INTERIM DESIGN GUIDANCE

DESIGN OF CONVENTIONAL STRUCTURAL SYSTEMS

October 2022 (v11)

CONTENTS

Document Status

This document was originally issued to provide guidance to structural and geotechnical engineers and to Territorial Authorities in the design of structures in the interim between the 2010/2011 Canterbury earthquakes and anticipated changes to the NZ Building Code.

As at April 2019, changes made to the NZ Building Code had superseded much of the original content of this document. Nonetheless SESOC see value in maintaining the document as an ongoing vehicle for disseminating new structural engineering knowledge and practice recommendations ahead of more formal document updates.

It is emphasised that the marking of information as superseded in this guidance does not connote that the subject issue is no longer important. Particularly where entire sections have been superseded it is instead generally the case that the issues now require mandatory consideration according to Standards and/or Building Code Verification Methods. This has generally been noted throughout the document.

SESOC intend to maintain the interim guidance as a means of disseminating updates regarding what constitutes good practice with respect to specific aspects of structural engineering. Such a vehicle is of increasing importance given the slow and in some cases confusing manner in which Standards and Verification Method B1/VM1 have been updated in recent years. It is anticipated that periodic updates to the interim guidance will be released, with the schedule determined to accommodate updated information as it becomes accepted.

It is important to note that this document is issued as guidance and that while it reflects the views of the Structural Engineering Society, it has no official status and its use may not be insisted upon in the processing of building consents. However, designers are advised to consider the issues raised and the possible solutions offered when preparing designs, and to exercise their engineering judgement in determining a suitable course of action in this regard.

Where errors or omissions are noted in the document, it is requested that users notify SESOC through exec@sesoc.org.nz.

Revision History

Version	Date	Notes
8	Sep. 2012	For formal review by SESOC membership
9	Mar. 2013	Minor corrections and typos
10	Sep. 2019	Amended to remove content superseded by Standards, Verification Methods, or other documentation as at mid-2019
11	Oct. 2022	Document reformatted to improve readability. Amended to remove content superseded by Standards, Verification Methods, or other documentation as at mid-2022. Updates to guidance related to damage reduction and other topics to include recent research learnings.

1 INTRODUCTION

Many observations were made of the performance of conventional structural systems following the 2010/2011 Canterbury earthquakes.

Additional observations were made following the 2016 Kaikoura earthquake, with particularly important information revealed and emphasised regarding the behaviour of hollow-core floors.

In general, it appears that the most modern structures (post-1995) performed acceptably. A further observation is that buildings which were well conceived, well designed, well detailed, and then well-constructed performed well, irrespective of their age.

However, some types of structures were found to perform poorly and some details were found to be grossly inadequate.

The original purpose of this document was to mitigate concerns that engineers may have been reusing structural forms or details that are inappropriate in the context of lessons learned from the 2010/2011 Canterbury earthquakes. In the current (2022) context, content retained in this revised version remains a useful source of general ‘best practice’ design guidance.

1.1 Scope

The scope of this document is generally limited to commercial building structures constructed of conventional materials, and of conventional form. It excludes buildings utilising energy dissipation or damage resistant design methods. It is noted however that aspects of this guidance may be applicable to those buildings, so designers are advised to review this guidance before undertaking design of such structures.

In general, the Building Act definition of non-residential structures is applicable, namely, all buildings except those:

“...used wholly or mainly for residential purposes unless the building:-

comprises 2 or more storeys; and

contains 3 or more household units.”

Although this document is not generally applicable to residential structures, there are sections that make reference to residential buildings, particularly with reference to slabs on grade. Designers of residential structures are referred to the DBH guidance document prepared by the Engineering Advisory Group [1]. Civil structures are excluded also, though some of the concepts discussed may be applicable.

1.2 Use of this Document

Recommendations are made throughout this document, at three different levels:

Verification Method requirement:	These are references to sections of the Verification Methods, to either emphasise or clarify the meaning of a particular clause.
SESOC Recommendation:	These are recommendations by SESOC for 'best practice' design or detailing of structures. In some instances these reiterate the requirements of Standards, and in other instances may suggest a more stringent requirement.
Damage Reduction Recommendation:	These are recommendations by SESOC for design or detailing improvements that will provide significant improvement in performance, in some cases, for little extra cost.

1.3 Acknowledgements

This document was prepared initially by Holmes, and then offered to SESOC for adaptation. SESOC gratefully acknowledges Tonkin & Taylor's contribution to Section 10. This document has subsequently been peer review by:

- SESOC
- NZ Society for Earthquake Engineering
- NZ Geotechnical Society

At the request of the Royal Commission, a previous version was also peer reviewed internationally.

1.4 Limitation

This interim design guidance has been prepared by SESOC for general distribution, for the guidance and assistance of structural engineers, although the observations herein are equally applicable to the whole country. Engineers using this information are not relieved of the obligation to consider any matter to which the information may relate.

Neither SESOC nor NZSEE and NZGS accept any liability for the application of this guidance in any specific instance.

This note has been prepared using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this note.

2 LOADINGS AND DESIGN PHILOSOPHY

New buildings in New Zealand are designed to conform to the compliance documents of the New Zealand Building Code, notably B1 [2]. The NZBC in turn sits beneath the Building Act [3]. B1 cites a number of documents as verification methods or acceptable solutions, commencing with the design actions standard, AS/NZS1170 [4]. The specific performance objectives are currently set in the design actions standard, which the materials standards are then intended to meet. The main material standards referred to in this document are the Concrete Structures Standard, NZS 3101 [5], the Steel Structures Standard, NZS 3404 [6], and the Composite Structures Standard, AS/NZS 2327 [7]. Note that the appropriate revisions must be used in each case.

The cited Standards together comprise the verification method VM1, which is a deemed-to-comply path to provide compliance with the Building Code. Designers may elect to follow the alternative solution path, using other means (such as industry guidelines, first-principle engineering, offshore or un-cited Standards; collectively ‘standards with a small s’). It should be noted by all designers that compliance with the Building Code is the minimum standard that must be achieved. There is nothing preventing designers (with their clients’ knowledge) providing a greater level of protection to buildings.

2.1 Seismic Design Actions

Amendment 1 to NZS 1170.5:2004 was published during 2016 [8], and contained a number of important updates to the Standard including:

- Improved categorisation of soils between types C and D (clause 3.1.2)
- Improved criteria for vertical earthquake actions (clause 3.2)
- Improved criteria for diaphragm analysis (clause 5.7)
- Changes to the inter-storey deflection used to calculate P-delta demands (clause 7.3.1), and
- Changes to requirements for parts and components (Section 8).

Notwithstanding these changes, Amendment 1 has not been cited by MBIE in Verification Method B1/VM1, and SESOC understand it is probable that Amendment 1 will never be cited. This is possibly due in part to a number of known problems with the published amendment. Notably, these include:

- Poorly defined changes to clause 6.1.1, which, if strictly interpreted, prohibit equivalent static and response spectrum analysis for many structures where such analysis should be considered appropriate,
- Requirements (in clause 4.5.3) that are intended to prevent 'ratchetting' of structures, but which are incorrect and inconsistent between the Standard and commentary.

Additional comments regarding provisions for diaphragm analysis can be found in Section 6.

It was anticipated that these issues and other updates would be encompassed in a planned Amendment 2 to NZS 1170.5 that was initiated following the 2016 Kaikoura earthquake, and that this second amendment would eventually be cited. However, this further amendment has been put on hold. Changes to B1/VM1 are now anticipated to occur over the next 3 years as elaborated on below.

Engineers are advised to adopt those parts of Amendment 1 that are known to improve the state of knowledge, notwithstanding that this results in a design being an Alternative Solution.

Additionally, it is emphasised that NZS 1170.5:2004 as cited is now an old Standard, and outdated in key respects including the underpinning seismic hazard model, the approach taken to forward directivity and ‘near fault’ effects, and the provisions provided for scaling of ground motion records for time history analysis.

Extensive work is currently ongoing that will result in production of a significant revision of the National Seismic Hazard Model (NSHM) [9]. The new NSHM was released on 4th October 2022. For the first time, the NSHM explicitly includes the uncertainty in the hazard estimates as an output.

It is expected that the results of the NSHM will be incorporated into Verification Method B1/VM1 in two stages. MBIE is planning to consult on the first stage in mid-2023. This initial review will consider how to incorporate the new hazard information into the current design approach. The timing for this consultation will depend on how long it takes to develop technically robust proposals, and MBIE intends to confirm timeframes in the coming months. The second and more substantial update is expected to follow in 2025 or 2026.

These Building Code updates will involve more than simply citing the technical outputs of the updated NSHM. In addition to seismic hazard, policy and risk settings are likely to be reviewed along with design practices. The Seismic Risk Working Group (SRWG) report of November 2020 [10] made a number of recommendations on these topics—and a further Seismic Risk Work Programme (SRWP) has commenced to lead the development of draft Building Code revisions.

Consequently, between now and 2026, there will be considerable uncertainty in the design community regarding how to approach seismic design of new buildings and seismic assessment/retrofit of existing buildings. SESOC, NZGS and NZSEE have jointly considered measures to mitigate this uncertainty, with the resulting recommendations contained in a guidance document titled *Earthquake Design for Uncertainty* [11]. SESOC recommend adoption of the content of that document.

SESOC Recommendation:	Those parts of NZS 1170.5 Amendment 1 that are known to improve the state of knowledge should be used. The recommendations detailed in the ‘Earthquake Design for Uncertainty document [11] be adopted.

2.2 Design Approach

One of the main cornerstones of structural design for earthquakes in New Zealand is capacity design. Arguably this design method was developed in New Zealand and our standards have embraced it since the mid-70s. Although there have been failures noted in buildings designed using capacity design, it is suggested that the failure is not with the capacity design philosophy, but with the structural systems or detailing.

Experience from past earthquakes has demonstrated that when capacity design principals have been followed, buildings are able to perform reliably even when subjected to shaking levels significantly greater than anticipated.

However, there is concern that buildings designed to be nominally ductile ($\mu = 1.25$) or for elastic response ($\mu = 1.0$) may not provide adequate resilience, particularly against shaking of significantly greater intensity than the design level. In the case of elastic response, this is compensated for at least in part by adopting $S_p = 1$, but it is debateable whether $S_p < 1.0$ should be used where no capacity design has been completed.

SESOC Recommendation:	Wherever practical, structures should be designed using a capacity design approach, regardless of the design seismic load level adopted. Where capacity design is not used OR sufficient resilience cannot be demonstrated, designers should adopt $S_p = 1.0$.

In the review of building damage in the earthquakes, it is noted that although most buildings have achieved the primary objective of protecting lives, levels of damage were often high. While this can be explained in many instances by the intense shaking experienced this was not always the case. It is recommended engineers discuss seismic performance criteria with their clients including consideration of the following for those projects where a greater level of seismic resilience is sought;

- Increasing the return period of the earthquake used for SLS1 from 25 years to 50 years. This is consistent with that used for new building design in Japan and performance-based design in the United States. It delays the onset of damage in smaller events and increases the probability that a building will be in a repairable state following a damaging event beyond SLS1 [12,13].
- Introduction of an intermediate "Damage Control Limit State" (DCLS) [14,15] (which may correlate to SLS2) between SLS1 and ULS. Objectives of the DCLS include limiting the extent of building damage to a level which is economically repairable and will enable the building to be reoccupied within a reasonable timeframe (to be agreed with the client).

Unless otherwise agreed with the client it is recommended design actions associated with a 250-year return period earthquake are used for the DCLS. For those cases when the DCLS is adopted for a project it is recommended the structural ductility factor, μ, used for the design of the primary structure should not exceed 2 for this limit state.

All structural elements expected to require inspection following an earthquake should be accessible without damaging elements that have a design life greater than 5 years (as defined in NZBC B2), and without removing items that would result in the need to shut down the building.

Furthermore, it is recommended that residual interstorey drifts should be limited to 0.5% to ensure building repair is possible [16]. This drift limit is consistent with work completed by McCormick et al. [17] and has been derived with consideration given to both human comfort, building functionality and repairability.

Buildings with residual drifts exceeding 0.5% may not be acceptable to occupy following an earthquake. A method to calculate residual drifts can be found in Section 5.4 of FEMA P-58 [16]. Damage to secondary and non-structural elements necessary for the building to be reoccupied should be controlled so as not to prevent reoccupation within prescribed timeframes (refer also Section 11).

The DCLS, if adopted, is not a Building Consent requirement, so this would be a matter of agreement between client and engineer only.

In general, unless a building contains highly sensitive or specialised equipment, stiffer buildings are likely to suffer less damage at lower levels of shaking. Restraint of plant and equipment can generally be achieved satisfactorily in stiff buildings. If a building contains high value or critical contents, consideration could be given to using other methods of protection such as base isolation.

Damage Reduction Recommendation:

Increase the return period of the earthquake used for SLS1 from 25 years to 50 years.

Damage in buildings be limited to economically repairable levels by introducing a DCLS. For the DCLS limit $\mu \leq 2.0$ and residual drifts $\leq 0.5\%$.

All structural elements that may require inspection following an earthquake should be accessible.

Damage to secondary and non-structural elements necessary for the reoccupation of the building to be controlled so as not to prevent reoccupation within prescribed timeframes.

2.3 Importance Level 4 Buildings

Buildings with special post-disaster functions are defined as Importance Level 4 (IL 4) buildings in AS/NZS 1170.0 [ref]. When designing IL 4 buildings AS/NZS 1170.0 requires consideration of the SLS2 performance limit state in addition to the requirements of SLS1 and ULS. An objective of SLS2 is the building maintains operational continuity after the SLS2 earthquake.

This guideline does not provide specific consideration of SLS2 requirements, however some of recommendations provided in relation to the DCLS may also be applicable to SLS2. Refer to AS/NZS 1170.0 and NZS 1170.5 for further guidance.

2.4 Margin Beyond ULS

The NZBC expectations for building performance are stated in NZS1170.5 [18]. The commentary notes performance expectations as:

Frequently occurring earthquake shaking can be resisted with a low probability of damage sufficient to prevent the building from being used as originally intended; and

The fatality risk is at an acceptable level.

It is further stated that buildings designed to the relevant materials Standards should have an acceptable margin against collapse in the event of earthquake shaking greater than the ULS design load. The commentary suggests the margin to be “at least 1.5 to 1.8” times the ULS level. This may be referred to as resilience. This requirement is generally satisfied by the materials codes, where the additional requirements of the seismic design procedures incorporate implicit ‘deemed to satisfy’ provisions around these higher levels of demand.

It has been mooted in some quarters that the NZBC should be revised to include reference to the Maximum Considered Earthquake (MCE). This is not currently explicitly referenced either as a load or performance objective. Furthermore, because it has been linked (artificially or otherwise) to the 2,500 year earthquake, regardless of building importance level, it could have implications for the design of IL3 or IL4 buildings, although there is no rational reason why those buildings should require less resilience than IL2 buildings. Because of this, the MCE is not referenced elsewhere in this document.

It is considered that, for the design of new structures, the margin of 1.5 to 1.8 as referred to in NZS1170.5 should be acceptable, although it is noted that deflections should not be reduced by the S_p factor (which otherwise reduces from peak drifts to average drifts) for elements that may be considered life safety hazards and which exhibit step changes in behaviour, such as floor seatings, seismic gaps and stairs.

In the case of buildings designed to IL3 or IL4, the margin of 1.5 to 1.8 should be relative to ULS design actions determined from use of R=1.3 or 1.8 respectively, recognising that the reasons for designing these buildings to a higher standard is to provide increased levels of resilience to key structures, or those that contain greater numbers of occupants. Although it is possible that the earthquake shaking resulting from distant faults may be unaffected by the increased local seismicity, it is felt that the increased resilience required of these buildings still warrants the same margins being maintained over the performance of IL2 buildings.

SESOC Recommendation:	For IL3 or IL4 buildings, the margin of 1.5 to 1.8 relative to ULS design actions is to be maintained.

2.5 Building Configuration and Redundancy

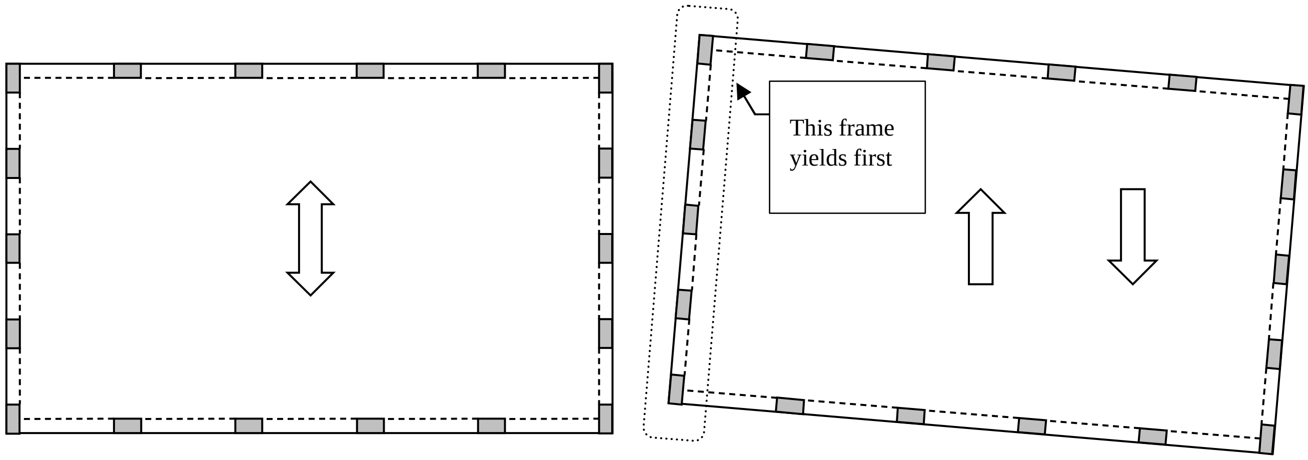
Observation shows that in general, regular buildings have behaved significantly better than irregular buildings. Well-proportioned regular buildings typically have a higher capability to withstand shaking levels even when they are significantly greater than anticipated. However, there is significant research and consideration required to establish a means of determining firstly the appropriate regularity provisions and secondly, the appropriate multipliers on loading, beyond what is currently in the Loadings Standard.

Building systems which have one face essentially open have been vulnerable to increased deflections on the open face, resulting in poor cladding performance. In such cases, attention is drawn to the existing regularity provisions, noting that the seismic displacements should be calculated at the worst location, typically on the line of the open face.

SESOC Recommendation: The combined primary structural gravity and lateral systems are proportioned with enough regularity that it is possible to identify a clear plastic mechanism for the structure as a whole, and also so that it avoids the application of provisions in NZS 1170.5 Amendment 1 related to unbalanced strength, ratcheting and inelastic unrestrained torsion.

Another observed issue is the inability of some regular orthogonal systems to resist torsional response in the case where an accidental eccentricity has developed. This may arise where one frame or wall in the stiffer direction inevitably hinges before the other, and significantly reduces in stiffness. In such cases, if the more flexible system does not have sufficient stiffness and strength to force the other frame or wall to hinge, it is possible that the building may develop an undesirable failure mode.

This is illustrated in Figure 1 below. The frames in the direction of loading are significantly stiffer than the orthogonal frames, therefore providing most of the resistance to torsion (noting that as this is a regular building, only accidental eccentricities are significant). Following the yield of the first frame, its stiffness drops considerably, shifting the centre of rigidity. The orthogonal frames may not have sufficient stiffness to force hinging of the other frame, resulting in increased drifts at the yielded frame. (Based on an observed example).



Before: Cr of Mass and Cr of Rigidity coincident

Post-yield: Cr of Mass and Cr of Rigidity separate, increasing deflection at end frame

Figure 1 Building Plan indicating possible torsional mode development

This may happen in the case of perimeter frames where there are only two main frame lines in the direction which contributes most of the torsional rigidity and where there is significant difference between elastic and post-yield stiffness of the frames and where rigid floor diaphragms exist. If the frames or walls in the opposite direction cannot contribute more than say 30% of the torsional rigidity, a third frame or wall of similar stiffness should be introduced into the stiffer direction. The three (or more) lateral load resisting elements in the stiffer direction should then be distributed approximately evenly over the length of the building, and should be given approximately equal stiffness and strength.

This could also be considered a redundancy provision.

SESOC Recommendation:	In all buildings with rigid diaphragms, each orthogonal direction should have a system capable of resisting torsion. Where this cannot be achieved, special study may be required to demonstrate that the building is able to resist torsional actions after a full mechanism has developed. Where the lateral force resisting system in one direction of a two-way frame structure contributes more than 70% of the resistance to torsion, and when there is a significant stiffness reduction as a result of yielding, the frame stiffness should be adjusted such that each contributes more evenly, or a third frame line should be introduced in the stiffer direction. The strength and stiffness required in that direction should be spread approximately evenly between the frames. The three (or more) frames should be spread approximately evenly over the length.

2.6 Compatibility Effects in Gravity Structure

Although gravity frames may not form part of the lateral load resisting system of a building, they are nevertheless expected to deform along with the primary system. All gravity structure should be detailed to accommodate the expected displacement demand from earthquakes greater than the ULS event. This may be assumed when detailing in accordance with the material Standards, but may require specific attention when designing novel structural systems that are not covered by New Zealand Standards.

Verification Method requirement:	Gravity structures should be detailed to accommodate the expected displacement demand from earthquakes. For designs not covered by B1/VM1 consideration shall be given to earthquakes greater than the ULS event.

2.7 Acceptance of Proprietary Systems

A number of manufacturers offer proprietary solutions, from simple details through to complete structural systems. Use of these systems may be promoted by owners, developers or contractors, but the final responsibility for their use remains with the building designer. Therefore it is the building designer's responsibility to verify that a proprietary item is suitable for use. It must be compatible with the overall structural performance expected, from both a strength and displacement perspective. Ultimately, the building must comply with the NZ Building Code, and the interaction of the proprietary elements with the rest of the structure can only be checked by the building designer. Therefore overall responsibility must rest with the designer.

Manufacturers' or distributors' claims for their products must be considered carefully. If a product has a New Zealand based accreditation, it should only be used within the limitations of that accreditation. Where a product carries certification from other sources, it needs to be more carefully considered. Even products that may have been in use within the industry for a long time may not be suitable for use in all locations.

Designers' attention is drawn to MBIE guidance regarding the Product Assurance Framework to Support Building Code Compliance[19], available at their website.

A significant concern with seismic performance is with the ability of elements to withstand the effects of inelastic drift associated with the development of ductility and from events greater than the design earthquake, as discussed in Resilience above. While the detailing requirements of the materials standards are deemed to provide the additional capacity required to meet these demands, proprietary items may not have had sufficient testing to achieve this.

SESOC Recommendation:	Proprietary systems must only be used in situations where there will be no inelastic demand on the system, unless the whole system has been designed or tested to $1.5/S_p$ times the inelastic drift demand imposed by its use and configuration within the structure, and should not exhibit sudden brittle or unpredictable behaviour immediately beyond this limit.
Verification Method Requirement:	Where proprietary systems have been accepted on the basis of a recognised New Zealand appraisal in accordance with the MBIE Guidelines, they should only be used strictly in accordance with the limitations of the appraisal.

2.8 Independent Peer Review

An independent peer review of the structural design is recommended for complex or unusual structures, or for those projects where a high level of structural performance is being sought. The peer reviewer should be independent and have the necessary technical competence to complete the review. Engineering NZ Practice Note 2 [20] provides guidance for engineers carrying out peer reviews.

SESOC Recommendation:	Independent peer review of the structural design is recommended for complex or unusual structures, or for those projects where a high level of structural performance is being sought.

3 ANALYSIS

Seismic analysis has often been regarded as secondary in importance to the actual design. This comment, while valid, ignores a significant fact – that assumptions that are made in analysis may have a profound impact on the design and hence must be validated through the design. Whether analysis is completed by hand using equivalent static analysis (appropriate for small simple structures), or using advanced computer analysis, it is important that designers do not lose sight of the building that they are analysing, for the sake of the analysis.

A common trap is to assume that something is valid ‘because the computer says so’. But the reality is that any analysis is only as good as the input. All computer analysis should be accompanied by sufficient reality checks that a designer can be satisfied that the virtual building that was analysed is indeed the same as the one that gets built.

3.1 Boundary Conditions and Assumptions

Assumptions made in analysis must be verified in the final design. This is emphasised in the CERC report (CERC R1.55). In particular, this applies to foundation flexibility and its impact on the super-structure (CERC R1.12, R1.13). Although there is no specific design guidance currently available for when foundation deformation may impact on the superstructure, it is recommended that consideration is given to this when analysis models are being prepared.

In particular:

- The impact of potential rocking should be considered, noting that NZS 1170.5 (cl 6.6) requires special study for such structures.
- Where yielding of foundations may occur, the foundation system should be explicitly modelled, with due allowance for cracking. Where appropriate, dummy storeys should be used to model the flexibility of the foundation system, using properties supplied by the geotechnical engineer.
- Where foundation flexibility is explicitly modelled, allowance should be made for the variability of soil properties [21]. Further guidance on this issue is provided in Section 0.

SESOC Recommendation:	The possible impact of foundation deformations should be considered in the seismic analysis (CERC R1.12). Foundation deformations should be assessed for the ULS load cases and overstrength actions, not just foundation strength (capacity). Deformations should not add unduly to the ductility demand of the structure or prevent the intended structural response.

3.2 Vertical Accelerations

NZS1170.5 includes a section giving the derivation of vertical seismic loads (clause 3.2, 5.4). In general, vertical actions may be shown to be non-critical compared to gravity actions, but designers should identify and address specific elements that may be vulnerable to vertical actions. Such elements may include cantilevered elements or elements with low live load, where the combination of dead load plus vertical load may exceed the factored gravity load combination. In particular, designers should consider the load case of self weight only with earthquake acting upwards, for elements such as cantilevered slabs that may require reinforcement on both faces in order to resist upwards accelerations.

A special case for consideration is that of transfer elements. Typically, earthquake induced bending and shear actions due to lateral loading may be limited by capacity design, but the effects of vertical accelerations may not be limited in that way, and so could increase for larger earthquakes. It is recommended in such cases that the effect of vertical load is included with a multiplier of 1.5 to allow for this effect

SESOC Recommendation:	Elements such as transfer structures where vertical seismic accelerations could add significantly to the design actions should have a multiplier of 1.5 to be applied to the vertical component of seismic loading, to allow for larger earthquakes.

3.3 Ratcheting Actions

Some structures may develop mechanisms that result in the formation of one-way hinges. This type of structure may progressively deflect in one direction, resulting in a p-delta effect due to the increasing displacement.

As noted in the CERC report Part One volume 2, *"potential problems may arise from ratcheting in structures where:*

- *gravity loads are resisted by cantilever action;*
- *structures or structural elements have different lateral strengths in the forward and backward directions; or*
- *transfer structures are incorporated in buildings."*

Designers should identify structures where this mode of behaviour may develop. Where ratcheting actions may result, either the structure should be balanced to reduce the impact of ratcheting, or allowance should be made for the added actions resulting from the ratcheting.

The need to consider ratchetting effects has now been included in Amendment 1 to NZS 1170.5, but this amendment has not yet been cited as part of Verification Method B1/VM1. Also as noted in Section 2.1 above there are a number of errors in the provisions included that may make their application difficult. Engineers are recommended to consider a recent paper based on University of Canterbury research addressing this topic which provides clarification and revision to these provisions [22].

SESOC Recommendation:	Buildings should be configured to minimise the potential for ratcheting. When ratcheting cannot be avoided appropriate allowances should be made where this action may occur. (CERC R2.56). Ratchetting should be considered using methods contained in Amendment 1 to NZS 1170.5 or other accepted method. Reference to paper by Saif et al. [22] is recommended.

3.4 Second Order Actions

Designers should be aware of second order actions that are not able to be modelled using conventional analysis. These actions include:

- P-delta effects. NZS 1170.5 clause 6.5 contains provisions that require the explicit consideration of P-delta.
- Elongation from the deformation of plastic hinge regions in beams, columns and structural walls. The impacts of member elongation need to be considered for:
  - Detailing supports of stairs and ramps.
  - Support and fixing of cladding panels and precast floor units.
  - Beams and ground lower-level columns in moment resisting frames.
  - Coupling beams and lower-level wall piers in coupled wall structures.
  - Geometric elongation associated with rocking foundations.

Verification Method Requirement:	Second order actions should be considered in accordance with relevant Verification Method provisions.

Buildings that are susceptible to P-delta effects will be prone to disproportionate displacement amplification if they are subjected to shaking that is more intense than anticipated. This particularly relates to systems with relatively low strain hardening and high stability coefficients [23]. It is recommended the stability coefficient as defined in clause 6.5.2 of NZS 1170.5 be limited to not more than 0.2 to provide increased resilience.

SESOC Recommendation:	Buildings be proportioned so the NZS 1170.5 stability coefficient is not greater than 0.2

4 REINFORCED CONCRETE

4.1 General

4.1.1 Strain Ageing of Reinforcement

Reinforcement used in potential plastic hinge regions and 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. 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.

Testing has confirmed that low carbon AS/NZS 4671 G300E [24] reinforcement is susceptible to strain-ageing [25,26] and therefore should not be used in potential plastic hinge zones. Micro alloyed high strength steel such as AS/NZS 4671 Grade 500E reinforcement is not affected by strain ageing and can therefore be used.

SESOC Recommendation:	AS/NZS 4671 Grade 500E reinforcement is used in potential plastic hinge regions because it is not susceptible to strain ageing effects.

4.1.2 Detailing for Resilience

Testing [27] has shown that moderately damaged reinforced elements can be repaired by means of epoxy injection and replacement of damaged concrete provided bar buckling does not occur. This is because bar buckling accelerates fatigue damage accumulation in longitudinal reinforcing bars [28,29].

Bar buckling in potential plastic hinge regions can be delayed, and the reparability of reinforced concrete buildings improved, if the centre-to-centre spacing of stirrup-ties in potential plastic hinge regions is reduced to four times the diameter of the longitudinal bar to be restrained. Furthermore, it is recommended that ductility demands on the seismic resisting system should be limited for the DCLS so $\mu \leq 2.0$.

Damage Reduction Recommendation:	For the DCLS limit so $\mu \leq 2.0$. Centre-to-centre spacing of stirrup-ties in potential plastic hinge regions should not exceed four times the diameter of any longitudinal bar to be restrained.

4.1.3 Shallow Embedded Anchors

Failures of precast panel connections with shallow embedded anchors were observed in the 2010/2011 Canterbury earthquakes.

Shallow embedded anchors (with and without tie bars) were observed to pull out of the face of precast panels when overloaded. When provided with cast-in inserts tie bars, the tie bars were not of sufficient diameter or length to provide effective restraint of the anchor.

Recent testing of a web cleat detail with shallow embedded anchors, commonly used to provide a shear connection between steel beams and supporting concrete elements, has identified a potential brittle failure mode that designers need to consider [30]. When the connection is subject to significant in-plane rotations there is the potential for significant moment demands to be developed within the anchor group. This can result in a brittle concrete breakout failure of the shallow embedded anchors unless the connections is detailed to mitigate this failure mode. Options to mitigate this include:

- Provide horizontally slotted holes in the web cleat to accommodate 1.5 times the ULS movement of the bolt group in accordance NZS 3101:2006 A3 clause 17.6.2 (refer Figure 2). The gap between the end of the beam and the weld plate should also be sized to accommodate the same movement. When a concrete slab is present it is recommended the length of the slotted holes be determined assuming the joint is rotating about the top of the concrete slab. To limit the about of bearing deformation in the cleat perpendicular to the long slotted holes, the bearing capacity of the ply should be limited to $V_b = 2.1 d_f t_p f_{up}$ [31].

- Apply capacity design principles in accordance with NZS 3101:2006 A3 clause 17.6.3 so that the capacity of the anchor group exceeds the overstrength yielding of the attachment.

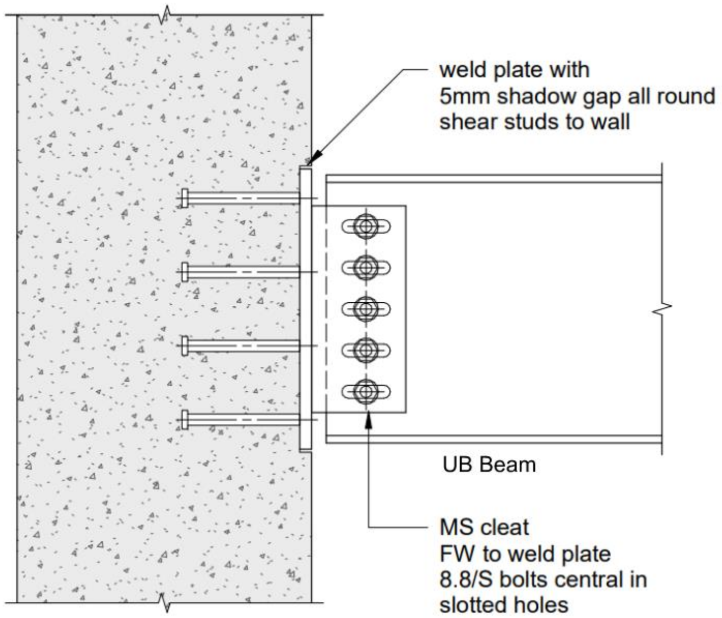


Figure 2 Recommended slotted cleat detail to accommodate steel beam support rotations.

Shallow embedded anchors should be designed in accordance with Section 17 of NZS 3101:2006 A3 including consideration of group and edge effects, and the additional seismic requirements detailed in NZS 3101 clause 17.6. Anchoring shallow embedded anchors in potential plastic hinge zones is not recommended. As noted in NZS 3101 A3 clause C17.6.6, when shallow embedded anchors are anchored in potential plastic hinge zones the cover concrete should be ignored and the concrete failure cone should be modified to account for the presence of flexural cracks.

For those cases when edge effects control the design of shallow embedded anchors clause 17.5 of ACI 318:2019 [32] provides guidance on the design and detailing of supplemental reinforcing to control concrete side face blow-out.

Verification Method Requirement:	Apply the seismic provisions of NZS 3101:2006 A3 clause 17.6 when designing shallow embedded anchors.
SESOC Recommendation:	Shallow embedded anchors should not be used in potential plastic hinge zones.

4.2 Structural Walls

Given the desire to design stiffer buildings to mitigate damage to drift sensitive building components it is likely that wall structures will be popular. However, the performance of wall structures in the 2010/2011 Canterbury Earthquakes was not as good as expected and we need to improve the future performance of these structures.

4.2.1 Effective Section Properties

Computed periods of vibration and lateral deflections of buildings with reinforced concrete structural walls are known to be particularly sensitive to the effects of cracking on member stiffness.

NZS 3101 clause C6.9.1 recommends that unless flexural cracking is likely to occur in the storey being considered, either gross section properties, or transformed section properties, should be used. If the effective section properties detailed in NZS 3101 Table C6.5 are used for structural walls these should be applied to the full wall height. This is because the section properties in Table C6.5 for walls have been calibrated so that when applied to the entire height of the wall the correct yield displacement can be calculated.

Clause 6.9.1.1 of NZS 3101 states "*Where analysis indicates tensile stresses due to flexure and axial load are less than* $0.55\sqrt{1.2f_c'}$ *in the ultimate limit state, flexural cracking is unlikely to occur and the section properties shall be based on gross or uncracked transformed sections properties*". This statement is open to misinterpretation, with some having taken it to mean that the likelihood of cracking should be checked at all positions within an element [e.g. 33]. However, it is now evident that the determination should be based solely on the tensile stress at the critical section of an element.

When determining interstorey drifts for DCLS load levels, the effective flexural stiffness of reinforced concrete walls (EI_{eff}) should be determined using the following moment curvature relationship [12]:

$$EI_{eff} = \frac{M_n}{\emptyset_y}$$

Where M_n is the nominal flexural strength of the wall and ϕ_y is yield curvature of the wall.

SESOC Reduction Recommendation:	When the effective section properties in NZS 3101 Table C6.5 are used for structural walls these should be applied over the full wall height.
Damage Reduction Recommendation:	Effective section properties of walls should be calculated when determining interstorey drifts for the DCLS.

4.2.2 Wall Thicknesses

Minimum wall thicknesses may be determined by a number of constraints – both with respect to the performance of the wall itself and to the connection of adjoining elements.

One constraint on minimum wall thickness is defined by the diameter of the reinforcing bars used. NZS3101:2006 (clause 11.3.12.2) defines the maximum bar diameter as $t_w/7$. This is further reduced to $t_w/10$ or $t_w/8$ for ductile and limited ductile regions respectively (clause 11.4.5).

Development of hooked starter bars into thin wall panels may also effectively restrict the minimum thickness of a wall, or alternatively, the wall thickness may limit the size of bar which may be anchored into the wall. For example, D10 bars have a hook development length of 90mm, while D12 bars have a development length of 110mm. Grade 500 reinforcing has larger development lengths again. It is generally recommended that Grade 300 reinforcement is used in such situations, due to its greater tolerance for potential bending and rebending.

Detailing of precast panel connections also has implications with respect to the minimum wall thicknesses achievable. Precast panel detailing is addressed in Section 4.5 below.

Verification Method Requirement:	Wall thicknesses should be at least 7 d_b, increasing to 10 d_b in yielding regions.
SESOC Recommendation:	Minimum structural wall thicknesses to accommodate reinforcement anchored into the wall should be used as shown in Table 1. Where possible, use of Grade 300 reinforcement is recommended.

Table 1 Minimum Structural Wall Thicknesses

Construction	Floor starters	Thickness
Any[1]	D10	150mm
Any[1]	D12	175mm
Any[1]	XD10	200mm
Any[1]	XD12	250mm
Precast[2]	-	200mm[3]

Notes:
1. Wall thickness limited by development of hooked floor starters (assumes *f'c* > 30 MPa, cover > 40mm, no more than 300mm concrete cast below the floor starters).
2. Wall thickness may also be limited by precast panel splices – refer to Section 4.2.6 below.
3. Unless greater wall thickness required for development of floor starters.

4.2.3 Minimum Reinforcement

Minimum reinforcement contents are required to ensure that well distributed cracks are formed in the concrete before the reinforcing steel yields in tension. Once a section of reinforcing steel yields it strain hardens, thereby forcing the lower strength reinforcing steel to yield at the next crack and so on. 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, if the effective concrete tensile strength is greater than that of the reinforcing steel, a single crack may form with all of the deformation concentrated at this location. The resulting strains imposed on the short section of reinforcing steel crossing this crack will cause fracture of the reinforcing steel at very low plastic deformations of the wall (refer to Figure 3).

Figure 3 Fractured bars in lightly reinforced wall

Minimum reinforcing steel contents for walls are specified in NZS 3101:2006 A3 as a function of the specified 28-day concrete strength. While lower bound 28-day concrete strengths are used for design, the strength of the concrete supplied for a project will be higher due to concrete production requirements. Higher target strengths are used for concrete production to ensure not more than 5% of representative samples fall below the specified 28-day concrete strength. Further strength enhancement will also occur due to age hardening and dynamic loading rates. Except as noted below for non-planar walls, the minimum reinforcing steel contents specified in NZS 3101:2006 A3 include consideration of these effects.

Designers should be aware that not all Ready-Mixed Concrete plants are certified to supply all the standard concrete strength grades. In some cases, a higher than specified grade may be supplied as a substitute for a project. When this occurs the implications on minimum reinforcing contents should be addressed.

In addition, precasters will often use higher concrete strengths in order to facilitate early lifting of units. Recognising that early lifting may be essential to maintain programmes, it is recommended that designers discuss precasting and lifting requirements as early as possible and if necessary, adjust reinforcement to suit the higher concrete strength. Alternatively, the pouring and lifting sequences may need to be reviewed.

Self compacting concrete (SCC) has inherently high strength (typically >70MPa), which will require large reinforcing contents. Designers need to be aware of this, and may need to avoid use of SCC for this reason.

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 strength (M_n) for planar walls. However, this will not necessarily be the case for non-planar (i.e. flanged) walls [33]. This is particularly likely to be a concern for nominally ductile walls with low reinforcement ratios.

It is recommended that designers ensure that the ratio $M_n/M_{cr} \geq 1.2$ to avoid concentration of plastic deformation at a single crack. For this purpose M_{cr} should be determined based on the average concrete tensile strength outlined in commentary clause C5.2.4 of NZS 3101, i.e. $f_t' = 1.43 \times 0.38\lambda\sqrt{f_c'} = 0.54\lambda\sqrt{f_c'}$.

SESOC Recommendation:	Engineers should ensure that the ratio $M_n/M_{cr} \geq 1.2$ for non-planar walls, with the cracking moment M_{cr} determined based on the average concrete tensile strength outlined in commentary clause C5.2.4 of NZS 3101, i.e. $f_t' = 0.54\lambda\sqrt{f_c'}$.

4.2.4 Distribution of Reinforcing Steel

For simplicity of construction, wall reinforcing steel is typically spread evenly along a wall. While this is rational and may perform adequately for a long, squat wall dominated by shear, for walls dominated by flexure the reinforcing steel will perform better when lumped at the ends.

The bars at the extreme fibre of a wall section undergo massive strains in order to develop the full nominal moment capacity of the section. If the reinforcing content is insufficient to force the development of distributed cracks up the extreme fibre of the wall, large isolated cracks may develop resulting in the fracture of the bars at the end of the wall (as seen in several buildings in Christchurch following the 2010/2011 Canterbury earthquakes) and the subsequent significant loss of flexural capacity.

By lumping steel at the ends of the wall, the reinforcing content in the end region containing high tensile strains is much higher. This in turn will force multiple cracks to develop, resulting in lower strain demands on the reinforcing. These walls will exhibit significantly higher ductility, although the over-strength capacity of the wall section is likely to be higher than for a wall with distributed reinforcing (where minimum steel governs along the wall).

In any case, the distribution of reinforcement in a wall must take into account the foundation conditions. For example, if a wall structure is founded on piles, the foundation beams under the wall must be capable of transferring the tensile loads from the intermediate reinforcing steel to the pile caps. This may be another point in favour of using lumped reinforcement.

SESOC Recommendation:	Reinforcing should be lumped at the ends of a wall, with minimum reinforcing distributed along the web.

4.2.5 Singly Reinforced Structural Walls

The stability of singly reinforced structural walls under earthquake loads is uncertain. Measures to reflect the uncertain performance of singly reinforced structural walls were introduced in Amendment 3 to NZS 3101:2006. In particular these limit the permissible ductility of singly reinforced structural walls, and require use of a low strength reduction factor. As noted in NZS3101:2006 A3 clause C2.3.2.2 concerns exist with the limited flexural resistance available to resist out-of-plane loads and the low buckling resistance of both the wall and reinforcement should flexural yielding occur.

Engineers should be cognisant of further concerns regarding the behaviour of singly reinforced structural walls, and apparent incompatibility between their behaviour and the structural mechanics routinely assumed in their design [34]. This is of particular concern when singly reinforced walls are subjected to biaxial loading and it is uncertain if this issue is adequately addressed in NZS3101:2006 A3.

Verification Method Requirement:	Apply the provisions of NZS 3101:2006 Amendment 3 when designing singly reinforced walls.
SESOC Recommendation:	Singly reinforced structural walls should not be used for primary structural load paths unless it can be shown all wall reinforcement remains elastic for 1.5 times ULS earthquake loading with S_p = 1.0. Effects of bidirectional earthquake loading should be considered. Engineers should be aware of further concerns regarding the behaviour and analysis of singly reinforced structural walls [34].

4.2.6 Wall Base Connections

Many connection details used historically at the base of (particularly singly reinforced, tilt-up walls) are now understood to perform poorly. This includes cast-in-inserts which have been observed to pull out of the face of precast panels. Engineers are advised to consider notes from a recent seminar that summarises current good practice regarding wall base connection detailing [35] along with a related paper [36].

4.2.7 Precast Panel Splices

Failures of poorly detailed precast panel splices were observed in the 2010/2011 Canterbury earthquakes. This highlighted the need for precast panel splices to be appropriately detailed to ensure they perform adequately when subject to seismic demands. Further guidance on how to design and detail grouted precast panel splices is provided in the SESOC Precast Concrete Grouted Connections and Drossbachs Guidelines [37–41].

4.3 Concrete Moment Resisting Frames

Generally concrete moment resisting frames performed as expected in the 2010/2011 Canterbury earthquakes. Capacity design principles appeared to work well, with damage concentrated in the beam hinges as expected. However, frames designed for high ductility suffered significant (and sometimes irreparable) damage, as well as causing significant damage to floor systems as a result of frame elongation. Concern has been expressed about the possible outcomes if the duration of shaking had been considerably longer.

Conventional ductile concrete moment resisting frames are not low damage systems but can still be designed to comply with the life safety provisions of the Building Act.

4.3.1 Detailing for Resilience

Regardless of the ductility assumed for the determination of ULS design loads, the design of the frame must incorporate a mechanism capable of resisting a significantly larger earthquake. This may be achieved in a number of ways;

- Follow a full capacity design procedure, OR
- Ensure a beam hinging mechanism is likely to develop to prevent the formation of a soft storey (refer to NZS3101:2006, clauses 2.6.6.1 and C2.6.6.1, only for nominal ductility structures)

$$\sum M_{n.col} \frac{L_{col(CL)}}{L_{col(clear)}} > 1.15 \sum M_{n,beam} \frac{L_{beam(CL)}}{L_{beam(clear)}}, \text{ OR}$$

- When the configuration of a structural system is such that a beam hinging mechanism cannot be assured, the relevant mechanism(s) should be identified. Potential plastic hinge regions should be identified and appropriately detailed so the strain limits given in NZS3101:2006 clause 2.6.1.3 are not exceeded for 1.5 times ULS design actions with S_p = 1.0.

When calculating the nominal flexural strength of beams, $M_{n,beam}$, the contribution of slab reinforcement when present should be included as defined in NZS 3101 clause 9.4.1.6.2.

These recommendations have been included to improve the resilience of seismic-resisting systems with low design ductilities and to ensure these structures have a minimum amount of dependable reserve inelastic capacity.

Verification Method Requirements:	Frames must be detailed to ensure sufficient capacity to resist earthquakes larger than the ULS earthquake.
SESOC Recommendation:	When a beam hinging mechanism cannot be developed potential plastic hinge regions should be designed and detailed for 1.5 times ULS design actions with S_p = 1.0.

4.3.2 Frame Elongation

Ductile moment resisting frames exhibit significant cracking due to yielding of the beams adjacent to the column faces. Each crack results in a small lengthening of the concrete beam - accumulated over several bays this elongation results in large tears across the floor diaphragm (refer Figure 4).

Figure 4 Tearing of insitu topping of precast flooring caused by frame elongation

Insitu floors tend to be able to accommodate severe damage of this form, and are arguably less likely to have damage concentrate in this manner because of their more uniform strength and stiffness. However precast flooring lacks this robustness and can result in severe collapse hazards if not detailed correctly (refer Section 4.4).

Designers should note NZS3101:2006, clause 2.6.5.10 requires deformation arising from frame elongation be considered when detailing items such as stairs, ramps, cladding panels, diaphragms and precast flooring. NZS 3101:2006 clause 7.8 provides guidance on how to calculate the magnitude of elongation in beams, columns and walls.

Verification Method Recommendation:	Deformations associated with frame elongation should be allowed for when detailing stairs, ramps, cladding panels, diaphragms and precast flooring.

4.4 Precast Flooring Systems

In general, precast flooring systems will not perform as well as steel deck or in-situ floors. While precast floors are perfectly capable of supporting gravity loads, they lack robustness to cope with damage to seatings, in-situ topping etc.

In-situ floors (conventionally reinforced or post-tensioned) are the preferred flooring system due to their superior robustness. However, in New Zealand in-situ floors tend to come at a premium, both with respect to design effort and construction cost – primarily due to their lack of use in our market.

A compromise is the use of steel deck flooring. This has a level of robustness approaching that of a one-way spanning insitu floor, but is substantially cheaper and faster to construct in the current New Zealand market.

The choice may depend on your particular project, client brief and contractor.

4.4.1 Double Tees

When double tees are used, preference should be given to flange hung supports utilising Cazaly hangers [42] similar to that shown below in Figure 6. Use of flange hung configurations are recommended due to the difficulty in providing seating for full depth ("stem sat") webs, and the improved geometry for shrinkage, thermal and support rotations. Recent testing [43] has demonstrated flange hung double tees supported on steel armoured corbels were able to sustain interstorey drifts of at least 1% without significant spalling of the supporting corbels.

SESOC Recommendation:	When used double tees should be flange hung utilising Cazaly hangers.
Damage Reduction Recommendation:	In addition to the above, when double tees are supported on reinforced concrete ledges, steel edge armouring should be provided to reduce the effects of spalling of the ledges and peak interstorey drifts for DCLS design actions shall be limited to 1%.

4.4.2 Hollow-core

Hollow-core floors have been widely used in New Zealand since the early 1980s. Poor performance of hollow-core floors in the 1994 Northridge and 2016 Kaikoura earthquakes has highlighted the vulnerability of the floor system to brittle failure modes during earthquakes [44].

Research undertaken following the 2016 Kaikoura earthquake [45] has confirmed the hollow-core support details illustrated in Figure C18.4, NZS 3101:2006A3 are inadequate and that there is no known alternative support detail for hollow-core floor units. In response to this SESOC, NZSEE and Engineering NZ have advised against the use of hollow-core floors in new buildings [46]. MBIE have subsequently signalled an intention to remove advice on how to detail hollow-core floors from B1/VM1 [47].

SESOC Recommendation:	Hollow-core floors should not be used for new construction (it is anticipated this will reflected in the November 2022 update of B1/VM1).

4.4.3 Rib and Timber Infill

Generally rib and timber infill has been found to perform adequately in the 2010/2011 Canterbury earthquakes, possibly due to the comparatively better distribution of cracks and greater topping thicknesses. Flooring details typically involved seating of the precast ribs on a steel angle or reinforced concrete corbel. Stirrup reinforcing from the ribs to the topping are important to ensure the robustness of the system in the event of cracking of the precast ribs.

SESOC Recommendation:	Prestressed ribs should be detailed with stirrups over the transfer length of the strands and with sufficient height to develop in the topping.

4.4.4 Seating Details

Recommended seating details for commonly used precast flooring systems are illustrated in Figure 5 to Figure 8 below. NZS 3101:2006A3 clause 18.7.4.3 requires precast flooring systems to be seated on bearing strips to reduce friction between the precast unit and the supporting corbel. This is not necessary when they are supported on structural steel members due to the smooth nature of the supporting surface.

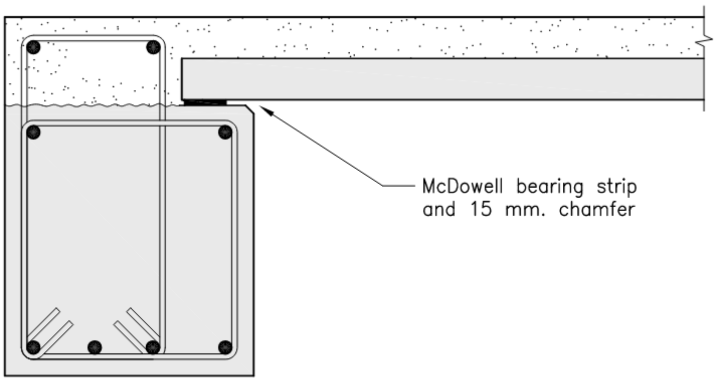


Figure 5 Recommended flat slab seating detail

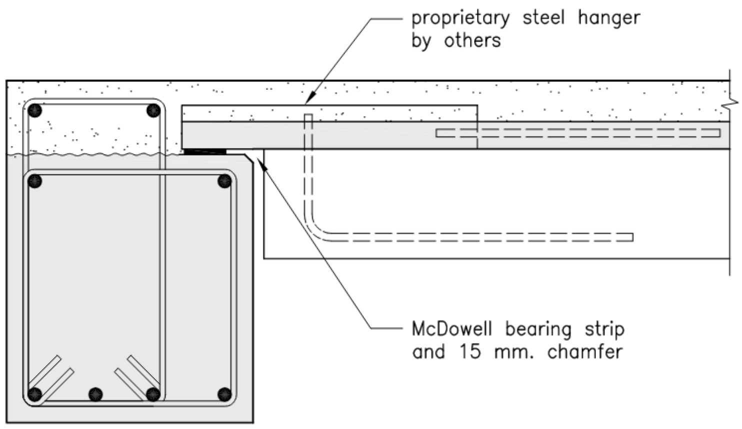


Figure 6 Recommended Cazaly flange hung double tee seating

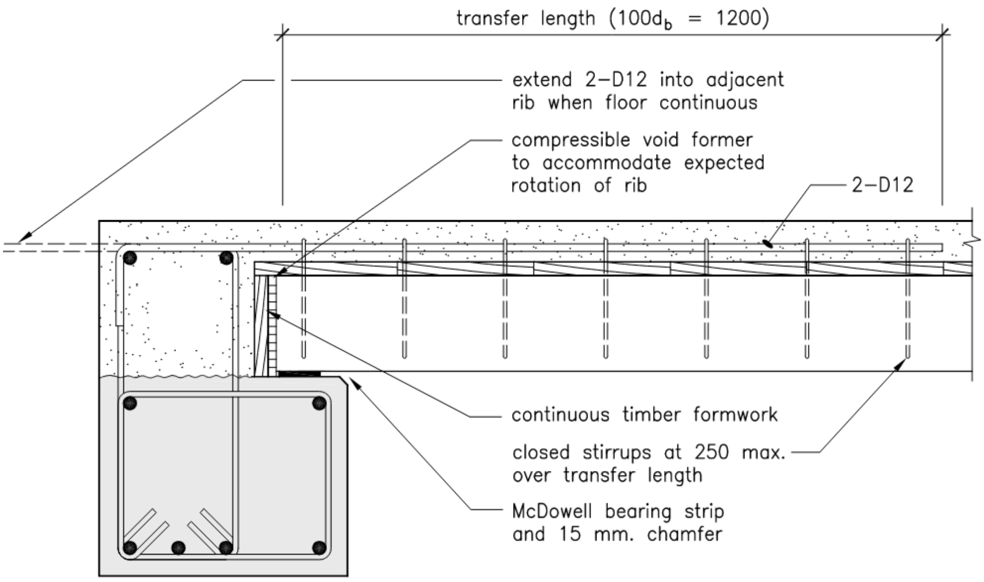


Figure 7 Recommended rib and timber infill seating

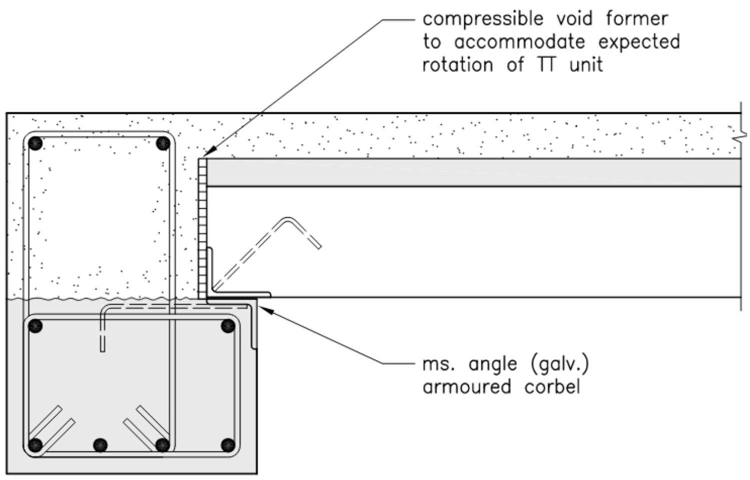


Figure 8 Recommended web supported double tee seating.

For all precast flooring systems over-stiffening of the floor system near the seating should be avoided in order to reduce the possibility of initiating a shear and/or moment failure at a location beyond the stiffened area. This is specifically highlighted in NZS 3101:2006A3 clause 18.8.2 for rib and timber infill systems. The haunch detail (where the last timber infill is sloped to meet the top of the supporting beam) commonly used in the past for rib and timber infill systems should be avoided because it restrains the web. The support detail illustrated in Figure 7 above shows an alternate detail which can be used (refer also NZS 3101:2006A3 Figure C18.8).

Precast floor systems should be detailed to preclude a negative moment failure near supports. Checks should be completed to ensure adequate reinforcement is provided in the topping slab to preclude this failure mechanism. For web supported double tees, and rib and timber infill systems, a compressible packer (or gap) should be provided between the end of the precast unit and the supporting beam to accommodate anticipated support rotations and reduce the potential for damage associated with deformation of the floor system (refer to Figures 7 and 8).

When using web supported double tees steel armouring should be provided at the ends of the precast floor unit and the supporting ledge to reduce the effects of spalling (refer Figure 8). Armoured ledges are not considered to be required (but may be used for improved performance) for pre-stressed flat slabs, prestressed ribs, or flange hung double tees. When steel armouring is used NZS 3101:2006A3 clause 18.7.4.3 requires the armouring to be fully engaged with the reinforcing of the supporting member.

SESOC Recommendation:	Figures Figure 55 to 8 provide typical seating details for commonly used precast flooring systems.
Damage Reduction Recommendation:	In addition to the above, when precast floors are supported on reinforced concrete ledges, steel edge armouring should be provided to reduce the effects of spalling of the ledges. When using double tee floor systems use flange hung seating details to minimise damage associated with support beam rotation.

4.4.5 Sliding Joints

Precast flooring typically relies on the transfer of pre-stressing forces at the ends of the unit over a relatively short distance. This may only just be enough to develop the strut and tie mechanism required to support gravity loads. Where precast flooring is detailed on sliding seatings (such as at a seismic gap), significant horizontal forces are induced on the bottom surface of the unit. These can cause cracking around the ends of the units, resulting in failure of the pre-stressing strand anchorage and subsequent loss of gravity load carrying capacity.

Because of this, sliding supports for precast flooring units should typically be avoided. Ideally double structure should be provided instead (refer to Section 9.2) although providing an area of insitu slab on a sliding seating is a reasonable compromise.

Damage Reduction Recommendation:	Double structure should be provided at seismic joints in preference to sliding details. Sliding seating details for precast flooring should be avoided.

4.5 Precast Concrete Cladding Panels

Precast concrete cladding panels are complex to design. In addition to designing panels to resist earthquake parts accelerations, panels and their fixings need to be detailed to accommodate differential displacements (in-plane and out-of-plane to the panel) and deflection induced actions including consideration of frame elongation [48].

Connection details should include adequate allowances for construction tolerance. As a minimum it is recommended connections be detailed to accommodate construction tolerances of at least ± 30 mm horizontally and ± 20 mm vertically.

Where connections to panels involve welding elements to cast in steel plates, care should be taken to mitigate the risk of cracking due to thermal expansion of the plates during welding. It is recommended that a shadow gap be provided around the outside of the plate. It is also recommended to avoid excessive weld sizes or lengths, and to ensure that welding procedures are carefully considered by the contractor.

Sliding joints for precast panels typically performed poorly in relation to their design intent in the 2010/2011 Canterbury earthquakes. Significant damage was observed to the panel connections, with some panels being dangerously close to falling off buildings.

4.5.1 Movement Allowances

When detailing precast concrete cladding panels in-plane and out-of-plane movement of the supporting building needs to be considered. NZS 3101:2006A3 clause 2.2.3 requires precast cladding panels be detailed for $1.5/S_p$ times the ULS displacements.

Verification Method Requirement:	Design panel joints to accommodate $1.5/S_p$ times the ULS displacements.

4.5.2 Detailing for In-Plane Actions

While many panel connection details were obviously designed to accommodate in-plane movement, these connections were rarely observed to slide in the earthquakes. A key cause of this is the tightening of bolts, resulting in a loss of sliding capability due to friction.

It is recommended that high density plastic shims and tube spacers are be provided at sliding joints to prevent the joints from locking up. Rubber gaskets should be provided to allow out-of-plane rotation of the panel [48].

Where mechanical fastenings are provided into precast panels, a lock nut should be provided directly against the back of the concrete panel and tightened to facilitate the mechanical fixing into the panel. The sliding connection can then be detailed as normal without worrying about losing the expansion fixing once the assembly becomes loose during sliding.

Damage Reduction Recommendation: Take care when detailing sliding joints as they tend to seize up

4.5.3 Detailing for Out-of-Plane Actions

When detailing precast panels for out of plane actions the following should be considered:

- Panel inertia forces (i.e. parts and components loads from NZS 1170.5 Section 8)
- Forces associated with yielding of the panel base connections, when this is needed to accommodate out-of-plane displacements of the supporting building (NZS 3101:2006A3 clause 17.6.3).
- Ensuring adequate clearances have been provided to prevent prying (refer Figure 9 below).

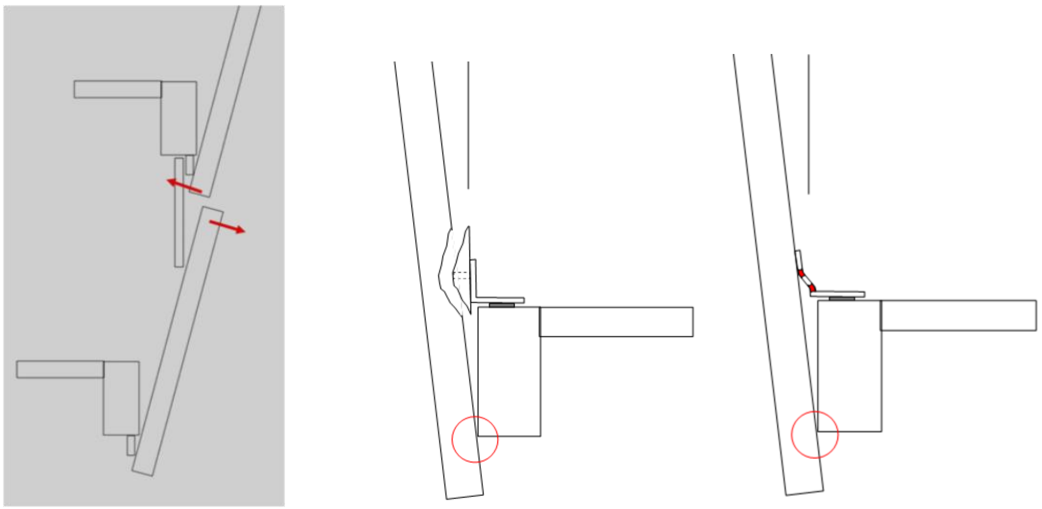


Figure 9 Potential precast panel prying failure modes (left) out-of-plane clash with bottom and top of panels, (middle) panels hitting the supporting beam causing concrete cone pull-out, (right) panels hitting the supporting beam causing yielding bracket

The practice of stiffening bottom panel brackets using vertical welded stiffeners is not recommended for non-stiff buildings. Should an earthquake occur, and the panels deform out-of-plane with the storey above, large restraint forces can be induced within the stiffened bracket. These restraint forces can be difficult to accommodate with conventional precast panel connection details which commonly include the use of brittle shallow embedded anchors.

Verification Method Recommendation:	Ensure adequate clearances have been provided to accommodate out-of-plane deformations of precast panels. Design panels and related connections for the overstrength capacity of bottom panel brackets when these brackets need to yield to accommodate out-of-plane displacements of the supporting building.

5 STRUCTURAL STEEL

5.1 General

5.1.1 Detailing for Resilience

It is recommended the NZS 3404 [6] structural system category used for the design of steel seismic-resisting systems should not exceed 3 and capacity design should be undertaken to prevent a soft storey mechanism. With reference to NZS 3404 Table 12.2.6 this recommendation limits the case number to no greater than 3. This recommendation is included so capacity design procedures are implemented and so that minimum connection robustness is achieved (by application of NZS 3404 clause 12.9.1.2.2 (4) (c)).

The limitations on axial demands on columns detailed in NZS 3404 clause 12.8.3.1 have been included in the standard to ensure steel columns can reliably sustain plastic rotations without buckling or excessive axial shortening. When reviewing these axial load limits capacity design principles should be used to establish the column design axial force. Instead of applying the compression axial force limitation waiver detailed in NZS 3404 clause 12.8.3.2, it is recommended the column axial force limitations still be met, determined using Eq. 12.8.3.1(1) with N^* taken to be 1.5 times the elastic design actions with $S_p = 1.0$ (notwithstanding that the use of N^*_{oc} to Eq. 12.8.3.1(2) remains preferred).

If the column is acting as part of a system in its major axis direction only, Eq. 12.8.3.1 may still need to be applied in the minor axis direction at column bases if the base fixity conditions result in a potential hinging region at this location (or other similar locations).

It is noted that NZS 3404 clause 12.9.1.2.2 (4) (c) contains an important requirement for category 1, 2 or 3 seismic resisting systems where capacity design principles are being applied. 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 1.25 times the actions generated by the design capacity of the primary member or members to which it is attached. This precludes the use of upper limit actions taken from analysis as defined in clauses 12.9.1.2.2 (4) (a) and (b), and can control connection detailing.

These recommendations have been included to improve the resilience of seismic-resisting systems with low design ductilities (where upper limit design actions can govern the design of members and connections) and to ensure these structures have a minimum amount of dependable reserve inelastic capacity. For the case of columns it will provide additional protection against an axial load failure.

Verification Method Requirement:	Axial demands on columns be limited to that permitted in NZS 3404 clause 12.8.3.1 to ensure they can reliably sustain plastic rotations without buckling or excessive axial shortening. For connections in category 1, 2 or 3 seismic resisting systems incorporating incomplete penetration butt welds, fillet welds, bolts and pins, and where capacity design is required in accordance with NZS 3404 clause 12.2.6, these connections be designed to resist 1.25 times the actions generated by the design capacity of the primary member or members to which it is attached.
SESOC Recommendation:	The seismic category used for the design of steel seismic-resisting systems should not exceed 3 and capacity design should be undertaken to prevent a soft storey mechanism. If designers choose to apply the compression axial force limitation waiver detailed in NZS 3404 clause 12.8.3.2, that they instead apply Eq. 12.8.3.1(1) with N^* taken to be 1.5 times the elastic design actions with $S_p = 1.0$.

For those cases when the DCLS is adopted for a project it is recommended the structural ductility factor, μ, used for the design of the primary structure should not exceed 2 for this limit state.

When determining concurrent actions on columns which are part of a two-way seismic resisting system, and one seismic system is category 2 and the other is category 3, in addition to the requirements of NZS 3404 clause 12.8.4 (b) it is recommended the column be designed to resist 100% of the capacity design actions from the category 2 system in conjunction with 100% of the earthquake forces acting on the category 3 system. This recommendation acknowledges the potential for significant design actions to be generated in the category 2 system when a building is subject to non-orthogonal earthquake actions and the additional interstorey drifts that could result from unanticipated column yielding.

Damage Reduction Recommendation:	For the DCLS limit $\mu \leq 2.0$. When determining concurrent actions on columns which are part of a two-way seismic resisting system, and one seismic system is category 2 and the other is category 3, it is recommended the column be designed to resist 100% of the capacity design actions from the category 2 system in conjunction with 100% of the ULS earthquake forces acting on the category 3 system.

5.2 Steel Moment Resisting Frames

There were relatively few large steel moment resisting frame (MRF) structures in Christchurch, by comparison with concrete structures.

Steel MRFs designed for other than elastic loads (i.e. category 4 systems) have restrictions on beam section geometry and minimisation of composite action at the column face which will suppress frame elongation.

5.2.1 Detailing for Resilience

Amendment 2 to NZS 3404:1997 published in 2007 included additional resilience requirements including limits on rotation demands of beams and columns, limits on the use of structural systems without capacity design and enhanced requirements for overstrength of connections.

NZS 3404 Table 12.2.6 specifies the relationship between structural category and member category. The intent of the table is to implicitly account for the inelastic behaviour under greater than ULS demands. It is noted the application of Table 12.2.6 to category 3 and 4 seismic resisting systems where capacity design is not undertaken, is limited to buildings not exceeding the critical height (over 4 storeys, or 5 storeys if the combined mass of the roof and top storey walls is less than 150 kg/m^2) and not irregular when assessed to the requirements of NZS 1170.5.

For seismic resisting systems that do not meet these requirements the member category for each member shall be determined explicitly by matching the plastic hinge demand determined to the plastic rotation limits of NZS 3404 clause 4.7.2. To remain consistent with the intent of Table 12.2.6, it is recommended that if required, this check be done using the total displacements calculated for 1.5 x ULS demands.

In Section 5.1.1 above, it was recommended that capacity design philosophies be applied, by selecting from case numbers 1 to 3 regardless of the design ductility selected. Published capacity design processes include the use of upper limit actions on members, and so further recommendations on how to do this are detailed below. These reiterate the preference to show (through calculation) that there is low potential for soft storey behaviour, and to ensure member ductility categories are consistent with the level of capacity protection achieved (and the level of inelastic demand that members could experience).

Steel moment resisting frames should incorporate a mechanism capable of resisting a significantly larger earthquake. This may be achieved in a number of ways:

- Follow a full capacity design procedure, OR
- If not applying a full capacity design procedure (including where actions on secondary/protected members are limited by upper limit actions) demonstrate the formation of a soft storey is unlikely. In category 3 or 4 MRFs a beam hinging or mixed sway mechanism may be assumed to form when the following equation is satisfied:

$$\sum M_{n.col} \frac{L_{col(CL)}}{L_{col(clear)}} > 1.15 \sum M_{n,beam} \frac{L_{beam(CL)}}{L_{beam(clear)}};$$

Where:

$\sum M_{n,col}$	= Sums of the nominal strength of the columns at the faces of the beam-column joint zones in the level being considered.
$\sum M_{n,beam}$	= Sums of the nominal strength of the beamss at the faces of the beam-column joint zones in the level being considered.
$L_{beam(CL)}$	= Centre to centre span of the beams.
$L_{beam(Clear)}$	= Clear spans of the beams.
$L_{col(CL)}$	= Centre to centre heights of the columns.
$L_{col(Clear)}$	= Clear heights of the columns.

When calculating the nominal flexural strength of beams, $M_{n,beam}$, the contribution of slab reinforcement when present should be included, OR

When the configuration of a structural system is such that a beam hinging mechanism cannot be assured, the relevant mechanism(s) should be identified. Plastic hinge regions should be identified, and appropriate member categories determined so the plastic rotation limits of NZS 3404 clause 4.7.2 are not exceeded for 1.5 times ULS design actions with Sp = 1.0. In addition to this the seismic member category used for column elements should not be greater than 2 recognising these elements have a lower level of protection against yielding.

Verification Method Requirement:	Follow the provision of NZS 3404 Amendment No 2 for the design of structural steel moment resisting frames. Ductility demands on members in Category 3 and 4 seismic resisting systems that do not meet the requirements of NZS 3404:1997 Table 12.2.6 shall be determined explicitly.
SESOC Recommendation:	When a column sway mechanism cannot be shown to be suppressed, member categories shall be determined so the plastic rotation limits of NZS 3404:1997 clause 4.7.2 are not exceeded for 1.5 times ULS design actions with S_p = 1.0 and the member category used for column elements should not be greater than 2.

5.2.2 Frame Elongation

Depending on the interaction between the slab and the columns, frame elongation may not be as significant an issue for ductile steel moment resisting frames as it is for concrete.

NZS 3404:1997 requires Category 1 and 2 members to be doubly symmetric. Provided doubly symmetric beams are detailed to be non-composite in potential plastic hinge regions, such beams will yield with a plastic neutral axis at mid-depth with half in tension and half in compression. By isolating the column from the slab, elongation can be minimised. Ductile steel moment resisting frames with composite floors require no special detailing for frame elongation effects on the floors.

The same may not be the case with precast floors on steel frames. No evidence of this being a problem was seen in the Christchurch following the 2010/2011 Canterbury earthquakes, although the number of such buildings is low and so it is not a sufficient data set on which to make definitive recommendations. If the same detailing is used for precast floors on steel frames that is being recommended for precast floors on concrete frames the performance is expected to be satisfactory, given that frame elongation is minimal in steel frames compared with that in reinforced concrete frames. This advice is likely to be conservative however given the generally poor performance of precast concrete floors it should be followed until more research is undertaken.

Damage Reduction Recommendation:	Isolate columns from the slab when using ductile steel moment resisting frames in order to effectively suppress beam elongation.
SESOC Recommendation:	Floor diaphragms comprising precast concrete floors on steel frames must be detailed to accommodate frame elongation where any yielding of conventional moment resisting frames is expected (note that this may not be possible to achieve). For a composite floor comprising concrete slab on steel deck on steel beams no special detailing is required however the diaphragm strength between the floor and the seismic-resisting system must be checked using a rational design procedure (as is required for all floor diaphragms)

5.3 Eccentrically Braced Frames

The general observation is that steel Eccentrically Braced Frame (EBFs) systems performed well during the 2010/2011 Canterbury earthquakes. However, there were (generally isolated) examples of EBF active link fractures where poor detailing, construction, or materials affected link robustness. Regardless, the concentration of forces in the ductile link sections makes repair relatively easy due to the isolated nature of the links.

The relationship between structural system category and member categories is given in NZS 3404 Table 12.2.6 with supplementary guidance provided in NZS 3404 clause 12.11.3.2. Utilising this guidance Table 3.1 in HERA P4001 [49] provides a concise summary of the relationship between structure category and member category.

As detailed in NZS 3404 clause 12.11.1.1 capacity design is required for category 1, 2 and 3 EBFs. It is recommended the guidance provided in HERA P4001 Section 5.5 be followed when sizing active link beams and detailing column splices. This is to minimise the potential for the development of excessive drifts at isolated levels within the EBF.

NZS 3404 clauses 12.8.3.1 and 12.11.7.5 (restricted to category 1 and 2 EBF systems only) need to be considered when checking the axial section capacity of EBF columns. When reviewing these axial load limits capacity design principles should be used to establish the column design axial force. If designers choose to apply the compression axial force limitation waiver detailed in NZS 3404 clause 12.8.3.2, it is recommended that they instead apply Eq. 12.8.3.1(1) with N^* taken to be 1.5 times the elastic design actions with $S_p = 1.0$.

If the upper limit actions of NZS 3404 clause 12.3.3.4 are used in lieu of capacity design derived axial loads when checking the section capacity of EBF columns, the seismic member category used for the EBF column elements should not be greater than 2 recognising these elements have a lower level of protection against yielding. The use of upper limit design actions for the design of EBF collectors and braces is not recommended. It makes little practical sense to design an EBF which could be controlled by inelastic behaviour or buckling in collectors or braces, rather than plasticity in the active link.

In a D-braced EBF the active link is situated at one end of a collector beam adjacent to a column. The connection between the active link and the column is detailed as rigid and substantial bending moments and axial loads need to be resisted by the column when the active link is deformed. When determining design actions on columns in D-braced EBFs capacity design principles should be used to ensure expected inelastic link behaviour can be sustained. If the designer chooses to apply upper limit design actions in accordance with NZS 3404 clause 12.3.3.4, it is recommended the EBF column design actions be determined using 1.5 times the elastic design actions with $S_p = 1.0$ and the seismic member category used for the column should not be greater than 2.

Verification Method Requirement:	Capacity design is required for category 1, 2 and 3 EBFs. NZS 3404 clauses 12.8.3.1 and 12.11.7.5 (restricted to category 1 and 2 EBF systems only) need to be considered when checking the axial section capacity of EBF columns.
SESOC Recommendation:	Follow the guidance provided in HERA P4001 Section 5.5 when sizing active links and detailing column splices to minimise the potential for the development of excessive drifts at isolated levels. If designers choose to apply the compression axial force limitation waiver detailed in NZS 3404 clause 12.8.3.2, that they instead apply Eq. 12.8.3.1(1) with N^* taken to be 1.5 times the elastic design actions with $S_p = 1.0$ and the member category used for column elements not be greater than 2. The use of upper limit design actions for the design of EBF collectors and braces is not recommended. If upper limit actions are used for the design of columns in D-braced EBFs it is recommended the design actions be determined using 1.5 times the elastic design actions with $S_p = 1.0$ and the seismic member category used for column not be greater than 2.

Some concern has been expressed by the CERC that there is insufficient redundancy in some EBF systems, noting that some links were observed to have fractured during the 2010/2011 Canterbury earthquakes. Although the buildings did not become unstable, it is noted that the short duration of the earthquakes may have masked the effects of this.

In order to provide a level of redundancy it is recommended that, in addition to the EBF frames themselves, all gravity columns are made continuous through the floors and spliced with connections capable of developing at least 30% of the section capacity in both principal directions.

SESOC Recommendation:	All gravity columns are continuous through the floors and are spliced in accordance with NZS 3404 clause 12.9.6.1 for the actions from clause 12.9.2.2.2 Beam connections onto these columns are designed and detailed to maintain beam vertical load carrying capacity when subjected to an inelastic rotation of 0.030 radians. (CERC R2.52)

When EBF columns are supported on base plates, the base plates should be detailed with shear keys to resist 100% of the design shear force to ensure even shear transfer between columns in tension and compression.

SESOC Recommendation:	When used, column base plates are to be detailed with shear keys to resist 100% of the design shear force.

5.4 Concentrically Braced Frames

Steel Concentric Braced Frame systems (CBFs) have not performed well in some cases, with failure in both proprietary and conventional systems. Failures observed include connection failure and secondary effects due to elongation of the braces and hence increased lateral drift. Observed conventional system failures have generally been due to inadequate strength of end connections or inadequate detailing for eccentricity of load path. This is commented on specifically below.

When subject to significant ductility demands CBFs are prone to the development of soft storeys which can be difficult to repair. Keeping the ductility demand low and ensuring that the connections are capacity designed are the best ways of ensuring good performance from CBF systems. For the DCLS it is recommended $\mu \leq 1.25$ for CBFs to minimise the potential for the significant contribution of inelastic deformation in a single storey.

It is recommended capacity design principles be used when determining design actions on CBF columns and collector beams. If upper limit design actions are used in lieu of capacity design, the seismic member category used for CBF column elements should not be greater than 2 recognising these elements have a lower level of protection against yielding.

SESOC Recommendation:	If upper limit actions are used for the design of columns in CBFs the seismic member category used for the columns should not be greater than 2.
Damage Reduction Recommendation:	For the DCLS limit $\mu \le 1.25$ for CBFs to minimise the potential for concentration of inelastic deformation in a single storey.

In order to provide a level of redundancy it is recommended that in addition to the frames themselves, all gravity columns are made continuous through the floors and spliced with connections capable of developing at least 30% of the section capacity in both principal directions.

SESOC Recommendation:	All gravity columns are continuous through the floors and are spliced in accordance with NZS 3404 clause 12.9.6.1 for the actions from clause 12.9.2.2.2

When CBF columns are supported on base plates, the base plates should be detailed with shear keys to resist 100% of the design shear force to ensure even shear transfer between columns in tension and compression.

For category 4 CBFs it may not always be practical to design base plate connections to develop the overstrength of the members to which it is attached. This is because columns in category 4 CBFs often have significant excess capacity. When capacity design principles have not been adopted for the design of CBF base plate connections it is recommended the design loads be determined using 1.5 times the elastic design actions with $S_p = 1.0$.

SESOC Recommendation:	When used column base plates are to be detailed with shear keys to resist 100% of the design shear force. When capacity design principles have not been adopted for the design of CBF base plate connections it is recommended the design loads be determined using 1.5 times the elastic design actions with $S_p = 1.0$.

5.4.1 Tension Only CBFs

For tension bracing systems where yielding of the braces can lead to increased drift, designers are advised to consider carefully the impact of the increased drift.

Notched braces to reduce the tension capacity must be designed and detailed to NZS 3404 clause 12.12.7.2 otherwise the notch is likely to have an adverse effect on brace and system performance. Note the increased effective length required by Amendment No 2 in subclause (h).

Proprietary systems should only be used within the limitations noted above in Section 2.5. At least one proprietary system was observed to have suffered failures in the 2010/2011 Canterbury earthquakes, and on review, product testing information was found to relate only to testing of a component, not to the system as a whole.

Proprietary bracing systems should only be used where they have been:

- Tested to dynamic loading conditions and shown not to suffer brittle failure, and
- Installed in accordance with the manufacturer's instructions and will dependably remain in the installed state in service. That means that any locating or restraining nuts on rods must remain in the installed condition and not loosen.

Verification Method Requirement:	Bracing systems and their connections must be designed and detailed to the provisions of NZS 3404. Note especially the connection strength requirements and detailing provisions for notches, if used.
SESOC Recommendation:	Proprietary systems shall have been subject to a comprehensive testing regime, and shall be installed in accordance with the manufacturer's instructions.

5.5 Composite Beams and Precast Flooring

Some designers over recent years have elected to use precast concrete floor systems in conjunction with steel composite floor members. However it is noted that most research on the use of composite beams has used composite concrete filled metal decking which has been specifically developed for this purpose. There are some exceptions that have been tested, but typically not in the configurations used in NZ.

It is a significant concern that precast flooring typically concentrates the effects of creep and shrinkage movements at the ends of the units, directly adjacent to the composite connectors to the steel beams (refer Figure 10). This may result in loss of confinement to the concrete adjacent to the studs, which in turn could lead to loss of composite behaviour.

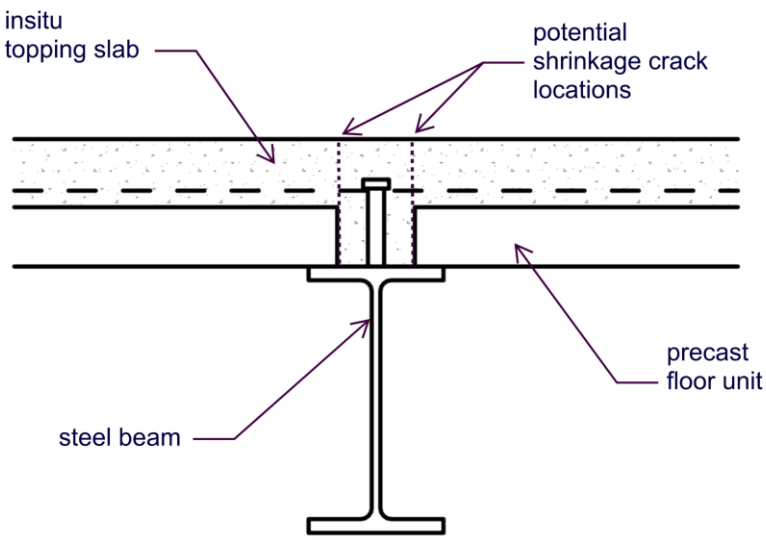


Figure 10 Cross section of composite steel beam supporting precast concrete floor units

The preferred option here is that there is no mixing and matching of precast flooring with composite steel beams unless or until research can be completed on the configurations of flooring used in NZ. At the very least, steel beams should be sized such that the maximum unfactored gravity load (*G & Q)* can be resisted on the bare steel section using the design section moment capacity of the steel beam in the event that composite connection is lost , assuming that the diaphragm actions may concentrate tensile strains at the beams.

Beam flange widths should be used that allow edge distances to the face of the precast unit to be treated as a slab edge in accordance with AS/NZS 2327:2017 clause 3.6.2.7.1. When the minimum edge distance requirements of AS/NZS 2327 clause 3.6.2.7.1 cannot be maintained transverse reinforcement, A_{rt}, should be provided in accordance with the equation below to control the post-splitting capacity of shear studs [50]:

$$A_{rt} > 430\,\frac{d_{sc}^2}{s_{sc}}\ (\mathrm{mm^2/metre\ length})$$

Where:

d_{sc} = Diameter of shear stud (mm)
s_{sc} = Average spacing along the line of the stud (mm)

When placed as bars the diameter of the transverse reinforcement should not exceed 16 mm. The transverse reinforcement should be located as low as possible and no less than 15 mm below the top of the shear stud. The transverse reinforcement shall be adequately anchored to develop the yield strength on both sides of the steel beam by embedment or hooks in accordance with NZS 3101:2006.

When determining the capacity of shear studs in accordance with AS/NZS 2327 clause 3.6.2.4.1 h_p shall be taken as the height of the precast floor unit above the top flange of the steel beam. When designing for longitudinal shear in accordance with AS/NZS 2327 clause 3.8.3 the contribution of any cohesion that may be present on the potential shrinkage cracks illustrated in Figure 10 above should be ignored.

Damage Reduction Recommendation:	When designing using composite steel beams, only use flooring systems that do NOT concentrate shrinkage and creep strains at the beam. This is deemed to be satisfied if using composite metal deck systems.
SESOC Recommendation:	If precast flooring systems are being used with composite steel beams, the bare steel member must be able to resist at least G&Q actions. The detailing of the shear connectors should be treated as if the precast flooring ends are a free edge, considering that shrinkage could open a crack in this location. Provide transverse reinforcement to control post-splitting capacity of shear studs as detailed above.

6 FLOOR AND ROOF DIAPHRAGMS

The performance of floor diaphragms in the 2010/2011 Canterbury earthquakes was varied. Significant damage was observed where diaphragms were required to drag large loads between lateral load resisting elements, or where diaphragm tearing caused consequential damage to non-robust flooring systems (refer Section 4.4).

The performance of thin toppings on precast flooring systems was mixed. Inelastic displacements tended to focus on pre-existing crack locations, resulting in large cracks which have often fractured the mesh. In general, the narrower modules of precast systems have performed better, possibly due to the greater number of shrinkage and creep-related pre-existing cracks between the precast units.

Older reinforced concrete insitu slabs have proven more robust, even where not specifically designed for earthquake actions. Such slabs typically have more reinforcement to resist earthquake actions, and their increased thickness provides greater stiffness.

Composite slabs on steel deck have also been shown to be robust as diaphragms and in general.

Roof diaphragms in lightweight structures have generally performed adequately, although in many cases there have been greater than expected deformations as a result of connection failure or tension brace yielding.

6.1 NZS 1170.5 Amendment 1 provisions for diaphragms

Amendment 1 to NZS 1170.5 introduces detailed provisions pertaining to analysis of diaphragms. While not cited in Verification Method B1/VM1, it is recommended that these be adopted for new designs.

Confusion can arise with respect to the overstrength base shear used for diaphragm design, $V_{ob,max}$, which is stated at page 65 of the commentary to be:

$$V_{ob,max} = 1.5V_{E,\mu=1.0,S_p=1.0}.$$

The following page 66 states further that "$V_{E,\mu=1.00,S_p=1.00}$ [cor]responds to 1.5 times the design base shear with the structure responding elastically and with an $S_p = 1.0$". This statement is incorrect and should be taken as "$V_{E,\mu=1.0,S_p=1.0}$ corresponds to the design base shear with the structure responding elastically and with an $S_p = 1.0$".

Clause C5.7.A2.3 notes that only the ESA part of the pESA represents an overstrength condition (even if limited by $V_{ob,max}$), and therefore suggests the use of a lower strength reduction factor to floors in the "PGA section" in order to maintain appropriate ULS reliability against inertial effects". This can lead to inconsistent and unintended outcomes, especially when transfer forces cause significant actions in diaphragms where the floor force is set by the PGA. Instead, it is recommended that PGA forces are scaled by 1/0.75 when being applied to the pESA, and the strength reduction ϕ taken as 1.0 for all floors. This is a better and simpler means of providing consistent reliability against transfer and inertia related action effects.

Further guidance on how to undertake grillage analysis for diaphragms can be found in assessment guidance for concrete buildings [51].

SESOC Recommendation:	Diaphragm provisions contained in NZS 1170.5 Amendment 1 should be used for design of diaphragms.

6.2 Collector Elements

Collectors are required to transfer significant tension/compression forces. Where compressive stresses exceed the strut and tie limits given in NZS3101:2006 clause A7.2 (a limit of 0.5 f'_c may be used conservatively), transverse confinement of collectors should be provided in accordance with NZS3101:2006, clause 10.3.10.6.

SESOC Recommendation:	Where compressive stresses exceed the strut and tie limits (conservatively taken as 0.5 f'_c), confinement should be provided in accordance with NZS3101:2006, clause 10.3.10.6.

6.3 Concrete floor diaphragms

Absolute minimum topping thicknesses of 75mm should be used on precast floors. Significantly greater topping thicknesses are likely to be required where transfer effects are present.

The actions in suspended floor diaphragms are extremely difficult to accurately determine. To provide a level of robustness, hard-drawn or other non-ductile mesh may NOT be used.

Ductile mesh or deformed bars should be provided. There are now several forms of ductile mesh on the market. Designers should verify that the specified ductile mesh, or any substitution offered by the contractor, meets the requirements of the Verification Methods. When using deformed bars, the maximum bar spacing permitted in NZS3101:2006 (clause 9.3.8.3) is 300mm for topping reinforcement on precast floors or 200mm for bars spanning across the infill slabs common to rib and timber infill.

Verification Method Requirement:	Concrete diaphragms must be designed using a strut-and-tie approach, or equivalent method. Hard-drawn or non-ductile wire meshes are NOT permitted to be used as reinforcement for floor diaphragms. If using deformed bars, maximum bar spacings are 300mm in toppings for precast floor systems, 200mm for infill slabs between precast units.
SESOC Recommendation:	The absolute minimum topping thickness should not be less than 75 mm.

6.4 Lightweight Roof Bracing

Lightweight roofs often use tension-only bracing. This bracing should comply with the same requirements as concentric bracing as noted in Section 5.4.1 above. Roof lateral load resisting elements (i.e. roof braces, struts and chords) should generally be designed to remain elastic by either:

- Following a capacity design approach whereby the roof lateral load resisting elements and their connections are detailed to resist the overstrength actions developed in the vertical lateral load resisting system, or
- The roof lateral load resisting elements and their connections are detailed to resist 1.5 times the elastic design actions with S_p =1 in accordance with Section 6 of NZS 1170.5, or
- For those situations when Section 8 of NZS 1170.5 has been adopted for the design of the lateral load resisting system (i.e. as is sometimes the case for penthouse structures), the roof lateral load resisting elements and their connections should be detailed to resist elastic loads determined using μ = 1 and S_p =1 in accordance with Section 8 of NZS 1170.5.

In certain cases, designers may have used ductile tension bracing in order to limit load input into the primary system. In such cases, the tension yielding elements must be capable of accepting the full displacement of the system without fracture or failure of connections and the C_s factors specified in NZS 3404 clause 12.12.6 should be applied.

Use of proprietary systems should be treated in the same manner as noted in Section 2.7 above.

Verification Method Requirement:	Yielding elements of the bracing system must be detailed with notches in accordance with NZS3404, unless it can be shown that they can accommodate the deformations associated with 1.5/S_p times the ULS drift.
SESOC Recommendation:	Roof lateral load resisting elements and their connections should be capable of developing the overstrength capacity of the vertical yielding element of the lateral load resisting system, unless designed to remain elastic as noted above. For steel systems the minimum design actions from NZS 3404 should be followed.

7 TRANSFER STRUCTURES

Transfer structures involve the transfer of vertical loads where a continuous load path to ground is not possible. They are often complex and may have significant consequences should failure occur.

Transfer structures may be simple gravity transfer structures, which typically are used where column lines do not extend all the way to ground. Such structures do not contribute significantly to the overall lateral load resisting system, but must maintain their gravity load carrying capacity through the full range of displacement that the building may be subject to.

Other transfer structures may have a similar function, but are in addition required to contribute significantly to the lateral load resisting system. In such cases, consideration must also be given to the possible overstrength actions that may result from larger earthquakes than the design basis event. Input actions to the transfer structure may be limited through capacity design, but this may not cover all actions. For example the effect of vertical loads is not amplified for such cases, and may be significant in the case of flexural elements below the transfer level.

7.1 Design Actions

While the detailing required by modern design codes will typically enable structural elements to sustain the deformations resulting from larger earthquakes than considered in design, transfer structure may not have this robustness.

As such, transfer structures should be designed for 1.5 times the ULS earthquake actions for forces or $1.5/S_p$ times the ULS displacement. Furthermore, the transfer structure should be designed for the concurrent actions of vertical and horizontal accelerations. A rational approach is considered to be to design the transfer structure for the SRSS of the design actions resulting from the vertical and horizontal accelerations.

SESOC Recommendation:	Design transfer structure for 1.5 times the ULS earthquake forces or $1.5/S_p$ times the ULS displacement. In the case of transfer structure that carries only gravity load, the increased vertical actions from 1.5 times the ULS earthquake design actions should be used, and the structure should be checked for its ability to carry its load through $1.5/S_p$ times the ULS displacement. In the case of transfer structures that contribute to the overall lateral load resistance, the derivation of design actions must include consideration of the overstrength actions of the structure above, as well as concurrency effects. Vertical actions should be added as noted in Section 3.2 above. If any part of the structure is designed for $\mu \leq 1.25$ actions, designers should use S_p=1, unless a capacity design approach has been followed.

8 STAIRS

Stairs were observed to perform poorly in many instances in the 2010/2011 Canterbury earthquakes. In particular, a lack of sliding capacity (elongation and compression) was responsible for the more publicised collapses.

It was also noted that typical sliding details involving a pocket in the landing tended to have been filled over the years by maintenance personnel, resulting in a removal of any compressive sliding capability.

For more detailed information than the summary given here, refer to the report to the Royal Commission [52].

8.1 Movement allowance

Detail sliding joints to Amendment 1 NZS 1170.5 clause 8.8. This includes accommodating inter-storey drifts associated with earthquakes that are $2/S_p$ times ULS event with additional allowances for construction tolerances, spalling, creep and shrinkage, temperature effects and deformations of the supporting structure and foundations.

SESOC Recommendation: Detail sliding joints for $2/S_p$ times the ULS displacements with additional allowances for construction tolerances, spalling, creep and shrinkage, temperature effects etc (refer Amendment 1 NZS 1170.5 clause 8.8).

8.2 Friction

Note that significant friction forces exist at sliding joints. Typical coefficients of friction are shown in Table 2 below.

Verification Method Requirement: Allow for minimum and maximum coefficients of friction in the design of stair connections.

Table 2 Coefficients of Friction - Maxima and Minima

Contact surfaces	μ (min)	μ (max)
Concrete on concrete[1]	0.5	1.0
Concrete on steel[2,3]	0.35	0.7
PTFE on stainless steel[4]	0.02	0.15

Notes:

1. From BS EN 12812:2004 Falsework – Performance requirements and general design. These values assume concrete has not been intentionally roughened.
2. Lower bound taken from NZS 3404:1997 for steel on steel
3. Upper bound taken from NZS 3101:2006 for concrete cast against steel and anchored using headed studs or reinforcing bars
4. Taken from requirements of Transit New Zealand Bridge Manual

8.3 Detailing

Stairs should typically be detailed with a fixed top connection and sliding base connection. The friction forces at the sliding connection should be evaluated and the stair detailed to either resist these forces (tension/compression), or accommodate the lateral displacements (transverse movement). Guidance on friction coefficients is provided in Section 8.2 above.

Split scissor stairs may be fixed at the floor levels and free to slide on their mid-height supporting beam. However, the horizontal friction forces should be considered in the design of the supporting beam.

Detailing should be such that maintenance contractors cannot easily fill the sliding joint. It is therefore recommended that the lower step be left to slide freely on top of the landing.

SESOC Recommendation:	Design stair for the friction forces induced (tension/compression and transverse shear). Provide sliding joints with details so they cannot be filled (refer to Figure 11)

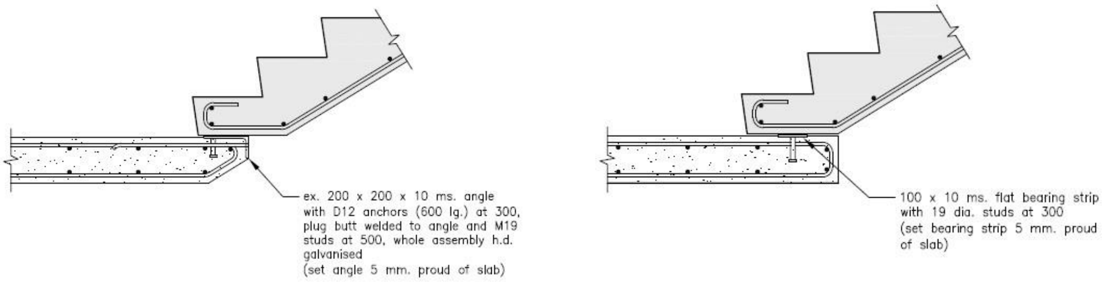


Figure 11 Typical stair details

9 SEISMIC JOINTS

Seismic joints are typically provided between buildings on a site that have been seismically separated.

9.1 Size of Joints

The size to be provided for the seismic gap will depend on the consequences of pounding. At the very least, a gap equivalent to the sum of the ULS displacements of the two buildings should be provided. If pounding in an event larger than ULS is not likely to cause catastrophic damage to the building structure, then this is probably sufficient.

However, if pounding has the potential to cause significant structural damage leading to collapse (as may be the case when adjacent floors are offset, or there are significantly different building heights), then the seismic gap should be increased in size to accommodate $1.5/S_p$ times the ULS displacements of the buildings. This is not critical where adjacent buildings have common heights and floor levels, such as in parts of a building that are separated by seismic joints detailed for the ULS drift.

Damage Reduction Recommendation:	Consider increasing seismic gaps where pounding has the potential to cause significant structural damage.

9.2 Detailing

Seismic joints have often been detailed with a sliding corbel to support the flooring from the adjacent building. This minimises the cost of double structure and reduces space requirements. However, the sliding induces significant friction forces (refer Table 2) which can alter the structural behaviour and cause significant local damage to the flooring.

It is recommended that double structure is provided at seismic joints, with the adjacent buildings each having their own vertical support system adjacent to the gap.

When a sliding corbel is used the seismic joint should be detailed in accordance with Amendment 1 NZS 1170.5 clause 8.8.

Damage Reduction Recommendation:	Double structure should be provided at seismic joints in preference to sliding details.

10 FOUNDATIONS

10.1 General

Poor performance of some buildings in the 2010/2011 Canterbury earthquakes highlighted the need for substantially more consideration of the seismic performance of foundation systems. The most appropriate foundation system for the site and structure should be selected, but the temptation to mix systems for cost-efficiency should be avoided. Where mixed foundation systems have been used, the different performance of the various bearing layers has resulted in significant residual deformations to an otherwise lightly damaged structure.

To improve the standard of earthquake geotechnical engineering and address the lessons from the 2010/2011 Canterbury and 2016 Kaikoura earthquakes, NZGS and the Ministry of Business, Innovation and Employment (MBIE) have jointly developed the Earthquake Geotechnical Engineering Practice Modules 1 – 6 [53].

Much of the material in the following sections has been sourced from Module 4 – Earthquake Resistant Foundation Design [21]. It is recommended that structural engineers familiarise themselves with the recommendations in Module 4.

10.2 Collaborative Approach to Design

Structural engineers are not experts with respect to geotechnical issues, and advice should be sought from appropriately qualified geotechnical engineers on all projects involving foundation works.

Foundation design is necessarily a combined effort between the structural and geotechnical engineers. For each project the geotechnical engineer and the structural engineer should agree what aspects of the design will be led by geotechnical and which will be led by the structural consultant. 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.

The structural engineer and geotechnical engineer should hold early discussions to allow the joint selection of compatible foundation and structural systems, in consultation with the client. The selected structural system should allow for any constraints which ground conditions may put on the performance or economics of the foundation system (e.g. the ground conditions may dictate that resisting high concentrated uplift is not practical or economic). The foundation system should have load displacement characteristics which are compatible with the performance requirements of the structural system.

Geotechnical reports should provide not only foundation design parameters, but also comment on the most appropriate foundation type for the proposed structure and site as agreed by the structural and geotechnical engineers. The structural engineer should discuss with the geotechnical engineer results of structural analysis based on parameters supplied by the geotechnical engineer. This will allow review of how parameters have been applied and challenge of parameters which prove to be critical to the design. During the design of the foundations, the geotechnical engineer should be asked to review the foundation design to ensure that their advice has been implemented and detailed in an appropriate manner.

The geotechnical engineer should be given the opportunity to review the foundation detailing on the plans prior to submission for Building Consent. The geotechnical engineer should also be involved in Construction Monitoring during the foundation phase of the construction.

<table>
<tr><td>SESOC Recommendation:</td><td>The structural engineer and geotechnical engineer should hold early discussions to allow the joint selection of compatible foundation and structural systems, in consultation with the client.

The design of the foundations and interaction with the structure should be a collaborative process between the structural engineer and the geotechnical engineer.

The geotechnical engineer should review the final foundation design.</td></tr>
</table>

10.3 Geotechnical Site Assessment and Selection of the Foundation and Structural Systems

Good foundation performance with earthquake shaking depends critically on the response of the site soils to shaking and the response of the site itself. 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 [54].

Site assessment will require geotechnical evaluation of ground conditions and likely site investigation. Module 2 [55] "Geotechnical investigations for earthquake engineering" provides guidance to geotechnical engineers on appropriate investigations to assess the ground conditions to support the seismic design of structures.

Foundation selection and design should be carried out in the context of a good understanding of the site soil response to earthquake shaking and the overall performance of the site including settlement and stability. These factors need to be considered in parallel with the development's design objectives and selection of the structural system. The following issues should be considered by the geotechnical engineer as part of a site assessment [21]:

- Soil response including liquefaction, cyclic softening and other changes in soil properties caused by shaking.
- Site performance including liquefaction severity, lateral spreading, settlement, and instability and the impacts of these on potential foundation systems.
- Building interaction effects. The presence of a building may significantly alter the response of the site and exacerbate the effects of ground failure and settlement.
- Foundation suitability. Including specific requirements and issues to be considered for different types of foundation systems taking account of the above effects.

Module 4 Section 4 [21] provides further detail on site assessment and foundation selection and with a structured approach.

In addition to these geotechnical considerations, during selection of the foundation and structural systems and design, the geotechnical and structural engineers need to collaboratively consider the following in consultation with the client:

- The objectives for the development including the client's acceptance or not of damage in specific events.
- The available structural options and the constraints they impose on the foundations including magnitudes of compression and tension loads and tolerance to deformation.
- Other merits of the foundation/structural systems including, cost, performance and sustainability.

The above issues should be considered for a range of earthquake shaking levels (refer next section), and the outcomes of this assessment summarised in the geotechnical report.

SESOC Recommendation:	Specific geotechnical advice must be sought for all sites. As part of a site assessment the geotechnical engineer should consider the overall performance of the site, including the

potential for liquefaction, cyclic softening and changes in soil properties caused by earthquake shaking and associated structural loading.

The foundation and structural systems need to be selected by the structural and geotechnical engineers collaboratively and in consultation with the client.

10.4 Geotechnical Step Change

Under verification method B1/VM1 there is no requirement to consider earthquake events between the SLS1 and ULS levels of shaking, or beyond ULS shaking (except SLS2 for IL4 buildings). The underlying assumption is that there would be a continuum of performance between SLS and ULS, and resilience beyond ULS. However, the behaviour of soils and foundation systems under earthquake shaking may be highly nonlinear and may even exhibit a pronounced 'step change' in performance [21]. Examples of this nonlinear behaviour include:

- Slope instability
- Shearing of the concrete/grout to ground resistance of a friction pile or anchor resistance
- Liquefaction and lateral spread

The geotechnical engineer should consider the full range of earthquake shaking including between SLS1 and ULS and beyond ULS in identifying susceptibility to 'step change'. Where 'step change' potential is identified the trigger and consequence of that step change should be assessed and the geotechnical and structural engineers should work together to avoid or mitigate adverse effects.

In general, non-ductile elements such as ground anchors in rock should be avoided to resist structural loads. If they cannot be avoided, they should be designed to take loads beyond ULS so that other more ductile failure mechanisms are critical.

The following subsections provide recommendations for allowing for this 'step change' behaviour:

- Between SLS1 and ULS
- Beyond ULS

10.4.1 Between SLS1 and ULS

Section 3.5 of Module 4 [21] provides guidance on considering potential 'step change' between SLS1 and ULS.

Figure 12 illustrates two hypothetical cases of geotechnical 'step change'. Case A shows a system for which a large ground response is triggered for a ground motion intensity corresponding to a 40 year return period, whereas in Case B the triggering occurs at a 400 year return period. Both cases show acceptable performance for SLS1 level of shaking. However Case A performs significantly worse with a step change occurring soon after the

SLS1 ground motions and the resulting damage might be excessive and inappropriate for such a high likelihood of occurrence.

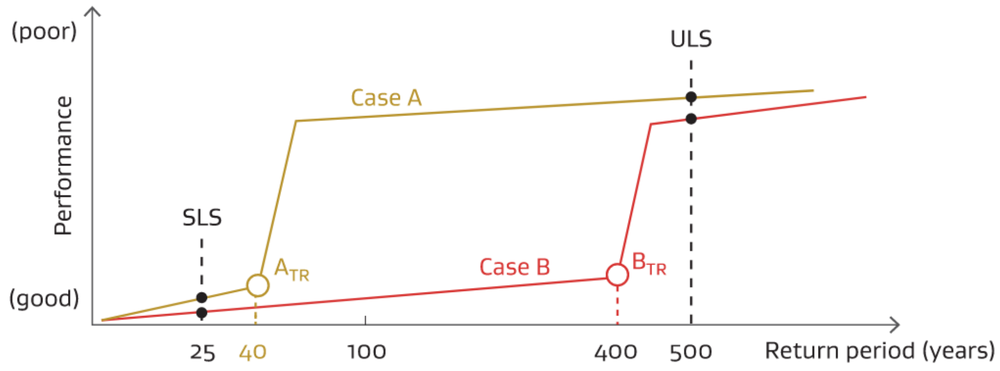


Figure 12 Step change in performance between SLS and ULS (NZGS, 2021)

Typical examples of 'step change' include sites affected by liquefaction or slope instability. For such cases, only considering performance at the SLS and ULS levels of shaking would fail to identify poor performance at intermediate return periods of shaking. It is important to discuss performance expectations with the client at the start of a project so the foundation system can be detailed appropriately [21].

Where a 'step change' is identified at or below ULS shaking this is to be allowed for in the ULS design. In addition, the geotechnical engineer should discuss with the structural engineer the assessed trigger and deformations or degradation of capacity associated with the 'step change'. The structural engineer should evaluate the consequences for the performance of the structure. The structural and geotechnical engineers should discuss with the client any adverse performance and consider possible options to mitigate.

An example where this 'step change' between SLS1 and ULS could be important is a building on shallow foundations in Wellington subject to differential settlement due to liquefaction effects. The SLS1 demand (0.13g M6.5) is unlikely to trigger liquefaction but a 100 year event (0.28g M7.1) could trigger the liquefaction and settlement. This settlement may be considered acceptable for ULS performance but it may not be acceptable performance for a 100 year event. This could dictate the use of piles rather than the shallow foundations.

10.4.2 Beyond ULS

Section 7 of Earthquake Design for Uncertainty [11] provides general guidance on allowing for 'step change' beyond ULS. Here further guidance is provided by way of an example of an Importance Level 2 (IL2) building on soils susceptible to liquefaction.

It is not the intent to apply ULS design requirements beyond ULS intensity of shaking, but rather to check the soil/foundation/structure system for potential instability/collapse mechanisms beyond ULS shaking and if collapse potential is identified modify the design to mitigate this. The intent is to provide some resilience in foundation performance beyond ULS and to allow for the uncertainty in predicting the level of shaking at which possible 'step changes' are triggered.

For the example of an IL2 building on ground susceptible to liquefaction, the following applied in conjunction with the Geotechnical Practice Modules [21,53], may be considered as a means of managing the risk. For IL3 and IL4 buildings a similar approach could be adopted but with further consideration of the demand and performance objectives.

- Liquefaction triggering analysis indicating adverse effects for ULS would require allowance for these effects in ULS design. This should be considered a current Verification Method requirement.
- If liquefaction susceptibility is identified but triggering analysis does not indicate adverse effects under 2500-year return period, then liquefaction effects may be an acceptable risk.
- If triggering analysis does indicate adverse effects under 2500-year return period (but not under the 500-year return period), then the effects may need to be given consideration, as follows. The intent is a stability check and not ULS design at a higher level of shaking
- For vertical effects (gravity support and overturning stability)
    - Structures, such as frames, which can tolerate a few hundred mm of differential settlement and a reasonable reduction in load bearing capacity without losing support or stability may not require further consideration. (These are expected to be largely qualitative reviews based on controlling behaviour, risk and consequence under this scale of effect. For example, curvature ductility checks are not expected)
    - If the general magnitude of these effects could lead to loss of support or instability (such as significant tilting, overturning instability of slender wall foundations, or loss of floor support), then assessment and design should be carried out. Moderately conservative assessment of degraded geotechnical resistance under 2500 year hazard with Strength Reduction Factor (SFR) = 1 could be used with ULS structural design action effects.
- For horizontal effects (lateral ground deformations (kinematics) and base shear)
    - Where lateral spread or cyclic displacements of more than 100mm are unlikely for the 2500 year event, consideration may be limited to providing robust foundation tying, and appropriate detailing to potential hinge regions. This should be considered a current verification method requirement.
    - If greater displacements are possible, specific assessment and design will likely be required. This could include further development of tying and detailing and checking capacity to tolerate possible residual differential lateral displacements. The predominant focus of this assessment beyond the ULS should be to maintain vertical support and global overturning stability—however base shear and kinematic effects may need to be given some consideration if those effects are significant and could cause a destabilising risk.

An example where this ‘step change’ beyond ULS could be important is a building on shallow foundations in Hamilton subject to reduced vertical support due to liquefaction effects. The ULS demand (0.25g M5.9) may not trigger liquefaction but a 1000 year event (0.32g M5.9) could trigger the liquefaction and reduced vertical support. If this reduced support could lead to instability of the structure this may dictate the use of piles rather than the shallow foundations to mitigate this adverse effect.

SESOC Recommendation:	The foundation/structure design should include consideration of geotechnical 'step change' over the full range of earthquake shaking including between SLS1 and ULS and beyond ULS.

10.5 Geotechnical Strength Reduction Factors

Geotechnical advice should be sought in selecting geotechnical strength reduction factors. Module 4 [21] guidance for selection of geotechnical strength reduction factors is as follows:

Vertical Design:
Shallow foundations: Refer Table 5.1 Module 4 [21]. A value of 0.45 to 0.6 is to be selected on the basis of a risk assessment.

Deep foundations: Refer AS 2159-2009 [56] Section 4.3. A structured risk assessment is applied determining a strength reduction factor of 0.4 to 0.76 if no pile load testing is undertaken and up to 0.9 with load testing. SESOC recommends that even if the AS2159 assessment determines a value greater than 0.7, this high value only be used with careful consideration of the load-deformation risk by the geotechnical and structural engineers.

Lateral Design:
Horizontal actions may be checked without applying strength reduction factors, noting that designers must take into account the effect of earthquake induced horizontal movements when considering these actions.

The Building Code compliance document B1, in B1/VM4 proposes the use of higher strength reduction factors in conjunction with load combinations involving earthquake overstrength. This proposal is superseded by the above Module 4 [21] guidance. SESOC recommends that the higher B1/VM4 strength reduction factors not be used.

Consideration should be given in a design to adopting different geotechnical strength reduction factors for earthquake load cases to those for the non-earthquake load cases. Earthquake shaking may reduce soil strengths and thus foundation capacities by liquefaction, pore water pressure increases, and cyclic softening within the influence zone of the foundation. These effects not only reduce foundation **capacity** but also increase **uncertainty** in foundation performance, thus warranting a lower strength reduction factor for the earthquake case than that used for the non-earthquake case, because of the uncertainty.

This is especially true where higher strength reduction factors are adopted following static load tests. Static load tests give no useful information about foundation capacity with earthquake shaking unless the soils are definitely not going to be affected thereby. In addition to applying the lower strength reduction factor the assumed capacity must be reduced to allow for any earthquake shaking effects.

SESOC Recommendation:

The geotechnical engineer to propose geotechnical strength reduction factors based on Module 4 [21] guidance. If values greater than 0.7 are to be used the load-deformation risk requires careful consideration.

The use of higher strength reduction factors in conjunction with load combinations including overstrength factors detailed in B1/VM4 are **not** recommended.

10.6 Lateral Spreading

When dealing with sites that are susceptible to lateral spreading advice should be sought from the geotechnical engineer. It is not possible to reliably predict the location, scale and magnitude of lateral spread movements. Any prediction of lateral spread should consider multiple methods [21].

Because lateral spread cannot be predicted designs which rely on the prediction of the magnitude of total or differential lateral spread should be avoided. Rather than developing designs to tolerate a magnitude of predicted lateral spread, it is recommended that designs be developed to resist the full passive or friction the lateral spread could impose on the foundations and structure. Or undertake ground improvement or retaining works to mitigate the lateral spread potential. Raft foundations have relatively good resilience in the event of lateral spread. In contrast tied together pad or pile systems will be more vulnerable.

For the purposes of design, it is recommended the geotechnical engineer develop a series of scenarios of potential lateral spread to be applied to the proposed foundation design(s). These scenarios should be developed in consultation with the structural engineer with the objective of the identifying possible scenarios which could be most damaging to the proposed foundation design(s) e.g. tie forces in ground beams and lateral deformations in piles.

Foundation elements should be detailed to ensure they have adequate capacity to support gravity loads while sustaining the chosen earthquake energy dissipating mechanism in the structure. Both lateral and vertical distribution of the lateral spread should be considered.

SESOC Recommendation:

Specific geotechnical advice should be sought to determine potential design actions on foundation elements associated with lateral spreading for affected sites.

When shallow foundations are used in those situations where there is potential for significant lateral spread raft foundations are preferred to tied pad systems.

10.7 Ground Water Pressures in Liquefiable Materials

Typically design of sealed basement structures considers the water pressures associated with the maximum water table. In the case of liquefaction occurring at the site, the pressures may be higher still, since the liquefied material has essentially been pressurised by the ground motion. In addition, the density of the silt laden fluid is likely to be significantly greater than that of water.

Advice should be sought from the geotechnical engineer with respect to the likely pressures at your site.

Damage Reduction Recommendation:	Ask the geotechnical engineer what pressure to allow for in submerged basements subject to liquefaction.

10.8 Soil Structure Interaction

10.8.1 Introduction

Soil Structure Interaction (SSI) can have a significant influence on the seismic performance of a building. In general, buildings with slender shear walls or braced frames will be most sensitive to SSI effects under seismic loading. Referring to Figure 13 below assuming unrealistically stiff foundations can result in an unrealistically low fundamental period for the structure, or underestimation of structural deformations. The converse also applies.

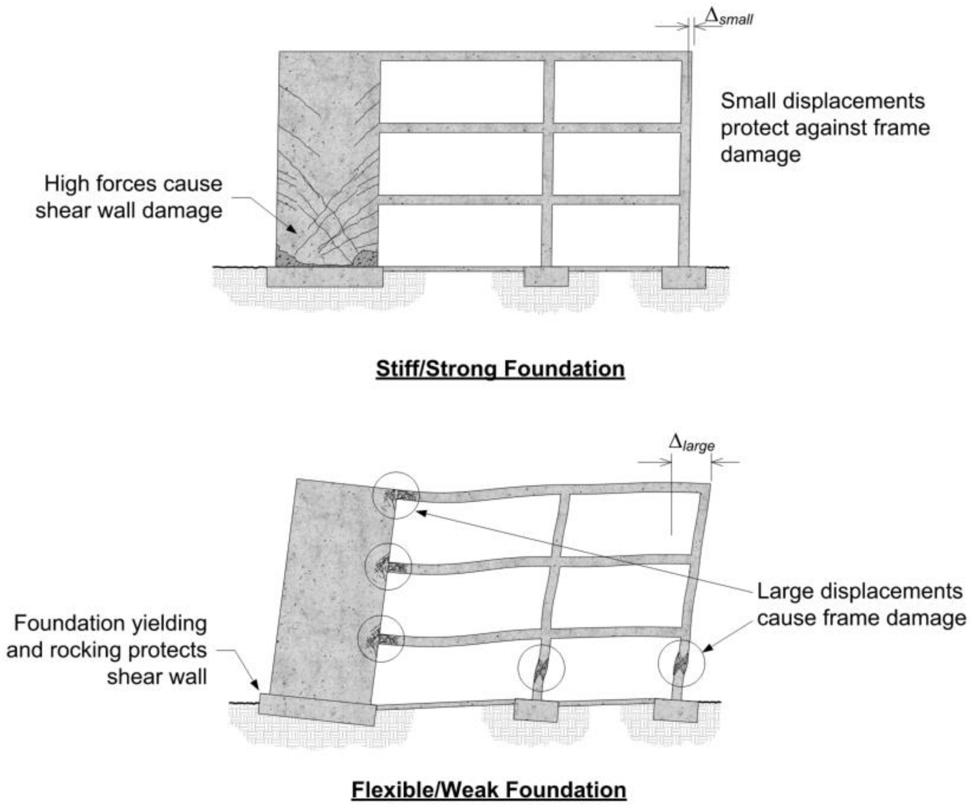


Figure 13 Influence of SSI on structural performance [57]

Prior to undertaking analysis, the geotechnical and structural engineer should agree the geotechnical parameters required and how these are to be applied in analysis. The geotechnical engineer should advise the range of vertical displacement of the foundations expected at ULS loading to inform this discussion. It may be concluded that SSI is unlikely to be critical to the design and thus SSI analysis is not required, or that analysis is required as described in the following sections

10.8.2 Foundation Modelling

Section 3.8 of Module 4 [21] discusses soil foundation structure interaction and the non-linear interaction between soil and foundations. This includes the possible design benefits of using these sophisticated analyses and performance-based design. Routinely applied structural software typically assumes elastic or elastic-plastic response of soil foundation interaction. These elastic analyses are current normal practice. In undertaking these elastic analyses, the designer must be aware of the simplification of the elastic model, i.e. assuming an elastic behaviour of the soil which is likely non-linear and highly variable.

To make some allowance for these simplifications the geotechnical and structural designers should collaboratively undertake sensitivity analyses and challenge the stiffness assumed for load range considered. There should be a feedback loop for the geotechnical engineer to check the assumed stiffness (secant stiffness) for the calculated loads. A possible means of undertaking this collaborative work is discussed below.

The geotechnical engineer should assess the expected load – displacement behaviour of the foundation (“idealised” behaviour in Figure 14) and the range of behaviour (shaded green in

Figure 14). This "idealised" behaviour will likely be non-linear. The range should be assessed by considering how stiff and how soft the ground could be. The possibility of shaking effects softening soils should be considered. The range is often expressed as "one-half and two times the expected soil properties". It is recommended that the range of stiffness be assessed before reporting the appropriate range. This appropriate range may or may not be "on-half and two times". Module 4 [21] provides guidance on assessing load displacement behaviour of foundations.

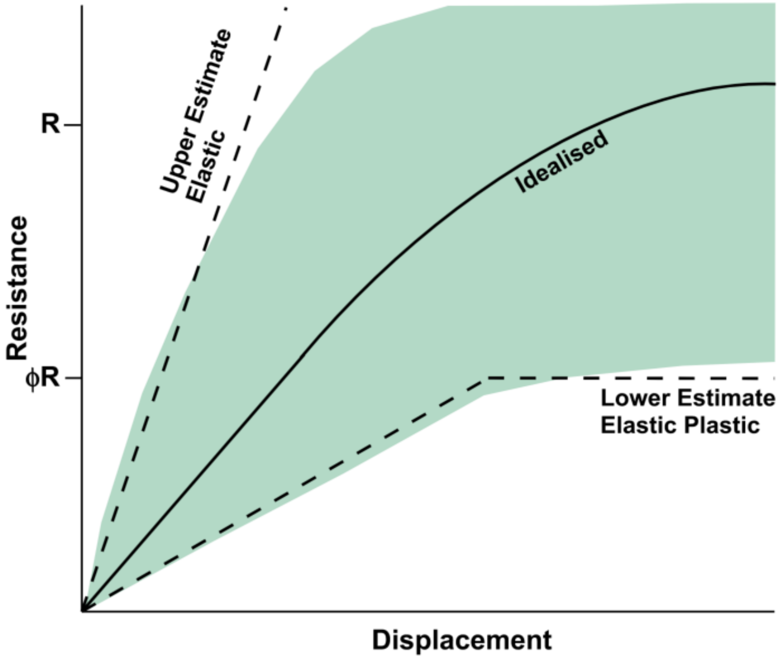


Figure 14 Foundation load displacement behaviour

The geotechnical engineer and structural engineer should jointly agree how this range of foundation load-displacement behaviour is to be modelled in the structural analysis.

The stiffer end of the possible range of foundation stiffness should be considered when assessing the seismic response (stiffer foundation = shorter period = larger spectral accelerations and seismic forces).

The softer end of the possible range should be considered when assessing possible deformations in the structure. Assessment of deformations should also consider possible variability in ground conditions as discussed in Section 10.8.3. Applying seismic loads determined assuming stiff foundations in conjunction with soft foundations to assess foundation deformations may be overly conservative. This needs to be considered by the structural engineer in sensitivity analyses.

The geotechnical engineer and structural engineer should discuss the conclusions of the analysis and what is critical to the building design. This provides the opportunity to check the geotechnical parameters have been applied as intended and to review/revise parameters if they appear unreasonably onerous to the structure.

Springs are typically used in structural analysis models to represent foundation flexibility when SSI effects are being considered. Elastic or nonlinear springs can be used in the structural analysis model depending on the analytical approach that has been adopted for a project.

Elastic soil springs are typically used when foundation displacements are expected to be low and soil yielding is not anticipated. When elastic springs are used the geotechnical engineer should review the loads determined by the structural analysis to check that they remain within the range that can be modelled as elastic i.e. yielding and associated larger displacements are not expected.

Nonlinear soil springs can be used when soil yielding is anticipated. Alternatively elastic springs with an appropriate secant stiffness based on the anticipated foundation deformations can be used in linear analysis models. When elastic springs are used the geotechnical engineer should review the loads determined by the structural engineer to ensure they are consistent with what was assumed when the spring parameters were derived.

Further guidance on modelling techniques can be found in FEMA P-2091 [58] and NZ Assessment Guidelines Section C4 [59].

SESOC Recommendation:	The geotechnical engineer and the structural engineer should jointly evaluate the significance of SSI on building performance. For those projects when SSI is likely to be significant foundation flexibility should be considered when analysis models are prepared.

10.8.3 Consideration of Spatial Variation

Consideration should be given to the spatial variability of the soils mechanical properties, and variability in the tolerance of the structure above to accommodate the resulting deformations, across a site. A process by which this could be considered is summarised below.

The geotechnical engineer and the structural engineer discuss:

- Possible variability of the soils mechanical properties (i.e. foundation stiffness) across the site.
- The distribution of loads applied to the foundations across the building's footprint.
- The buildings tolerance to the range of foundation deformations anticipated.

The geotechnical engineer advises how the soft end and stiff end of the foundation stiffness range could vary across the site and develops a series of potential scenarios in consultation with the structural engineer. It is not generally possible to reliably predict how foundation stiffness could vary across the site. These potential scenarios are proposed to test the structure for possible unfavourable conditions e.g.:

- For a building on pad or piled foundations: One foundation could be stiff and another foundation a specified distance away could be soft.
- For a building on a raft foundation: The subgrade could be generally stiff with a local area of a specified diameter being soft (a soft spot). Or, the subgrade could be generally soft with a local area of a specified diameter being stiff (a hard spot).

In addition, if ground settlement in addition to that due to foundation loading is possible, e.g. due to post liquefaction consolidation this should be assessed by the geotechnical engineer and allowed for in the structural analysis.

When considering potential scenarios, the structural engineer should consider possible distributions of stiffness across the building footprint which could be unfavourable to the building. These could include:

- Soft foundations beneath heavier loaded foundations and stiff foundations beneath lighter loaded foundations.
- Change from stiff to soft foundations across a section of the structure with low tolerance to deformation.

The structural engineer analyses the proposed foundation to assess the performance of the structure for the range of potential foundation stiffness scenarios that could be critical. The geotechnical engineer and structural engineer should review the conclusions from the analysis and identify what is critical to the building design. This provides the opportunity to check the geotechnical parameters have been applied as intended and to review the parameters if they appear unreasonably onerous to the structure.

SESOC Recommendation:	Consideration should be given to the spatial variability of the soils mechanical properties, and variability in the tolerance of the structure above to accommodate the resulting deformations, across a building's footprint.

10.9 Lateral Load Take Out

Lateral load take out involves soil structure interaction and its design will require collaborative work by the geotechnical and structural engineers. Detailed guidance for geotechnical engineers on how to determine the lateral resistance of foundation systems can be found in Module 4 [21].

Seismic shaking is transmitted into buildings via the foundations. This shaking can be amplified by the buildings dynamic response which is fed back down to the foundations. This results in differential shaking between building and ground with associated differential movements and loads. This concept can sometimes be forgotten when traditional force-based design principles are followed.

Lateral movement of foundation elements relative to the ground should be limited to tolerable values to prevent damage to buried service connections and accessways for SLS1and DCLS load levels unless these are specifically detailed to accommodate the anticipated deformations.

Significant lateral movement of foundation elements may be acceptable at ULS load levels provided the foundations have been designed to accommodate the expected deformations and they have adequate capacity to support vertical loads while sustaining the chosen earthquake energy dissipating mechanism in the structure.

A cautious approach is recommended for those buildings located on sloped sites, or which are retaining soil, as these structures will be subject to down slope ratchetting effects and likely larger residual deformations. Consult with the geotechnical engineer.

Sites prone to liquefaction will require special consideration. Consult with the geotechnical engineer.

10.9.1 Shallow foundations

For structures on shallow foundations, provided the foundations lateral capacity is greater than lateral demand the lateral deformations are likely to be tolerable for the ULS case. Strength reduction factors need not be applied provided tolerance to deformations has been considered. Specific geotechnical checks of vertical bearing capacity of the shallow foundations in combination with lateral loads will be required. Imposing a lateral load or overturning moments on a shallow foundation reduces its vertical bearing capacity [21].

Limited technical guidance is available to estimate the magnitude of lateral movement of sliding shallow foundations. Observations of buildings following the 2010/2011 Canterbury and the 2016 Kaikoura earthquakes suggests the magnitude of lateral foundation movement on flat sites is likely to be modest. In many circumstances there will be no need to quantify the magnitude of movement provided the building foundation is designed to move as a single block. For those projects when an estimate of the magnitude of foundation lateral sliding is required an idealised nonlinear analysis model similar to that illustrated in Figure 15 could be used.

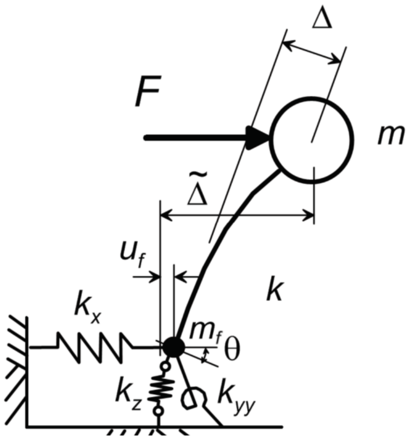


Figure 15 Idealised building model [60]

When shallow foundations have embedded elements (i.e. lift pits) additional checks should be undertaken as detailed in the Section 10.9.2 below to ensure the embedded element(s) have been designed to resist anticipated soil loads.

10.9.2 Deep foundations

For structures on deep foundations, or for structures on shallow foundations with embedded elements, structural checks of the embedded elements under the lateral loading will be required. This will need to consider the relative lateral stiffnesses of the various components providing lateral restraint between ground and structure.

Module 4 (2021) [21] Section 6.3.2 describes 3 methods of analysis for design of deep foundations to resist lateral loads. The method used will depend on the level of complexity and importance of the design and the magnitude of lateral loads to be resisted. Refer to Module 4 for more details on each of these methods:

Method 1: Limiting equilibrium calculation with factor of safety.
A strength reduction factor of 0.3 on passive resistance is suggested. The intent is to limit pile demands such that flexural and shear strengths are not exceeded. This method is likely to only be appropriate in low seismicity zones where lateral design of foundations may not be critical.

Method 2: Quasi static beam spring analysis
Quasi static beam spring analyses are applied to calculate shear and bending actions in piles. This Method is appropriate for most cases. An important part of this method is the distribution of lateral load between the various foundation elements providing lateral resistance. Due allowance should be made for torsion when this is significant. Further information on this load distribution is provided below to supplement the information in Module 4.

Method 3: Numerical time history analysis.
This more complex method would normally only be considered for complex cases (e.g. piled waterfront structures or buildings of high performance (e.g. IL 4 buildings)

An example of Method 2 is illustrated in Figure 16 below for a building that is founded on piles and has a shallow basement. For convenience the piles have been categorised into two groups, ‘Stiff’ and ‘Soft’. Lateral shear vs displacement relationships have been determined for the basement, and the stiff and soft piles. These have then been combined to form the yellow curve which is the total lateral shear vs displacement relationship for the building.

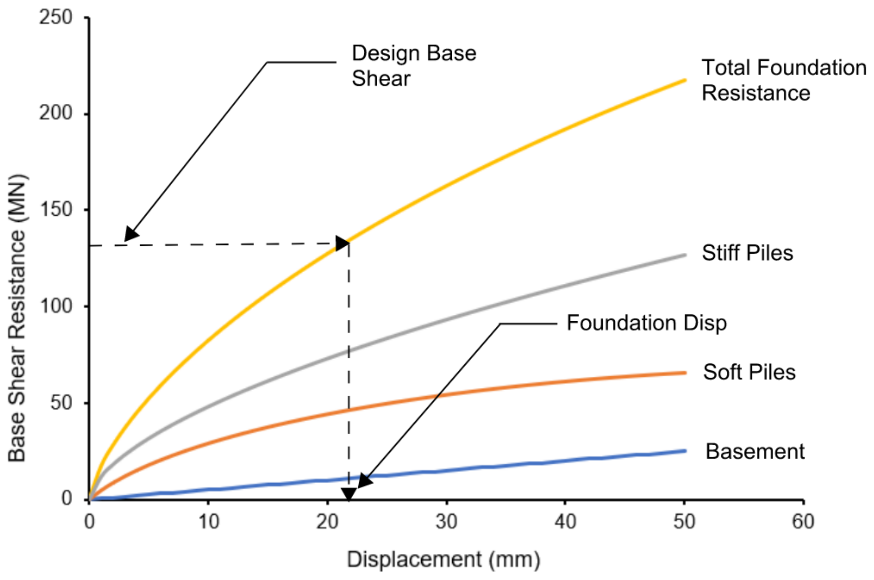


Figure 16 Example of a pseudo-static displacement-based assessment of foundation components.

The displacement of the foundation for the design base shear can then be determined from yellow curve as shown. For this example, the design base shear for the building was 130 MN and the corresponding foundation displacement is approximately 20 – 25 mm. This displacement and the base shear load distribution between the various elements (stiff piles, soft pile and basement) can then be applied in the structural design of each of these elements.

Verification Method Requirement:	Lateral movement of foundation elements relative to the ground for SLS1 load levels should be limited to tolerable values to prevent damage to buried service connections.
SESOC Recommendation:	Lateral movement at ULS load levels may be acceptable provided the necessary geotechnical and structural engineering checks have been completed to ensure the foundation has adequate capacity. This includes bearing capacity of shallow foundations and lateral structural capacity of embedded elements.

10.10 Slabs on Grade

Slabs on grade have performed poorly where they are on soft or liquefiable material. In these cases, they have been subject to severe differential settlement or heaving. Where the liquefaction has been sufficiently severe, liquefied material has in some cases come up through the slabs.

Slabs on grade are seldom critical for seismic performance but in some instances may be required to act as diaphragms to transfer seismic load between the main lateral load resisting elements and the foundations. Where this is the case, similar requirements exist as for regular floor diaphragms as detailed in Section 6.

Design of slabs on grade should take sufficient cognisance of the soil conditions. If there is insufficient ‘crust’ (depth of non-saturated soils) over the liquefiable material, it may be necessary to consider ground improvement, using techniques such as stone columns, dynamic compaction or deep soil mixing. In all cases, geotechnical advice should be sought prior to undertaking soil improvement or repair.

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.

For residential houses designed to NZS3604 [61], recent revisions to the Building Code require the use of Ductility Class E reinforcing steel - this may comprise deformed bars or welded mesh. Unreinforced slabs are not permitted.

However, on liquefiable sites more robust foundations are required. Waffle slabs, rafts, piles, or other suitable foundations should be designed to accommodate the ground deformations expected. For more information refer to ‘Guidance on house repairs and reconstruction following the 2010/2011 Canterbury earthquakes’ [1].

For commercial floor slabs on non-liquefiable sites, traditional slabs reinforced with cold drawn wire mesh and constructed with dowelled control joints and saw cuts at regular centres are still appropriate. The CCANZ publication ‘Concrete ground floors and pavements for commercial and industrial use’ [62] is a useful document for the design of the slab reinforcing and joint spacing. Post-tensioned floor systems have also performed well with little damage noted at any point.

For commercial floor slabs on liquefiable sites, post-tensioned floor systems have in some cases been effective at reducing damage from ground movement. However, for complete mitigation of movements, a low damage solution is likely to come at substantial cost. Site specific advice should be sought from the geotechnical engineer to explore various options for reducing the damaging effects of liquefaction. Consideration could be given to accepting the risk of their damage due to settlement and replacing the slab after a damaging event. The client should be fully involved in selecting the optimum outcome, acknowledging the level of damage they are prepared to accept in relation to construction cost.

Verification Method Requirement:	Unreinforced slabs are not permitted, including for residential construction.
SESOC Recommendation:	For residential slabs in liquefiable areas refer to the MBIE document ‘Repairing and rebuilding houses affected by the Canterbury earthquakes’ [1] and seek geotechnical advice.
Damage Reduction Recommendation:	For commercial slabs in non-liquefiable areas, traditional design using dowelled control joints and reinforced slabs is still appropriate, as is post-tensioning. For commercial slabs in liquefiable areas, seek specific geotechnical advice and involve the client in the decision with respect to expected damage versus construction cost. Post-tensioning may offer some damage reduction in areas of lower predicted movement. If the slab on grade is required to act as a diaphragm, proceed generally as for suspended floor diaphragms. For piled structures consider the cost benefit of a suspended ground floor slab.

10.11 Shallow Foundations

As a result of the liquefaction that occurred under a significant portion of the Christchurch CBD during the 2010/2011 Canterbury earthquakes (even where not evident at the surface), shallow foundations have tended to result in significant differential settlements - particularly between internal and external foundations.

As such, it is unlikely that pad foundations will be used as extensively in the future, unless the sites have an acceptably low liquefaction probability.

Raft foundations performed significantly better during the 2010/2011 Canterbury earthquakes, although residual deformations such as global rotation occurred in many cases, requiring substantial re-levelling works or complete demolition.

Surface loading adjoining existing shallow foundations can induce settlement of the existing foundation. This needs to be considered in conjunction with the geotechnical engineer if new shallow foundations are to be constructed adjoining existing.

10.11.1 Pad Foundations

If shallow pad foundations are to be used, it is important to provide tie beams between all pads to prevent relative lateral movement. When dealing with sites that are susceptible to lateral spreading advice should be sought from the geotechnical engineer when determining the suitability of a pad foundation system and if suitable, design actions for tie beams (refer Section 10.6). Tied together pad or pile systems are more vulnerable to lateral spread damage than raft foundations and are not generally recommended when significant lateral spreading is expected.

In a number of cases hardfill rafts (typically 400-600mm deep) have been used under shallow foundations, in order to reduce the likelihood and magnitude of differential settlement in liquefiable materials. It seems likely that these rafts have helped to disperse bearing pressures and to minimise differential settlement.

SESOC Recommendation:

All shallow foundations must have tie beams or a ground diaphragm between the pads, capable of providing a reasonable lateral tie force. A minimum recommended level of resistance is 10% of the gravity load on the foundation pad element, but not less than 150kN for commercial structures. If a diaphragm is to be used, reinforcement should comply with Section 6 above.

For those sites affected by lateral spreading advice should also be sought from the geotechnical engineer when determining if pad foundations are suitable, and if suitable, design actions for tie beams provided.

Subject to the geotechnical engineer’s recommendations, hardfill rafts may be used beneath the foundations in order to reduce differential settlement, provided that this may only be done for isolated buildings.

10.11.2 Mat and Raft Foundations

Module 4 [21] Section 5.5 describes mat and raft foundations as follows:

Mat foundations distribute concentrated column and wall loads and reduce differential settlement on variable ground. A mat has some flexibility. Raft foundations are a special case of mat foundations that are sufficiently stiff and strong to distribute the entire superstructure load uniformly across the base and to behave as a rigid unit (rare in practice). In reality there is a continuum between a flexible mat and a rigid raft.

Raft foundations generally performed well in the 2010/2011 Canterbury earthquakes. Global settlement and lateral movement have occurred. Out of plane differential settlement can be controlled to some extent with raft foundations but tilting can still occur. Re-levelling of the raft is potentially feasible by resin injection. Raft foundations provide a level of resilience against differential lateral spread.

Mat foundations will be at risk of some differential settlement. It is recommended that raft foundations be used rather than mats, or at least design move as far as practical along the continuum to a raft.

Damage Reduction Recommendation:	All mat foundations are to be made sufficiently stiff and strong to act as far as practical as rafts.

10.12 Deep Foundations

Typically, foundations are considered deep when the depth to breadth ratio is greater than 5 (D/B >5). Deep foundations comprise mainly piles.

Lateral loads are resisted by passive soil pressure acting against embedded foundation elements including piles, pile caps, foundation beams and other structural elements such as basement walls and lift pits etc [21]. The contribution of any friction developed underneath the building should be ignored as this will typically fail quickly with any settlement of the ground.

Piles should be designed to withstand relative lateral movements of intermediate soil layers (kinematic effects) including both transient and permanent lateral movement of the ground (lateral spread) without excessive damage which might compromise the ability of the piles to support the structure above. This design for kinematic effects should be undertaken collaboratively between the structural and geotechnical engineers.

The following sections outline the characteristics of each generic pile type. In all cases, specific geotechnical advice needs to be sought as to the appropriate form of pile for any given site.

SESOC Recommendation:	The contribution of any friction developed underneath buildings with deep foundations should be ignored when evaluating the lateral load capacity of the foundation system.

10.12.1 Settlement of Piled Foundations

Module 4 [21] provides information on settlement of piled foundations. Highlights are discussed here.

Piles which experienced significant settlement during the 2010/2011 Canterbury earthquakes may have simply been overloaded by the earthquake induced axial loads. The Building Code VM4 document permits use of a generic geotechnical strength reduction factor of $\phi_g = 0.8 – 0.9$ for load combinations including earthquake "overstrength" loads, which is much higher than factors typically used for other load combinations. Refer to Section 10.5 above for further guidance.

Pile settlement may also be from liquefaction effects reducing shaft and/or end resistance or as a consequence of post liquefaction settlement below the founding layer. Many parts of Christchurch have dense gravel or sand layers that may be several metres thick but underlain with much looser sands. Deeper liquefaction may not have been considered in the pile design, particularly of older buildings.

Loss of side resistance (skin friction) in piles may occur from pore water pressure increase during shaking, even if full liquefaction does not trigger. Where full liquefaction is triggered above the pile base, all side resistance above may be effectively lost or reversed because of settlement of the overlying strata. In such cases so called "negative skin friction" may contribute to pile settlement.

Unless they are adequately embedded in dense soils, bored cast-in-place piles are perhaps the most susceptible to settlement caused by pore water pressure rise and liquefaction above the base of the pile because the gravity loads are carried initially almost entirely by side resistance. If this mechanism is overloaded, the pile will settle until the end bearing mechanism is mobilised (which could be as much as 5% – 10% of the pile diameter). This can potentially be exacerbated if poor construction has left a zone of disturbed material at the base of the piles.

Piles should not be founded within different bearing strata at different depths beneath a building. Doing so might result in differential pile settlement during shaking, particularly if liquefaction of intermediate sand layers occurs.

Cyclic axial loading during the earthquake may cause loss of capacity and settlement especially for piles that carry only light gravity loads and rely mainly on side resistance.

10.12.2 Interconnectivity of Pile Caps

If piled foundations are to be used, it is important to provide tie beams and/or a full floor diaphragm between all pile caps to prevent relative lateral movement. When dealing with sites that are susceptible to lateral spreading advice should be sought from the geotechnical engineer when determining design actions for tie beams or floor diaphragms (refer Section 10.6).

SESOC Recommendation:	All pile foundations must have tie beams between the pile caps capable of providing a reasonable lateral tie force. A recommended level of resistance is 10% of the gravity load on the pile or pile group, but not less than 150kN for commercial structures. For those sites affected by lateral spreading advice should also be sought from the geotechnical engineer when determining design actions for tie beams.

10.12.3 Driven Piles

Module 4 [21] Section 6.1 discusses the types of deep foundations (driven, bored and screw) commonly used in New Zealand practice.

Driven piles used to be the norm, but this has reduced over recent years due to concerns about vibration and noise. Driven piles have benefits over bored piles of less settlement potential (refer Section 10.5.1) and are therefore use of driven piles is encouraged in a recommendation of the Canterbury Earthquakes Royal Commission. Corrosion allowance for steel piles should be to SNZ TS 3404:2018 [63].

10.12.4 Bored Piles

Bored piles now take two basic forms – conventional bored piles, or continuous flight auger (CFA) piles. The former generally require casing in order to avoid collapse of the sides of the excavation. CFA piles avoid collapse by displacing the soil as it is extracted with concrete under pressure.

One of the main potential shortcomings of bored piles is the potential for settlement at the tip due to compaction of the disturbed soils. CFA piles may avoid this problem, but there is still a practical limit as to the length of pile achievable. In many cases, CFA piles will not be able to reach lower founding levels. For CFA piles the reinforcing cage is plunged into the wet concrete after pouring. This may limit the amount of reinforcing which can be installed.

Another significant shortcoming of bored piles in soils subject to liquefaction and pore water pressure changes is that initially most of the building weight is carried by side resistance (skin friction). Following an earthquake, side resistance may be lost due to liquefaction (or worse, negative side resistance occurs) and the pile may settle significantly as load is transferred to the base resistance mechanism which is comparatively soft (vertical displacements in the order of 5% - 10% of the diameter of the pile base are typically required to mobilise the base resistance mechanism of a pile).

For the case of large diameter concrete belled piles this can translate to vertical deformations in the order of 100 mm. Support deformations of this magnitude can have a significant impact on global building performance and it is important therefore that this behaviour is accounted for in the building design.

10.12.5 Screw Piles

Screw piles rely on the enhanced bearing of the steel flights that are attached to the pile shafts. For end bearing piles, there is normally a single flight at the tip. Multiple flights can be used along the shaft to distribute load through the bearing stratum.

Designers should be aware that the flights must deflect significantly for the pile to develop its full capacity and therefore have relatively high settlement potential. Compared to a driven or bored pile in similar ground conditions a single helix screw pile will have a greater proportion of its capacity derived from end bearing. This can make screw piles more vulnerable to settlement particularly in end bearing conditions subject to liquefaction or increase in pore water pressure effects. These settlement and reduced capacity vulnerabilities should be considered in design.

Screw piles are routinely static load tested providing valuable information to inform design. If the ground conditions in which the screw piles are founded are subject to liquefaction or increase in pore water pressure effects, the results of static load tests need to be reduced before applying to a seismic design case. Consideration of corrosion should also be made, using the provisions of SNZ TS 3404:2018 [63], given that the greatest stress is at the root of the weld of the flight to the shaft.

10.12.6 Pile Depth

The use of piles relies heavily on the identification of a sufficiently good bearing layer at a consistent depth. If there is doubt about the integrity of a bearing layer (for example where a lens of material may taper off part way across a site), then deeper layers may need to be identified. If piles are required to resist tension uplift loads, allowance must be made for the reduced capacity of potentially liquefiable upper layers.

10.13 Mixed foundations

Mixed foundations, where part of the building was supported on deep piles and part on shallow foundations, performed poorly in the Canterbury earthquake sequence, because of complex dynamic interactions resulting in differential movements between the two systems. Mixed foundation systems are to be avoided [21].

Interconnecting adjacent structures should generally be avoided. If this cannot be avoided employing similar foundations for the two structures and tying them together should be considered.

11 NON-STRUCTURAL ELEMENTS

11.1 General

The CERC has made several recommendations regarding the protection of life from hazards created by non-structural elements. In particular, recommendation 2.64 states:

> *In designing a building, the overall structure, including the ancillary structures, should be considered by a person with an understanding of how that building is likely to behave in an earthquake.*

Non-structural elements and/or their supports are often not designed by the structural engineer responsible for the overall building design. While this is often a contractual matter, it is nevertheless important that the structural engineer for the building provide, at the minimum, sufficient information for the designers and suppliers of secondary elements to ensure that their systems are compatible with the overall building behaviour. One way of doing this is to ensure that a comprehensive Design Features Report is supplied, recording the relevant information for each limit state in an unambiguous way. A sample Design Features Report is available for SESOC members on the website.

SESOC Recommendation:	A Design Features Report (DFR) should be provided for all significant buildings, providing sufficient information so that the designers of non-structural elements and their supports are aware of the building behaviour and expected performance. This should record relevant seismic design criteria and expected displacements that should be allowed for at each limit state.

11.2 Non-Structural Seismic Restraint Specialists

Historically the design and co-ordination of the seismic restraint of non-structural elements (i.e. building services, partitions, ceilings, facades etc) has been undertaken during the construction phase. This approach can lead to poor project outcomes including non-compliance, poor performance in earthquakes and re-work [64].

It is recommended project teams should seek advice from specialist non-structural seismic restraint specialists early during the design phase of projects e.g. at the same time or soon after the engagement of fire engineering, structural and building services consultants. Early engagement of specialist non-structural seismic restraint specialists will enable the design and detailing of non-structural elements to be co-ordinated during the design phase and ensure appropriate clearances are provided thereby minimising the need for design changes during the construction phase.

SESOC Recommendation:	Advice from non-structural seismic restraint specialists should be sought early during the design phase of projects. This is to enable the design and detailing of non-structural elements to be

co-ordinated and minimising the need for disruptive design changes during the construction phase.

11.3 Reoccupation Considerations

For those projects where reoccupation of the building following DCLS level of shaking has been identified as a priority, the project team should establish which non-structural components are required for the building to be reoccupied. Generally this will require input and collaboration across the entire design team to ensure all of the necessary building components are identified.

Some building services equipment is acceleration sensitive and may require seismic verification to ensure reoccupation timeframes can be met. It is noted that not all building services equipment have the same criticality and this should be considered when the need for seismic verification is evaluated.

It might be acceptable to permit some non-structural components to sustain damage during DCLS level of shaking provided those components are not required for reoccupation, can be repaired within acceptable timeframes, or other short term remedial measures can be implemented while necessary repairs are completed. Otherwise, damage to these components should be limited to the extent that they can continue to function as necessary for reoccupation of the building.

Damage Reduction Recommendation:	Non-structural components required for reoccupation of the building should be identified by the wider project design team. Damage to these components should be limited to the extent that they can continue to function as necessary for reoccupation of the building, or they can be repaired within acceptable timeframes, or other short term remedial measures can be implemented.

11.4 Parts Ductility Factors for Non-Structural Elements

For those projects where reoccupation of the building following DCLS level of shaking has been identified as a priority, non-structural components necessary for the reoccupation, and their related seismic restraints, should be designed to remain elastic. In this instance it is recommended the design actions on the non-structural elements should be determined using an NZS 1170.5 parts ductility factor, μ_p, = 1.0.

Damage Reduction Recommendation:	When determining design actions for the DCLS μ_p, = 1.0 should be used.

11.5 Restraint of Non-Structural Elements to Lightweight Roofs

When building services equipment, suspended ceilings and partitions are to be seismically restrained by lightweight roofs special consideration is required to avoid overload of the roof structure. It is important an appropriate allowance for the weight of these elements is made when the roof is designed by the structural engineer and details of this are recorded in the Design Features Report (DFR).

When designing and detailing the restraint of non-structural elements, the non-structural seismic restraint specialist should ensure the base structure has adequate capacity to resist the design actions associated with the non-structural elements.

The practice of relying on the out-of-plane capacity of light gauge purlins to resist seismic restraint design actions is not recommended (refer Figure 13). Light gauge purlins generally have little ability to resist out-of-plane loads and the light gauge steel sheet has a low bending capacity to resist connection forces not applied through in-plane shear in the major axis of the purlin. Additional elements are often required to transfer horizontal brace forces into main roof bracing structure

Figure 17: Connecting seismic restraint braces to light gauge purlins

SESOC Recommendation:

An appropriate allowance for the anticipated weight of non-structural elements should be made when light weight roofs are designed, and this allowance should be recorded in the DFR.

The non-structural seismic restraint specialist should then ensure the base structure has adequate capacity to resist the design actions associated with non-structural elements.

Reliance on the out-of-plane capacity of light gauge purlins to resist seismic restraint design should be avoided.

11.6 Restraint of Non-Structural Elements to Base Structure

For those projects when building services equipment, suspended ceilings and partitions are to be seismically restrained to the underside of suspended floors, prying effects associated with proprietary brackets shall be considered. Prying in proprietary brackets can amplify anchor tension forces by 1.5 to 2.7 times due to the geometry and thickness of the brackets. It is a requirement that seismic restraint designers account for prying when detailing connections, and the effects of combined shear and tension on anchorages are considered.

Verification Method Requirement: The effects of prying and combined actions shall be considered when detailing connections in seismic restraint systems.

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