building Code generally through an acceptable solution or verification method, or by guidance issued under Section 175 of the Building Act.

It is envisaged the Prescriptive Design Method will be applicable to most buildings. For buildings using this pathway, the LDSD design process is similar to the current design process in the Building Code system via cited design standards, albeit there is an increased requirement for design collaboration and reporting. Provided the prescribed design parameters and/or methods are used, and the type of system or component fits within the scope, then the Prescriptive Design Method is deemed to satisfy the LDSD provisions, provided the process, reporting and documentation expectations outlined in this document are satisfied.

2.4.2 Direct Design Method

The Direct Design Method pathway is used for building systems or components that don't meet the requirements of Prescriptive Design Method pathway. It is for those building systems or components that are considered innovative, new or alternative solutions via the Building Code system.

Demonstrating compliance using this method will require the design team to follow recognised analytical methods, or experimental testing in accordance with recognised standard testing procedures, to validate the

performance of potentially affected building systems or components will not compromise the LDSD performance goals detailed in Volume 2.

Commentary

An example situation when the Direct Design Method pathway might be adopted by a design team would be for the case when a DCLS inter-storey drift limit of 0.75% is adopted for a project. This exceeds the 0.5% inter-storey drift limit prescribed in the Prescriptive Design Method pathway. The Direct Design Method pathway could be used to validate adequate performance of drift sensitive components such as partition walls, façade systems, passive fire systems etc. This could include experimental testing of proposed 'improved' partition wall detailing to demonstrate that damage sustained by partition walls at 0.75% inter-storey drift met the Physical States damage criteria detailed in Volume 2.

In a similar way, all other building elements and components would be required to sustain the 0.75% inter-storey drift while meeting the Physical States damage criteria. However, other unaffected building components (such as those that are sensitive to floor acceleration and not storey drift demands, or structural elements in which damage is limited through ductility checks rather than storey drift) could still be design using the Prescriptive Design Method pathway.

2.4.3 Direct Assessment Method

The Direct Assessment Method pathway is an explicit assessment method that enables design teams to demonstrate compliance with the LDSD performance goals detailed in Volume 2. Design teams should use recognised performance assessment procedures such as the FEMA P58 (FEMA, 2018) approach. Fragility and loss functions required for the Direct Assessment Method pathway should be adopted considering New Zealand construction practice. Further guidance on this pathway is provided in Appendix C.

3 Design and Briefing Process

3.1 Project Establishment

3.1.1 General

LDSD requires a comprehensive approach to a project ranging from early site selection through to post-occupancy preparedness for a seismic event. It is important to ensure clients understand the value of getting appropriate early advice during a project because it can be very difficult to meet the necessary LDSD requirements later in the design process.

The following material may seem quite involved because it is required to cover a wide range of project sizes and complexities, but it should be reasonably straightforward to apply the guidance to smaller and simpler projects by making sensible decisions on the appropriate way to achieve the same outcomes. For example, certain roles can be combined to a single consultant, or reports may be simple memos.

3.1.2 Developing the Client's Expectations for the Building

It is often an architect or project manager, rather than an engineer, who is the first consultant approached by a client. Whoever is engaged early on can play a key role in briefing and site selection. The first step is to discuss overall expectations for the building. Does the client understand what the Building Code requires or the difference between operational continuity and low damage? What does the building owner or tenant expect to happen after an earthquake? How soon do they want to re-occupy the building? How much might they be willing to spend on earthquake damage repairs? Are there valuable contents that might need to be protected? Does the client understand there may be damage to the site beyond the building and beyond the site that could impact functionality and what are they and/or the building owner or tenant willing to accept? Are they willing to accept loss of functionality of the building because of damage to lifeline utilities beyond the site or loss of access to the site?

3.1.3 Critical Early Decisions

Section 3.1.4.1 describes the recommended LDSD briefing process, however in many cases it may not be possible to implement this process before initial design work begins. This can happen when investigating the feasibility of developing a site, or evaluating several sites, for a variety of reasons such as, uncertainty of tenants and their needs, availability of resources to test initial briefing requirements (time, financial, personnel etc) or uncertainty that such a process will be beneficial. There are nevertheless some critical early decisions that can lead to good outcomes.

3.1.3.1 Briefing: High Level Brief

Volume One of this LDSD Guidance Series includes a LDSD Project Brief Sheet that is an excellent starting point for initial client discussions. Table 3-1 below contains some examples of supplementary questions and supporting commentary to illustrate how the LDSD Project Brief Sheet can be used. Agreement on critical aspects of the project should happen before site evaluation or design work commences.

Table 3-1 Discussion Points to assist with establishing the High-Level Brief

Example Primary questions	Example Secondary questions	Commentary
Building Use		
What is the building to be used for?	What is its primary use?	Note that the use may change over the building lifetime so consider futureproofing for this.
	What is its secondary use?	A requirement for business-critical activities after an earthquake may trigger a LDSD requirement.
Will the building have some form of post-disaster function?	If so, is this: a regulatory requirement (e.g. the building will be an emergency service facility or has been designated as an essential facility); or	Will trigger Building Code IL4 designation. This has a focus on operational continuity but could be supplemented by additional LDSD requirements.
	Is it desirable (e.g. shelter in place for residential use)?	May trigger LDSD or a requirement for operation continuity of critical functions/partial functionality Refer to LDSD Volume Two for information on building operational states.
Building Location		
What type of event do you wish to limit damage or maintain operational continuity for?	Such as: Earthquake Tsunami Windstorm Flood Landslide	May trigger a bespoke response. May lead to discussions on site selection if a site is vulnerable to these issues.
How reliable are the regional lifeline utilities?	Water Wastewater Transport Energy Telecommunications	May lead to discussions on site selection or requirements for onsite generators, water storage etc.
What level of performance is expected outside the building footprint?	Will the building be able to be accessed post-event?	Achieving LDSD beneath the building footprint may be the primary focus but beyond the building footprint there may be additional requirements to achieve the desired functionality outcomes.
Safety and Wellbeing		
How important is it to show occupants they are safe?	Will having a building with enhanced safety/lower damage be attractive to future tenants and employees?	Presents the potential for a marketing value-add.
Business Continuity		
If you lost the building, could you move /replicate operations to another geographic location?	If not, what would you do?	The ability to move operations may make individual buildings less critical. If operations can't be moved, it is more likely LDSD might be appropriate.
How long could you cope with reduced or no operation?	What does reduced operation mean? After what scenario event might this be acceptable? Can the reduced function be segregated into a discrete part of the building or is it spread throughout? How quickly should the full function be restored?	Post-earthquake repair times for non-LDSD buildings can often take many months or more. If disruption to operation could only be tolerated for days or weeks, this may trigger a LDSD requirement.
Building Management		
How is the building managed (e.g. is there a facilities manager, or body corporate structure etc)?	Will it be possible to establish a process for ensuring that building additions and alterations adhere to the LDSD requirements?	This helps make it clear that LDSD requires an ongoing commitment from the building owner and manager.

Building Priorities		
How soon do you want to be able to occupy your building after a large earthquake?	Hours, days, months? Or is there no reoccupation time requirement?	Relatively rapid reoccupation (within days rather than months) may trigger a LDS requirement.
Building Contents		
Are there particularly hazardous or valuable contents?	If hazardous: What is the associated risk? What containment measures are taken?	Hazardous contents may trigger a LDS requirement. Possibly bespoke and specific to the hazardous content area.
	If valuable: How vulnerable are the contents to earthquake damage? What protection is available?	Valuable contents may trigger a LDS requirement. Possibly bespoke and specific to the valuable content area.
Commercial Considerations		
Is there a significant business partner? If so, should they also have input into this process?	Bank/Lender? Major tenant? Insurer?	Other parties' interests may trigger a LDS requirement.
What is the insurance approach for the building?	Possibility of insurance market requiring higher levels of seismic performance in the future. Possibility of self-insurance?	May trigger a LDS requirement or opportunity.
Design and Leadership		
Are architectural style and flair important to your project?	Are the architect and client receptive to a regular building form?	Aside from having a well-designed and regular form, LDS does not have to limit architectural opportunities.

If some of these questions are unable to be answered, this may inhibit later aspects of LDS. For example, it can be a major change to a project to add base isolation to the project brief because of the requirements to form the base isolation plane and rattle space. One way around this is to explore some of these options in early design studies.

3.1.3.2 The Importance of Site Selection

For some projects the site has already been determined before design consultants are appointed, but in many cases a building developer or client may be considering some alternative sites. Similarly, masterplans may be assessing a number of possible building locations within an existing site.

Good site selection can be fundamental to achieving LDS. For some poor sites it may not be practicable to achieve LDS because of high costs associated with mitigating seismic hazards that affect the site. It is therefore important to ensure that the appropriate specialist engineers, in particular a geotechnical engineer, are part of the core design team at an early stage. This is to ensure site specific hazards are understood and appropriately considered. It is also important to consider other resilience factors that may affect the site and so civil or services engineering input may also be required.

Items to consider as part of site selection include:

- What seismic geotechnical hazards – including slope stability, liquefaction, lateral spreading, weak ground or fault ruptures – might constrain site development or influence project costs? For example, at a waterfront site does liquefaction and lateral spreading originating outside the site boundary constrain the potential for LDS?
- What seismic hazards might originate beyond the site, such as tsunamis, slope instability and rockfall, and constrain the potential for LDS?

- Is the site in an area of high seismicity such that significant non-linear soil/rock response is anticipated, and more detailed site assessment might be required? Section 4 provides further detail on considerations at high levels of shaking.
- What is the seismic resilience of access and services to the site? Do these meet the high-level brief for the development? Note the seismic resilience of incoming services infrastructure is unlikely to be accurately quantified, consequently only a high-level qualitative assessment based on judgement may be possible.
- What hazard do neighbouring sites or buildings present to the site at a high level (i.e. without detailed assessment of neighbouring sites)? For example, does the age of an immediate neighbouring building indicate a potential higher risk of collapsing onto the site during a significant earthquake? Or what is the risk that damage to neighbouring buildings could limit access and business continuity in the area?
- In a master planning situation, for example on a campus site, are critical services located in less resilient buildings? In this case, advice from a services engineer would be relevant.
- Is there any site contamination? This could affect the decision to excavate a basement which is sometimes required to form a base isolation plane.
- Are there any features within the site, such as existing buildings or retaining walls, that could pose a hazard and therefore require survey or structural assessment? Will the site beyond the building footprint meet performance requirements for the building to achieve LDSD?

Consider how these hazards could be mitigated to achieve LDSD as well as the possible constraints they could impose on the development. Mitigation could include considering alternative sites. Comparison can help inform an understanding of the relative shortcomings of a site. Also consider the constraints these hazards might impose on the site layout or building form, and how these issues compare at any alternative sites.

3.1.3.3 Bulk and Location

Bulk and location studies are often undertaken during the feasibility phase of a project. The overall form of the building is fundamental to LDSD. While the basic building form is typically designed by the architect, it is recommended advice be sought from a structural engineer as a design is developed, or at the very least the information in Appendix D on structural system considerations be considered. This is because regular form can make a big difference to the seismic performance of a building, and it is important the client and architect consider this as part of their early decision making.

In many cases the form of a building is determined by trying to maximise the developable area on a site, especially during the early feasibility stage of a project. The architect may need to communicate to the client that this may not be the best strategy for LDSD. A reduction in area, for example to accommodate a rattle space for a base isolation system or to provide a wider set back to a neighbouring building, may be a better strategy for improved seismic resilience. Refer to Section 3.2.2.1 for more information on building form.

If the project is part of a campus or master planned development, then site-specific hazards such as proximity to other buildings or localised geotechnical conditions, will also be factors that should be considered when locating and shaping the building.

3.1.3.4 Early Consultant Advice

During the early establishment and feasibility stages of a project the client should be encouraged to engage the right consultants at the right time to ensure LDSD can be successfully implemented in a cost-effective manner – ideally the consultants should be engaged as early as possible. For example, an electrical engineer may need to be commissioned early to assess the existing electrical infrastructure on a site. This input is to avoid having to make fundamental changes to the design of the building during later stages of the design process which, referring to Figure 3-1, can have significant implications on project costs and schedules.

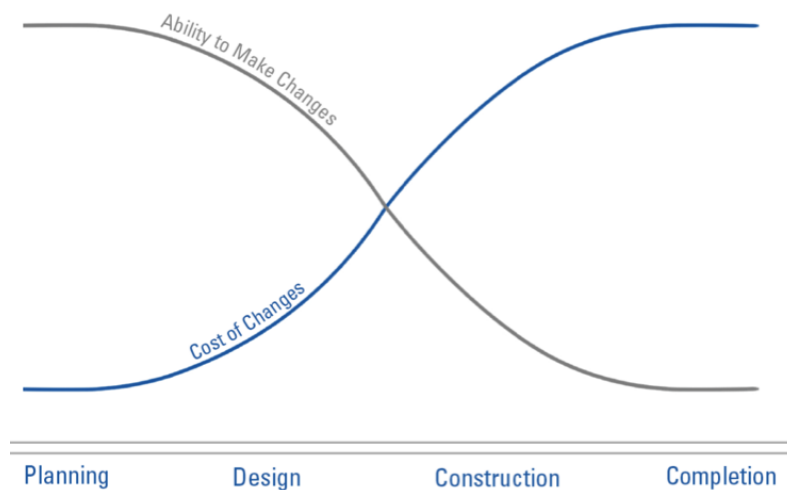


Figure 3-1 The relationship between the cost of changes to a design and the ease of ability to make these changes (A.W. Hooker, 2015).

3.1.4 Setting the Project Up to Achieve Low Damage Seismic Design

Once the client has confirmed that they want to adopt LDS for their project it is important to take the time to set the project up well. As well as implementing a good briefing process and sourcing detailed site (or existing building/services) information, the appropriate consultants should be engaged early with a clear understanding of how their roles and responsibilities will be affected by the LDS objectives of the project. This includes consideration of the roles of specialist Non-Structural Element Seismic Designers and Peer Reviewers. The Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025) has a design responsibilities matrix which is a useful starting point for this aspect.

Depending on the construction procurement strategy adopted for the project, there may also be an opportunity for early input from relevant contractors and sub-contractors. Clarity about their design roles and responsibilities is also required.

The client should be informed about the proposed deliverables, for example the contents of a LDS Brief or Design Features Reports (DFRs), to enable them to understand the LDS aspects of the project and then approve these at each design milestone as the project progresses.

3.1.4.1 Briefing: Getting it Right

It is important for the client and the design team to be clear about how the briefing process is intended to translate the client's overall objectives for the building into detailed technical design criteria which can be used by the design consultants for the project. As illustrated in Figure 3-2 this process typically happens in three general steps across the early design phases of a project.



Figure 3-2: Evolution from broad project goals (from the client) to technical design criteria (from and for the consultants)

As the design progresses from the concept phase, the design team will use the agreed LDSD Performance Goals to develop detailed LDSD criteria which will be used in subsequent design phases of the project. For example, at the beginning of the concept phase there will be the general aspiration to reduce damage after an earthquake, but by the end of preliminary design, the design team will have agreed technical design criteria for building inter-storey drifts, floor accelerations, seismic verification of proprietary equipment and other aspects of the building design.

A reflected or iterative briefing process is a key tool in this translation from client goals (which are often subjective and qualitative) into quantitative technical design criteria that can be used by the consultants in the design.

Table 3-2 below summarises how the process evolves from the initial high-level briefing at the project establishment phase, to the development and documentation of technical design criteria and associated reporting to keep the client and consultants informed. There is some flexibility between stages depending on specific project scale and circumstances. Note that the client's budget can also be a reason for an iterative briefing and design process.

For smaller scale and simple projects, the required documentation can be quite brief providing it addresses and records the relevant material.

Table 3-2 Low Damage Seismic Design Briefing Process

	Project Phase			
	Project Establishment	Concept Design	Preliminary Design	Developed and Detailed Design
Briefing Requirement	High Level LDSD Brief.	LDSD Project Brief.	Coordinated consultant response to the LDSD Brief.	Design continues to respond to the brief.
Briefing Process	Series of prompting questions to the client to determine the high-level objectives.	An iterative process where the LDSD requirements are gone into in more detail and the implications of these requirements are explored via concept design options and discussions with the client where the more technical aspects of the brief are reflected back to them.	An iterative process where the consultant responses to the LDSD Brief are worked through in a coordinated manner and to a greater level of detail. Client to also be involved as key decision maker.	Some aspects of the design process may reveal situations where consensus-based solutions are required e.g. damage limitation vs acoustic performance, or cost. These should be discussed/recorded for client information.
Briefing Deliverable	Short summary of high-level requirements to enable early feasibility/ site selection work to commence. Generally qualitative.	LDSD Brief with agreed Outcome Objectives and related Performance Goals. Ideally a client adopts standard LDSD Category Levels 1,2 or 3. There can be additional bespoke elements if required. Report detailing the proposed Building Movement Strategy ¹ .	LDSD Section in each of the Consultant DFRs in response to the LDSD Brief, with agreed technical design criteria for building drifts, acceleration and other aspects of the building design. Updated Building Movement Strategy Report.	LDSD Section in each consultant DFR and a Building Movement Strategy Report. This should report on compliance with the LDSD Brief or departures from it (with explanation); it will also report on the granular detail briefing that comes out of the Developed Design.
How is the brief used in the design process?	Allows site selection, and high-level bulk and location studies to be informed.	Used during the concept design phase to test design options. Allows Preliminary Design to commence with appropriate level of information and certainty.	LDSD Brief used during Preliminary Design and subsequently revised depending on design outcomes. LDSD Section in DFRs allows Developed Design to commence.	Consultant LDSD Reporting is required to demonstrate compliance with LDSD objectives and technical design criteria adopted for the project. Also useful for contractor design items and ultimately serves as a repository for information post construction.

Notes:

1. A specific Building Movement Strategy Report is required in addition to the consultant Design Features Reports (DFRs) from concept design onwards to ensure good coordination and to report on any compromises (refer to Section 3.2.2.3).

3.1.4.2 Encouraging Good Collaboration

Seismic damage often occurs where building components immediately interface or are close together, so good coordination and collaboration between all members of the design and construction teams at all stages of the project is very important. The aim should be to work together holistically to minimise design and coordination conflicts.

The Code of Practice for the Seismic Performance of Non-structural Elements (BIP, 2025) provides guidance for design consultants, contractors and subcontractors on the design, coordination and construction of non-structural elements (NSEs). This code of practice details a recommended process design teams and contractors can use to design and coordinate NSEs and the code of practice should be adopted for LDSD projects.

Leadership of this process and ensuring collaboration could sit with the architect (typically the lead consultant) but could also be led by a design manager, structural engineer or non-structural seismic designer. In any case a LDSD 'Champion' or Lead should be identified early during the project establishment phase.

Ways to ensure good collaboration will be dependent on the project scale and complexity but include:

- Using the NZCIC Guidelines (NZCIC, 2023) as a starting point to make sure that design work associated with LDSD is included in the consultants' scopes of services. The Code of Practice for the Seismic Performance of Non-structural Elements has detail that expands on the NZCIC Guidelines, as well as material on the demarcation of the design roles and responsibilities of the design engineer vs the contractor.
- Identifying which consultant will be the LDSD Lead for the project.
- Engaging a quantity surveyor early during the project to assist with costings and optioneering.
- Engaging a non-structural seismic designer during the project establishment phase for input into key decisions
- Having a specific LDSD project induction process where the objectives and processes to achieve them are explained to all the consultants
- Making appropriate design decisions at the early stages that will aid LDSD
- Developing a Building Movement Strategy commencing at Concept Design.
- Ensuring the design team meet regularly to address LDSD issues and that there is a formal workshop at each workstage, to discuss how the LDSD objectives are being met, how potentially conflicting requirements are being resolved, and the recording thereof.
- Making LDSD an agenda item in Design Team Meetings
- Clearly defining design responsibilities during the project, including which consultant is responsible for leading design work on particular interface areas.
- Identifying critical interface areas and workshopping solutions to those.
- Acknowledging the broad skills of the design team and encouraging members to speak up, even about matters outside their area of specific expertise, to encourage holistic thinking.

Topic-specific workshops that involve multiple disciplines are already common in projects and include Risk, Safety in Design, and Sustainability Certification. A similar approach for achieving LDSD is recommended. Leadership of this process should be agreed early on to identify the LDSD Lead and will depend on the scale and type of project. It could be any of the following: design manager, non-structural seismic designer, architect, or structural engineer.

Good construction is also important to achieve LDSD. So, consideration should be given to involving contractors and sub-contractors, particularly those that manufacture and supply proprietary devices and equipment, in the design process. This is discussed in more detail in Section 7.

3.1.4.3 Non-structural Seismic Designer

This is an emerging consultant role reflecting the growing importance of good seismic design and has been included in the latest NZCIC Guidelines (NZCIC, 2023). All LDSD projects are required to have a non-structural seismic designer is part of the consultant team from the early design stages right through to the construction stages. They will assist with the detailing of NSEs such as partitions, ceilings, and building services.

It is important to note that for simple, or small-scale projects, the non-structural seismic designer does not necessarily have to be a separate consultant, but could simply be an extension to another consultant's scope.

3.1.4.4 Independent LDSD Reviewer

For LDSD Category Level 2 and 3 buildings independent peer review of the building design is required to give clients confidence that their projects are compliant with the LDSD requirements. Independent peer review is also recommended for LDSD Category Level 1 buildings but is not a requirement.

The scope of the independent peer review is limited to the LDSD aspects of the project. The LDSD peer reviews are completed by the Independent LDSD Reviewer at each workstage and at completion. The requirements of this role are detailed in Section 3.6.3.

3.1.4.5 Building Information Modelling (BIM)

Building Information Modelling (BIM) is an excellent way to promote and map out collaboration on a project, and can be useful for LDSD projects, but is not mandatory. A good BIM brief from a client is the first step in consultant commissioning arrangements so that the scope is well understood. The next step is a BIM Execution Plan (BEP) developed with the consultants to set a good foundation for the BIM process.

Agreement on appropriate and consistent Levels of Development (LODs) for building components is part of the BEP. Components will need to be accurately modelled, including information about clearance zones between different building components that are required to achieve LDSD. Clash detection checking can be used to identify 'soft clashes' and minimise the risk of damage due to interactions between different building components during an earthquake.

The New Zealand BIM Handbook (CSA, 2023) provides excellent guidance on all aspects of BIM including appropriate LODs. In particular the document defines LOD 350 as being where the particular model element is graphically represented within the model as a specific system, object, or assembly in terms of quantity, size, shape, location, orientation, and interfaces with other building systems, and so in some cases LOD 350 will be the appropriate selection for relevant aspects of a LDSD project because it addresses interface issues. It is important to note that this does not mean there has to be a blanket requirement for LOD 350 for all model elements, but rather that specific elements should be modelled at this LOD. LOD requirements for different model elements should be clearly defined in the BEP.

For many projects there will be advantages in maintaining and updating the BIM model through the construction stage and, just as importantly, to the future occupation and management of the building. This will help ensure that future alterations to the building are consistent with the LDSD performance requirements. These benefits should be discussed with the client, and the relevant requirements included in tender and construction documents (refer Section 7.2).

3.1.4.6 Safety in Design Considerations for LDS

Consultants will be familiar with their responsibilities under the Health and Safety at Work Act 2015 in respect to design. Good safety in design procedures is an excellent way to encourage collaboration and holistic thinking and so align with LDS. This kind of design is in itself, a way to improve safety by reducing damage and resultant repair work.

LDS projects should not be more onerous than typical projects in respect to safety. Some examples of considerations are:

- Site preparation (for example, excavation for a base isolation basement).
- Construction, including temporary works.
- Safe and convenient access for regular maintenance.
- Access for easily and safely checking structure and critical building components (e.g. seismic joints and seismic isolators) after an event.
- Access for repair of critical components. This is an important aspect because speedy repair is crucial for building reoccupation and so careful thought should be given to how this can be facilitated in the design (refer to Section 7.6).

The outcomes of design discussions, risk registers etc, should be communicated to the contractor and client in an agreed format.

3.2 Concept Design and the Low Damage Seismic Design Brief

With a site selected and with knowledge of the client's high-level LDS requirements, the design can move to the Concept Design stage. The NZCIC Guidelines (NZCIC, 2023) describe Concept Design as the stage for the exploration of sufficient design concepts so that the client (and consultants) can test the brief.

The Code of Practice for the Seismic Performance of Non-structural Elements (BIP, 2025) has a helpful flowchart describing the design process during Concept Design, as well as a detailed list of design tasks.

Some fundamental decisions that affect LDS outcomes are made at this stage, so it is important to explore a good range of design options for foundations, structure, services and architectural form and treatment, but also to engage in an iterative design versus brief process with the client.

3.2.1 Briefing: The Low Damage Seismic Design Brief

As the project moves from the early establishment/feasibility stage, based on a high-level brief, it is important to work towards a more detailed brief during concept design. This is known as a reflected or return briefing process and confirms to the client what they are getting in respect of LDS, as well as helping to brief the project team about the required building performance and ensuring the design is well coordinated.

The easiest way for a client to give a brief for Low Damage Seismic Design is to simply adopt one of the LDS Category Levels provided in this document.

As described in Volume Two of this guidance series, three different LDS categories are defined – Level 1, Level 2 and Level 3 (refer also related summary in Section 1.4.2 of this volume). These LDS category levels can be considered industry best practice for LDS for non-specialist buildings, such as commercial offices or apartments. One of these levels may also be appropriate for some specialist facilities, such as laboratories, museums, or retail facilities.

Alternatively, project specific LDS requirements can be set. Because building seismic resilience is dependent on equitable resilience of many interdependent systems and components, project specific requirements should avoid the disconnection of critical interdependencies and so should be limited in nature whenever possible. Detailed discussion about building performance, with input from the whole design team, will be required to ensure a coordinated and holistic outcome is realised.

Possible discussion points to help inform the best way to adopt LDS for a project are included in Table 3-3 below. More detail on capturing the critical seismic design parameters to meet the brief is in Section 3.3.

Table 3-3: Discussion Points to assist with establishing the Low Damage Seismic Design Brief

Possible Discussion points/questions		Commentary
Primary question	Secondary questions	
How involved does the client want to be involved with determining project specific LDS requirements?	Is the client interested in the detail of the seismic performance? Do they already have a good understanding of engineering and design terminology?	Some clients may prefer the ease of simply specifying one of the recommended LDS category levels or minimum Building Code. Other clients may prefer the flexibility of developing project specific LDS requirements.
What is the building to be used for?	What is its primary use? What is its secondary use?	Impacts on Building Importance Levels. Impacts LDS criteria.
How many occupants are there likely to be?	In normal operation? During peak occupancy?	Informs Building Importance Levels.
How soon do you want to be able to occupy your building after a large earthquake?	Minutes, hours, days, months? Or is this not a significant issue for the client?	Determines if recommended LDS Category Levels can be used. Informs project specific LDS criteria.
Any specific building functions or features that may warrant specific consideration of LDS criteria?		Examples include valuable contents in museums, laboratories, etc
Is some level of repairable damage tolerable in reasonably foreseeable events?	What potential repairs might be considered acceptable? By extent By cost By time to complete.	Can be used to determine a specific response if required.
How reliable is the infrastructure to the site?	Electrical Comms/Data Three Waters	Information on the seismic resilience of public infrastructure is not typically available, and hence any assessment is likely to be indicative only. Consider need for on-site water storage and power generation as below.
Indicative requirements with regard to critical building services.	Is there a requirement for onsite water storage? Are emergency generators or facilities for portable generators needed to cover network outages? Where are heavy building services plant to be located in the building? If base-isolated, where is the proposed location of the isolation plane. Indicative floor-to-floor heights	Refer to Section 6.6.2. Consider what critical services are required for building occupation and how these are to be inspected and repaired.
What performance criteria is required for the site beyond the building footprint and at the interface with the building?	Consider access to the site and to the building on the site, underground services and the general location and likely operation (or not) following an event	Refer to Section 6.13 and consider the different performance objectives for the site versus that for the building and interface with the ground and connecting services
Is the client open to Early Contractor Involvement (ECI)	Is it possible to engage with potential suppliers of proprietary seismic devices in the early design stages?	Understanding the technical requirements of these devices will permit appropriate spatial allowances to be made during the early stages of a project.

3.2.2 Concept Design Key Considerations

The emphasis for design during this stage is typically focused on items that have spatial impacts. This will require input from all the main consultants because it involves discussions about the overall building form and floor-to-floor heights, structural strategy, foundation strategy, building stiffness, spatial requirements for plant items and in-ceiling services, etc.

3.2.2.1 Building Form and Basic Arrangement

The ideal for LDSD is for the primary structure to be as regular as possible, with a well-balanced structural layout – in both plan and building cross-section (refer Figure 3-3). The location of the building core and bracing walls, or frames, should also be arranged in a balanced and symmetrical way. Rectilinear arrangements are usually best, but it's important to note that this does not necessarily have to inhibit the design because much of this can be achieved using secondary structural and non-structural components.

Complex building forms should be avoided or minimised as this may lead to a requirement for a number of seismic joints. These can be costly and complicated to design and construct; and can require ongoing maintenance. Complex building forms can also have implications for fire and acoustic design which can add cost to a project. A more regular-shaped building is a good way to avoid these issues but, if that's not possible, then ensure that these joints are in locations that are easily accessible for maintenance and repairs. Providing easy access to building components that need to be maintained or inspected following an earthquake is important from both a Safety in Design and speed of repair perspective.

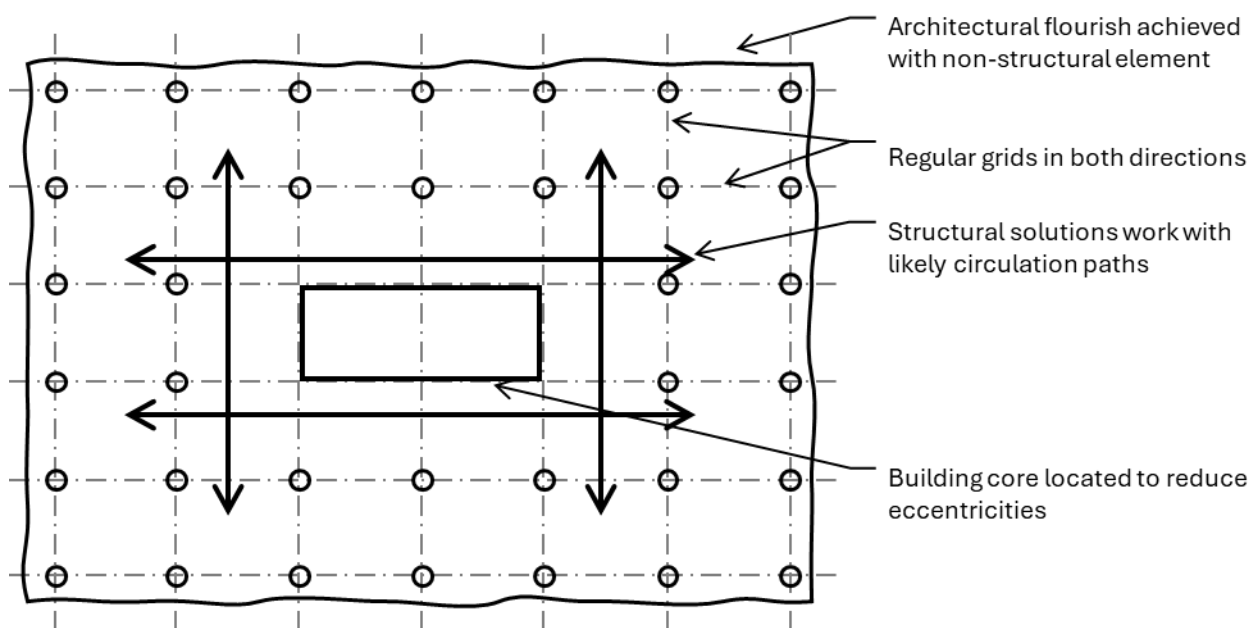


Figure 3-3: Diagrammatic plan showing desirable features to achieve LDSD

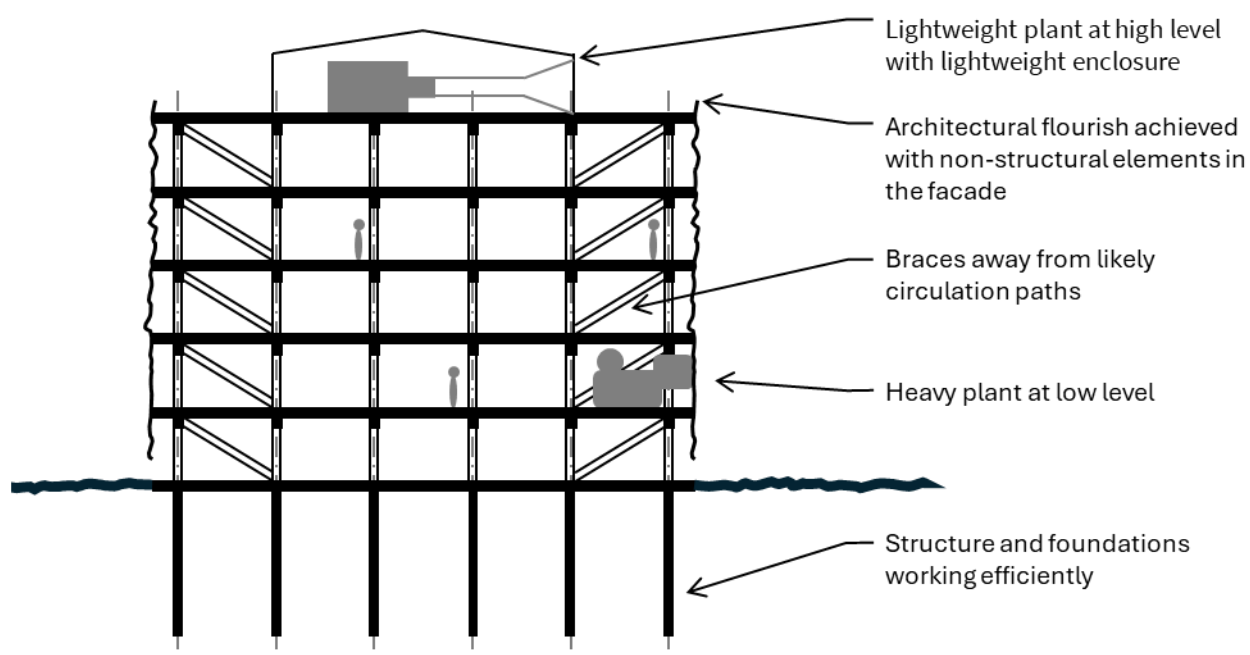


Figure 3-4: Cross section showing desirable features to achieve LDS

The form of the building will impact foundation requirements and the ground conditions should in turn inform the structural solution. During concept design information should be gathered about the site and ground conditions using available and recent site investigations. This information should be used by the geotechnical engineer with the design team to collaborate on ideas, opportunities, and challenges for the building design at the site. Foundation options such as shallow pads, deep piles, and ground improvement should be considered in conjunction with the implications these would have on the building form and the effects of soil-structure interaction (SSI). In all cases, early options for building form (structural walls vs moment resisting frame etc) should be developed in conjunction with options for the foundations to ensure a solution best suited to the site conditions is realised.

Commentary:

An example of the impact of particular ground condition is in the lower Hutt Valley in Wellington which include deep soft sediments over dense artesian aquifer gravels. The soft sediments have limited load carrying capacity and founding in the artesian aquifer gravels is very expensive and complex because of the need to protect this valuable water supply source. Consequently, a structural design needs to be developed that steps lightly on the ground. Building loads should be distributed to foundations rather than concentrated.

Detailing required to accommodate movement associated with base isolation systems can be quite complicated when dealing with services or partitions that cross the isolation plane. To minimise these issues, it is recommended base isolation planes be located in a sub-basement or basement zones (for example, a basement carpark) when this is possible. Try to avoid heavily serviced areas in these zones like plant areas or end-of-trip facilities for cyclists.

Technologies like fluid viscous dampers and buckling restrained braces can introduce large diagonal elements into a building interior. These can block views and present detailing challenges for walls that cross them, but they can also provide occupants some visible assurance about a building's seismic resilience. The location of these diagonal elements should be planned logically so they won't unduly impinge on likely future floor layouts (e.g. check head heights in likely circulation paths etc).

Also consider locating heavy items of plant lower down in the building if practicable. This is to reduce the weight at high levels of the building and reduce the potential for shaking damage to that plant (i.e. seismic floor accelerations are typically less in the lower levels of buildings).

While the illustrations show a multi-storey example, the principles of form and basic planning arrangement also apply to lower scale buildings.

3.2.2.2 Building Stiffness (and Impact on NSEs)

Selecting a structural form and system compatible with your performance requirements is a critical early decision. Some structural systems control stiffness and limit flexibility of the building, while other structural systems control floor accelerations. Selecting a system that is compatible with your performance requirements will make the rest of the design process as simple as possible and should result in the most economical outcome.

A decision about building flexibility can have major ramifications for many aspects of the building design, so careful consideration should be given to this aspect at an early stage. For example, during a major earthquake compliance with the Building Code may involve allowing horizontal inter-storey building deflections up to 2.5% of the storey height (i.e. 75 – 125 mm) (SNZ, 2025). Yet damage to some building components, such as partition walls, can be initiated at much lower inter-storey deflections, typically around 0.3% of the storey height (i.e. 10 – 15 mm) (SANZ, 2011).

Experience around the world from past earthquakes shows that buildings that are stiff and regular suffer less damage.

As noted in Section 1.4.2 this statement is supported by observations of building performance in past earthquakes which have consistently demonstrated that stiff, regular buildings on good ground generally perform well in earthquakes provided they are well detailed and non-structural elements (NSEs) are adequately restrained. Limiting building displacements is an effective means to mitigate damage to the primary structure and deformation sensitive NSEs. But there is an important caveat to this which is that plant and equipment can be problematic with respect to acceleration.

While building plant and equipment can be restrained under seismic shaking, the restraint will not typically significantly reduce the seismic inertia forces the component experiences during an earthquake. Consequently, acceleration sensitive plant and equipment can still sustain significant shaking damage even when adequately restrained. Such shaking damage can render them inoperative (i.e. component will no longer function) or lead to a loss of a structural integrity (i.e. yielding and fracture of component resulting in a potential falling hazard).

Information relating to the seismic performance of acceleration sensitive plant and equipment is often not readily available. While it may be possible to calculate if the plant and equipment will maintain structural integrity under seismic shaking, confirmation of continued functionality will likely require seismic verification (either shake table testing or relevant experience data).

Buildings which rely on moment resisting frames to resist seismic loads are typically relatively flexible structures. These buildings can be expected to sustain relatively high inter-storey deflections during earthquakes, but lower floor accelerations. Buildings that utilise stiffer structural systems, such as reinforced concrete walls or structural steel braced frames, to resist seismic loads can be expected to sustain lower inter-storey deflections, but higher floor accelerations.

Selection of the structural system should also consider available foundation options in parallel. This is because the structural system will influence the distribution and intensity of loading on the foundations. The ability of the ground/foundation to practically or economically support concentrated loads could influence the selection of the foundation system and vice versa. For example, a building with relatively short structural walls or braced bays can impart very large, concentrated loads on the ground. For sites with deep weak ground conditions, resisting these loads may not be practical or may prove relatively expensive.

Further information on building form, primary structural system options, and the relative benefits of each system, is provided in Appendix D.

3.2.2.3 Building Movement Strategy

Closely associated with the approach to building flexibility is the development of a Building Movement Strategy. The Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025) provides guidance on how to develop a Building Movement Strategy. A report detailing the proposed Building Movement Strategy should be developed during Concept Design and updated as the project proceeds.

As noted in the previous section, a structure that is stiffer and therefore deflects less during an earthquake, means that deformation sensitive building components, such as facades, ceilings, partitions, pipes, ducts, cable supports etc, do not have to be detailed to accommodate such large movements at interfaces. Correspondingly, flexible structures require these building components be detailed to accommodate much higher movements. In complex building types, such as apartment buildings, this can have a major impact on the successful design and detailing of deformation sensitive NSEs such as tiled membraned showers and inter-tenancy walls that have to achieve, and maintain, acoustic and fire performance requirements. Particular attention is required at the interfaces of these elements, such as pipe or service ducts penetrating partition walls.

The Building Movement Strategy Report is a way for the project team to collaboratively identify at a more granular level some of the conflicting requirements for acoustic/fire/vibration/thermal/passive fire etc vs seismic movements. Compromises may be required in some locations where not all of the requirements can be met. In these locations the design team should discuss and agree what aspects are to take precedence and which can be compromised, and why the decision was made. This process should be documented in the Building Movement Strategy Report and communicated to the client.

Note that where there are conflicting requirements the following hierarchy should generally be observed:

- Regulatory Compliance (e.g. passive fire performance).
- Day-to-day amenity (e.g. desirable acoustic performance).
- LDSD requirements (e.g. accommodating movement).

Commentary:

It's important for the design team to collaboratively consider the implications of building movement. While a stiff building has advantages for detailing interior partitions etc it may require deeper beams, or a greater number of structural walls, which could significantly impact available routes and/or clearances for building services, or the stiffness may be detrimental to acceleration sensitive components. Resolving these conflicting requirements in specific multi-disciplinary workshops early in the design process and also recording key decisions in the Building Movement Strategy Report, is important.

3.3 Preliminary Design

The Preliminary Design stage is critical for achieving LDSD because it is the stage when key, often irreversible design decisions are finalised. These include the following:

- Foundation selection including indicative solutions for basements and retaining walls.
- Overall building layout, grid layout, and cross section including floor-to-floor heights.
- Core layout with vertical circulation and egress paths.
- Gravity load-carrying structural system with indicative member sizes.
- Lateral force-resisting system with layout and indicative sizes.
- Location of heavy plant and main vertical and horizontal services distribution routes.
- Confirmation of facade treatment and secondary elements like canopies etc

In order to achieve a coordinated response that meets the LDSD brief, it is important to have a number of consultant workshops to resolve any conflicting requirements. The compatibility between the foundation and primary structure options should be tested and refined at this stage and preliminary sizing undertaken. Any issues arising from the Building Movement Strategy should also be managed at this design stage. The Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025) has more detailed description of decisions relating to non-structural elements.

3.3.1 Confirmation of Project LDSD Design Criteria

The Design Team should capture the critical seismic design parameters and how these are addressed, in their respective Design Features Reports at the end of Preliminary Design.

It is expected the design team will make heavy use of the Design Requirements sections of this document (i.e. Sections 5 and 6) to achieve the technical design criteria for the building. It will also depend on building form. For example, a project might adopt a low drift structure (perhaps via base isolation), allowing for conventional partition and façade detailing. Alternatively, a project might adopt special partition and façade detailing to accommodate higher drifts of a more conventional structure. These decisions will need to be made in a collaborative way by the whole design team.

Separate sections should be added to the Design Features Report (DFR), particularly the geotechnical report, that confirm the LDSD design criteria for the building footprint and interface with the site and for the site beyond the building footprint (refer to Section 6.13). The specific performance objectives for the building and the interface should be well established as part of defining the LDSD project objectives. For the site beyond the building footprint, the DFR should also assess the seismic performance of this area and document the agreed level of mitigation as well as the residual issues that have been accepted.

A suggested format for capturing the LDSD Design Criteria is provided in Table 3-4 below, along with some example situations to provide context. The LDSD Design Criteria forms the link between the client's aspirations, as captured through the briefing process, and the detailed building design.

Table 3-4 Project LDS Design Criteria – Example Only

Design Criteria	Benchmarks		Criteria Adopted for Project ¹ (To be Completed by the Design Team)
	NZBC Minimum Requirement	LDSD Recommendation	
General			
Seismic Hazard Definition	NZS 1170.5:2004 ²	TS 1170.5:2025	TS 1170.5:2025
Importance Level	As per NZS 1170.0	As per NZS 1170.0	IL 2
Design Life	Default 50 years.	Default 50 years.	50 years
Will LDSD be adopted?	N/A	Adopt to suit client requirements	No, use code minimum, or Yes, adopt LDSD Category Level 2, or Yes, adopt LDSD Category Level 2 along with the following project specific LDSD requirements.
SLS1 Design Criteria			
SLS1 Seismic Hazard	APoE = 1/25	APoE = 1/50	APoE = 1/50
SLS1 Performance Objective	Avoid damage that would prevent the building from being used as intended without repair. Ductility limited to μ=1.0	Same as NZBC minimum requirement	Avoid damage that would prevent the building from being used as intended without repair. Inter-storey drifts limited to 0.3%. Peak floor accelerations limited to 0.3g.
SLS2 Design Criteria			
SLS2 Seismic Hazard	APoE = 1/500	N/A	N/A
SLS2 Performance Objective	Only applicable to IL 4 buildings ² . The building maintains operational continuity.	N/A	N/A
DCLS Design Criteria			
DCLS Seismic Hazard	N/A	Varies by LDSD Category: Level 1 APoE = N/A Level 2 APoE = 1/250 Level 3 APoE = 1/500	APoE = 1/250
DCLS Performance Objective - General	N/A	Adopt Damage Control and Improved Functionality performance goals from Volume 2 Section 7.	<u>Damage Control:</u> For LDSD Category Level 2 as per Vol 2 Table 7-1. <u>Improved Functionality:</u> OS-II Partial functionality as per Vol 2 Table 7-2.
DCLS Performance Objective – Building Foundations	N/A	Refer to Section 6.3. Direct Design Method pathway	Follow guidance in NZGS Module 4 including allowance for step-change in performance.
DCLS Performance Objective – Building Structure	N/A	Refer to Section 6.4. Inelastic demands on building structure limited to ensure structure is readily repairable. PFAs limited to < 1.25g to limit damage to acceleration sensitive NSEs.	Primary structure ductility limited to μ= 2.0 Building inter-storey drift limited to 0.5% PFAs limited to 0.9g to limit damage to acceleration sensitive NSEs.
DCLS Performance Objective – Building Envelope	N/A	Refer to Section 6.5.	Facade designed to accommodate inter-storey drift of 0.5%

Design Criteria	Benchmarks		Criteria Adopted for Project ¹ (To be Completed by the Design Team)
	NZBC Minimum Requirement	LDSD Recommendation	
DCLS Performance Objective – Ceilings	N/A	Refer to Section 6.9	Ductility of part limited to $\mu_p = 1.25$ Ceilings designed to resist parts acceleration of 2.0g
DCLS Performance Objective – Partitions	N/A	Refer to Section 6.10	Partitions designed to accommodate drift of 0.5%
DCLS Performance Objective – Building Services	N/A	Refer to Section 6.6	
DCLS Performance Objective – Landscaping and External Service	N/A	Refer to Section 6.13	
ULS Design Criteria			
ULS Seismic Hazard	Varies by Importance Level: IL2 APoE = 1/500 IL3 APoE = 1/1000 IL4 APoE = 1/2500	Same as NZBC minimum requirement	Building is IL 2. APoE = 1/500
ULS Performance Objective – Building Structure	Maintenance of structural integrity and gravity load support.	Refer to Section 6.4.	Primary structure ductility limited to $\mu = 3.0$ Building inter-storey drift limited to 2.0%
ULS Performance Objective – NSEs	Avoidance of collapse or loss of support to parts of Categories P.1, P.2, P.3 and P.4. Avoidance of damage to Category P.4 parts necessary for building evacuation that will render them inoperative.	Same as NZBC minimum requirement.	

Notes:

1. Content in column four of the table has been added as a typical example of a potential LDSD criteria for a project. Design teams are expected to populate this column with the design criteria developed for their project.
2. As specified in NZS1170.5 (2004) which is cited in the Building Code via B1/VM1.

3.4 Developed and Detailed Design

Developed Design is an important stage to resolve any outstanding inter-disciplinary design issues, although inevitably design coordination issues continue to be worked on during the subsequent Detailed Design stage. In the past it has often been left until the construction stage for the seismic design of NSEs. However, this approach can often lead to poor or inefficient project outcomes and should be avoided.

The Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025) recommends there is a lag between the completion of the building design and the delivery of the NSE documentation. This is good practice but does not mean that the non-structural seismic designer is not an active participant during these stages. There should also be the opportunity for other consultants to update their documents after the NSE documentation and before final issue.

The NZCIC Guidelines (NZCIC, 2023) provides a detailed list of the requirements for consultants' design and documentation for conventional projects. Additional requirements for LDS projects at these stages include:

- Ensure ongoing and well thought out coordination.
- Workshop any situations where there are incompatible requirements.
- Consider access for post-event inspections.
- Consider access for ease and convenience of repairs.
- Update DFRs with LDS content.
- Update the Building Movement Strategy Report, in particular to record decisions on conflicting requirements and how these have been resolved.
- Ensure drawings and specifications align with the content of any reports related to LDS.
- Ensure that specifications contain detailed requirements for Operating and Maintenance (O&M) Manuals to contain relevant LDS information.
- Depending on the procurement strategy, consider how LDS requirements will be communicated to any tenderer, contractor or sub-contractors. Refer to Section 7 for more information on this aspect.
- Think ahead to how the drawings, specifications, and reports can be used for the post-construction stage of the building's life (e.g. for use in a Building Re-occupation Strategy Report).

Sometime around the transition between these stages it is important to establish the strategy for Building Consent (refer Volume 2 Section 2.1) and Peer Review (refer Section 3.6), noting that this should have commenced during the Preliminary Design stage according to the NZCIC Guidelines

Commentary

An example of the kind of coordination issue that might arise during these workstages is where an architect is working on the detailing of walls between bedrooms and the living area in an apartment building. They note that there is a requirement from the structural engineer for a deflection head to all partitions; the non-structural seismic designer requests that these stop short of any 'T' junction of partitions; the acoustic engineer requires that there is no gap at the top of the walls that would allow noise transfer to occur. After some discussion, including with the developer client, it is agreed that the day-to-day acoustic performance takes precedence over damage limitation and so the detail is revised to achieve the required acoustic performance. This decision is recorded by the non-structural seismic designer in the Building Movement Strategy Report.

3.5 Construction Stage Design

Whether or not there has been Early Contractor Involvement (ECI), there will always be a considerable quantum of design, and confirmation of proprietary equipment selection undertaken during the Construction stage. It is important sub-contractors and their submittal/shop drawing teams are fully aware of the specific requirements for LDS. The same applies to the consultant teams reviewing the submittals and shop drawings. It can be helpful for salient aspects of LDS to be clearly noted on drawings and specifications where they are more likely to be seen and allowed for, than in separate reports that are sometimes not included in contract documents.

In addition, the need to ensure building elements are well co-ordinated remains a priority throughout the construction stage. This is covered in detail, along with the expectations of the contractor in Section 7. This includes a requirement for a NSE Seismic Coordinator to be part of the contractor's team.

3.6 LDS Compliance and Peer Review

3.6.1 General

Demonstrating compliance with the LDS requirements detailed in this document is the responsibility of the design team.

3.6.2 Demonstrating Compliance with LDS Requirements

Compliance with the LDS requirements detailed in this document should be demonstrated by a section within the Design/Design Features Reports (DFRs) provided by each design consultant. These reports should include a description of how compliance with the LDS requirements has been demonstrated with specific references to the relevant Design Requirements sections (i.e. Sections 5 and 6).

A statement from each design consultant within this section of their DFR is also required to confirm their design meets the LDS requirements.

A PS1 *Design* Producer Statement is purely for purposes of obtaining a Building Consent and would not normally be sufficient to establish compliance with the LDS requirements, given that aspects of the design may go beyond minimum Building Code requirements.

3.6.3 LDS Peer Reviews

Peer reviews should be undertaken in accordance with the Engineering NZ Practice Note 2: Peer Review (Engineering NZ, 2018).

LDS peer reviews are not the same as regulatory peer reviews and have a different scope. Regulatory peer reviews are undertaken to establish compliance with the requirements of the Building Code. For those projects where there is a structural engineer engaged to undertake a peer review for the purpose of regulatory compliance with the Building Code it would typically make sense for this person to have their role expanded to be the Independent LDS Reviewer.

The independent LDS peer review would consist of the following:

- Milestone reviews completed by the Independent LDS Reviewer at the end of each workstage.
- Peer review of the design of key building components by an independent peer reviewer at the end of the detailed design phase.

Details of which building components are judged to be critical and will require an independent peer review are detailed 3.6.3.2 below. Overall, the LDS Peer review need not be onerous, especially for small or simple buildings.

3.6.3.1 Milestone LDSR Reviews

Milestone LDSR Reviews should be completed by the Independent LDSR Reviewer (refer Section 3.1.4.4) at the end of each work stage. Scope of the milestone reviews includes the items detailed in Table 2-1 that are appropriate to the design phase of the project and LDSR Category of the building.

The Milestone LDSR Reviews are to be undertaken at a high level and should confirm the items in detailed in Table 2-1 have been completed and the outcomes are consistent with what would be reasonably expected.

A short report summarising the key findings of each milestone review should be prepared by the Independent LDSR Reviewer and issued to the client with a copy sent to the project LDSR Lead (refer Section 3.1.4.2).

3.6.3.2 Detailed LDSR Review

Key building components should be subject to an independent LDSR peer review at the end of the detailed design phase to determine whether the design of these components complies with the LDSR requirements detailed in this document. Key building components subject to independent peer review should as a minimum comprise of the following:

- Foundations.
- Building Structure.
- Building Envelope.

When the Direct Assessment Method pathway is adopted for a project, whereby a project specific loss assessment is used to validate the LDSR performance goals specified in Volume 2 have been met, the loss assessment should also be subject to an independent peer review.

The scope of the independent peer review can be extended to include other building components as needed on a project-by-project basis if the design team judges this to be necessary.

A short report summarising the key findings of each independent review should be prepared by the independent peer reviewers and issued to the client with a copy sent to the project LDSR Lead.

4 Seismic Hazard

4.1 Overview

Seismic hazard is the intensity of ground shaking hazard associated with potential earthquakes in a particular area. This relates to the size of the earthquake a building is designed for and varies for each building performance limit state.

Volume Two Section 8 defines the design intensity levels of ground shaking (expressed in terms of APoE) to be used for the design of LDS Design Category Level 1 to 3 buildings.

The procedures detailed in the New Zealand earthquake design actions technical specification TS 1170.5 (SNZ, 2025) have generally been adopted. For the case of base isolated buildings, designers are also referred to the NZSEE Draft Seismic Isolation Guidelines (NZSEE, 2019).

Commentary:

When using this document earthquake design actions should be determined in accordance with the latest New Zealand earthquake loadings standard or technical specification. At time of writing this is TS 1170.5 (SNZ, 2025), however it is anticipated in the future that either TS 1170.5 or NZS 1170.5 (SNZ, 2004) will be updated and when this occurs the more recent document should be used. When this document refers to TS 1170.5 this should be taken to be the latest New Zealand earthquake loadings standard or technical specification. It is understood there are also plans to update the NZSEE Draft Seismic Isolation Guidelines (NZSEE, 2019). When an updated version of this NZSEE document is published it should also be used for LDS projects.

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4.2 Acceleration and Displacement Seismic Demands

4.2.1 Elastic Site Spectra for Horizontal Loading

The elastic site hazard spectrum for horizontal loading should be determined in accordance with TS 1170.5 Section 3.1 except as modified below, or by a site-specific probabilistic hazard analysis in accordance with Section 4.7.

The 5% damped elastic site displacement spectra for horizontal loading, $S_d(T)$, in units of mm can be determined using Equation 4-1:

$$S_d(T) = \frac{T^2}{4\pi^2} C(T)g \quad \text{Eqn 4-1}$$

where:

$C(T)$ = elastic site spectrum for horizontal loading determined in accordance with TS 1170.5.

T = period (seconds).

g = gravitational acceleration (9,810 mm/s²).

4.2.2 Elastic Site Spectra for Vertical Loading

The elastic site hazard spectrum for vertical loading should be determined in accordance with TS 1170.5 Section 3.2 or by a site-specific special study in accordance with Section 4.7.

4.3 Earthquake Design Actions

Earthquake design actions should be determined in accordance with TS 1170.5 except as modified below. When evaluating earthquake design actions for geotechnical assessment of ground deformations and site stability, such as the effects of liquefaction, cyclic displacement or lateral spreading, refer to guidance provided in NZGS Module 1 (NZGS/MBIE, 2021). When using NZGS Module 1 seismic design parameters PGA and Magnitude from TS 1170.5 should generally be adopted (refer to Section 4.6). Earthquake foundation design actions should consider kinematic loads in combination with the building inertia loads and NZGS Module 4 provides some guidance on these design combinations (refer to Section 6.3 for more information).

At high levels of design earthquake shaking, consideration should be given to nonlinear soil/rock response and whether the design actions derived from elastic methods (i.e. including those from TS 1170.5) are appropriate. Plastic soil/rock behaviour can be induced at high levels of earthquake shaking and nonlinear methods for determining earthquake design actions could be considered in these cases. Refer to Sections 4.6 and 4.7 below for more information.

4.3.1 Equivalent Static and Modal Response Spectrum Analysis Methods

Earthquake design actions for equivalent static and modal response spectrum analysis methods should be calculated in TS 1170.5 Section 5.2.

4.3.2 Displacement Based Analysis Methods

The design displacement spectrum for horizontal loading, $\Delta_d(T)$, for different levels of system equivalent viscous damping, ξ_{sys} for displacement-based analysis methods can be calculated using Equation 4-2:

$$\Delta_d(T) = S_d(T) K_\xi \left(\frac{1 + S_p}{2} \right) \quad \text{Eqn 4-2}$$

where:

$S_d(T)$ = elastic design spectrum for horizontal loading determined from Equation 4-1.

K_ξ = spectral damping reduction factor.

S_p = structural performance factor determined from TS 1170.5.

The spectral damping reduction factor, K_ξ , should be determined in accordance with Equation 4-3:

$$K_\xi = \left(\frac{0.07}{0.02 + \xi_{eq}} \right)^\alpha \quad \text{Eqn 4-3}$$

where:

α = 0.25 for near-fault sites.

= 0.5 for other situations.

ξ_{eq} = equivalent viscous damping for the system.

A near-fault site is defined as a site that is within 10 km of a major fault as defined in TS 1170.5 Figure 3.2.

Commentary:

Displacement-based design based on using the inelastic secant stiffness as detailed in Priestley et al. (2007) provides an alternative approach to the force-base design approach specified in TS 1170.5 for determining the earthquake demands for buildings. The Direct Displacement Based Design (DDBD) procedure requires the structural system to be converted to an equivalent single degree of freedom system having an effective elastic stiffness, effective mass, effective height and damping corresponding to the target displacement. The procedure is as follows:

- *Select a design inter-storey drift limit. The design inter-storey drift limit would be typically determined by considering (1) peak ULS inter-storey drift limit as defined in TS 1170.5, (2) deformation limits in potential plastic hinge regions, (3) residual drift limit at the DCLS and (4) limiting drifts at SLS1 and DCLS to mitigate damage to parts of structures and non-structural components. Higher mode drift amplification should be considered when selecting inter-storey drift limits (refer Section 5.4 of Priestley et al. (2007)).*
- *Evaluate the equivalent SDOF properties, defining the target displacement (at the effective height), $\Delta_d(T)$.*
- *Calculate the equivalent viscous damping for the building. This is related to the system ductility, μ , and the typology of the structure. Enter a design displacement spectrum, reduced to account for the ductility/damping of the system, and determine the effective period, T_{eff} .*
- *Evaluate the effective stiffness, K_{eff} knowing effective mass and effective period.*
- *Calculate the base shear by multiplying the secant stiffness by the target displacement, $V_b = K_{eff}\Delta_d(T)$.*
- *Distribute the base shear throughout the primary structure and design members. Capacity design should be applied to the structure to meet the requirements of TS 1170.5.*

For further details refer to Priestley et al. (2007) and NZCS (2010). Refer to (Millen, Pampanin, & Cubrinovski, 2020) for details of how to include soil structure interaction (SSI) in displacement-based design. Further information on SSI is expected to be included in the 2025 update to Section C4 of the NZ Seismic Assessment Guidelines (NZSEE, 2025).

Implementation of the NZS 1170.5 structural performance factor, S_p , in Equation 4.2 is consistent with that recommended in the NZSEE Draft Seismic Isolation Guidelines (NZSEE, 2019) and by Marriot (2018).

4.4 Nonlinear Response History Analysis

Ground motion selection and scaling procedures detailed in the NZ Industry Nonlinear Response History guidelines (SESOC, NZSEE and NZGS, 2025) should be used.

When nonlinear response history analysis (NLRHA) is being undertaken for the structural design of buildings in high seismic regions, and where nonlinear soil/rock behaviour is expected to significantly affect site response, nonlinear site-specific response analysis is recommended to determine the appropriate design spectra for ground motion selection and scaling (refer to Section 4.6).

Commentary:

As detailed Morris et al in (2019), the ground motion selection and scaling requirements in NZS 1170.5 (SNZ, 2004) were established in 2004 and are outdated. Seismic demands have large record-to-record variability and concerns exist that the specification of a minimum of three records in NZS 1170.5 may be unconservative (Bradley, 2011), (Hancock, Bommer, & Stafford, 2008). Ground motion selection and scaling procedures in the NZ Industry Nonlinear Response History guidelines (SESOC, NZSEE and NZGS, 2025) are based on more recent research and address these issues (Haselton, et al., 2017).

4.5 Seismic Design Actions for Parts and Components

Seismic design actions (accelerations and inter-storey deflections) for parts of structures and non-structural components and their connections should be determined by the structural engineer and communicated to the wider project team. This should include seismic design actions for SLS1, DCLS (for LDSD Category Level 2 and 3 buildings only) and ULS load combinations.

Seismic design actions for parts and components should be determined in accordance with TS 1170.5 Section 8. Alternatively, when NLRHA is used as the structural analysis method, building specific floor response spectra can be developed in accordance with the NZ industry Nonlinear Response History guideline Section 5 (SESOC, NZSEE and NZGS, 2025).

When computing peak inter-storey displacement demands, additional deformation from member elongation and support rotation should be included when these are significant. Analysis models used to compute building specific floor response spectra should include foundation flexibility when this significantly affects the dynamic properties of the building.

For seismically isolated buildings, and buildings with supplemental damping systems, it is recommended building specific floor response spectra be developed from NLRHA.

Commentary:

While some standards – i.e. NZS 4219 (SNZ, 2009) and AS/NZS 2785 (SANZ, 2020) – permit seismic design actions for non-structural components to be calculated using simplified methods, it is recommended the structural engineer for the project provides these loads to the design team. This will enable a more consistent level of seismic performance to be achieved across the various parts and components that may be present.

When reporting seismic design actions for building parts and components to the wider design team, it is important this information is communicated for each structural limit state in an unambiguous way. Suggestions here include:

- When TS 1170.5 Section 8 is used to derive design actions, the design actions should be communicated in terms of the design response coefficient for parts, $C_p(T_p)$. Design actions for rigid and flexible components should be provided, and in some instances, it may be appropriate to provide design actions for long-period components. Part ductility factors, μ_p , used to compute the design actions, and $T_{p,long}$ appropriate for the structure for each structural limit state considered, should be reported.*
- When building specific floor response spectra are developed, background information used in the analysis, including the non-structural damping ratio and any allowance for inelastic response of the non-structural elements, should be reported.*

Due allowance should be made for the flexibility of connections, including deformation of anchors and secondary supporting elements when these are present, when determining the period of building parts and components.

NZS 3101 (SNZ, 2017) provides guidance which can be used to compute member elongation for DCLS load combinations.

Use of building specific floor response spectra for building parts and components would be considered an alternative solution in the NZBC and therefore a peer review is recommended.

4.6 Seismic Demands for Geotechnical Considerations

Seismic demands for geotechnical considerations should be derived in accordance with the requirements of TS 1170.5, with consideration of the guidance in NZGS Module 1 except as modified below. Three available methods are available for deriving shaking hazard parameters:

- Method 1: Refer to TS 1170.5 (PGA and Magnitude pairs).
- Method 2: Site-specific probabilistic seismic hazard analysis (refer to Section 4.7 and NZGS Module 1 for guidance).
- Method 3: Site-specific site response analysis (refer to Section 4.7 and NZGS Module 1 for guidance).

Method 1 is appropriate for routine engineering design projects whereas Methods 2 and 3 are preferred for more significant or complex projects. For LDS, at high levels of earthquake shaking where significant nonlinear soil/rock behaviour is anticipated and will likely affect seismic demand, Method 3 could be adopted with allowance for nonlinear soil/rock response.

In accordance with TS 1170.5, deformations of the foundation soils should not compromise the assumed structural behaviour, so it is important to consider the effects of a potential step-change in soil behaviour between SLS and ULS as well as beyond ULS. In the absence of other specific guidance, step-change behaviours should be considered for shaking intensity up to 150% ULS (for further details refer to TS 1170.5).

Commentary:

At the time of preparing the LDS document, TS 1170.5 had not been issued as final, but it is understood that it will be released before this document is published. In practice, geotechnical seismic design parameters were being derived directly from NZGS Module 1 (NZGS/MBIE, 2021) before the issue of TS 1170.5, i.e. for Method 1 above. However, the authors understand that the parameters provided in Module 1 were interim values and TS 1170.5 will supersede these parameters when it is published.

4.7 Site-specific Hazard Studies

4.7.1 General

Site-specific seismic (probabilistic) hazard analyses should be completed in accordance with the requirements of TS 1170.5 and NZGS Module 1 as appropriate. The constraints noted in B1/VM1 of the NZBC regarding the results from a site-specific hazard analysis apply.

An element of nonlinear site response will be included in the probabilistic site-specific seismic hazard analysis (typically through use of V_{s30}). However, for sites with a high level of seismic shaking and/or where significant nonlinear soil/rock behaviour is anticipated, the site-specific hazard analysis could be combined with site-specific site response analysis (i.e. Method 3 in Section 4.6) to account for these effects.

Specific consideration should be given in the site-specific analyses for DCLS, in addition to SLS and ULS, as well as beyond ULS. Annual probabilities of exceedance (APoE) considered for site-specific analysis should consider the specific limit states and should also allow for seismic performance to be considered in between the limit states, and beyond ULS, to achieve the objectives of LDS.

The use of site-specific hazard studies to determine seismic loads should be subject to independent peer review.

Commentary:

Additional background information related to site-specific probabilistic seismic hazard analyses is available from Quake Centre (Dismuke & Fraser, 2020).

4.7.2 Building Parts and Components

TS 1170.5 Section 8 can be used to determine seismic design actions for parts and components using the PGA computed from site-specific seismic hazard analyses.

4.8 Other Seismic Hazards

4.8.1 Fault Rupture

Fault rupture may extend to the ground surface and generate ground deformations that could cause significant damage to foundations and buildings. NZGS Module 1 provides guidance on considerations for fault rupture in geotechnical design and references the Ministry for the Environment (MfE) document - Planning for Development of Land on or Close to Active Faults (MfE, 2003).

LDSD Category Level 1 to 3 buildings are not permitted to be located within fault avoidance zones associated with faults which have fault recurrence interval less than or equal to 10,000 years (i.e. fault recurrence class IV or less).

Fault avoidance zones defined in the MfE document have been adopted by a number of Territorial Authorities around NZ (e.g. Wairarapa and Greater Wellington Regional Councils) and are available on the GNS Active Faults Database (GNS, 2024). A minimum buffer zone is set at 20 m either side of a known active fault trace, or likely fault rupture zone, where development is restricted to ensure life safety, refer to Figure 4-1.

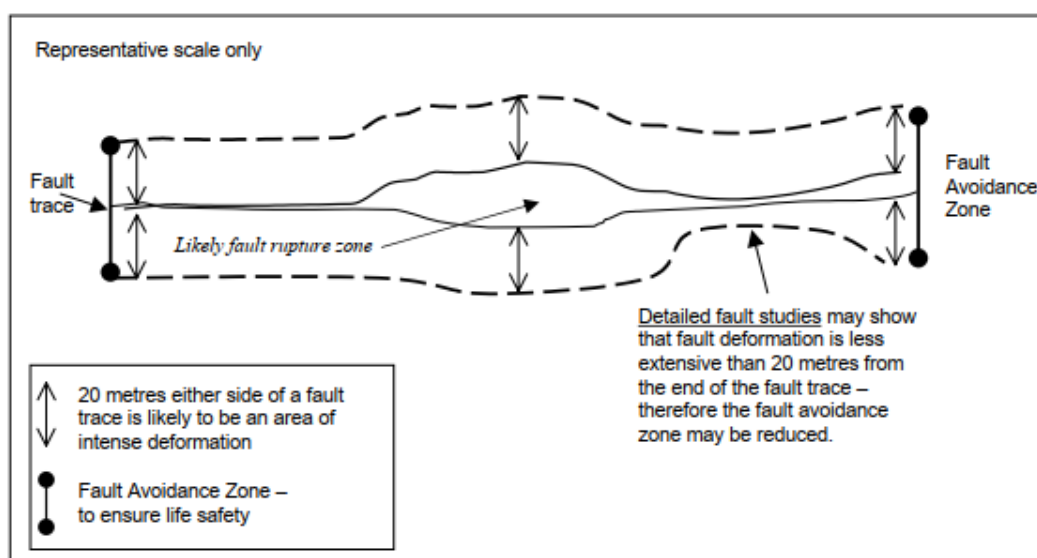


Figure 4-1 Example of a fault avoidance zone on a district planning map (MfE, 2003).

When designing buildings within close proximity to faults (even outside fault avoidance zones), design options that mitigate potential building damage due to ground deformations should be adopted. This includes stiff raft foundations not tied into the ground (i.e. without piles or ground anchors) and structures designed to accommodate significant foundation deformations. Bray (2009) provides some guidance on designing buildings to accommodate earthquake surface fault rupture.

Commentary:

Building within a Fault Avoidance Zone is generally beyond the scope of achieving LDSD. As stated in the MfE document, “buildings within a fault avoidance zone, particularly buildings crossing active faults, are very likely to be damaged in a fault rupture.” When considering a site for LDSD, the client and the design team should consult the GNS Active Fault Database (GNS, 2024) and Territorial Authority maps to identify whether a fault avoidance zone may limit the viability of LDSD, and this should be considered as part of the briefing process (refer Section 3).

As well as the location and complexity of the surface fault rupture, the activity of the fault, as measured by its average recurrence interval, is also an important consideration. Faults with short recurrence intervals are generally more likely to rupture in the near future than faults with a longer recurrence interval.

The GNS Active Fault Database provides six classes of average fault recurrence intervals (I to VI) and the MfE document outlines building importance categories that can be considered for construction in the vicinity of faults with different classes. Building limitations within fault avoidance zones are consistent with MfE recommendations (MfE, 2003) for IL 3 buildings in previously subdivided or developed sites, and for IL 2 and IL 3 buildings in greenfield sites.

It is important to note a fault is generally a zone of deformation rather than a single linear feature and fault avoidance zones may be defined to combine nearby faults. Furthermore, the size of the avoidance zone depends on the complexity of the fault, with GNS providing 5 categories: well-defined; well-defined extended; distributed; uncertain; uncertain constrained; and uncertain poorly constrained (Morgenstern & Van Dissen, 2020).

4.8.2 Seismic Slope Instability

Strong earthquake induced ground shaking can significantly reduce the stability of inclined masses of soil and rock and cause landslides. Earthquake induced landslides can cause damage to buildings through ground deformations and/or impact of slope debris. The instability can originate from outside the site being considered and should be dealt with as part of standard design practice. Different mechanisms for slope movement potentially causing building damage are shown in Figure 4-2.

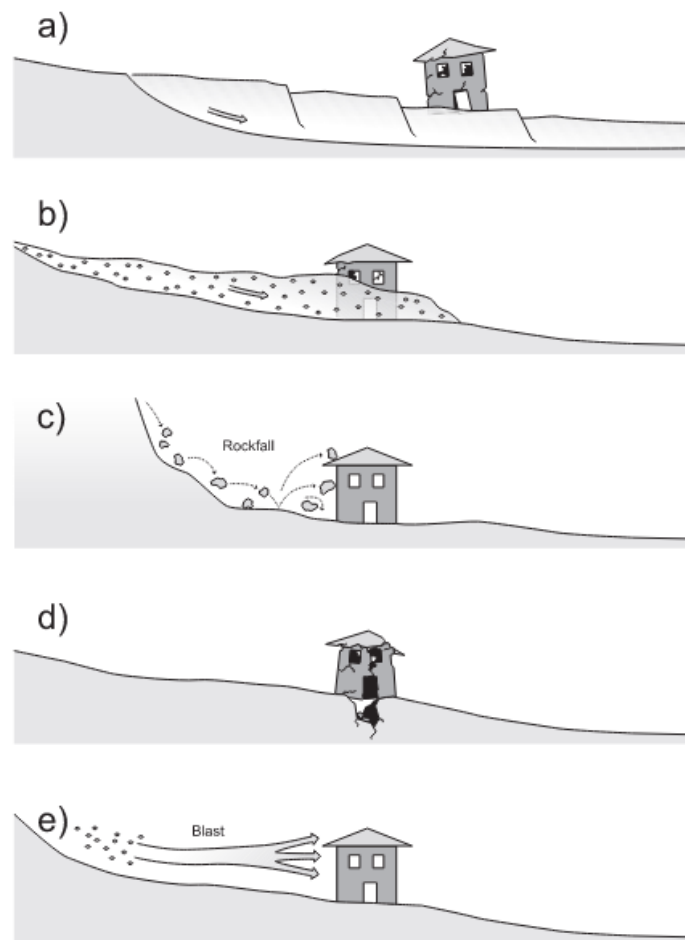


Figure 4-2 Potential damage due to landslides depends on the style and velocity of movement. (a) shallow translational or rotational movement (b) debris flow (c) rock fall (d) subsidence and (e) rock avalanche

The most common landslides during earthquakes are rock and soil falls on very steep cliffs, escarpments, gorges, gravel banks, and high unsupported man-made cuts. Liquefaction induced lateral spreading is also a common earthquake landslide mechanism (refer to Section 6.3 for further details and consideration).

Site selection during the briefing process (refer Section 3) should consider the potential for damage due to seismic slope instability. Further details on assessing slope instability, including a risk assessment process that could be considered early in design, is provided in the *NZGS Slope Stability Geotechnical Guidance Series* (NZGS, 2025).

4.8.3 Tsunami

Tsunamis can be “near field”, generated from the tectonic plates close to our land mass, or “far-field” from the more distant plate boundaries (e.g. along the South American western coast). The proximity of tsunami can affect both the magnitude and more limited warning times for evacuation, meaning that consideration of tsunami impacts and loads are required for LDSD as well as providing for evacuation methods for events where there are longer warning times.

While large-scale, destructive tsunami have not recently been generated in the vicinity of New Zealand, there is a history of large local and distant source tsunami occurring with severe damage occurring in affected coastal cities and communities (Paulik, Craig, & Popovich, 2020). National tsunami amplitude curves and deaggregation plots for 20km coastal sections around the country has have been prepared by GNS (Power, Burbridge, & Gusman, 2022). Tsunami evacuation zone maps have been prepared based on this national data by many councils using the guidelines developed by the Ministry for Civil Defence and Emergency Management (MCDEM, 2016).

The Orange Zone as defined by MCDEM (2016) shown on these maps, is based on a 500-year event and characterises the area to be evacuated in most, if not all, distant and regional-source official warnings of tsunami. It is suggested that if a building is to be sited in the orange zone, or seaward of it, LDSD consider the potential effects of tsunami. Where no maps exist and development is on the open coast, consider If flooding or impact loading associated with tsunami present unacceptable consequences for a project, then a site-specific assessment of the likelihood of tsunami inundation should be undertaken, irrespective of whether or not it is located within the orange zone.

Additional regional and local assessments have been completed by a number of regional councils, so design teams are advised to check if local data sources exist which might provide a more refined tsunami hazard information.

Consideration could be made in LDSD for buildings on ground levels lower than 6m above Mean High Water Springs. It is noted the available information (MCDEM, 2016) has been prepared for evacuation warning purposes and not for assessment of a specific building site.

There is limited New Zealand specific design guidance for tsunami effects. Design guidance on considering tsunami effects on bridges is included in the NZTA Waka Kotahi Bridge Manual (NZTA, 2022).

The most current international guidance for a wider range of buildings is provided in ASCE 7-22 (ASCE, 2022). This standard also provides risk categories, assessment methods and design procedures including hydrostatic, hydrodynamic and debris impact loads including for preventing failures of multistorey buildings during tsunamis, enabling risk reduction in planning and siting of facilities, designing to mitigate damage.

Technical information was released by MBIE, (2020) on the tsunami loads and effects on vertical evacuation structures intended to provide short-term refuge during a tsunami event and provides useful local context to determining potential tsunami loads and effects.

LDSD for tsunami should focus on control of damage at DCLS earthquake intensities. Mitigation measures could include:

- Setting ground floor levels higher to reduce risk of inundation.
- Selecting a purpose, finishings and details for the ground floor which is relatively tolerant to inundation.
- locating sensitive or critical items in the building above tsunami inundation level.
- Designing structural elements, and where practical other elements, to tolerate assessed tsunami impact loads.

Commentary:

It is noted that hazard assessments and design guidance are regularly being updated, and it is important that design teams check for the latest guidance.

Uncertainty exists regarding the performance of base isolated buildings when subject to tsunami effects due to the potential for tension on the isolators due to overturning and buoyancy actions. Guidance (in Japanese) for designing tsunami resistant base isolated buildings is available (JSSI, 2020). It is anticipated further guidance might be provided in future updates to the NZSEE Seismic Isolation Guidelines (NZSEE, 2019).

5 Design Requirements for LDSD Level 1 Buildings

5.1 Overview

This section sets out recommendations for LDSD Category Level 1 buildings. Unlike LDSD Category Level 2 and 3 buildings, Level 1 buildings don't have specific damage control performance goals and do not require design checks at DCLS load levels. Instead, design requirements for LDSD Category Level 1 buildings have been established to ensure these buildings will behave in a predictable, desirable manner when subject to earthquake shaking by applying fundamentally good engineering practice such as ensuring buildings are regular, redundant and outputs from each design consultant are co-ordinated.

Commentary:

Design requirements for LDSD Category Level 1 buildings include some of the recommendations from the NZSEE Earthquake Design for Uncertainty advisory (2022) and the SESOC Interim Design Guidance for Conventional Structural Systems (2022). This section also references the BIP Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025) for the design and co-ordination of non-structural elements.

5.2 Seismic Hazard

Earthquake design actions should be determined using TS 1170.5 (SNZ, 2025) in accordance with Section 4. When evaluating earthquake design actions for geotechnical considerations, such as slope stability and the effects of liquefaction, refer to guidance provided in NZGS Module 1, Module 3 (NZGS/MBIE, 2021) and the NZGS Slope Stability Geotechnical Guidance Series (NZGS, 2023), with consideration for LDSD outcomes. Design parameters (PGA and Magnitude) detailed in TS 1170.5 should generally be adopted for geotechnical design. Earthquake foundation design actions should consider kinematic loads in combination with the building inertia loads and NZGS Module 4 provides some guidance on the appropriate design combinations. Buildings should not be constructed within fault avoidance zones associated with faults which have recurrence intervals of less than or equal to 10,000 years.

Commentary:

Section 4 provides guidance on how earthquake design actions for geotechnical considerations, structures, parts of structures and non-structural elements should be determined. NZGS Modules 1, 3 and 4 and NZGS Slope Stability Geotechnical Guidance Series (NZGS, 2023) are the primary references for geotechnical considerations. For further information related to fault rupture considerations and building close to active faults refer to Section 4.8.

5.3 LDS Category Level 1 Design Requirements

5.3.1 Design Requirements for Foundations

Early engagement between geotechnical and structural engineering, and collaboration with the wider design team on the foundation types most suitable for the ground conditions is an important part of achieving LDS Category Level 1. NZGS Module 4 (NZGS/MBIE, 2021) should form the basis of foundation design to achieve LDS and should be read in conjunction with this Section.

A key aspect of foundation design for LDS Category Level 1 buildings is the consideration of potential step-change in performance of soils and geotechnical systems, particularly related to liquefaction triggering. Even though Level 1 buildings do not require design checks at DCLS load levels, the potential for step-change in behaviour between SLS and ULS as well as beyond ULS should be considered.

Foundation deformation limits at SLS and ULS should be agreed by the design team in the early stages of the project and the impact of potential step-change in soil performance on those deformation limits should be addressed (refer to TS 1170.5 and NZGS Module 4). Consideration should be given to foundation performance between SLS and ULS, as well as beyond ULS. The type of foundation should be selected to mitigate potential issues related to ground deformation with consideration of the building form and tolerance to settlement. Ground improvement can also be considered to mitigate the effects of ground deformation on the building. However, in situations where there is a high level of seismic shaking, nonlinear soil/rock behaviour and soil yielding should be considered to achieve LDS Category Level 1.

Rocking and sliding of foundations are permitted for ULS load combinations and should be designed in accordance with TS 1170.5 Section 5.9.

Commentary:

A foundation design that responds to the ground conditions and where foundation performance is consistent with the design assumptions and objectives of other disciplines provides the best opportunity to achieve LDS Category Level 1. In general, a well tied together regular foundation option, or stiff concrete raft, on a thick natural soil raft (refer to NZGS Module 4), or in combination with ground improvement to mitigate undesirable soil behaviour, provides a good opportunity to achieve LDS Category Level 1. However, this will only be achieved when the building structure, and other non-structural elements, are integrated into the foundation design.

Pile foundations provide the potential for additional foundation capacity and reduced settlement compared to shallow foundations, however, there is the risk of unseen damage that needs to be considered and mitigated. Accordingly, pile foundations should generally be designed to remain elastic, which can be difficult in areas of high seismicity.

Shallow foundations present the opportunity to decouple ground response from foundation response, but they require careful consideration of the potential for differential settlement across the building platform, particularly due to liquefaction effects.

5.3.2 Design Requirements for Building Structure

In addition to the requirements of the NZBC, the primary structure is to be designed in accordance with the material standards cited in NZBC B1/VM1 with the additional structural design criteria detailed in this section.

The structural ductility factor, μ , used for the design of the primary structure should not exceed 3 for ULS load combinations. Peak inter-storey deflections for ULS design actions, including consideration of foundation flexibility, should not exceed 1.5%.

Except for single storey buildings, or the uppermost storey of multi-storey buildings, capacity design should be used to ensure that a suitable sway mechanism develops under very rare shaking levels. Potential plastic hinge

regions should be detailed in accordance with the relevant material standard to ensure they have adequate deformation capacity to reliably resist anticipated seismic demands.

Upper limit shear design actions on reinforced concrete and masonry walls should be determined as 1.5 times the ULS seismic design action with $\mu = 1.25$ and $S_p = 0.9$.

Structures should not have vertical or plan irregularities as defined in Section 4.5 of TS1170.5, except that a vertical stiffness irregularity is permitted for structures when capacity design has been used to provide a level of protection against the formation of a column storey sway mechanism.

No single line of lateral force resistance to have a single element or assembly resisting more than 60% of the total lateral demand in the building in that direction.

Reinforcement used in potential plastic hinge regions that is expected to yield under ULS design actions should be produced from a type of steel that has been demonstrated not to be susceptible to strain ageing. In New Zealand currently this will typically require the use of AS/NZS 4671 Grade G500E steel (SANZ, 2001).

When vertical wall reinforcement is spliced by means of staggered laps in potential plastic hinge regions the deformation capacity of the wall should be reduced to account for the negative effects of anticipated lap splice bond failure. The approach by Kerby et al. (2025) should be used.

Non-planar reinforced concrete walls (e.g. C, T, I or L shaped walls) should have sufficient reinforcement to ensure the ratio of $M_{n,wall}/M_{cr} \geq 1.2$, where $M_{n,wall}$ is the nominal flexural strength of the wall and M_{cr} is the cracking moment of the wall. M_{cr} should be determined using an average concrete tensile strength, $f'_t = 0.55\sqrt{1.2f'_c}$.

Foundation flexibility, including piles and the supporting soils with which they interact, should be included in the analysis when they significantly affect the dynamic properties of the building.

Buildings should not be founded on mixed foundation systems (i.e. piles and shallow foundations).

Shallow reinforced concrete pad foundations and pile caps to have interconnecting tie beams or equivalent capable of resisting 10% of $G&U$ vertical load acting on the foundation component, but not less than 150 kN. The tie element may be contained within a slab on grade provided the slab has positive connections to restrain the structure across all construction and movement joints.

When there is a potential for lateral spreading to occur in CALS ground motions, foundation tie elements should be sized to ensure the continuity of the foundation system is maintained.

Commentary:

Amendment 3 of NZS 3404 (SNZ, 2026) was expected to be published in mid-2025, however this process has been delayed with publication not expected until later in 2026. For the interim period until Amendment 3 of NZS 3404 is published the recommendations in Section 5 of the SESOC Interim Design Guidance (SESOC, 2022) should be adopted for LDS Category Level 1 buildings. The additional 'Damage Reduction Recommendations' in this document need not be considered.

The requirements for capacity design, regularity and for limiting the structural ductility factor used for the design of the primary structure levels, is intended to provide a higher level of confidence buildings designed to these criteria will perform well in significant earthquakes. For moment resisting frames the sway index check detailed in Section 6.4 can be used to demonstrate that a beam sway mechanism can be expected.

Where buildings do not meet regularity provisions, experience has shown these buildings typically perform poorly in earthquakes. For this reason, those buildings that do not meet the regularity limitations in TS 1170.5 have been excluded. The exception provided for vertical stiffness irregularities acknowledges the relatively common occurrence

of buildings having a ground floor height significantly higher than that used for levels one and two to accommodate the needs of ground floor retail tenancies. Capacity design is anticipated to mitigate potential negative affects related to such a vertical stiffness irregularity.

When assessing if an element or assembly is resisting more than 60% of the total lateral demand in a building in a given direction, accidental eccentricity need not be considered.

Testing completed by Pussegoda (1978) and Loporcaro et al. (2019) has confirmed low carbon AS/NZS 4671 G300E (SANZ, 2001) reinforcement is susceptible to strain-ageing. Effects of strain-ageing include an increase in the yield and ultimate tensile strength, and reduction in ductility, both of which can have a detrimental effect on seismic performance.

Mixed foundation systems, where part of a building is supported on deep piles and part on shallow foundations, have performed poorly in past earthquakes and are not recommended (CERC, 2012).

Interconnecting tie beams are intended to prevent lateral extension and dislocation of building foundations. Refer to Section 6.3 for recommendations on how to determine design actions on foundation tie elements when lateral spreading is anticipated.

5.3.3 Design Requirements for Non-Structural Elements

In addition to the requirements of the NZBC, non-structural elements (NSEs) should be designed and co-ordinated in accordance with the BIP Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025). This includes:

- Holistic Design Process that includes clarity of roles and responsibilities, identification of responsibilities at specific interfaces as well as early-stage multi-discipline coordination and potential multi-discipline iteration of proposed solution to achieve the holistic building performance objectives.
- Supporting the recommended design process, is the requirement to develop a Building Movement Strategy for the project which defines an agreed approach for accommodating building movements that occur during the day-to-day movements and function of the building, as well as movements that will occur to the building and components during an earthquake.
- Ensuring the project has a specialist NSE Seismic Designer to enable the design of the non-structural elements to be co-ordinated during the design phase; and to review the sub-contractor shop drawings to check seismic restraint design intent is considered and incorporated.
- Ensuring the main contractor has a NSE Seismic Coordinator, and there is a handover between the NSE Seismic Designer and the NSE Seismic Coordinator to confirm the NSE seismic strategy and key design considerations incorporated in the building design at the commencement of construction. The NSE Seismic Coordinator is to work with the sub-trades to coordinate the seismic design and installation of NSEs with the finalised sub-contractor designs.

Seismic design actions (accelerations and inter-storey deflections) for NSEs should be determined by the project structural engineer in accordance with Section 4.5 or 4.7. Design actions for ULS and SLS1 limit states (and SLS2 when required) should be provided.

Commentary:

The holistic design approach recommended in the BIP Code of Practice supports good early decisions which can make a significant difference to the overall seismic performance and resilience of buildings. The response of the structural system often dictates the seismic performance of NSEs, but without early consideration of NSE seismic

performance during the conceptual structural system selection, the seismic performance of the building may not be achieved or might require significant additional effort and cost to achieve.

A dedicated entity (NSE Seismic Designer) who is responsible for the coordination of all NSEs, and how the seismic detailing is coordinated within the design, is an important aspect to achieve the expected seismic performance for the building. It is also important the NSE Seismic Designer is not only involved in a review role during construction, but actively provides a brief to the NSE Seismic Coordinator as well as the main contractor, buildings services subcontractor, ceiling and partition subcontractors prior to construction commencing to discuss general principles to be followed, shop drawing process (if any), standard details to be used and agreed procedures for areas where standard details may not apply.

While the wider project team will contribute to the development of the Building Movement Strategy, and this strategy will inform the selection of the structural system, the structural engineer is responsible for determining and communicating the seismic design actions required for the design of non-structural components.

6 Design Requirements for LDS Level 2 and 3 Buildings

6.1 General

6.1.1 Section Overview

This section contains technical design criteria for building components which can be used by design teams to demonstrate a building design meets the requirements for LDS Category Level 2 or 3. Design requirements for LDS Category Level 1 buildings are detailed in Section 5.

The term building component has been adopted recognising a building is made up of a number of components, ranging from the foundations through to structural and non-structural elements, including building services items. Some items, such as landscaping around a building, and the building contents within, may not strictly be components, but are included for completeness.

Technical design criteria are provided for the following:

- Section 6.2 – General Requirements for Non-Structural Components
- Section 6.3 - Foundations
- Section 6.4 - Building Structure
- Section 6.5 - Building Envelope
- Section 6.6 - Building Services
- Section 6.7 - Stairs and Ramps
- Section 6.8 - Lifts
- Section 6.9 - Ceilings
- Section 6.10 - Partition Walls
- Section 6.11 - Passive Fire
- Section 6.12 - Building Contents
- Section 6.13 - Landscaping and External Services

The format of each building component section is set according to the following:

- Subsection 1, *Scope*, this describes the elements of the building the component section addresses.
- Subsection 2, *Detail and Interface Issues*, this section is intended to encourage good detailing and consultant collaboration. It is where common areas of failure, both in detail and at interfaces, are noted. It is also where examples of ‘what good looks like’ are given.
- Subsection 3, *Prescriptive Design Method*, this describes the prescriptive design criteria applicable for the building component.
- Subsection 4, *Direct Design Method*, this describes an alternative design criterion which can be used for building components that don’t meet the requirements of the prescriptive design method.

Commentary:

Technical design criteria provided in Section 6 are not intended to provide detailed guidance for the general design of buildings, or to replace existing building standards and guidelines. Rather the LDSD design criteria are intended to act as an overlay and provide designers with a methodology to achieve the LDSD outcome objectives and post event performance goals detailed in Volume 2.

6.1.2 The Importance of Good Coordination

The importance of early coordination is to minimise challenges often encountered in the later stages of the design and through construction. It has been shown that either delaying coordination, or not undertaking good coordination at all, often results in compromises to the seismic performance of components and ultimately the overall seismic performance of the building as well as resulting in potential project delays and increased costs (BIP, 2020).

Historically seismic restraints for non-structural elements (NSEs) were designed during the construction phase, with different seismic restraint engineers responsible for each sub-trade. This historical focus was solely on seismic restraints (life safety), rather than seismic performance of the building as a system (BIP, 2020).

LDSD requires coordination between all disciplines throughout the project lifecycle commencing early during the design phase. The level of detail and coordination to be provided by the design team at each phase of the project is detailed in Section 3 and in Section 6.1.3.

Design and documentation of NSEs will be provided through a combination of disciplines and subtrades. It is the responsibility of the disciplines responsible for each NSE (architect, building services, fire/passive fire engineer, civil) to consider in collaboration with the structural engineer (who will confirm structural response for various APoEs, i.e. inter-storey deflections and accelerations at various heights in the building) and the NSE Seismic Designer:

- The operational requirements for all NSEs;
- Ensure each NSE can be located where the discipline lead intends it to be;
- Achieve the required clearances;
- Meet the necessary performance requirements to achieve LDSD.

Early collaboration and coordination have other significant benefits beyond improving seismic performance, including significantly less clashes to be resolved on site, improve efficiency and lower project costs.

Development of the Building Movement Strategy (refer Section 3.2.2.3) is an important coordination requirement for every LDSD project, and is scalable from small simple LDSD projects up to large complex LDSD projects. The Building Movement Strategy forms part of the ongoing collaboration and coordination between disciplines where building and component movements are discussed and, where they occur, conflicting movement requirements agreed and documented. Note that there are often conflicting building/component movement requirements at various locations within buildings (e.g. gaps for acoustic/vibration control, verses no gaps for passive fire, or thermal movements of components required for day-to-day function of the building versus seismic restraint of components).

Further information about the Building Movement Strategy, including its scalability to small and larger projects, can be found in the Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025).

Finalisation of the design, and detailing of some NSEs, is still likely to be required during the construction phase. It is important to note design and coordination activities should still occur for these elements, but this final design needs to be checked or confirmed during the construction phase. This requirement should be clearly communicated to the relevant sub-contractors. A recommended approach to integrate the sub-contractor design of these non-structural elements and equipment selections with the overall seismic design and coordination of the building is provided in the Code of Practice (BIP, 2025).

6.1.3 Roles and Responsibilities

The roles and responsibilities documented in the NSE Code of Practice (BIP, 2025), should be adopted for LDSD projects.

Commentary:

The roles and responsibilities described and documented in the NSE Code of Practice (BIP, 2025) incorporates and expands the tasks and roles included in the 2023 NZCIC Guidelines. No additional roles and responsibilities were identified as being necessary for LDSD projects.

6.1.4 Designing for Inspection and Repairability

LDSB Category Level 2 and 3 buildings are expected to achieve *Operational State Category OS II - Partial Functionality* following a DCLS intensity earthquake provided wider impacts beyond the site do not prevent this from occurring (refer Volume 2). To enable the specified reoccupation timeframes to be met, designers should consider how key building components are to be inspected following a significant earthquake, how long the inspections are likely to take and the potential for disruption to building occupants should earthquake damage repairs be necessary.

To be considered LDSB:

- All key building components that may require inspection following an event must be accessible with no (or easily repairable) damage to any other element which has a design life of greater than five years (as defined by NZBC B2).
- All disciplines are to include a focus on repairability for all components associated with their discipline. This may involve consideration of alternative detailing that allows components to be easily repaired, or consideration of the best location for components for ease of access to undertake repairs.
- Holistic design is required when repair or replacement of damaged building components could impact other components. For example, repairing damage to a building services component or equipment may require partial or full removal of ceilings. However, if the component can be repositioned in an area where less damage to other components would be required to affect the repair (or replace the component) this should be done.

Commentary:

In practical terms, this means that access hatches, ladder access, 'viewing windows' etc, may be appropriate in locations where they may not have been required before. But it does not mean for example, that areas of suspended plasterboard ceilings or plasterboard partition walls require access hatches, because these linings are 'easy to access and replace' and therefore fall within the 5-year durability requirement of the Building Code.

6.1.5 Buildings with Multiple LDS Category Levels - Impact on Services, Structure & other NSEs

6.1.5.1 General

This section is to be used for LDS projects when the following circumstances are applicable:

- A building appears to be a single entity from an architectural, space planning and services perspective, but has been separated into seismically separate 'structures', for structural reasons, and the different buildings ('structures') have been assigned different LDS Category Levels (e.g. one building has no LDS rating whilst another linked/adjacent building is a LDS Category Level 1, 2 or 3 building).
- Where there are multiple buildings on a site which rely on common building services infrastructure, but the various independent buildings have different LDS Category Levels.

The impact of different LDS Category Levels can be complex in terms of the location, design, operation and maintainability of services infrastructure (plant, pipes, ducts, cables) that are common to more than one building. The appropriate services solution for the site may impact the seismic design of all buildings.

A collaborative process between the Building Services, Structural and Non-structural Seismic Designer is required. This includes consideration of the principles outlined below to holistically evaluate options, including relative merits vs costs (capital, operational, maintenance, overall robustness, durability, etc) to collectively determine the most practicable coordinated services, structural and NSE seismic performance solution for the project.

6.1.5.2 Damage Considerations for Building Services that are Common to Multiple Buildings

Where common building services run between multiple buildings that do not share the same performance requirements, damage to common services (e.g. plant failure, pipe failure, electrical fault) can disable that service(s) within a designated LDS building such that function is impaired in the LDS building and the service requires repair at a lower level of seismic shaking than required for the LDS building to meet its LDS performance requirements.

Examples of damage to services in a lower LDS Category Level building that compromises the required Operational Integrity of connected services within the higher LDS Category Level building can include:

- Plant failure within a lower LDS Category Level building (or where a building has no LDS rating) where such plant serves the higher LDS Category Level building.
- Pipe or duct failure, and consequent leakage of contents within the lower LDS Category Level building could compromise the operational integrity of connected services within the higher LDS Category Level building.
- Electrical fault within the lower LDS Category Level building could trip upstream circuit protection devices that could isolate power to services within the higher LDS Category Level building.

Failures described above could be caused by either:

- The seismic event exceeding the services restraint strength, the seismic capacity of plant or equipment or the displacement capacity of services within the lower LDS Category Level building, or
- Interactions between services and (potentially damaged) structure or other NSEs (e.g. ceilings, risers, services that pass-through partition walls or seismic gaps, etc) within the lower LDS Category Level building.

6.1.5.3 LDSD Design Requirements for Buildings Where Building Services are Common to Multiple Buildings

The risks of damage and loss of service discussed in the previous section are not avoided by simply assigning a higher LDSD Category Level to the building services components within the lower LDSD Category Level building. This does not address the potential for damage to services via interactions between services and structure, or other NSEs, that may not have been designed for the required performance requirements for the higher LDSD Category Level building.

The following two sections provide the LDSD design requirements for buildings where building services are common to multiple buildings.

Common buildings services plant

Where multiple buildings contain common building services plant (transformers, main switchboards, generators, fire pumps, water tanks, chillers, boilers, etc.) the LDSD performance requirements can be achieved using one of the following options:

- Assigning and designing all buildings to achieve the same LDSD Category Level as the most onerous building.
- Locate common services plant in the highest LDSD Category Level building (or another structure with the same LDSD Category Level).
- Separate services into discrete plant located within each respective LDSD Category Level structure i.e. do not have common building services for multiple buildings.

Common building services distribution infrastructure

Where multiple buildings contain common distribution infrastructure (pipes, ducts, cables) serving, or passing through, different buildings the LDSD performance requirements can be achieved using one or a combination of the following options:

- Assigning and designing all buildings to achieve the same LDSD Category Level as the most onerous buildings.
- Assess the potential for damage to services within the lower LDSD Category Level building. If the services are assessed to meet the performance requirements of the highest LDSD Category Level building this is Deemed to Comply. Such an assessment would require combined input from the building services, structural & NSE teams. Even then, such an approach is likely to include inherent risks that should be communicated to the client.
- Undertake 'enhanced design' of the building with lower performance requirements. Detail selected services (and those non-structural components that could interact with the selected services) such that the holistic design of the structure, building services and other non-structural elements will meet the performance requirements of the highest LDSD Category building.
- Provide automatic isolating/discrimination systems to avoid downstream damage compromising upstream functionality in the highest LDSD Category building. This strategy is not an appropriate solution for services that pass through a building with lower performance requirements on the way to a building with higher performance requirements and it may not be acceptable for fire water systems.
- Provision of dual redundant distribution, with at least one route entirely contained within the building with the most onerous performance requirements, combined with automatic isolating/discrimination systems to avoid a services outage if one or more of the other routes is damaged.

6.1.6 NSE Seismic Designer Engagement

A NSE Seismic Designer is required for all LDSD projects. The client should engage an NSE Seismic Designer as early as practicable (i.e. at the same time or soon after engagement of fire engineering, structural and building services consultants).

Refer to 'Design Methodology & Design Phases' in Part A of the Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025) which offers comprehensive guidance on navigating the Roles and Responsibilities of the NSE Seismic Designer and the design process from project briefing to construction phase design.

6.1.7 Building Functionality

LDSD Category Level 2 and 3 buildings are expected to achieve *Operational State Category OS II - Partial Functionality* following a DCLS intensity earthquake provided wider impacts beyond the building site, outside the control of the design team, do not occur (refer Volume Two). Wider impacts that could impact building functionality include public utility outages (i.e. power, water, wastewater and data), post-earthquake cordons established by emergency services limiting building access, and labour or material shortages. Volume Two also provides guidance on the expected physical state of building components following a DCLS intensity earthquake.

Critical non-structural components and systems necessary for *Operational State Category OS II* (or a higher Operational State Category if this is a specific project requirement) should be identified by the project team. A strategy should be established for each critical non-structural component and system to enable the specified operational state to be returned within the acceptable timeframe (refer to Volume Two Section 7.2 for more information).

For those projects when clients have a lower tolerance for loss of building functionality, consideration could be given to improving building system reliability by means of the provision of back-up systems or redundancy. This could include the provision of emergency power or on-site water storage.

6.2 General Requirements Applicable to Non-structural Elements

This section sets out LDSD recommendations that are applicable to many non-structural components.

6.2.1 Seismic Design Actions for Non-structural Components

Seismic design actions (accelerations and inter-storey deflections) for non-structural elements (NSEs) should be determined by the project structural engineer in accordance with Section 4.5 or 4.7. Design actions for ULS, DCLS and SLS1 limit states (and SLS2 when required) should be provided. The project structural engineer should also advise if peak floor accelerations exceed the limits specified in Section 6.4.3 thereby triggering seismic qualification requirements for acceleration sensitive components.

Commentary:

While the wider project team will contribute to the development of the Building Movement Strategy (refer to Section 3.2.2.3), and this strategy will inform the selection of the structural system, the structural engineer is responsible for determining and communicating the seismic design actions required for the design of non-structural components.

6.2.2 Classification of Parts and Components

Parts of structures and NSEs that have the potential to interact during an earthquake should generally be designed using the most critical TS 1170.5 parts and components category (refer TS 1170.5 Table 8.1) that is applicable to either component. In some cases, this may require a component to be designated a parts and components category that would otherwise not be applicable.

An exception to this requirement would be if the NSE Seismic Designer, and the relevant designers and seismic specialists responsible for each interacting component, determine via a holistic assessment this is not necessary. Furthermore, any adjustment of seismic performance criteria (e.g. building Importance Level, LDSD Category Level, selected APoEs) should be applied consistently across all structural and non-structural elements to maintain the hierarchy of building component seismic resilience.

Commentary:

Parts and components can have multiple TS 1170.5 parts categories, all of which need to be considered when they are designed or specified. For example, a building component could represent a life safety hazard (i.e. Category P2) requiring consideration of ULS, while at the same time being needed for operational continuity of the building (i.e. Category P5) which will require consideration of SLS2. SLS1 checks will also typically be needed for most components (i.e. Category P6 or P7).

6.2.3 Part Ductility Factors for Non-Structural Components

When determining horizontal seismic design actions on parts and components for all LDSD Level buildings, the following part ductility values, μ_p , should be used:

- SLS1: $\mu_p = 1.0$.
- SLS2: $\mu_p = 1.25$.
- DCLS: The recommendation for SLS2 should be applied.
- ULS: As per Table C8.3 of TS 1170.5 (SNZ, 2025).

Commentary:

TS 1170.5 Table C8.3 provides recommended part ductility values that reflect inherent damping and other nonlinear behaviour characteristic of NSEs (which can include such mechanisms as yielding of the part, fastener slips etc).

Larger part ductility values for parts and components may be able to be used for NSEs through the application of the Direct Design Method. The use of larger part ductility values would need to be justified by experimental testing, or similar as part of a special study completed in accordance with TS 1170.5 (SNZ, 2025).

6.2.4 Connections for Non-Structural Components

Designers should consider the potential for non-linear behaviour of the structure to which NSEs and seismic restraints are connected to. This is to ensure the integrity of the connections and related fasteners are maintained should the supporting elements of the structure crack or yield during an earthquake. Refer to Section C.4 of the Code of Practice (BIP, 2025) for more information.

Seismic design actions on non-ductile connections for parts and components for all LDSD Level buildings should be computed using a part ductility value, m_p , of 1.0 or derived using capacity design in accordance with TS 1170.5 Section 8.8.

Commentary:

Refer to TS 1170.5 for the definition of a non-ductile connection.

6.3 Foundations

6.3.1 Scope

This section sets out recommendations for LDSD of building foundations including:

- Site assessment – ground model and site response/performance.
- Selection of foundation system.
- Design of selected foundation system:
 - i. Shallow foundations.
 - ii. Deep foundations.
 - iii. Ground improvement.

This section includes geotechnical engineering, and the interface between geotechnical and structural engineering, but excludes the structural design of foundation elements such as piles. Refer to Section 6.4 Building Structure for the structural design of these elements.

Commentary:

This section is not intended to provide detailed guidance for the design of foundations or to replace existing standards and guidelines. Rather this section is intended to act as an overlay and provide designers with a methodology to achieve the LDSD outcome objectives and post event performance goals as detailed in Volume Two.

6.3.2 Detail and Interface Issues

6.3.2.1 General

Early engagement between the geotechnical and structural engineer, as well as the wider design team regarding, the foundation types most suitable for the ground conditions and proposed building is an important part of achieving LDSD (refer to Section 3). Historically, the geotechnical engineer has communicated with the project team through the structural engineer. It is now becoming common for the geotechnical engineer to directly communicate with the project team, including with the client, architect and structural engineer. This direct communication is considered important in all projects, and particularly for the case of LDSD projects to enable a holistic understanding to be developed and applied in building design.

Seismic geotechnical considerations that can impact the performance of foundations and buildings include:

- Vertical and lateral load-displacement behaviour of foundations.
- Liquefaction.
- Cyclic softening.
- Lateral spreading.
- Earthquake-induced settlement.
- Down drag on piles.
- Lateral soil loads.
- Fault rupture.
- Buoyancy effects.
- Slope stability.
- Rockfall.

The process for developing a good early understanding of the project is outlined below along with critical early decisions. Examples of detailing, coordination, and collaboration issues that may arise in the design and construction phase are also presented. Tasks and inputs from various team members will vary depending on the nature of the site and development.

6.3.2.2 Initial Understanding of the Project

Site selection and/or understanding site constraints is an important part of any project and particularly for LDS. The steps involved in site selection and/or understanding site constraints include:

1. The geotechnical engineer identifies and considers seismic geohazards and potential geotechnical issues. This will require a desktop study, refer to NZGS Module 2 (NZGS/MBIE, 2021).
2. The architect, geotechnical and structural engineer, and other team members, participate in a project briefing with the client.
3. During the briefing, the geotechnical engineer provides a representation of the ground model, (e.g. a cross-section sketch of the soil profile), and a high-level description of identified geotechnical issues, constraints and opportunities that could influence building design and behaviour. Ground behaviour is considered at all levels of shaking, including beyond ULS. Intensity of shaking to trigger any geotechnical 'step-change' is identified, refer to TS1170.5 (SNZ, 2025).
4. Similarly, other team members describe the constraints and opportunities their discipline presents to the project.
5. Desired performance objectives and the ability to achieve these are discussed by the project team, leading to the brief for the project.
6. The structural and geotechnical engineers discuss and agree the split of responsibilities to be applied through the design and construction process.

6.3.2.3 Critical Early Decisions

In the early phases of the project, structural and geotechnical engineers should:

- Review available geotechnical investigation information uncovered by the desktop study and consider the extent and timing of further investigations (refer to NZGS Module 2). Geotechnical investigations to

supplement the desktop study may be critical to undertake before the design team is brought together as the ground conditions could impact the type of development that could be undertaken at a site.

- Hold early discussions to allow the selection of compatible foundation and structural systems. The selection of the structural system must allow for any constraints that ground conditions may put on the performance or economics of the foundation system, e.g. the potential for a geotechnical step-change, or the ground conditions may dictate that resisting high concentrated uplift loads is not practical or economic. The selected foundation system must have a load displacement performance compatible with that of the structural system.
- Jointly select the foundation and structural systems in consultation with the client and architect.
- Discuss load paths in the structure, and load path from foundation to ground, including considerations of base shear take out options.
- Discuss structure and foundation behaviour and associated risks and uncertainty.

As part of the development of the Building Movement Strategy (refer Section 3.2.2.3) the design team should:

- Jointly consider tolerable differential foundation deformations. Also consider the tolerable differential displacements between the building and the adjoining ground. These assessments will be influenced by the type of; structure, building façade, architectural finishes, non-structural elements and service and access connections.
- Discuss the selection and detailing of pavements, hard landscaping and buried services to respond to the expected seismic performance of the ground relative to the seismic performance of the foundations and building. Refer to Section 6.13 for further information.

6.3.2.4 Preliminary and developed design

For the preliminary and developed design stages, the key detailing, coordination, and collaboration issues should be addressed by following the below process:

1. Jointly confirm tolerable differential foundation deformations and confirm tolerable differential displacements between the building and the adjoining ground.
2. Agree approach and parameters required for the design of the foundation system:
 - The geotechnical engineer needs to determine realistic geotechnical parameters (i.e. not conservative) and then apply sensitivity ranges on those parameters for determining foundation capacity and stiffness.
 - Vertical and lateral load paths between structure and ground are discussed and possible issues of foundation capacity and relative stiffness are explored. The geotechnical engineer presents to the structural engineer the range of possible load and displacement behaviour of foundations. The structural engineer presents the demand and tolerance of structure to displacement.
 - They then agree:
 - How this load and displacement behaviour is to be modelled, for example using foundation springs or integrated finite element modelling;
 - Parameters to be supplied by the geotechnical engineer, the nonlinearity and sensitivity of those parameters, and the form in which they are to be provided;
 - Acceptable displacement limits at SLS, DCLS, and ULS.
3. The geotechnical engineer provides the agreed parameters, and the structural engineer applies these in the structural analysis and design.

4. Then there is a feedback loop that involves:

- The structural engineer reporting back to the geotechnical engineer the conclusions of analyses based on the parameters provided by the geotechnical engineer.
- Discussion of what parameters were found to be critical.
- Confirmation or not that the critical parameters are appropriate and have been applied appropriately.
- An opportunity to review and refine parameters or how they are applied, and some design iterations undertaken to reach consensus.

6.3.2.5 Detailed design

Following the design development, the geotechnical engineer provides construction specifications for geotechnical aspects of the work. The geotechnical engineer also reviews geotechnical aspects of the structural engineers' drawings.

6.3.2.6 Construction

The geotechnical engineer undertakes inspections of ground conditions and geotechnical aspects of construction. The structural engineer undertakes inspections of structural aspects of the foundations. As built records are collated, particularly for buried elements.

Commentary:

Foundation design is necessarily a combined effort between the geotechnical and structural engineers. For each project the geotechnical and structural engineers should agree what aspects of the design will be led by geotechnical and which will be led by structural. All aspects of the design will need to be undertaken in a collaborative manner. This collaborative approach allows a holistic view to be developed and applied to the benefit of the project.

6.3.3 Prescriptive Design Method

NZGS Module 4 (NZGS/MBIE, 2021) should form the basis of foundation design to achieve LDSD and should be read in conjunction with this Section. The Module 4 guidance is not repeated here, but is built on to promote LDSD. The subsections below use the same main headings as used in Module 4. Commentary is provided in each subsection to build on the Module 4 content regarding LDSD and key sub-headings are used to link back to Module 4.

Commentary:

NZGS Module 4 has been issued under Section 175 of the Building Act and provides guidance for earthquake resistant foundation design. The guidance has been established to promote compliance with the NZBC. Module 4 also provides some commentary on design beyond the minimum requirements of the NZBC.

NZBC B1/VM2 provides information for foundation design but is not considered to be appropriate for LDSD projects. This is because the verification method has limited application and unlike NZGS Module 4 does not provide a procedure to determine foundation deformations or address liquefaction-prone sites.

6.3.3.1 Introduction (Earthquake Observations and Professional Collaboration)

Module 4 presents valuable information on observations of foundation performance in the 2010/2011 Canterbury earthquake sequence. This information, plus further information from Wellington observations following the 2016 Kaikōura earthquake, is important to consider as part of foundation system selection and assessment and is summarised below with an emphasis on LDSD. The majority of, but not all, foundation damage observed in recent earthquakes was related to liquefaction effects.

Shallow foundations:

- Variable ground conditions and liquefaction effects resulted in differential settlement.
- Stiff raft foundations performed relatively well.
- Ground improvement beneath shallow foundations reduced but did not remove differential settlements.

Deep foundations:

- Pile foundations generally performed better than shallow foundations.
- Differential settlements were observed as a consequence of pile tip punching through relatively thin founding layers, reduced shaft resistance due to liquefaction effects, and liquefaction and settlement of underlying soils.
- Observations of differential settlement of floor slabs on grade relative to piled buildings.
- Differential settlement of surrounding pavements, hard landscaping and facilities was observed relative to piled structures.

Hybrid foundations:

- Structures founded on a mixture of shallow and deep foundations generally performed poorly.

General foundation observations:

- Differential lateral displacement between structures and the surrounding ground surface - particularly for long buildings or for buildings on piles.
- Differential lateral displacement of structures poorly tied together.
- Heave of basement floors and other buried vessels due to liquefaction effects.
- The ability to repair buildings was severely constrained by the unknowns relating to the condition of foundations and their subgrades (including possible structural damage to piles, possible voids or loosened ground beneath foundations).
- The ability to repair buildings was severely constrained by the high cost of repair work to foundations.

A common theme between Module 4 and LDSD is the collaboration between the structural and geotechnical engineers, particularly during the critical early stages of selecting the primary structural and foundation systems. Considering options of foundation and structural systems in parallel presents the opportunity of selecting a holistic solution to the overall benefit of the project.

6.3.3.2 Site Geotechnical Model

Section 2 of Module 4 outlines the development of the site geotechnical model, usually graphically and in tabulated format, as well as the procedures for determining representative geotechnical design parameters. Module 2 (NZGS/MBIE, 2021) describes in detail the process for undertaking geotechnical investigations and testing to develop the geotechnical model and determine the design parameters. Refer to those documents to establish the geotechnical model for LDSD projects.

6.3.3.3 Foundation Performance Objectives for Earthquake Loading

Minimum design in accordance with the NZBC requires specific building performance requirements to be met at SLS and ULS. These code minimum foundation design requirements are detailed in Module 4.

Additional design requirements for LDSD are discussed here. Developing a foundation design with vertical and lateral stiffness which is compatible with that of the primary structure and the remainder of the building, and with

the overall performance objectives for the building, is critical for design and particularly LDSD. The structural and geotechnical engineers need to agree the building's tolerable limits for foundation deformation at each of the limit states. The building performance limit states to be considered in LDSD are:

- Serviceability (SLS);
- Damage control (DCLS);
- Ultimate (ULS).

In design, foundation and building performance is to be checked against agreed requirements at each limit state. For LDSD this includes a check at DCLS. In addition to these specific checks, foundation behaviour across the full range of possible earthquake shaking intensities, including between the limit states and beyond ULS should be considered.

Any potential step-change in the behaviour is to be identified. Step-change behaviour includes triggering of liquefaction, or slope instability, or other sudden adverse changes in foundation behaviour with increasing intensity of ground shaking. The level of earthquake shaking at which any identified step-change is triggered is to be evaluated and the consequences for building performance assessed. This triggering level and assessed consequences are to be discussed with the project team, including the client, and any mitigation agreed. For example, if a step-change is triggered at a level of shaking just above DCLS ground shaking intensity, this performance may not be considered acceptable and mitigation measures may be required.

This consideration of foundation behaviour between limit states, and beyond ULS, is discussed in Module 4 and the in TS1170.5 commentary with reference to Figure 6-1.

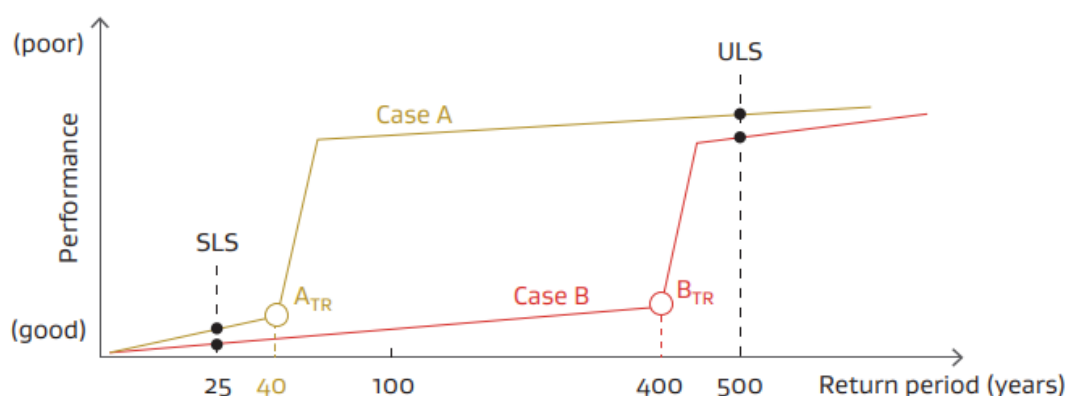


Figure 6-1: Step-change in performance between SLS and ULS (NZGS/MBIE, 2021)

Structures Deformation Limits and Estimating Foundation Deformations

Foundation deformation limits should be agreed and documented during the early stages of a LDSD project as part of the development of the Building Movement Strategy (refer Section 3.2.2.3). The importance of agreeing displacement limits is also emphasised by NZGS Module 4, as is highlighted below in the respective limit state subsections.

In estimating foundation deformations and stiffnesses the following should be considered:

- In predicting differential deformations/stiffnesses, allowance should be made for possible spatial variation of soils. Deformation potential could be greater at one location than another. Refer to the “Soil structure interaction” sub-heading below for further details.
- Best estimate parameters should be applied and the uncertainty in predicting deformations/stiffnesses should be captured by adopting appropriate sensitivity ranges (upper and lower estimates).

- With the sensitivity assessments, the stiff end of the possible range of stiffness is likely to be critical for building response and the soft end of the range of stiffness is likely to be critical for total deformations. Spatial variation from stiff to soft is likely to be critical for differential deformations. Refer to the “Soil structure interaction” sub-heading below for further details.

Serviceability Limit State (SLS)

With reference to Volume 2, for LDSD Category Level 2 and 3 buildings the Annual Probability of Exceedance for (APoE) adopted SLS for LDSD is 1/50. This varies from minimum NZBC requirements where an APoE of 1/25 is mandated.

NZGS Module 4 states: “The selection of settlement limit criteria for building foundations at the SLS should not be a decision of the geotechnical engineer in isolation, but should be agreed and documented with the structural engineer, architect and owner, as appropriate, depending on the structural form, building fabric, and intended use.”

Damage Control Limit State (DCLS)

Performance requirements (foundation displacement limits) for the DCLS are to be agreed by the structural and geotechnical engineers in consultation with the project team.

Ultimate Limit State (ULS)

Vertical and lateral displacement of foundations at ULS should be considered in addition to strength requirements as part of LDSD.

NZGS Module 4 states: “Tolerable limits for foundation deformation at the ULS will depend on the structural form of the building and the building response mechanism intended by the structural engineer. Deformation limits should be agreed and documented between the geotechnical engineer and the structural engineer.”

In accordance with TS 1170.5 and NZGS Module 4, consideration should also be given to foundation behaviour beyond ULS. This includes assessing the potential for, and consequences of, step-change behaviour beyond ULS. In line with the commentary for TS 1170.5, a shaking intensity up to 150% ULS should be considered and checks made that stability (maintaining gravity support) of the building is provided for at this intensity of ground shaking.

LRFD Design with Earthquake Loading

Under verification method B1/VM1, the design procedure specified for buildings in AS/NZS 1170.0 (SANS, 2011) is a strength based, Load and Resistance Factor Design procedure (LRFD). This procedure consists of a check on the capacity (strength) of foundations at ULS.

NZGS Module 4 provides guidance on selection of strength reduction factors to be applied in LRFD. These strength reduction factors apply to all ULS load cases, including load combinations with overstrength factors determined using capacity design. Guidance in Module 4 allows strength reduction factors to be increased with pile load testing or other means to reduce uncertainty.

In addition to the guidance in Module 4, for LDSD strength reduction factors of greater than 0.7 are not permitted. This is to further reduce the risk of large or less predictable displacements.

For LDSD in addition to this capacity check, displacement potential at ULS is to be considered relative to the LDSD building performance requirements.

Soil Structure Interaction

Soil structure interaction (SSI) aims to model the soil, foundation, and structure as a single system. The introduction of SSI generally “softens” the building response and lengthens the building period, usually resulting in reduced spectral accelerations but possibly at the cost of increased displacement. FEMA P-2091 (FEMA, 2020) provides useful practical guidance for SSI, but consideration should be given for the applied structural performance and ductility factors adopted in the structural design (refer to Section 6.4).

The behaviour of the foundation elements is often modelled as soil springs in SSI analysis. It is important to consider the potential change in soil response (i.e. stiffness and capacity) due to seismic shaking and consider this when determining foundation springs.

For LDSD, foundations should be robustly tied together to resist lateral soil structure interaction effects (refer also related requirements in Section 6.4). Raking piles should be avoided as they have been frequently observed to perform poorly in previous earthquakes.

Given differential foundation settlement is a critical contributor to foundation damage, the following guidance is provided on selection of vertical spring stiffness values and application to soil structure interaction analysis:

1. The structural and geotechnical engineer should discuss and agree how foundation stiffness values are to be assessed and applied. The stiffness values must be relevant to the load range of interest in the structural analysis and the proposed dimensions of the foundations. The level of uncertainty in the values must be understood. How performance beyond the elastic range is to be modelled is to be agreed, for example using of bilinear or elastic-plastic springs.
7. Spring stiffness values can be calculated by assessing the foundation displacement under the load of interest and dividing the load by the displacement. A range of values from the softest to the stiffest expected should be assessed and reported. The common ‘rule of thumb’ 50% and 200% range of stiffness can be considered but should also be critically evaluated for the likely expected range, with consideration of best estimate (rather than conservative) soil parameters as the baseline.
8. The stiff end of the assessed range of spring stiffnesses should be assumed when assessing the natural period of the structure that that is likely to result in the highest seismic (force) demand, i.e. a stiffer response, lower natural period and therefore generally higher seismic demand. Or other adverse combination.
9. The soft end of the assessed range of spring stiffnesses should be assumed when assessing the potential of differential foundation displacements. Differential displacement due to possible variable ground conditions should also be considered, e.g. a soft or hard spot. In developing scenarios to model the effects of variable ground conditions, the geotechnical engineer needs to consider the geological conditions and over what area a soft or hard spot could apply. The structural engineer needs to identify the critical locations within the building footprint with respect to tolerance to differential vertical displacement.
 - a. For individual pile or shallow pad foundations, to test for a soft spot, model all foundations as stiff springs except for the one which is modelled as soft. This soft spring is to be located in the most adverse location for the structure. The value of the soft spring is at the bottom end of the assessed range of stiffness, and the stiff spring the top end. The reverse approach can be applied to test for a hard spot.

- b. For the case of raft foundations, the soil reaction is typically modelled as a grid of springs. Spring stiffnesses are normally reported as pressure (rather than load) divided by displacement. The structural engineer converts this to load/displacement on the basis of the spacing of springs used in the structural analysis. For the raft, a soft or hard spot will be modelled as a zone of soft or hard springs rather than as an individual spring. The area of this zone will be selected by the geotechnical engineer considering the area of influence of the raft loading and the geology and associated soil variability.
10. Once the spring stiffness values, and various scenarios have been applied to the structural analysis, the structural engineer is to discuss the outputs with the geotechnical engineer and a check made that the springs are appropriately modelling the expected range of ground behaviour.

Commentary:

When considering the effects of liquefaction on seismic soil-foundation response, it is generally considered overly conservative in pseudo-static analysis to adopt the peak seismic shaking at the same time as adopting liquefied soil strength parameters and liquefaction induced foundation settlement. This is because time is required for pore water pressures to be generated to trigger liquefaction, and the peak seismic shaking generally occurs early in the earthquake time history before these pore pressures are sufficiently generated.

The NZTA Bridge Manual (NZTA, 2022) provides some guidance on this in Section 6.3.5 where three scenarios are considered:

- i. Peak seismic shaking preceding liquefaction developing.*
- ii. 80% of peak seismic shaking with 100% kinematic soil loads and fully degraded soil strength and stiffness parameters.*
- iii. 0% - 25% of the structure inertia forces with lateral spreading displacement loads.*

The focus of Bridge Manual Section 6.3.5 is for cyclic ground displacement and lateral spread and associated kinematic loading on piles, however, a similar concept can be considered for the effects of liquefaction induced vertical effects on piles and shallow foundations. For vertical bearing capacity and displacement scenarios (i) (peak shaking preceding liquefaction) and (ii) (100-80% peak shaking and fully degraded soil strength and stiffness), should be considered. Scenario (iii) should consider 0%-25% shaking in conjunction with post liquefaction ground settlement down drag effects on piles.

Further research and guidance is required in this area, however, engineering judgement can be applied in selecting the load/displacement combinations for application to LDSD with consideration of the design magnitude of shaking and therefore the expected duration of shaking as well as the consequence of liquefaction induced ground softening and displacement on the performance of the foundation and structure, particularly differential settlement of the foundation.

6.3.3.4 Site Assessment and Foundation Selection

Assessment of the response of the site soils, and the overall site to earthquake shaking, is critical for good foundation performance and achieving LDSD. “Most observations of poor foundation performance during earthquakes have been associated with ground failure including liquefaction or cyclic softening of the site soils and lateral spreading effects” (NZGS Module 4). In accordance with NZGS Module 4, the following matters for site assessment and foundation selection require careful consideration:

1. Soil response – liquefaction, cyclic softening, cyclic and yield displacements, etc.
2. Site performance – severity of liquefaction, lateral spreading, settlement, and instability

3. Building interaction effects – does the building exacerbate the effects of ground failure?
4. Foundation suitability – different types of foundation having specific requirements and considerations.

A useful flow chart in NZGS Module 4 (Figure 4.1) provides a strategy for site assessment and foundation selection, working through whether liquefaction, cyclic softening, lateral spreading, and other stability issues affect the site and how the geotechnical engineer and the design team may select the appropriate foundation solution.

Soil Liquefaction

The possibility of liquefaction and consequences for the foundations and building needs to be considered over the full range of earthquake shaking intensity (SLS, DCLS, and ULS). Shaking intensity beyond the ULS event also needs to be considered in terms of building stability (refer Section 6.3.3.3).

Lateral Spreading

We may not be able to reliably predict the magnitude and distribution of lateral spread across a site, but for the purposes of design the geotechnical engineer can develop a series of potential lateral spread scenarios to be applied to the structural design. These scenarios should be developed in consultation with the structural engineer with the objective of identifying possible scenarios which could be most damaging to the proposed structure and mitigating these in the design.

Each lateral spreading scenario is to define an assumed magnitude and distribution of lateral spread plus the lateral stiffness and capacity of the interaction between the soil and the structure. The potential stabilising effect of existing structures (e.g. basements, piled buildings, seawalls) should be considered and the resulting influence on the subject site and proposed building foundations assessed. The structural and geotechnical engineers can then quantify possible design actions in the structure due to lateral spreading and allow for these in design (e.g. additional tie forces in ground beams and bending moments in piles). Both lateral and vertical distribution of the lateral spread is to be considered. These scenarios are often best communicated via sketches.

NZGS Module 4 provides guidance for design of deep foundations allowing for the combination of inertia loads from the building (base shear) and kinematic soil loads on the piles (due to lateral spread or cyclic displacement). However, at high levels of shaking when significant nonlinear site response is expected and potentially additional lateral soil loading on foundations, alternative solutions should be considered beyond those in Module 4 (refer to Section 6.3.4 Direct Design Method).

Other Site Stability Issues

In addition to liquefaction and lateral spreading issues, earthquake shaking may also significantly increase the risk of other types of instability on sites with:

- Sloping ground.
- Cut batters.
- Fill batters.
- Retaining structures.

“Assessment of potential hazards needs to include a wider perspective than just the immediate boundaries of the site” (NZGS Module 4), and further information is provided in Section 3 regarding site selection and Section 4 regarding other seismic hazards. Requirements for retaining structures are detailed in Section 6.13 along with ground performance at and beyond the interface between the building and the surrounding ground.

Natural Raft

Where a thick layer of non-liquefying soils is present at the ground surface and creates a 'natural raft', shallow foundations present an effective option for LDSD as it potentially decouples ground movements from deformation of the foundation and building structure. The surface crust must be thick enough and strong enough to bridge over the underlying liquefiable soils and the properties of the crust must be consistent across the site and building platform. For multi-storey buildings (including those with basements), a raft/mat foundation should be used, and these should be capable of resisting buoyancy forces from high pore-water pressures from underlying liquefied soil.

Where there is uncertainty in the consistency of a 'natural raft', an artificial raft may be created using ground improvement as discussed below.

Ground Improvement

Ground improvement has benefits over the alternative of piling in that potential differential settlement and lateral displacement between building and surrounding ground may be reduced and, as a consequence, less potential damage at the building interface. However, if the depth and extent of ground improvement is significant, potential impacts of the ground improvement on seismic site response should be considered, particularly at high levels of shaking.

Design of ground improvement needs to consider the following relative to the LDSD objectives for the project:

- Uncertainties in the degree of improvement which can be achieved
- Magnitude of potential settlement of the improved ground.

NZGS Module 5 (NZGS/MBIE, 2021) provides guidance on ground improvement design.

Shallow and Deep Foundation Requirements

Selection of the foundation system should be undertaken in parallel with selection of the structural system (refer to Section 3.2.2.2). This selection of foundation and structural systems is to be undertaken in collaboration between the structural and geotechnical engineers, in consultation with wider members of the project team, as part of the development of the Building Movement Strategy (refer to Section 3.2.2.3). Considering foundation system options in parallel with wider project requirements presents the opportunity

Along with the requirements outlined in NZGS Module 4 for shallow and deep foundations, the LDSD merits of shallow foundations, deep foundations and ground improvement detailed in Table 6-1 should be considered as part of the selection of the foundation system.

Table 6-1 LDSO merits of shallow foundations, deep foundations and ground improvement

Foundation Component	LDSO Considerations
Shallow foundations	Less potential differential between building and surrounding ground than deep foundations.
	Stiff raft foundations may mitigate differential settlement and consequences of lateral spread.
	Resin injection technology is available to repair settled shallow foundations.
Deep foundations	Generally, less total settlement potential than shallow foundations and ground improvement.
	Risk of unseen structural damage to piles in an earthquake (but potential to mitigate with instrumentation installed within piles such as optical strain fibres or grout tubes).
	Expensive to repair.
	Suspending the ground floor slab may be warranted if there is ground surface settlement potential.
	Settlement potential between building and surrounding ground/pavements/buried services.
Ground improvement	Improvement of performance of the site beyond the building footprint possible.
	Potential influence on seismic site response and design spectra if ground improvement deep and extensive, particularly at high levels of shaking.
	Some settlement potential is likely to remain after ground improvement; the magnitude of this will vary depending on the ground improvement technique used.
	Can be used in conjunction with shallow or deep foundations.

6.3.3.5 Shallow Foundation and Deep Foundation Design

Section 5 and Section 6 of NZGS Module 4 present simplified approaches for practical seismic design of shallow and deep foundations, respectively. These approaches are appropriate for LDSO. However, in LDSO of foundations, the following should also be considered for both shallow and deep foundations:

Damage control considerations:

- For each limit state design to provide lateral and vertical deformation potential less than the respective tolerance of the structure and other building components and meet the project performance objectives.
- Earthquake induced building settlements relative to surrounding ground should be limited so that immediate foundation levelling post event is not required, and so that the potential for flooding from overland flow is not significantly increased (refer also Section 6.13).
- Liquefaction induced buoyancy/uplift effects from high pore pressures should be allowed for in the design of foundations to avoid failure of ground floor and basement slabs.
- Lateral and vertical deformation potential between the building and surrounding ground/pavements/underground services should be allowed for (refer also Section 6.13).
- Buried elements are difficult/expensive to repair and therefore a higher level of robustness should be allowed for in their design. This includes limiting ductility demands on pile design (refer also Section 6.4.3.5).

- Foundations should be robustly tied together (refer also Section 6.4.3.5).
- Consider the use of lower strength reduction factors in geotechnical design and/or more realistic (i.e. not conservative) parameters with suitable sensitivity range to make deformations more predictable to provide greater redundancy.
- Consider what part of the foundation system is most vulnerable to damage and how the robustness of that part could be improved.

System repairability:

- Damage to foundations and their subgrades as a consequence of an earthquake is difficult to detect. Investigate details which could be included in the design and construction to facilitate detecting damage (e.g. access conduits down piles for sonic testing, or building instrumentation).
- Consider how foundation damage could be repaired and incorporate features into the design to facilitate this (e.g. if there is settlement potential of shallow foundations, consider where resin/grout injection points to relevel would be practical and, if necessary, modify the design to facilitate this).

Durability and maintenance:

- The foundation system should comply with the durability requirements of the NZBC Clause B2.
- A maintenance schedule for the foundation system should be developed for the building owner (refer also Section 7.4). As a minimum the maintenance schedule should include:
 - i. Any maintenance program.
 - ii. Any recommended periodic inspections.
 - iii. Recommended post-earthquake damage inspections and testing, including description of any details of the foundation design which have been incorporated to facilitate inspections or repair, e.g. conduits down piles for sonic testing.
 - iv. As built records of foundations and records of ground conditions.

Commentary:

NZBC Clause B2 specifies a minimum design life for durability of 50 years for foundation components providing structural stability to buildings. Designers are required to demonstrate that such foundation components meet this requirement.

Foundations are generally not accessible for inspection, maintenance or repair and therefore design and construction of the foundation system should be robust providing durability and resilience beyond minimum code requirements.

6.3.4 Direct Design Method

The Direct Design Method may be appropriate for foundations that do not meet the Prescriptive Design Method criteria detailed in Section 6.3.3. This includes new or innovative foundation systems, or when performance-based design procedures are adopted to better assess anticipated building performance.

It is recommended performance-based design be considered for scenarios with high levels of ground shaking when significant nonlinear soil-foundation response is expected. Such an approach generally requires more sophisticated modelling of building response, including rigorous modelling of non-linear soil-foundation interaction, and close collaboration between geotechnical and structural engineers. This may be an attractive approach for LDS as the performance, and potential damage, may be more readily quantified.

From a geotechnical perspective, one approach that shows promise is to replace site soils beneath a shallow foundation with engineered soils with more uniform and predictable characteristics. This enables decoupling of

building and ground performance. However, consideration needs to be given to how the ground improvement affects seismic site response, potentially in a negative way, if the depth of ground improvement is significant or the extent of ground improvement across the site is large.

Other alternative approaches include utilising nonlinear ground response analysis to better understand site specific seismicity and then account for potential beneficial effects of soil yielding; and/or allowing shallow foundations, particularly raft foundations, to slide and reduce seismic actions on the structure and allow for releveling post event.

Uncertainty in the earthquake loading must be accounted for in these alternative solutions as well as uncertainty in foundation performance and soil response (via a parametric study and sensitivity analysis). However, variability in the earthquake loading may be more significant than the variability in the soil response and foundation performance (Tow, Pender, & McCully, 2011).

6.4 Building Structure

6.4.1 Scope

This section sets out recommendations for LDSD of the following building structure components:

- Primary lateral and vertical force resisting systems.
- Energy dissipating devices.

The scope here does not include detailed design guidance for specific lateral force resisting systems.

Commentary:

This section is not intended to provide detailed guidance for the design of low damage lateral force resisting systems, or to replace existing building standards and guidelines. Rather this section is intended to act as an overlay and provide designers with a methodology to achieve the LDSD outcome objectives and post event performance goals as detailed in Volume Two.

6.4.2 Detail and Interface Issues

Selecting a structural system that will enable wider project performance requirements to be achieved is critical for LDSD and requires early discussions with the project team and the development of a Building Movement Strategy (refer to Section 3.2.2.3). This should include discussions with the architect and façade engineer regarding the building form, emphasising the importance of a regular building form for LDSD projects. Similarly, discussions between the structural and geotechnical engineers should allow the selection of compatible foundation and structural systems (refer Section 6.3).

Deformation compatibility between the primary and secondary structural systems, and other non-structural elements (NSEs), should be considered in the building design to ensure damage associated with the deformation of the primary structure is minimised to acceptable levels. An example of this would be consideration of the elongation of rocking elements, such as walls, beams and columns; and the damage this might cause to floors, facades etc.

As noted in Section 1.4.2 this document generally recommends a LDSD philosophy of providing stiff, regular, conventional lateral load resisting systems to mitigate damage to primary structure and deformation sensitive secondary and non-structural elements. However, it is acknowledged for regions subject to very high levels of seismic shaking (i.e. Wellington and Wairarapa) more sophisticated structural systems may be required when higher levels of seismic performance are being sought.

Conventional structures that contain potential plastic hinge zones that are expected to yield and sustain large inelastic deformations under DCLS design actions should be avoided. This is because conventional lateral force resisting systems which experience high ductility demands are also expected to sustain significant damage and residual deformations which may not be economically repairable following a large earthquake.

Lateral force resisting systems containing energy dissipating devices that can be unbolted and replaced following large earthquakes have LDSD performance advantages over conventional systems where this is not typically possible. Replaceable energy dissipating devices should be subject to sufficient testing and validation to ensure they perform reliably over the range of earthquake shaking to which the building may be subjected.

The primary structure needs to comply with the durability requirements of the Building Code to ensure the building can perform reliably over its economic life. This includes energy dissipating devices and post-tensioning systems which may have specific inspection and maintenance requirements that need to be communicated to the building owner. Where access is needed to monitor or maintain components of the primary structure this should be provided for in the building design (refer Section 6.1.4). The design team should aim to facilitate easy post-

earthquake inspection of those parts of the building that are designed to undergo large deformations during an earthquake.

6.4.3 Prescriptive Design Method

In addition to the requirements of the NZBC, the primary structure is to be designed in accordance with the material standards cited in NZBC B1/VM1 with the additional design criteria given below.

The structural ductility factor, μ , used for the design of the primary structure should not exceed the maximum values detailed in Table 6-2. Further limitations for specific structural systems are defined later in this section. The structural performance factor, S_p , for the DCLS should be determined in accordance with the methodology used in TS 1170.5 (SNZ, 2025) for the ULS.

Table 6-2 Maximum structural ductility factor

Building Limit State	Maximum structural ductility factor ¹
SLS1	1.0
DCLS	2.0
ULS	3.0

Notes:

1. As defined in TS 1170.5 Section 4.3 for use with equivalent static and modal response analysis method

Member design capacities for DCLS design procedures should be calculated using strength reduction factors and nominal material strengths as detailed in AS/NZS 1170.0 (SANZ, 2011) for the ULS.

Peak floor accelerations (PFAs) for the DCLS should generally be less than 1.25g and should not exceed 1.6g unless the seismic performance of acceleration sensitive components required to achieve Operational State II – Partial Functionality have been validated for higher shaking intensities (refer Section 6.6.2.14). PFAs should be calculated using TS 1170.5 Eqn 8.1 with the part spectral-shape coefficient, $C_i(T_p)$, and the part horizontal response factor, C_{ph} , taken equal to 1.0.

Except for single storey buildings, or the uppermost storey of multi-storey buildings, capacity design should be used to ensure that a suitable sway mechanism develops under very rare shaking levels. Potential plastic hinge regions should be detailed in accordance with the relevant material standard, and the additional LDSD requirements as defined in the following sections, to ensure they have adequate deformation capacity to reliably resist anticipated deformation demands.

Structures should not have vertical or plan irregularities as defined in Section 4.5 of TS1170.5, except that a vertical stiffness irregularity is permitted for structures when capacity design has been used to provide a level of protection against the formation of a column sway mechanism.

No single line of lateral force resistance to have a single element or assembly resisting more than 60% of the total lateral demand in the building in that direction. When assessing if an element or assembly is resisting more than 60% of the total lateral demand in a building direction accidental eccentricity need not be considered.

Peak and residual inter-storey deflections for DCLS load combinations, including consideration of foundation flexibility, should not exceed the limits detailed in Table 6-3. Peak inter-storey deflections should be calculated in accordance with Section 7.3.1 of TS 1170.5.

Table 6-3 DCLS inter-storey deflection limits

Building Inter-storey Deflection	Inter-storey Deflection Limit ¹
Peak Inter-storey Deflection ²	0.5%
Residual Inter-storey Deflection	0.5%

Notes:

1. Inter-storey deflection limit is expressed in terms of the percentage of the corresponding storey height.
2. Peak inter-storey deflection limit can be increased by means of the Direct Design Method (refer Section 2.4.2).

Allowance should be made for the deformations arising from member elongation for DCLS load combinations.

All elements that may require inspection following a DCLS event must be accessible, refer to Section 6.1.4 for more information).

Foundation flexibility, including piles and the supporting soils with which they interact, should be included in the analysis when they significantly affect the dynamic properties of the building.

Commentary:

This document aligns with recommended industry practice (NZSEE, 2022) whereby designers should deliberately proportion structures with redundancy and enough regularity so that it is possible to identify a clear plastic mechanism. The latter will enable successful implementation of capacity design which will deliver robust structures that can be expected to perform well even when subjected to stronger than expected ground shaking.

Loss modelling studies undertaken to date (refer Appendix C) to validate the LDSD design criteria detailed in this document confirmed the recommended approach of controlling building damage by limiting building deformations and bracing non-structural components is effective for low to significant levels of ground shaking. However, under higher levels of ground shaking, acceleration sensitive components can be susceptible to shaking damage and this could result in building damage which exceeds that specified in Volume Two.

Results of the loss modelling studies show considerable variability across lateral force resisting system type and building location. Other studies (John, Williamson, & Sullivan, 2023) have also demonstrated building height influences building damage. When PFAs exceed 1.25g there is an increased risk the damage control performance goals specified in Volume Two might not be achieved and designers are advised against exceeding this acceleration.

When PFAs exceed 1.25g design teams should consider alternative lateral force resisting systems which are characterised by lower floor accelerations or, consider limiting the specification of acceleration sensitive components to components that have been validated for the levels of shaking expected. The 1.6g limit is a 'hard' limit which is not to be exceeded for LDSD Category Level 2 and 3 buildings unless the performance of acceleration sensitive building components is validated i.e. through seismic verification or similar (refer Section 6.6.2.14).

The requirement for capacity design, and limiting the structural ductility factor used for the design of the primary structure to be not greater than 2 at DCLS load levels, is intended to provide a high level of confidence that (1) any damage sustained to the primary structure is economically repairable and (2) the building will not sustain safety-critical structural damage that would require repair before the building could be reoccupied. Additional system specific local deformation checks are also included in subsequent sections to mitigate the localisation of inelastic deformation.

For moment resisting frames a beam sway mechanism may be assumed when the following equation is satisfied:

$$\sum M_{n,col} \frac{L_{col}}{L'_{col}} \geq 1.15 \sum M_{n,beam} \frac{L_{beam}}{L'_{beam}} \quad \text{Eqn 6-1}$$

where:

$\Sigma M_{n,beam}$	=	sums of the nominal flexural strength of the beams at the faces of the beam column joint zones in the level being considered
$\Sigma M_{n,col}$	=	sums of the nominal flexural strength of the columns at the faces of the beam column joint zones in the level being considered
L_{beam}	=	centre to centre spans of the beams
L'_{beam}	=	clear spans of the beams
L_{col}	=	centre to centre spans of the columns
L'_{col}	=	clear height of the columns

The sway index concept was originally developed by Priestley (1996) for reinforced concrete moment resisting frames. Consideration should be given to biaxial bending when calculating the nominal flexural strength of columns, $M_{n,col}$, that act as part of two-way frames. When calculating the nominal flexural strength of beams, $M_{n,beam}$, the contribution of slab reinforcement when present should be included in accordance with the relevant material design standard.

Where buildings do not meet regularity provisions, experience has shown these buildings typically perform poorly in earthquakes. For this reason, those buildings that do not meet the regularity limitations in TS 1170.5 have been excluded from the Prescriptive Design Method pathway. The exception provided for vertical stiffness irregularities acknowledges the relatively common occurrence of buildings having a ground floor height significantly higher than that used for levels one and two to accommodate the needs of ground floor retail tenancies. Capacity design is anticipated to mitigate potential negative affects related to such a vertical stiffness irregularity. In this instance Equation 6-1 can be used to demonstrate whether a column sway mechanism is expected or not.

Drift limits specified in Table 6-3 are not intended to mitigate damage to deflection sensitive elements such as conventional partition walls, however, overall building damage is expected to be controlled such that the building damage performance goals detailed in LDSO Volume Two are met. Limited research is currently available on the seismic performance of apartment buildings and the drift limit in Table 6-3 for apartment buildings should be considered tentative.

Peak inter-storey deflection limits in Table 6-3 can be increased by means of the Direct Design Method pathway by demonstrating potentially affected deformation sensitive components (e.g. partition walls, facades, precast floor units etc.) are able to accommodate the increased deformations without damage that would impact LDSO performance goals (refer Section 2.4.2).

Residual inter-storey deflections are limited to 0.5% of the corresponding storey height to ensure building repair is possible (FEMA, 2018). This limit is consistent with work completed by McCormick et al. (2008). The limit has been derived with consideration given to both human comfort, building functionality and building safety. Buildings with residual drifts exceeding this limit may not be acceptable to occupy following an earthquake. In many cases it may not be practicable to straighten a building following a major earthquake.

Residual inter-storey deflections can be calculated in accordance with FEMA P-58-1 Section 5.4 (FEMA, 2018). However, note that if a maximum drift of 0.5% is maintained then the residual drift limit can be assumed met and will not be critical.

NZS 3101 (SNZ, 2017) provides guidance which can be used to compute member elongation for DCLS load combinations.

When the geotechnical investigation identifies the potential for liquefaction, the effects of liquefaction on the building and its contents should be mitigated to ensure the performance objectives specified here are met. Refer to Section 6.3 for further information.

Building parts should be designed for the earthquake actions, deflection induced actions and differential displacements detailed in Sections 4.5 or 4.7.

6.4.3.1 Additional Design Criteria for Reinforced Concrete Buildings

Reinforcement used in potential plastic hinge regions that is expected to yield under ULS design actions should be produced from a type of steel that has been demonstrated not to be susceptible to strain ageing. In New Zealand currently this will typically require the use of AS/NZS 4671 Grade G500E steel (SANZ, 2001).

Structures should generally be proportioned to avoid localisation of inelastic deformation to mitigate bar buckling and fatigue damage. For the case of reinforced concrete coupling beams, where some localisation of inelastic deformation is unavoidable, the recommended deformation limits detailed in Table 6-4 should not be exceeded for DCLS design actions. The deformation limits in Table 6-4 are in terms of the equivalent chord rotation (combined elastic and inelastic) as illustrated in Figure 6-2.

Table 6-4 Recommended DCLS deformation limits for reinforced coupling beams

Component Description	L_n/h_b^1	Acceptable deformation (rad) ²
Diagonally reinforced coupling beams ³	≤ 0.5	0.011
	≥ 3.5	0.032
Conventionally reinforced coupling beams ⁴	≤ 1.0	0.012
	≥ 5.0	0.020

Notes:

1. L_n is the clear span of the coupling beam and h_b is the height of the coupling beam.
2. Linear interpolation between values listed in the table is permitted.
3. Designed and detailed in accordance with NZS 3101 Cl 11.4.9.3.
4. Designed and detailed in accordance with NZS 3101 Cl 11.4.9.5

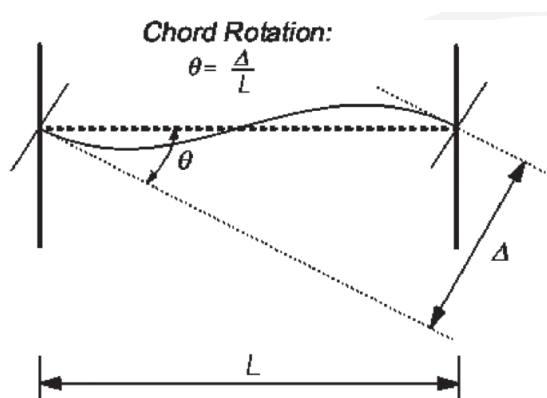


Figure 6-2 Rotation definition for a coupling beam (ACI, 2022)

Potential plastic hinge regions in reinforced concrete buildings should as a minimum be detailed as limited ductile plastic regions in accordance with NZS 3101 (SNZ, 2017). Centre-to-centre spacing of stirrup-ties in potential plastic regions should not exceed four times the diameter of any longitudinal bar to be restrained in the outer layers.

Unless the structural analysis indicates that flexural cracking is likely to occur in a structural member for DCLS load levels, either gross section properties, or transformed section properties, should be used. When flexural cracking is expected to occur in components, other than reinforced concrete walls, effective section properties

should be determined in accordance with NZS 3101 Section 6.9.1 The effective stiffness of reinforced concrete walls, El_{eff} that are expected to crack at their base should be determined using the following moment curvature relationship:

$$El_{eff} = \frac{M_{n,wall}}{\phi_y} \quad \text{Eqn 6-2}$$

where:

$M_{n,wall}$ = nominal flexural strength of the wall.

ϕ_y = yield curvature of the wall.

The effective stiffness computed using Eqn 6-2 can be assumed to be constant over the full height of the wall.

When vertical wall reinforcement is spliced by means of staggered laps in potential plastic hinge regions the deformation capacity of the wall should be reduced to account for the negative effects of anticipated lap splice bond failure. The approach by Kerby et al. (2025) should be used.

Non-planar reinforced concrete walls (e.g. C, T, I or L shaped walls) should have sufficient reinforcement to ensure the ratio of $M_{n,wall}/M_{cr} \geq 1.2$, where $M_{n,wall}$ is as defined above and M_{cr} is the cracking moment of the wall. M_{cr} should be determined using an average concrete tensile strength, $f'_t = 0.55\sqrt{1.2f'_c}$.

Upper limit shear design actions on reinforced concrete walls should be determined as 1.5 times the ULS seismic design action with $\mu = 1.25$ and $S_p = 0.9$.

Singly reinforced concrete walls are not permitted.

Precast concrete hollowcore or web supported double tee floor systems should not be used. Where other precast floor systems are used, peaked inter-storey deflections for DCLS load levels should be limited to 1.0%, unless it can be demonstrated the floor system is able to accommodate increased deformations without damage that would impact on the DCLS performance goals.

When precast floor systems are supported on reinforced concrete ledges, structural steel armouring should be provided to prevent spalling of the ledges.

Ductile jointed precast concrete systems designed in accordance with NZS 3101 Appendix B (SNZ, 2017) are not considered to be low damage structures when following the Prescriptive Design Method pathway in this document.

Commentary:

Testing completed by Pussegoda (1978) and Loporcaro et al. (2019) has confirmed low carbon AS/NZS 4671 G300E (SANZ, 2001) reinforcement is susceptible to strain-ageing. Effects of strain-ageing include an increase in the yield and ultimate tensile strength, and reduction in ductility, both of which can have a detrimental effect on seismic performance. Micro alloyed high strength steel AS/NZS 4671 Grade 500E reinforcement is not affected by strain ageing.

Coupling beam deformation limits in Table 6-4 have been derived from FEMA P-2335 (FEMA, 2025) and are intended to limit damage to reinforced coupling beams to moderate levels. Research by Marder et al. (2020) has shown that moderately damaged reinforced elements can be repaired by means of epoxy injection and replacement of damaged concrete. Moderate damage excludes bar buckling. Bar buckling accelerates fatigue damage accumulation in longitudinal reinforcing bars (Dhakal, Tripathi, & Dashti, Bar buckling in ductile RC walls with different boundary zone, 2019) and (ATC, 2020). Reduction in centre-to-centre spacing of stirrup-ties in limited ductile plastic regions is intended to delay the onset of bar buckling. Work completed by Zhong and Deierlein (2019) has shown reducing the

spacing of stirrups to four times the diameter of Grade 80 (550 MPa) longitudinal reinforcement improves the reparability of reinforced concrete buildings. Refer to Section 6.4.4.3 for more information related to the DCLS deformation limits in Table 6-4.

Computed periods of vibration and lateral deflections of buildings with reinforced concrete walls are known to be sensitive to the effects of cracking on member stiffness (SNZ, 2017). NZS 3101 Section C6.9.1 provides a methodology which can be used for the assessment of flexural cracking of reinforced concrete walls. The procedure detailed above to determine the effective stiffness of reinforced concrete walls (Priestley, Calvi, & Kowalsky, 2007) is expected to provide a reasonable estimate of the effects of cracking on wall stiffness.

Research by Sullivan (2005) has shown that above the mid-height of a wall, the effective stiffness may be different to that computed using Eqn 5-2, however the displacement of the wall will be most sensitive to the assumptions related to the stiffness distribution at the bottom of the wall. It was therefore concluded use of a constant cracked stiffness full height of the wall is likely to provide a sufficiently accurate estimate of the walls yield displacement. This approach is supported by Paulay (1999) and Stuart & Brooke (2024) where it is noted that, provided undesirable failure mechanism are precluded through the use of capacity design, the estimation of wall strength demands need not be viewed as a particularly critical issue.

Research at the University of Canterbury (2024) has shown that walls controlled by flexure and containing lap splices can fail at smaller drifts than walls without lap splices. This observation has been made for both non-staggered and staggered lap splices. This research shows that lap splices in walls can reduce rotation capacity via large reductions in the effective plastic hinge length and/or limited splice strain capacity.

Minimum reinforcement ratios specified in NZS 3101 are sufficient that they should ensure an acceptable margin exists between the cracking strength (M_{cr}) and the nominal flexural strength (M_n) for planar walls. This is required to ensure that well distributed flexural cracks are formed in the concrete before the reinforcing steel yields in tension. The result is that bars strain harden over a substantial length, enabling the wall to sustain significant plastic curvatures before fracture of the reinforcing steel. However, the minimum reinforcement ratios in NZ 3101 may not be adequate for non-planar (i.e. flanged) walls (O'Hagen & Stuart, 2021). This is particularly likely to be a concern for nominally ductile walls with low reinforcement ratios. Detailing walls to have a ratio $M_n/M_{cr} \geq 1.2$ is intended to avoid concentration of plastic deformation at a single crack.

Upper limit shear actions on reinforced concrete walls have been increased above the minimum requirements prescribed in NZS 3101 to provide a higher level of confidence LDSD damage control performance goals are met.

Testing (Mathews, 2004) and (Woods, 2008); and field observations by Brunsdon et al. (2017) has demonstrated that, compared with other floor systems, precast concrete hollowcore and web supported double tee floor systems are generally less robust and less tolerant of both in-plane racking movements and torsional warping about a horizontal axis induced under earthquake actions. Earthquake damage to precast hollowcore and web supported double tee floor systems is also difficult to repair. More robust floor systems include:

- Conventionally reinforced or post-tensioned insitu concrete floor systems.
- Insitu concrete composite metal deck floor systems.
- Flange hung precast concrete double tee floor systems that utilise Cazaly hangers (Slatter, 1966).
- Precast rib and infill floor systems.

Shake table testing (Lu, et al., 2019) has shown flange hung double floor units supported on steel armoured corbels were able to sustain inter-storey deflections of at least 1% without significant spalling of the corbels. Due allowance should be made for member elongation when these are significant (SNZ, 2017).

Seismic performance of singly reinforced concrete walls is uncertain (SNZ, 2017) and these have been excluded accordingly.

The extent of testing of ductile jointed precast concrete systems undertaken to date is not sufficient to verify their performance against the objectives of this document. Consequently, at this stage there is not sufficient confidence that these objectives will be met except if a Prescriptive Design Method is undertaken as described in the following section.

6.4.3.2 Additional Design Criteria for Structural Steel Buildings

The NZS 3404 (SNZ, 2026) seismic category used for the design of steel seismic-resisting systems should not exceed 3.

For moment resisting frames with a structural ductility factor, μ , greater than 1.25 at the DCLS load level:

- Shear studs should not be placed on the top flange of steel beams within potential plastic hinge zones.
- Columns should be isolated from composite slabs when they are present using a fire rated compressible material.
- Isolated columns should be restrained by steel beams to prevent lateral movement of the column about both principal axis and to prevent twist.

Active link beam deformations in eccentric braced frames (EBF) should not exceed the recommended limits in Table 6-5 for DCLS load combinations.

Table 6-5 Recommended DCLS inelastic deformation limits for EBF active links

Conditions ¹	Acceptable inelastic deformation (rad)
$e \leq \frac{1.6M_{sp}}{V_w}$ (shear controlled)	0.030
$e \geq \frac{3M_{sp}}{V_w}$ (flexure controlled)	Same as for beams, refer Table 6-10
$\frac{1.6M_{sp}}{V_w} < e < \frac{3M_{sp}}{V_w}$ (shear-flexure)	Linear interpolation should be used

Notes:

1. Where e is the clear length of the active link, M_{sp} is the nominal plastic moment capacity of the active link and V_w is the nominal shear capacity of the active link as defined in NZS 3404 Cl 12.11.3.

For concentric braced frames (CBFs) the structural ductility factor, μ , should not exceed 1.25 for DCLS load combinations.

Buckling restrained braced frames (BRBFs) should be designed in accordance with the procedure detailed in NZS 3404 Cl C12.13. Buckling restrained braces are considered to be proprietary energy dissipating devices and require prequalification testing as detailed in NZS 3404 Cl 12.13.3. Buckling restrained braces should be detailed to ensure global and local stability of the brace is reliably maintained. This should include consideration of in-plane rotation of the brace/beam/column connection due to in-plane deformation of the BRBF and also out-of-plane deformation due to seismic action in the other principal direction of the building. For DCLS load combinations maximum core strain in the buckling restrained braces should not exceed 1.5%.

Commentary:

Amendment 3 of NZS 3404 (SNZ, 2026) was expected to be published in mid-2025, however this process has been delayed with publication not expected until later in 2026. For the interim period until Amendment 3 of NZS 3404 is published the recommendations in Section 5 of the SESOC Interim Design Guidance (SESOC, 2022) should be adopted.

Amendment 3 of NZS 3404 is expected to include a number of significant changes to the seismic design provisions in the standard to improve seismic resilience and ensure acceptable margins against collapse in the event of shaking of significantly greater intensity than the ULS intensity level. This has included increasing the upper limit design axial compression force in columns acting as part of seismic-resisting systems, N'_{lim} , to 1.5 times the ULS seismic design

action with $\mu = 1.25$ and $S_p = 0.9$ (notwithstanding that the use of N_{oc}^* in NZS 3404 Eqn 12.8.3.1(2) remains preferred). New continuous column requirements are also expected to minimise the concentration of drift in multi-storey frames.

It is noted that NZS 3404 clause 12.9.1.2.2 (1) (c) contains an important requirement for category 1, 2 or 3 seismic resisting systems where for most connection types (specifically, those incorporating incomplete penetration butt welds, fillet welds, bolts or pins) it restricts the use of upper limit design actions taken from structural analysis. Instead, these connections are required to be designed to resist capacity design derived actions or the overstrength of the element to which it is attached.

A maximum NZS 3404 seismic category of 3 is recommended so capacity procedures are implemented and to provide increased steel impact toughness requirements to steel members, and related endplates, that would not otherwise be required for category 4 seismic load resisting systems.

Testing completed by Chaudari et al. (2015) has shown isolating columns from composite slabs is effective at reducing damage to the slab. However, when this is done the column should be adequately restrained against out-of-plane lateral movement and twisting. This restraint is necessary for column member compression capacity development. When columns are not isolated from the surrounding slab the effect of this on the moment demand onto the columns needs to be considered.

Localisation of inelastic deformation in EBF link beams is unavoidable, the recommended deformation limits in Table 6-4 are intended to minimise the need for repairs to link beams and adjacent sections of slab. Note that same slab damage is unavoidable unless steps are taken to vertically separate the link beam and adjacent collector beams from the slab. The recommended deformation limits are applicable to conventional EBFs and EBFs with bolted replaceable active links. Refer to the commentary of Section 6.4.4.3 for details of how the deformation limits for EBF link beams were derived.

BRBFs have been widely used in Japan and North America for over 20 years, and when detailed correctly, are a robust system with superior performance when compared with conventional CBFs (SNZ, 2026). BRBFs have been included in NZS 3404 for the first time as part of Amendment 3. Seismic design provisions for BRBFs have been included in the standard commentary rather than the normative part recognising that aspects of BRBF design, including those related to global and local stability of the buckling restrained brace, are not sufficiently prescribed to the point they could be included in NZBC B1/VM1 (SNZ, 2026).

Seismic design provisions for BRBFs exist in other international steel standards, including AISC 341-22 (AISC, 2022) and CSA S16-24 (CSA, 2024). Recognising this, BRBFs have been included in the LDS Design Prescriptive Design Method as a recognised system with the caveat that careful consideration of the in-plane and out-of-plane stability of the BRBF system is required. Takeuchi & Wada (2017) provide a comprehensive overview of this behaviour and related stability considerations for BRBFs. Design procedures have been developed to address these affects, including (Zaboli, Clifton, & Cowie, 2018), (Zaboli, 2021) and (MacRae, et al., 2021), however as noted in NZS 3404, some judgement is still required to ensure that all necessary aspects are covered.

An upper limit on the maximum strain of the BRB core for DCLS load combinations has been included to limit the excessive localisation of plastic strains.

6.4.3.3 Additional Design Criteria for Reinforced Concrete Masonry Buildings

For DCLS load combinations the structural ductility factor, μ , should not exceed 1.25. Height of reinforced masonry buildings should be limited to three storeys.

When a capacity design approach is not followed in accordance with NZS 4230 Cl 3.7.3.2 (SNZ, 2006) shear design actions on reinforced masonry walls should be determined as 1.5 times the ULS seismic design action with $\mu = 1.25$ and $S_p = 0.9$.

Stack bonding of masonry units is not permitted, and all unit cores should be fully grouted.

Commentary:

Experience has shown buildings constructed using stack bond or partially grouted masonry have generally performed less satisfactorily in earthquakes.

6.4.3.4 Additional Design Criteria for Timber Buildings

Lateral load resisting systems that have had their capacity determined through a P21 test (Shelton, 2010), to meet the requirements of B1/AS1, are outside the scope of this guideline.

For limited ductile or ductile structural systems, other than plywood-sheathed shear walls, the structural ductility factor, m , should not exceed 1.25 for DCLS load combinations.

The provisions of NZS AS 1720.1 (SNZ, 2022) may be utilised for design, however it is noted that:

- NZS AS 1720.1 is not cited in NZBC B1/VM1, therefore these provisions should be applied in addition to the current cited Standard, NZS 3603 (SNZ, 2005), or it must be treated as an alternative solution, and;
- There are known errata and omissions within NZS AS 1720.1 and reference should be made to published guidance on use of the Standard (TDS, 2025)

Elastic structural systems that are not capable of, or have not been designed for, inelastic deformation should be designed using $\mu = 1.0$ and $S_p = 1.5$ at ULS. Examples of these structural systems include timber portal frames with glued knee joints and embedded cantilever timber pole structures. Further background of this definition, and more examples, can be found in the Seismic Design Chapter of the NZ Wood Design Guides (WPMA, 2020).

Diaphragms should be designed to remain elastic under DCLS loads, except for limited localised redistribution by yielding of fasteners at areas of concentrated stress, such as re-entrant corners. The deformation of these yielded fasteners should be calculated at ULS and should not exceed the plastic deformation limits in NZS 3603. Furthermore, capacity design principles should be applied to surrounding elements to ensure non-ductile failure modes are suppressed.

Commentary:

Lateral load resisting systems that are not within the scope of B1/VM1, for example cross-laminated timber walls, are outside the scope of this Prescriptive Design Method. It is intended the Direct Design Method be utilised for these types of systems.

Limiting the structural ductility factor, μ , to 2 at DCLS is reasonable for timber structures that rely on large numbers of fasteners in lateral load, such as plywood-sheathed shear walls. There is a large margin between the design strength and probable strength of fasteners in lateral load, such that a design that adopts $\mu = 2$ and follows NZS 3603 (or NZS AS 1720.1) is unlikely to fully yield all fasteners. For nails in plywood, the expected fastener deflection (nail slip) would be in the order of 1.0-1.5 mm at DCLS, which correlates to the nominal yield point of the nail (Kho, Li, & Lee, 2019) and a minimal reduction of fastener strength and stiffness. Consequently, a plywood-sheathed shear wall designed for $\mu = 2$ at DCLS can be assumed to meet the residual drift limits of this guideline. This conclusion is supported by loss studies undertaken to date (refer Appendix C).

Seismic behaviour of ductile timber systems that fall within the scope of B1/VM1, and are not plywood shear walls, are not as well studied. The increase of peak deflections due to the pinched hysteresis of ductile plywood-sheathed walls has been shown to be negligible (Dean, Stewart, & Carr, 1986), however for systems other than ductile plywood-sheathed walls that exhibit a pinched hysteresis, reference should be made to NZS AS 1720.1, which requires a special study to determine the increase in peak deflections. Similarly, the method to estimate residual drift in FEMA P-58-1 (FEMA, 2018) has been derived from studies on systems with elastic-perfectly-plastic or general-inelastic hysteretic behaviour, so should not be assumed to be applicable to systems with pinched hysteresis without a special

study. Accordingly, for this prescribed design approach, the ductility limit of 1.25 is recommended at DCLS for these systems.

To promote nail pull-out and distribute yielding along the length of the shank, such that the number of nails that will fail by fracture is minimised, a minimum plywood thickness of 9 mm is recommended based on testing (Deam, 1996). Similarly, galvanised nails should only be used in plywood-sheathed elements where they are required for durability, because they tend to have less ductility due to their greater withdrawal resistance (Buchanan, 2007). Alternatively, instead of applying these ‘rules of thumb’, the expected failure mode of nails can be calculated using the detailed method in NZS AS 1720.1, and the connection proportioned in such a way that the governing failure mode includes one or more plastic hinges in the nail.

The recommendation of using $\mu = 1.0$ and $S_p = 1.5$ for brittle structures is consistent with NZS AS 1720.1 and the intent of TS 1170.5 (SNZ, 2025), whereby brittle structures should be avoided.

6.4.3.5 Additional Structural Design Criteria for Foundation Elements

Buildings should not be founded on mixed foundation systems.

Deformation demands on foundation elements should not exceed those detailed in Section 6.4.4.3. for DCLS load combinations.

Shallow reinforced concrete pad foundations and pile caps should have interconnecting tie beams or equivalent capable of resisting 10% of $G + \frac{1}{2}Q + E_u$ vertical load acting on the foundation component, but not less than 150 kN. The tie element may be contained within a slab on grade provided the slab has positive connections to restrain the structure across all construction and movement joints.

When there is a potential for lateral spreading to occur in CALS ground motions foundation tie elements should be sized to ensure the continuity of the foundation system is maintained.

For piled structures, where ground surface settlement is possible due to floor loading, on-going consolidation or liquefaction effects, consideration should be given to suspending the ground floor slab on piles.

Commentary:

Mixed foundation systems, where part of a building is supported on deep piles and part on shallow foundations, have performed poorly in past earthquakes and should not be used (CERC, 2012).

Foundation elements such as piles, ground beams and rafts are not typically accessible for inspection or repair. Recognising this capacity design of these foundation elements is recommended to ensure they have a nominal strength greater than the total seismic reaction that can be transmitted from the superstructure. When inelastic deformations have not been precluded (e.g. including for piles subject to significant kinematic soil loads) deformation limits have been provided to reduce the likelihood for repairs following DCLS intensity ground motions.

Interconnecting tie beams are intended to prevent lateral extension and dislocation of building foundations. Refer to Section 6.3.3.4 for recommendations on how to determine design actions on foundation tie elements when lateral spreading is anticipated.

6.4.4 Direct Design Method

6.4.4.1 General

The Direct Design Method may be more appropriate for components that would generally demonstrate compliance with the NZBC through an alternative solution, or which cannot meet the Prescriptive Design Method

criteria detailed in Section 6.4.3. This would include, for example, new structural forms and systems that are not covered in B1/VM1, and irregular buildings.

Where a solution is being considered that adopts a displacement-tolerant approach, for example a PRESSS system (Priestley, Sritharan, Conley, & Pampanin, 1999), it is important the design team addresses the implications of the increased movement associated with that system on the primary and secondary structure, non-structural elements (NSEs) and building contents.

In addition to meeting the requirements of NZBC B1 Structure, building designers still have the responsibility to demonstrate the building complies with other parts of the NZBC (i.e. B2 Durability, C6 Structural Stability etc) when these apply.

When peak floor accelerations or inter-storey drifts exceed the DCLS limits specified in Section 6.4.3 the performance of potentially affected secondary and NSEs should be validated to ensure the DCLS performance goals detailed in LDSD Volume Two have been met. Refer to Section 6.6.2.14 for guidance on how the performance of acceleration sensitive components are to be validated.

Commentary:

Although the Direct Design Method permits the design of irregular buildings, experience has shown these buildings typically perform poorly in earthquakes. Design teams are strongly encouraged to minimise horizontal and vertical irregularities when this is possible.

The following documents are available to assist with the design of low damage structural systems:

- *ASCE 7-22 Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Chapter 18 Seismic Design Requirements for Structures with Damping Systems (ASCE, 2022).*
- *Draft Guideline for the Design of Seismic Isolation Systems for Buildings (NZSEE, 2019).*
- *Design Guide for Controlled Rocking Steel Braced Frames (SCNZ, 2015).*
- *ACI ITG-5.2-09 Requirements for Design of a Special Unbonded Post-Tensioned Precast Shear Wall Satisfying ACI ITG-5.1 (ACI, 2009).*
- *NZCS PRESSS Design Handbook (NZCS, 2010).*
- *NZS 3101 Concrete Structures Standard, Appendix B Special Provisions for the Seismic Design of Ductile Jointed Precast Concrete Structural Systems (SNZ, 2017).*
- *Prescriptive Seismic Design Procedure for Post-Tensioned Mass Timber Rocking Walls (Busch, et al., 2022).*

Efforts are underway to introduce mass timber rocking wall provisions into future updates to the American Wood Council seismic design provisions for wood structures (AWC, 2015) so this may be a useful reference in the future.

Efforts are underway to introduce mass timber rocking wall provisions into future updates to the American Wood Council seismic design provisions for wood structures (AWC, 2015) so this may be a useful reference in the future.

6.4.4.2 Structural Analysis

The complexity of analysis method needed to demonstrate compliance depends on the level of inelastic demand on the primary structure, building regularity, response characteristics of the LDSD system being considered and the availability of applicable design standards and guidelines. Table 6-6 provides recommendations on the methods of analysis which can be used.

Table 6-6 Recommended methods of analysis

Method of Analysis	Limitations on Use ¹
Equivalent Static or Modal and Response Spectrum Analysis	In addition to the limitations specified in NZS 1170.5, the design ductility demand on the building should not exceed 1.25 at the ULS load level or; Structures that comply with the regularity, ductility demand limits specified in Section 6.4.3, and, recognised New Zealand or international design standards, or New Zealand industry guidelines, are used to demonstrate compliance with the performance requirements Clause B1 Structure of the NZBC ²
Displacement-Based Design	Structures that comply with the regularity limits specified in Section 6.4.3 and Application should be limited to those structural systems for which displacement-based design guidelines have been developed and tested, as reported in Priestley et al. (2007) and Sullivan et al. (2012)
Nonlinear Response History Analysis	Permitted for all structural systems.

Notes:

1. Refer NZSEE (2019) for specific recommendations for isolated buildings
2. Refer Table 6-7 for recognised NZ or international design standards, or NZ industry guidelines.

Table 6-7 Recognised NZ or international design standards, and NZ industry guidelines

Low Damage Systems	Reference Document
Controlled Rocking Steel Braced Frames	Design Guide for Controlled Rocking Steel Braced Frames (SCNZ, 2015)

While a nonlinear response history analysis may be required to model and validate the likely performance of some LDSD systems, a simple equivalent static analysis should be sufficient in most cases to advance the design for preliminary layout and budgeting purposes.

The structural analysis model should comply with TS 1170.5, except as modified by this document.

Adequate consideration should be given to the sensitivity of the structural system to variations to boundary conditions and material properties, including time-related effects.

Foundation flexibility, including piles and the supporting soils with which they interact, should be included in the analysis when they significantly affect the dynamic properties of the building.

Design strengths and expected member stiffness for use with the modal response spectrum and equivalent static methods should be determined in accordance with the relevant New Zealand standards (i.e. NZS 3101, NZS 3404 etc) when these are applicable. For the cases when New Zealand standards do not apply relevant international standards or guidelines should be used when these are available.

Nonlinear response history analysis should be undertaken in accordance with the NZ Industry Nonlinear Response History Analysis guidelines (SESOC, NZSEE and NZGS, 2025).

When required for nonlinear analysis methods, expected material strengths should be based on applicable data for the project, or from other projects using similar materials and construction. Where project specific data is not available the default expected material strengths detailed in Table 6-8 can be used.

Table 6-8 Default expected material strengths (PEER, 2017) & (NZSEE, 2017)

Material	Strength Parameter	Expected Strength
Concrete	Compressive Strength	$1.3f'_c$ ¹
Reinforcing Steel - G300E	Yield Strength	325 MPa
	Tensile Strength	475 MPa
Reinforcing Steel - G500E	Yield Strength	540 MPa
	Tensile Strength	680 MPa
Structural Steel	Yield Strength	$1.1f_y$ ²
	Tensile Strength	$1.1f_u$ ³

Notes:

1. f'_c is the specified compressive strength of concrete as defined in NZS 3101. Expected concrete compressive strength is the expected compressive strength at approximately one year.
2. f_y is the design yield stress as defined in NZS 3404.
3. f_u is the design tensile stress as defined in NZS 3404.

Commentary:

TS 1170.5 does not consider displacement-based design and analysis procedures such as the Direct Displacement Based Design (DDBD) method. Refer to Priestley et al. (2007) and Sullivan et al. (2012) for guidance when a DDBD approach is used. When DDBD is used, adequate consideration should be given to the following:

- *Increase of displacements resulting from torsional eccentricity. Refer Priestley et al. (2007) and Fox et al. (2017) for further details.*
- *Capacity design should be applied to the primary structure to meet the requirements of TS 1170.5.*
- *Reduction in lateral resistance, and hence increase in displacements as a consequence of P-delta effects. Refer to TS 1170.5 Section 5.5 for limitations when P-delta effects can be ignored and the maximum stability coefficient permitted for ULS load combinations.*
- *Amplification of inter-storey drifts due to higher mode effects. Refer to Sullivan et al. (2012) Section 5.9.*
- *Priestley et al. (2007) and Sullivan et al (2012) should be adapted considering the performance objectives set out in this document.*

The nonlinear response history analysis (NLRHA) model used for assessing building damage is intended to provide an unbiased, best estimate of the expected response of the building when subjected to earthquake ground motions. For this reason, NLRHA models should be developed using expected member stiffness, strengths and related force deformation relationships, rather than the more conservative design values used with the linear methods detailed in NZBC B1/VM1. Notwithstanding this, when assessing the adequacy of members, sections or connections, appropriately conservative design values should be used to ensure that necessary levels of structural reliabilities are achieved.

6.4.4.3 Structural System Repairability

The primary structure should be economically repairable following DCLS intensity shaking and any damage sustained should not preclude the building from achieving *Operational State Category OS II - Partial Functionality* (refer Section 6.1.7). The primary structure can be considered economically repairable if one of the following conditions is satisfied:

1. The structural ductility factor, μ , used for the design of the primary lateral load resisting system does not exceed 2 for DCLS load combinations, and capacity design has been used to ensure that a suitable sway mechanism develops under very rare shaking levels; or
2. Deformation demands on components of the primary lateral and vertical load resisting systems that are expected to yield should comply with the deformation limits detailed in Table 6-9, Table 6-10 and Table 6-11 for DCLS load combinations; or
3. Components of the primary lateral and vertical load resisting systems that are expected to yield or dissipate seismic energy by other means, including friction and viscous damping, should as a minimum have sufficient capacity to reliably withstand the following seismic demands before requiring repair:
 - a. Replaceable devices: Replaceable devices should be able to sustain at least one DCLS intensity event and still have adequate reserve capacity to meet the life safety requirements of the NZBC.
 - b. Encapsulated components: Encapsulated components should be able to sustain at least two DCLS intensity events and still have adequate reserve capacity to meet the life safety requirements of the NZBC.

All elements that may require inspection following a DCLS event must be accessible with no (or easily repairable) damage to any other element that has a design life greater than 5 years (as defined by NZBC B2) (refer Section 6.1.4).

Table 6-9 Recommended DCLS deformation limits for reinforced concrete components

Component type ¹		Acceptable deformation ² , (rad)
Beams		0.020
Columns ³		$0.032 - 0.035 \frac{N^*}{A_g f'_{cE}} \leq 0.030$
Walls ⁴		$0.0046 \leq 0.018 - 0.00017 \frac{l_w c}{b^2} \leq 0.016$
Coupling Beams – Diagonally Reinforced ⁵	$L_n/h_b \leq 0.5$	0.011
	$L_n/h_b \geq 3.5$	0.032
Coupling Beams – Conventionally Reinforced ⁵	$L_n/h_b \leq 1.0$	0.012
	$L_n/h_b \geq 5.0$	0.020

Notes:

1. Components detailed in accordance with Section 6.4.3.1.
2. Deformation limits in table are expressed in terms of total rotations.
3. Where N^* is the axial load, A_g is the gross area of the column, f'_{cE} is the expected concrete compressive strength.
4. Where l_w is the wall length, c is the compression zone depth and b is the wall thickness.
5. Where L_n is the clear span of the coupling beam and h_b is the height of the coupling beam.
6. Linear interpolation between values listed in the table is permitted.

Table 6-10 Recommended DCLS deformation limits for structural steel beams and columns

Component type	Acceptable plastic deformation ¹ (rad)
Columns ²	
1. Where: $\frac{b_f}{2t_f} \leq 0.03 \sqrt{\frac{E}{f_{yE}}}$ and For $\frac{N_G^*}{N_{C,E}} < 0.2$, $\frac{h}{t_w} \leq 2.45 \sqrt{\frac{E}{f_{yE}}} \left(1 - 0.71 \frac{N_G^*}{N_{C,E}}\right)$ For $\frac{N_G^*}{N_{C,E}} \geq 0.2$, $\frac{h}{t_w} \leq 0.77 \sqrt{\frac{E}{f_{yE}}} \left(2.93 - \frac{N_G^*}{N_{C,E}}\right) \leq 1.49 \sqrt{\frac{E}{f_{yE}}}$	$0.5 \left(1 - \frac{N_G^*}{N_{C,E}}\right)^{2.2} \left(0.1 \frac{L}{r_y} + 0.8 \frac{h}{t_w}\right)^{-1} \geq 0$
2. Where: $\frac{b_f}{2t_f} \geq 0.38 \sqrt{\frac{E}{f_{yE}}}$ or For $\frac{N_G^*}{N_{C,E}} < 0.2$, $\frac{h}{t_w} \geq 3.76 \sqrt{\frac{E}{f_{yE}}} \left(1 - 1.83 \frac{N_G^*}{N_{C,E}}\right)$ For $\frac{N_G^*}{N_{C,E}} \geq 0.2$, $\frac{h}{t_w} \geq 1.12 \sqrt{\frac{E}{f_{yE}}} \left(2.33 - \frac{N_G^*}{N_{C,E}}\right) \geq 1.49 \sqrt{\frac{E}{f_{yE}}}$	$0.7 \left(1 - \frac{N_G^*}{N_{C,E}}\right)^{1.2} \left(1.4 \frac{L}{r_y} + 0.1 \frac{h}{t_w} + 0.9 \frac{b_f}{2t_f}\right)^{-1} \geq 0$
3. Other: Linear interpolation between the values on lines 1 and 2 for both flange slenderness (first term) and web slenderness (second term) should be performed, and the lower resulting value used.	
Beams ^{2,3}	
1. Where: $\frac{b_f}{2t_f} \leq 0.03 \sqrt{\frac{E}{f_{yE}}}$ and $\frac{h}{t_w} \leq 2.45 \sqrt{\frac{E}{f_{yE}}}$	$2\theta_y$
2. Where: $\frac{b_f}{2t_f} \geq 0.38 \sqrt{\frac{E}{f_{yE}}}$ and $\frac{h}{t_w} \leq 3.76 \sqrt{\frac{E}{f_{yE}}}$	θ_y
3. Other: Linear interpolation between the values on lines 1 and 2 for both flange slenderness (first term) and web slenderness (second term) should be performed, and the lower resulting value used.	
Reduced Beam Section Connection ⁴	$0.014 - \frac{d}{300 \times 10^3}$
Eccentric Braced Frame (EBF) Link Beam ⁵	
1. $e \leq \frac{1.6M_{sp}}{V_w}$ (shear controlled)	0.030
2. $e \geq \frac{3M_{sp}}{V_w}$ (flexure controlled)	Same as for beams
3. $\frac{1.6M_{sp}}{V_w} < e < \frac{3M_{sp}}{V_w}$ (shear-flexure)	Linear interpolation should be used

Notes:

- Deformation limits in table are expressed in terms of plastic rotations. Where b_f is the width of the flange, t_f is the thickness of the flange, h is the height of the web, E is the modulus of elasticity, f_{yE} is the expected yield strength and θ_y is the member yield rotation. θ_y is to be calculated in accordance with AISC 342-22 (AISC, 2022) Section C2.
- Where d is the depth of the beam (mm).
- Where e is the clear length of the active link, M_{sp} is the nominal plastic moment capacity of the active link and V_w is the nominal shear capacity of the active link as defined in NZS 3404 Cl 12.11.3.

Table 6-11 Recommended DCLS deformation limits for structural steel braces

Component type	Acceptable plastic deformation ¹ , (mm)
Concentric Braced Frame Braces ²	0
Buckling Restrained Braces ^{3,4}	$7.5\Delta_y$

Notes:

1. Deformation limits in table are expressed in terms of brace axial plastic displacements.
 2. Concentric braces should remain within the elastic range for DCLS load combinations.
 3. In addition, maximum core strains in the buckling restrained brace should not exceed 1.5%.
- Δ_y is the axial deformation at the expected brace strength.

Commentary:

DCLS deformation limits detailed in this section are intended to limit the likelihood of safety-critical structural damage occurring, where safety-critical damage is damage that that must be repaired for occupants to be able to return safely to the building (Opabola & Elwood, 2024).

For those cases when LDS structural system repairability requirements are met by limiting the structural ductility factor, μ , to 2 or less, designers should ensure local ductility demands are appropriately controlled. Ductility demands in some structural systems are prone to localisation effects. Examples include high local ductility demands on active links in steel eccentric braced frames, and on coupling beams in coupled reinforced concrete structural walls. For these cases the DCLS deformation limits detailed in Table 6-9, Table 6-10, and Table 6-11, can be used to assess local ductility demands.

DCLS deformation limits have been derived targeting a 20% probability of exceeding the deformation at the initiation of lateral strength loss (LSL). This is consistent with the approach proposed by Opabola and Elwood (2024).

DCLS deformation limits in Table 6-9 for reinforced concrete beams, columns and walls were derived from Opabola and Elwood (2024). Deformation limits for coupling beams were derived from FEMA-P2335 (FEMA, 2025). Deformation limits in FEMA P-2335 are reported as medium values and these have been modified to a probability of exceedance of 20% assuming a total dispersion (β_{tot}) of 0.65.

DCLS deformation limits in Table 6-10 and Table 6-11 for structural steel components have been derived in a similar manner to that used by Opabola and Elwood (2024) for reinforced concrete elements. Table 6-12 summarises the sources of uncertainty assumed for the development of deformation limits. Uncertainty related to the capacity model, β_c , has generally been assumed to 0.30. This is based on engineering judgement and further research is required to confirm it is appropriate.

DCLS deformation limits in Table 6-10 and Table 6-11 for structural steel components have been derived in a similar manner to that used by Opabola and Elwood (2024) for reinforced concrete elements. Table 6-12 summarises the sources of uncertainty assumed for the development of deformation limits. Uncertainty related to the capacity model, β_c , has generally been assumed to 0.30. This is based on engineering judgement and further research is required to confirm it is appropriate.

Table 6-12 Sources of uncertainty adopted for structural steel components

Uncertainty source	Value	Reference
Capacity model (β_c)	0.30	Estimated
Influence of loading protocol on capacity (β_{LP})	0.15	FEMA 355D
Design requirements uncertainty (β_{DR})	0.20	FEMA P695
Construction quality assurance (β_{CQ})	0.25	FEMA P58
Modelling uncertainty (β_M)	0.25	FEMA P58
Record-to-record uncertainty (β_{RTR})	0.40	FEMA P58
Total	0.66	

The ASCE 41-17 'a' parameter for structural steel columns is the expected deformation at the initiation of LSL minus z times the standard deviation to target a 90% confidence level. This has been modified in Table 6-10 to equate to a probability of exceedance of 20% assuming a total dispersion (β_{tot}) of 0.66.

The ASCE 41-17 'a' parameters for fully restrained moment frame connections were derived from FEMA 355D (2000) and represent the maximum plastic rotation at which initial fracture occurred or where the resistance dropped below 80% of the plastic moment capacity. Based on a limited review of reduced beam section connection data (FEMA, 2000), (Uang, Yu, & Gilton, 2000) and (Zhang, Ricles, Lu, & Fisher, 2004) the expected deformation at the initiation of LSL has been estimated to be approximately 0.5 times the ASCE 41-17 'a' parameter. These has been modified in Table 6-10 to achieve the target probability of exceedance.

A median DCLS deformation limit for EBF link beams of 0.05 radians was informed by FEMA P58 (FEMA, 2010) and EBF repair guidance developed following the 2010/2011 Canterbury earthquakes (Clifton & Ferguson, 2016). This has been modified in Table 6-10 to achieve the target probability of exceedance. The deformation limits in Table 6-10 are applicable to conventional EBFs and EBFs with bolted replaceable active links.

The median DCLS deformation limit for buckling restrained braces (BRBs) was taken as the ASCE 41-17 'a' parameter. This has been modified in Table 6-11 to achieve the target probability of exceedance. An upper limit on the maximum strain of the BRB core for DCLS load combinations has been included to prevent excessive localisation of plastic strains.

Lateral load resisting systems which rely on energy dissipating devices that can be unbolted and replaced (i.e. have a 'plug and play' capability) following a large earthquake have repair cost advantages over other systems where this is not possible. Note that for replacement to be facilitated, residual deformations and forces in the building should be limited. Examples of 'plug and play' energy dissipating devices used in the construction industry include:

- Seismic isolators (i.e. lead rubber bearings, curved surface slider isolators etc).
- Fluid viscous dampers.
- Buckling restrained braces.
- U-shaped flexural plate (UFP) dampers.
- Friction devices.

Energy dissipating elements that are encapsulated inside structural elements, or otherwise not readily replaceable, are typically characterised by higher repair costs should this be necessary following a significant earthquake. Examples of this form of construction include:

- Conventional structural systems.
- Mild steel reinforcement encapsulated inside reinforced concrete elements.

Encapsulated mild steel reinforcement is commonly used in reinforced concrete structures as a source of hysteretic damping. Low-cycle fatigue performance of encapsulated mild steel reinforcement is a topic of active research (Zhong & Deierlein, 2019), (Elwood, et al., 2021) and (ATC, 2023). Inelastic strains in encapsulated reinforcement should be limited to control low-cycle fatigue damage. Tentative recommended maximum strains for encapsulated mild steel reinforcement are detailed in Table 6-13. Strain limits for DCLS load combinations have been derived from Ingham et al. (2001), FEMA P-2335 (2025) and ATC 145-2 (ATC, 2020). Recommended strain limits for ULS and CALS load combinations are based on the NZTA Bridge Manual (NZTA, 2022) and ITG-5.2-09 (ACI, 2009).

Table 6-13 Tentative recommended strain limits for encapsulated mild steel reinforcement.

Building Limit State	Strain Limit ^{1,2}
DCLS	0.025
ULS	$0.5 \epsilon_{su} \leq 0.05$
CALS	$0.85 \epsilon_{su} \leq 0.08$

Notes:

1. ϵ_{su} is the strain at the tensile strength of the steel reinforcement.
2. The strain limits provided are intended to be used with the peak strains computed for the mild steel reinforcement for a given building limit state without consideration of cumulative strain effects.

The strain limits are applicable to mild steel reinforcement that is not susceptible to strain aging and is prevented from bar buckling by means of a debonding layer (applies to hybrid systems), well confined concrete and transverse reinforcement. The transverse reinforcement in the form of crossties should comply with the seismic detailing requirements in NZS 3101 with the following additional constraints:

- Centre-to-centre spacing of crossties should not exceed four times the diameter of the encapsulated longitudinal mild steel reinforcement.
- Crossties should be arranged so that each encapsulated longitudinal bar is restrained against buckling by a 135 degree or 180-degree hook. Stirrup-ties with 90-degree hooks should not be used as testing has shown these can be ineffective at providing restraint against buckling in the direction parallel to the face of the member.

Project specific low cycle fatigue damage assessments can be undertaken in lieu of the strain limits in Table 6-13 to validate the structural system reparability requirements of this section are met. FEMA P-2335 (2025) provides a methodology which can be used.

International standards including ASCE 7-22 (ASCE, 2022), FEMA 461 (FEMA, 2007), ANSI/AISC 341-22 (AISC, 2022) and EN 15129 (CEN, 2018) detail prototype and production testing that can be used to validate the performance of energy dissipating components. Refer also requirements in TS 1170.5 Appendix B.

6.4.4.4 Maximum Residual Drift

Residual inter-storey deflections following DCLS intensity shaking should not exceed 0.5% of the corresponding storey height.

Commentary:

Some lateral load resisting systems, such as PRESSS hybrid walls (NZCS, 2010), have active self-centring by means of unbonded post-tensioned tendons which minimise residual drifts following an earthquake. Base isolation systems are typically fully or nearly self-centring. It is unclear at time of writing how much of a premium should be placed on active self-centring systems.

Experience from earthquakes has typically shown conventional buildings that have not experienced foundation failures have generally sustained residual building drifts that are close to the tolerances permitted in material standards for new construction (Hare, Oliver, & Galloway, 2012).

Residual building deformations can also be minimised by ensuring that the post-yield stiffness of the whole building (i.e. including both primary and secondary structural elements) is at least 5% – 10% of the initial elastic stiffness (Pettinga, Christopoulos, Pampanin, & Priestley, 2007). As suggested by Pettinga et al. (2007) in some cases the required post yield stiffness could be provided by introducing a secondary, more flexible, seismic resisting system which is designed to remain elastic for the DCLS earthquake. An example of this would be a BRBF system with a number of parallel secondary ‘gravity’ frames detailed to provide the necessary post yield stiffness once the BRBs have yielded.

Residual drifts are limited to 0.5% to ensure that building repair is possible (FEMA, 2018). This limit is consistent with work completed by McCormick et al (2008). The limit has been derived with consideration given to both human comfort, building functionality and building safety. Buildings with residual drifts exceeding this limit may not be acceptable to occupy following an earthquake. In many cases it may not be practicable to straighten a building following a major earthquake.

Residual inter-storey deflections can be calculated in accordance with FEMA P-58-1 Section 5.4 (FEMA, 2018). However, note that if a maximum drift of 0.5% is maintained then the residual drift limit can be assumed met and will not be critical.

6.4.4.5 Durability and Maintenance

The building structure should comply with the durability requirements of the NZBC Clause B2.

A maintenance schedule for the elements of the building structure should be developed for the building owner (refer also Section 7.4). As a minimum the maintenance schedule should include:

- A maintenance program
- Recommended periodic inspections
- Recommended post-earthquake damage inspections and testing.

Commentary:

NZBC Clause B2 specifies a minimum design life for durability of 50 years for elements providing structural stability to buildings, although there is no consequent driver to require removal of the building. However, it should be assumed that any structural system must achieve this life, either through durability or an effective maintenance regime.

A difficulty with this can be anticipation of the possible environmental exposure over time. What environmental testing has been completed to demonstrate adequate durability performance? For the case of energy dissipating devices, is the device suitable only interior environments, or can it also be used in damp places such as on the cold side of the dew point, or is it suitable for general use in exterior environments? What is the consequence of poor maintenance? Does the device have fail-safe mechanisms such that if the device were to lock, an acceptable capacity designed failure mechanism would ensue?

For the case of proprietary energy dissipating devices, environmental testing by the manufacturer is typically required to demonstrate adequate durability performance and a recommended maintenance schedule should also be provided.

In the case of post-tensioned systems, such as PRESSS walls, access for re-stressing should be allowed for and an effective maintenance and monitoring process should be established.

6.4.4.6 Collapse Avoidance Limit State

The structure should meet the life safety and amenity objectives of Building Code Clause B1. This includes consideration of the seismic performance of the structure when subject to earthquake shaking intensity levels considerably more than that corresponding to ULS. Seismic resisting systems outside the scope of NZBC B1/VM1 should be assessed at the Collapse Avoidance Limit State (CALS).

Satisfactory performance at the CALS loading level for seismic resisting systems that do not comply with NZBC B1/VM1 should be demonstrated by one of the following methods:

1. Demonstrate the design of the structure complies with the requirements of a New Zealand industry guideline, or an international design standard, which meets the performance requirements of Building Code Clause B1.
2. Nonlinear response history analysis of the building undertaken in accordance with the NZ Industry Nonlinear Response History Analysis guidelines (SESOC, NZSEE and NZGS, 2025).

For lateral force resisting systems containing energy dissipating devices (i.e. seismic isolators, fluid viscous dampers, friction devices etc) an appropriate allowance for variations in device properties caused by manufacturing variances and environmental conditions should be considered. Environmental conditions to be considered include the effects of aging, creep, loading history, loading rate and contamination when these are relevant.

Commentary:

NZBC B1/VM1 has no requirement to explicitly consider CALS. This is because the margins inherent within the ULS design procedures implemented within NZ materials standards are expected to provide sufficient confidence that acceptable collapse and fatality risks are achieved. This includes seismic detailing provisions, and the application of capacity design procedures for structures that are expected to respond beyond their strength limit when subjected to earthquake shaking corresponding to ULS. Similar margins may not always exist for structural systems that do not comply with NZBC B1/VM1.

The following documents could be used to demonstrate satisfactory performance at the CALS:

- *ASCE 7-22 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE, 2022)*
- *ACI ITG-5.2-09 Requirements for Design of a Special Unbonded Post-Tensioned Precast Shear Wall Satisfying ACI ITG-5.1 (ACI, 2009)*
- *ANSI/AISC 341-22 Seismic Provisions for Structural Steel Buildings (AISC, 2022)*
- *Design Guide for Controlled Rocking Steel Braced Frames (SCNZ, 2015)*
- *Draft Guideline for the Design of Seismic Isolation Systems for Buildings (NZSEE, 2019)*

The NZ Industry Nonlinear Response History Analysis guidelines (SESOC, NZSEE and NZGS, 2025) present an alternate design methodology whereby building performance at CALS is used to demonstrate that the Building Code life safety performance requirements have been achieved. The design methodology is based on similar procedures in ASCE 7-22 and related US performance-based design guidelines (PEER, 2017) and (LATBSDC, 2023).

6.5 Building Envelope

6.5.1 Scope

This section sets out recommendations for LDSO for the following components which form part of the external building envelope:

- The vertical façade.
- Balustrades.
- Roofs.
- Decks and balconies.
- Canopies.

This section contains a number of references to the Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025). This document provides relevant information for the seismic design and performance of vertical façade systems and so should be read in conjunction with this section.

Commentary:

This section is not intended to provide detailed guidance for the design of building envelope components, or to replace existing building standards and guidelines. Rather this section is intended to act as an overlay and provide designers with a methodology to achieve the LDSO outcome objectives and post event performance goals as detailed in Volume Two.

6.5.2 Detail and Interface Issues

6.5.2.1 Key Coordination Considerations

To achieve LDSO of building envelope components, early consideration of potential envelope systems, the ability of these systems to accommodate seismic building movements and cost related implications is required during feasibility stages. Building envelope designers are expected to be a key contributor to the Building Movement Strategy (refer BIP NSE CoP and Section 3.2.2.3) with the ability of vertical façade elements to accommodate in-plane building movements potentially being a governing factor for determining the required stiffness of the structural system.

Specific considerations for coordinating the seismic performance of vertical facade systems are provided in Part C.12 of the BIP NSE CoP (BIP, 2025).

6.5.2.2 Detailing and Interface Considerations

“Buildings are dynamic; they move. Exterior enclosure systems are dynamic; they move. Beginning with the primary building superstructure: columns, beams, slabs and the like deflect, sway and drift. Similar to the human body, the primary building superstructure is the bones, and the exterior building enclosure system is the skin. As the bones move, the skin reacts in kind” (Boswell, 2013).

Designers usually deliberately decouple the exterior envelope systems from the building structure, usually by designing the envelope to accommodate imposed seismic movements without contributing to primary load-bearing structural systems (Arnold, 2016). For further information on detailing also refer to Charleson (2007).

Conceptually the movement accommodation and detailing philosophy of building envelope systems during seismic events will typically either:

- Divide the building envelope into discrete movement joint locations, and areas between these joints which are separated panels that are intended to remain planar during seismic events, or

- Allow larger areas of integrated panels within the building envelope to undergo in-plane deformations in a more distributed way. Either as a consequence of being part of the primary load bearing structure of the building, or due to the approach taken to connect the exterior envelope systems (skin) to the primary building superstructure (bones).

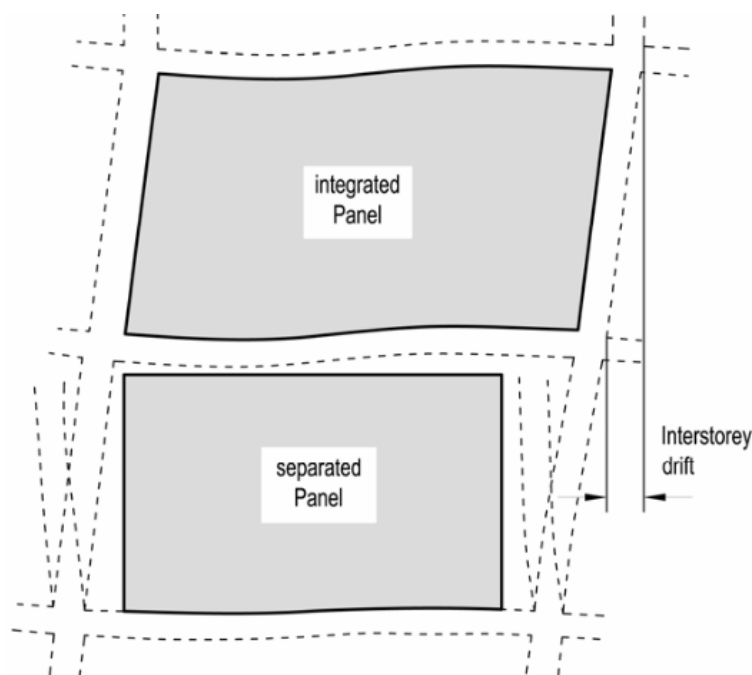


Figure 6-3 The diagram illustrates in an exaggerated way the flexure of a frame structure under lateral loading and the clearances required around the infill panel to avoid it making contact with the structure (Charleson, 2007)

Façade damage arising from earthquakes will generally occur when the movement clearances are exceeded, or deformations are induced in the envelope which exceed the elastic capabilities of the materials used. Examples of façade damage which could be expected in these situations include torn membranes and seals, deformed flashings and broken glass.

LDSD buildings will usually avoid incompatible interfaces by avoiding mixed façade systems which, if used, can introduce complex movement incompatibilities. Detailing methods for reducing the risk of earthquake damage to façade elements for common façade systems are well-documented, particularly in the Japan (AIJ, 1996) and US (FEMA, 2011) where damage mitigation philosophies are more mature.

6.5.2.3 Cladding as Load-Bearing Structure

Many residential buildings up to 3-storeys high, that utilise the acceptable solution cladding details of the NZBC E2/AS1, have cladding directly fixed to timber framing which forms part of the buildings primary structure, usually with a 20mm drained and ventilated cavity between. Generally, these timber 'stick framed' houses have performed well in past earthquakes, unless affected by significant ground deformation (i.e. due to liquefaction or lateral spreading) (Buchanan, Carradine, Beattie, & Morris, 2011).

The mechanisms for accommodating movement in these systems can be complex and may include:

- The cumulative effects of small movements by using cladding materials and modulation that better accommodate movement, e.g. weatherboards fixed with a traditional single nail per stud thus allowing for the boards to effectively slide past one another.
- Localised movement at fixings, e.g. nail slip, elastic deformation of fixings and brick ties, and fixing hole clearances
- Stretch and/or deformation, e.g. membranes, wraps and profiled metals.

Concrete and masonry buildings frequently utilise lateral load resisting systems as part of the primary building structure that are congruent with the exterior envelope system. These buildings typically only sustain modest inter-storey drifts during seismic events, and normally use externally applied control layers (paint, membranes, plaster and stucco) applied continuously over the primary structural substrate to form the envelope. These layers may in themselves be fairly stiff, or have inherent moisture or thermal movements, requiring the introduction of movement control joints to control cracking. Designers need to be aware the additional mass inherent in these buildings can introduce significant inertial forces during seismic events within elements. At the connections of parts, these forces may be more consequential in design and risk considerations than is the case for lighter envelope solutions.

6.5.2.4 Horizontal Elements

Roofs, decks, membrane systems and canopies also form part of the exterior envelope system. These horizontal elements are generally not required to accommodate the same magnitude of building deformations during seismic events as vertical façade components. However, designers should apply similar design philosophies to horizontal elements to ensure the LDS performance objectives are met.

Common LDS detailing issues to be considered for horizontal elements include:

- Interfaces with adjacent vertical components may need to accommodate relative movement. For example, at some roof to vertical façade interfaces.
- Building deformations to be accommodated by the building envelope for DCLS load combinations will typically be higher than would be case for SLS1. For example, a torch-on membrane over a plywood timber framed roof diaphragm will need to be designed to accommodate increased in-plane racking deformations.
- Seismic separations of the primary structure should be articulated in roofs and membranes. Such seismic jointing, usually proprietary, is typically designed to maintain weathertightness to the SLS1 movement conditions. The repercussion, in terms of damage and replacement expectations, of more onerous movement ranges at the DCLS requires consideration.

6.5.2.5 Interfaces

In considering the design of interfaces between façade systems, and between the façade and other building components, there are additional design complexities that do not exist when contemplating two dimensional in-plane performance of individual systems. These often stem from adjoining systems' differing strategies for accommodating seismic induced movements. For example a unitised façade system will typically be hung from the floor at the top of the panel, whereas architectural precast systems will typically be supported at the floor at the base of the panel.

At corners the reality that buildings have three dimensional envelopes reveals complexities that two-dimensional consideration of system performance overlook. A corner is where the in-plane movement strategy of one system necessarily transitions to the out-of-plane strategy of the adjoining system around the corner. The movement mechanisms at the corner can be more complex than those of the individual systems. The geometric simplicity of an in-plane sliding strategy on one side of the corner, combined with an out-of-plane tilt on the other, may require a warping transition at the corner.

Fire rated control joints are commonly required at the interface between the façade and the floor, and at fire rated intertenancy walls (refer Section 6.11). Seismic movements to be accommodated by the fire control joints need to be communicated to the passive fire designer so correct systems can be specified. Similar considerations apply to other gaps/movement joints between fire rated elements of the building envelope i.e. joints between fire rated decks and adjacent building facade.

These interfaces will often be quite bespoke to a particular project. The performance of the interface, in terms of weathertightness, thermal, acoustic and fire, should be equal to the adjacent systems. This is not unique to LDSD, as accommodating building movements (be they from seismic actions, wind, or imposed loads acting on the primary structure etc) is a key driver for the design and detailing of façade interfaces on any building.

In conventional seismic design a typical approach to an interface might be:

- Design elastomeric seals or flashings to accommodate SLS1 building movements without damage. The imposed movements on elastomeric seals would be demonstrably less than their design capacity. Bending of flashings is permitted provided they remain elastic and return to their pre-event shape.
- For building movements greater than expected at SLS1, accept that the seals or flashings may be damaged and need to be replaced.
- Contact between the structural components of adjacent systems would generally be avoided in a ULS event, and flashings would be designed to resist ULS design actions (to avoid causing a falling hazard)

For LDSD the location and ease of access to inspect and repair earthquake damage are key considerations in addition to the degree of damage to elastomeric seals and flashings at interfaces (refer also Section 6.1.4). For example:

- If seals are concealed behind internal linings their damage post-event cannot be easily assessed, and replacement may be complicated. Leaving seals exposed, designing to allow assessment and replacement from the exterior, detailing linings to be easily removable for inspection, or designing a sealant joint and/or specifying a sealant which accommodates larger movements (i.e. no damage at DCLS) are possible options.
- The performance of a flashing is often reasonably critical to the performance of the interface (in terms of weathertightness) as it often protects an associated seal from UV radiation. For LDSD an assessment should be made as to how much damage would be acceptable during a DCLS event, both in terms of the effect on the performance of the interface (prior to damaged element being fixed) and the cost, extent and ease of replacement.

NZS 3101 (SNZ, 2017) has specific CALS requirements for concrete cladding panels to remain attached to buildings following a maximum considered earthquake. This requirement may affect the detailing of concrete cladding panel connections and interfaces immediately adjacent cladding panels.

6.5.3 Prescriptive Design Method

In addition to the requirements of the NZBC, building envelope components should be designed in accordance with the additional technical criteria detailed below.

Seismic design actions (accelerations and inter-storey deflections) required for the design of the building envelope should be provided by the project structural engineer in accordance with Section 6.2.1 and provided to the building envelope designers.

External building envelope components required for *Operational State Category OS II - Partial Functionality* should be designed to accommodate DCLS design actions (relative building movements and inertia forces) without the need for repair. The building envelope should remain effective in preventing air and water intrusion to the extent required for post-earthquake functionality.

Some damage at discontinuities such as corners and transitions is permitted provided it is easily repairable and does not prevent reoccupation of the building within the specified timeframes.

Commentary:

For many projects the ability of building envelope components to accommodate DCLS seismic movements may be a governing factor for the structural system selection. It is therefore important building envelope designers contribute to the development of the Building Movement Strategy to ensure an appropriate structural system is adopted for the project (refer BIP NSE CoP and Section 3.2.2.3).

Adopting a ‘without repair’ design strategy (i.e. the performance requirements conventionally adopted for SLS1) to the DCLS fulfils LDSD damage control and improved functionality performance goals for the building envelope. This approach is consistent with that adopted in other damage control/ seismic resilience frameworks such as the REDI Rating System (Arup, 2013).

Because of the inherent complexity, and limitations of available information, regarding the seismic performance of building envelope systems sustaining design actions greater than SLS1, it was judged appropriate to limit the seismic demands on these systems so as to provide the necessary confidence that LDSD performance goals would be achieved. For example, by designing a building that has low drifts, limiting corners and geometric complexity, and implementing simple, regular, well distributed lateral bracing systems (refer Section 3.2.2.1 on building form).

6.5.4 Direct Design Method

The Direct Design Method may be appropriate for building envelope components that do not meet the Prescriptive Design Method criteria detailed in Section 6.5.3. This includes façade components that have been allowed to sustain higher deformations under DCLS design actions and as a consequence may not meet the ‘without repair’ requirements detailed in the Prescriptive Design Method.

To demonstrate LDSD damage control and improved functionality performance goals are met (refer Section 6.1.7) the following will need to be considered when applying the Direct Design Method:

- Quantification of the anticipated extent of façade damage expected following DCLS intensity ground shaking.
- Access requirements to inspect damage to the building envelope, and related methodology and cost to effect repair.
- Short term management plans required for building reoccupation until and repairs are completed.

Analysis or experimental testing of proposed façade systems is likely to be necessary to validate their seismic performance.

Commentary:

Refer to Appendix J for additional information on New Zealand and Australian façade testing standards which can be used to validate the seismic performance of façade systems. It should be noted that testing for validation could be beneficial under either the Direct Design Method or Prescriptive Design Method, and that in addition to the New Zealand and Australian Standards in Appendix G, there are numerous other international testing standards that may offer potential methods for prototype and/or proof testing of façade systems.

6.6 Building Services

6.6.1 Scope

This section sets out recommendations for LDSD of the following building services components:

- Linear components:
 - i. Pipes.
 - ii. Ducts.
 - iii. Cables and cable support systems.
- Self-contained components:
 - i. Major equipment, e.g. switchboards, control panels, generators, pumps, tanks, chillers, boilers, lift control systems and equipment, fans, air handling units, fan coil units, etc.
 - ii. Minor equipment, e.g. light fittings, air diffusers and grilles, emergency lighting, exit signs, fire detectors, evacuation sounders, etc.

Refer to also Section 6.8 for additional LDSD requirements for lift cars, lift shafts and related hardware.

Commentary:

Building Services disciplines include mechanical, electrical, plumbing and drainage (internal), fire alarm and protection, lifts, security and other miscellaneous building infrastructure systems. Design of related building components such as lift pits, lift shafts, services risers and the like is by other consultants.

This section is not intended to provide detailed guidance for general seismic design or to replace existing standards and guidelines. Rather this section is intended to act as an overlay and provide designers with a methodology to achieve the LDSD outcome objectives and post event performance goals as detailed in Volume Two.

The Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025) provides general guidance for the seismic design of NSEs including building services systems and components.

6.6.2 Detail and Interface Issues

6.6.2.1 General Functionality Considerations

LDSD Category Level 2 and 3 buildings are expected to achieve *Operational State Category OS II - Partial Functionality* following a DCLS intensity earthquake provided public infrastructure remains functional (refer Section 6.1.7).

On-site back-up systems can be considered to provide increased reliability of building functionality for those projects where clients want to reduce their dependency on public infrastructure, although such back-up systems are not a requirement for LDSD Category Level 2 or 3 buildings. Refer further details in:

Section 6.6.2.7 – On-Site Potable Water Storage

Section 6.6.2.8 – On-Site Wastewater Storage

Section 6.6.2.9 – Emergency Power

6.6.2.2 Building Movement Strategy

Section 3.2.2.3 provides guidance on how to develop a Building Movement Strategy. Since building services distribution systems (pipes, ducts, cable supports) are displacement sensitive components and are typically subject to damage control and improved functionality performance requirements at higher earthquake intensities

in LDSD buildings than is the case for code minimum IL 2 and IL 3 buildings, the development of an agreed approach (i.e. the Building Movement Strategy) is critical for such services.

Particular attention is drawn to the following detail and interface issues:

- Services crossing structural seismic separation – refer to Section 6.6.2.5 below.
- Services and full height partitions – refer to Section 6.6.2.10 below.
- Underground services – refer to Section 6.6.2.13 below.
- Coordination of linear component seismic restraint and flexibility provisions, particularly for pipework, to cater for the combination of thermal, seismic and gravity induced movements, such that restraint systems do not compromise flexibility provisions. Refer to Section 6.6.2.12 – Liquid Containment or discussion on component categorisation of pipework, which determines the applicable Limit State seismic movement at which pipework should remain operational and avoid consequential damage due to content leakage.

Commentary

The Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025) provides general guidance on seismic design of linear services, including discussion regarding coordination of restraint and flexibility provisions to avoid the former compromising the latter.

6.6.2.3 Designing for Inspection and Repairability

LDSD requires designers to consider how key building components required for building reoccupation are to be inspected and repaired following a significant earthquake (refer Section 6.1.4). This requires a specific focus on access provisions to critical building services that might not otherwise be needed for minimum code buildings. Particular attention should be paid to:

- Piped services: accessible and labelled isolation points for rapid isolation of selected pipe branches to facilitate section-by-section repair of earthquake damage should this be required (and potentially reduce the extent of consequential damage).
- Layout of and access to in-ceiling services, particularly above solid plasterboard or similar ceilings.
- Access to services in riser shafts.
- Underground services: access for inspection and repair where underground services enter and exit the building. Refer to Section 6.6.2.13 below.

Commentary

The intent of this requirement is to consider practicable and cost-effective steps to configure NSEs and improve access, rather than significantly increase provisions beyond those which are appropriate for normal maintenance and end-of-life replacement. It is not intended, for example, to avoid the need to remove sections of plasterboard or substantially increase services spatial requirements

NZBC H1/VM3 (HVAC Energy Efficiency) notes good practice guidance on space requirements for equipment access can be found in the UK Defence Works Functional Standard, Design & Maintenance Guide 08 (MoD, 1996).

6.6.2.4 Building Services Plant Locations

Heavy equipment breaking loose is a potential source of injury and building damage. Preferably, heavy plant (e.g. chillers, boilers, generators, bulk liquid storage tanks) should be located as low as practicable within the building to minimise both building and plant seismic loading. Such locations also tend to make it easier to repair plant and/or connect temporary plant, as well as assist design for major plant replacement at end of life.

Consideration should also be given to tsunami and other flood risks when locating plant. Elevating plant to avoid the risk of water inundation may be appropriate for some projects.

Fuel burning equipment should be located where it is accessible, to facilitate firefighting. Storage of fuel above ground and in buildings should be minimised.

Commentary:

Engine exhaust, or gas appliance flue discharge, requirements may in some cases make locating such equipment at, or near, the top of a building preferable.

Decarbonisation is reducing the application of fuel burning equipment and its associated seismic risks.

6.6.2.5 Services Crossing Structural Seismic Separations

When required, linear systems (e.g. pipes, ducts, and cables) should cross structural seismic separations as close to ground level as practicable to minimise the space required for seismic displacement provisions. It is generally preferable to limit services crossing structural seismic separations to distribution mains infrastructure only, i.e. avoid or otherwise minimise distribution runout systems crossing seismic separations.

For base-isolated buildings, provide input to locate the isolation plane to minimise impact on building services systems, or otherwise achieve an appropriate compromise with other structure and/or non-structural component objectives (i.e. as part of the development of the Building Movement Strategy). For example, for buildings with basements, locating the isolation plane at high level directly below the ground floor slab (as opposed to higher in the building) is preferable because this:

- Minimises the number of linear services crossing the isolation plane requiring space intensive seismic flexibles that increase the number of potential failure points.
- Enables location of the space-intensive seismic flexibles with least spatial impact on other non-structural elements and/or space utilisation.
- Potentially reduces impact on lift shafts crossing the isolation plane.

6.6.2.6 Buildings with Multiple LDS Category Levels or Multiple Importance Levels

Refer to Section 6.1.5 above for details of additional considerations for multi-building projects which include buildings with different LDS Category Levels/Importance Levels.

Commentary

Sometimes projects warrant different LDS Category Levels/Importance Levels across a multi-building site that relies on some common building services infrastructure. While the impact of different LDS Category Levels/Importance Levels is relatively straightforward for structural and architectural components, it can be much more complex in terms of the location, design, operation and maintainability of services infrastructure (plant, pipes, ducts, cables) that is common to the buildings. The appropriate solution may impact seismic design of the structure, other NSEs, as well as that of the building services systems.

6.6.2.7 On-Site Potable Water Storage

The Building Code mandates on-site water storage for the building types detailed in Table 6-14 below and also suggests consideration of such facilities for other buildings.

Table 6-14 Minimum NZ Building Code water storage requirements.

Service	Relevant acceptable solution or verification method	Buildings Requiring On-Site Water Storage
Secondary fire water supply	NZS 4541 (SNZ, 2020) as cited by NZBC C/VM2 & Fire and Emergency NZ Regulations.	Height > 45m from point of lowest entry to floor level of highest normally occupied floor. Total floor area > 11,000m ² AND including managed evacuation strategies.
Potable (cold) water storage	NZBC G12/AS1	Community Care, e.g. hospitals, old people's homes, prisons. Minimum 50 litres/person.

Mains water supply could be impaired following a significant earthquake, and the resilience of these systems is unlikely to be quantifiable. Therefore, for buildings other than those listed in Table 6-14, where the client chooses to reduce their dependency on public infrastructure systems, on-site water storage can be considered for both fire supply and potable water to provide some degree of increased post-earthquake building functionality.

On-site water storage requires additional space and structural consideration, as well as an appropriate means of replenishment e.g. mobile tanker access). Table 6-15 below provides indicative potable water consumption rates which can be used to assist with project planning.

Table 6-15 Indicative potable water consumption (for hygiene and drinking).

Application	Potable Water Consumption ¹
Small Hospital	450 – 600 Litres / person / day
Residential	200 – 350 Litres / person / day
Office	2.0 – 5.5 Litres / m ² (NLA) / day

Notes:

- Consumption rates provided should be considered indicative only and design teams should complete their own project specific assessments of likely potable water requirements.

Using the indicative potable water consumption rates detailed in Table 6-15, it can be observed that providing even 24 hours of potable water will typically require a substantial storage volume that will need to be accommodated within the site. Additional water may also be required for cooling systems that are necessary for post-earthquake building functionality (i.e. where space cooling is dependent on evaporative cooling, including cooling towers).

6.6.2.8 On-Site Wastewater Storage

Public sanitary wastewater systems could be impaired following a significant earthquake, and the resilience of these systems is unlikely to be quantifiable. Therefore, where the client chooses to reduce their dependency on public infrastructure systems, on-site sanitary wastewater storage should be considered to provide some degree of increased post-earthquake building functionality.

On-site sanitary waste storage requires:

- Septic tank, typically underground, storage volume based on post event operational requirements and available frequency of portable pump-out facilities, e.g. mobile tankers.
- Changeover valves (typically manual), to divert wastewater from normal sanitary drainage systems to septic tank in an emergency.
- Suitable access for mobile tankers

6.6.2.9 Emergency Power

Mains electrical supply could be impaired following a significant earthquake, and the resilience of the supply network is unlikely to be quantifiable. Therefore, for those projects when the client chooses to reduce their dependency on public infrastructure systems on-site emergency generation, or facilities for portable generators, can be considered.

For those projects where clients require rapid power restoration, permanent on-site emergency generation should be included. Portable generators are likely to be in high demand after a significant earthquake, and the limited availability of these generators could extend post-earthquake power restoration times.

Where extended autonomy between refuelling is required (e.g. days rather than hours), bulk on-site storage tank(s) and tanker access thereto will likely be necessary.

If a portable generation option is selected, facilities should include:

- Permanent Main Switchboard (MSB) facilities for temporary connection of portable generator cables.
- Space for portable generator(s), considering:
 - i. Access for unloading / loading portable generator(s).
 - ii. Temporary cable routes from generator(s) to Main Switchboard (MSB).
 - iii. Ensuring engine exhaust, heat and noise emissions are non-objectionable.
 - iv. Tanker access for refuelling (likely to be frequent).

Commentary:

Central battery systems are typically limited to backup power to emergency lighting/exit signage and/or data systems, providing hours rather than days of autonomy. While proprietary higher energy density battery storage system technologies are continually improving, current applications are typically limited to small residential, shorter term & focused backup in commercial buildings (i.e. systems have insufficient capacity for the essential lighting and power typically required for post-earthquake operational continuity in the event of an extended main electrical supply power outage.

Dual site power supplies from alternative substations can improve power resilience, but these systems are typically targeting to add to, rather than substitute for, resilience provided by on-site generators.

6.6.2.10 Services and Full Height Partitions

Collaborative development of an appropriate project solution for detailing the interface between linear systems and non-loadbearing partitions is an important part of a Building Movement Strategy. This strategy should also include consideration of an appropriate ceiling void depth to accommodate the proposed building services. Section 6.10 provides guidance on key coordination considerations for services that penetrate non-loadbearing partitions.

6.6.2.11 Heating and Cooling Plant

It is considered the provision of reasonably tenable conditions will be adequate to meet the performance requirements of *Operational State Category OS II - Partial Functionality* (as defined in *Volume Two, Section 7.2 – Improved Functionality*) following a DCLS intensity earthquake.

Requirements for heating and cooling plant to achieve reasonably tenable conditions will vary depending on the location of the building (i.e. the climate zone), building use (including occupant quality expectations), building characteristics (e.g. thermal envelope, internal heat loads, openable or non-openable windows), heating and cooling details (e.g. plant redundancy, 100% outdoor air economiser capability).

Heating and cooling plant redundancy decisions are typically based on a wide range of variables, and post-earthquake functionality is another consideration that needs to be included in those decisions.

Examples of situations when post-earthquake functionality of heating and cooling plant may be needed to meet *Operational State Category OS II - Partial Functionality* requirements:

- Heating systems in a Dunedin apartment building, but less important in a double-glazed office building in Auckland.
- Cooling systems in a premium Auckland office building, but less important in a modest rent Wellington office building with small floorplate and operable windows.

Building owners should be made aware the provision of post-earthquake functionality for heating and cooling plant for commercial buildings to satisfy building owner or tenants expectations, but that would otherwise be unnecessary to meet LDSO requirements, potentially exposes them to more sources of argument with, and liability to, tenants, who may not understand proprietary equipment seismic qualification limitations (refer Section 6.6.2.14 for further information related to proprietary equipment seismic qualification).

Commentary:

While the Building Code mandates ventilation for all occupied spaces in all buildings, it only mandates heating in aged and early child-hood care facilities and does not mandate cooling in any building type. Consequently, operational continuity of heating and cooling is usually a commercial rather than code compliance matter.

6.6.2.12 Liquid Containment

Liquid (water, fuel oil, etc) containing components may leak or fail in normal use, as well as during a seismic event, and this can cause significant consequential damage. Builder's work bunding and floor drainage can be provided around tanks, vessels and zones of concentrated pipework such as plantrooms and pipe corridors, particularly where appropriate to protect electrical equipment from water ingress.

Liquid containing components would normally be classified at least Category P6 (disproportionate consequential damage) due to potential for liquid spill. Such components could be reclassified Category P5 to minimise the likelihood of loss of containment (i.e. rupture of tanks or piping) during a DCLS intensity earthquake. However, the following should be considered:

- Most tanks and vessels will be above the TS 1170.5 (SNZ, 2025) weight/height thresholds that trigger Category P1/P2/P3. This will require the components to be restrained for structural integrity for ULS design actions. Such restraint is likely to minimise damage at the DCLS seismic intensity level that could cause loss of containment.
- Hazardous contents trigger Category P1/P2/P3 which has a corresponding requirement of avoiding loss of contents at ULS.
- In order to meet the LDSO *Operational State Category OS II - Partial Functionality* performance requirements most piping distribution systems will be classified as Category P5.
- If components are assigned Category P6 they are required to retain operational integrity at 2 x SLS1 design actions. For many LDSO Category Level 2 buildings the resulting design actions will be similar to those associated with the DCLS i.e. to be used for the design of Category P5 components.

6.6.2.13 Underground Services

Services entering and exiting buildings should have their connections designed to be appropriately flexible and robust at easily identifiable and reasonably accessible locations such that they may be readily repaired if damaged.

NZS 4219 (SNZ, 2009) requires underground pipes penetrating building foundations to include provision for relative movement between the building and the surrounding soil, including a minimum 25mm clearance between pipes and adjacent foundation elements. However specific ground conditions (e.g. liquefaction potential) can result in expected relative movements exceeding the above NZS 4219 minimum. Hence expected relative movements should be provided by the project structural and geotechnical engineers and clearances should be detailed accordingly.

NZS 4219 also recommends small underground pipes from storage tanks be installed inside larger diameter pipes where they pass through building foundations.

Consideration should be given to facilitating access to underground services in the event of earthquake damage as far as reasonably practical to ensure the inspection and reparability requirements of Section 6.1.4 are met. For example, consider minimising underground services with concrete or asphalt cover where practicable.

Commentary

Access manholes to drainage pipework passing through building foundations are often not practicable, and where they are, the additional cost/construction complexity may or may not be warranted.

Covered pipe trenches should be designed to prevent, or otherwise minimise water ingress, include water monitoring/pumping drainage systems. Provision of improved accessibility for the repair of damage to underground services outside the property boundary is outside the scope of LDSD.

6.6.2.14 Seismic Qualification of Proprietary Equipment

The Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025) includes guidance on seismic qualification of proprietary equipment to demonstrate compliance with relevant seismic performance criteria. This guidance is based on US industry practice (ASCE, 2022), following the principle that it is generally not considered economic to seismically certify all building services components, the Code of Practice instead focuses on the more critical equipment and includes a list of equipment types deemed “inherently rugged” that are exempt from seismic qualification.

The Code of Practice requires seismic qualification of plant and equipment assigned Category P5, i.e. necessary for operational continuity at SLS2 in IL4 buildings. TS 1170.5 (SNZ, 2025) does not apply the Category P5 to other than IL 4 buildings. However, the Category P5 classification is relevant to LDSD Category Level 2 and 3 buildings where some building services systems are categorised P5 to enable LDSD damage control and improved functionality performance goals to be met.

Ideally building services systems assigned Category P5 to meet LDSD requirements should be subject to seismic qualification because without such qualification there will be a lower confidence LDSD performance goals will be met. Seismic qualification of proprietary equipment typically comes at additional cost (and possibly time) and building owners should be made aware these additional costs can be significant.

Section 6.4.3 includes a requirement that building structures should be designed so peak floor accelerations (PFAs) for the DCLS are generally less than 1.25g, and should not exceed 1.6g, unless acceleration sensitive components (i.e. components not deemed “inherently rugged”) required to *achieve Operational State II – Partial Functionality* have been seismically certified for the higher floor accelerations.

These PFA limits were primarily established to ensure LDSD damage control performance goals are met. However, limiting floor accelerations will also reduce, but not necessarily eliminate, the likelihood of post-earthquake loss of equipment functionality. Furthermore, loss modelling studies (refer Appendix C) indicate proper seismic fixing and restraint of proprietary equipment, and the flexibility of connected pipes, ducts, and cable supports, is generally expected to result in low or relatively easily repairable damage following a DCLS intensity event. However, this does not assure the continued functionality of acceleration sensitive components.

While seismically qualified proprietary equipment is available on the NZ market, it is:

- At the time of writing, relatively limited in terms of the range of equipment types that could require such qualification, and the number of brands within any one equipment type.
- Brand-specific, i.e. not representative of the respective kind.
- May have originated from project-specific seismic testing and therefore may not be applicable to all NZ locations, Building Importance Levels and building designs.
- May not be readily identifiable without specific questioning of individual suppliers.
- May subsequently be found (via review of equipment technical submission during the construction design phase) to have unacceptable non-compliances in other areas of performance, quality, durability, etc.

Refer to Appendix H for additional information on factors that affect the cost and reliability of the seismic qualification of proprietary equipment in NZ.

It is recommended building owners, in consultation with their building services engineer and NSE Seismic Designers (and contractor if the project includes early contractor engagement):

- Consider the specifics of their building, its seismic performance criteria, and the potential consequences of significant damage to, or loss of functionality of, Category P5 proprietary equipment.
- Consider the likely time and costs related to seismic qualification of Category P5 proprietary equipment, and
- Decide whether to include or exclude such seismic qualification based on a high-level cost versus benefit assessment.
- Document the outcomes of this process in the LDSD section of the respective Design Features Reports.

Examples of building specifics that may influence a building owner's decision

- Whether or not the building structure meets the PFA limits for acceleration sensitive components as noted above.
- Potential economic loss or service disruption in the event of loss of functionality (e.g. loss of power and/or cooling equipment in a commercial data centre).
- Main switchboards (other than those necessary for evacuation after fire or human life support and hence designated P4 and requiring seismic qualification anyway) are essential to operational continuity but are often located on or below the ground floor and therefore subject to lower seismic accelerations than equipment higher in the building.

In lieu of seismic qualification, building owners may wish to consider alternative strategies to manage the risk of damage to Category P5 proprietary equipment:

- Establish repair or replacement strategies for the damaged equipment. This could include the retention of spare parts or equipment on or off-site. However, uncertainties in attempting to predict proprietary equipment failure modes, and therefore what spare parts or equipment to retain, the cost of storing spare parts and equipment, labour uncertainties following a significant earthquake etc, means that such strategies may not be effective in practice.
- Establish alternative short term management strategies which can be implemented while repairs to the damaged equipment are completed.

Commentary:

For comparison seismic qualification requirements of the US Functional Recovery provisions and the Japanese Building Code are:

- *US Functional Recovery provisions (refer Section 1.6) mandate seismic qualification of plant and equipment required for building occupation (i.e. needed to support the basic intended functions of the building) for those buildings that have target recovery times of less than 1 week. For buildings with target recovery times of 3 to 6 months seismic qualification requirements are less onerous. For these buildings acceleration limits are provided for each category of NSE, and seismic qualification is required when these limits are exceeded. (Molina Hutt & Zimmerman, 2024).*
- *The Japan Building Code has a requirement that plant and equipment required for building occupation be seismically certified to a minimum of 1.0 g (Stanway, 2024).*

6.6.3 Prescriptive Design Method

In addition to the requirements of the NZBC, building services should be designed in accordance with the additional technical criteria detailed below.

Seismic design actions (accelerations and inter-storey drifts) for building parts and components should be determined by the project structural engineer in accordance with Section 6.2.1 and provided to the NSE designers. The project structural engineer should also advise if peak floor accelerations exceed the limits specified in Section 6.4.3 thereby triggering seismic qualification requirements for acceleration sensitive components.

The general design requirements detailed in Section 6.2 also need to be complied with including:

- When determining the categorisation of building parts and components consideration should be given to the potential for interaction as detailed in Section 6.2.2.
- Horizontal seismic design actions on building parts and components, and their connections, are to be determined using the part ductility values, μ_p , detailed in Sections 6.2.3 and 6.2.4.

Building services systems required for *Operational State Category OS II - Partial Functionality* should be classified as TS 1170.5 Category P5 and should be designed to achieve Operational Integrity at the DCLS as per Table 6-16 below.

When practicable, bunding and floor drainage is to be provided around tanks, vessels and zones of concentrated pipework such as in plantrooms and pipe corridors, as required to protect electrical equipment from water ingress.

Table 6-16 LDSD OS-II – Partial Functionality - Operational Integrity Component Categorisation for LDSD Category Level 2 and 3 buildings

Services System/Function	Operational Integrity Category ¹	Comments ²
Fire sprinkler systems	P5	Required by NZBC/NZS 4541 (SNZ, 2020) to be P5 in IL 4 buildings, otherwise P6. For LDSD assign as P5.
Hydrant systems	P4	Required by NZBC/NZS 4510 (SNZ, 2022) to be P4. Hydrant systems are required for firefighting operations within buildings. For LDSD assign as P4.
Fire alarm, escape route pressurisation, smoke control systems	P5	Required for building occupation ³ .
Emergency lighting, escape route signs	P4	Required for evacuation after earthquake.
Mains & emergency power	P4 where necessary for evacuation after earthquake or human life support, otherwise P5	Emergency power not mandated by NZBC or Electrical (Safety) Regulations ⁴ .
Artificial lighting and general power	P5	Artificial lighting required by NZBC to enable safe movement.
Water supply and drainage	P5	Required by NZBC for building occupation.
Domestic hot water supply	P5	Typically appropriate for LDSD OS-II. Required by NZBC if water supply used for utensil washing in any building, or personal washing in residential, aged or early childhood care centres.
Cooking facilities ⁵	P5	Where required for LDSD OS-II. Required by NZBC in residential, aged & early childhood care only.
Ventilation – occupied spaces	P5	Required by NZBC for building occupation. Specifically includes (but not limited to) heating / cooling air distribution systems necessary for proper distribution of outdoor air ventilation, and laboratory fume cupboards.
Space heating	P4, P5 or P6/P7 dependent on building specifics and client quality expectations ⁶ .	Building services engineer to provide building-specific recommendations to maintain reasonably tenable post-earthquake conditions, refer Section 6.6.2.11. Mechanical cooling for P4 components would typically also be categorised P4. P6 applies to liquid containing equipment if not P5.
Space cooling		
Equipment/plantroom ventilation, heating or cooling	To match equipment served.	For example, fire pumps, generators, UPS/batteries, transformers, chillers.
Lifts ¹¹	P5 or P7	Typically P5 for LDSD OS-II. Required by NZBC for building occupation except as per Note 7.
Escalators	P5 or P7	Typically P5 for LDSD OS-II. Required by NZBC if used as escape path (not typical) ⁸ .
BMS (Building Management System)	P5	Typically required for LDSD OS-II for critical control services systems required for building occupation. Required by code if essential to ventilation, escape route pressurisation or smoke control.
Electronic security	P5 or P7	Typically appropriate for LDSD OS-II unless manual security measures are implemented ⁹ .
Audio visual	P5 or P7	Typically P7, unless selected otherwise by the client. Refer also note 10 for assistive listening systems.
Data systems		
Carpark management		

Notes:

1. All components within a given building services system (i.e. all pipes, ducts, cables and equipment within each sprinkler, power supply, lighting, ventilation, heating, cooling, water supply, etc. systems respectively) should typically be designated the same Operational Integrity Category to ensure the system retains operational integrity at the applicable building performance limit state.
2. NZBC requirements detailed in table were current at time of writing but are subject to change in future building code updates. Designers are advised to check current building code requirements when applying this table.
3. Classified by NZS 4219 (SNZ, 2009) as P4. However, this is based on NZS 1170.5: 2004 applying P4 classification to components required for “evacuation and life safety systems” and doesn’t take cognisance of TS 1170.5 revision to “evacuation (after earthquake) & human life support systems”. NZS 4541 (SNZ, 2020) already reflects this amendment, and future NZS 4219 / NZBC B1/VM updates are expected to do the same. However, P5 categorisation should currently be considered an NZBC alternative solution.
4. Evacuation systems with battery backup are only dependent on mains or emergency power beyond the battery autonomy duration (usually minutes or hours rather than days).
5. Applies to electrical and/or gas supplies and ventilation to cooking facilities.
6. Prescribing minimum uniform heating and cooling requirements is not considered appropriate given the large number of variables that affect occupant comfort expectations (refer to Section 6.6.2.11).
7. NZBC Clause D1.3.4 does not mandate lifts (for accessibility by people with disabilities) in the following buildings; 3 storeys where 2 upper floors have total design occupancy <50; 2 storeys where upper floor has design occupancy <40, or upper floors are not used as public reception for banks, local/central/regional government facilities, hospitals, medical/dental surgeries, medical / paramedical & other primary healthcare centres
8. Although non-functional escalators are usable as escape paths subject to BCA waiver regarding uniform stair rise requirements. Ensure seismic damage does not block escalator.
9. Not required by NZBC since escape path exits and fire shutters include electronic security override.
10. Assistive listening systems required by NZBC in Communal Non-Residential spaces with >250 people; theatres, cinemas or public halls; and aged care assembly spaces with > 20 people.
11. Refer also to Section 6.8 for additional LDSD requirements for lift cars, lift shafts and related hardware.

Commentary:

It is common for building parts and components to have more than one TS 1170.5 parts category. This means that in addition to the Operational Integrity Categories detailed in Table 6-16 above, other TS 1170.5 parts categories might also apply (refer to Section 6.2.2).

Refer to Section 6.6.2.12 – Liquid Containment for more information related to bunding requirements to protect electrical equipment from water ingress.

The Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025) provides guidance on NZBC seismic compliance requirements.

6.6.4 Direct Design Method

The Direct Design Method is applicable to building services systems or components that do not meet the Prescriptive Design Method criteria detailed in Section 6.6.3. This could include the use of experimental testing or analytical study to demonstrate adequate LDSD performance of building services systems.

Commentary:

An example of when the Direct Design Method would be applicable would be for a project when part ductility values, μ_p , greater than 1.25 are adopted for DCLS load combinations. This would be a departure from the Prescriptive Design Method and as detailed in Section 6.2.3 and such values would need to be justified by experimental testing, or similar as part of a Special Study.

6.7 Stairs and Ramps

6.7.1 Scope

This section sets out recommendations for LDSD of the following components:

- Stairs.
- Ramps.

Not included in the scope of this section are:

- Passive fire design requirements for wall and ceiling penetrations related to stairs and ramps when these occur. Refer to the Section 6.11 for further information and detail.
- Requirements for external stairs and ramps are provided in Section 6.13.

Commentary:

This section is not intended to provide detailed guidance for the design of stairs and ramps, or to replace existing building standards and guidelines. Rather this section is intended to act as an overlay and provide designers with a methodology to achieve the LDSD outcome objectives and post event performance goals as detailed in Volume Two.

6.7.2 Detail and Interface Issues

An essential part of achieving LDSD of stairs and ramps is a coordinated approach for accommodating inter-storey building deflections in stair flights, stair landings, balustrades, handrails and partition walls. For switch back stairs constructed using precast concrete a common approach is to provide a seismic movement joint at the mid-height landing as is shown in Figure 6-4 below.

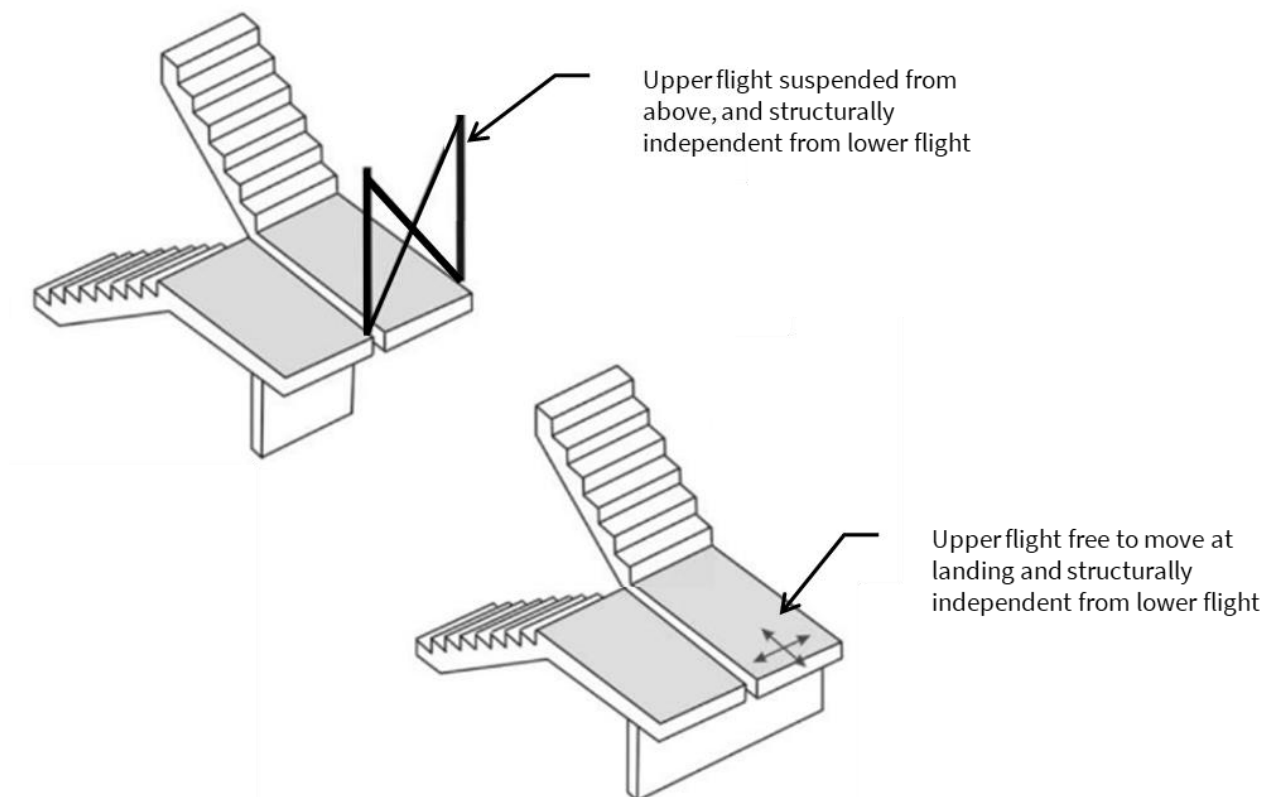


Figure 6-4 Examples of seismic detailing to accommodate inter-storey deflections in precast concrete stair flights (adapted from NZCS, 2015)

In the 2011 Christchurch earthquake several buildings which had precast concrete stairs with sliding joints at one end experienced stair collapse (CERC, 2012). In the notable case of the 18-storey Forsyth Barr building, the building was otherwise largely undamaged and building occupants had to be evacuated by means of a crane (Kam, Pampanin, & Elwood, 2011). In the 2011 Christchurch earthquake several buildings which had precast concrete stairs with sliding joints at one end experienced stair collapse (CERC, 2012). In the notable case of the 18-storey Forsyth Barr building, the building was otherwise largely undamaged and building occupants had to be evacuated by means of a crane (Kam, Pampanin, & Elwood, 2011).

It is important for the structural engineer, architect and passive fire designer to work collaboratively to ensure movement requirements are understood and stairs and ramps are properly detailed. The Building Code has a number of balustrade and handrail requirements that need to be addressed:

- NZBC F4/AS1 limits maximum opening widths in stair balustrades to 100mm. If seismic movement requirements can be less than this, then detailing of the balustrade becomes easier.
- NZBC D1/AS1 includes a requirement for handrails to be continuous on accessible stairways. This means that thought should be given to how this can be achieved while allowing for seismic movement, for example by designing a moveable/removable/easily replaceable section of handrail.

In some cases, seismic movement joints may need to be fire rated to ensure fire separation requirements are met. This is of particular importance for scissor stairs, where there is a fire requirement for each stair to be a separate fire compartment. Consequently, the stair flights themselves are fire rated, and any gaps between the stairs and internal dividing wall also need to be fire rated.

When seismic movement joints are required to be fire rated, expected movements should be communicated to the passive fire designer so correct passive fire systems can be specified.

6.7.3 Prescriptive Design Method

In addition to the requirements of the NZ Building Code, stairs and ramps are to be designed in accordance with one of the material standards cited in NZBC B1/VM1 with the additional design criteria given below.

Seismic design actions (accelerations and inter-storey drifts) required for the design of stairs and ramps should be provided by the project structural engineer in accordance with Section 6.2.1. TS 1170.5 (SNZ, 2025) should be used to determine minimum seating requirements for stair and ramp sliding joints supported on ledges.

Where seismic joints are needed in floor finishes, balustrades and handrails these should be detailed to accommodate anticipated movements associated with a DCLS event with minimal damage.

Minor damage to non-structural elements of feature stairs (i.e. stairs that are not required for emergency egress) during a DCLS event is permitted, subject to the following:

- The building owner and tenant have been advised of the reduced performance expectation for the feature stair; and
- Anticipated damage and loss of function of the stair will not prevent the building achieving *Operational State Category OS II - Partial Functionality* following a DCLS intensity earthquake.

Commentary:

Minimum seating requirements for stairs and ramps prescribed in TS 1170.5 are intended to provide confidence that gravity support of these elements will be maintained in shaking larger than implied by the levels assumed for design, and also to make an allowance for unintended or unexpected behaviour in the structure.

TS 1170.5 requires frictional forces developed at sliding supports to be considered in stair and ramp design.

In buildings with low seismic drifts, stairs and ramps are sometimes detailed without sliding joints. Careful attention must be given to deformation compatibility and the potential for interaction with the primary structure when this approach is adopted. Provision of sliding joints is generally preferred as this will typically result in a design which is more tolerant to unintended or unexpected behaviour in the structure.

Maintenance contractors filling sliding joints in stair flights, which prevented the joints from closing, was identified as a contributing factor to a number of stair collapses in the 2011 Christchurch earthquake (CERC, 2012). For this reason, joint details that are susceptible to being filled because of maintenance or construction errors should be avoided (DBH, 2011). Refer to Figure 6-5 for examples of precast concrete stair flight details which are and are not susceptible to being filled.

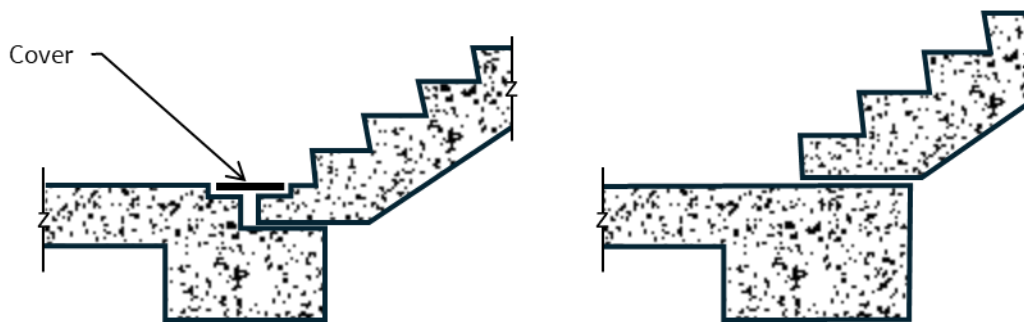


Figure 6-5 On the left is an example of precast concrete stair flight sliding support detail that is susceptible to being filled, or where the cover may pop up and prevent an adjacent egress door from opening fully, should be avoided. On the right is the preferred sliding detail that is not is susceptible to being filled.

Consideration of inertia forces and inter-storey building deflections parallel and perpendicular to the longitudinal axis of the stair flight or ramp needs to be addressed in the design. Steel framed stairs can often be detailed with sufficient flexibility to accommodate required seismic movements perpendicular to the stair flight without the need for sliding joints in this direction. However, it is important this detailing considers seismic movement in all potential horizontal directions.

In many cases it will not be possible to adopt a similar strategy for precast concrete stair flights due to the higher stiffness of these elements. Precast concrete stair flights and ramps often need to resist inertia forces perpendicular to their longitudinal axis by cantilever action. Refer to (Bull, 2011) and (NZCS, 2015) for further information on seismic detailing requirements for stairs and ramps.

LDSD requirements for feature stairs have been relaxed when they are not required for building reoccupation. This recognises these elements often have aesthetic requirements which may not be compatible with LDSD.

6.7.4 Direct Design Method

The Direct Design Method is applicable for stairs and ramps that would generally demonstrate compliance with the NZBC through an alternative solution, or which cannot meet the Prescriptive Design Method criteria. For these situations LDSD compliance could be confirmed by physical testing, or analytical study, which meets the requirements in TS 1170.5 for a Special Study.

6.8 Lifts

6.8.1 Scope

This section sets out recommendations for LDSD of passenger carrying lifts for the following components:

- Lift shaft.
- Lift car and related retaining devices.
- Guide rails.

Refer to also Section 6.6 for requirements for building services systems associated with lifts (i.e. control systems, lift machinery etc).

Commentary:

This section is not intended to provide detailed guidance for the design of lifts, or to replace existing building standards and guidelines. Rather this section is intended to act as an overlay and provide designers with a methodology to achieve the LDSD outcome objectives and post event performance goals as detailed in Volume Two.

6.8.2 Detail and Interface Issues

6.8.2.1 Seismic Performance of Lifts

Observations from past earthquakes, and industry feedback, has found counterweight derailment due to rail distortion (FEMA, 2018) tends to be the primary seismic damage state for lifts in terms of magnitude of damage and repair cost. Porter (2016) provides a detailed description of damage to lifts in past US earthquakes. Through discussions with industry, Griffiths & Chu (2023) identified a preliminary insight that other common failures in lifts (e.g. rail failure and realignment) are secondary to counterweight derailment.

Lifts typically contain seismic movement sensors which detect if the counterweight displaces. Upon detecting movement that exceeds specified limits, the sensor triggers safety mechanisms within the lift control system, stopping the lift at the nearest floor and opening the doors to allow occupants to exit. Such sensors are commonly activated during significant earthquakes and are effective at reducing damage to lifts.

Observations from past earthquakes indicate there is a strong correlation between the direction of ground shaking, damage to guide rails and rail brackets, and inter-story drift locking up door movement. To enhance resiliency, it is thus desirable to orient lift door openings so that not all are parallel to one another. This will be difficult in many buildings since lifts are typically on one side of a lobby or on opposite sides, so the doors are rotated at either zero degrees to another or 180 degrees to one another, and thus they all experience the same earthquake shaking direction. Orienting at least one lift at 90 degrees to the other lifts means that at least one lift will experience a lower level of shaking in the key direction of interest.

Experience from past earthquakes typically show after a seismic switch has been triggered and/or power is lost, the time to regain lift function is dependent on the availability of qualified lift technicians to inspect the lift, and supporting systems, and bring the lift back to function. Establishing master procurement contracts with a lift vendor might help mitigate this issue.

6.8.2.2 Seismic Isolation Plane Considerations

Care is required when lift shafts extend through isolation planes in base isolated buildings to ensure the seismic isolation plane is maintained. Referring to Figure 6-6 an approach that is commonly used is to hang the lift shaft from the superstructure. The lift shaft is also braced to the superstructure above to ensure it has adequate lateral stiffness to resist anticipated seismic inertia forces.

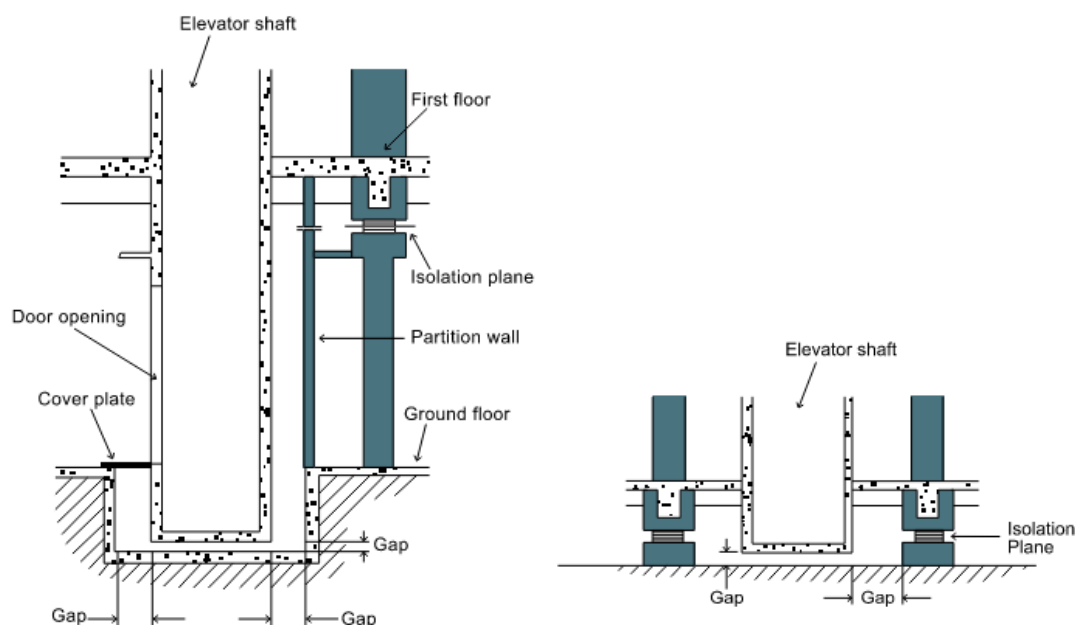


Figure 6-6 Examples of seismic isolation plane detailing at lift shafts (NZSEE, 2019)

6.8.2.3 Building Code Compliance

The NZBC compliance pathway for the structural design of passenger lifts is not straight forward:

- NZBC D2/AS1 cites NZS 4332 (SNZ, 1997) and EN 81-20 (CEN, 2014) as acceptable solutions for the design of passenger lifts. NZS 4332 has not been updated since 1997 so lift manufacturers are increasingly adopting EN 81-20 as their preferred compliance pathway.
- NZBC D2/AS1 notes EN 81-20 excludes the structural design of the lift installation and the structure supporting it, in particular with respect to seismic actions. EN 81-77 (CEN, 2013) is used for seismic design in of lifts in Europe, however the standard does not contain seismic design information for New Zealand.
- NZBC D2/AS1 instead requires the structural design of the lift installation to be undertaken by a suitably qualified designer, using NZBC B1/VM1 and EN 81-77 as considered appropriate.

Further structural design guidance for lifts is provided in Section 6.8.3 below.

6.8.3 Prescriptive Design Method

Seismic design of lifts is to be undertaken in accordance with NZBC D2/AS1 via the EN 81-20 compliance pathway described above, with the additional technical criteria detailed below. D2/AS1 cites EN 81-77:2013, however this standard has been superseded by EN 81-77:2022 and it is recommended the more recent standard be used.

Minor damage to lifts is permitted provided the damage will not prevent the building achieving Operational State Category OS II - Partial Functionality following a DCLS intensity earthquake.

Seismic design actions (accelerations and inter-storey drifts) required for the design of lifts should be provided by the project structural engineer in accordance with Section 6.2.1.

The ULS load combination for strength (refer EN 81-20:2020):

$$E_d = [1.2G + 1.5(\psi_c Q_{bldg} + Q_{lift})] \quad \text{Eqn 6-3}$$

where:

E_d = ULS design action effect.

G = permanent action or 'dead' action.

Q_{bldg} = imposed action due to building occupancy use.

Q_{lift} = supplied lift loads including dynamic or impact factors.

The 1.5 factor in Eqn 6-3 is only applied to live load for strength combinations and is not used for determining deflections or in seismic load combinations.

For seismic design of lifts in accordance with EN 81-77:2022:

- EN 81-77 'a_d' should be replaced by TS 1170.5 parts loads computed using a part ductility factor, μ_p , of 2.0 for ULS. For DCLS a part ductility factor, μ_p , of 1.25 should be used.
- In accordance with EN 81-77 Cl. 6.8.1, guide rails, their joints and attachments should comply with EN 81020:2020 Section 6.7, and they should withstand the loads and forces generated by the design acceleration. Refer to Section 6.2.4 for additional requirements for connections.
- ULS and DCLS seismic design actions are to be communicated to the lift supplier. The lift supplier is to confirm the proposed lift system has the appropriate seismic qualification for these loads.
- As per EN 81-77 Cl. 6.8.2.1, where lift seismic retaining devices are not provided, the maximum permissible deflections of the car guide rail system should comply with the requirements of EN 81-20:2020 Section 6.7, taking into account the load and forces generated by the lift system, including the forces generated by the design seismic acceleration.
- As per EN 81-77 Cl. 6.8.2.2, where seismic retaining devices are provided, refer to this clause for the permissible stresses and deflections during seismic events.
- As per EN 81-77 Cl 6.4.2, the depth of the seismic retaining device should be limited to avoid collision with guide rail attachments or other fixed devices, but long enough to guarantee a minimum overlapping length between the retaining devices and the guide rail blade of at least 5mm during an earthquake – the drift of the building should be checked to ensure this requirement is met.
- As per EN 81-77 Cl 6.4.2, the car structure and the retaining device should withstand the loads and forces imposed on them, including forces generated by the design acceleration, without permanent deformation, where the maximum clearance between the guide rail and retaining device should not exceed 5mm and the dimensions chosen should not cause accidental tripping of the safety gear during an earthquake.

Additional LDS requirements for the design of lifts:

- Guide rail midspan deflection limit: For typical guide rails, the maximum midspan guide rail deflection should be less than 12.5 mm for ULS load combinations.
- Seismic switch and annunciator power supply: The seismic switch and the annunciator should be connected to the essential electrical system.
- Go slow feature: After a seismic switch has been triggered, the elevator should have the ability operate at a "go slow" speed until the elevator can be inspected. "Go slow" speed is defined as a travel speed of not more than 0.76 m/s.
- Direction of lift door opening: When practicable orientate one or more lifts at 90 degrees to the primary lift bank to enhance seismic resiliency.

Commentary:

As noted in Volume Two for buildings less than or equal to three stories high, minor damage to lift systems may be appropriate for some building occupancy types provided the damage is readily repairable.

When considering lift design, for most projects the structural engineer will be responsible for:

- *Providing the seismic actions (floor accelerations and inter-storey drifts) required for the lift design to the lift supplier.*
- *Designing the structure supporting the lift installation (i.e. structure required to support guide rails, lifting and hitching beams, car and counterweight buffers etc). This includes ensuring the structure has adequate strength and stiffness to resist the lift design loads provided by the lift supplier.*

Similarly for most projects the lift supplier will be responsible for:

- *Designing the lift car, related retaining devices, guide rails, support brackets, car and counterweight buffers etc to resist the seismic design actions provided by the structural engineer, including related safety devices.*
- *Providing car and counterweight design loads, as well as other normal operating loads (including lifting beam loads, machine room loads, dynamic rail/pit loads etc), necessary for the design of the supporting structure to the structural engineer.*

Annex B of EN 81-77 specifies a behaviour factor of $q = 2$ which is comparable to a TS 1170.5 part ductility factor. A ductility factor of 2 is considered reasonable for the design of the guiderails and is consistent with TS 1170.5. Refer to Section 6.8.2.1 for background information related to lift door orientation seismic resiliency considerations.

6.8.4 Direct Design Method

The Direct Design Method is applicable for lift systems that would generally demonstrate compliance with the NZBC through an alternative solution, or which cannot meet the Prescriptive Design Method criteria. For these situations compliance could be confirmed by physical testing, or analytical study, which meets the requirements in TS 1170.5 for a Special Study.

6.9 Ceilings

6.9.1 Scope

This section sets out recommendations for LDSO of the following ceiling components:

- Proprietary suspended ceilings (e.g. exposed grid and tile ceilings, and plasterboard lined concealed grid ceilings).
- Specific design suspended ceilings (e.g. exposed grid and tile ceilings, and plasterboard lined concealed grid ceilings).
- Ceilings directly fixed to structure, e.g. via battens fixed to timber trusses.
- Ceilings under lightweight roof structures.
- Bulkheads (typically used to create boxed-in areas to conceal structural elements/services or to separate ceilings of different heights).

The following are not included in the scope of this section:

- The passive fire design for junctions with walls and penetrations through ceilings. Junctions with walls and penetrations through fire-rated ceilings need to be protected to the same rating as the ceiling and adequately restrained so that, in the event of seismic movement, the fire rating is not compromised. It is important the ceiling designer communicates the design movement requirements at DCLS to the fire engineer to ensure both the seismic and fire rating requirements are achieved. Refer to Section 6.11 for further information and detail related to passive fire requirements.
- Acoustic design for junctions with walls and penetrations through ceilings. Discussions regarding movement with the acoustic engineer will be required, similar to that described above for coordination with the fire engineer.

This section contains a number of references to the Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025). This document provides relevant information for the seismic design and performance of ceilings, and interfaces between ceilings and other components, and so should be read in conjunction with this section.

Commentary:

This section is not intended to provide detailed guidance for the design of ceilings, or to replace existing building standards and guidelines. Rather this section is intended to act as an overlay and provide designers with a methodology to achieve the LDSO outcome objectives and post event performance goals as detailed in Volume Two.

The most common ceiling type in commercial office buildings is a proprietary system using lightweight mineral fibre tiles set in an aluminium grid. Because of their lightweight nature, if well designed and coordinated with other components in the ceiling void, and clipping or securing of the tiles is undertaken, these ceiling systems can perform well in earthquakes.

6.9.2 Detail and Interface Issues

6.9.2.1 Key Coordination Considerations

The Low Damage Seismic Design of suspended ceilings includes the support structure, ceiling linings/tiles and the seismic restraint of the ceilings. Importantly, it also includes coordination with the architect, structural engineer, fire engineer, specialist NSE seismic designer and building services engineers to ensure the suspended ceiling design meets the required performance objectives. The design of the suspended ceiling should ensure that it does not compromise the seismic performance of other elements and contents.

Refer to 'Design Methodology & Design Phases' in Part A of the NSE CoP (BIP, 2025) which offers comprehensive guidance on the design process from project briefing through to the construction phase design. The code of practice recommends an integrated approach to incorporating NSEs into the design methodology and phases. For simple buildings all of the listed disciplines may not exist, or one entity may be responsible for multiple components.

Refer to the NSE CoP for details of the required coordination of ceilings throughout the design process. Noting that for simple projects the coordination of the ceiling with other components below can be as simple as a conversation between the parties responsible for the various components but may need dedicated coordination meetings for more complex projects.

Early coordination, for example via Early Contractor Involvement (ECI), ensures the Building Movement Strategy (refer Section 3.2.2.3) is aligned between disciplines, including ceilings, and also ensures the gravity support for the ceiling can be achieved with the necessary clearances. When considered necessary, extra space should be provided in congested areas during the design phase to allow for secondary structure. Providing seismic restraint frames that support multiple services within the ceiling void can reduce congestion and complexity.

Overall, collaboration and coordination are key to optimising the ceiling void depth and ensuring seismic performance and clearances are met.

6.9.2.2 Detail and Interface Considerations

Refer to 'Seismic Design of Suspended Ceilings' in Part C of the NSE CoP which describes key performance considerations for the design of suspended ceilings, including examples of detailing and interface issues to be aware of with suspended ceilings. Important detail and interface considerations include the following (refer to the BIP NSE CoP for more information):

- Coordination from early in project.
- Input and coordination with the Building Movement Strategy.
- Minimise ceiling penetrations. Where penetrations are required provide detailing to prevent damage, e.g. flexible droppers.
- Ensure there is deformation compatibility and agreement between partition design and ceiling design if ceilings are to be connected to partitions.
- Holistic consideration of all components in the ceiling void. Where large ceiling voids occur, diagonal braces can cause significant coordination challenges.
- Consider using tile clips or Velcro to restrain ceiling tiles and provision of maintenance access hatches.
- Consideration of components supported by the ceiling on the seismic performance of ceilings.
- Direct fixed ceilings have a lower risk of damage compared to suspended ceilings.

6.9.2.3 Examples of Good Low Damage Detailing

Examples of good low damage detailing for ceilings and their seismic restraints are included in Part C of the NSE CoP (BIP, 2025).

One of the most important aspects to achieve LDSD outcomes for ceilings is coordination of all components in the ceiling void. Figure 6-7 and Figure 6-8 below have been taken from the AWCINZ CoP (AWCINZ, 2015) and are good examples of coordination between in-ceiling services and ceiling design.

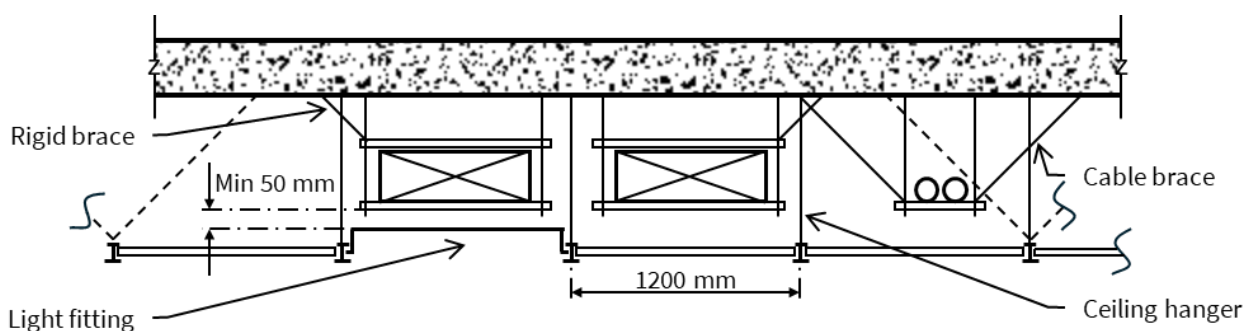


Figure 6-7 Examples of good coordination between ceiling and building services components, hangers and seismic restraints (adapted from AWCINZ, 2015).

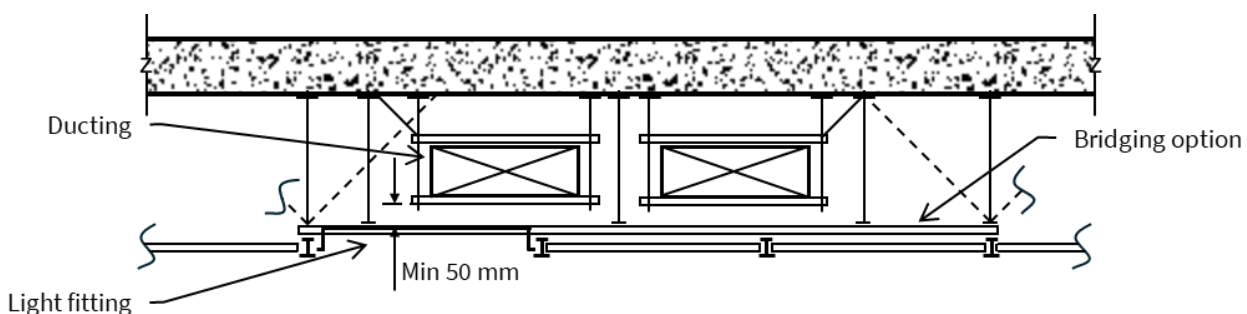


Figure 6-8 Examples of achieving required horizontal and vertical clearances and gravity support to ceiling where ducts or services obstruct access for required ceiling hanger points (adapted from AWCINZ, 2015).

6.9.2.4 Discipline Responsibilities for the Design of Ceilings

Refer to the ‘Seismic Design of Suspended Ceilings’ in Part C of the NSE CoP which outlines the specific roles and responsibilities of key parties involved in developing and coordinating the seismic performance of ceilings.

Unlike some other building components, there are often a large number of design disciplines that interface with ceilings. Their input into the design of ceilings is not always undertaken or considered in current design practice. Inter-discipline responsibilities and input into the design of ceilings is required to achieve LDSD. Discipline specific responsibilities included in the NSE CoP are to be included in designer scope of work and complied with.

6.9.3 Prescriptive Design Method

6.9.3.1 General

In addition to the requirements of the NZBC, ceilings and bulkheads should be designed in accordance with the additional technical criteria detailed below.

Seismic design actions (accelerations and inter-storey drifts) required for the design of ceilings should be provided by the project structural engineer in accordance with Section 6.2.1.

6.9.3.2 Suspended Ceilings

Suspended ceilings are to be designed in accordance with the requirements of NZBC B1/VM1 and AS/NZS 2785 (SANZ, 2020).

Designers are to refer to Part C of the NSE CoP (BIP, 2025) and (Browne, Long, & Stanway, 2024) for the design methodology and seismic performance requirements for suspended ceilings. These documents detail various design limit states and provide the recommended performance criteria to achieve the desired performance, including for the DCLS limit state.

Table 6-17 below details additional LDS design criteria for ceilings. Refer also to Section 6.11 for further information regarding passive fire requirements necessary for *Operational State Category OS II - Partial Functionality*.

Table 6-17 Additional design criteria for LDS

LDS Requirement	Description
Penetrations through acoustic and fire rated ceilings	Penetrations through acoustic and/or fire rated ceilings should consider DCLS design actions (accelerations and inter-storey drifts) and design the penetration to maintain the specified acoustic and/or fire rating for the ceiling ¹ .
Coordinate 3D movement requirements of ceilings with acoustic and fire engineers	Communicate required seismic movements and collaborate with the fire and acoustic engineers to detail a solution that meets seismic, acoustic and fire rating requirements.
Damage States	Each ceiling should be designed to achieve the appropriate Damage State detailed in Table 6-18 below.

Notes:

1. This requirement can be waived if there are compelling reasons why it cannot be achieved (i.e. would lead to unacceptable compromise of another more important aspect of the building design) provided required passive fire performance requirements are met. Furthermore, the client should be made aware of this non-compliance, and it must be clearly documented.

6.9.3.3 Ceilings Directly Fixed to Structure

Plasterboard ceilings directly fixed to the structure (e.g. ceiling battens fixed to timber trusses) should be designed in accordance with Section 6.4 while also detailed to achieve the appropriate Damage State (DS) detailed in Table 6-18 below for DCLS load combinations.

The limiting mid-span horizontal deflection of direct fixed plasterboard ceilings for DS1 can be taken as span/300.

Passive fire requirements for direct fixed ceilings should be determined in accordance with Section 6.11.

Commentary:

Sometimes direct fixed ceilings have a dual structural purpose i.e. such as a ceiling diaphragm to resist wind and seismic loading. Mid-span diaphragm deflections are to be computed with respect to diaphragm support elements (i.e. walls, braced frames etc). DCLS deflection limits for direct fixed plasterboard ceilings is a topic of active research (Liu, Li, & Shelton, 2019) and the recommendations provided in this section have been informed by judgement. Further experimental work is required validate the recommended limits.

6.9.3.4 Bulkheads

Bulkheads and related interfaces with ceilings should be designed and detailed for DCLS design actions (accelerations and inter-storey drifts), with due consideration given to deformation compatibility, to achieve the appropriate Damage State (DS) detailed in Table 6-18 below.

Bulkhead interaction with ceilings often occur at riser faces. These areas require careful consideration and detailing to meet LDS performance requirements.

Passive fire requirements for bulkheads should be determined in accordance with Section 6.11.

Commentary:

The AWCINZ Code of Practice for Suspended Ceilings (AWCINZ, 2015) provides advice for detailing bulkheads.

6.9.3.5 Damage States for Ceilings to Achieve LDSD Criteria

Refer to Table 6-18 for the ceiling criteria for different damage states.

Table 6-18 Damage states for ceiling to achieve LDSD criteria

Damage State	Description
DS1	No loss of fire-rated ceiling functionality.
DS2	Repairable damage with no loss of operational continuity ¹
DS3	Damage to joints and damage to linings. No damage to ceiling grid.

Notes:

1. For concealed grid suspended plasterboard ceilings this is defined as no damage to the ceiling grid and no cracks to the plasterboard ceiling linings. Damage to joints is acceptable, no aesthetic restoration work required. For exposed grid and tile ceilings, no damage to the ceiling grid and a maximum of 5% loss of individual ceiling tiles.

The limiting damage state applicable for ceiling design for DCLS design actions will be informed by the fire rating of the ceiling:

- Fire rated ceilings at DS1 would be considered acceptable for Operational State Category OS II - Partial Functionality because cracking at sheet joints should have framing behind, and any joint that is designed to accommodate movement should be detailed as a fire rated control joint (refer Section 6.11).
- Fire rated ceilings at DS2. Damage at this drift limit might compromise the fire rating of the ceiling. Whether such damage is acceptable for Operational State Category OS II - Partial Functionality is dependent upon the size, quantity and location of damage. This determination is to be made by the project fire engineer.
- Non-Fire rated ceilings at DS3. Damage at this limit might compromise the fire rating of the ceiling and would not generally be acceptable for fire rated ceilings is therefore applicable for non-fire rated ceilings.

Designers should also consider the functionality and finishes proposed for each ceiling to determine the damage state which will achieve LDSD performance requirements.

6.9.4 Direct Design Method

The Direct Design Method is intended for complex projects, or projects that use a new or innovative ceiling systems which cannot meet the Prescriptive Design Method criteria. For these situations LDSD compliance could be via physical testing, or analytical study, which meets the requirements in TS 1170.5 for a Special Study and confirms the ceiling system complies with the applicable damage states detailed in Section 6.9.3.5

Commentary:

An example of this approach would be a new type of floating ceiling system proposed by Pourali et. al. (2017). This ceiling performed very well under laboratory shake table testing, without the need for seismic restraints in the ceiling void. The ceiling is supported from the structure above through vertical hanger wires, the edge of the ceiling has a perimeter channel, and that channel has a clearance all around the perimeter that is filled with industrial foam (refer Figure 6-9 below).



Figure 6-9 Isolation foam placed between ceiling framing and the timber boundary member (Pourali, Dhakal, MacRae, & Tasligedik, 2017).

6.10 Partitions

6.10.1 Scope

This section sets out recommendations for LDSD of the following non-loadbearing partition components:

- Non-loadbearing lightweight proprietary partition walls.
- Non-loadbearing specifically designed timber or steel framed plasterboard partition walls.
- Glazed partition walls.

This section includes the wall types listed above that are constructed to achieve specific fire, acoustic and fire resistance ratings.

The design of non-loadbearing partition walls includes the stud/glazing design, partition linings and finishes (e.g. tiles), doors and seismic restraint of the partitions.

Not included in the scope of this section are proprietary toilet cubicle partitions and operable walls although the principles of LDSD included in this section can be applied to the design of these elements.

This section contains a number of references to the Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025). This document provides relevant information for the seismic design and performance of ceilings, and interfaces between ceilings and other components, and so should be read in conjunction with this section.

Commentary:

This section is not intended to provide detailed guidance for the design of non-loadbearing partition walls, or to replace existing building standards and guidelines. Rather this section is intended to act as an overlay and provide designers with a methodology to achieve the LDSD outcome objectives and post event performance goals as detailed in Volume Two.

There are a number of different situations where non-loadbearing partition walls will be used typically, including:

- *Walls from floor to underside of floor (or roof) over.*
- *Walls from floor to just above ceiling level.*
- *Walls from floor to underside of ceiling over.*
- *Part-height walls like glazed shower partitions.*

6.10.2 Detail and Interface Issues

6.10.2.1 Key Coordination Considerations

To achieve LDSD of non-loadbearing partition walls, early consideration of damage states, estimation of inter-storey drift demands from the primary structure, and cost implications during feasibility stages is required. A Building Movement Strategy (refer Section 3.2.2.3) is required to be developed for the project. Throughout the concept, preliminary, and detailed design stages, coordination with the design team is required to manage:

- Multi-discipline interfaces at partition walls.
- Minimise penetrations through partition walls, particularly through fire and acoustic walls, and minimise potential interactions between building components and non-loadbearing partition walls (refer also to discussion on this in Section 6.11).

- Building and component movements.
- Considerations of inspection and repairability. The partition wall, all elements that hang off it, are integrated into the wall, and penetrate the wall, should be designed so they can be readily inspected and repaired (refer also Section 6.1.4).
- Consideration of services coming into walls from the ceiling void.
- Integrate all building components with the partition walls and their seismic restraints.
- Standard proprietary fire and acoustic sealing systems typically allow for nominal (small) relative movement between the partition wall and the component penetrating the partition. Specifically designed fire rated systems are tested for movement and can cater for higher relative movement (both lateral and longitudinal where applicable). This is typically achieved by using wider sealant-filled gaps between the component penetrating the partition and the edge of the penetration. However, these systems are supported to a maximum displacement corresponding to the gap used in the product's fire rating test.
- In many cases available fire and acoustic rated systems on their own are unlikely to cater for the required combination of relative seismic, thermal and gravity displacements. This means additional services flexible connections are likely to be required. Such flexible connections require additional ceiling space and are not typically as robust as the parent service (particularly for piped services). They therefore add potential failure points and increase failure probability under both normal operating and seismic conditions. It is essential there is detailed coordination between the partition wall designer, fire engineer, building services engineer and acoustic engineer to manage these interfaces. It is recommended that flexible connections in services distribution systems be minimised as far as reasonably practicable.
- Appropriate project-specific deflection head locations should be agreed between structural, architectural, building services, passive fire and NSE seismic designers, and appropriate ceiling void depths (impacting floor-to-floor heights) determined to suit.
- Wall finishes. Brittle finishes, such as adhered veneer are particularly vulnerable to damage, either as a result of accelerations, or as a consequence of damage to the wall sheathing and should be avoided for low damage buildings.
- Ability to undertake cosmetic repairs quickly and with minimal disruption to other components after an earthquake.
- Where contents or equipment (or potential for future contents or equipment) is to be seismically restrained by a partition wall, provide studs at 400 mm centres to facilitate seismic restraint of the equipment/contents.

Refer to 'Design Methodology & Design Phases' in Part A of the BIP NSE CoP (BIP, 2025) which offers comprehensive guidance on navigating the design process from project briefing through to construction phase design. The code of practice recommends an integrated approach to incorporating NSEs into the design methodology and phases. This section should be read alongside 'Roles and Responsibilities', which defines the obligations of various design disciplines and highlights key design interfaces (including partition walls) to be addressed throughout the design process.

Specific considerations for coordinating the seismic performance of non-loadbearing partition walls are provided in Part C of the BIP NSE CoP, 'Seismic Design of Lightweight Partition Walls'.

Commentary:

Full height fire or acoustically rated partition walls in LDS buildings require careful consideration in relation to the location of the wall deflection head track and the associated detailing required. Options for the location of the deflection head track and related considerations include

- Full height partition wall with braced downstand where the deflection head track is located at the bottom of the downstand just above the ceiling (refer Figure 6-10 a). For this scenario the partition is split into two sections, the top section (known as the downstand) is hung off the floor above and moves with the floor above, a deflection head track that allows independent horizontal in-plane sliding to the lower portion of the partition wall is provided just above the ceiling. As horizontal in-ceiling services distribution systems are also hung from the floor above this system minimises the extent of differential displacement between services that penetrate the partition and the displacement of the partition itself. This option increases partition design and construction complexity, including above-ceiling partition bracing competing with services space.*
- Full height partition where the deflection head track is located at the underside of the floor structure (refer Figure 6-10 b). This option reduces the partition design and detailing complexity and therefore may provide a less expensive partition option. However full consideration of the cost and performance of partitions and services needs to be quantified. This is because this option is likely to require a combination of specifically designed fire rated partition penetrations and flexible connections for services, the latter tending to reduce distribution system robustness under seismic and normal operating conditions.*

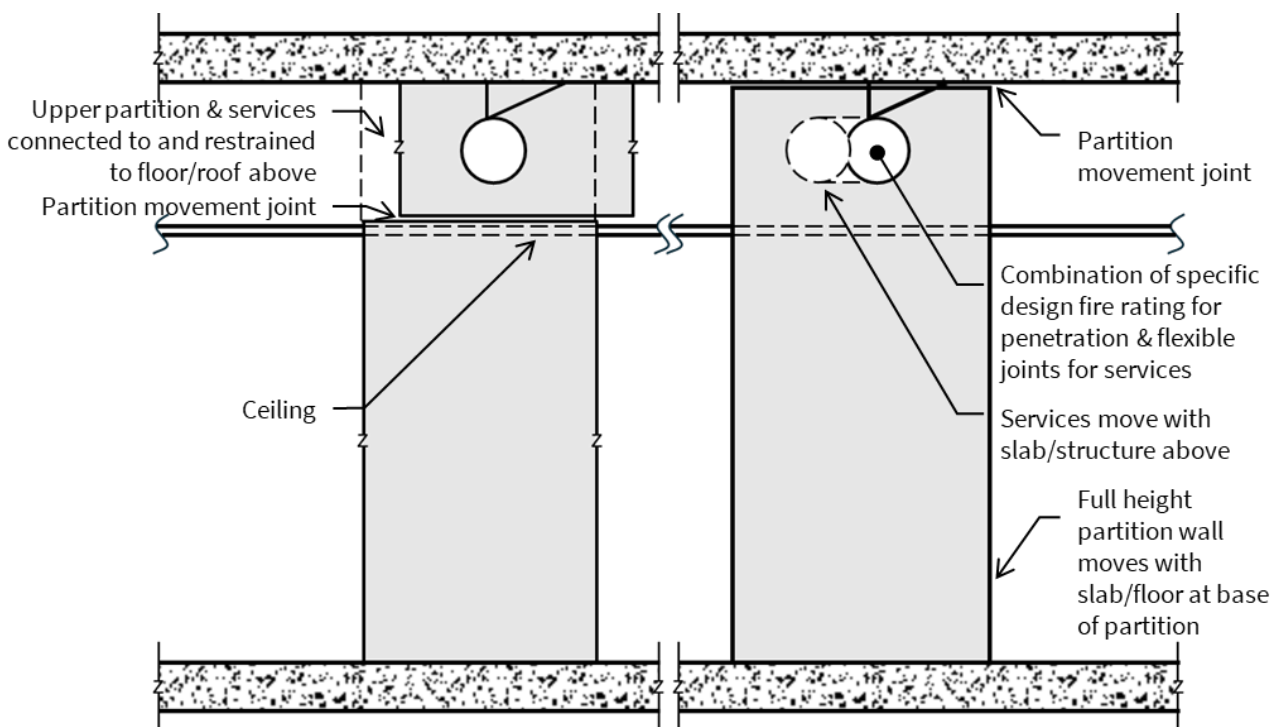


Figure 6-10 Location of deflection head track in non-loadbearing partition walls: (a) bottom of the downstand just above the ceiling (left) and (b) underside of the floor structure (right).

Appropriate allowances for thermal displacements of services that penetrate the partition walls is also required

While wider sealant-filled gaps (and correspondingly higher differential displacement capacities) may be technically feasible, many sealants have yet to be fire tested and certified for use in NZ. Such testing requires a laboratory fire test, plus a separate relative displacement test, the latter typically includes visual inspection only for physical integrity after undergoing the maximum permissible cyclical displacement, i.e. does not typically include re-testing of fire integrity after displacement. Therefore, the service penetration is likely, but not necessarily assured, of retaining fire integrity after a significant earthquake event.

6.10.2.2 Detail and Interface Considerations

The LDSD process for non-loadbearing partition wall systems should include coordination with the architect, structural engineer, non-structural seismic designer, fire engineer, acoustic engineer and building services engineer to ensure the non-loadbearing partition wall design meets the performance requirements for all structural and non-structural elements; and contents that are close to, hung off, connected to, or penetrate the partition. The design of the partition should ensure that it does not compromise the performance of other elements and contents.

Should compromises on the performance of the partition or other components/contents be found to be necessary this should be managed in accordance with the process detailed Section 3.2.2.3.

If there are non-loadbearing partition walls that need to be inspected following an earthquake to enable building reoccupation (i.e. non-redundant fire rated walls), designers should consider how the partition walls are to be inspected, including access to areas above ceilings (refer to related requirements in Section 6.1.4).

Refer to ‘Seismic Design of Lightweight Partition Walls’ in Part C of the BIP NSE CoP which describes key performance considerations for the design of partitions walls, including examples of important detailing and interface considerations such as:

- Coordination from early in project.
- Early consideration of anticipated building inter-storey drifts and potential locations for partition wall movement joints.
- Input and coordination with the Building Movement Strategy.
- Minimising partition wall penetrations (refer also related discussion in Section 6.11 for fire rated partition walls).
- Partition wall corners and intersections.
- Considerations for partition walls in stair wells and vertical shafts.
- Impact of partition wall movement joint location on building services (refer Figure 6-10).
- Ensure there is deformation compatibility and agreement between partition designer and ceiling designer if ceilings are to be connected to partitions.
- Reduce drift limits for partitions in wet areas with membrane. In the case of wet areas, drift limits should avoid damage to the waterproof membrane at DCLS intensity shaking.
- Coordinate with in-ceiling components.
- Impact of linings and finishes on the seismic performance of partitions. Detailing and wall finishes can affect the seismic performance of the partitions.
- Doors can cause concentration of damage in partitions.
- Avoid fire doors becoming jammed.
- Detailing of fire and acoustic seals.
- Consideration of contents on the seismic performance of the partitions.

6.10.2.3 Examples of Good Low Damage Seismic Design Detailing

Good LDSD detailing for partition walls will vary for each project as the ‘low damage partition’ depends on the drift of the building at each performance limit state.

Examples of good LDSD detailing for plasterboard and glazed partition walls are included in Part C of the BIP NSE CoP in 'Seismic Design of Lightweight Partition Walls' and also discussed in the *AWCINZ – Code of Practice – For Seismic Design and Installation of Non-Structural Internal Walls and Partitions* (AWCINZ, 2018).

'Box within box' construction

One way to eliminate damage to partitions caused by inter-storey drifts is to not connect the tops of walls to the floor above. One way to achieve this is by building a box within the 'box' of the structure. This is suited to small rooms like bathrooms where ceilings can be lower, structural spans for the 'roof' of the box (which is in fact the ceiling) are shorter, and there are often more solid walls that can brace and stiffen the box. Moreover, in the case of bathrooms, the stiffer lightweight structure is less prone to the sorts of movements that can also damage waterproof membranes.

6.10.2.4 Discipline Responsibilities for the Design of Non-loadbearing Partition Walls

Refer to the 'Seismic Design of Lightweight Partition Walls' in Part C of the BIP NSE CoP which outlines the specific roles and responsibilities of key parties involved in developing and coordinating the seismic performance of non-loadbearing partition wall systems.

Unlike some other building components, there are often a large number of consultants that interface with partition walls. Their various inputs into the design of non-loadbearing partition walls are not always undertaken or considered in current design practice. Inter-discipline responsibilities and input into the design of non-loadbearing partition walls is required to achieve LDSD. The discipline specific responsibilities included in the BIP NSE CoP are to be included in designer scope of work and complied with.

6.10.3 Prescriptive Design Method

In addition to the requirements of the NZBC, non-loadbearing partition walls should be designed in accordance with the additional technical criteria detailed below.

The design recommendations for non-loadbearing partition walls in Part C.6 of the BIP NSE CoP should be used for LDSD.

Seismic design actions (accelerations and inter-storey drifts) required for the design of non-loadbearing partition walls should be provided by the project structural engineer in accordance with Section 6.2.1.

Commentary:

This Prescriptive Design Method is limited to plasterboard lined partition walls. Other types of non-loadbearing partition walls (i.e. glazed partitions, prefabricated steel sheet with aerated concrete core partitions etc) do not currently have sufficient testing available to be included in the Prescriptive Design Method.

The BIP NSE CoP (BIP, 2025) provides guidance on the minimum acceptable standards for the design, coordination, selection, and construction of non-loadbearing partition walls to meet the performance requirements of the NZBC. The document also provides guidance on how to design and construct systems that perform better than code minimum.

The BIP NSE CoP (BIP, 2025) provides guidance on the minimum acceptable standards for the design, coordination, selection, and construction of non-loadbearing partition walls to meet the performance requirements of the NZBC. The document also provides guidance on how to design and construct systems that perform better than code minimum.

6.10.4 Direct Design Method

The Direct Design Method is applicable for non-loadbearing partition walls that would generally demonstrate compliance with the NZBC through an alternative solution, or which cannot meet the Prescriptive Design Method criteria (i.e. including non-loadbearing partition walls other than plasterboard lined walls). For these situations, designers should coordinate with partition wall suppliers to confirm the detailing required to ensure the LDSD damage control and improved functionality performance goals defined in Volume Two are met.

When using the Direct Design Method LDSD compliance can be confirmed through one or more of the following options:

- Analytical analysis, where acceleration and inter-storey drift demands are provided by the structural engineer and the stiffness of the partition system is calculated using material standards. The analysis should confirm damage sustained by the partition systems at each building limit state is acceptable.
- Physical testing in accordance with a recognised standard testing procedure such as ICC-ES AC156 (ICC-ES, 2007) provided the substantiated seismic capacities equal or exceed the seismic demands provided by the structural engineer.
- Use of experience-based evidence to demonstrate LDSD compliance. In order to meet this compliance route:
 - i. The detailing of the proposed partition type must match the partition experience data being referenced
 - ii. The designer must be able to confirm the floor accelerations and building inter-storey drift demands the subject partition was subjected to equal or exceed the seismic demands provided by the structural engineer.
 - iii. Damage sustained by the subject partition wall was acceptable in terms of meeting LDSD requirements.

6.11 Passive Fire

6.11.1 Scope

This section sets out recommendations for LDSO of the following passive fire components:

- Fire rated walls and floors (also refer to Sections 6.9 and 6.10 for related recommendations for ceilings and partitions).
- Fire rated doors, windows and hatches.
- Pipe, cable and duct penetrations through fire rated elements (walls and floors).
- Non fire rated building elements penetrating fire rated elements (e.g. rafters, joists and braces penetrating fire rated walls).
- Fire rated control joints between fire rated elements (wall and ceiling elements and wall/ceiling junctions).
- Junctions between stair treads and walls; and landings and walls.
- Fire rated control joints between facades and fire rated walls and floors.
- Fire rated seismic control joints in walls, floors and fire rated roofs.
- Fire rated cavity barriers within external cladding systems.
- Fire protection of structural members (e.g. steel and timber).

Commentary:

This section is not intended to provide detailed guidance for the design of passive fire components, or to replace existing building standards and guidelines. Rather this section is intended to act as an overlay and provide designers with a methodology to achieve the LDSO outcome objectives and post event performance goals as detailed in Volume Two.

6.11.2 Detail and Interface Issues

6.11.2.1 General

Passive fire systems must integrate with many other building components. Collaboration with other members of the project design team is essential to achieve this integration. Passive fire designers should review related subsections of Section 6, particularly those on Building Structure, Building Envelope, Stairs and Ramps, Building Services, Partitions and Ceilings to align performance goals and constraints effectively.

6.11.2.2 Good Design Principles

The concept and preliminary design stages are critical for shaping passive fire designs to ensure products and systems can be selected that meet both seismic and fire rating performance criteria during detailed design.

The first steps should be to simplify the project by designing out requirements for elaborate passive fire detailing or systems. Options available to the design team in conjunction with the fire engineer are to:

- Limit inter-storey drift (Section 6.4) and locate zones of low relative inter-storey drift to run services. Limiting inter-storey drifts will increase the range of passive fire systems that could be suitable.
- Minimise fire separations by incorporating additional fire safety features (such as sprinklers) and reconfiguring building layouts.
- Reduce service penetrations through fire-rated elements by redesigning or rerouting building services systems around them, for example by installing plumbing on non-fire rated walls.

- Optimise structural steel fire protection using performance-based structural fire engineering.
- Address three-dimensional movement in buildings by incorporating details or components that accommodate the movement, while accepting that controlled easily repairable damage may need to be accepted to simplify the design. This approach is particularly relevant for plasterboard walls and ceilings, which pose significant challenges in achieving compliance with three-dimensional movement requirements. It is essential that this three-dimensional movement is considered and documented in the Building Movement Strategy (refer Section 3.2.2.3).
- Ensure access for inspection and replacement of fire rated systems when damage is expected (refer related discussion in Section 6.1.4).
- Select fire-stop systems with inherent seismic capability.
- Select easily repairable firestop systems where damage is expected.

6.11.2.3 Impact on fire rated building elements

There are many fire-rated elements in a building, and damage after a seismic event can be categorised into two main types:

1. Damage to the fire rated element itself such as a wall or a floor because it is not sufficiently flexible to accommodate inter-storey building movement that occurred during the earthquake.
2. Damage to a fire-stopping system at a junction between two different fire rated elements (e.g. a wall to floor interface or the firestopping between a pipe and a wall or floor) due to differential movement.

Table 6-19 below identifies generic issues and solutions for each type of fire rated element.

Table 6-19 Impact of seismic movements and acceleration on fire rated building elements

Building Element	Impact of seismic movements and accelerations	Solutions
Fire rated walls and floors	Cracks in the tape and plaster in sheet joints. Fixings in plasterboard walls become loose. Cracks in the wall or floor material (plasterboard, timber, concrete etc) result in a loss of integrity allowing smoke and fire to pass through.	Refer Sections 6.9 and 6.10 for design advice for ceilings and partition walls. Provide access for inspection and repair of damage. Refer to advice below on interim tolerable crack limits.
Fire rated doors windows and hatches	Fire doors and hatches may jam open or closed. Cracks can occur in wall linings around doors. Glazing is unable to accommodate significant shear or torsion forces. Differential movement can occur in fire-rated control joint around door/window/hatch frames.	Refer Sections 6.9 and 6.10 for design advice for ceilings and partition walls. Ensure walls around fire and smoke control doors are sufficiently rigid to maintain necessary functionality post-seismic event.
Pipe and cable penetrations through fire rated elements (walls and floors)	Services may experience up to three degrees of movement relative to penetrated walls or floors. Performance of firestopping might be degraded by movement.	Consider services routes that do not pass through fire rated elements. Accommodate movement in the pipe or cable away from the firestop. Locate in zones of low inter-storey drift.
Duct penetrations through fire rated elements (walls and floors)	Fire dampers are fixed relative to walls or floors, requiring fixings and supports that can withstand seismic accelerations.	Fire dampers need breakaway joints on both sides to prevent being dislodged during a fire. Choose flexible duct sections to also accommodate seismic movement. Optimise duct routes to minimise penetrations through fire rated elements. Fire damper installation is to be as per manufacturers recommendation.
Non fire rated building elements penetrating fire rated elements (e.g. rafters and joists penetrating walls)	Differential movement possible between the service and the wall or floor being penetrated. Fire-rated sealants used to seal gaps often have limited movement capacity and the performance of the sealants might be degraded.	Refer Sections 6.9 and 6.10 for design advice for ceilings and partition walls. Use sealants designed to accommodate seismic movement while maintaining fire ratings. Ensure access for post-seismic inspection and sealant replacement.
Control joints between fire rated elements	Differential movement can occur between fire-rated elements (e.g., walls and floors). Fire-rated sealants used to seal gaps often have limited movement capacity and the performance of the sealants might be degraded.	Refer Sections 6.9 and 6.10 for design advice for ceilings and partition walls. Use sealants capable of accommodating seismic movement while maintaining fire ratings. Ensure access for post-seismic inspection and sealant replacement.
Fire rated control joints between facades and fire rated walls and floors	Differential movement occurs between facades and fire-rated floors and walls.	Limit curtain wall movement to within the joint capacity. Select control joint systems that have been subject to movement before fire testing. ASTM E2307 (ASTM, 2023) has provisions for seismic cyclical testing before fire testing. ASTM E2307 (ASTM, 2023) has provisions for seismic cyclical testing before fire testing.
Fire rated seismic control joints	Differential movement between the different building elements.	Ensure the control joint in its compressed state has adequate space for the firestopping system.

Building Element	Impact of seismic movements and accelerations	Solutions
Fire rated cavity barriers in external ventilated cladding systems	Differential movement between the cladding and base wall system. Cavity barriers can become dislodged or fall out.	Select intumescent systems which have a gap between the base wall and cladding. For mineral wool systems that fully fill the gap, ensure they allow for differential movement. Fix all systems securely to withstand seismic accelerations.
Fire protection of structural steel and timber (board systems)	Building inter-storey movements generate shear forces, leading to cracks in board protection systems.	Design the framing supporting the board system to move independently of the structure or provide sufficient clearance for movement.
Fire protection of structural steel and timber (spray on systems)	Spray-on systems like intumescent paint and cementitious coatings typically tolerant to seismic demands. However, damage can occur if the surrounding structure physically contacts/impacts the fire-rated systems.	Maintain sufficient clearance between spray-protected systems and adjacent building elements to prevent damage/contact.

Effective solutions require collaboration across disciplines. As part of the development of a Building Movement Strategy (refer Section 3.2.2.3), the passive fire designer works collaboratively with the wider project team to develop an agreed approach that satisfies the necessary project requirements. These discussions are critical during early design stages to avoid locking in decisions prematurely.

6.11.2.4 Performance of fire-stop systems after a seismic event

LDSD Category Level 2 and 3 buildings are expected to achieve *Operational State Category OS II - Partial Functionality* following a DCLS intensity earthquake (refer Section 6.1.7). Volume 2 provides guidance on the expected physical state of building components following such an event.

Damage can be categorised into various levels:

1. Damage that is minor and does not significantly reduce the fire rating performance of the element (e.g. minor cracks in partition walls that are considered acceptable in terms of passive fire requirements but will eventually need to be repaired for aesthetic reasons).
2. Damage that is readily accessible and repairable.
3. Damage that requires removal and replacement of the entire element (e.g. such as relining an entire wall or rehanging a door).

Consideration can also be given to the level of compliance that is required to achieve *Operational State Category OS II - Partial Functionality*. For example:

1. A lower than specified fire rating might be acceptable for a time if the rating still exceeds the expected time for building evacuation to occur.
2. A management plan which includes a security warden acting in lieu of fire separations or an operational fire alarm system, until necessary repairs are completed.
3. Reduced occupancy may be an acceptable means of lowering the risk. Fire safety features increase with the number of occupants. A temporary reduction in occupants may allow the building to fall under a lower threshold.

Walls, floors, and passive fire systems are typically tested to AS 1530.4 (SA, 2005) and AS 4072.1 (SA, 2005). AS 1530.4 offers some guidance on how cracks impact the integrity of walls, floors, and firestop systems. BRANZ reports SR304 (Collier, 2013) and SR410 (Frank, Baker, & MacIntyre, 2018) provide additional insights into fire

performance for cracked plasterboard walls and non-compliant fire-stopping systems. These references can assist in evaluating post-earthquake compliance and determining building re-occupancy safety.

However, these standards lack requirements for pre-seismic movement testing. Some fire-stop systems retain fire performance after seismic movement due to their design, but many do not. ASTM E3037 (ASTM, 2020) outlines a methodology for movement testing before fire performance assessment. Although not cited in New Zealand building code regulatory system, the standard is a valuable reference to use as a starting point for evaluating post-seismic fire-stop performance.

With the limited availability of tested systems (i.e. evaluating seismic and fire performance), passive fire designers need to understand the interactions between fire-rated elements, building services, and the seismic movements that need to be considered.

6.11.2.5 Fire-stop systems with inherent seismic capability

Examples of fire-stop systems and interfaces that are able to inherently accommodate seismic movement are shown in Figures 6-11 to 6-14 below. In addition, refer to Section 6.10 which provides guidance for partition walls, including placing movement joints in walls to minimise differential seismic movement at service penetration locations.

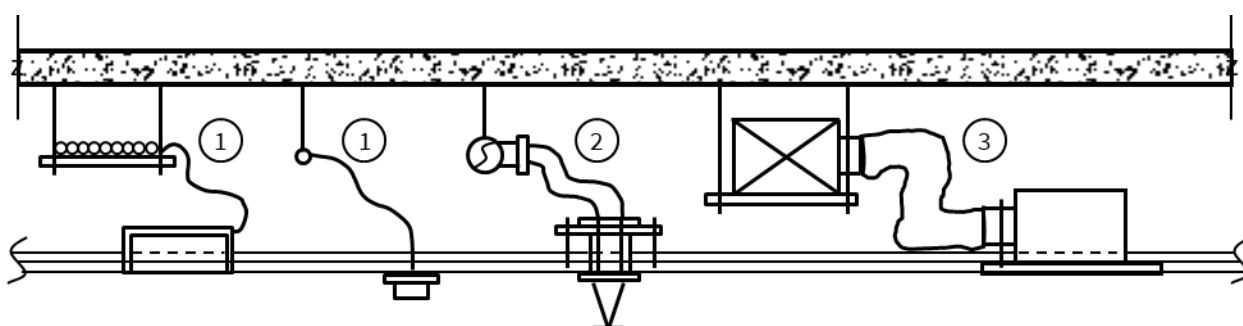


Figure 6-11 Inherently flexible connections to services penetrations in a fire rated ceiling. (1: Cables; 2: Flexible sprinkler pipe; 3: Flexible duct.)

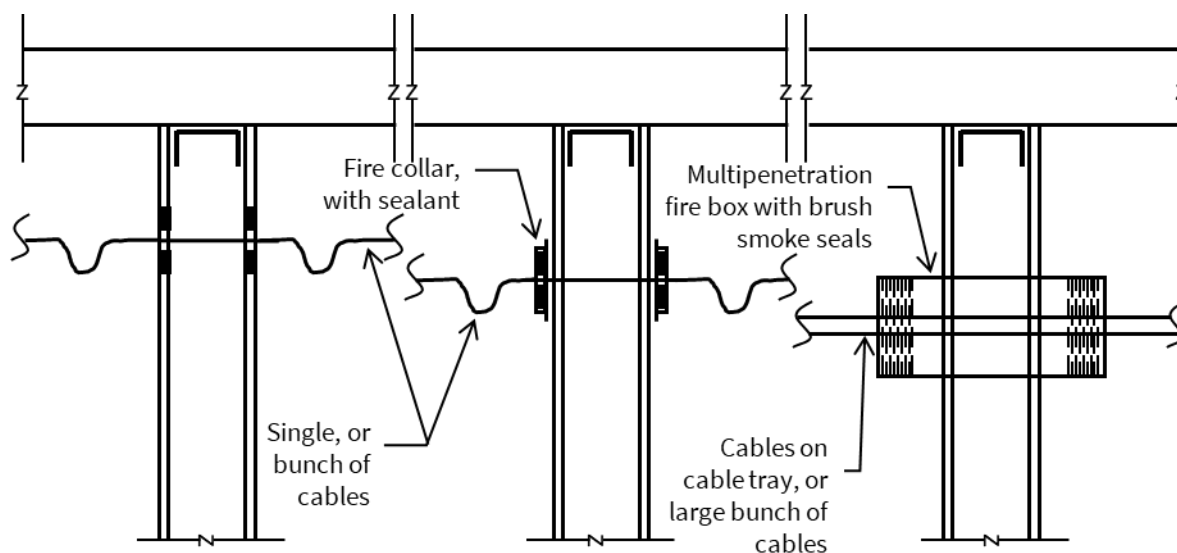


Figure 6-12 Wall penetrations that can be inherently flexible. Cables with slack either side of the walls. Multi-penetration firebox with sufficient clearance between services and the box to accommodate seismic movement

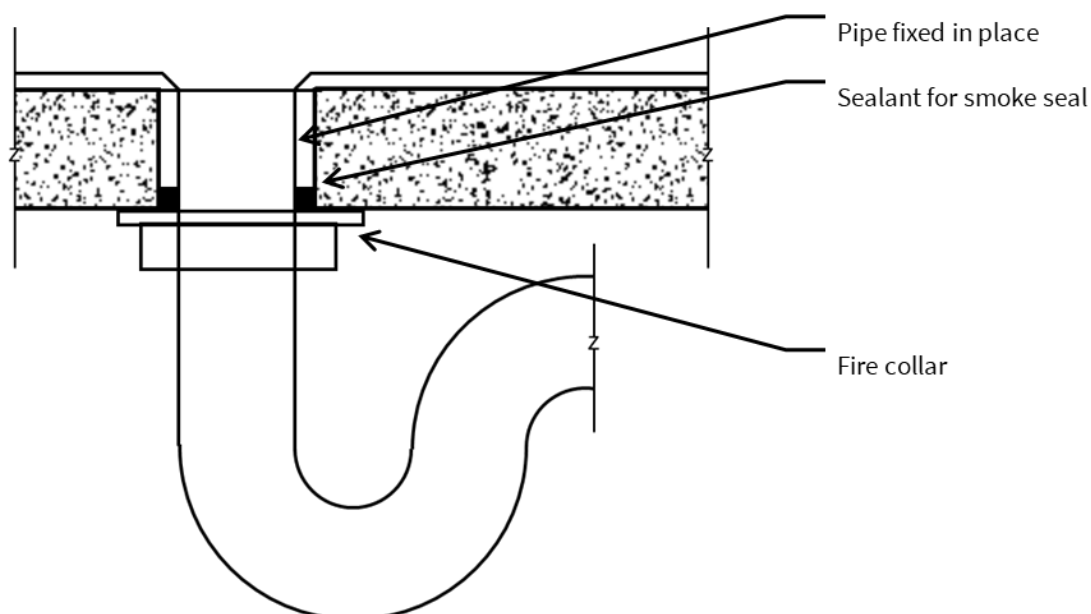


Figure 6-13 Floor penetrations with the pipe fixed rigidly to the floor so that firestop does not need to accommodate any movement

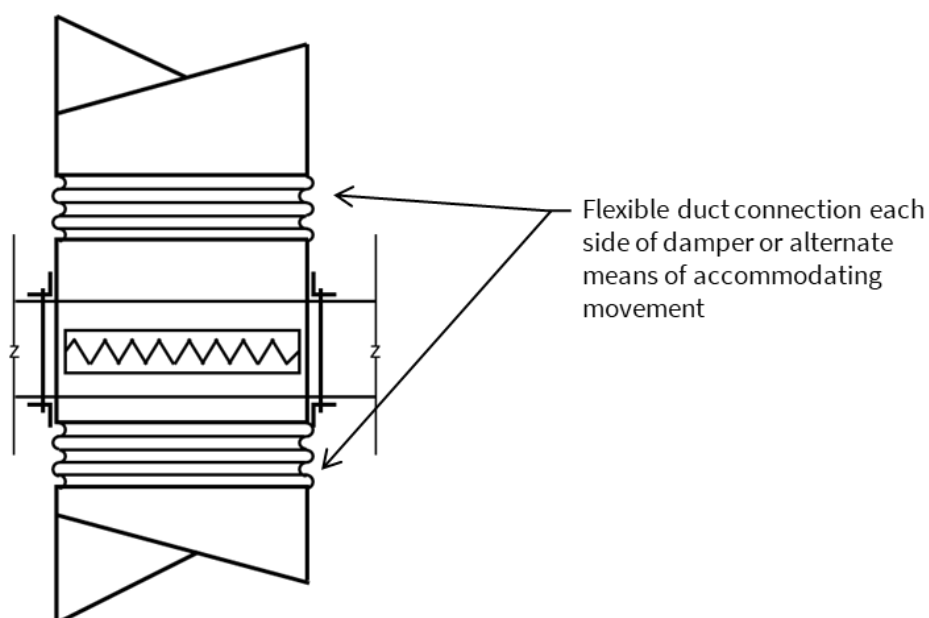


Figure 6-14 Fire dampers must be rigidly fixed to the wall or floor they are passing through. Movement needs to be accommodated either side of the damper.

6.11.3 Prescriptive Design Method.

In addition to the requirements of the NZBC, passive fire systems should be designed in accordance with the additional technical criteria detailed below.

Passive fire systems required for *Operational State Category OS II - Partial Functionality* should be designed and detailed to meet one of the following criteria:

1. The system has adequate capacity to withstand DCLS intensity earthquake design actions without a significant reduction in fire rating performance, or
2. The system can be economically repairable within acceptable timeframes (refer to Volume Two).

DCLS seismic movements needed for the design of passive fire systems should be provided by the consultant(s) that has primary responsibility (refer to Section 6.1.3) for the design of the affected building element(s).

Table 6-20 details compliance pathways which can be used to demonstrate adequacy of passive fire systems.

Table 6-20 Passive fire performance after DCLS intensity event and compliance pathway

Building Element	Performance	Compliance pathway
Fire rated walls and floors	Cracks in the tape and plaster in plasterboard sheet joints. No damage to primary structure (concrete walls and floors). Refer to control joints requirements below. Fixings in plasterboard walls become loose.	Refer Sections 6.9 and 6.10 for design methodologies for partition walls and ceilings, including inter-storey drift limits. No damage is expected to primary structural walls and floors that would affect their performance as fire separations.
Fire rated doors windows and hatches	Fire doors and hatches must not jam closed or open. Cracks can occur in plasterboard wall linings around the doors. Refer to control joints for fire-stopping between the door/window frame and wall.	Refer to Section 6.10 for the design methodology and inter-storey drift limits for partition walls to achieve the performance criteria.
Pipe and cable penetrations through fire rated elements (walls and floors)	Fire-stopping has the same performance after an DCLS event. Where fire stop systems are easily accessible use systems that are easily repairable or replaceable.	Shake test system to ASTM E3037 and then fire test to AS 1530.4. Restrain services relative to the wall/floor they are penetrating, then use standard non seismic tested systems. Use firestop systems that inherently accommodate movement due to large clearances.
Duct penetrations through fire rated elements (walls and floors)	Fire dampers have the same performance after a DCLS event. Refer to control joints below for fire-stopping between the damper and wall or floor. Fire rated Kitchen extract ducts maintain the fire rating through a fire rated wall or floor.	Specify fire damper that has adequate restraint in the wall or floor it is installed in. Design or specify system so the required seismic movement can be accommodated with flexibility in the ductwork either side of the fire damper.
Non-fire rated building elements penetrating fire rated elements (e.g. rafters and joists penetrating walls)	Fire-stopping has the same performance after an DCLS event. Where fire stop systems are easily accessible use systems that are easily repairable or replaceable.	Shake test system to ASTM E3037 or similar and then fire test to AS 1530.4. Restrain non-fire rated building elements relative to the wall or floor it is penetrating then use standard fire-stopping system tested to AS 1530.4. Use firestop systems that inherently accommodate seismic movement due to large clearances.
Control joints between fire rated elements	Fire-stopping has the same performance after an DCLS event. Where fire stop systems are easily accessible use systems that are easily repairable or replaceable.	Use fire-stopping systems that have shake-tested to ASTM E2307 or similar and then fire test to AS 1530.4.
Fire rated control joints between facade and fire rated walls and floors	Fire-stopping has the same performance after an DCLS event. Where fire stop systems are easily accessible use systems that are easily repairable or replaceable.	Use systems that have been shake tested to ASTM E3037 or similar then fire tested to ASTM E2307 or BS EN 1364-4. ASTM E2307 also has provisions for seismic movement testing prior to the fire test which can be used.
Fire rated seismic control joints	Fire-stopping has the same performance after an DCLS event.	Movement tests followed by fire test to AS 1530.4
Fire rated cavity barriers in external ventilated cladding systems	Fire-stopping has the same performance after an DCLS event.	Ensure clearance between the cladding and base wall can accommodate the movement. Select a passive fire-stop system (usually an intumescent strip) that can span the gap if there is a fire.
Fire protection of structural steel and timber (board systems)	Damage limited to cracks at sheet joints.	Refer to Section 6.10 for advice.
Fire protection of structural steel and timber (spray systems)	Fire protection system has the same performance after an DCLS event.	Detail system so movement of adjacent components will not physically contact and damage the spray applied system.

Commentary:

Default clearances provided in NZS 4219 (SNZ, 2009) Table 15 should not be relied upon because they may not always be adequate. Seismic movement requirements should be computed from inter-storey drift information provided by the structural engineer (refer Section 6.2.1).

6.11.4 Direct Design Method

6.11.4.1 General

The Direct Design Method may be appropriate for passive fire systems that do not meet the Prescriptive Design Method criteria detailed in Section 6.11.3. This includes passive fire systems that cannot maintain their fire rating or be repaired within acceptable timeframes to achieve *Operational State Category OS II - Partial Functionality* following a DCLS intensity earthquake (refer to Section 6.1.7)

6.11.4.2 Methodology

A risk-based methodology which can be used to assess adequacy of passive fire systems is outlined below. The methodology only applies within the following limits:

- Fire doors must operate effectively in both open and closed positions.
- Other doors must remain openable.

Step 1

Determine the extent of defects (i.e. gaps or cracks) that are expected to be present following a DCLS intensity earthquake.

Step 2

Determine if an integrity failure has occurred using AS 1530.4, which permits gaps up to 6mm by 150mm. If gaps or cracks are expected to exceed this threshold, redesign the system to comply. If they are likely to remain below the threshold, proceed to Step 3.

Step 3

Prepare a risk matrix using AS/NZS ISO 31000 (SANZ, 2009) to compare damage impacts on life safety, property protection, repair time, and cost. Defects in fire ratings for escape routes carry greater consequences than those between firecells. For example, a fire rating defect in a lower roof protecting other property poses less risk than defects in single means of escape stair firewalls. Similarly, defects in structural steel protection may reduce performance but not necessarily to unsafe levels.

Use NZBC C/VM2 to assess the life safety and property protection consequences of the post-earthquake scenario when the passive fire system is damaged (i.e. has defects).

Assess the performance of the damaged passive fire systems to resist spread of fire using BRANZ Study Reports SR304 (Collier, 2013) and SR401 (Frank, Baker, & MacIntyre, 2018) and any available manufacturer test data.

Determine time and cost estimates to repair the defects.

Categorise the life safety and property protection consequences to High, Medium or Low. Then rank the time and costs to repair the defects Low, Medium or High. As detailed in Table 6-21 below assess the compliance of the passive fire system design for the post-earthquake scenario:

- Defects that have a high consequence, and a high time and cost to repair, do not comply and the fire protection system will need to be redesigned.

- Defects that have a low consequence, and a low time and cost to repair comply and are acceptable.
- Defects that fall in between require further consideration, but where possible they are designed out of the project.

For defects that have an acceptable repair cost but do not meet reoccupation timeframes, consider management plans, or reduced or restricted occupancy until necessary repairs are completed (refer Section 6.11.2.3).

Table 6-21 Risk matrix for assessing compliance

Life Safety and Property Protection Consequence	Time and Cost to Repair		
	Low	Medium	High
Low	Green	Green	Yellow
Medium	Green	Yellow	Red
High	Yellow	Red	Red

Legend	
Green	Design complies
Yellow	Further consideration Required
Red	Design does not comply

Step 4

In the Operating and Maintenance manual for the building include the following (refer Section 7.4):

- Quantify the acceptable level of damage after a seismic event (fire engineer, structural engineer).
- Document firestops that do not meet the differential movement criteria and will require inspection and repair or replacement (passive fire designer).
- Document the provision for access to inspect damage to passive fire components, and the methodology to repair the damage (passive fire designer).
- Document any management plans that need to be put in place between the time that occupancy occurs, and repair of the defects is completed (fire engineer).

Commentary:

The lead designer for the affected component will provide the displacements required for the passive fire design.

6.12 Building Contents

6.12.1 Scope

In general, LDSD does not apply to contents. However, there will be some situations where the protection of contents will be important for the building to continue its function following a seismic event, or because of the inherent value of the contents. Examples include retail and data storage, museum and art gallery exhibits, hospital and laboratory equipment etc. These considerations will be identified during the briefing process and may require a project specific LDSD response as noted in Section 3.2.1.

In other situations, the movement of contents may cause consequential damage to other important building components (e.g. when liquids are stored above sensitive electronic equipment). In these instances, the design team should consider restraint of these contents as a minimum.

It is also acknowledged many items of content will be provided and installed by a tenant or user without the knowledge or input from the design team but, where possible and where the knowledge exists, advice should be given for these items. This could be in the form of a Tenant Design Guide (refer Section 7.4).

Finally, there are compliance related situations where contents should be restrained or relocated (e.g. if their failure might block an egress pathway).

This section sets out recommendations for LDSD, should this be required, for the following typical building contents:

- Built-in joinery and furniture.
- Loose furniture and joinery items.
- Desking.
- Shelving systems
- Mobile compact shelving.
- Safes.
- Computers and monitors.
- Audio Visual Equipment.
- IT Server racks.
- Equipment including printers, copiers etc.
- Appliances.
- Loose contents like glassware, crockery etc.
- Artworks.
- Valuable or irreplaceable items.

This section does not provide detailed guidance for hazardous substances. When present, risks associated with storing hazardous substances should be managed in accordance with the requirements of the Health and Safety at Work (Hazardous Substances) Regulations 2017 and NZBC Clause F3 Hazardous Substances and Processes.

For items of building plant and equipment, e.g. air handler units, emergency generators etc., refer to the relevant components section as these items will usually be specified or designed by the relevant specialist consultant.

Commentary:

While contents are usually covered by a separate insurance policy to the building, it is still worthwhile considering a LDS approach to building contents because damage caused to and by contents can affect building functionality following an earthquake and can be costly.

6.12.2 Detail and Interface Issues

6.12.2.1 Key Coordination Considerations

Discipline coordination considerations for the seismic design of contents are included in Table 6-22 below.

Table 6-22 LDS coordination considerations.

Discipline	Design Coordination Considerations
All	At key project stages consider the following: Concept Design: a) Consider if there are any unusual or important contents and how they are best dealt with to minimise the risk of damage from earthquake, tsunami or settlement etc. b) Consider the best location for large or heavy building contents that will impact the overall building design¹. c) Consider the best location for building contents that will affect the time to return the building to an operational state following a seismic event and those items that are of high value. Preliminary/Developed Design: a) Decide which contents require specialised seismic protection, for example, by means of individual base isolation for the item of content only, or by additional bracing to the content, fitout, building fabric or structure or, where appropriate, allowing sliding of the item of content can be an effective seismic design strategy. b) Develop a project strategy to seismically restrain contents. c) Coordinate with the wall/partition designer for all contents that require a wall restraint, to ensure the wall and all connections are designed to provide a robust load path from the content item and that the system achieves the necessary seismic performance. Detailed Design: a) Coordinate to ensure the movement of contents will not impact or be impacted by other building components. b) Include details of seismic restraints or a contractual monetary allowance for this.

Notes:

1. Refer to TS1170.5 (SNZ, 2025) Clause C8.1.1 for guidance on when a part or component is likely to affect the response of the building.

Commentary

Mobile compact shelving commonly presents a challenge because of the dynamic loads that can develop during an earthquake and the higher live load demands that are associated. Generally, this type of shelving should be discouraged in the upper floors of buildings in areas of high seismicity.

6.12.2.2 Detail and Interface Considerations

Examples of detailing or coordination issues to be aware of with contents are included in Table 6-23 below.

Table 6-23 Detail and Interface Issues

Item	Issue to be considered
Seismic response	The dynamic response of contents can be improved by changing the item's structure, dimensions and support conditions. This can be achieved by anchoring the content to the building or allowing the items to slide easily on the floor (albeit tethered where they may hit other components or building parts). Generally, the best solution is chosen on a case-by-case basis.
Holistic consideration of contents	Designing seismic restraints for contents in a well thought out and integrated manner, that isn't inconvenient for day-to-day use, is a better solution than a potentially ugly array of wires and brackets being added as a solution later. Also remember that contents can experience vertical as well as horizontal accelerations in an earthquake.
Consider consequence of failure of individual contents	Damage associated with building contents can range from the inconvenient, for example, some broken glasses that can be reasonably quickly swept up, to significant, such as industrial storage racks falling over. These can affect an organisation's ability to function for days or weeks. There are also high value contents that require consideration, and in some cases, injuries and fatalities can also arise from falling items or blocked exit paths etc.
Provide guidance for future use	Many building contents are supplied and installed at the direction of the owner or tenant. It is important that those doing this work are aware of the requirements for LDSD. The owner should ideally provide a guidance document for the installation of contents such as appliances, shelving, furniture etc.

6.12.2.3 Examples of Good Low Damage Seismic Design Detailing

NZS 4104 (SNZ, 1994) and FEMA E-74 (FEMA, 2011) provide detailing solutions which can be used to restrain various types of contents. The BRANZ Design Guide *Seismic Design of High-Level Storage Racking Systems with Public Access* (BRANZ, 2006) provides specific advice for the design of high shelving and racking systems.

Other things that can also be considered include:

- Consider reducing accelerations and drifts by either moving the items to a lower floor, using base isolation of the content item or, where there are a lot of high value/important pieces of contents or equipment, consider base isolation of the whole building.
- Seek advice on additional structure to resist the dynamic load of mobile compact shelving systems (or consider not using this kind of shelving).
- Get advice on the restraint of large monitors hung from walls or ceilings as these can impose large loads on the walls and, when they move around during seismic shaking, they can damage adjacent ceilings and walls if there is insufficient clearance. Ensure there is a clearly designed load path, for example:
 - a) Connection of content item to mounting bracket/fixing.
 - b) Connection of mounting bracket/fixing to wall component (stud/nog).
 - c) Connection of the wall component (stud/nog) to the wall framing/connection to primary structure.
- Recommend that sit-to-stand desks are connected to other desks rather than freestanding and therefore vulnerable to tipping.
- Encourage the use of monitor arms rather than freestanding monitors.
- Design storage areas like kitchens and utility areas so that large, heavy or breakable items are located at low level.
- Provide upstand lips on the front of kitchen or utility area shelves to prevent items sliding off.

- Provide wire restraints on shelving in cupboards to prevent items falling off.
- Use clear nylon cables to restrain small items as this is less visible.
- Use non-slip finishes to prevent items sliding about where they could hit other components or parts of the building.
- Consider specifying earthquake-proof latches to cupboards, particularly seldom used high level cupboards.
- At locations with extensive benchtop or side-by-side floor-mounted equipment such as in laboratories, install a continuous strut on the wall or bench to facilitate the installation of removable restraints.
- If possible, connect tall contents with high centre of gravity to a weighted base to lower the centre of gravity and reduce/eliminate toppling during earthquakes.
- Additional strength can be provided to contents/objects by providing supportive mounts that both cradle and restrain the object as well as actually improving the display.

6.12.3 Prescriptive Design Method

In addition to the requirements of the NZ Building Code, building tenants and users could choose to undertake a risk-based procedure to identify high risk building contents that could be restrained to help mitigate damage during significant earthquakes. Such a procedure is outlined below.

Seismic design actions (accelerations and inter-storey drifts) required for the restraint of building contents should be provided by the project structural engineer in accordance with Section 6.2.1.

Step 1

Complete a risk assessment to determine the seismic load that is to be considered for the restraint of contents.

Table 6-24 LDSD risk assessment table for building contents

Risk of failure of item of content	Example	Applicable Limit State
Low Risk	Contents of shelves because these are relatively low cost and easily replaced.	SLS1
High Risk	Valuable contents/equipment required for operational continuity. Contents that could obstruct egress or continued operation or use of the building if they fell or overturned. High value contents, e.g. museum exhibit.	DCLS

Step 2

Ensure that a robust load path is provided from the item of content back to primary structure (refer to Section 6.12.2.3 for guidance).

Commentary

TS 1170.5 Section 8.1 provides guidance on when an item within a building is considered to represent a life safety hazard (i.e. would be classified as a P2/P3 part) and therefore is required to be restrained for ULS loading.

An example of a poor load path is where a robust bracket has been designed to connect an item of content to a wall. The bracket is then fixed into the wall cladding but not the framing behind or it is connected to wall framing but with an insufficiently strong fixing of the framing component to transfer the seismic load from the item of content to the structure.

Examples of supplier information that need to be considered for applicability include server cabinets that are seismically rated for a base fixing only for a specified load in the cabinet and a given seismic acceleration. A structural engineer would be required to determine the seismic demand and, if it is higher than the cabinet is rated for, additional seismic restraints may be required to restrain the top of the cabinet.

Another example are library bookshelves which are free-standing and seismically rated, but the actual demand and performance should be assessed at various levels of buildings.

6.12.4 Direct Design Method

The Direct Design Method may be more appropriate for contents that would generally demonstrate compliance with the NZ Building Code through an alternative solution, or which cannot meet the Prescriptive Design Method criteria. This could include the use of experimental testing or analytical study to demonstrate adequate LDS performance.

Commentary:

When it is not possible to restrain building contents, FEMA P-58 Appendix K (FEMA, 2018) can be used to calculate the sliding and overturning capacities of unrestrained building contents. Work by Yeow et al. (2018) provides estimates of dynamic coefficients of friction for common office furniture. These capacities should then be validated against the seismic design actions provided by the project structural engineer (refer Section 6.2.1).

6.13 Landscaping and External Services

6.13.1 Scope

This section sets out recommendations for LDSO of the following landscaping and external services components:

- Pavements, stairs, ramps, landscaping, retaining structures, external services and other facilities at the following locations:
 - i. The interface between the building and surrounding ground.
 - ii. Beneath the building.
- Within the site but beyond the building footprint and beyond the interface with the building.

When considering landscaping and external services, LDSO requirements are limited to maintaining access and function of the building. This section provides information to assist geotechnical, civil and building services engineers identify potential damage to landscaping and external services as a consequence of earthquakes; and to develop designs to mitigate damage to the extent needed to meet LDSO damage control and improved functionality performance goals.

Commentary:

This section is not intended to provide detailed guidance for general design or to replace existing standards and guidelines. Rather this section is intended to act as an overlay and provide designers with a methodology to achieve the LDSO outcome objectives and post event performance goals as detailed in Volume Two.

6.13.2 Detail and Interface Issues

Design of landscaping and external services, and any other facilities bearing on or in the ground, needs to consider the potential for ground damage as a consequence of earthquake shaking. This requires collaboration between the architect, landscape architect, and structural, geotechnical, civil and building services engineers.

The geotechnical engineer should clearly explain and document potential ground settlement and lateral deformations, such as that resulting from liquefaction and lateral spreading, as a consequence of ground shaking associated with each of the design limit states (SLS, DCLS, ULS). This includes potential differential settlements/lateral deformations between the building and surrounding and underlying ground. Other disciplines should allow for these deformations as they develop designs in consultation with the geotechnical engineer and others.

Examples of detail and interface issues to consider include:

- Interface between the building and surrounding or underlying ground.
- Detailing of building entries to ensure egress is maintained and to control damage.
- Detailing underground services entering or beneath the building.
- Detailing of pavements, stairs, ramps and landscaping abutting the building.
- Detailing of any structures which span between the building and the surrounding ground.
- Detailing of 'rattle spaces' associated with base isolated buildings and, in particular, how these will be remediated.
- Detailing of basement subsoil drainage and waterproofing.
- Detailing any facilities to manage sub-terranean hazards (e.g. gases).
- Consideration of the consequences of failure of the surrounding ground - e.g. a barrier to prevent rockfall from causing damage.

Within the site but beyond the building footprint and beyond the interface with the building:

- Detailing of pavements, retaining walls, landscaping, underground services and other facilities to allow for potential ground deformations.

6.13.3 Prescriptive Design Method

In addition to the requirements of the NZBC, recommendations in NZGS Modules 1 - 6 (NZGS/MBIE, 2021) should form the basis for LDSD of landscaping and external services. This recognises that ground damage as a result of earthquake shaking is a key consideration for these elements.

Collaboration across the design team is required to understand the implications of ground damage on these elements and the design features report should document the anticipated ground deformations and potential mitigations or residual risk accepted for these elements. Information is provided below on identifying and mitigating the effects of earthquake-induced ground damage for LDSD of landscaping and external services.

Commentary:

While NZBC B1/VM2 provides information for foundation design it is not considered to be appropriate for LDSD projects because it has limited application and does not provide a procedure to determine foundation deformations or address liquefaction-prone sites. The NZGS Modules 1 - 6 provide guidance for “Earthquake resistant design” and consideration of ground deformations. NZGS Module 3 is particularly relevant to LDSD of landscaping and external services.

6.13.3.1 Differential Displacement Between Building and Surrounding Ground

The geotechnical engineer should assess the potential for differential displacement at the interface between the building and surrounding ground to inform design of services, landscaping and other facilities connecting to, or abutting the building.

Potential damage to facilities at the interface with the building should be addressed and mitigated to enable LDSD damage control and improved functionality performance goals to be met. This includes consideration of *Operational State Category OS II - Partial Functionality* following a DCLS intensity earthquake (refer Section 6.1.7). The following outlines potential mechanisms of differential displacement at the building interface.

Ground Settlement

Earthquake shaking can result in compaction and settlement of loose granular soils. In the case of saturated loose granular soils this can be associated with liquefaction and post liquefaction consolidation. Loose granular soils above the water table can also be subject to “shake down”. NZGS Module 3 (NZGS/MBIE, 2021) provides guidance for estimating the potential for and magnitude of this settlement.

For buildings on deep foundations, ground settlement can appear as a sharp differential of the ground surface relative to the building. For buildings on shallow foundations any differential will be less marked because the building can settle with the ground. However, for buildings on shallow foundations there could be potential for the building to settle due to earthquake shaking relative to the surrounding ground. This should also be considered. NZGS Module 4 (NZGS/MBIE, 2021) provides guidance on building settlement, as discussed in Section 6.3.

Lateral Ground Displacement

Sites with a potential for liquefaction can also be subject to lateral displacement due to earthquake shaking in the form of cyclic displacement or lateral spread. NZGS Module 3 provides guidance for evaluating this potential.

Sloping sites, or sites adjoining slopes, can have a potential for lateral displacement as a consequence of earthquake shaking. Further details on assessing slope instability are provided in the NZGS *Slope Stability Geotechnical Guidance Series* (NZGS, 2025).

For buildings on laterally stiff deep foundations, lateral ground displacement can appear as a sharp differential at the interface between the building and ground. For buildings on shallow foundations this differential can be less marked because the building can move with the ground.

Constructed items which cross or abut the interface between building and surrounding ground, which could be damaged by the ground displacements described above, include:

- Pavements, stairs, ramps and landscaping.
- Vehicle entry/exit ramps.
- Pedestrian entry/exit landings or steps.
- Underground services.
- Basement subsoil drainage and waterproofing.
- Facilities to manage sub-terranean hazards (e.g. gases).
- Base isolation rattle space elements.

6.13.3.2 Ground Deformation Mitigation Measures at Building Interface

For LDSD, the design team should either be satisfied expected earthquake-induced differential deformations between the building and ground are small such that any resulting damage is acceptable or should include measures to mitigate damage to an acceptable level. Possible measures to mitigate damage resulting from differential displacements include:

- Selection of a foundation system with less potential for differential displacement. Shallow foundations may have benefits over deep foundations. Ground improvement beneath and beyond the building footprint may offer an opportunity to spread the differential over a greater distance.
- Pavements, landscaping, building entry:
 - i. Develop connection details which can tolerate the differential displacement including articulated joints and settlement slabs.
 - ii. Select pavement types and other facilities which are more tolerant to differential displacement. This could include a narrow strip of more tolerant surfaces adjoining the building, for example, as a strip of soft landscaping against the building.
- Services:
 - i. Avoid locating services in the ground beneath the building. Make the services part of the building rather than in the ground by using service channels or similar. Where possible, select an alternative route for the services which does not go beneath the building.
 - ii. Provide inspection chambers and flexible joints for services at the interface between building and ground.
 - iii. Minimise the number of entry and egress points for services around the building to allow any mitigation details or damage repair to be focussed.
- Base isolation 'rattle space':
 - i. Select the level of the base isolation plane to simplify detailing and control damage. Avoid passing isolation plane through tenanted spaces.
 - ii. Detail rattle space covers to:
 - Avoid damage (e.g. floor above isolation plane cantilevering over)

- Control damage (e.g. perimeter covers connected to the building and sliding over adjoining ground surface)
- Be easily repairable (e.g. replaceable covers).

6.13.3 Retaining walls

Retaining walls may be integral to, supporting or protecting, a building and may be providing access to services and/or be beyond the building interface. Figure 6-15 presents typical situations where retaining walls are used for building development. NZGS Module 6 (NZGS/MBIE, 2021) provides guidance on the earthquake resistant design of retaining walls in these situations.

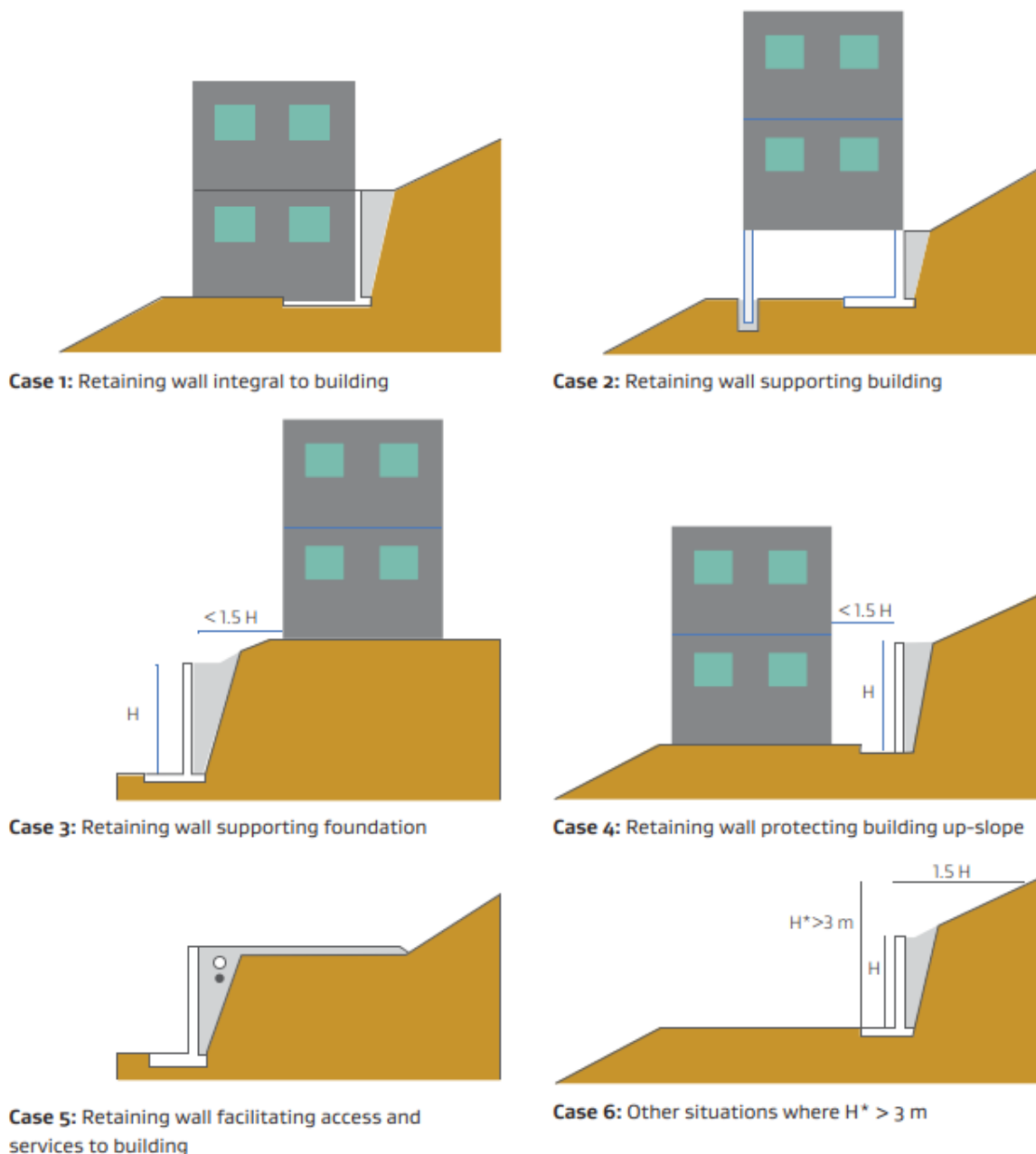


Figure 6-13 Typical situations where retaining walls are used for building development (NZGS/MBIE, 2021).

When possible, retaining wall performance should be decoupled from the building performance. This is because such an approach makes building and retaining wall performance more predictable and enhances LDS. Where retaining wall performance is linked to building performance, NZGS Module 6 provides guidance, and careful consideration of potential wall deformations and implications for building performance should be given.

The performance requirements provided in NZGS Module 6 for retaining structures during earthquakes focus on SLS and ULS (refer to Table 4.1 in NZGS Module 6). For retaining walls integral to and supporting buildings (Cases 1 to 3 in Figure 6-15) the performance requirements in NZGS Module 6 for SLS should also be adopted for DCLS.

For walls in close proximity to buildings, but beyond the building interface (Case 4 in Figure 6-15), seismic performance of the wall at SLS and DCLS should consider access and functionality of the building and should be agreed between the design team and the client.

When retaining walls are supporting services (Case 5 in Figure 6-15), specific consideration of the effect of expected wall/ground deformations on the services and related connections to the building should be made.

Further to the retaining wall performance requirements outlined in NZGS Module 6, the wall displacement factor for SLS and DCLS should be 1.0 for all LDSD cases. Consideration can be given to using lower displacement factors (typically 0.7) for walls that are not integral or supporting the building, or building service connections, however the implications this might have (i.e. including larger wall/ground deformations) on LDSD project objectives should be carefully considered.

6.13.3.4 Site beyond building interface

For LDSD of facilities within the site, but beyond the building interface, design teams should identify potential issues due to earthquake-induced differential deformations of the ground and consider potential measures to mitigate damage in accordance with the LDSD design objectives agreed with the client. Possible measures to mitigate damage as a result of these potential differential deformations are outlined in Table 6-25 below.

Facilities that are to be considered include: underground services, pavements, hard landscaping and other built features. As the primary cause of damage from ground deformations in an earthquake are due to liquefaction and lateral spread, refer to NZGS Module 3 (NZGS/MBIE, 2021) for further guidance on design of these mitigation measures.

Table 6-25: Damage to facilities beyond the building interface

Potential cause of damage	Possible type of damage	Possible mitigation measures
Liquefaction: lateral spread	Displacement, stretch and cracking.	Assess damage potential and acceptability of it. Locate critical facilities in areas less prone to lateral spread. Undertake ground improvement (refer to NZGS Module 5). Robustly tie facilities together. Include repairable isolation joints.
Liquefaction: sand boils	Subsurface damage; subgrade weakening, voids. Surface damage; ejecta, cracks, settlement.	Assess damage potential and acceptability of it. Locate critical facilities in areas less prone to liquefaction. Ground improvement, e.g. a confining cap across the site of geotextile and compacted fill (refer to NZGS Module 5). Found selected facilities on isolated raft foundations which can be relevelled by resin injection or similar techniques.
Liquefaction: buoyancy	Uplift of buried vessels.	Assess damage potential and acceptability of it. Locate critical facilities in areas less prone to liquefaction, this could include inside the building if appropriate. Ground improvement (refer to NZGS Module 5). Make vessel shallower. Pressure relief details. Provide tie downs.
Settlement: Liquefaction, lateral spread, shake down	Differential settlement.	Assess damage potential and acceptability of it. Locate critical facilities in areas less prone to settlement. Ground improvement (refer to NZGS Module 5). Repairable details. Details to tolerate settlement. Found selected facilities on isolated raft foundations which can be relevelled by resin injection or similar techniques.
Slope movement	Displacement, stretch and cracking.	Assess damage potential and acceptability of it. Locate critical facilities in areas less prone to slope movement. Undertake ground improvement (refer to NZGS Module 5). Robustly tie facilities together. Include repairable isolation joints. Found selected facilities on isolated raft foundations which can be relevelled by resin injection or similar techniques.

6.13.4 Direct Design Method

The Direct Design Method may be appropriate for landscaping and external services that do not meet the Prescriptive Design Method criteria detailed in Section 6.13.3. This could include the use of experimental testing or analytical study to demonstrate adequate LDSD performance of new or innovative systems.

7 Procurement, Construction and Post Construction

7.1 Procurement

While the focus of this document is on the design aspects of LDSD, it is also important to ensure the design is properly constructed, maintained and repaired. These aspects should be considered at the early stages of a project so they can be properly included in the consultants' scopes of services.

Most projects commence with consultants, with contractors being selected at a later stage. There are many paths to procuring a contractor for a project - often driven by specific requirements of the client.

LDSD projects are not necessarily any different to typical projects where some degree of Early Contractor Involvement (ECI) may provide useful input into the building design, although this will depend on the specifics of the project. For the case of LDSD projects this could include early advice on:

- Sources and availability of proprietary seismic technologies.
- Sources and availability of proposed materials, systems and components.
- Indicative costs for different component options.
- Buildability challenges of a project and how these can be improved.
- Input into the design of non-structural elements.
- Input into Building Information Modelling.
- Safety in Design considerations.

Along with main contractor input, the input of specialist subcontractors can also be very useful because they often have more intimate knowledge of the availability, suitability and costs of building components. ECI doesn't necessarily mean a project can't be competitively tendered in total, or for some - or all - sub-trades.

No matter which procurement approach is adopted, the selection of contractors or subcontractors should consider their track record and relevant experience. Good proven experience of LDSD projects is obviously very useful, but just as important are:

- A willingness to take on projects that may be a little different to the norm.
- Project team members with experience that can be applied to LDSD, e.g. a NSE Seismic Coordinator.
- A proven collaborative approach.
- Clearly demonstrated Quality Assurance procedures.
- Good references from clients and consultants on previous projects.

These aspects could be included in a weighted attributes selection process.

7.2 Tender and Construction Documents

To ensure LDS is properly allowed for in a contractor's tender, and then properly executed, it is important to make sure the requirements are made clear. At the very least the following information should be set out in the tender and construction documents:

- The fact the building has been designed as a LDS project, to what Category Level, and what this means for the project.
- Project specific seismic design actions (i.e. seismic loads and building inter-storey drifts) needed for the design and/or specification of secondary and non-structural components (refer Sections 5.3.3 and 6.2.1).
- Relevant sections of the Design Features Reports and the Building Movement Strategy Report (refer Section 3.2.2.3)
- Documentation that shows movement zones, identifying any sacrificial elements and how these are to be replaced (refer Section 3.2.2.3 Building Movement Strategy).
- Details of any project specific prototype or production testing when this is needed to validate the seismic performance of specific building components.
- Requirements for specific LDS related Quality Assurance processes.
- A requirement for a NSE Seismic Coordinator to be part of the contractor's team, as recommended by the Code of Practice for the Seismic Performance of Non-Structural Elements (BIP, 2025).
- Where the contractor or sub-contractor has design responsibility for a component, the documents should clearly set out how this component interacts with other elements and, in particular, who will take primary responsibility for critical interfaces.
- Requirements for Producer Statements.
- Requirements for as-built documentation (refer 7.4 below).

It is important this information is made clear to tenderers so that unexpected variation claims don't arise during construction.

7.3 Construction Monitoring

With well detailed and co-ordinated design documentation there should not generally be a need for a higher level of construction monitoring or observation than normal for LDSD projects. However, this will depend on a number of factors and consultants should ensure their scope of services include the necessary level of construction monitoring.

As recommended in Section 3.1.4.3, there should be a NSE Seismic Designer whose role will continue throughout the construction monitoring stage leading to the issue of a Producer Statement - Construction Review (PS4). Section 3 also includes a requirement that the NSE Seismic Designer is involved in an initial briefing at the commencement of construction to discuss the design strategy, the Building Movement Strategy (refer Section 3.2.2.3), design of the interfaces between trades/disciplines, and to discuss the general principles and coordination to be followed for any contractor designed NSEs. The purpose of this briefing is to smooth the handover of the NSE design and seismic detailing from the detailed design to the construction phase.

Several aspects of the building are likely to have a contractor designed components. These can include:

- Precast concrete floor systems.
- Proprietary seismic devices such as proprietary seismic isolators and dampers.
- Moat covers for base isolated buildings.
- Facades including seismic movement joints.
- Building services plant and components, and the support and restraint of these.
- Lifts.
- Ceilings.
- Active and passive fire protection systems etc.

It is important to ensure these components are coordinated, designed, documented, reviewed by consultants, and built in accordance with the seismic requirements of the project. With the latter including additional LDSD requirements where these are applicable.

Other tasks relevant to LDSD to be mindful of when carrying out construction monitoring include the following:

- Carry out a gap analysis to ensure that all relevant aspects of LDSD are being reviewed and monitored by the appropriate consultant.
- Review the contractor's and sub-contractors' QA processes to ensure they include the necessary LDSD requirements. This is one area where performance expectations should be high.
- Ensure items are viewed before they are closed in or difficult to inspect. Similarly view an installation early to iron out any issues before the installation is repeated multiple times.
- Review the construction phase of the BIM Execution Plan (where applicable) and ensure modelling and shop drawings are consistent with the plan.
- Review shop drawings for services to ensure these show all seismic restraint, flexibility and clearance requirements.
- Review shop drawings for proprietary façade systems to ensure these show all seismic movement and clearance requirements.
- Review technical submissions to ensure these include information on seismic restraint, flexibility and clearances.

- Obtain PS3s from relevant contractors and subcontractors as evidence the work has been carried out in accordance with the design.
- Obtain PS1s and PS4s where relevant for design by the contractor's specialist suppliers.

Even for smaller projects where a number of the tasks described above will be simplified, it is still worth reminding all involved in site work of the LDSD aspects of the project, and so a final but important suggestion is to make LDSD an item on the agenda of Site Meetings.

Commentary:

For any project, construction monitoring verification of the ground conditions relative to design assumptions is important, but particularly so for LDSD projects. In LDSD expectations for foundation performance may be higher than they are for a code minimum design and therefore a higher level of confidence in ground conditions relative to design assumptions may be necessary. Construction inspections and testing should be planned and implemented accordingly.

7.4 As-builts, Maintenance and Building Alterations

To ensure a building continues to perform in accordance with the LDSD performance criteria it is important that it is well maintained and that any building alterations are consistent with LDSD requirements. Building alterations can range from the replacement of large plant items to the cycle of tenant fitouts, to major building upgrades. In all cases this work must be done in a way that does not impact adversely on the LDSD aspects of the building.

In the first instance, it is important to encourage clients to follow good facilities management practices. This would include good as-built records and if there is a BIM model, this could be developed to an asset information model (AIM), which the client's facilities manager would then maintain for the life of the building. Such a model could have built into it the necessary clearance, movement and restraint requirements for building components.

To be considered a LDSD building, consultants must ensure that information on these design aspects of the building is included in the Operating and Maintenance manuals or equivalent digital information. This could be a simple list of all the LDSD aspects of the building, and the relevant design parameters, so that these can be used by service personnel, designers and contractors in the future. This should include documentation on movement zones and information on specific LDSD details and how these should be maintained, inspected and reinstated after an event. This would apply particularly to sacrificial elements, but also some energy dissipating and post-tensioned systems which may require regular inspections and maintenance.

Another document that consultants could prepare LDSD-related material for, if required, is a Tenant Design Guide that a building owner can give to a tenant design team when a new fitout is being designed. This should set out in detail the requirements for tenant fitout work, for example, to ensure the fitout partition walls are constructed in accordance with the Building Movement Strategy, or that contents are seismically restrained in critical areas.

Commentary:

After a seismic event it may be necessary to review the condition of a building and foundations to justify their continued use with or without repair. To complete this review, it is essential to have reliable 'as-built' records from the investigation, design and construction stages, including relating to ground conditions.

7.5 Monitoring and Post-event Resilience

To complete the resilience and low damage ‘lifecycle’, it is important building owners and tenants have agreed procedures for what happens immediately after a significant earthquake. The consultant team is ideally placed to provide assistance with developing these procedures. It will generally make sense for the structural engineer to review the building’s condition, but other specialist consultants or tradespeople may also be required depending on the severity and nature of the event. For example, a façade engineer may be required to assess the performance of the façade, or a drainage contractor may be required to check for broken pipes.

Some of the agreed LDS performance goals will assume that repairs take place quite promptly, so it is important for the building owner to have arrangements in place to get consultants and tradespeople to the building as soon as possible. This is part of normal facilities management and, as well as a record of important contact arrangement, there should be an agreed inspection plan, fulsome building information etc.

The installation of seismic sensors can assist the speed and effectiveness of post-earthquake assessment of buildings. This kind of instrumentation offers a range of information of value to a building owner and could include one or more of the following:

- Confirm if the actual building performance aligned with the design assumptions.
- Provide an early indication of seismically induced building damage and potential life safety concerns.
- Information on the short and long-term structural health of buildings.
- Improve industry understanding of the behaviour and potential for damage of buildings under the dynamic loads of earthquakes.
- As a result of this improved understanding, codes, modelling assumptions, design and construction practices can be modified so that future earthquake damage is minimised.
- Response triggering at an agreed threshold value. For example, shutting down certain building systems (through an electronic Building Management System) if peak floor accelerations exceed a specified value.
- Possibly quicker resolution of legal disputes by identifying factors contributing to damage.
- Early warning of an event.
- Provide accurate data for a parametric insurance claim.

There is also the possibility the building could become part of Geonet’s Building Instrumentation Programme (NHC & Earth Sciences New Zealand, 2025) where the information is gathered for a wider good.

At the same time that the building is being assessed, tenants should be implementing their own Business Continuity Plans (BCPs). The expectation is that, with a well-designed LDS building, building reoccupation times will be minimised. Experience also shows that a speedy implementation of a BCP is also an important part of reducing reoccupation times and making staff feel attended to.

There are other sources of information on this topic and the BCP will be very bespoke to the tenant and the building. However, there may be some basic things that tenants could agree to do to reduce damage to buildings and their fitout, especially if the landlord or building manager is remote. For example, does the tenant know how to turn off the water supply in the event of broken pipes? Or the gas supply? This would also apply to body corporates who are responsible for apartment buildings.

7.6 Post-event Recovery

7.6.1 The Decision to Reoccupy

The decision to reoccupy a building after an earthquake will be mainly influenced by health and safety considerations. Local authorities will rely on building owners, boards, body corporates and the like to have processes in place to make an informed decision about reoccupation. The desire to be operational in the case of a business, or to reoccupy in the case of an apartment, must be balanced against risks associated with a damaged building. These risks range from risks affecting life safety to ones that simply reduce amenity to occupants.

The strategy for carrying out earthquake damage repairs should correlate with those risks, with a greater emphasis on repairing damaged items that present the greatest hazard. Occupants will also have a range of tolerances for how long things take to repair. In the case of apartment dwellers, the stress and inconvenience of relocation means that those tolerances could be quite high. On the other hand, remote working typically provides more options for office workplaces and so tolerance levels in this situation may be lower.

The Building Code provides a useful framework to consider which items are life safety/injury related vs amenity related. Performance-critical damage to the building structure should be repaired prior to reoccupation, where performance-critical damage is defined as damage to the building structure that initiates lateral strength loss. FEMA P-2335 (FEMA, 2025) provides a post-earthquake assessment process which could be used to identify and repair earthquake damage to the building structure.

Table 7-1 below is a simplified example of how a building owner or body corporate could prepare a plan for building reoccupation. A document like this would follow on from the LDSB Brief, be regularly updated, and completed in detail towards the end of the construction stage. The building design consultants, contractors and sub-contractors etc. could all make suggestions for how hazards are managed in the short term after an event. Insurers could also be involved. Ideally it would become part of an organisation's business continuity planning.

Using this format can assist decision-makers in assessing how and when to reoccupy. The Building Code framework may also assist with related discussions with the local authority.

If the earthquake causes considerable damage a Civil Emergency may be declared, in which case a number of statutory processes will be implemented which may impact such a plan (NEMA, 2024).

Table 7-1 Example Building Reoccupation Strategy Report

Building Code Clause ¹	Highest Impact of hazard ²	Inspection by and of what ³	Likely repair requirements for occupation ⁴	Interim 'alternative solution' ⁵
B1 Structure	Death/Injury	Structural engineer to inspect primary structure, e.g. ductile yielding plates - note inspection hatches giving access to these on ground floor.	Performance-critical structural damage must be repaired before reoccupation E.g. Spare yielding plates are stored in the L7 plantroom	Not applicable
	Death/Injury	Structural engineer to inspect ground conditions and request input from geotechnical engineer if required.		
	Injury	Structural engineer /Architect to inspect non-structural elements, e.g. ceilings to be inspected for any loose or dangerous items.	Loose/broken ceiling tiles to be removed or reinstated. Areas of damaged plasterboard ceiling to be screwed back in place. Full repairs to occur when convenient to occupier.	
C1-C6 Fire	Death/Injury/illness	Fire engineer to inspect		
C2 Prevention of Fire	Death/Injury/illness	Fire engineer	Flues must be repaired before occupation or	Device not used until flue repaired.
C3 Fire Spread	Death/Injury/illness	Fire engineer to inspect all passive fire items, e.g. fire walls around stairs using ceiling access hatches provided on level 7.	E.g. Cracks in Fireline plasterboard greater than XX mm to be repaired. Any torn intumescent sealant to be re-sealed.	Manual security measures.
C4 Egress	Death/Injury/illness	Fire engineer to check all doors on escape routes open. Where they are fire doors that they also close and latch closed.	Non fire rated doors may be able to be left open or removed if they can't open/close. Fire doors may need to be rehung, or the door leaf edges planed so there is no binding.	Manual security measures may be satisfactory subject to the Fire Engineers approval.
C5 Firefighting	Death/Injury/illness	Fire engineer to inspect fire fighter vehicle and personnel access to the building. Fire Protection engineer to check operation of sprinkler, fire alarm and hydrant systems.		
C6 Structural Stability	Death/Injury/illness	Structural engineer to check structural performance of the structure that is supporting fire rated elements. Fire engineer to inspect the fire rating applied to the structural support system.		
D1 Access	Injury	Architect to inspect building access, handrails etc.		
	Injury	Structural engineer to inspect stairs.		
D2 Lifts etc	Injury/amenity	Lift technician to inspect lifts.		
E1 Surface Water	Injury/illness			

Building Code Clause ¹	Highest Impact of hazard ²	Inspection by and of what ³	Likely repair requirements for occupation ⁴	Interim 'alternative solution' ⁵
E2 External Moisture		Architect/Façade engineer to inspect building envelope		Adhesive tape or plastic sheeting, tarpaulins etc.
E3 Internal Moisture	Injury/illness	Services engineer or maintenance contractor to inspect mechanical ventilation system.		
	Illness/infection	Architect to inspect impervious surfaces including waterproof membranes.	Temporary sealant/adhesive tapes to prevent leakage.	
G1 Personal Hygiene	Illness/infection	Architect/Services engineer to inspect sanitary facilities. Access to review sewer pipes is in room XX. Check reduced numbers of facilities against reduced building occupancy.	Temporary repairs to drainage pipework.	There is a range of options here. Is there an emergency public facility within 400m walking distance? Could chemical toilets be stored in a basement for distribution to residents etc.?

Notes:

1. Not all sections of the Building Code have been included in this illustrative example.
2. The impact will vary depending on the building, contents etc. The Building Code does not refer to loss of life but, for the purposes of considering health and safety, this has been added as a possible impact.
3. This should identify who is doing the inspection, what they are inspecting, and where there are specific provisions for ease of inspection. The purpose of the inspection is primarily to assess safety for re-occupation and suggest the likely repairs to enable the reoccupation.
4. This will prompt thinking about and planning how repairs are carried out and if spare parts etc. should be kept.
5. This list acknowledges that repairs may take longer than a few days and so gives some thought to alternative ways to allow occupation

In its final form this kind of table would be more detailed, and the contents would be customised to the client brief, the building, its components, and the contents.

7.6.2 Building Consent Considerations

A LDS building should be able to be repaired on a like-for-like basis and so would generally not require a new building consent for this scope of work. Such repairs are permitted by Schedule 1, Part 1, of the Building Act, with certain provisions, under the criteria of 'General repair, maintenance, and replacement'.

If a new building consent was required, for example because there was the 'substantial replacement of a component or assembly contributing to the building's fire-safety properties' [Schedule 1, Part 1, 1 (3)(b)], then a consent may have to be obtained. This doesn't necessarily have to be an onerous process because the territorial authority has the power to grant an exemption for a building consent or to issue a Certificate of Acceptance once the work is completed. This would avoid the time delay associated with processing the consent application but does require good records to be kept of the work carried out.

7.6.3 Building Warrants of Fitness

An annual building warrant of fitness (BWOFF) demonstrates the specified systems in a building are operating adequately. After a significant event it may be prudent for a building owner to instruct their incumbent building services (mechanical, electrical, fire, P&D, lift, BMS, etc.) maintenance contractors to inspect, restart systems and implement damage repairs as soon as possible after an earthquake, followed by BWOFF type tests to check functionality, particularly of normally dormant safety systems. The building owner may then involve a services engineer to provide specialist advice where specific circumstances warrant.

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Appendix A – Definitions and Acronyms

Acceleration Sensitive Components	Those building components most adversely affected by a sudden sideways or upwards shift of the building in an earthquake. An example is a piece of mechanical plant that during an earthquake, by virtue of the momentum of its own weight, may cause damage to itself or items to which it is attached
Accessible	Features to permit use by people with disabilities.
Active component	A component which has parts that rotate, move mechanically, or are energised during operation
APoE	Annual probability of exceedance. It is the likelihood that a specific event at a given location will be exceeded in any given year.
Base isolation	This building construction method places seismic isolators (base isolators) between the foundation and the bottom of the building superstructure. During an earthquake, the isolators effectively decouple the superstructure from the foundation, avoiding damage to the building.
Base isolation plane	The boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure, which moves rigidly with the ground.
Building Information Modelling (BIM)	A process of creating and managing detailed digital representations of a building project throughout its entire life cycle
BIM Execution Plan (BEP)	A document that outlines how a BIM project will be executed, managed, and delivered
Building Act	Building Act 2004
Building Code	Schedule 1 to the Building Regulations 1992
Building Movement Strategy Report	A report that details the outcomes of the Building Movement Strategy.
Building Re-occupation Strategy Report	A building-specific report that sets out the requirements for inspections, repairs, and any interim solutions to allow a building to be re-occupied following a significant seismic event.
Collapse Avoidance Limit State (CALS)	Building performance limit state at which collapse of the structure is to be prevented with reasonable reliability in accordance with the requirements of the NZBC
Containment	Maintaining the containment of liquid, steam, gas or hazardous contents within tanks/pressure vessels and reticulated services by avoiding rupture of those components (as opposed to containment of spilled hazardous contents from ruptured tanks within bunded areas).
Damage Control Limit State (DCLS)	A non-mandatory LSD limit state at which damage to the building is controlled so there is a low probability of damage leading to significant economic loss and a reduced probability of loss of functionality.
Discrimination	Discrimination, also known as selectivity, is a process that coordinates protective devices in an electrical system to limit the impact of a fault. The goal is to ensure that the affected circuit is disconnected while leaving other circuits powered.
Drift Sensitive Components	Those components that are most adversely affected by the change of shape of a building when it moves out of plumb in an earthquake. An example is a partition wall which is built plumb and level and uses large sheet materials that suffer damage when pushed out of square.

Design Features Report (DFR)	A document that details the design criteria adopted by a design consultant for a project, including the assumptions, materials, and calculation procedures. The document also records key decisions and outcomes, and how the design meets the LDSB Brief. Some consultants refer to these as Design Reports.
Direct Assessment Method	An explicit assessment method that enables design teams to demonstrate compliance with LDSB performance goals using recognised performance assessment procedures such as the FEMA P58 (FEMA, 2018).
Direct Design Method	A verification pathway for LDSB for building systems or components that don't meet the requirements of Prescriptive Design Method. This method uses recognised analytical methods, or experimental testing in accordance with recognised standard testing procedures, to validate the seismic performance.
Early Contractor Involvement (ECI)	A construction contracting method that involves a contractor in the design phase of a project.
Embodied carbon	Greenhouse gas emissions associated with materials and construction processes throughout the whole lifecycle of an asset, including maintenance and replacement. This could be a whole building, or only a building's structure, known as 'structural embodied carbon'.
Enhanced Design	Designing the critical services within the building with the lower performance requirements for the seismic accelerations and displacements imposed by this building when subjected to the seismic demands used for the design of the building with the higher performance requirements (refer Section 6.1.5)
Expected Annual Loss (EAL)	Represents the average economic loss in dollars resulting from seismic hazards each year.
HCAI	California Department of Health Care Access and Information
Importance Level (IL)	Building Importance Level as defined in NZS 1170.0 (SANZ, 2011).
In-plane deformation	Deformation in the plane of a surface (e.g. change of a window shape from a rectangle to a parallelogram).
Inherently rugged	Building services components that have been shown by past earthquake experience to be linearly elastic and therefore inherently rugged under the relevant design earthquake actions (this does not apply to equipment anchorage, which should be subject to specific design).
Inter-storey deflection	The horizontal displacement of one story of a building relative to the story below, typically caused by lateral forces like wind or earthquakes.
Inter-storey drift	See inter-storey deflection.
Isolation system	The collection of structural elements that includes all individual isolators, all structural elements that transfer force between elements of the isolation system, and all connections to other structural elements. The isolation system also includes the wind restraint system, energy dissipation devices, and/or the displacement restraint system if such systems and devices are used to meet the design requirements.
Isolation plane	The boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure, which moves rigidly with the ground. Also referred to as the isolation interface.
Lateral strength loss (LSL)	Used to define the deformation capacity of structural components at which the lateral resistance starts to degrade.
Levels of Development (LOD)	A standardised system used in Building Information Modelling (BIM) to define the level of detail and accuracy of a model element at different stages of a project

Life Cycle Assessment (LCA)	A method used to quantify the greenhouse gas emissions and other environmental impacts of assets or products over their whole lifecycle. An LCA approach is used to calculate embodied carbon (but will also often include calculations of operational carbon).
Lift seismic retaining device	A specialised mechanism installed on a lift car designed to prevent it from moving excessively or becoming dislodged from its guide rails during an earthquake, essentially holding the elevator car securely in place during seismic activity by providing additional contact points with the guide rails.
Linear components	Building services components such as Pipes, ducts, cables and cable support systems.
LDSD Brief	A live document that captures the client's requirements for the LDSD aspects of the project. The nominated LDSD Lead for the project will update this as the project proceeds. The consultants Design and Design Features Reports will make reference to the LDSD Brief.
Low Damage Seismic Design	A building design philosophy that achieves better than New Zealand Building Code minimum requirements. A key goal is to deliver buildings that are less likely to be damaged and thereby limit disruption and losses in future earthquakes.
LDSD Category Levels	Used to describe categories of building which meet specific Low Damage Seismic Design requirements as detailed in the Low Damage Seismic Design: Performance Framework (Volume Two).
LDSD Category Level 1	LDSD Category Level 1 buildings incorporate some design features that help to minimise damage in addition to complying with minimum seismic design standards. LDSD Category Level 1 building are designed with lesser performance standards than LDSD Category Level 2 and 3 buildings.
LDSD Category Level 2	LDSD Category Level 2 buildings are designed to have a high level of seismic performance including specific damage control performance goals.
LDSD Category Level 3	LDSD Category Level 3 buildings are designed to have a very high level of seismic performance including specific damage control performance goals.
Maximum Considered Earthquake (MCE)	The maximum level of earthquake shaking that needs to be considered in a building design.
New Building Standard (NBS)	An index used to characterise the expected seismic response of an existing building to earthquake shaking. It identifies buildings that represent a higher seismic risk than a similar new building, built to the minimum life safety requirements of the Building Code (or New Building Standard)
NLA	Net lettable area.
Non-linear behaviour	When a building component is loaded beyond its elastic limit and cracks or yields as a result.
Non-structural Element (NSE)	All parts of a building that are essential to its functioning, but are not part there to hold the building up, such as ceilings, partitions, HVAC systems, electrical systems, plumbing and drainage etc.
New Zealand Building Code (NZBC)	See Building Code
Out-of-plane deformation	Deformation perpendicular to the plane of a surface (e.g. bowing of a window).
Operational Continuity	The ability of an organization to keep its core functions running despite disruption (i.e. earthquake).

Operational Integrity	Able to function in operational state at the applicable Limit State, or for the case of P5 components, able to be returned to an operational state within an acceptably short time frame.
Parametric insurance	Parametric Insurance is an insurance product that offers a pre-agreed payout based upon a trigger event. Trigger events can include an earthquake of a certain magnitude. The main benefit of parametric insurance policies for buildings over traditional asset insurance is that payouts are made much more quickly, i.e. as soon as the trigger event has been confirmed.
Peak Floor Acceleration (PFA)	The highest acceleration at a specific floor of a building.
Performance critical damage	Damage to the primary structure that initiates lateral strength loss.
Prescriptive design method	A verification pathway for LDS for building systems or components which meet certain regularity restrictions or building systems or components where expected performance is reasonably reliable, and the design method is covered by the Building Code generally through an acceptable solution or verification method, or by guidance issued under Section 175 of the Building Act.
Primary Structure	The use of the term Primary Structure in this document refers to the structure required for the vertical and lateral support of a building's floors and enclosure. It includes the floor structure, columns, walls and roof framing such as posts, rafters and bracing which provide support for the gravity loads and loads on the building from wind and earthquake. It excludes the enclosure itself
Project brief	Description of the key elements of the project including project goals, project drivers and performance requirements. It is used to communicate project requirements to the Design and Construction Team.
Proprietary equipment	A product marketed under, and protected by, a registered trade name. Recommended: Indicates practices that are advised for higher levels of seismic performance. Seismic restraint frames:
Rattle space	The gap around a base-isolated building that allows the building to move independently during an earthquake.
Safety-critical structural damage	Structural damage that that must be repaired to enable occupants to return safely to the building.
Secondary Structure	Structural components that are not part of the gravity and lateral load structural system of the building. For example, secondary columns provided to support façade components that are connected to, and supported by, the primary beams of the main structure.
Seismic Restraint	A structural assemblage of elements designed and installed to transfer earthquake-induced forces from a component to the supporting structure.
Serviceability Limit State 1 (SLS1)	Refer to TS 1170.5.
Serviceability Limit State 2 (SLS2)	Refer to TS 1170.5.
Seismic carbon	Greenhouse gas emissions associated with building repairs or demolition resulting from earthquake damage. For the case of demolition, includes emissions associated with construction of a replacement building.
Seismic hazard	The intensity of ground shaking hazard associated with potential earthquakes in a particular area.
Seismic isolation	See base isolation

Seismic isolators	A horizontally flexible and vertically stiff structural element of the isolation system that permits large lateral deformations under seismic shaking. An isolator is permitted to be used either as part of, or in addition to, the weight-supporting system of the structure.
Seismic separation	Separation (or gap) between adjacent structural elements to allow for differential seismic movement (building sway) between the structures.
Singly Reinforced Concrete Wall	A reinforced concrete wall with one layer of vertical reinforcement, usually in the centre of the wall.
Should	A requirement that must be adopted in order for a building to achieve the performance of this advice.
Special study	Refer to TS 1170.5.
Specification	A written document that describes in detail the scope of work, materials to be used, methods of installation, and quality of workmanship for a parcel of work to be undertaken under contract.
Step change	A sudden adverse change in behaviour of a foundation or structure with increasing level of shaking, load or displacement, e.g. triggering of liquefaction, or triggering of a landslide, or brittle failure of a structural element
Structural integrity	Component should not collapse or lose support. Where component contains hazardous contents, should not rupture or otherwise lose hazardous contents.
Substructure	Part of the structure located under the isolation plane including the foundation.
Superstructure	Part of the structure located above the isolation plane.
Tenant Design Guide	A document that a building owner, landlord, body corporate etc provides to tenants that sets out the design and construction requirements for the construction of an office fitout or apartment alteration etc. This may include requirements for the storage of contents and other matters related to LDSD.
Total loss	When an item (i.e. building) is damaged beyond repair or the cost to repair it is more than the items value.
Ultimate Limit State (ULS)	Refer to TS 1170.5.
Upfront carbon	Greenhouse gas emissions associated with the materials and construction processes up to the point of practical completion of a building.

Appendix B – Notation

Unless stated otherwise, this guideline uses the following notations:

A_g	Gross area of the section (mm ²) (see 6.4.4.3)
b	Wall thickness (mm) (see 6.4.4.3)
b_f	Width of beam flange (mm) (see 6.4.4.3)
c	Wall compression zone depth (mm) (see 6.4.4.3)
d	Depth of the beam (mm) (see 6.4.4.3)
$C(T)$	Elastic site spectrum for horizontal loading determined in accordance with TZS 1170.5 (see 4.2.1)
$C_i(T_p)$	Spectral shape coefficient in accordance with TS 1170.5 (see 5.3.3)
C_{ph}	Part horizontal response factor in accordance with TS 1170.5 (see 5.3.3)
h	Height of beam web (mm) (see 6.4.4.3)
h_b	Height of the beam (m) (see 6.4.3.1 and 6.4.4.3)
e	Clear length of eccentric braced frame active link (m) (see 6.4.3.2)
E	Modulus of elasticity (MPa) (see 6.4.4.3)
E_d	ULS design action effect (see 6.8.3)
f'_c	Specified compressive strength of concrete (MPa) (see 6.4.3.1)
f'_{cE}	Expected compressive strength of concrete (MPa) (see 6.4.4.3)
f'_t	Average tensile strength of concrete (MPa) (see 6.4.3.1)
F_u	Design tensile strength (MPa) (see 6.4.4.2)
f_y	Design yield strength (MPa) (see 6.4.4.2)
$f_{y,E}$	Expected yield strength (MPa) (see 6.4.4.3)
G	Permanent (self-weight or 'dead') action (KN) (see 6.4.3.4 and 6.8.3)
g	Gravitational acceleration (9,810 mm/s ²) (see 4.2.1)
K_ξ	Spectral damping reduction factor (see 4.3.2)
L_{beam}	Centre to centre spans of the beams (m) (see 6.4.3)
L'_{beam}	Clear spans of the beams (m) (see 6.4.3)
L_{col}	Centre to centre spans of the columns (m) (see 6.4.3)
L'_{col}	Clear spans of the columns (m) (see 6.4.3)

L_n	Clear span of the coupling beam (m) (see 6.4.3.1)
l_w	Wall length (mm) (see 6.4.4.3)
$M_{n,beam}$	Sums of the nominal flexural strength of the beams at the faces of the beam column joint zones in the level being considered (kNm) (see 6.4.3)
$M_{n,col}$	Sums of the nominal flexural strength of the columns at the faces of the beam column joint zones in the level being considered (kNm) (see 6.4.3)
$M_{n,cr}$	Cracking moment of the wall (kNm) (see 6.4.3.1)
$M_{n,wall}$	Nominal flexural strength of the wall (kNm) (see 6.4.3.1)
M_{sp}	Nominal plastic capacity (kNm) (see 6.4.3.2)
N^*	Axial load (kN) (see 6.4.4.3)
PFA	Peak floor acceleration (g) (see 6.4.3)
Q	Imposed or 'live' action (due to occupancy or use) (kN) (see 6.8.3.4)
Q_{bldg}	Imposed action due to building occupancy (kN) (see 6.8.3)
Q_{lift}	Supplied lift loads including dynamic or impact factors (kN) (see 6.8.3)
$S_d(T)$	5% damped elastic site displacement spectra for horizontal loading (mm) (see 4.2.1)
S_p	Structural performance factor (see 4.3.2 and 6.4.3)
t_f	Thickness of beam flange (mm) (see 6.4.4.3)
t_w	Thickness of beam web (mm) (see 6.4.4.3)
V_w	Nominal shear capacity (kN) (see 6.4.3.2)
T	Period (seconds) (see 4.2.1)
$T_{p,long}$	Reference translational period of vibration in accordance with TJS 1170.5 (seconds) (see 4.5)
α	Spectral damping parameter (see 4.3.2)
β	Dispersion associated with structural components (see 6.4.4.3)
$\Delta_d(T)$	Design displacement spectrum horizontal loading (mm) (see 4.3.2)
Δ_y	Axial deformation at the expected brace strength (mm) (see 6.4.4.3)
ε_{su}	Axial strain at the tensile strength of steel reinforcement (see 6.4.4.3)
ξ_{eq}	System equivalent viscous damping (see 4.3.2)
θ_y	Yield rotation in accordance with AISC 342-22 (rad) (see 6.4.4.3)
μ	Structural ductility factor

μ_p	Ductility of a part or component
ϕ_y	Yield curvature (see 6.4.3.1)
ψ_E	Earthquake imposed action factor in accordance with TS 1170.5 (see 6.4.3.5)

Appendix C – Direct Performance Assessment

The Low Damage Seismic Design (LDSD) performance framework (refer Volume Two) for LDSD Category Level 2 and 3 Buildings includes performance goals to limit the expected annual loss (EAL) and to limit the probability of exceeding 5% replacement cost in an earthquake over a 10-year period. These targets are referring to direct losses; the costs of repairing damage to the building due to earthquake shaking (as opposed to indirect losses which could be affected by factors outside the control of the design team).

Three verification pathways are available to demonstrate a building meets the outcome objectives and post event performance goals for LDSD Category Level 2 and 3 Buildings:

1. Prescriptive Design Method;
2. Direct Design Method; and
3. Direct Assessment Method

Further information on the application of Direct Assessment Method and the benchmarking studies completed for the Prescriptive Design Method are provided below.

C.1 Direct Assessment Method

The Direct Assessment Method enables design teams to demonstrate LDSD compliance via application of recognised performance assessment procedures and guidelines, such as the FEMA P58 (FEMA, 2018) approach.

There are many factors that will affect the repair cost of a building. Figure C-1 provides an overview of the detailed loss-assessment framework described in FEMA P-58. The framework proposes that performance be evaluated through a four-stage analysis process: (i) hazard analysis to identify the probability or rate of different levels of ground shaking intensity being exceeded at a site, (ii) structural analysis to estimate the likely response of the building for a given level of ground shaking intensity, quantified in terms of engineering demand parameters (EDP) such as peak floor acceleration or peak storey drift, (iii) damage analysis to identify the likely damage to the various components in the building for a certain value of EDP, and (iv) loss analysis from which the repair costs (or other consequences relevant for decision making such as injuries or downtime) are quantified considering the expected damage states for the various components in the building. By integrating the results of these four analyses phases over a relevant range of ground shaking intensity levels, one can identify loss measures for the building such as the EAL.

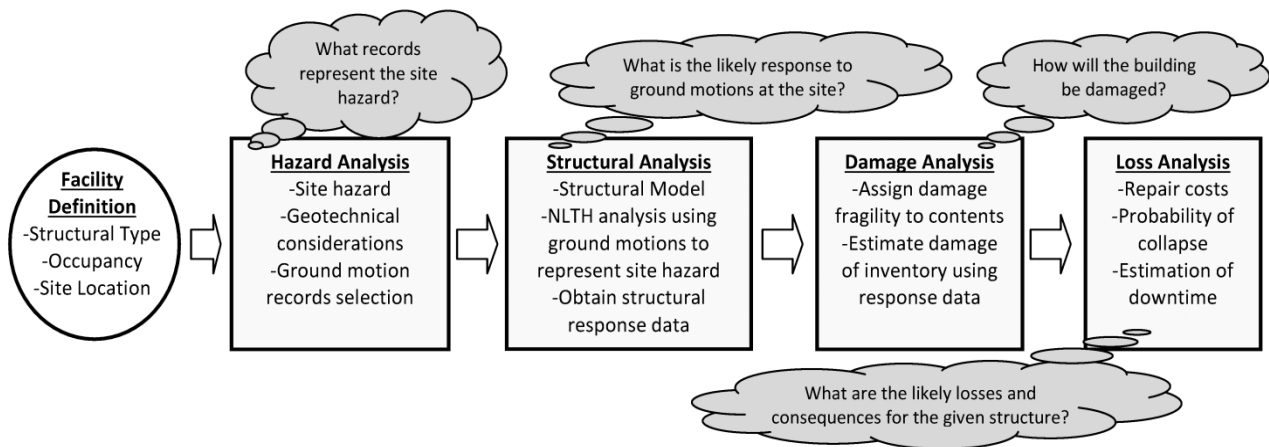


Figure C-1 Overview of the four stages of the FEMA P-58 loss assessment framework (Calvi, Sullivan, & Welch, 2014).

The quality of the results obtained via a Direct Assessment Method will be affected by the quality of the inputs provided at each stage of the loss assessment framework. Fragility functions, required for damage analysis, and loss functions, required to quantify the costs associated with repair at a given damage state, should be adopted considering New Zealand construction practice. Currently, many manufacturers do not possess information on the fragility of their building components and research continues to provide new insight into both fragility and loss functions.

Quantity surveyors from building contractors associated with the Christchurch rebuild, following the 2010 - 2011 Canterbury earthquake sequence, were consulted in the development of repair cost (loss) functions reported in Fox et al. (2024). In addition, a number of studies have been conducted into the fragility of different building components in New Zealand. For a list of functions and references for loss assessment, available at the time of writing of this document, refer to Dong et al. (2024) and Sullivan et al. (2025).

It is expected the number and quality of such functions will increase in the coming years. For example, at the time of writing this document it is understood new fragility functions for lifts will soon be available via the ATC-138 project. When the Direct Assessment Method is adopted design teams should ensure the fragility and loss function used are current and relevant.

C.2 Prescriptive Design Method

The Prescriptive Design Method has been developed so that if a building is designed according to the LDSD design criteria specified in Section 6, it should generally satisfy the target LDSD performance goals in terms of damage and losses.

C.2.1 How will the LDSD Design Criteria Limit Damage and Repair Costs?

The LDSD design criteria limit the likely damage to the structural elements by setting a low structural ductility factor, m , for the damage-control limit state (DCLS). However, in modern buildings non-structural elements (NSEs) tend to be considered the most vulnerable building components, often requiring repair after only low levels of shaking intensity (Filiatrault & Sullivan, 2014). For modern commercial buildings, the types of NSEs that typically suffer earthquake damage and require repairs are:

- Suspended ceiling systems.
- Suspended pipes and mechanical and electrical equipment (when inadequately braced or when inadequate clearances are provided to other elements).
- Lifts (including the rails, cab, motor and cables).
- Non-loadbearing plasterboard partition walls.
- Certain types of cladding systems (dependent on the connection detailing adopted).

The first two types of components listed above are commonly referred to as “acceleration-sensitive” non-structural elements, recognising that the acceleration of the supports to the suspended elements will cause vibration of the components and this in turn may lead to excessive displacements and damage. With good detailing and appropriate design, the vulnerability of these types of non-structural elements can be greatly reduced. Thus, by designing restraints for acceleration-sensitive components to remain undamaged at the DCLS, the expectation is that damage to acceleration-sensitive components will be reduced compared to buildings designed via standard methods.

The last two types of components listed above are typically referred to as “drift-sensitive”, implying that as the inter-storey drift demand increases, the likelihood of damaging these NSEs increases. Thus, by limiting the storey drift to 0.5% for DCLS load combinations, the expectation is that damage to drift-sensitive components will be limited. The specific value of 0.5% was informed by work of Dhakal and Saha (2017), shown in Figure C-2. This figure illustrates the expected loss due to damage of drift sensitive elements in a typical office building is 5% when the storey drift is 0.5% (using the curve corresponding to the slightly conservative mean plus one standard deviation; note that the drift is 0.6% if the mean value is used). Thus, this figure supports the assumption that a drift limit of 0.5% will limit repair costs in a typical office building to 5% the replacement cost, if it is also assumed that suitable measures will be taken to minimise losses to acceleration-sensitive components at this shaking intensity level too.

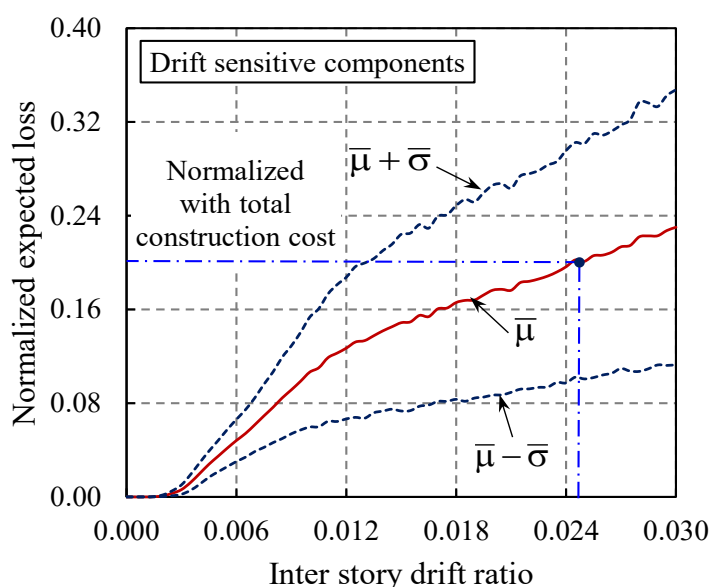


Figure C-2 Expected loss ratio (normalised by total construction cost, slightly less than typical replacement costs) obtained for typical New Zealand office buildings by Dhakal and Saha (2017).

Some NSEs can be considered sensitive to both storey drift and floor accelerations. This could include pre-cast concrete cladding panels, masonry infills (in older buildings) and lifts. Currently, however, the fragility of lifts is quantified using peak-ground acceleration as an intensity measure (Porter, 2016), suggesting that design provisions are not currently very effective in limiting damage to lifts, even though rails and other lift components are checked for acceleration demands during design. As such, future research could help identify better means of reducing the fragility of lifts.

C.2.2 Benchmarking the Prescriptive Design Method

To benchmark the Prescriptive Design Method for LDSD Category Level 2 and 3 buildings a set of case-study buildings were designed using draft LDSD criteria. These case study buildings were subsequently assessed using FEMA P-58, using fragility and loss functions applicable to the New Zealand construction industry where possible. The hypothetical case study buildings were located in Auckland, Christchurch and Wellington. The seismicity of these three cities could be described as low, high and very-high respectively.

Three lateral force resisting systems were examined at each site: 4-storey steel moment resisting frames (MRF), 4-storey cantilever reinforced concrete (RC) walls; and 3-storey timber-framed walls. The buildings were designed using the equivalent static method in accordance with NZ draft technical standard TS1170.5 (2024), with the draft LDSD criteria referred to earlier. The designs were developed in consultation with practicing engineers to ensure they were consistent with common New Zealand practice. Foundation flexibility was accounted for in a simplified manner (linear rotational springs) for the steel MRF and RC wall buildings, but non-linear soil-structure interaction effects were not included in the design or later performance assessment. For details of the building designs see Sullivan et al. (2025).

Following design, non-linear analysis models of the buildings were developed and subject to NLRHA at multiple intensity levels. Subsequently the resulting peak floor accelerations (PFAs) and peak storey drifts were input into PACT (FEMA P58.3) and the EALs and probability of realising 5% replacement cost, were computed.

For the loss assessment, an inventory of damage components was developed from the work of Dong et al. (2024) with adjustments for LDSD described in Sullivan et al. (2025). Component fragility and repair cost functions were taken from New Zealand literature where possible. However, the fragility for much of the mechanical and electrical equipment was taken from the FEMA P-58 database (highlighting the potential need for more research into such fragility functions in the future). HVAC units were assumed to be supported on vibration isolators in accordance with industry practice.

The median capacity for damage states related to bracing of acceleration-sensitive components was assumed to be proportional to the design intensity level. Consequently, for such elements, the median capacity was scaled in proportion to the PGA at the DCLS intensity level. A threshold loss value of 0.5 was adopted as part of the loss assessment in PACT. For additional details on the performance assessment, refer to Sullivan et al. (2025).

Figures C-3 and C-4 present an overview of the results obtained. From Figure C-3 it can be seen that the likely damage and expected annual losses are reduced, but the amount of reduction depends on the structural system and the local seismicity. In regions of low seismicity, such as Auckland, it would appear that there is no significant benefit of designing for LDSD category Levels 2 and 3, with Level 1 being sufficient. In contrast, in regions of very high seismicity, such as Wellington, the introduction of tighter drift limits led to the design of stiffer buildings with larger PFA demands.

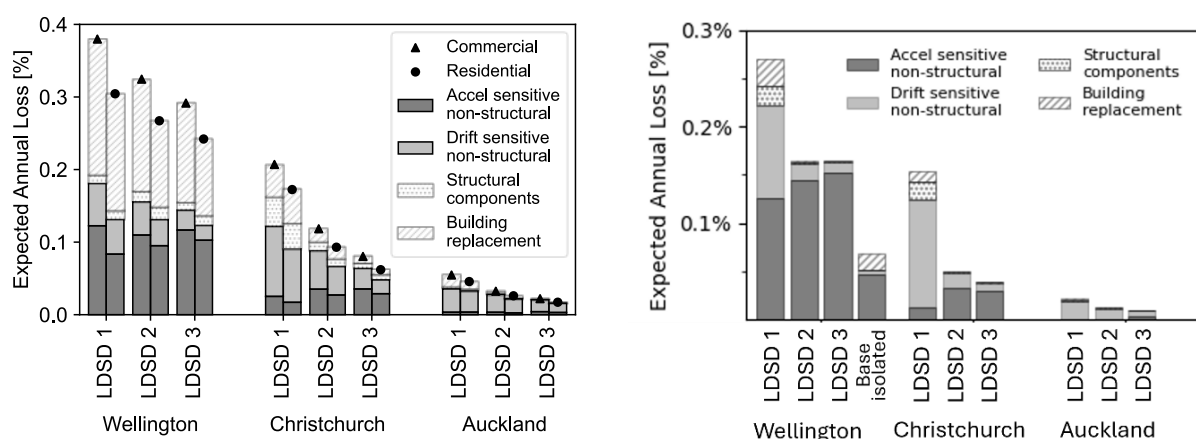


Figure C-3 Expected annual loss (EAL) values obtained for the cantilever RC wall structures (left) and steel MRF structures (right) obtained for the different LDSD buildings (from Sullivan et al. 2024).

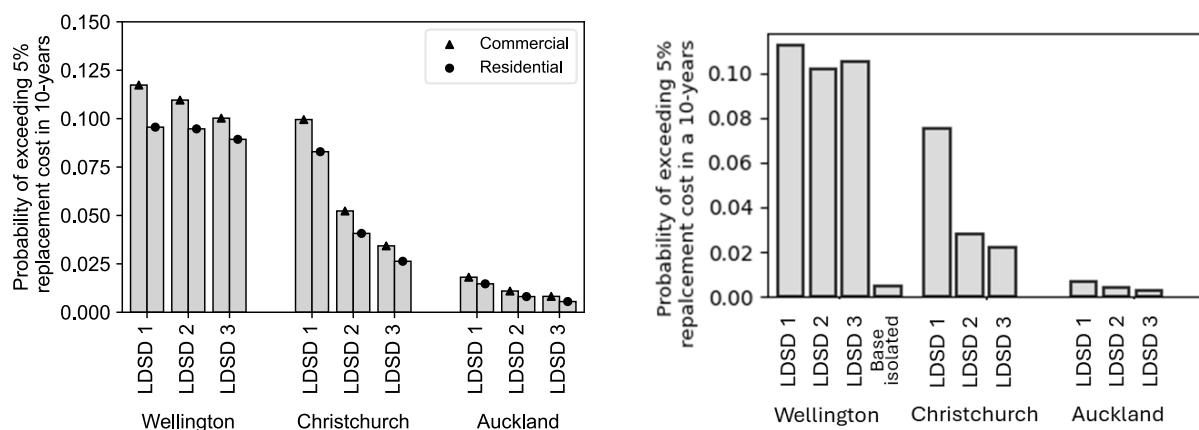


Figure C-4 Probability of earthquake repairs exceeding 5% of replacement cost over a 10-year period for the RC wall structures (left) and steel MRF structures (right) obtained for the different LDSD categories (from Sullivan et al. 2024).

The figures also show the contribution acceleration-sensitive NSEs and drift-sensitive structural and non-structural elements make to the EAL. The draft Prescriptive Design Method was seen to be relatively ineffective in Wellington due to damage to acceleration sensitive mechanical and electrical equipment, including lifts. As the damage levels in Wellington were high for the case study buildings, EAL values obtained for a base-isolated MRF building, designed and assessed by Dong et al. (2024), are also shown.

The damage to the acceleration-sensitive components in the non-isolated Wellington buildings almost completely offset the reduced damage and losses to drift-sensitive components. To address this issue PFA limits were introduced (using the results from the performance assessment) and seismic qualification requirements implemented, as a means of achieving desirable performance via the Prescriptive Design Method.

The results also demonstrate the benefits of using base-isolation in regions of very high seismicity, as an alternative approach.

Appendix D – LDSD Primary Structure System Option Summaries

D.1 General

This section summarises primary lateral structural systems commonly used in New Zealand. It provides high level LDSD information about each lateral system for the structural engineer to consider and communicate to the client and wider design team. In selecting the appropriate structural system for a LDSD solution the design team should consider if controlling drift or floor accelerations is a primary driver for structural system selection. This choice will be an important part of the Building Movement Strategy and will be informed by design requirements for non-structural elements (refer Section 3).

Regularity, symmetry and having the centre of mass close to the centre of rigidity are also important considerations in choosing a primary structural system.

There is no ‘one size fits all’ structural solution. Some projects may warrant base isolation whilst for others LDSD performance may be achieved via conventional structural wall or frame systems. The important point is that the whole design team must consciously select the structural system that best achieves agreed LDSD performance levels within the agreed project constraints.

The selection of an appropriate structural system must be done in concert with other project factors, in particular the foundations and the architectural aspects of the project. Section 6.3 has information on foundations and working closely with a geotechnical engineer to ensure an appropriate solution is the key to this design aspect.

Matters that should be discussed with the architect as part of LDSD thinking include the following:

- The basic floor plate arrangement and building cross-section to maximise regularity.
- The location of the building core(s) as these are opportunities for structural walls but are also present large penetrations in floor diaphragms.
- Likely plan layout of any fitout (applies to apartment buildings and offices), including circulation paths as this may impact on where any diagonal braces may be located, but also intertenancy walls can provide locations to hide structural elements.
- The material selection and the factors to make this decision ranging from basic suitability for structural purpose, carbon considerations, aesthetics, likely sizes of members, cost, availability, fire related issues etc,
- Detailing issues associated with more flexible structural systems, or structural systems with diagonal elements.
- Detailing associated with base isolation systems, in particular details for the perimeter rattle space and isolation plane, including any heavily serviced areas that cross the isolation plane.

D.2 Summary of Typical Primary Lateral Structural System Options

Table D-1 below summarises the relative performance of typical primary structural system options. The ratings provided are intended to be for general guidance only. It is acknowledged the ratings are subjective and depend on many factors, including stiffness and ductility demands on the structural system. Further information on drift and acceleration control is provided in the following sections.

Table D-1 Relative LDSD performance of typical primary structural systems

Key: x = poor, ✓ = good, ✓✓ = better, ✓✓✓ = best

	Moment Frame	Structural Wall	Braced Frame	BRBF	Supplem. Damped Systems	Base Isolation	Post Tensioned Wall	Post Tensioned Frame
Drift control	✓	✓✓✓	✓✓✓	✓✓✓	✓✓	✓✓✓ ¹	✓✓✓	✓
Floor acceleration control	✓✓✓	x	x	x	✓✓	✓✓✓	✓✓	✓✓✓
Self-centring ability	x	✓	✓	✓✓	✓✓✓	✓✓✓	✓✓✓	✓✓✓
System reparability ease	✓	x	✓✓	✓✓✓	✓✓✓	✓✓✓	✓✓✓	✓✓
Damage mitigation effectiveness	✓	✓	✓	✓✓	✓✓	✓✓✓	✓✓	✓✓
Simple construction	✓✓✓	✓✓✓	✓✓✓	✓✓✓	✓✓	✓✓	✓✓	✓

Notes:

1. Drift control rating in the table applies to the superstructure. Large inter-storey drifts will occur at the isolation plane.
2. It is noted that additional design effort above a standard verification method (VM1) is required for many of the systems listed above and this should be considered when discussing with your client.

D.3 Implications of Structural Response on Building Performance

D.3.1 Drift vs Acceleration

As discussed in Section 3 early decisions around selection of structural form can have significant impact on the cost and ease of achieving LDSD. This document recommends a Building Movement Strategy be collaboratively developed with the project team to ensure wider project performance requirements are identified and appropriately met.

Stiffer structural systems enable drift sensitive components to sustain higher intensity seismic events compared to the same component installed in an equivalent more flexible structure (i.e. because inter-storey movements are less). Drift sensitive components include facades, partition walls & ceiling interfaces. Limiting the seismic movements drift sensitive components need to be designed for can make a significant difference to performance and cost.

Flexible structural systems enable acceleration sensitive components to sustain higher intensity seismic events compared to the same component installed in an equivalent stiffer structure (i.e. because the floor accelerations are less). Acceleration sensitive components include some building services equipment, linear services, heavy façade items.

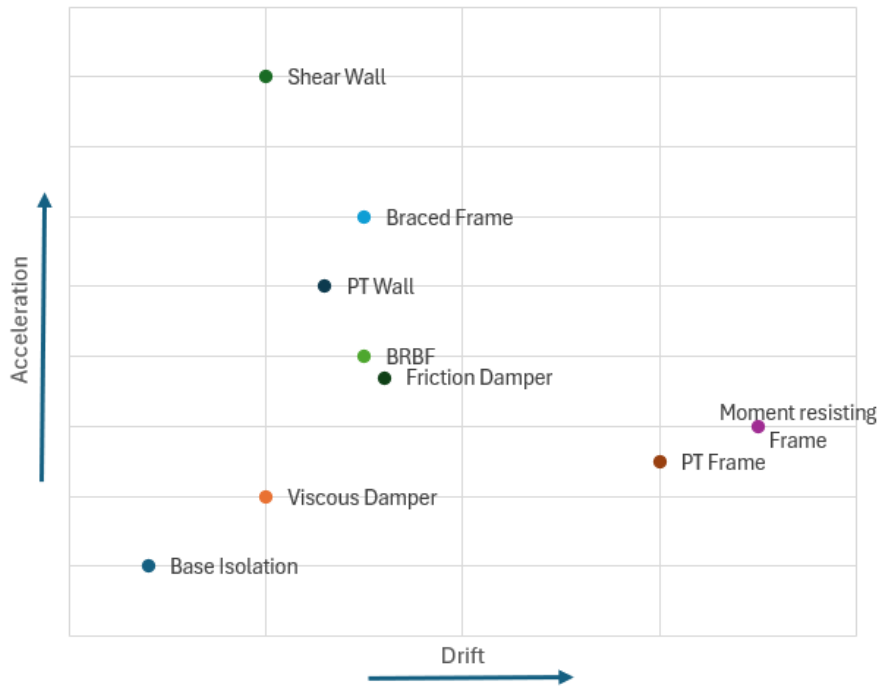


Figure D-1 Drift vs acceleration performance of typical structural systems

D.3.2 Plan and Vertical Regularity

Ensuring that structures are regular is very important for LDS. Figures D-2 to D-6 illustrate how undesirable building configurations can be turned into more desirable ones via good early decisions on structural form. While this is for the structural form it is critical that the whole design team is involved in setting up the structural for to achieve the performance goals.

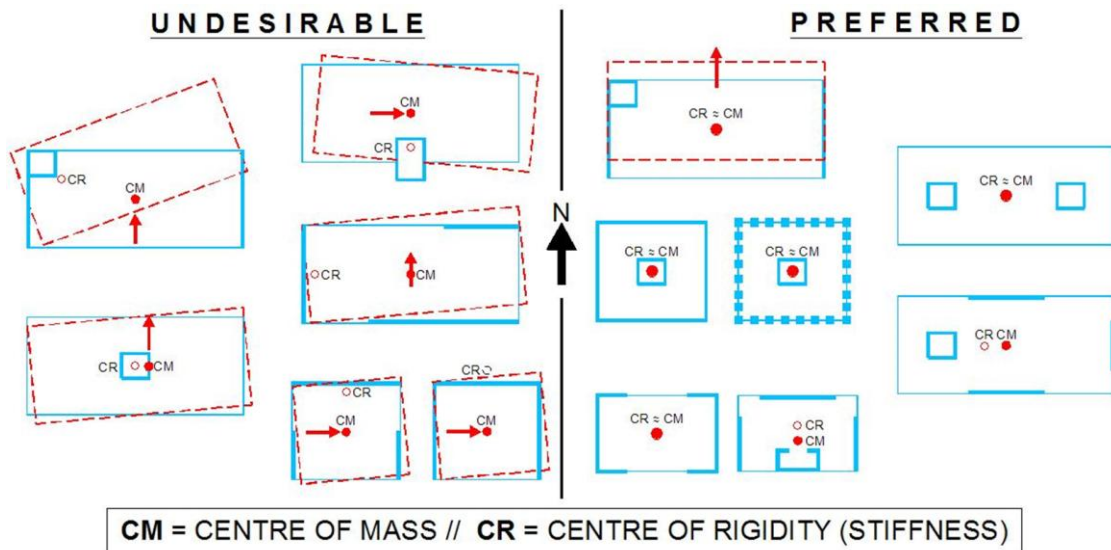


Figure D-2 Limiting undesirable torsional response.

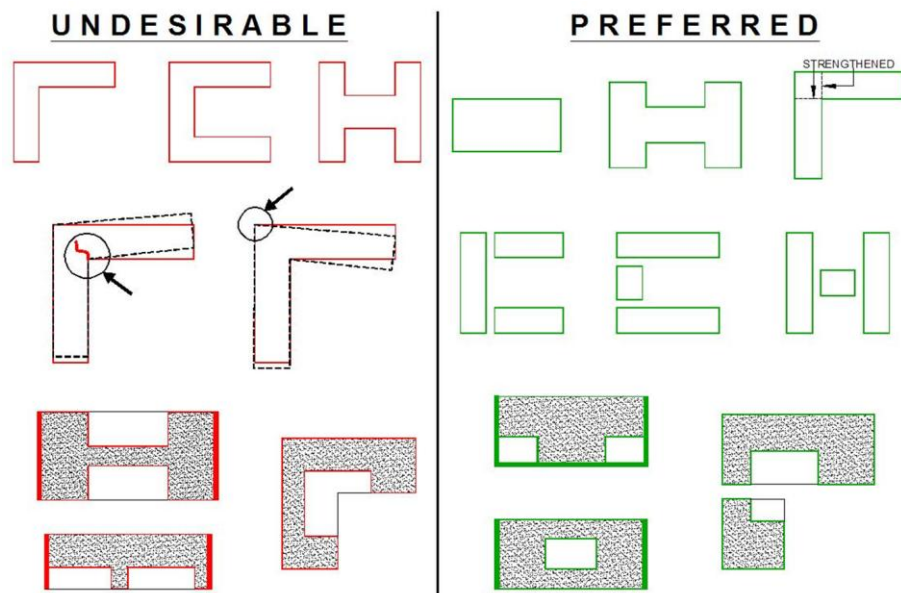


Figure D-3 Limiting undesirable plan configurations.

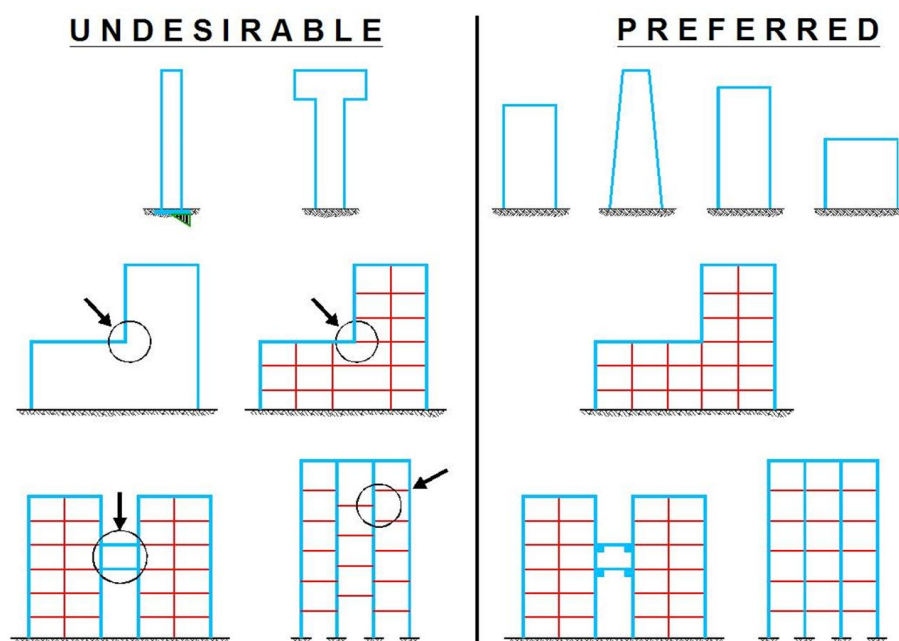


Figure D-4 Limiting undesirable vertical irregularities - multistorey.

LOW RISE EXAMPLES

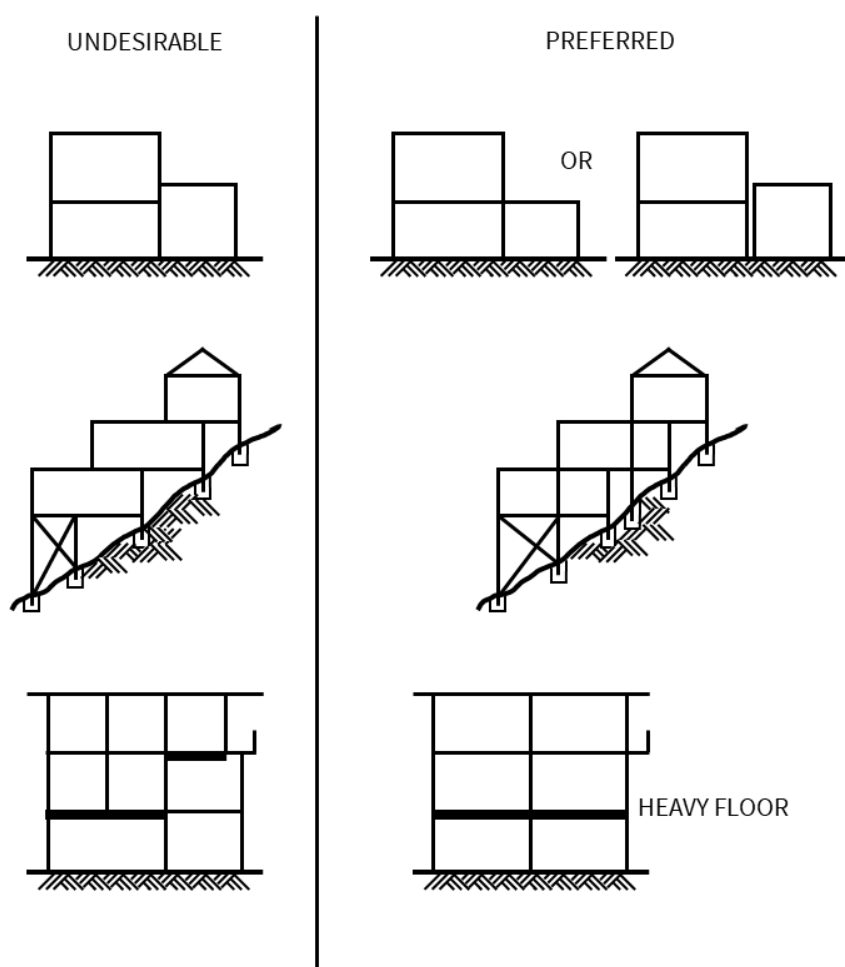


Figure D-5 Limiting undesirable irregularities - low rise

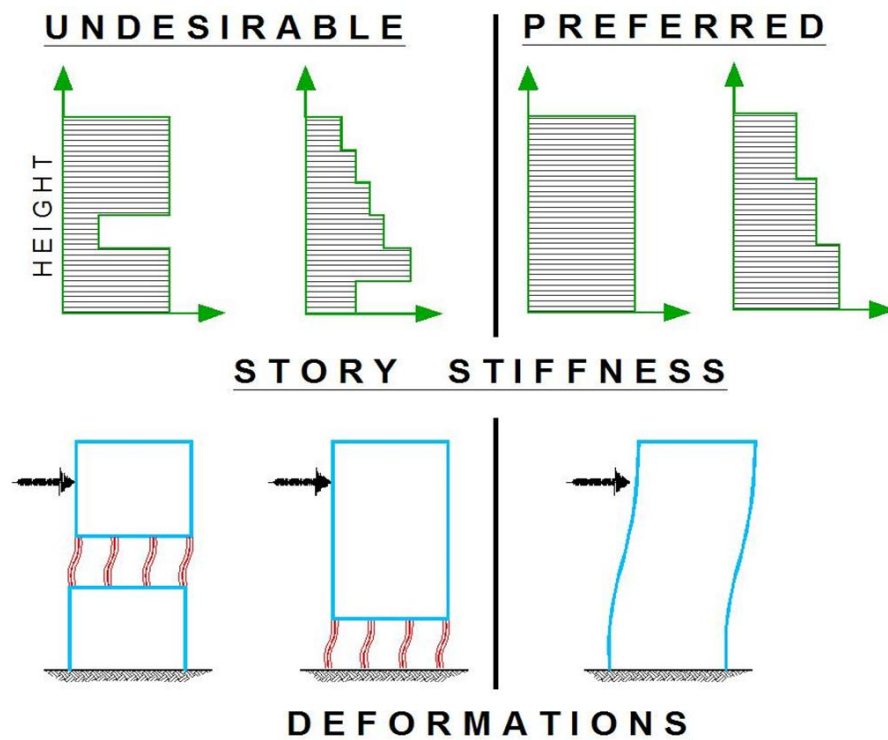


Figure D-6 Limiting undesirable vertical irregularities.

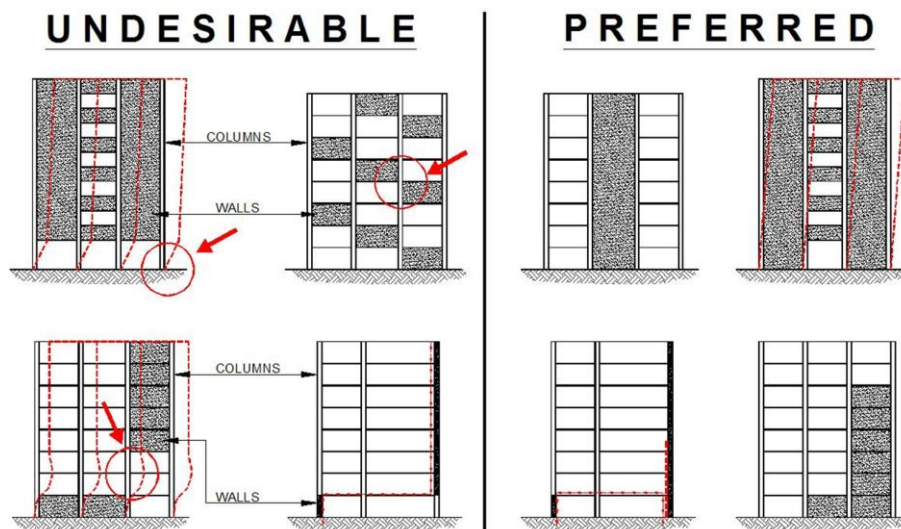


Figure D-7 Limiting undesirable vertical irregularities.

D.3.3 Other Considerations

Mixed structural systems in the same direction should generally be avoided where possible. If mixed systems are used consideration of deformation compatibility is critical.

Floor diaphragms are critical elements and must be designed to ensure the assumed load path is realised. Connectivity of the diaphragm to the lateral load resisting system is important and collector elements must be robust. Displacement compatibility of the diaphragm is important as is modelling the stiffness of the diaphragm.

Where possible the structural system and load path should be simple, symmetrical and include some redundancy.

D.4 Primary Lateral Structural System Summaries

This section describes typical primary lateral structural systems used in New Zealand. It provides high level LDSD information about each lateral system for the structural engineer to consider and communicate to the client and wider design team. This resource may be useful when project teams are considering which lateral systems may be most suitable for their LDSD project as part of the develop the Building Movement Strategy (refer Section 3.2.2.3).

D.4.1 Structural Walls

D.4.1.1 Description

A structural wall is a structural system containing load bearing structural walls that resist both vertical and horizontal loads and supplemented with beams and columns to resist vertical loads. Structural walls are typically constructed from reinforced concrete, reinforced masonry and engineered timber (i.e. cross-laminated timber).

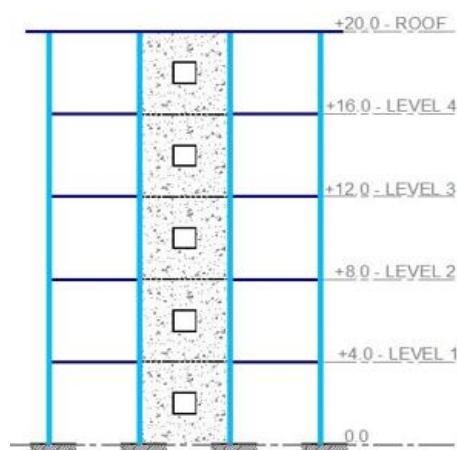


Figure D-7 Example of a structural wall.

The critical element is the yielding plastic hinge regions which generally occur at the bottom corners of the wall.

Reinforced concrete structural walls are suitable for low to high-rise structures. Whilst timber structural walls are typically suitable for low to medium-rise structures. Reinforced masonry structural walls are typically suited to low-rise structures.

D.4.1.2 Performance Considerations

Refer to Table D-2 below for a summary of LDSD performance considerations for structural walls.

Table D-2 Performance considerations for structural walls.

Drift control	Stiff structural form governed by wall length. Drift sensitive components can sustain higher-level seismic events without being damaged compared to the same component installed in more flexible structures.
Floor acceleration control	High accelerations compared more flexible structural systems. Acceleration sensitive components will be damaged in lower-level seismic events compared to the same component installed in more flexible structures. Therefore, this system will require greater level of attention to DCLS accelerations, detailing and selection of plant and equipment to achieve DCLS performance for elements such as ducts, pipework, plant and equipment.
Self-centring Ability	This system does not have active recentring capability. The system is prone to significant residual drifts should it experience high ductility demands. Conversely residual drifts can be expected to be low for low ductility demands.
System reparability ease	Plastic hinge damage usually confined to small areas. Damaged reinforcement is difficult to repair
Damage mitigation effectiveness	Relatively good up until low/modest levels of plastic deformation without the need for major repairs.
Simple construction	A conventional system that competent contractors will be experienced in.

D.4.1.3 General Considerations

Length and placement of structural walls can make a significant difference in performance.

Structural walls are commonly located adjacent diaphragm penetrations (i.e. around lift shafts and stairwells) and require thoughtful collector design to ensure reliable loadpaths are maintained.

Penetrations in structural walls require thoughtful design and detailing.

Coupled structural walls require thoughtful design and detailing. Coupling beams will often be subject to high local ductility demands. Specific checks on these local ductility demands are required to ensure damage to coupling beams is limited to acceptable levels.

Structural wall foundation loads can be concentrated so the system may not be compatible for sites with poor ground conditions.

D.4.2 Moment Resisting Frames

D.4.2.1 Description

A moment resisting frame (MRF) is a structural system containing beams and columns that resist both vertical and horizontal loads. MRFs can be constructed from steel, concrete, timber or a hybrid structure that contains mixed materials in the structural system.

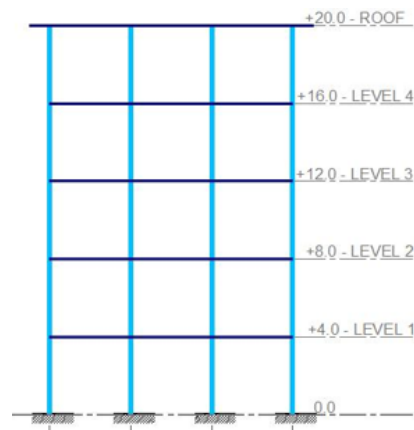


Figure D-8 Example of a moment resisting frame.

The critical element is the beam column joint and energy dissipation is normally via plastic hinges (damage) in steel and concrete MRFs that form in the beams adjacent to columns, and via the connections in timber MRFs.

Steel and concrete MRFs are suitable for low to high-rise structures. Whilst timber MRFs are typically suitable for low to medium-rise structures. One storey, one bay MRFs are often called portal frames

D.4.2.2 Performance Considerations

Refer to Table D-3 below for a summary of LDS performance considerations for moment resisting frames.

Table D-3 Performance considerations for moment resisting frames.

Drift control	Flexible structural form governed by number of bays and bay length. Drift sensitive components will be damaged in lower-level seismic events compared to the same component installed in stiffer structures. Therefore, this system will require greater level of attention to DCLS drifts and detailing to achieve DCLS performance for drift sensitive elements such as facades, partitions and brittle cladding.
Floor acceleration control	Relatively low. Generally, will provide the lowest accelerations for conventional structural systems (i.e. not including base isolated or viscous damped structures).
Self-centring Ability	This system does not have active recentring capability. The system is prone to significant residual drifts should it experience high ductility demands. Conversely residual drifts can be expected to be low, for low ductility demands.
System reparability ease	Plastic hinge damage usually confined to small areas. Damaged reinforcement is difficult to repair
Damage mitigation effectiveness	Relatively good up until low/modest levels of plastic deformation without the need for major repairs.
Simple construction	A conventional system that competent contractors will be experienced in.

D.4.2.3 General Considerations

Number and length of bays can make a significant difference in building stiffness and performance.

Regularity of bay length is important for efficient design, and this must be balanced with functional layouts

Column design needs to consider if the frame is a one-way vs two-way so biaxial demands are appropriately account for.

Good performance can generally be achieved with stiff frames designed to have low ductility demands.

D.4.3 Conventional Braced Frames

D.4.3.1 Description

There are various types of braced frames, typically constructed from structural steel (refer Figure D-9):

- Centrally braced frames (CBF)
- Eccentrically braced frames (EBF)

The critical element for EBFs is the active link and energy dissipation is normally via plastic deformation (damage) forming in the active link of the beams.

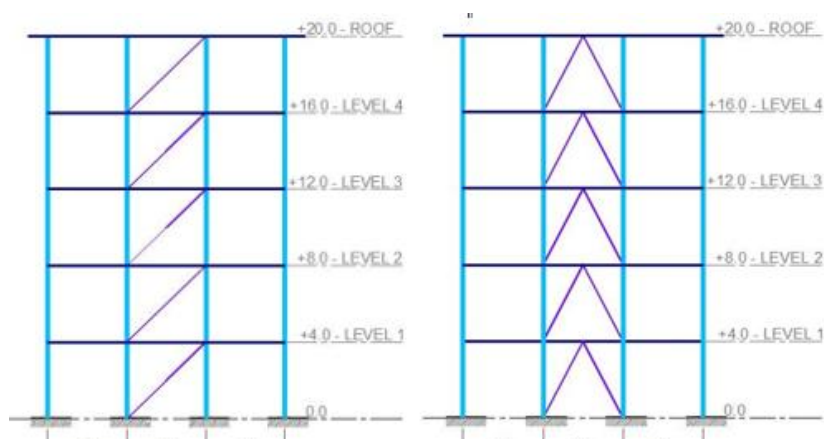


Figure D-9 Examples of concentric braced frames (left) and eccentric braced frame (right) systems.

The critical element for CBF's is the brace and energy dissipation is via brace yielding (damage). Conventional CBF's usually rely on a pair of opposing braces acting in tension as the tension/compression capacity of braces is usually vastly different due to slenderness.

Braced frames are suitable for low to high-rise structures.

D.4.3.2 Performance Considerations

Refer to Table D-4 below for a summary of LDSD performance considerations for braced frames.

Table D-4 Performance considerations for braced frames.

Drift control	Stiff structural form governed by number of braced bays. Drift sensitive components can sustain higher-level seismic events without being damaged compared to the same component installed in more flexible structures.
Floor acceleration control	High accelerations compared with more flexible structural systems. Acceleration sensitive components will be damaged in lower-level seismic events compared to the same component installed in more flexible structures. Therefore, this system will require greater level of attention to DCLS accelerations, detailing and selection of plant and equipment to achieve DCLS performance for elements such as ducts, pipework, plant and equipment.
Self-centring ability	This system does not have active recentring capability. The system can be prone to significant residual drifts should it experience high ductility demands. Conversely residual drifts can be expected to be low, for low ductility demands. Residual drifts can be controlled using a secondary, elastic seismic resisting system.
System reparability ease	Plastic hinge damage usually confined to small areas. CBF is repaired by replacing the brace and EBF by replacing the active link and potentially repairs to the floor slab above. Replaceable active link EBFs are easier to repair. Repairability depends on access to the damaged element.
Damage mitigation effectiveness	Relatively good up until post elastic response.
Simple construction	A conventional system that competent contractors will be experienced in.

D.4.3.3 General Considerations

Number and length of bays can make a significant difference in performance. Braces are most efficient at 45 degrees so braced bay length should be approximately two times the storey height.

Column design needs to consider if the frame is a one-way vs two-way so biaxial demands are appropriately accounted for.

Location of braced bays should consider both functional layout requirements and availability of tributary building 'dead' load to resist brace uplift demands.

Thoughtful detailing of floors/diaphragms adjacent active links is required to aid reparability, and to ensure a reliable load path for diaphragm collector forces is provided.

Braced frame foundation loads can be concentrated so the system may not be compatible for sites with poor ground conditions.

D.4.3 Buckling Restrained Braced Frames

D.4.3.1 Description

Buckling restrained braced frames (BRBFs) are typically used in concentrically braced frame systems. The buckling restrained brace (BRB) can yield in both tension and compression. The BRB is the primary energy dissipating element.

The supply of the BRB is usually via a manufacturer and requires the structural engineer to work closely with the BRB manufacturer to ensure the BRB performs as intended.

Steel BRBs braces can be used in conjunction with structural steel, concrete and timber frames.

Braced frames are suitable for low to high-rise structures.



Figure D-10 Example of a structural steel BRBF (NIST, 2015).

D.4.3.2 Performance Considerations

Refer to Table D-4 below for a summary of LDSD performance considerations for braced frames.

Table D-5 Performance considerations for BRBFs.

Drift control	Stiff structural form governed by number of braced bays. Drift sensitive components can sustain higher-level seismic events without being damaged compared to the same component installed in more flexible structures.
Floor acceleration control	Higher accelerations compared with more flexible structural systems. Acceleration sensitive components will be damaged in lower-level seismic events compared to the same component installed in more flexible structures. Therefore, this system will require greater level of attention to DCLS accelerations, detailing and selection of plant and equipment to achieve DCLS performance for elements such as ducts, pipework, plant and equipment.
Self-centring ability	This system does not have active recentring capability. The system can be prone to significant residual drifts should it experience high ductility demands. Conversely residual drifts can be expected to be low, for low ductility demands. Residual drifts can be controlled using a secondary, elastic seismic resisting system.
System reparability ease	Relatively simple to replace the brace provided access is adequate. The braces will be bespoke so replacement may be subject to reasonable lead times. Inspection of post elastic yielding of the brace can be problematic as it is a sealed system.
Damage mitigation effectiveness	Relatively good up until modest levels of plastic deformation without the need for major repairs.
Simple construction	A conventional system that competent contractors will be experienced in. The BRB itself is a specialist item that is bespoke for the building and may be subject to lead times.

D.4.3.3 General Considerations

Number and length of bays can make a significant difference in performance. Braces are most efficient at 45 degrees so braced bay length should be approximately two times the storey height.

Column design needs to consider if the frame is a one-way vs two-way so biaxial demands are appropriately accounted for.

Location of braced bays should consider both functional layout requirements and availability of tributary building 'dead' load to resist brace uplift demands.

Braced frame foundation loads can be concentrated so the system may not be compatible for sites with poor ground conditions.

As BRBs are a proprietary item, the designer must work with the brace supplier to ensure the device performs as expected. Supply and lead times for the brace needs to be considered.

D.4.5 Supplementary Damped Systems - Fluid Viscous Dampers

D.4.5.1 Description

Fluid viscous damped systems typically consist of proprietary fluid viscous dampers designed to work in parallel with conventional lateral structural systems such as moment resisting frames, structural walls or buckling restrained braced frames.



Figure D-11 Example of fluid viscous dampers in a steel structure (Taylor Devices, 2025).

Fluid viscous dampers work by dissipating energy from earthquakes as heat thereby reducing inter-storey deflections and floor accelerations.

Fluid viscous dampers can reduce foundation loads.

Fluid viscous dampers are suitable for low to high-rise structures.

D.4.5.2 Performance Considerations

Refer to Table D-6 below for a summary of LDSD performance considerations for lateral systems with supplementary fluid viscous dampers.

Table D-6 LDSD performance considerations for lateral systems with supplementary fluid viscous dampers.

Drift control	Reduced drifts compared to equivalent conventional system. Dampers effective over wide range of seismic demand. Drift sensitive components can sustain higher-level seismic events without being damaged compared to the same component installed in more flexible structures.
Floor acceleration control	Reduced accelerations compared to walls and braced frames. Generally, will provide lower accelerations compared with conventional systems.
Self-centring ability	Good self-centring ability if used in parallel with conventional structural systems that are designed to have low ductility demands.
System reparability ease	Fluid viscous dampers designed so repairs will not typically be required except for very large earthquakes. Dampers can be easily removed and replaced if required. Dampers are bespoke devices so replacement or repairs may be subject to reasonable lead times.
Damage mitigation effectiveness	Good up until the conventional parallel system sustains significant yielding.
Simple construction	A system that shouldn't be an issue for competent contractors. The damper itself is a specialist item that is bespoke for the building and may be subject to lead times.

D.4.5.3 General Considerations

Fluid viscous damped frames in seismically active regions can have larger grids and typically smaller internal columns when compared with conventional MRFs. Architecturally, the damper braces are typically located around lift cores and stairwells as well as the perimeter of the building. This can impose some limitations to the floor plan layout to achieve a regular building. As gravity loads are resisted by columns, floor plates are still relatively open otherwise.

Diagonal fluid viscous dampers located on the perimeter of the buildings will need to be integrated into the external architectural aesthetic and daylight considerations. From a planning perspective diagonal dampers are fixed in place and size for life of the structure, and will need to be worked around if the building is to be repurposed in the future.

Fluid viscous damped structures typically have lower foundation loads when compared with equivalent conventional structural systems.

As fluid viscous dampers are a proprietary item, the designer must work with the damper supplier to ensure the device performs as expected. Supply and lead times for the dampers need to be considered.

D.4.6 Supplementary Damped Systems - Friction Dampers

D.4.6.1 Description

Friction damped systems typically consist of proprietary friction devices designed to work in series with conventional lateral structural systems such as moment resisting frames, structural walls or braced frames.

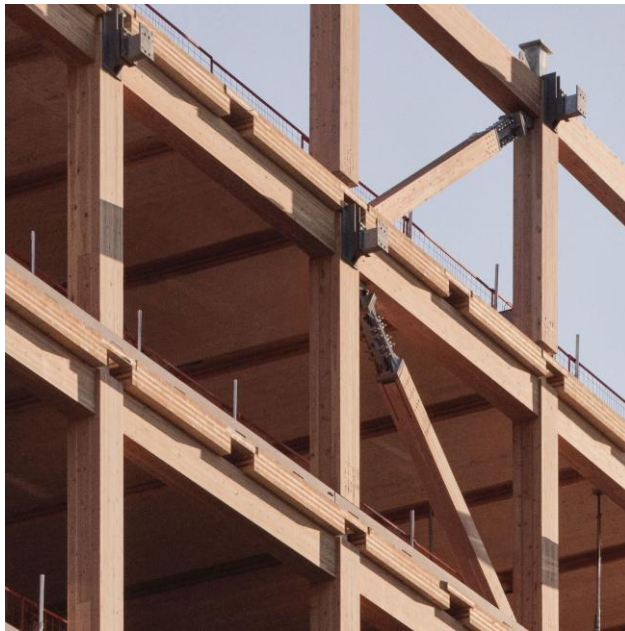


Figure D-12 Example of a friction damper systems (left) as a friction brace in a mass timber building (Fast + Epp, 2025) and (right) as a hold-down device in reinforced concrete structural wall (Tectonus, 2025)

Friction dampers work by dissipating energy from earthquakes as friction thereby reducing inter-storey deflections and floor accelerations.

Friction dampers are suitable for low to medium-rise structures.

D.4.6.2 Performance Considerations

Refer to Table D-7 below for a summary of LDSD performance considerations for lateral systems with supplementary friction devices.

Table D-7 LDSD performance considerations for lateral systems with supplementary friction devices.

Drift control	Stiff structural form governed by number of devices and device characteristics. Drift sensitive components can sustain higher-level seismic events without being damaged compared to the same component installed in more flexible structures.
Floor acceleration control	Generally reduced accelerations compared to conventional walls and braced frames, but maybe higher compared with moment resisting frames. Generally, will provide lower accelerations compared with conventional systems.
Self-centring ability	Some friction dampers have self-centring characteristics, so when these are used good control of residual drifts can be expected.
System reparability ease	Friction dampers designed so repairs will not typically be required except for very large earthquakes. Dampers can be easily removed and replaced if required. Dampers are bespoke devices so replacement or repairs may be subject to reasonable lead times.
Damage mitigation effectiveness	Good.
Simple construction	A system that shouldn't be an issue for competent contractors. The damper itself is a specialist item that is bespoke for the building and may be subject to lead times.

D.4.5.3 General Considerations

As friction dampers are a proprietary item, the designer must work with the damper supplier to ensure the device performs as expected. Supply and lead times for the dampers need to be considered.

D.4.6 Base Isolation System

D.4.6.1 Description

A base isolation system is a method of seismic protection where the building superstructure is separated from the base (foundation or substructure) by seismic isolators. The seismic isolators decouple the building superstructure from earthquake induced ground shaking limiting (refer Figure D-13). This limits the amount of seismic energy that is transferred to the superstructure. Typical seismic isolator types used in New Zealand include:

- Lead rubber bearings.
- High damping rubber bearings.
- Curved surface slider (i.e. Friction Pendulums™).
- Flat sliders (i.e. pot bearings).

The building superstructure is typically constructed using either reinforced concrete or structural steel, although more recently a number of isolated buildings have been constructed using engineered timber. Moment resisting frames or braced frames are typically used as primary lateral resisting systems for the superstructure.

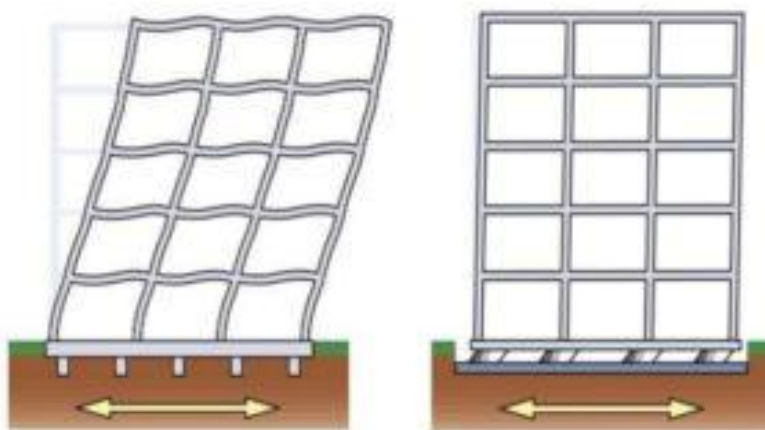


Figure D-13 Example of a conventional (left) vs base isolated building (right) (Quanser, 2025)

The isolation plane can be at foundation level or above, usually ground level (refer Figure D-13). Base isolated buildings need sufficient mass to engage the isolation devices and require movement space around the building often termed 'the moat or rattle space'. The structure behaves like a conventional building until seismic demands are sufficient to activate the seismic isolators.

Base isolated buildings are suitable for low to medium rise structures.

D.4.6.2 Performance Considerations

Refer to Table D-8 below for a summary of LDSD performance considerations for viscous damped systems.

Table D-8 Performance considerations for base isolation buildings.

Drift control	Reduced drifts compared to an MRFs. Drift sensitive components can sustain higher-level seismic events without being damaged compared to the same component installed in more flexible structures. Large drifts at the isolation plane which will require special detailing (can be several hundred millimetres). Any services or components crossing the isolation plane need careful detailing to ensure they can accommodate the expected movement.
Floor acceleration control	Reduced accelerations compared to walls and braced frames. Generally, will provide the lowest accelerations for typical structural systems
Self-centring ability	Very good. Isolation systems typically have active self-centring
System reparability ease	Isolators designed so repairs will not be required except for very large earthquakes. Isolator can be readily removed and replaced if needed.
Damage mitigation effectiveness	Very good. Localised non-structural repairs may be needed at the isolation plane following larger events.
Simple construction	Construction requirements for the isolation plane won't be familiar to many contractors. The seismic isolators are specialist devices and may be subject to lead times.

D.4.6.3 General Considerations

Base Isolated MRF's in seismically active regions have modest grid spacing requirements and work best with a regular grid layout. Architecturally, they allow for floor plates that are relatively open and the external façade in only obstructed by columns on the grid lines.

The isolators concentrate the displacements at the isolation plane elsewhere drifts are generally lower. Fluid viscous dampers can be used in parallel with seismic isolators across the isolation plane to reduce rattle space requirements.

The rattle space (or moat) can take up site area and needs careful detailing and integration with hard/soft landscaping elements.

Accommodation of the rattle space (or moat) means that it will not be possible to build the structure up to the property boundary.

As base isolator supply is a proprietary item, the designer must work with the supplier to ensure the brace performs as expected.

As seismic isolators are proprietary items, the designer must work with the isolator supplier to ensure the devices perform as expected. Supply and lead times for seismic isolators need to be considered.

Appendix E – Industry Research Needs

Industry research needs identified during the preparation of this document are detailed in Table E-1 below.

Table E-1 Summary of industry research needs identified during the preparation of this document.

No	Description	Section Reference
1	<u>Guidance for existing buildings</u> The scope of this document is primarily aimed at new buildings and does not consider in detail how the performance of existing buildings could be assessed. Existing buildings might have details or systems which might not comply with the requirements of this document i.e. reinforcement which is prone to strain ageing or non-rugged proprietary services equipment which is not seismically certified. Further guidance is needed on how to assess the LDS performance of existing buildings.	1.2
2	<u>Nonlinear soil/rock response and soil yielding at high levels of seismic shaking</u> At high levels of seismic shaking, typically above a PGA of about 0.8g, nonlinear soil/rock response and soil yielding can have an effect on seismic site response, irrespective of whether liquefaction or other softening effects occur. This generally makes elastic site response methods inaccurate and potentially leads to unconservative design. Further research is needed into the levels of shaking and ground conditions where these effects can influence the structural design, particularly from a displacement perspective.	3.4 and 6.2
3	<u>NZ database of seismically qualified proprietary equipment</u> Seismic qualification of proprietary services equipment can add significant time and costs to projects. Research is needed to determine if there is value in establishing and maintaining a NZ database of seismic qualified proprietary services equipment noting the number of impeding factors identified in Appendix H. Furthermore, this should include identifying which organisation(s) would be best place to host such a database.	6.6
4	<u>Performance of base isolated buildings when subject to tsunami effects</u> Uncertainty exists regarding the performance of base isolated buildings when subject to tsunami effects due to the potential for tension on the isolators due to overturning and buoyancy actions. Further design guidance is needed to address this uncertainty both in terms of limiting damaging and ensuring life safety risks are appropriately addressed.	4.8.3
5	<u>Impact of ground improvement on seismic site response</u> Ground improvement provides an opportunity to replace site soils beneath a shallow foundation with engineered soils with more uniform and predictable characteristics, which allows decoupling of building and ground performance. However, consideration needs to be given to how the ground improvement affects seismic site response, potentially in a negative way if the depth of ground improvement is significant or the extent of ground improvement across the site is large. Further research is needed into how much of an effect different types, extents, and depths of ground improvement can modify site response and change the design loads considered for the structure supported on the ground improved soil.	6.3
6	<u>Effects of liquefaction on seismic soil-foundation response</u> When considering the effects of liquefaction on seismic soil-foundation response, it is generally considered overly conservative in pseudo-static analysis to adopt the peak seismic shaking at the same time as adopting liquefied soil strength parameters and liquefaction induced foundation settlement. Some guidance on this topic is provided on the NZTA Bridge Manual (NZTA, 2022), although this is more focused on cyclic ground displacement and lateral spread and associated kinematic loading on piles. Further research and guidance are needed for the effects of liquefaction induced vertical effects on piles and shallow foundations.	6.3.3.3
7	<u>DCLS structural ductility factor limits</u> DCLS structural ductility factor limits detailed in Table 6-2 for the Prescriptive Design Method imply near elastic performance when considering that nominal strengths and strength reduction factors are used, furthermore they are more restrictive than that prescribed in the US Functional Recovery provisions. Further research is needed to confirm if these could be relaxed.	6.4.3

8	<u>DCLS Inter-storey drift limits</u> Inter-storey drift limits detailed in Table 6-3 were originally derived for commercial office buildings. Limited validation has been completed for apartment buildings. More research is required to confirm if the inter-storey drift limits detailed in Table 5-3 are appropriate for apartment buildings, and other building occupancy types.	6.4.3
9	<u>PFA thresholds for NSE seismic qualification</u> Section 6.3.3 details PFA thresholds for seismic qualification. If this limit is exceeded, the seismic performance of non-rugged acceleration sensitive NSEs is required to be validated by seismic verification. The methodology could benefit from a more granular approach whereby acceleration sensitive NSEs are appropriately categorised and then PFA thresholds are assigned to each category. This might have the benefit of reducing the amount of NSE seismic qualification required on projects.	6.4.3
10	<u>NZS 3101 design provisions for ductile jointed precast concrete systems</u> Research is needed to confirm if the design provisions in NZS 3101 Appendix B for ductile jointed precast concrete systems will meet LDSD damage control and improved functionality performance goals. This includes confirming if the reinforcement strain limits are sufficient to address low cycle fatigue considerations (refer related item 13 below).	6.4.3.1
11	<u>Design guidance for structures with damping systems</u> New Zealand lacks design provisions for structures with supplemental damping systems such as fluid viscous dampers or friction dampers. Further research and guidance are needed to enable design teams to design and specify damping systems to ensure acceptable levels of seismic performance are provided.	6.4.4
12	<u>DCLS deformation limits for structural components</u> Research is needed to confirm if the DCLS deformation limits detailed in Section 6.4.4.3 ensure an appropriate level of seismic performance is achieved. In particular, deformation limits proposed for structural steel components have been derived using a limited data set and a more rigorous review of the underlying sources of uncertainty and related experimental results would be beneficial.	6.4.4.3
13	<u>DCLS strain limits and low cycle fatigue coefficients for G500E reinforcement</u> Research is needed to confirm low cycle fatigue coefficients (i.e. appropriate to the Coffin-Manson equation or Koh and Stephens equation) for Grade 500E reinforcement so the industry can have greater confidence of the accuracy of fatigue damage assessments undertaken in accordance with FEMA P-2335 (FEMA, 2025). Furthermore, there is a need to validate the tentative strain limits provided in Table 6-13 to ensure acceptable levels of seismic performance are provided.	6.4.4.3
14	<u>Design guidance for lifts</u> Limited experimental evidence on the seismic performance of passenger lifts commonly used in New Zealand is available. Further research and guidance is needed to enable design teams to design and specify lifts with greater confidence that target building reoccupation times will be achieved.	6.8.3
15	<u>Deflection limits for direct fixed plasterboard ceilings</u> Deflection limits for direct fixed plasterboard ceilings is a topic of active research and the recommendations provided in this document have been informed by judgement. Further experimental work is required validate the recommended limits to ensure acceptable levels of seismic performance are provided.	6.9.3.3
16	<u>Seismic and passive fire performance of 'seismically resilient' non-load bearing partitions</u> 'Seismically resilient' non-load bearing partition walls commonly used in buildings with post-earthquake functionality requirements have limited experimental evidence to validate their performance. This often results in design teams having to adopt conservative estimates of their in-plane seismic deformation capacity and post-earthquake fire rating performance. This can add significant cost to projects. Further research is needed to validate the performance of these partition walls including consideration of interaction effects with suspended ceilings.	6.10
17	<u>Design guidance for non-load bearing partitions in wet areas</u> The seismic performance of non-load bearing partition wall systems appropriate for wet areas (i.e. bathrooms and shows) is not well understood. This often results in design teams having to adopt	6.10

	conservative estimates of their in-plane seismic deformation capacity. Further research and guidance is needed to for non-load bearing partition wall systems appropriate for wet areas.	
18	<u>Design guidance for other the plasterboard non-load bearing partitions</u> Design guidance for non-load bearing partitions in the BIP NSE CoP (BIP, 2025) is generally limited to plasterboard lined walls. Further research and industry guidance is need for the design and detailing of other non-load bearing partitions including glazed partition walls, aerated concrete partition walls and masonry partition walls.	6.10
19	<u>No standard cited for assessing performance of firestops subject to movement</u> Section 6.11.3 recommends the use of ASTM E3037 for testing, subjecting fire stop systems to movement prior to fire testing. There is no standard cited in the current building regulations to mandate this. While this is not a problem for a voluntary compliance, from an industry perspective mandating such an approach might be beneficial for selected building types i.e. potentially some IL 3 and IL 4 buildings etc.	6.11.3
20	<u>Limited testing of seismic performance of firestop systems</u> To date there is limited testing of commercially available firestop systems for fire performance after being subject to seismic movement. Development, testing and certification of proprietary fire stopping products to specified drifts is required for services penetrations throughout the building. Note the design, construction, and performance of any one brand/model of proprietary product will not typically be representative of the product kind. Hence testing will need to be brand/model specific.	6.11.3
21	<u>ANARP guidance</u> Very little guidance is available to assist building owners and consultants determine what is ‘as nearly as is reasonably practicable’ (ANARP) when making building reoccupation decisions following an earthquake.	7.6.1
22	<u>Fragility functions for NZ building components</u> Fragility functions for building components commonly found in NZ buildings does not exist. As a consequence, when competing loss and down time assessments analysts often have to rely on overseas data which may not always be appropriate. In particular fragility and loss functions for the following components is required; lifts, ‘seismically resilient’ non-load bearing partition walls, firestop systems	App C
23	<u>Additional research to support benchmarking studies</u> Benchmarking studies completed as part of this project to validate the Prescriptive Design Method for LDSD Category Level 2 and 3 Buildings were limited in scope i.e. only considered 3 building typologies, two occupancy types and a restricted number of stories. Extending this benchmarking study to include a greater range of building typologies, occupancies and heights would be beneficial.	App C2.2

Appendix F – Downstream Social Benefits

LDSD buildings provide several direct and indirect benefits and opportunities. The benefits will resonate differently depending on the use of the building and the building owner's long-term interest in the property. Key benefits for building developers, owners and their tenants are summarised below.

F.1 Increased Certainty of Performance

There is a lot of uncertainty about the nature of earthquakes and how buildings will respond to them. LDSD can provide building owners (and their tenants) with greater certainty about how their building will perform in a given earthquake. This can help with planning for emergencies, business continuity, and financial management. This is particularly important for organisations that have time-critical operations and/or strongly depend on their building to operate.

F.2 Increased Property Value

LDSD has the potential to increase the capital value of a building and financial return from rent. The degree of increase will depend on the property market at the time and will likely differ depending on the seismic hazard of the region where the building is located and can be influenced by recency of a seismic event. For example, a study from 2017 (Filippova, Rehm, & Dibble, 2017) found that there was a rental premium for buildings with higher %NBS ratings in Wellington but not Auckland.

Even if the market is not valuing the investment, being a LDSD building will still be a differentiating feature that will appeal to some tenants.

F.3 Reduced Losses

LDSD buildings have clear financial benefits, especially over the lifetime of a building. These include reducing the likelihood of costly repairs and minimising the time required to return a building back to an operational state following an earthquake.

There are significant risks and uncertainties when repairing or rebuilding a building following an earthquake. Resource constraints (i.e. building materials and labour) can exacerbate the cost and time it takes to repair and rebuild buildings (King, Middleton, Charlotte, Johnston, & Johal, 2014 and Chang, Wilkinson, Potangaroa, & Seville, 2012). These constraints can exacerbate losses, including business disruption losses.

Following the Canterbury earthquakes, even those building owners and tenants that thought they were fully insured, found themselves suffering losses that were not covered. In many cases, business interruption losses extended beyond the indemnity period organisations were covered for (Brown, Seville, & Vargo, 2017). Similarly, residential building tenants were often displaced for longer than allowed for in the temporary accommodation allowances.

F.4 Improved Insurability

New Zealand's insurance market is marked by increasingly high premiums and coverage limitations. By investing in more seismically resilient LDSD buildings, building owners or tenants may improve their ability to secure ongoing insurance coverage. Furthermore, when coupled with post-earthquake damage inspection and recovery planning (refer Sections 7.5 and 7.6), LDSD can provide greater certainty of cost-effective building repair and quicker reoccupation times which may have insurance advantages.

F.5 Reduced Stress

During an earthquake, observation of significant building damage can contribute towards post-traumatic stress disorder (McBride, et al., 2018). In the days, weeks and months following an earthquake, dealing with insurance and disruption during repairs, or having to relocate temporarily or permanently, can be very stressful for building owners (Eade, Brown, Bird, & Brunsdon, 2023), tenants and staff (Bell, et al., 2016) (Stevenson, et al., 2011). By reducing building damage, LDSD can reduce the stress experienced, and this can contribute toward the mental health and wellbeing of occupants. For organisations this has benefits in terms of increased productivity and employee wellbeing.

F.6 Supporting Community Resilience

Collectively, individual buildings contribute to the overall functionality of communities. Having more buildings designed with LDSD means less damage and disruption after an earthquake. This can contribute to not only accelerated built environment recovery, but social and economic recovery too, by enabling access to essential goods and services, providing places for people to connect, providing business confidence, and quickly restoring a sense of normalcy (Brown, Abeling, Horsfall, Ferner, & Cowan, 2022). By adopting LDSD building owners are helping to improve the seismic resilience of their community.

Appendix G – US Functional Recovery Provisions

The United States Federal Emergency Management Agency (FEMA) is developing similar guidance for increasing the seismic resilience of buildings. A summary of the new guidance, and how it relates to LDSD, is included in the following sections.

G.1 Background

The US Building Seismic Safety Council (BSSC) has established a Functional Recovery Task Committee to develop seismic design provisions for functional recovery performance. The committee intends to integrate such provisions into the 2026 National Earthquake Hazards Reduction Program (NEHRP) Recommended Seismic Provisions for New Buildings and Other Structures. Ultimately, the goal is for the provisions to also inform the development of the 2028 edition of ASCE 7: Minimum Design Loads for Buildings and Other Structures, and, in turn, the 2030 edition of the International Building Code (Molina Hutt & Zimmerman, 2024).

G.2 Functional Recovery Performance Targets

Reasons for developing the new provisions include increasing the seismic resilience of buildings and infrastructure and thereby reducing injury and loss of life, community disruption (i.e. population displacement) and economic losses in future earthquakes.

To support resilience goals at the community level a new link has been established between the design, construction and retrofit of individual buildings and lifeline infrastructure systems, and community resilience, as measured by time to recovery of function. This includes the concepts of reoccupancy, and functional recovery defined as below (FEMA, 2021):

- *Reoccupancy* is a post-earthquake performance state in which a building is maintained, or restored, to allow safe re-entry for the purpose of providing shelter or protecting building contents.
- *Functional Recovery* is a post-earthquake performance state in which a building is maintained, or restored, to safely adequately support the *basic intended functions* associated with the pre-earthquake use or occupancy of a building.

A key concept for functional recovery is that basic intended function is less than full pre-earthquake functionality, but more than would be considered the minimum sufficient for reoccupancy of buildings. In the context of LDSD this is broadly similar to achieving Operational State II - Partial Functionality.

The provisions include the development of new building Functional Recovery Categories, Target Functional Recovery Times and a risk targeted Functional Recovery Earthquake (FRE_R), which is lower than that currently employed in US life-safety design. With reference to Table G-1, recovery time targets are defined by the building Functional Recovery Category. The risk-based characterisation of the earthquake ground motion hazard includes both a risk and a reliability target (Zimmerman, 2024):

- *Risk Target*: 10% chance of exceeding recovery time target in 50 years.
- *Reliability Target*: 25% chance of exceeding recovery time target conditioned on FRE_R ground motions.
- No explicit damage control performance targets (i.e. limiting EALs or similar) are expected to be a feature of the US Functional Recovery provisions. However, recognising the link between building damage and recovery times, the functional recovery provisions are expected to result in buildings that have a

significantly lower probability of sustaining damage that would lead to significant economic losses when compared with minimum code compliant buildings.

Table G-1: Proposed Functional Recovery Categories and Target Functional Recovery Times (Zimmerman, 2024)

Functional Recovery Category	Contribution of Structure to Community Resilience	Target Recovery Time
A	Structures with a basic intended function that the community relies on for immediate post-earthquake safety and survival i.e. buildings with special post-disaster functions, assisted living facilities, acute healthcare facilities	0 hrs
B	Structures with a basic intended function that supports urgent and basic safety and survival needs and prevents escalation of adverse disaster consequences i.e. supermarkets, non-urgent healthcare facilities, residential housing, vital utilities services	72 hrs
C	Structures with a basic intended function that support routine self- and group-preservation, and sustain short- and long-term economic, educational and governance activities and services i.e. recovery focused local government services, childcare, primary and secondary school, healthcare facilities providing routine or elective treatments, services and organisations critical to regional economic stability.	2 months
D	Structures with a basic intended function that enhance a community's general well-being and expedite the return to normalcy i.e. tertiary education facilities, routine local government services, services and organisations critical to local economic stability.	6 months
E	Structures not assigned to Functional Recovery Category A, B, C or D.	No target

The APoE of the risk targeted Functional Recovery Earthquake (FRE_R) is typically somewhere between the NZ DCLS earthquake APoEs recommended for LDSO Category Level 2 and 3 buildings. It is not possible to provide a direct comparison because the FRE_R APoE varies geographically across the US due to the risk targeted approach taken.

G.3 Structural and Non-structural System Design for Functional Recovery

The design provisions for Functional Recovery, which feature prescriptive and explicit (i.e. performance based) design requirements, will include new requirements for structural and non-structural system design aimed at enabling faster post-earthquake recovery.

For structural systems prescriptive design provisions will introduce new functional recovery response modification factors (R_{fr}) and drift limits for common structural systems to minimise the probability of safety-critical structural damage (i.e. structural damage that must be repaired before a building can be safely reoccupied) (Blowes, et al., 2025). These factors are dependent upon the Functional Recovery Category and are structural system specific.

For non-structural elements, the new provisions will include a seismic component importance factor for functional recovery, which is applied to both drift and acceleration sensitive components, as well as seismic qualification of equipment, to ensure recovery time targets are met (Blowes, et al., 2025). These new requirements are also dependent upon the building's Functional Recovery Category. Some exceptions to the seismic qualification of equipment are provided when the peak floor acceleration demands are lower than prescribed thresholds (Molina Hutt & Zimmerman, 2024). Blowes et al. (2025) also noted the need to perform separate studies to evaluate the impact of damage to elevators and fire rated partitions on building function.

G.4 Other Criteria to Achieve Functional Recovery

In recognition that structural and nonstructural design criteria alone are insufficient to guarantee speedy recovery, functional recovery provisions are also expected to include new requirements related to peer review, construction monitoring and quality assurance requirements. Furthermore, it is understood the functional recovery provisions may also include recommendations for the preparation of post-earthquake functional recovery plans, and introduce a new role of Functional Recovery Coordinator, i.e. a registered design professional tasked with the review and coordination of the documentation, observation, quality assurance and recovery plan for all elements, components, equipment and systems required to achieve basic intended function.

Appendix H – Factors Affecting Cost and Reliability of Seismic Qualification of Proprietary Services Equipment

Factors affecting the cost and reliability of seismic qualification of proprietary services equipment in NZ include:

- The seismic resilience of any one brand / model of services equipment will not typically be representative of the equipment kind.
- A significant portion of imported building services products available in the NZ market are sourced from Europe, Asia, or Australia, with variable and often unknown levels of seismic resilience and typically no seismic qualification. Equipment sourced from US is typically designed and manufactured for a certain degree of seismic resilience, and many have been seismically certified. However, many global building services equipment manufacturers supply fundamentally different products to their US, European, and Asia / Pacific markets. Sample enquires of NZ importers of US branded products (not a rigorous survey) suggest that a significant portion of such products are different to those supplied to the US market. Therefore, overseas product qualifications (e.g. from US) may not be transferable to the same brand products supplied to the New Zealand market. If such seismic qualifications are for the identical product supplied to the NZ market, they may not cover the levels of earthquake action applicable to their use in NZ.
- The NZ building services proprietary equipment manufacturing / importing industry should ideally be developing their ability to provide seismically certified proprietary equipment, “type tested” to cover a suitable range of project applications, with costs distributed across their respective current and future customer base. The sensible approach to seismically certifying a component for Operational Integrity is to shake it, see what breaks, fix what has broken (without voiding manufacturer’s warranty), shake again, repeating this process until the item still works after shaking. Voiding of manufacturer’s warranty shouldn’t be a major issue for locally manufactured products, but could be a significant issue for imported products, with local importers / distributors having limited experience in managing this risk.
- The consistency, reliability and availability of US proprietary equipment seismic qualifications are aided by California Department of Health Care Access and Information (HCAI) acting as an authority having jurisdiction (for healthcare in California), mandating protocols including:
 - i. equipment deemed inherently rugged,
 - ii. rules for sample type testing of product ranges (different sizes, options, etc),
 - iii. qualifications of engineers overseeing the manufacturer’s testing regime and approving qualification outputs,
 - iv. central approval and registration of seismically certified equipment (on a publicly available register),
 - v. re-qualification in the event of product changes,
 - vi. application of expired certificates, etc.

There is no equivalent seismic qualification authority having jurisdiction in NZ.

- NZ manufacturers/importers will need to determine the seismic accelerations (and vibration isolators where applicable) against which they analyse/test. Some, particularly importers given the potential warranty issues outlined above, may be reluctant to risk analysing / testing for the worst-case seismic accelerations in NZ (e.g. for an IL 4 building in Wellington) and may select a lower threshold. This is also more likely if their first analysis / testing process is driven by a specific project (that is not an IL 4 building in Wellington). While manufacturers / importers will typically need to engage engineers, without an overseeing agency similar to HCAI there is increased potential for inconsistent advice to their respective clients. Hence resultant seismic qualifications may be limited in their application, and designers are not assured of easy access to those limitations.
- Given all the above, plus the small size of the NZ market making it challenging for the industry to spread qualification costs across the market, some NZ building services proprietary equipment manufacturers / importers may consider there to be insufficient demand for seismically certified equipment to warrant their investment in type tested seismic qualification. Hence tenderers may opt to allow for one-off testing within the contract price. For example, we are aware of one tender for a regional hospital building including ~\$1m for mechanical services seismic qualification, including ~\$700k for testing an air-cooled chiller to destruction and then replacing it. One counter to this approach is for the building owner to accept the cost of one-off shake table testing subject to them being assigned ownership of the intellectual property associated therewith. At least one NZ supplier eventually agreed to accept the testing cost under these circumstances, so this is a sound and compelling response but does not guarantee the same outcome in every case.
- Hence seismic qualification could add reasonably significant cost (and possibly time) to LDS building services systems, exacerbated by the potential for limited confidence as to the extent of cost (and time) premiums until part way into the construction design phase. Furthermore, should a seismically certified product subsequently be found (via equipment technical submission reviews during the construction design phase) to have unacceptable non-compliances in other areas of performance, quality, durability, etc, it may be necessary to repeat the process with alternative suppliers, or accept some sort of compromise.
- While NZ already needs to develop suitably efficient and equitable solutions to the above challenges, the greater the scope of seismic qualification, the greater the scope of the challenges.

Appendix J – Façade Testing Standards Information

Information on Australian and New Zealand façade testing standards that may be useful for LDSD projects is included below.

J.1 NZS 4211 and AS/NZS 4420

NZS 4211 (SNZ, 2008) and the accompanying AS/NZS 4420 (SANZ, 2016) form the principal route for demonstrating compliance of windows and doors with clauses B1 and E2 of the NZBC. NZS4211 is cited in B1/VM1 and E2/AS1.

AS/NZS 4420 includes consideration of structural wind performance at SLS1 and ULS, and air infiltration and water penetration after the SLS1 structural (deflection) stage of the test. Notably, this standard does not include seismic structural performance testing, or post-seismic event testing for air and water penetration. As such (historic) NZS4211 test reports for doors and windows are unlikely to provide any useful information regarding seismic performance and as such are unlikely to be sufficient to demonstrate compliance with LDSD (or SLS2) performance requirements.

J.2 AS/NZS 4284

AS/NZS 4284 (SANZ, 2008) contains a more comprehensive testing regime for determining façade performance and includes consideration of SLS1 and ULS seismic test stages. AS/NZS 4284 testing is commonly used on more complex façade systems (e.g. curtain wall systems) to demonstrate compliance with NZBC using the alternative solution route and could possibly be used for more onerous project-specific performance requirements.

The building standard includes seismic racking at SLS1 drifts which is proceeded by air leakage and water penetration testing - this aligns with the SLS 'without repair' performance requirements. The test concludes with racking to ULS inter-storey drifts, with the condition the test specimen shall not collapse which is aligned with the ULS performance condition. Some limited glass breakage is permitted.

Where NZS/AS 4284 testing is being used to demonstrate compliance, historic test reports will commonly be used, particularly for locally (or Australian) produced façade systems. Consequently, these tests will not typically report on DCLS or SLS2 performance requirements. NS/NZS 4284 permits additional steps in the test sequence. If project specific testing is being considered the test regime could be tailored to test structural performance at DCLS (or SLS2) and weathertightness after this event.

It is worth noting that NZBC E2 includes a modified version of AS/NZS 4284 in E2/VM1 but, as noted in the scope section of the standard, this modified version is orientated towards cladding for 'domestic buildings' (i.e. buildings falling within the scope of NZS 3604). The abridged version in E2/VM1 includes only water penetration testing, not structural, seismic or air infiltration test components and may not be appropriate for LDSD projects.