

INTERIM DESIGN GUIDANCE

DESIGN OF CONVENTIONAL STRUCTURAL SYSTEMS FOLLOWING THE CANTERBURY EARTHQUAKES



Version No. 10 – September 2019

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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 Canterbury earthquakes and anticipated changes to the NZ Building Code.

As at April 2019, changes made to the NZ Building Code have superseded much of the original content of this document. Nonetheless, it is felt important to revise, rather than withdraw this document for two reasons:

1. Some aspects of the document remain current and relevant. Revision and republication makes clear which parts fall in this category, and
2. On the other hand, it is necessary to avoid outdated information retaining implied currency through lack of a clear statement to the contrary. To achieve this aim, superseded information is struck out in this edition of the interim guidance rather than being simply deleted.

This revision has not aimed to update technical information in the guidance except to the extent implied above.

It is additionally emphasised that the marking of information as superseded in this guidance does not connote that the subject issue is no longer important. Particularly where entire sections have been superseded it is instead generally the case that the issues now require mandatory consideration according to Standards and/or Building Code Verification Methods.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

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:

- Revision 1, Internal to SESOC Mancom
- Revision 2, Draft for review by SESOC Mancom, NZSEE, NZGS, 7 Nov, 2011
- Revision 3, Draft update, 19 December 2011
- Revision 4, Submitted to Royal Commission, released 22 December 2011
- Revision 5, Internal to SESOC Mancom
- Revision 6, internal for review by CERC
- Revision 7, internal for review by SESOC, 4 May 2012.
- Revision 8, For formal review by SESOC membership, 18 September 2012
- Revision 9, Minor corrections and typos, 26 March 2012
- Revision 10, Amended to remove content superseded by Standards, Verification Methods, or other documentation as at mid-2019

1 INTRODUCTION

Many observations have been made of the performance of conventional structural systems following the Canterbury Earthquakes. In general, it appears that the most modern structures (post-1995) have performed acceptably. A further observation is that buildings which have been well conceived, well designed, well detailed, and then well constructed have performed well, irrespective of their age.

However, some types of structures have been found to perform poorly and some details have been 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 earthquakes during the Canterbury rebuilding. In the current (2019) 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 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:-

- (a) comprises 2 or more storeys; and
- (b) 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. Civil structures are excluded also.

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 has been prepared initially by Holmes Consulting Group, and has been offered to SESOC for adaptation, with subsequent review by

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

At the request of the Royal Commission, it has also been 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 involved in particular in the preparation of designs for the Canterbury area, 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

2.1 Seismic Loads

New buildings in New Zealand are designed to conform to the compliance documents of the New Zealand Building Code, notably B1¹. The NZBC in turn sits beneath the Building Act². B1 cites a number of documents as verification methods or acceptable solutions, commencing with the loading standard, AS/NZS1170³. The performance objectives are currently set in the loadings 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 3101⁴, and the Steel Structures Standard, NZS 3404^{5,6}. 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.

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.

Moreover, there is concern that buildings designed to be nominally ductile ($\mu = 1.25$) or for elastic response ($\mu = 1$) 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 $S_p < 1$ should be used where no capacity design has been completed.

SESOC Recommendation: Ideally all structures should be designed using a capacity design approach, regardless of the design seismic load level adopted. This may not be practical in all circumstances. Where capacity design is not used OR sufficient resilience cannot be demonstrated, designers should default to $S_p = 1$ (but refer also to Section 4.2).

2.2 Design Approach

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 have been high. Shaking intensity from all of the major events has exceeded SLS limits, so it has not to date been possible to assess whether the SLS performance objectives have been met, but in general, it could be observed as follows:

- For contents or non-structural elements that are force controlled (such as plant and equipment, or contents), high floor accelerations are most damaging, therefore stiff structures are generally undesirable.
- For contents or non-structural elements that are deformation controlled (such as partitions or cladding), high displacements are undesirable.

This means that in general, flexible structures are likely to have more damage to non-structural elements than stiff buildings, but stiffer buildings may result in more damage to unrestrained contents. As general guidance, damage to partitions may be assumed at drifts in excess of 0.3-0.5%. It is recommended that engineers discuss performance criteria with their clients, noting that the introduction of an intermediate "Damage Control Limit State"^{7,8} (which may correlate to SLS2) between SLS1 and ULS, may be an appropriate outcome. The Damage Control Limit State (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: Unless a building contains high value and/or sensitive equipment and processes, designers are advised to use stiffer lateral load resisting systems.

2.3 Resilience

The NZBC expectations for building performance are stated in AS/NZS1170.5⁹. The commentary notes performance expectations as:

- (1) *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*
- (2) *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.

~~In some instances, observations from the earthquakes have suggested that the implied levels of resilience are not always achieved. This may be because elements requiring added resilience are not implicitly covered by the ductile detailing provisions, or because the provisions themselves have proved to be inadequate. A simple example is stairs, where the requirement to consider support of the stairs under the greater imposed displacements is not explicitly stated, and for which the inter-storey drifts are otherwise only at the ULS level.~~

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 AS/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, such as floor seatings and stairs. ~~Hence it is proposed that a multiplier of 1.5 is used for forces, 1.5/ S_p for displacements of such elements. Note however that, where there are requirements in the materials standards to check material strains, the strain limits set already include allowance for the increased drift and S_p factor, so there is no need to use any further multiplier.~~

In the case of buildings designed to IL3 or IL4, the margin of 1.5 to 1.8 should be relative to 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, use of $R=1.3$ and 1.8 respectively are to be maintained for all levels of seismicity.

content below now superseded by Verification Methods
~~Where the margin between ultimate limit state and collapse is not specifically addressed by the materials or loading standards as noted herein, a multiplier of 1.5 or $1.5/S_p$ is to be applied to ensure sufficient resilience, for forces and displacements respectively. This is not to apply to the checking of material strains against code limits, for which no multiplier is required.~~

2.4 Building Configuration and Redundancy

Observation shows that in general, regular buildings have behaved better than irregular buildings. 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.

Another observed issue is the inability of some regular orthogonal systems to reverse 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).

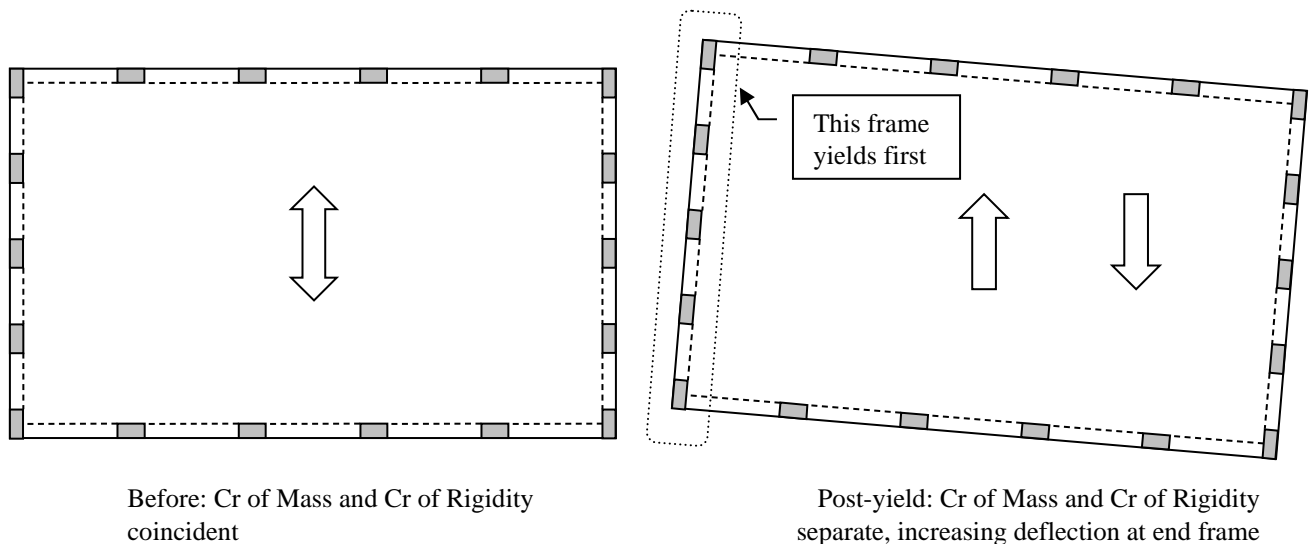


Figure 1: Building Plan indicating possible torsional mode development

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. ~~It is recommended that frame stiffnesses are adjusted so that each of the orthogonal lateral load resisting systems has approximately equal torsional rigidity.~~ 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. ~~Clearly this could equally be treated by introducing more stiff elements into the softer side, or by softening the stiffer direction, in order to even up the torsional resistance.~~

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.

~~In two-way frame systems, it is recommended that each direction contribute approximately equally to the torsional resistance of the building.~~

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.

2.5 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 Compliance¹⁰, 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.

Verification Method Requirement: Where proprietary systems have been accepted on the basis of a recognised New Zealand appraisal in accordance with the DBH Guidelines, they should only be used strictly in accordance with the limitations of the appraisal.

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 stories 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. A variation of +/- 50% in soil properties is recommended, in accordance with ASCE 7-05¹¹.

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 (cl 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, ~~taking account of the possibility of increased drift from earthquakes larger than the design (ULS) case.~~

~~This is reflected in the CERC recommendation, adopted by SESOC:~~ 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. There are also a number of errors in the provisions included that may make their application difficult.

SESOC Recommendation: Appropriate allowance should be made for ratcheting 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.

~~Where ratcheting cannot be suppressed, additional allowance for the resulting P-delta effects should be made using amplified 1.5 times the ULS drift.~~

3.4 Second Order Actions

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

- P-delta effects. Note that the Loadings standard NZS 1170.5 provides (cl 6.5) provisions that require the explicit consideration of P-delta.
- Beam elongation. Beam elongation may result in increased column design actions, particularly at the level immediately above the column base hinge level. This will also result in compression of the beam at this level. The impacts of beam elongation are now required to be considered

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

~~Where ratcheting cannot be suppressed, additional allowance for the resulting P-delta effects should be made using amplified 1.5 times the ULS drift.~~

4 CONCRETE WALLS

Given the desire to design stiffer buildings (as outlined above), it is likely that wall structures will be popular. However, the performance of wall structures in the Canterbury Earthquakes has not been as good as expected and we need to improve the future performance of these structures.

4.1 ~~Singly Reinforced Walls~~

content below now superseded by Verification Methods

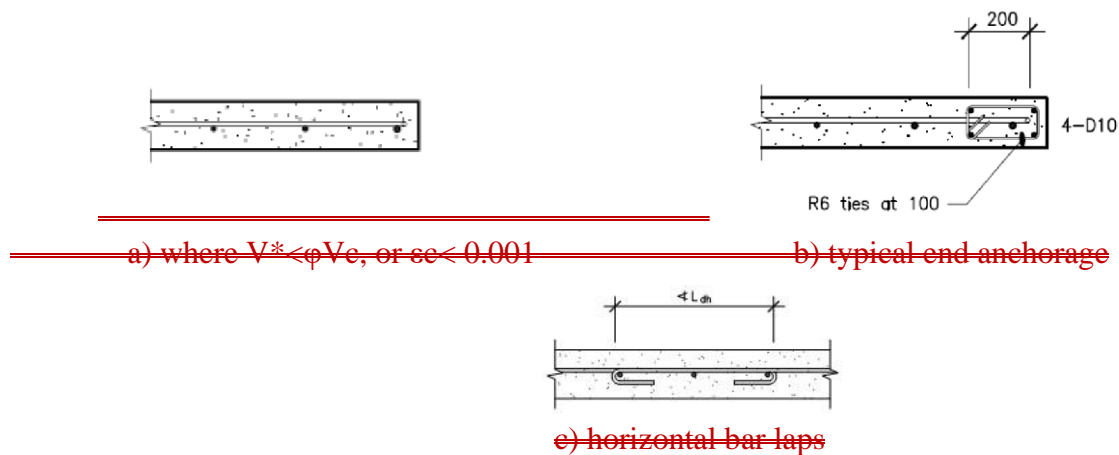
~~Singly reinforced walls lack the robustness to suffer significant damage while retaining lateral stability. As such, they should be designed for the nominally ductile ($\mu \leq 1.25$) actions. It is considered that singly reinforced walls should only be used for low-rise development, or where axial loads are low (i.e. when an alternative gravity load carrying system exists).~~

~~In some cases, use of non-ductile mesh in wall panels has been noted. Even where walls have been designed for fully elastic actions, this is inappropriate as these materials do not have sufficient resilience for accidental overload situations.~~

~~Where the horizontal steel is not required (other than minimum steel requirements) to resist in-plane shear (i.e. $V^* \leq \phi V_c$) at the ULS, or spalling of cover concrete is not expected (compressive strain, $\epsilon_c < 0.001$, assuming a triangular stress block), no special detailing of the horizontal reinforcing anchorage is required.~~

~~In all other situations, the ends of wall segments should be locally confined to ensure development of the horizontal reinforcing. Specific design of the confining steel is not required — R6 stirrups with 4 no. D10 bars will suffice — refer to Figure 2 below.~~

~~It is recommended that horizontal bars should be lapped with overlapping 180-degree hooks, anchored around vertical reinforcement — refer to Figure 2.~~



~~Figure 2: Detailing of singly reinforced walls (note hook anchorages must be used on horizontal reinforcement).~~

Verification Method Requirement:	Non-ductile reinforcement or mesh shall not be used in any walls, regardless of assessed ductility demand.
SESOC Recommendation:	Singly reinforced walls should be designed for nominally ductile ($\mu = 1.25$) actions. Typically provide closed stirrup cages at each end of a wall segment to confine the anchorage of the horizontal reinforcing steel (refer to Figure 2) unless $V^* \leq \phi V_e$, or $\epsilon_c \leq 0.001$.
	Lap horizontal bars with 180 degree hooks anchored around vertical reinforcement as per Figure 2.

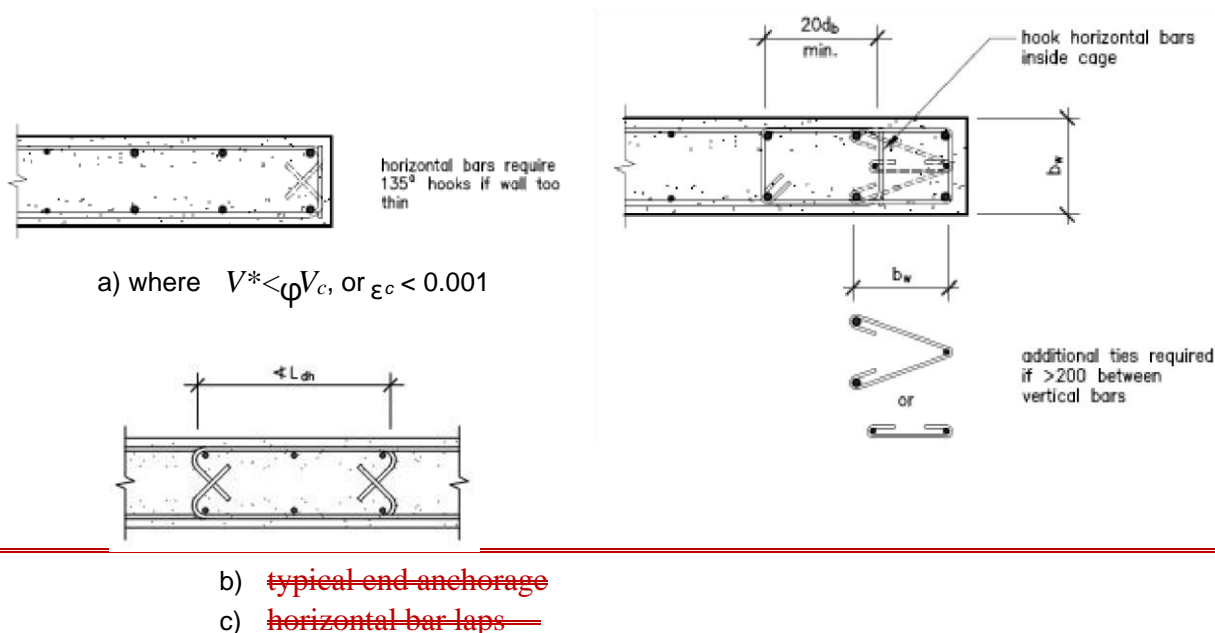
4.2 Doubly Reinforced Walls

~~**content below now superseded by Verification Methods**~~

~~Provided the reinforcing steel was well confined, doubly reinforced walls generally performed well in the Canterbury Earthquakes. However, local bar buckling was a common problem as highlighted in Section 4.4.~~

~~Where the horizontal steel is not required (other than minimum steel requirements) to resist in-plane shear (i.e. $V^* \leq \phi V_e$) at the DBE, or spalling of cover concrete is not expected (compressive strain, $\epsilon_c \leq 0.001$, assuming a triangular stress block), no special detailing of the horizontal reinforcing anchorage is required.~~

~~In all other situations, the ends of wall segments should be locally confined to provide development of the horizontal reinforcing — refer to Figure 3 below. It is recommended that horizontal bars should be lapped with overlapping 180 degree hooks, anchored around vertical reinforcement — refer to Figure 3.~~



~~Figure 3: Detailing of doubly reinforced walls~~

~~SESOC~~ ~~Refer to Section 4.4 for requirements to address local bar~~
~~Recommendation:~~ ~~buckling~~

~~Typically provide closed stirrup cages at each end of a wall segment to confine the anchorage of the horizontal reinforcing steel (refer to Figure 3) unless $V^* < \phi V_c$, or $\epsilon_c < 0.001$. Lap horizontal bars with 180 degree hooks anchored around vertical reinforcement as per Figure 3.~~

4.3 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.

The minimum wall thickness is defined by the reinforcing bar size used. NZS3101:2006 (clause 11.3.11.2) 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 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 4.8 below.

Verification Method Requirement: Wall thicknesses should be at least 7 d_b , increasing to 10 d_b in yielding regions.

SESOC Recommendation: Minimum 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 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 > 40 mm, no more than 300mm concrete cast below the starter)
 2. Wall thickness limited by precast panel splices – refer to Section 4.8 below
 3. Unless greater wall thickness required for development of floor starters

4.4 ~~Local Bar Buckling~~

content below now superseded by Verification Methods

~~Buckling of reinforcing steel in concrete walls with confined boundary regions was observed in several buildings (refer Figure 4, for example). Although some older walls may have had no particular confinement requirements, a significant number of walls that had well confined end regions also had buckled bars between the confined zones. In some cases this also resulted in horizontal steel being exposed, losing bond.~~

~~There is an anomaly here. The steel in the middle regions of walls may not be in compression when analysing wall sections for ultimate limit state in either direction of loading. However, if it yields in tension with any appreciable strain, it must yield again in compression as the load reverses, before it can again yield in tension.~~

~~Another concern is that in extreme shaking, walls may pick up increased axial loads from other elements of adjacent structure, due to elongation effects in the wall. Although this may not be quantifiable, the possibility underscores the need to reconsider confinement of intermediate bars.~~

~~NZS3101:2006 requires confinement to be provided throughout walls where the longitudinal reinforcing content exceeds 1.0% by area (clause 11.3.11.5). Note that this applies to the entire wall section and is NOT just limited to plastic hinge or boundary regions. More restrictive limits apply in yielding regions (defined by clause 11.4.6). However, confinement is generally only required in the outer portion of the wall, beyond the neutral axis. Unless~~

~~there is unusually high axial load, the neutral axis is unlikely to exceed half the depth of a wall.~~



~~Figure 4 Buckling of concrete wall with confined boundary regions~~

~~It is believed that providing confinement of the central portion between boundary regions will improve the integrity of walls subjected to high levels of ductility.~~

~~Verification Method Requirement:~~

~~For all walls (ductile or nominally ductile), confining steel should be provided throughout in accordance with NZS3101:2006, clause 11.3.11.5.~~

~~In yielding regions additional confining steel should be provided in boundary regions in accordance with NZS3101:2006, clause 11.4.6.~~

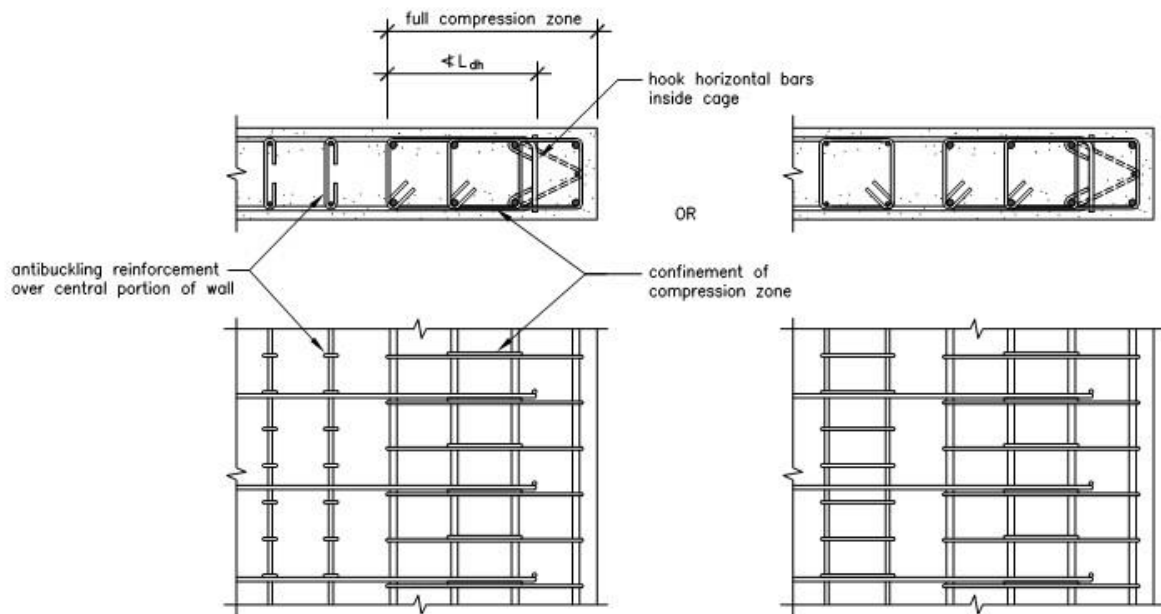
~~SESOC Recommendation:~~

~~For nominally ductile walls, full anti-buckling and confinement (NZS3101:2006, clause 11.4.6) should be provided over the full length of the compression zone unless it can be shown that the wall has sufficient capacity to resist 1.5 times the ULS forces without yielding any bars.~~

~~For the ductile detailing length of ductile walls, transverse reinforcement shall be provided over the full wall length as follows;~~

- ~~• Confinement of the boundary regions shall be provided in accordance with NZS3101:2006, clause 11.4.6 modified to provide confinement over the full length of the compression zone.~~

- ~~Transverse reinforcement in the central portion of the wall shall satisfy the anti-buckling requirements of NZS3101:2006, clause 11.4.6.3, as shown in Figure 5.~~



~~Figure 5: Confinement requirements for doubly reinforced walls~~

4.5 ~~Global Wall Buckling~~

~~**content below now superseded by Verification Methods**~~

~~Global buckling should be considered in particular lateral flexural torsional buckling effects for slender walls. As a general rule of thumb, lateral buckling should be checked for all walls with h/t ratios in excess of 20.~~

~~NZS3101:2006 provides simplified methods for determining the limiting height to thickness ratios for concrete walls in clause 11.3.5 (singly reinforced walls), clause 11.3.6 (doubly reinforced), and clause 11.3.7 (high axial loads). More restrictive height to thickness ratios apply in yielding regions (defined by clause 11.4.2).~~

~~Verification Method Outside plastic hinge regions for slender walls ($h/t > 20$, where Requirement: h is the storey height), wall buckling should be checked in accordance with clauses 11.3.5, 11.3.6, and 11.3.7. In yielding regions wall buckling should be checked in accordance with clause 11.4.2.~~

~~The CERC has recommended that walls should be subjected to a further check where the axial load exceeds minimum levels. This is assumed to apply only to ductile walls, i.e. $\mu > 1.25$.~~

~~SESOC Where there is a ductile detailing length in the wall and the Recommendation: axial load ratio $N/A_g f_c \geq 0.1$, the ratio of clear height to thickness should not exceed the smaller of 10, or the value derived from clause 11.4.2 in NZS3101:2006 (CERC R2.44)~~

4.6 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 6).



Figure 6: Fractured bars in lightly reinforced wall

Minimum reinforcing steel contents for walls are specified in NZS3101:2006 as a function of the 28 day concrete strength. While lower bound 28 day concrete strengths of around 30MPa are assumed for design, in reality the mean strength (further increased by age hardening and dynamic strength enhancement) could be anywhere up to 2-3 times larger. As a result, designers should specify a minimum and maximum concrete strength to be supplied for certain elements.

Following consultation with ready mix suppliers it is proposed to specify a maximum strength of double the specified minimum strength. This should give the suppliers a reasonable margin to work within. ~~It is also proposed to determine minimum steel contents using 2.5 times the specified minimum strength to allow for further strength development with time. The equation of clause 11.3.11.3 (e) has been modified in the requirement below.~~

In addition, note that 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.

content below now superseded by Verification Methods

~~These minimum reinforcing requirements apply to walls where bar yielding is expected at ULS. In walls where yielding of reinforcement is not expected, the minimum reinforcement requirements of the Standard are still applicable~~

~~Minimum reinforcing requirements for columns are not currently dependent on concrete strength and the appropriateness of the requirements for beams is still being considered.~~

~~SESOC Recommendation: Where there is potential inelastic demand at ULS (i.e. $c_s > c_y$ at any section), the minimum reinforcing content calculation of NZS3101:2006 (clause 11.3.11.3) shall be amended in accordance with the formula below, using the specified 28 day strength:~~

$$\rho_n \geq \frac{0.4 \sqrt{f'c}}{f_y}$$

~~and ρ_n is calculated for the gross dimensions of the concrete member.~~

~~A clause defining minimum ($f'c$) and maximum ($2 \times f'c$) acceptable concrete strengths shall also be included in the Specification.~~

4.7 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) 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 has on piled foundations, 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

4.8 Precast Panel Splices

Failures of precast panel splices have been observed in the Canterbury Earthquakes.

The concrete standard NZS3101:2006 (11.4.8.1) requires that splicing in potential yielding regions should be avoided, but that splices must be staggered. It effectively precludes use of grouted splices in potential plastic hinge regions. Such splices may only be used in protected zones or in nominally ductile or elastically responding walls.

The concentration of tension forces at panel splices involving Drossbach ducts results in the strain hardening being concentrated right at the panel joint. This can lead to premature bar fracture. To mitigate this, it is recommended that the splice bar is de-bonded as shown in Figure 7.

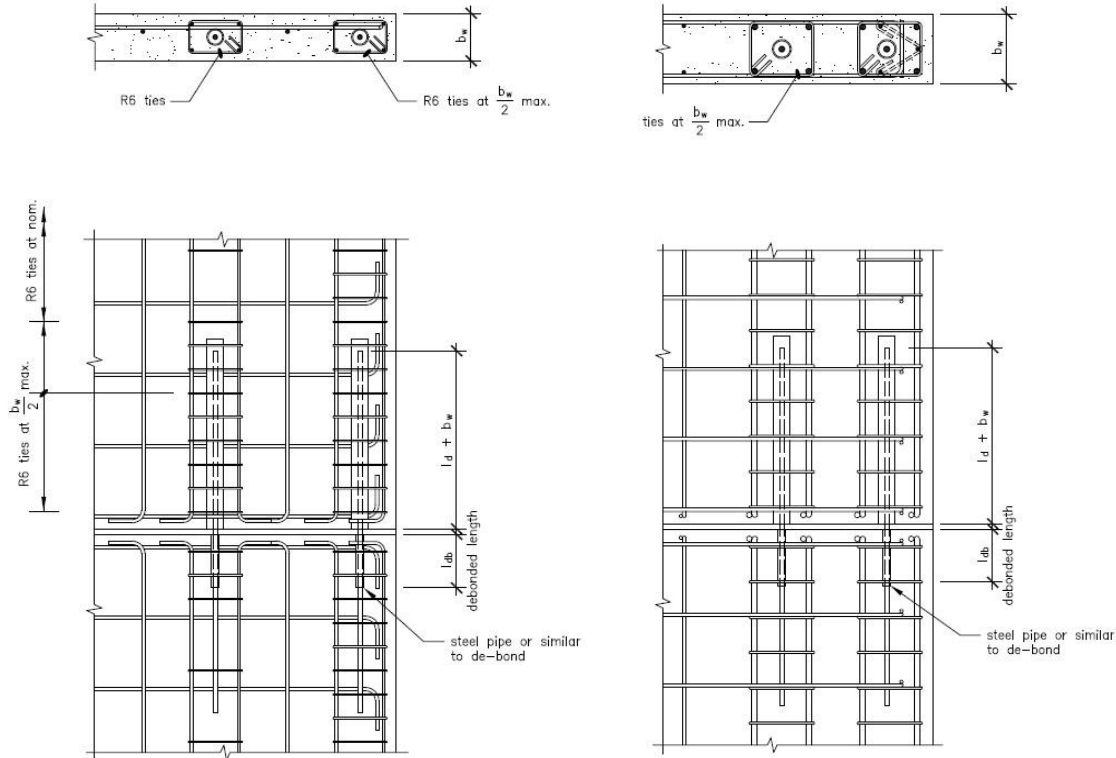


Figure 7: Typical grouted duct details

The large diameter Drossbach ducts also result in a reduction in the concrete section area and a subsequent weakness develops. Thicker panel sizes are therefore recommended for precast

panels (as outlined in Section 3.3 above) and all ducts should be confined with stirrups as shown in Figure 7. The area of the stirrups should be calculated as:

$$\frac{A_{tr}}{s} = \frac{d_b f_y}{48 f_{yt}}$$

A simplified calculation of the de-bonded length to be provided can be derived from the wall geometry and expected wall rotation as shown in Figure 8 below;

De-bonded length,
$$L_{dh} = \Delta_w \frac{L_w}{0.025 h_w}$$

Where Δ_w is the expected wall displacement at the ULS.

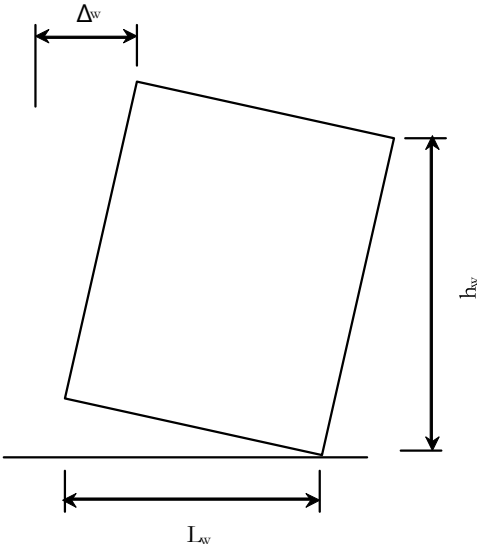


Figure 8: Simplified calculation of debonded length

~~Verification Method: Grouted splices may not be used in the yielding regions of ductile or limited ductile walls.~~
~~Requirement: ductile or limited ductile walls.~~

SESOC Recommendation:

Precast panel splices should allow for de-bonding of reinforcement where yielding is expected.

Grouted splices should not be used in the yielding regions of ductile or limited ductile walls.

Drossbach ducts must be fully confined.

4.9 Precast Panel Embedded Anchors

Failures of precast panel connections have been observed in the Canterbury Earthquakes.

Cast-in inserts (with and without tie bars) have been observed to pull out of the face of precast panels. The tie bars are not of sufficient diameter or length to provide effective restraint of the cast-in insert.

SESOC Recommendation: Shallow embedded connections should not be used for ~~primary~~ structural load paths.

4.10 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. In any other cases such frames should have detailing to accommodate $1.5/S_p$ times the ULS drifts. Note that the material strain limits in the Standard are already calibrated to include the effects of this drift so this factor need not be included when checking material strains.

This may be achieved though use of pinned joints, but it is noted that a true pin is difficult to achieve in a concrete structure. It is recommended that designers use the detailing provisions in Section 5 below to ensure that the required level of resilience is achieved.

SESOC Recommendation: All gravity frames and members in wall structures shall be detailed to accommodate $1.5/S_p$ times the ULS drifts. Gravity structure detailed in accordance with relevant material Standards will achieve this level of performance without specific consideration being required.

5 CONCRETE MOMENT RESISTING FRAMES

Generally concrete moment resisting frames performed as expected in the 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 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.

5.1 Frame Ductility

The larger buildings in Christchurch were typically concrete MRF structures, designed in the 1980s for high levels of ductility. While the frames performed as expected in the earthquakes, the yielding of the beam reinforcing contributed to repair of many buildings being deemed uneconomic.

Capacity design principles should continue to be used for frame structures, however higher load levels may reduce damage in earthquakes that are between the SLS and ULS levels of

shaking. This is to control both structural damage in a significant aftershock and frame elongation effects as described below. Designers should take special note of clause 2.6.6.1 of NZS3101:2006, which requires that suitable collapse mechanisms are identified irrespective of the ductility chosen for a concrete building.

Damage Reduction Recommendation: Damage in conventional concrete moment resisting frames may be reduced by designing for higher levels of load, but without departure from the appropriate detailing for ductility that would otherwise be appropriate. This includes at the very least, detailing for resilience as described in section 5.2 below.

5.2 Frame Detailing for Resilience

Regardless of the ductility assumed for the determination of 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)

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

OR

- ~~Check column rotations at the ULS displacements assuming full inelastic drift is concentrated in a single storey (refer to NZS3101:2006, clause 2.6.1.3.2.). These drifts should be calculated assuming probable strengths, and verified against the rotation limits given in NZS3101.~~

Verification Method Requirement: Frames must be detailed to ensure sufficient capacity to resist earthquakes larger than the ULS earthquake.

5.3 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.

content below now superseded by Verification Methods

~~In situ floors tend to be able to accommodate severe damage of this form; however precast flooring lacks this robustness and can result in severe collapse hazards (especially flange hung double tees and hollowcore).~~

~~It is recommended that, where any frame ductility is anticipated, the floor diaphragm connection to the end frames be detailed to accommodate the elongations expected. This will be no easy task — a good reason not to design for high levels of ductility in concrete moment resisting frames.~~



Figure 9: Floor tearing caused by frame elongation

~~Maximum elongations of 3.5% of the beam depth are expected for fully ductile frames. However, it should be noted that some frame elongation is still expected at yield, with recoverable geometric elongations in the order of 0.25% of the beam depth per plastic hinge are to be considered. This may apply even in buildings where the frames are not part of the primary lateral load resisting system, or for secondary (gravity) systems in moment resisting frame structures.~~

~~SESOC
Recommendation:~~

~~Floor diaphragms must be detailed to accommodate significant frame elongation where any yielding of conventional moment resisting frames is expected (note that this may not be possible to achieve).~~

~~Where conventional moment resisting frames are designed to remain elastic, geometric elongation of approx 0.25% should be considered in design and detailing of floor diaphragms.~~

5.4 ~~Shear Reinforcement in Deep Beams~~

~~**content below now superseded by Verification Methods**~~

~~Failure of lap splices was noted in one instance where stirrups were lapped in a deep (transfer) beam, in the cover concrete. Although such beams may not be expected to develop ductility, accidental overload may cause greater loads than the design loads. Such actions may lead to spalling at which points laps may become ineffective. Although lap splices of U-bars are permitted by the Standard (cl 8.7.2.8), the CERC has recommended that lap splices in such locations are limited.~~

~~SESOC Recommendation: Where the Standard permits use of stirrups in the form of lap spliced U-bars, the proportion of such bars lapped in the cover concrete shall not exceed 50%. All other stirrups shall be anchored conventionally with 135-degree hooks around the principal reinforcement. (CERC R2.51)~~

6 STEEL MOMENT RESISTING FRAMES

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

Steel moment resisting frames (meaning systems which are reliant on yielding of the steel in the frame) designed for other than nominal ductility have restrictions on beam section geometry and elimination of composite action at the column face in order to suppress frame elongation.

6.1 ~~Frame Ductility~~

~~Refer to Concrete Moment Resisting Frames section above. Following a similar reasoning, it is recommended that the maximum category of steel moment frame used be category 2, i.e. $\mu_{des} = 3$. By using a lower ductility demand in conventional buildings, it will effectively raise the damage threshold and make repair either less onerous or unnecessary following a severe earthquake.~~

~~A lower value, say $\mu_{des} = 2$, will raise the damage threshold further and therefore further reduce the need for post earthquake repair without much penalty on frame member sizes. However it should be noted that category 2 MRFs performed well in the 2010/2011 Christchurch earthquake series so $\mu_{des} = 3$ remains a suitable starting point when low damage behaviour is not being specifically sought.~~

~~Damage Reduction Recommendation: Conventional steel moment resisting frames should be limited to Category 2, i.e. $\mu_{des} = 3$~~

6.2 Frame Detailing for Resilience

NZS 3404:1997, when published, did not contain specific provisions to ensure that buildings will be capable of surviving an earthquake larger than design. However, that was a principal focus of the Amendment No 2: 2007 and those provisions must be followed to provide this resilience. Amendment No 2 imposes limits on rotation demand of beams and columns, limits on the use of structural systems without capacity design and enhanced requirements for overstrength of connections.

Verification Method Requirement: Follow the provision of NZS 3404 Amendment No 2 for the design of structural steel moment resisting frames.

6.3 Frame Elongation

Depending on the interaction between the slab and the columns, frame elongation is not as significant an issue for steel MRFs as it is for concrete.

Ductile moment resisting steel frames with composite floors require no special detailing for frame effects on the floors.

The same may not be the case with precast floors on steel frames. No evidence of this being a problem has been seen in the Christchurch area 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.

The provisions of Amendment 2 require the yielding beam framing into the column to be doubly symmetric and detailed to avoid composite action. Such beams 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 if not eliminated.

Damage Reduction Recommendation: Isolate the columns from the slab when using structural steel MRFs 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 in with all floor diaphragms)

6.4 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 should be noted that the current code provisions are suitable to spans of up to 12m, beyond which different configurations of composite connectors may be required to achieve the required extent of composite connection. Steel Construction New Zealand may be able to provide further advice.

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. This may result in loss of confinement to the concrete adjacent to the studs, which in turn could lead to loss of composite behaviour.

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 free edge in accordance with clause 13.3.2.3 (d) of NZS3404:1997. The requirement of NZS 3404 Clause 13.4.10.4 for transverse reinforcement to control post-splitting loss of strength is particularly important with precast floors on steel beams.

Note hollowcore is the most problematic precast floor system in this regard as the shrinkage at the base of the hollowcore units will put a transverse tension field across the concrete rib enclosing the studs over time, enhancing the likelihood of a crack developing along the line of the shear studs in service. This is independent of any earthquake effects and applies to all steel beams supporting precast floor systems. Use of flange-hung double tees will reduce these actions as the support is then above the neutral axis, but overall shrinkage will still focus at the ends of the units and over a multi-bay system, there is insufficient control of where the crack may form.

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 tray 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 gap in this location. Apply NZS 3404 Clause 13.4.10.4 for transverse reinforcement to control post-splitting loss of strength.

7 STEEL BRACED FRAMES

The general observation is that steel eccentrically braced frames (EBFs) have performed well. However, there have been examples of EBF active link fractures and in other instances, inelastic demand requiring replacement of active links. Regardless, the concentration of forces in the ductile link sections makes repair relatively easy due to the isolated nature of the links.

In addition, tension bracing systems 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. The

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.

Ductile steel braced frames designed to NZS 3404 Section 12.12 will have relatively low ductility demand due to the penalty of the Cs factor and increased inelastic deflection requirements. Keeping the ductility demand low and ensuring that the connections are designed to the required overstrengths are the best ways of ensuring good performance from conventional braced steel framed systems.

The integration of floor slabs with eccentrically braced frame systems is a subject of new research, as evidence from Christchurch is that composite floors may have been beneficial in increasing stiffness and reducing ductility demand while suffering minimum damage. This is covered more under EBFs below

7.1 Ductile EBFs

Ductile EBFs in theory have a low damage threshold, although performance in Christchurch showed that to be higher than expected. Nevertheless, EBFs should be designed for $\mu = 3$ max.

EBF systems using bolted-in replaceable active links will like be easier to repair following a severe earthquake.

Damage Reduction Recommendation: When designing EBFs, consider using bolted in replaceable active links in order to facilitate future replacement in the event of damage.

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 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 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: The associated structural system 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)

7.2 Tension Only Bracing

The design of concentric bracing systems is prescribed in NZS3404:1997. It is considered that provided that the provisions of section 12.12 are followed, adequate performance is expected. Designers' attention is drawn specifically to the C_s factor, which is used to increase the design base shear for concentrically braced systems.

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 or 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).

Note that NZS 3404 requires capacity design on all category 1, 2 and 3 concentrically braced systems.

Proprietary systems should only be used within the limitations noted above in Section 2.5. Note that at least one proprietary system has suffered failures, 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:

- 1 Tested to dynamic loading conditions and shown not to suffer brittle failure, and
- 2 Are 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.

SESOC Recommendation: Bracing systems and their connections must be designed and detailed to the provisions of NZS 3404. Note especially the connection strength requirements.

Proprietary systems shall have been subject to a comprehensive testing regime, and shall be installed completely to the manufacturer's instructions.

8 STEEL CONNECTIONS

Reliable performance of structural steel connections is required in order to achieve the required performance of the overall system. In many cases where failure or poor performance of structural steel systems was noted, this was attributable to poor connections, either by design to previous standards, or poor workmanship.

8.1 Steel connections subject to inelastic demand

For steel connections subject to inelastic demand or connecting elements of a seismic resisting system that is expected to perform inelastically the following requirements are essential to ensure good performance:

1. Load path to be as simple and direct as practicable

2. Determine internal forces generated in the members being connected
 - recognise primary torsion and other actions
3. Incoming force to be transferred into components parallel to it
4. Provide for reactions when component forces change direction
5. Design actions based on system response
6. Suppress connector only failure modes through detailing and overstrength design
7. Don't mix bolts and welds to carry the same design action
8. Fillet welds must be double sided and balanced
9. Design connection components and connectors for design actions including overstrength where required to suppress connector failure
10. Detail connections to sustain inelastic rotation of connected members
11. Ensure that all materials are suitably notch tough for their in-service condition

Connections that met these requirements performed well. However connections that failed some of these provisions, especially poor lining up of incoming elements with stiffeners or where the welds or bolts were the weakest link, performed badly.

Verification Method Requirement: Make sure the above points are adhered to in both design and as constructed. The photo below shows the consequences of misalignment of an incoming brace flange with the stiffener above.



Figure 10: Fabrication eccentricity causing fracture of EBF

9 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, topping etc.

Insitu floors (conventionally reinforced or post-tensioned) are the preferred flooring system due to their superior robustness. However, in New Zealand insitu 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 and contractor.

9.1 ~~Double Tees~~

content below now superseded by Verification Methods

~~If double tees are used, flange hung supports are common due to the difficulty in providing seating for full depth webs, and the improved geometry for shrinkage and thermal movements. However, the flange hung details need to incorporate a robust hanger, detailed to maintain seating for the inter-storey drifts expected in a significantly larger earthquake than the ULS event. Refer to the SESOC guidelines¹³ for further guidance.~~

~~The CERC has recommended that double tee units are supported on low friction bearing strips in order to isolate the units and supporting structure from friction forces.~~

~~Verification Method Provide robust hanger seatings to accommodate 1.5/Sp times the Requirement: ULS drifts (including effects such as frame elongation as appropriate)~~

~~SESOC Double Tee units shall be supported on low friction bearing Recommendation: strips in similar fashion to hollowcore, in accordance with cl 18.7.4 (e) of the Standard (CERC R2.50).~~

9.2 ~~Hollow-core~~

content below now superseded by Verification Methods

~~Hollow-core flooring is prone to damage when forced to undergo rotation. A key issue is that once the web is cracked, the capacity of the unit is severely compromised. The introduction of reinforced webs to the New Zealand hollow-core market is desirable but unlikely given the current investment in precasting technology.~~

~~Methods are provided in the University of Canterbury research report 'Assessment of hollowcore floors for seismic performance' (2010)¹⁴ to detail hollow-core seating to accommodate a variety of situations. The units should be detailed to maintain seating for the inter-storey drifts expected in a significantly larger earthquake than the ULS event.~~

~~Verification Method Use recommended hollow-core seating details and provide Requirement: seating ledges to accommodate 1.5/Sp times the ULS drifts (including frame elongation, etc as appropriate).~~

9.3 ~~Rib and Timber Infill~~

content below now superseded by Verification Methods

~~Generally rib and timber infill has been found to perform adequately in the 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.~~

9.4 Seating Details

content below now superseded by Verification Methods

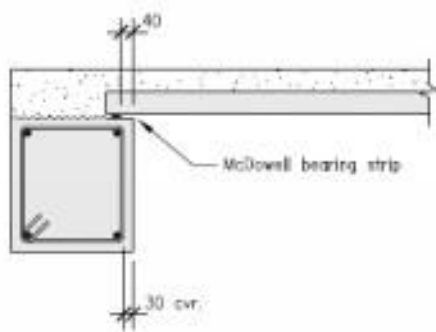
~~Significant guidance is provided with respect to seating details for precast floors — for more detail refer to the University of Canterbury research report ‘Assessment of hollow core floors for seismic performance’ (2010)~~

~~Precast flooring systems should 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.~~

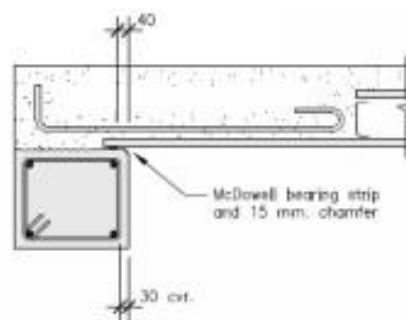
~~For web supported double tees, armoured corbels should be provided to reduce the effects of spalling on the precast flooring seating. Armoured corbels are not considered to be required (but may be used) for pre-stressed flat slabs, prestressed ribs, or hollow core.~~

~~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 failure at a location beyond the stiffened area. In particular, for rib and timber infill systems, the commonly used haunch detail (where the last timber infill is sloped to meet the top of the supporting beam) should be avoided. Instead a vertical form should be used to the underside of the timber infills.~~

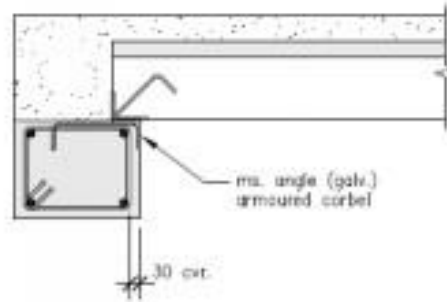
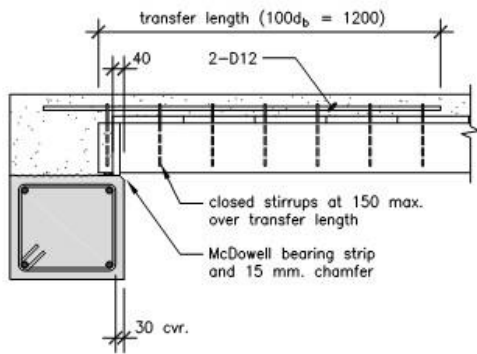
~~SESOC Recommendation: The following figures provide typical seating details for commonly used precast flooring systems~~



~~Figure 11: Flat slab seating~~

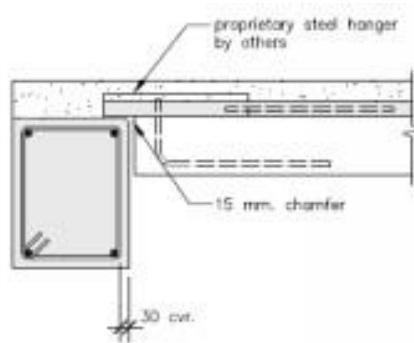


~~Figure 12: Hollowcore seating~~



~~Figure 13: Rib and timber infill seating~~

~~Figure 14: Web supported tee seating~~



~~Figure 15: Flange hung double tee seating~~

9.5 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 on Seismic Joints below) 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.

10 FLOOR AND ROOF DIAPHRAGMS

The performance of floor diaphragms in the Canterbury Earthquakes has been varied. Significant damage has been observed where diaphragms are required to drag large loads around between lateral load resisting elements, or where diaphragm tearing has caused consequential damage to non-robust flooring systems (see above).

The performance of thin toppings on precast flooring systems has been mixed. Inelastic displacements have 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 distribution of shrinkage and creep-related pre-existing cracks between a greater number of 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. However, these slabs are also less likely to have specifically addressed collector or transfer forces.

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

~~Little guidance is currently available for either the assessment of design actions or the design of diaphragms and collectors. One possible source of information is a US document published by the National Institute of Standards and Technology (NIST), *Seismic Design of Cast-in-place Concrete Diaphragms, Chords and Collectors* 15. Care must be taken to account for the effects of using thin toppings when using this document. More recently, research completed at the University of Canterbury (Gardiner) 16~~

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.

10.1 Collector Elements

Where forces need to be transferred into a lateral load resisting element, the collector element must be able to maintain the load path without failure in an event significantly larger than the ULS level event. The collector element should therefore be designed to resist the forces associated with the lesser of 1.5 times the ULS actions or over-strength actions as appropriate.

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: Collector elements must be designed for the lesser of 1.5 times the ULS actions or over-strength actions as appropriate. 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.

10.2 Suspended Floors

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. If 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 or hollowcore flooring systems.

SESOC Recommendation: Absolute minimum topping thickness of 75mm. Hard-drawn or non-ductile wire meshes are NOT to be used in floor diaphragms.

Verification Method Requirement: If using deformed bars, maximum bar spacings are 300mm in toppings for precast floor systems, 200mm for infill slabs between precast units.

10.3 Roof Bracing

Lightweight roofs often use tension bracing. This bracing should comply with the same requirements as concentric bracing as noted in Section 7.2 above. These systems should either be designed elastically for the capacity of the primary structure, or for $\mu=1$ actions using parts and portions derived loads.

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.

Use of proprietary systems should be treated in the same manner as noted in Section 2.5 above.

SESOC Recommendation: Connections for tension only bracing systems must be capable of developing the overstrength capacity of the yielding element of the system, unless designing for $\mu=1$. For steel systems the minimum design actions from NZS 3404 Amendment No 2 should be followed.

Yielding elements of the bracing system must be detailed with notches in accordance with NZS3404, unless it can be shown that they are capable of extending to the amount required to accommodate $1.5/S_p$ times the ULS drift.

10.4 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.

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 residential houses designed to NZS3604, 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 Canterbury earthquake’¹⁷ published by the DBH.

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’¹⁸ is a very 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 specialist geotechnical engineers to explore various options for reducing the damaging effects of liquefaction. 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 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’ 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. Posttensioning 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.

11 TRANSFER STRUCTURE

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.

11.1 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 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 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 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 above. If any part of the structure is designed for $\mu=1.25$ actions, designers must use $S_p=1$, unless a capacity design approach has been followed.

Damage Reduction Recommendation: Due to the complexity of transfer structures that form part of the lateral load resisting system, full independent peer review should be carried out, at least for the transfer structure.

12 SEISMIC JOINTS

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

12.1 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 is likely to cause significant structural damage.

12.2 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 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.

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

13 FOUNDATION ISSUES

Foundations will require substantially more consideration in future. The most appropriate system for the site 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.

13.1 Geotechnical Advice

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.

Geotechnical reports should provide not only foundation design parameters, but also comment on the most appropriate foundation type for the particular structure and site. Closer collaboration will therefore be required.

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

SESOC Recommendation: Geotechnical advice should include the most appropriate foundation types for the specific project. The geotechnical engineer should review the final foundation design.

13.2 Geotechnical Strength Reduction Factors

The Building Code compliance document B1, in B1/VM4 nominates strength reduction factors for foundations, from $\phi = 0.8$ to 0.9 for load combinations involving earthquake overstrength; and from $\phi = 0.45$ to 0.6 for other load combinations. Geotechnical engineers advise that there is no merit in this variation for vertical actions (gravity or gravity and seismic combinations), given the uncertainties in foundation design. However, it is not considered that strength reduction factors always need to be applied to horizontal earthquake actions.

It is recommended that designers do not use the higher strength reduction factors, ϕ_{bc} for shallow foundation design (from Table 1 of B1/VM4) and ϕ_{pc} for deep foundation design (from Table 4 of B1/VM4), associated with earthquake overstrength combinations, but instead default to the appropriate values for the normal load combinations, for vertical actions. Alternatively a revised strength reduction factor should be selected based on a proper risk assessment procedure such as that given in AS2159-2009, following specific geotechnical engineering advice. Horizontal actions may be checked without applying strength reduction

factors, noting that designers must take into account the effect of earthquake induced horizontal movements in considering these actions.

SESOC Recommendation: The use of higher strength reduction factors is not recommended unless specifically instructed by the geotechnical engineer.

13.3 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.

At this stage it is not known what pressure should be used for the design of basements to resist pressures resulting from liquefaction – advice should be sought from your 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.

13.4 Reconstruction in Christchurch

Widespread liquefaction has occurred throughout areas of Christchurch, although the visible evidence is not always obvious. In addition there has been significant lateral spread at riverbanks, both existing and on abandoned channels. Frequently both have occurred in many sites. Where liquefaction has not been observed, the possibility of liquefaction at deeper levels still exists. On the worst sites, it is possible that liquefaction may occur even under the SLS event, ~~noting that the seismic hazard factor has been raised from R=0.25 to 0.33.~~

SESOC Recommendation: Specific geotechnical advice must be sought for all sites in Christchurch. Geotech reports must make a recommendation on the type of foundations to be used in all cases. Liquefaction potential should be addressed at both SLS and ULS shaking level.

14 SHALLOW FOUNDATIONS

As a result of the liquefaction that occurred under a significant portion of the CBD (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 as previously in the Christchurch re-build, unless the sites have an acceptably low liquefaction probability.

Raft foundations performed significantly better, although residual deformations such as global rotation occurred in many cases, requiring substantial re-leveling works or complete demolition.

14.1 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.

Designing shallow pad foundations to exert similar bearing pressures at a potentially liquefiable layer could be undertaken to mitigate the effect of differential settlement. However, the high degree of uncertainty involved will make this a difficult task to get right.

In a number of cases hardfill rafts (typically 400-600mm deep) have been used under shallow foundations, in order to minimise the likelihood of differential settlement in liquefiable materials. It seems likely that these rafts have helped to disperse bearing pressures and to minimise differential settlement. This approach has however not worked as well where the shallow foundation is immediately adjacent to piled foundations. In at least one such case, there has been significant differential movement, both lateral and vertical.

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 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 10.2 above.

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

14.2 Mat Foundations

Mat foundations are continuous structural slabs spanning between columns and walls etc. Their resistance to differential ground movements will vary according to their strength and stiffness. The level of damage will also depend on the extent of differential movements both vertical and lateral.

In general, the same comments apply as are noted below for Rafts. However it is recommended that only rafts are used unless bearing pressures are sufficiently low.

SESOC Recommendation: All mat slabs are to be made sufficiently stiff and strong to act as rafts.

14.3 Rafts

Raft foundations have generally performed well in the Canterbury Earthquakes. Global settlement and lateral movement have occurred. However, the buildings have remained generally level in their new position. In addition, re-levelling of the raft is potentially feasible. Exceptions are where lateral spread has occurred.

In general it should be avoided, but if adjacent structures are to be interconnected, consideration should be given to tying the foundation together. If there is sufficient time available, preloading may reduce differential settlement. Equally grouting could be considered. Otherwise the rafts are prone to moving in independent directions, resulting in level and alignment offsets between the adjacent structures and a subsequent reduction in amenity of the buildings. Use of piles in such cases is generally not effective at avoiding differential settlement.

Damage Reduction Recommendation: Before constructing a new structure immediately adjoining an existing raft structure, preloading or grouting (or other nonharmful compaction) should be considered. In any case, raft slabs of interconnected buildings should be connected together as thoroughly as possible in order to minimise differential settlement.

15 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.

Damage to deep foundations may not always be evident from the surface, particularly where a large area has been subject to lateral displacements. Where there is evidence of relative motion between the structure and the ground, pile heads and the connection to the structure should be checked for overload in shear.

Shear transfer from the ground to the building is typically assumed to be carried by friction underneath the building and by passive resistance of the soil against buried foundation beams and walls etc. The friction mechanism will typically fail quickly with any settlement of the ground and the passive mechanism degrades rapidly with development of gapping. For this reason (and because the earthquake shaking was stronger than design levels) it is likely that the piles may have carried far more shear than the designer ever intended. In such situations it is important to expose the piles for observation to identify damage.

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.

15.1 Settlement of Piled Foundations

Settled piles may simply have 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 13.2 above for further guidance.

Pile settlement may also be from liquefaction of sand layers 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 at depth, all side resistance above may be effectively lost or reversed because of settlement of the overlying strata. In such cases so called “negative skin friction” may contribute to pile settlement.

Unless they are adequately embedded in dense soils, bored cast-in-place piles are perhaps the most susceptible to settlement caused by pore water pressure rise and liquefaction above the base of the pile because the gravity loads are carried initially almost entirely by side resistance. If this mechanism is overloaded, the pile will settle until the end bearing mechanism is mobilised (which could be as much as 5% – 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.

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.

15.2 Interconnectivity of Pile Caps

If piled foundations are to be used, it is important to provide tie beams or a full floor diaphragm between all pile caps to prevent relative lateral movement.

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.

15.3 Driven Piles

Driven piles used to be the norm, but this has reduced over recent years due to concerns about vibration and noise. They are now more often used in residential situations in smaller sizes, either concrete or timber or steel. Although now rarely used in larger structures, these are now more likely to be steel sections. Corrosion allowance for steel piles should be to NZS 3404 Part 1:2009.

15.4 Bored Piles

Bored piles now take two basic forms – conventional bored piles, or 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 adequate to reach the lower founding levels.

15.5 Screw Piles

Screw piles rely on the enhanced bearing of the steel flights that are attached to the pile shafts. For lightly loaded piles, there may be only a single flight at the tip, but for heavily loaded piles, more flights may be used.

Designers should be aware that the flights must deflect significantly for the pile to develop its full capacity. This is an important consideration in that, as the pile is screwed into the ground, the pile is in tension mode from the beginning and therefore if settlement is critical, preloading may be required to ensure that the pile is in compression mode. Consideration of corrosion should also be made, using the provisions of NZS 3404 Part 1:2009, given that the greatest movement is at the root of the weld of the flight to the shaft.

15.6 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.

16 STAIRS

Stairs were observed to perform poorly in many instances in the 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 Commission¹⁹.

16.1 ~~Movement allowance~~

content below now superseded by Verification Methods

~~Detail sliding joints to accommodate inter-storey drifts associated with earthquakes that may significantly exceed the ULS event.~~

~~SESOC ————— Design sliding joints for 1.5/Sp times the ULS displacements.
Recommendation:~~

16.2 Friction

Note that significant friction forces exist at sliding joints. Typical coefficients of friction are as follows;

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
 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

SESOC Recommendation: Allow for minimum and maximum coefficients of friction in the design of stair connections.

16.3 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 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 16)

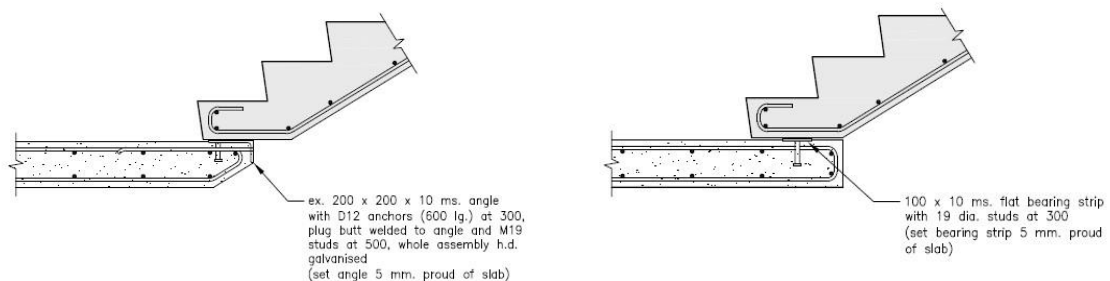


Figure 16: Typical stair details

17 PRECAST CLADDING PANELS

Sliding joints for precast panels typically performed poorly in relation to their design intent. As more panels have been exposed, significant damage has been observed to the panel connections, with some panels being dangerously close to falling off buildings.

For multi-storey buildings, there were no observed cases of complete loss of panels. Several warehouse type structures (such as malls, supermarkets) had precast cladding panels fall off, landing on the street below.

17.1 ~~Movement allowance~~

content below now superseded by Verification Methods

~~Sliding joints should be detailed to accommodate drifts associated with earthquakes that may significantly exceed the ULS event.~~

~~SESOC ————— Design panel joints to accommodate 1.5/Sp times the ULS
Recommendation: ————— displacements.~~

17.2 Detailing

While many panel connection details were obviously designed to accommodate 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 plastic washers should be provided in sliding joints. These washers should break when panel movement is initiated, resulting in a loosening of the bolted connection and reduction in frictional resistance.

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

18 GENERAL NON-STRUCTURAL ELEMENTS

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 engineer responsible for the overall building design. While this is often a contractual matter, it is

nevertheless important that the 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. 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 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 both SLS and ULS levels.

19 REFERENCES

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