Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury

Part 2
Evaluation Procedure

Draft Prepared by the Engineering Advisory Group

Revision 5, 19 July 2011

The contents do not represent government policy

Document Status

It is intended that this document will provide guidance to structural and geotechnical engineers and to Territorial Authorities in the assessment of earthquake-damaged buildings. The purpose of the assessment is primarily to assist in determining whether buildings should be occupied, noting that absolute safety can never be achieved.

Ideally, a document such as this should have been in existence prior to the Canterbury Earthquakes, as it is needed almost immediately. Consequently, this document has been prepared with considerable urgency, acknowledging that comprehensiveness and depth may be compromised as a result. This document is likely to require significant further revision in order to be applied more broadly than the Canterbury earthquake recovery.

This document is part of a series of documents, as follows:

Part 1	Background
Part 2	Evaluation Procedure
Part 3	Technical Guidance

The sequence of release of the documents is deliberately out of numerical order, recognising the need for engineers to begin the detailed evaluations as soon as possible.

Where errors are omissions are noted in the document, it is requested that users notify the Engineering Advisory Group through John Hare at johnh@holmesgroup.com.

Revision history:

Revision 1, Released through CSG, 29 May
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Revision 5, Released through CSG, 19 July

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1 INTRODUCTION

The post-disaster Building Safety Evaluation process endorsed by DBH involves three levels of assessment, as follows:

- **Initial assessment** - a quick walk around the exterior of the building to identify signs of imminent danger.

- **Rapid assessments** (level 1 and level 2) – a walk around and through the building (if it is safe to do so) looking for visible signs of significant structural damage.

- **Detailed engineering evaluation** - review of the building design, construction, and how the building has performed in recent earthquakes to understand its potential performance in future earthquakes and to determine what repair or strengthening is required to bring it into a satisfactory level of compliance or to simply improve its future performance

The first two have a clearly defined process1 but the third does not.

The need for a clearly defined Detailed Engineering Evaluation (DEE) procedure for buildings was highlighted initially following the September 4 earthquake, but is now even more evident post February 22. Initial and Rapid Assessments for buildings are a basic sifting method for identifying the worst of the immediate hazards, but the fact that a building may have a green placard does not mean that it has behaved satisfactorily and nor does it mean that it will behave satisfactorily in a future event. It simply identifies that no significant damage has been identified, that is, it is not known to be unsuitable for occupation. This means it is important for the engineering community to reinforce the message that further evaluation is generally needed, even where a building has been green placarded.

It is important that engineers completing detailed assessments do not rely unduly on the rapid assessments, but must rather form their own views based on a fully considered assessment. The rapid assessments should be taken as a guide only.

Longer term building performance is a significant concern for the general public, particularly with the continuing aftershock sequence. They are naturally lacking confidence in our building stock, particularly the older structures. Although there is a reasonable understanding of the general meaning of the placards, it is clear that there is some confusion amongst building owners and the public in general as to how much assessment is required to determine if a building may be considered safe enough to occupy.

There are several problems with this:

- Firstly, there is a lack of definition as to what a DEE comprises. A recommended process follows.

- The second issue is that there is not a legislative framework supporting this process. It logically resides in the Building Act, but this would require an amendment to the Act. This is addressed under the CERA legislation[2] for Canterbury, but it is considered by the Engineering Advisory Group that future wider application must be considered. There may well be implications for the insurance industry with respect to post-earthquake legislation, but public safety and confidence are an essential part of the recovery.

- A third, highly technical issue is the question of the incremental damage and how to evaluate it. In the previous earthquakes (September 4 and December 26), most of the damage was sustained by masonry buildings, with relatively limited damage to reinforced concrete and steel structures. Modern capacity design was barely tested.

  That changed with the February 22 earthquake. Following that, there were many damaged buildings of all forms, raising the question of how we assess their residual capacity. The June 13 earthquake caused even more damage, in some cases causing partial collapse of buildings that had survived earlier events of higher intensity. The assessment and repair of these structures must take into account future performance, notably the possible long-duration shaking that could result from an earthquake on the Alpine fault.

  This is not something that has been previously considered to this extent or level of detail in New Zealand. However, with the number of buildings affected, there is a need to quickly develop an assessment methodology, and ensure that it is applied.

  Guides for such evaluations have been developed overseas, notably in the US under the Federal Emergency Management Association (FEMA) programme. However their applicability in New Zealand is limited by variations in our design and construction methodologies. For example our use of precast is much more extensive than most other countries and this has considerable bearing on the way we should assess our building stock.

The form and extent of further evaluation should be appropriate to the individual building. Clearly a building of low occupancy that has no structural damage evident may require less intensive evaluation than a damaged building with great occupancy.

Acknowledgements

This document has been prepared by the Department of Building and Housing Engineering Advisory Group, comprising:

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Important Note

It is recommended that those carrying out evaluations and reviews using this guidance recognise the responsibilities involved and the liabilities to which they may be exposed

Neither DBH nor any member of the Engineering Advisory Group accepts any liability for the application of this guidance in any specific instance.

It is recommended that engineers providing advice based on the application of this guidance take appropriate steps to define the limits of their responsibilities and liabilities.

2 OBJECTIVES

The overarching primary objective of the Detailed Engineering Evaluation procedure is to provide confidence in our remaining building stock in order to assist the recovery from the Canterbury earthquakes. The measures of success include the appropriate reduction of risk of future building collapses in earthquakes; and when and if people return to the CBDs of the respective centres, whether as developer, owner, tenant or the general public.

This requires a process that is:

- Consistent – by the common application of the process described herein.
- Comprehensive – by ensuring that an appropriate evaluation process is applied to all buildings that could have suffered damage, or which may otherwise have significant vulnerabilities.
- Auditable – by requiring a consistent quality of information to be lodged with the Building Consent Authorities (BCAs).
- Able to be understood by lay people – by describing a process that is transparent and well communicated.

Secondary objectives include:

1. Ensuring that the process offers sufficient flexibility that no more effort is spent on a building than is necessary, in order to avoid unnecessary time and expense for owners, and to help speed the process.
2. The gathering and storage of information pertinent to the buildings

3 SCOPE

It is strongly recommended that affected TA's request further evaluation for all buildings not exempt from the Earthquake Prone Building (EPB) legislation, i.e. excluding only residential structures unless the building comprises two or more stories and contains 3 or more household units. This is broadly all non-residential structures, extended to include apartment buildings.

As these buildings are already under potential consideration as EPBs, it follows that detailed evaluation may be required in any case. This means that the main limitation will be geographic, i.e. how far from the main affected zones should this process spread? For now it is assumed that this will be at least in the three main TA's in the Canterbury area – Christchurch, Waimakiriri and Selwyn, but in practice this must be confirmed by CERA and the TAs.

The form of the evaluation should be appropriate to the individual building. For low risk buildings that have suffered no significant structural damage, a simple IEP procedure may be sufficient. For other buildings, the DEE procedure offers flexibility to engineers, with the proviso that a superficial walk-through offers little more real value than the Rapid Safety Evaluation. The exclusion of smaller buildings, for example buildings of three storeys and below, was considered. However it is noted that many of the buildings that collapsed or shed masonry into the street during the February 22 earthquake were one or two storeys only.

In addition to the structural and geotechnical engineering aspects of the buildings, there are a number of non-structural matters that should be checked prior to occupation. These checks are outside the scope of this guidance, and it is not generally expected that they will be completed under the supervision of the structural engineer. It is recommended that owners should be advised that these checks, which may be undertaken by the IQPs and other specialists. Such items may include:

- Compliance items covered by the building Warrant of Fitness. A list of these items is included in Appendix B.
- An electrical safety review
- A fire safety review.

These additional inspections will not require structural engineering review, but it is recommended that they be completed and submitted at the same time as the structural report, in order to simplify the reoccupation process. It is recommended that structural engineers brief owners and their IQPs on the need to identify loose and/or inadequate fixings and to notify the engineers if these are found.

4 THE PROCEDURE

The overall Detailed Engineering Evaluation process is presented graphically in Figure 4-1: Detailed Engineering Evaluation - Overall Procedure Outline on page 13.

It is recognised that not all buildings will need the same level of review to achieve sufficient confidence over their likely future performance. At either extreme of the red or green placarding, the engineering evaluation should be relatively straightforward. Therefore the major effort could be reserved for those buildings that are the most complex and which generally may be yellow placarded.

However, it must be noted that any green placarded buildings may harbour hidden damage or vulnerabilities which require an appropriate level of investigation to be detected. Engineers will need to exercise judgement in this, but evidence of distress or movement should inform the decision as to the extent of lining removal and testing required.

It is proposed to complete the evaluation in two parts, the first qualitative and the second, quantitative. The extent of the qualitative assessment will be determined initially from the placard and then from detailed damage observations, recognising that the Rapid Safety Evaluation (RSE) Procedure is superficial in nature, intended only to give a broad picture of overall damage levels for planning. The detailed evaluation process is outlined in Figure 4-1 below on page 13. Both the qualitative and the quantitative procedures are described separately below.

Following the qualitative assessment procedure, those buildings requiring no further action (other than non-consentable repairs) may be occupied (or have their existing occupancy continue). A report will still need to be submitted for approval, but no further action may be required.

The remaining buildings will then require quantitative assessment. The form of the quantitative assessment will vary according to the nature and extent of damage.

For many buildings, the extent of damage may be such that it is clear from the outset that a quantitative assessment will be required. In such cases, it may be efficient to commence the quantitative assessment in parallel with the qualitative assessment, but it should be noted that findings of the qualitative assessment will be a necessary input into the quantitative assessment before any conclusions can be reached. In particular, the qualitative assessment will help to indentify significant boundary condition issues for analysis models and to point assessors to potential weaknesses requiring further investigation.

4.1 QUALITATIVE ASSESSMENT PROCEDURE

The qualitative assessment process is presented graphically in Figure 4-2: Qualitative Assessment Procedure on page 14.

The purpose of the qualitative procedure is to develop a picture of the damage that a building has sustained, its causes, and the overall impact on the building's future performance. For this phase it is intended that no detailed analysis needs to be performed, although an assessment of likely building capacity will be made in terms of %NBS (New Building

Standard), either in accordance with the NZSEE Initial Evaluation Procedure (IEP), or by a simple comparison with current code according to the original design.

It is considered that the Qualitative procedure will be as follows, noting that in some cases, an abbreviated scope may be adequate:

1. Determine the building's status following the RSE. If possible, contact the building reviewer and ascertain the reasons for the assessed rating. At the very least, review the placard date and wording to ensure that the posted placard matches the building records. Note however that assessors should not rely on the RSE assessment, which is a visual assessment only.

   Should the building placard be inappropriate, it may be necessary to have it changed, particularly if the building has a green placard but is not considered appropriate for continued occupation. In such cases, advise the building owner and follow the official procedure to have the placard updated immediately.

2. Review existing documentation. An initial understanding of the expected structural performance is best obtained from review of the drawings and possibly the calculations or Design Features Report (if available). If no documentation is available, site measurement may be required in order to provide enough detail for the assessment.

   For additional guidance, refer to Appendix A – Generic Building Types and Expected Damage.

> Note that in many cases, documentation may be difficult to source, if it exists. Council records are not always comprehensive and may not include all buildings on a site. In some cases, building files may spread over more than one address, so all possible addresses should be searched. In other cases, better records may be held by previous owners or the original designers.
>
> Where no records are available, any assumptions must be made with caution, and on an informed basis. Reasonable attempts should be made to investigate the critical elements, including destructive sampling and testing if required. Assumptions of minimum reinforcement and steel grades must only be made with reasonable knowledge of the time of construction and prevailing standards at the time.
>
> A fall-back position is to make the most conservative assumption regarding the capacity of the existing structure (which in many cases may be to neglect its contribution completely), and insert supplementary structure to make up the shortfall.

3. The review must include consideration of the foundation performance, including an assessment of local soil behaviour. This requires the assessor to establish what the foundations are, and whether they are of an appropriate form for the nature of the building and the soil profile, assessed in light of our recent learnings. If no site specific geotechnical report is available, review general area soils information in order to form a picture of the likely soil behaviour. If in doubt, consult a geotechnical

engineer.

4. From the documentation review, the assessor should have:

    a. A reasonable expectation of the likely building performance and damage patterns.

    b. A mark-up of areas of the building requiring special attention. Matters to be considered include identification of potential 'hot-spots', areas where critical weaknesses have been identified or where damage is expected to be focused. These areas are to be exposed for inspection, noting that if necessary, destructive investigation may be required

5. Site investigation should follow. At all stages, safety precautions should be observed. Independent safety advice should be sought if necessary.

    The investigation should commence with a review of the surrounding buildings and soil performance. Initial review of overall behaviour should be followed by detailed observations where required, informed by the documentation review as noted above. Survey information may be required at this stage, including a detailed level survey and a verticality survey if rotation of the buildings is suspected. If doing a level survey, consider surveying both the ground floor (or basement if applicable) and a suspended floor, in case of flotation or settlement of the base level independent of the main structure.

    Removal of linings should be completed as needed, according to the expected damage, commencing initially with identified hot-spots. Intrusive investigations should be spread evenly across areas where damage may be predicted, even if this may be inconvenient.

    If the damage observed does not match expectations, it may be necessary to extend the investigation, or to iterate between observation on site and further review of the documentation. The building's placard status should be taken into account, but must not be relied upon. Absence of damage in a green placarded building should not be taken for granted, but if sufficient investigation has been completed with no discovery, can be assumed.

    A list of elements to be considered in the site investigation is given in Table 4-1: Schedule of Recommended Inspections below. Note this list is given for guidance and is not necessarily comprehensive.

6. With reference to Table 4-2: Soil Damage Assessment Criteria, if it is determined that geotechnical advice is required, the geotechnical engineer should be engaged at this stage. For further guidance of areas of local ground damage, refer to Figure 4-3: Observed Soil Damage Within Four Avenues from Feb 22nd, below.

    A minimum level of investigation in all cases should include the following:

    a. Foundation drawings from records (if available)

b. Geotechnical report for site from records and/or relevant nearby geotechnical data from records (if available)

c. Visual observations of foundation performance and adjacent ground damage.

d. Levelling of ground floor and/or basement floor (relative levels – external benchmarks will be unreliable)

e. Check to see if property is identified in orange and red zones on the CERA land damage hazard map

Where geotechnical data and foundation data is not available for the site and ground damage and / or building performance indicate problems with the foundations, it may be necessary to carry out new investigations including borings/CPT etc. and exposure of foundation elements. Guidance on the appropriate scale of such investigations and when specialist geotechnical engineering input is needed is given in Table 4-2: Soil Damage Assessment Criteria.

Generic "local" sub-soil profiles and data from nearby borelogs etc may be very unreliable in Christchurch where soil conditions are known to vary significantly across individual sites, let alone between sites or across city blocks.

Visual observations of performance may be unreliable and much evidence of ground movement and liquefaction will have been lost since the event. Photographic evidence from immediately after the event may be useful. Evidence of relative movement between the structure and adjacent ground should be sought but should not be relied on to give a complete picture of structure or ground movements

7. An investigation of possible collapse hazards or critical structural weaknesses (CSWs) should be made.

   Some examples include:

   a. A steel tension brace may be vulnerable to fracture at threaded ends, where there may be insufficient threaded length to allow the required inelastic drift to develop.

   b. A shear wall may lack adequate collector elements from the structural diaphragm, either from inadequate anchorage, or insufficient area of steel.

   c. An exterior column may not have sufficient connection back into its supporting diaphragm.

   Note that it is not adequate to assume that a detail formed from a ductile material will behave in an acceptable fashion. Refer to Section 6 for further guidance.

8. An assessment must be made of both the original and the post-earthquake capacity of the building, taking into account the damage it has suffered. This may be achieved in

a number of ways:

a. An Initial Evaluation Procedure (IEP) may be performed, in accordance with the NZSEE procedures[3]. If so, allowance should be made in the IEP for detail CSWs in accordance with Section 6.3.1.

b. In the case of buildings that have suffered insignificant damage, this may come from a simple comparison against the design standards and procedures used for the original building design. For example, if a building has suffered no significant damage and is less than 15 years old, it is likely that it complies in most respects with current detailing provisions. Hence, given the recent change of seismic hazard coefficient (from Z=0.22 to Z=0.3), its capacity could be expressed as:

$$\%NBS = 100\% \times \frac{.22}{.3} = 73\%$$

This method also requires evaluation of the CSWs, which can be undertaken using the simplified analysis method presented in Section 6.3.2.

c. More refined analysis may be used if deemed necessary or desirable, but note that this will be an output of the quantitative assessment.

d. Note also that further detailed evaluation guidelines are to be issued to provide guidance on how to assess the capacity of damaged elements

9. An assessment must be made as to whether or not the building has sustained substantial damage, in accordance with Section 5. This will be used to assist in the determination of a repair and/or strengthening strategy for the building.

On completion of the qualitative assessment, a preliminary evaluation of the required course of action may be appropriate. According to the damage observed and the %NBS assessment, broad options are as follows:

1. For a building that has insignificant damage, no collapse hazard or critical structural weakness and that has %NBS>33%, **no further assessment is required**. Strengthening is however recommended for any building with %NBS<67%.

2. For a building that has insignificant damage, that has %NBS>33%, but which has a potential collapse hazard or critical structural weakness, **mitigation of the collapse hazard or CSW is strongly recommended**. Overall strengthening is also recommended for any building with %NBS<67%.

3. For buildings with insignificant damage, but that have %NBS<33%, and buildings with significant damage, a quantitative assessment is required. Note that according to the extent of damage, it may be possible to complete a quantitative assessment for part only of the structure, with a qualitative analysis for the structure as a whole. This could be sufficient when there is highly localised severe damage but the building has otherwise suffered little or no damage.

On completion of the qualitative assessment, the engineer should have a comprehensive understanding of the building's performance, the reasons why it has behaved as it has and a general understanding of its expected future performance. In the case of buildings which have suffered damage, it may be possible at this stage to complete a preliminary assessment of the required repairs and strengthening, to a suitable level for owners to consider their preferred strategy for future retention or demolition.

4.2 QUANTITATIVE PROCEDURE

The Quantitative Procedure is intended initially to assess the residual capacity of the building in its damaged state, and then to assess the efficacy of proposed repairs and strengthening. The Quantitative Procedure must be used where triggered by the Qualitative Procedure. The extent of quantitative assessment will have been informed by the outputs of the qualitative assessment. It is not intended that all buildings should undergo quantitative assessment. However, in those cases where the need for a quantitative assessment is clear from the outset, the two processes may run in parallel, at the engineer's discretion.

Where the Qualitative Procedure has determined that a geotechnical evaluation is required, it will generally be necessary to complete this prior to the structural quantitative assessment being completed. **The geotechnical evaluation is required to confirm boundary conditions for any structural analysis and without it, any preliminary results should be heavily qualified.**

In some cases where the primary structure is relatively undamaged but the foundations have been significantly affected by settlement, liquefaction or lateral spread, it is theoretically possible that only a geotechnical quantitative assessment may be required. However, assuming that some form of repair will be required, it is likely that a structural model may have to be developed to determine the impact of any re-levelling or foundation repair or replacement, particularly if load paths may be affected by the proposed work.

A set of detailed guidelines for the Quantitative Procedure is to come in Part 3 of the Detailed Engineering Evaluation Guidelines.

4.2.1 Geotechnical Evaluation

Where a quantitative assessment of the ongoing suitability of a structure is to be carried out, a quantitative assessment of the foundation capacity should also be undertaken. The quantitative assessment should be based on informed knowledge of the soil conditions and foundation dimensions.

This foundation assessment should probably be in advance of the structural assessment, as upgrading foundation performance may be much more difficult to achieve technically and economically than for the building itself. For instance, if the foundations to a significantly tilted building cannot be corrected, then demolition is likely and a quantitative assessment of the building may be superfluous. In some cases, a quantitative assessment of the foundation capacity should be undertaken even where a quantitative assessment of the structure is not considered necessary. This is particularly applicable where there has been significant liquefaction and/or lateral spreading. Some guidance on the appropriate levels of investigation and analysis required is given Table 4-2: Soil Damage Assessment Criteria.

Quantitative assessment may include a simple check of liquefaction susceptibility and bearing capacity, pile capacity checks incorporating pore water pressure changes, assessment of lateral load paths, through to a full assessment of pile-soil kinematic interaction effects. It should include an assessment of deformations likely in a future earthquake and how these might impact on the foundations in their current post-earthquake condition.

Lack of evidence of settlement or lateral movement should not be taken as proof of suitability of a foundation. Absolute measurements of either settlement or lateral movement are likely to be impossible to obtain given the damage to the existing benchmarks and the lack of pre-earthquake data in most cases. Also, there has been a wide variability in intensity of shaking around the region for various reasons and individual buildings may not have been subject to such strong shaking as others.

Where there is any suspicion that foundation movements in excess of the triggers may have occurred, a geotechnical engineer should be consulted.

4.2.2 Structural Assessment

Quantitative assessment may take a variety of forms according to the damage suffered and building form and configuration. This should take into account the possible collapse hazard or CSWs identified in the qualitative assessment. Quantitative assessment should generally be approached using the standard assessment procedures used in the evaluation of existing buildings, in accordance with the NZSEE guidelines[3] (including the most recent masonry research[4,i]), but will require modification in order to accommodate observed damage.

It is recognised that earthquake damage to existing building elements may reduce capacity and/or available ductility. Methods of assessment and repair are available under a range of international guides[5,6,7] but these may not always be applicable to the New Zealand context. It is intended as part of the Engineering Advisory Group activities to publish further guidance on the applicability of such guides and/or local adaptations for use in the assessment.

Analysis may be generally in accordance with NZS1170.5[8] and the NZSEE guidelines[3], taking into account the recent amendment to B1[9]. Use of linear or non-linear techniques should be chosen according to the type and complexity of the structure.

The output from the Quantitative procedure will initially be an assessment of the %NBS of the building in its damaged state, leading to an assessment of the required repairs.

[i] There are some known errors in this document in need of correction, but this is otherwise the most authoritative guide available for Unreinforced Masonry in NZ conditions

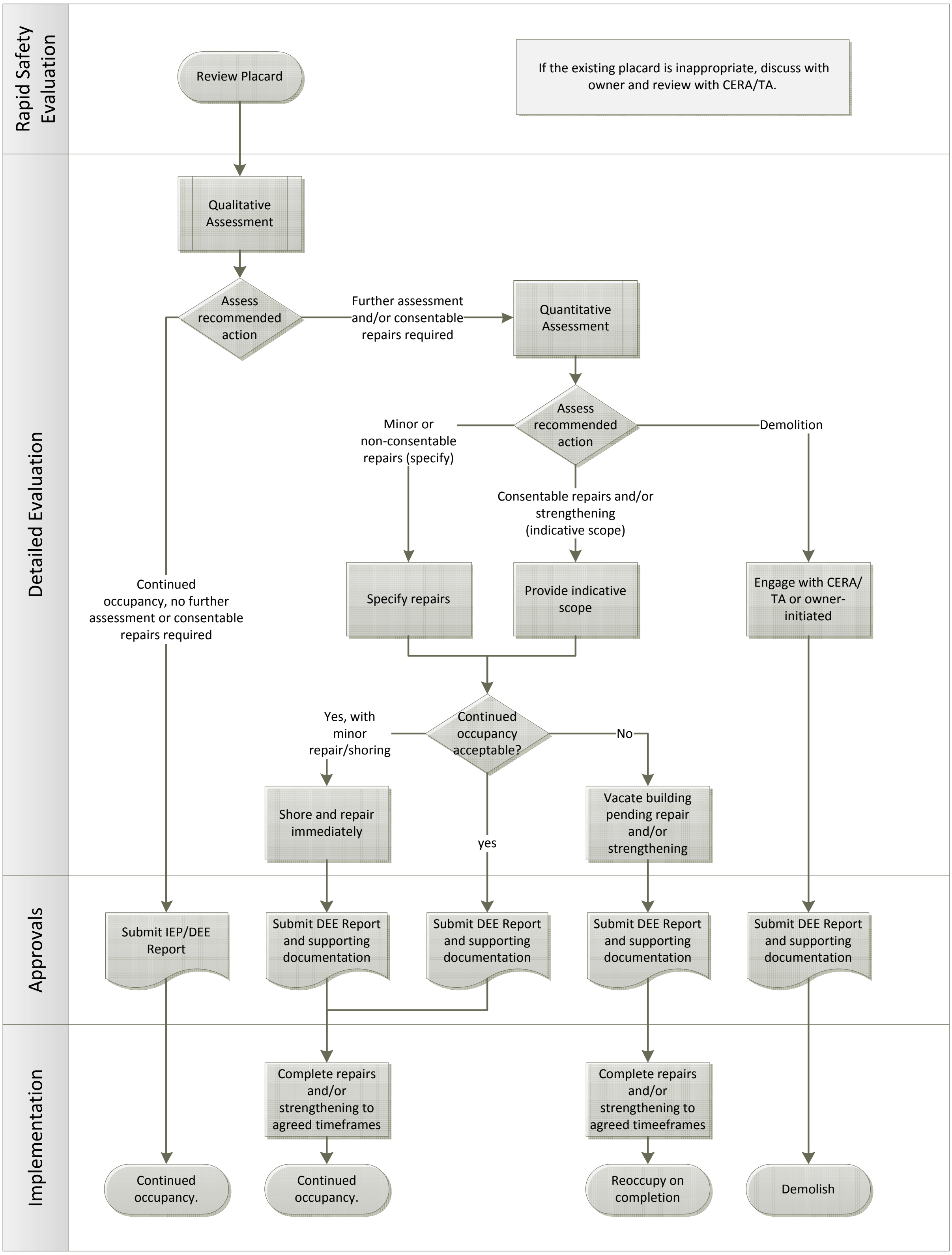

Figure 4-1: Detailed Engineering Evaluation - Overall Procedure Outline

Note: It may be advantageous to submit the DEE for discussion and approval ahead of any subsequent building consent application.

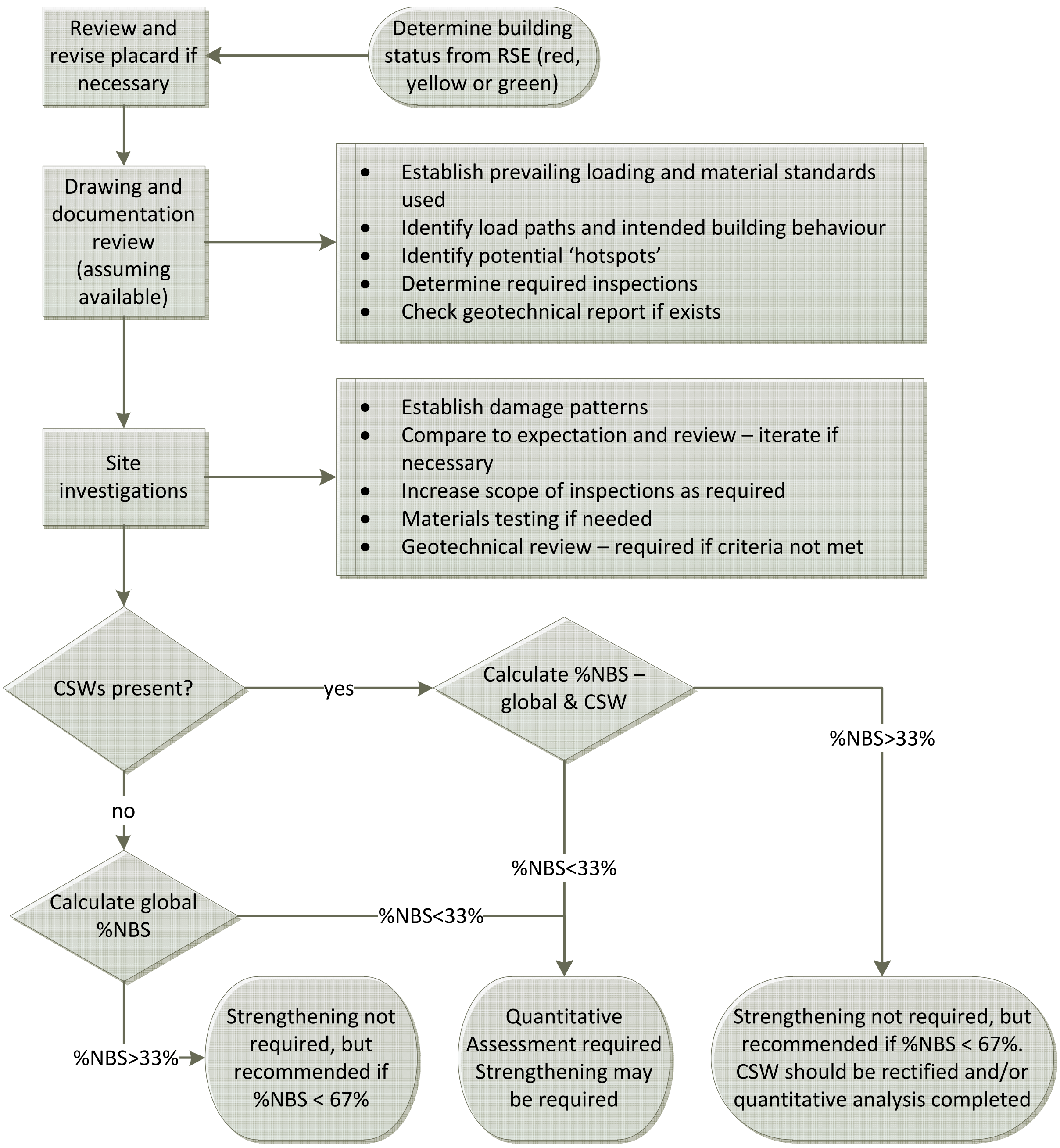

Figure 4-2: Qualitative Assessment Procedure

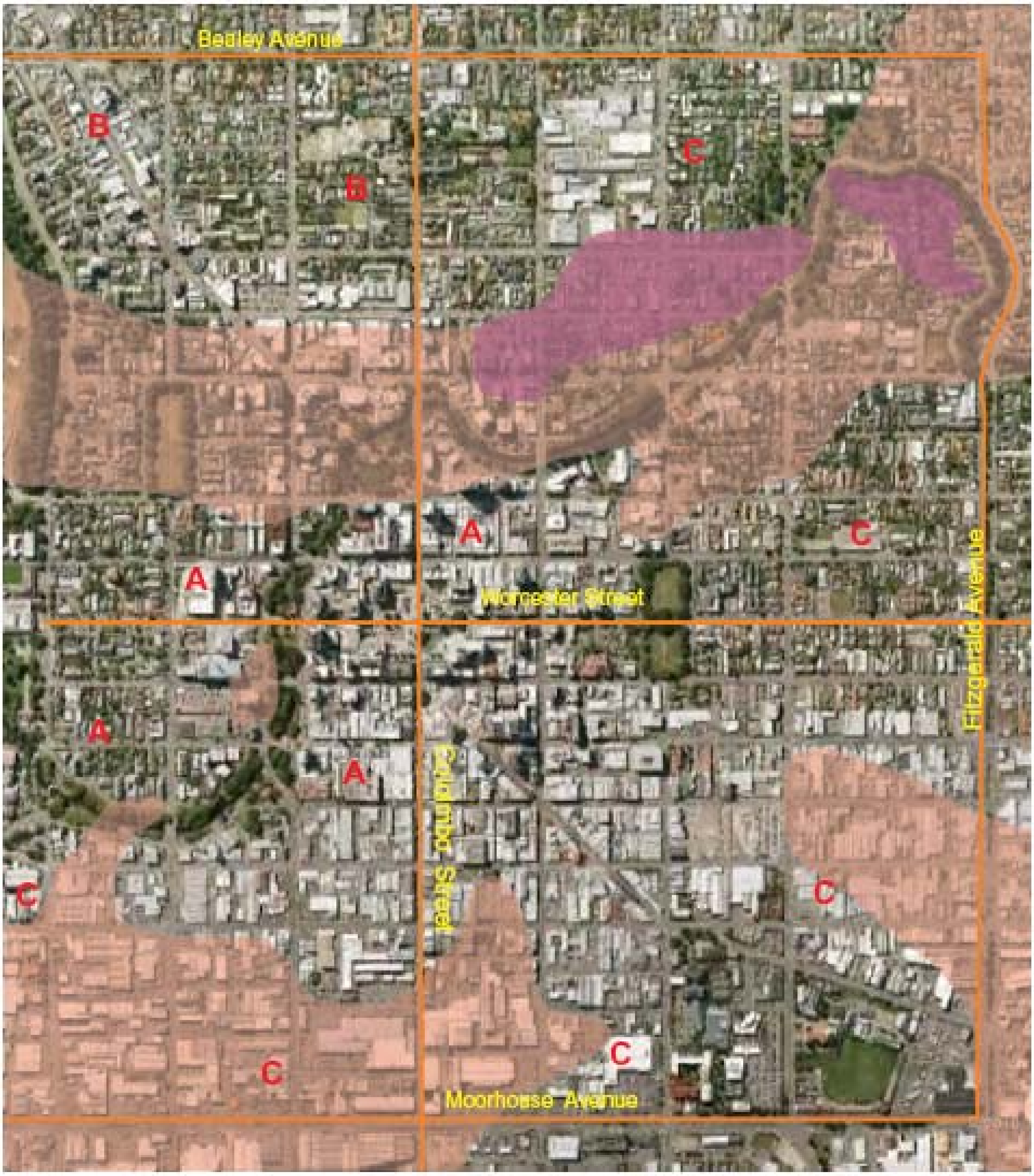

Figure 4-3: Observed Soil Damage Within Four Avenues from Feb 22nd (Misko Cubrinowski and others)

Notes:
Shaded areas denote liquefaction damage with sand and water ejection, ground fissuring etc. They are indicative only in that not all parts were damaged to the same degree, or damaged at all. Small areas of sand ejection also occurred in places outside shaded areas. Liquefaction may also have occurred in areas without surface damage.

Red shaded	22 February 2011. Most severe in Kilmore-Peterborough Street east of Colombo Street and in Avon Loop
Purple shaded	4 September 2010.
Area A	typically underlain with shallow gravel 6-8m thick
Area B	typically soft silty and some peat soils to 7-10m over dense sand and gravel
Area C	variable shallow soil profiles, frequently soft to 10-12m, fewer gravel layers to wards south and east

Table 4-1: Schedule of Recommended Inspections

Area	Element	Notes
Foundations	Ground conditions	• Verify whether liquefaction has occurred at or near the site[ii] • Verify whether lateral spread has occurred at or near the site • Check whether geotechnical information is available for the site • Look for signs of obvious settlement
	Foundations	• Investigate possible movement, lateral and vertical • If piled and lateral movement is observed, expose a pile or piles in order to verify the condition of the pile and connectivity to the building
Exterior	Roof	• Check for movement at flashings • Check parapets and other roof level appendages • Review connections at parapets
	Overall alignment and verticality	• If obvious movement or rotation (especially foundation level) consider survey.
	Surrounding buildings	• Visual inspection of surrounding buildings that may represent a hazard to the subject building
Main structure	Moment frames	• Column bases – hinging? • Beams – investigate potential plastic hinges and beam elongation • Beam-column joints – crack patterns • Possible fracture in steel frame joints
	Shear walls	• Crack patterns • Possible base hinging or shear failure?
	Bracing systems	• Extension in braces • Shear or flexural yielding in links of EBFs • Lateral buckling of brace elements • Yielding or damage to connections

[ii] Note that the detection of liquefaction or lateral spread can be difficult, and may sometimes not be apparent at ground level. If the surrounding ground conditions suggest either of these, or if the geotech report indicates possible vulnerability, it is recommended that a geotechnical engineer is engaged. Refer Table 2 below for guidance as to what type of review may be applicable.

Area	Element	Notes
	Diaphragms	• Transfer or inertial? • Floor type? • Precast floors – investigate seatings (above and below), crack patterns in topping, review ties at perimeter, saddle bars, topping reinforcement integrity
	Connections	• Verify grouted ducts fully grouted.
Secondary structure	Stairs	• Review seating and connections • Review intermediate landings – compression or tension failure
	Cladding	• Check whether cladding may have modified structural behaviour • Identify areas where structural interference has occurred due to drift • Investigate connections
	Ceilings	• Review fixing of grid (if applicable) • Fixing/support of lights, a/c grilles etc. • Damage to/at sprinkler systems
	Building services	• All plant items connected and restrained suitably
Non-structural elements (by others)	Compliance Schedule items	• Refer Appendix B.
	Electrical	• Electrician to inspect wiring.
	Fire Safety	• Fire engineer/IQP to inspect fire cell linings and active/passive systems

Table 4-2: Soil Damage Assessment Criteria

Parameter	Level of geotechnical assessment: Desk study	Geotechnical investigations if good borehole CPT data not available and limited exposure of critical ground connections	Full exposure of typical foundation elements and intrusive investigations of foundations if questions remain
Geotechnical engineering	Geotechnical engineering input to be considered	Involvement of appropriately qualified and experienced geotechnical engineer is essential	
Settlement (mm)	25	100	200
Differential Settlements	1:350	1:250	1:150
Liquefaction ($m^3/100m^2$)	2	5	10
Lateral Spreading total (mm)	50	250	500
Lateral spreading differential	1:400	1:100	1:50
Cracks (mm/20m)	20	100	200
Damage to superstructure	Cosmetic	Minor to Significant Structural	Severe to major structural
Damage in Area (Major remedial works)	Slight	Moderate to substantial (1 in 5)	Widespread to major (1in 3 to most)

Note: If any one parameter exceeds the limits set out in a column, then the scale of investigation is to be increased to the next level.

5 DAMAGE THRESHOLDS FOR REPAIR OR STRENGTHENING

5.1 INTRODUCTION

It is necessary in considerations of building assessment after earthquake, to set thresholds for when damage may be 'significant', therefore determining whether strengthening is required in addition to simply repair. In addition, the levels of damage may be used in tandem with residual capacity to determine acceptable timeframes for strengthening, and interim occupancy conditions.

International practice in this regard has been referenced in order to arrive at definitions to suit the Christchurch context. The main point of reference in this regard has been US practice, given their leadership in planning for earthquake. Of most notable importance is the recent CAPSS study in San Francisco, culminating in the publication of ATC52-4.

This chapter is intended to present a definition of substantial structural damage which will later be used to determine a repair and strengthening strategy for buildings, according to the damage level and residual capacity.

5.2 DEFINITION OF SUBSTANTIAL STRUCTURAL DAMAGE

The definition for substantial structural damage has been drawn from current US practice, with minor change. For the purposes of building evaluation (after any possible damaging event), the following is proposed for a definition of substantial structural damage, taking into account any reduction due to soil conditions:

1. In any storey, any elements of the lateral force-resisting system have suffered damage such that the lateral load carrying capacity of the structure in any horizontal direction has been reduced by more than 20% from its pre-damaged condition; or

2. The capacity of any vertical gravity load-carrying component, or any group of such components, that supports more than 30% of the total area of the structure's floor(s) and roof(s) has been reduced more than 20% from its pre-damaged condition and the remaining capacity of such affected elements, with respect to all dead and live loads, is less than 75% of that required by this code for new buildings of similar structure, purpose and location.

For the purposes of assessing the lateral load capacity above, damage must be considered both for individual lines within the structure and for the structure as a whole. This is relevant to the scale and extent of repair and retrofit.

For example, if a building, considered in one direction only, has a front wall with 50% damage but the damage level otherwise in that direction is less than 20%, then only the front wall may need to be repaired to have the building as a whole suffering from minor damage only. This implies that a simple repair and strengthening of the front wall may allow the building to be occupied (subject to overall capacity) while further evaluation is completed and a long-term strengthening policy developed.

6 RESILIENCE

The Christchurch earthquakes have re-emphasised the need for resilience. Although the duration of shaking was relatively short, the intensity of shaking was in many cases considerably higher than the design level. Consequently, some building performance was poorer than expected, or less than might be considered acceptable.

The collapse or partial collapse of buildings may simply be a result of low building strength, but it is noted that there are many cases of buildings of low assessed capacity which have nevertheless performed well, due to regularity and inherently good detailing. Conversely, there have been other buildings that failed to achieve their full capacity because of the failure of secondary details, or buildings (and parts of buildings) that behaved dangerously because displacements exceeded expected limits.

The purpose of this section is to discuss the identification of possible design or configuration issues that may result in potential collapse or dangerous behaviour of buildings, in shaking of greater intensity than anticipated in design or evaluation. It presents a simplified analysis method to evaluate such hazards, and recommendations for further action.

6.1 PERFORMANCE OBJECTIVES

In common with most countries that have advanced seismic engineering standards, New Zealand adopts a probabilistic hazard analysis approach to seismicity, and then a tiered approach to seismic design. For design, we have stated performance objectives:

1. Frequently occurring earthquakes 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.

These objectives are met differently in new building design than in the evaluation of existing buildings.

6.1.1 New Building Design

Objective 1 is satisfied by the serviceability limit state (SLS) requirements relating to earthquake, and is not relevant to this document (although it may be subject to separate review).

Objective 2 is deemed to be satisfied for new buildings by designing to the ultimate limit state (ULS) procedures set out in NZS1170.5 and associated material design Standards. Although new buildings are designed to achieve ULS at what might be considered a design level of earthquake shaking, it is generally implicit in the Building Code that a building that has been designed accordingly may also withstand significantly larger intensity earthquake shaking with an appropriately low probability of collapse. It is generally accepted that there is a margin of at least 1.5 to 1.8 over ULS capacity for well detailed new structures.

However, there are some more recent or current design practices that may significantly erode the resilience available and which are required to meet the expected performance in events larger than the design earthquake shaking.

These practices include but are not limited to:

- Not making sufficient allowance for the inherent poor performance observed in irregular buildings
- Allowing capacity design to cut off at design loads with μ =1.25
- Use of details where there is no resilience beyond the drifts predicted by the design Standards

These practices, although currently acceptable under the Building Code, should be identified by designers, who may then consider means of addressing them.

6.1.2 Existing Building Evaluation

The evaluation of existing buildings generally assumes that the original building design does not confirm to current standards, either in design or detailing. Hence the evaluation is intended to assess the building's capacity in a way that takes its potential lack of resilience into account.

Assessment of Objective 1 is outside the scope of this document, which is concerned with life safety only, although it could be noted that for those owners who wish to reduce the cost impact of future earthquakes, SLS performance should be evaluated, and may be enhanced by appropriate strengthening or other means.

Objective 2 is deemed to be satisfied for existing buildings if the requirements of the NZSEE guideline document and/or Building Code can be met. The guidelines allow some relaxation of requirements for existing buildings compared with new. Probable material strengths can be used and the guidelines recognise that conservatisms in some areas (eg calculation of shear capacity) that can be built in for relatively modest additional cost in a new building may not be appropriate or necessary when assessing existing buildings.

Implicit in the acceptance of relaxed requirements is that an existing building shown to achieve 100%NBS may not achieve the same level of seismic performance as a new building designed to achieve minimum compliance with the building code. However, the NZSEE guideline document recognises that existing buildings that meet 67%NBS (as determined by the guidelines) will still achieve an acceptable level of performance when measured against the performance objects outlined above and in the Building Code.

Assessors must recognise that an important aspect of resilience is determining the ability of the structure to deform beyond the displacements predicted for the ULS. If the assessment of an existing building is focussed purely on the overall building strength, it may not verify that the required level of resilience is being achieved. This is illustrated in Figure 6-1 below, which illustrates the load displacement relationships of structures of differing levels of ductility, and hence resilience. In this figure, the blue lines represent new buildings of differing levels of design ductility and the red lines represent existing buildings with differing %NBS capacity.

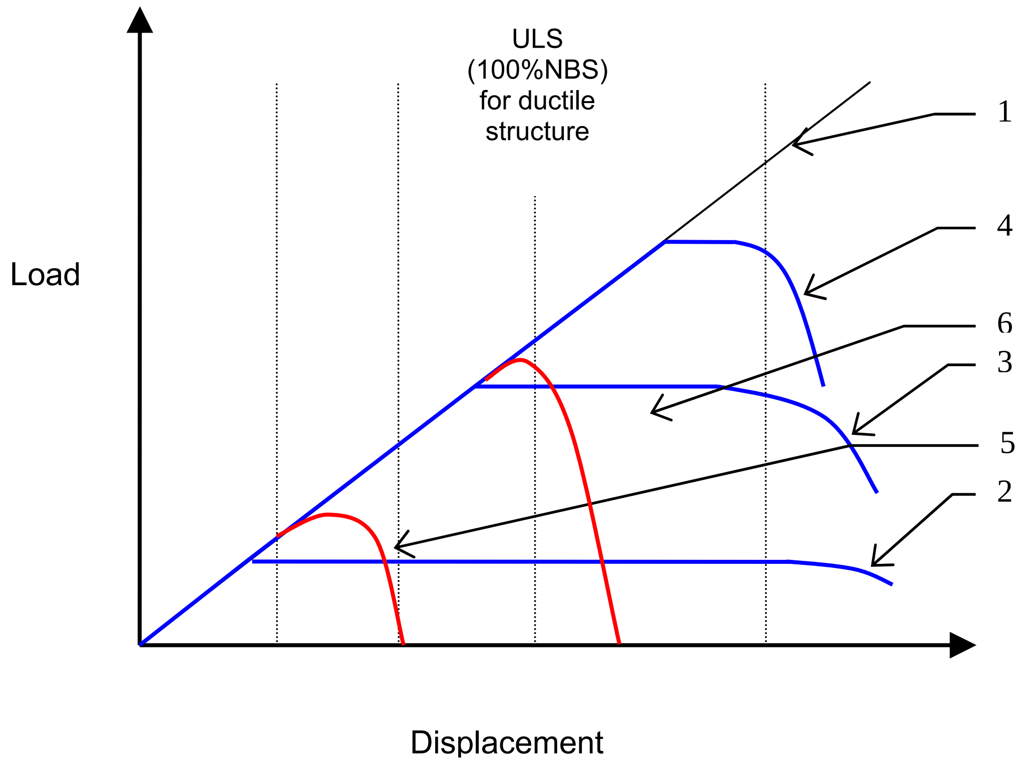

Figure 6-1: Load-Displacement relationships for buildings

Notes:

Line 1 represents a fully linear elastic approach, that is, the building has been designed to simply resist the full applied load in proportion to the imposed displacement.

Line 2 represents a high ductility level. The required strength is reduced according to the ductility, and capacity design is used to ensure that the building yields in a controlled fashion. The design detailing provisions of the standards should ensure in the majority of cases that the buildings will displace to significantly greater levels of displacement with acceptably low risk of collapse.

Line 3 represents a building of limited ductility. If higher strengths are provided, designers may reduce the detailing standards. However, this may mean that the margin between ULS and collapse is reduced. This is explicitly checked in the concrete standard, at least in respect of soft-storey mechanisms, but is implicit in the steel standard.

Line 4 represents a structure that is designed to remain fully elastic for the ULS. Such buildings are penalised (a higher Sp factor is specified) and are therefore required to have a higher design capacity than a ductile structure. However, because there are no implicit or explicit checks, there is no guarantee that they do not contain a critical structural weakness beyond the design capacity.

Line 5 represents a building that may just exceed the EPB threshold. Even if similar margins between ULS and collapse available in a new building are maintained it is apparent that there may be little capability to survive anything other than a moderate earthquake, which is only a little greater than a SLS event for a modern building.

Line 6 represents a building that may have been strengthened to 67%NBS. Because there is no requirement to add ductility, the onset of collapse is still only marginally above the design load.

6.2 CRITICAL STRUCTURAL WEAKNESSES

The term "Critical Structural Weakness" (CSW) is used in the New Zealand Society for Earthquake Engineering (NZSEE) Red Book[3]. These are used as the basis for a modifier to the Initial Evaluation Procedure (IEP) process used in the identification of EPBs and ERBs. Factors that are used in the identification of CSWs are:

1. Plan irregularity – identifying vulnerable floor diaphragm shape characteristics and potential torsional behaviour
2. Vertical irregularity - identifying possible storey failures of variation in mass and/or stiffness distribution
3. Short columns – identifying potential soft storey or torsional behaviour that may result
4. Pounding potential – due to inadequate clearance, with or without floor misalignment
5. Height differences – related to pounding, where adjacent buildings of different height my impact
6. Site characteristics – looking at land instability, possible landslide from above, or liquefaction
7. 'Other factors', factor F – essentially at the reviewer's discretion, an assessment of other compensating factors that may exist to reflect likely better or worse than expected behaviour.

Because of the nature of the IEP these are factors are typically those that can be determined from visual observation of the building without reference to plans or details. The extent and severity of observed CSWs is used to calculate the Performance Achievement Ratio (PAR), which is used to modify the baseline percentage of New Building Standard (%NBS).

The intention of the IEP is that all issues known to the assessor, that could potentially affect the seismic performance of the structure, be included in the assessment of the final score, albeit qualitatively.

The Christchurch earthquakes have highlighted the need to identify and assess the potential effects of a number of other CSWs that can only be identified from a review of drawings.

Some examples of such further detail CSWs that can be identified from plan review include:

- Areas of precast floor where the supports are short enough that the floor could drop under the actions of beam elongation and rotation under imposed displacements significantly in excess of the ULS drift.

- Stair supports that have insufficient seating or where filling of gaps to surrounding structure has reduced clearance such that yield of the stair in compression could occur, causing shortening of the stair with subsequent reduction of seating.

- Non-ductile connectors between precast panels and structure, or connections that have insufficient clearance to main structure or are incapable of accommodating interstorey drift at greater than ULS levels.

- Lack of adequate collector elements to transfer load from a floor diaphragm into a supporting shear wall or other discrete bracing element.

- Significant gravity load bearing columns or wall elements that fail in shear, leading to potential loss of support. This is a criterion for short column behaviour, but may equally apply in other cases, and should be identified.

- Large differential settlement of foundations which could lead to failure of superstructure components or unseating of floor units or similar.

Use of the IEP procedure therefore requires a means of addressing such issues, ensuring that there is not a double-up in the assessment. Other methodologies need to have a means also of addressing these, as well as the CSWs noted in the IEP. This is discussed below.

Note that the list of detail issues above can be split into displacement controlled and force controlled elements.

- A displacement controlled CSW is one which may contribute nothing to the resistance of the building as a whole, but which is not able to tolerate deformation of the structure. A simple example is a precast panel which must have fixings able to accept the proportion of the lateral drift that occurs over its height. Note that in cases where such an element reaches the limit of its displacement capacity, it may modify the behaviour of the building as a whole. The Building Code requires that such aspects are dealt with even if there is no affect on the building as a whole and therefore they should be factored into the IEP.

- A force controlled CSW is one that develops increasing load as the force or deformation on the overall structure increases, and the failure of which may cause premature failure of the structure as a whole, ie it acts as an unintended fuse for the structure, in a way which is insufficiently ductile.

This distinction is important in the assessment of the overall building capacity, and how it may be improved.

6.3 ALLOWANCE FOR RESILIENCE IN ASSESSMENTS

The resilience available in a structure should be reflected in the %NBS score given to the building. This applies to both the qualitative (IEP or simple code comparison) and the quantitative assessments.

It is apparent that what has previously been thought to be new building standard may require adjustment for some structural and non-structural aspects of the building. For a qualitative assessment, such as the IEP, these aspects will typically require consideration of factors that will not necessarily be apparent from an external inspection.

For both the qualitative and quantitative assessments, comparison with the revised new building standard (NBS) outlined below will be required. Methods are presented for incorporation into the IEP and a simplified analysis method is presented for incorporation with either the qualitative or quantitative analysis.

It is not considered that ground conditions generally need to be analysed as CSWs. Possible exceptions that are considered to present risk of brittle collapse are:

- Where differential settlement becomes so great that there is risk of the structure above failing.
- Where the stability of the structure is reliant on an uplift device such as a tension pile or ground anchor that may lose capacity in liquefaction conditions.

Where either of these possibilities exists, geotechnical advice should be sought and a detailed quantitative analysis completed.

6.3.1 CSW Analysis for the IEP

As global CSWs are already addressed in the IEP, it is necessary only to include allowance for additional detail CSWs not currently included, but which could be considered as part of the Factor F.

The recommended process is as follows:

1. From the plan review, identify potential detail CSWs, ie force-controlled elements that may cause premature failure, or displacement controlled elements that will fail at low levels of displacement.

2. Assess the severity of the force controlled CSWs in series with the other CSWs (if they exist). If the detail CSWs have a lesser impact than the global CSWs, they may be ignored in the Factor F assessment. If the detail CSWs have a greater impact than the global CSWs, then allowance should be made in Factor F, as follows:

    a. Calculate the capacity of the detail CSW, using the probable strength values and a strength reduction factor, ϕ=1.

b. Calculate the estimated global building capacity, using the IEP process (with global CSWs included, but excluding detail CSWs from Factor F) as an effective multiplier on the estimated base shear.

c. Calculate the capacity/demand ratio of the above detail CSWs.

d. Use the following modifiers to the assumed Factor F values used in step b above:

Table 6-1: Factor F multipliers for IEP CSW process

Capacity/Demand ratio	<1	<2	≥2
Factor F multiplier	0.5	0.75	1

3. For the displacement controlled elements, calculate a %NBS value for each of the CSWs individually, by comparing the expected displacement at 100% to the available clearance, modified with the K_d factor as noted in Table 6-2 below.

4. The %NBS for the building is the lesser of all of the calculated %NBS values.

Note that the IEP process is intended primarily as a sifting method to determine if a building is potentially earthquake prone or earthquake risk (i.e. less than 67%NBS). For buildings with an assessed capacity from the IEP of less than 33%, it is possible that a full analysis may result in a higher value. However, if the overall %NBS for the building without the detail CSW analysis is above 33%, owners should be encouraged to at least mitigate the CSWs.

6.3.2 CSW simplified assessment methodology

The objective is to develop an assessment methodology to ensure that there is an adequate margin between the performance of the primary system and the possible generation of alternative premature collapse mechanisms, and/or other significant hazards to safe egress or life safety. Note that the intention of this methodology is not to be used as a process to force upgrade of the primary systems.

The current EPB legislation uses a threshold of 33%NBS for all buildings. The PAR calculations are applicable to the IEP process, but do not have the same application to a more quantitative analytical process. A means of restoring the relativity of ULS to ultimate collapse for assessing buildings with CSWs is therefore required to ensure that resilience is achieved.

The proposed methodology is as follows:

1. Identify the collapse hazards in accordance with the Qualitative Procedure or otherwise.

2. Determine whether the CSW is displacement or force controlled.

3. From Table 6-2 below, identify the demand side multiplier, K_d

4. Calculate the limiting drift or force that the element may be subjected to.

5. Calculate the %NBS of the element in the normal way, but including the impact of K_d:

$$\%NBS_{element} = \frac{capacity}{K_d \times demand}$$

6. If the element is earthquake prone (i.e. %NBS<33%, to current EPB legislation), the element must be upgraded. If retrofit is required, the level of retrofit must be 100% of the factored load or displacement, using the Target Capacity Multiplier K_c, i.e.

$$\phi R \geq K_c \times R_{demand} \text{, where } \phi \text{ may be taken as 1}$$

Alternatively a full analysis of the building can be completed and the element subjected to capacity design procedures to ensure that non-ductile failure is suppressed.

Examples are given of this process at the end of this section, for both a displacement-controlled and a force-controlled element.

Table 6-2: Detail CSW demand side multipliers

Element	Force or displacement controlled?	Demand-side multiplier, K_d 1	Target capacity multiplier K_c 2
Torsional response	Displacement	2	2
Short columns	Displacement	2	2
Short columns	Force	1 flexure/2 shear/2 axial 3	1 flexure/2 shear/2 axial 3
Adjacent building clearance	Displacement	Refer to NZSEE guidelines for further guidance	
Precast floor seating	Displacement	2	2
Stair, ramp and escalator supports	Displacement	2	2
Shear wall collectors	Force	2	2
Non-ductile panel connectors	Force	2	0.67/2
Inadequate panel clearance	Displacement	2	0.67/2
Face-loaded masonry anchors	Force	2	0.67/2 4

Notes: 1. In the case of the force-controlled elements, the K_d factor is included in lieu of a formal full analysis. As an alternative, a full capacity design procedure may be followed.
2. Where there are two factors given for K_c, the greater factor represents the case where the hazard presents a risk to egress paths or access routes for emergency personnel
3. Where K_d/K_c values are given for axial load, this applies to the seismic component only
4. In the case of face-loaded masonry wall anchors, it is noted that the minimum effective anchor spacing is determined by the geometry at which anchor pull-out cones overlap. In cases where this cannot be achieved, supplementary support may need to be added.

6.3.3 CSW full detailed assessment

In the case of a full detailed evaluation using advanced forms of assessment, care should be taken to ensure that the treatment of the CSWs is consistent with the method being used. The factors presented in Table 6-2 for force controlled elements are not intended to apply to a comprehensive analysis. Instead, a full assessment must take the CSWs into account and treat them in accordance with the appropriate assessment methodology.

If using conventional linear analysis, in accordance with the NZSEE guidelines or the relevant Standards, all elements should be assessed against the appropriate ductility limits and element strain limits. If considering torsional or irregular buildings, imposed displacements must take into account the full accidental eccentricity, including the effect of yield and ductility in opposing elements.

If using non-linear analysis, the element strain limits used should take into account the mode of failure, whether force-controlled or deformation controlled. In the case of force controlled elements, the strain limits should be selected so as to provide an acceptable margin over the collapse prevention limit, for any load-bearing elements.

In either case, residual deformations of the structure and foundations must be taken into account in the analysis. In particular, for residual differential settlement, if there is differential settlement, allowance should also be made for future additional settlement. Geotechnical advice should be sought, but a minimum allowance of 30% of the existing residual differential should be added.

For displacement controlled aspects the assessor should note recommendations for additional allowances for clearances/seating lengths where these alone are likely to limit the resilience of the structure or non-structural components.

6.4 MITIGATION

It is strongly recommended that all CSWs are mitigated, at least to provide a margin for the overall capacity of the building, including any strengthening or other improvement. This may be achieved in a number of ways, for example:

- In the case of force controlled elements where there is no associated displacement issue (i.e. the element is capable of accommodating the full inelastic displacement demand for the building), the weak element may be strengthened to the lesser of the overstrength of the system or K_c times the intended standard for the building as a whole.

- In the case of force-controlled elements where strengthening will result in a structure that is incapable of accommodating the full inelastic displacement demand (eg squat piers in a pierced wall system), it may be preferable to provide an alternative load path for the gravity system, or to add or identify a secondary lateral load support mechanism that provides residual support to the full displacement

- In the case of displacement controlled elements which may impact on the structure and modify its performance (eg concrete wall panels), create sufficient clearance for the full inelastic design drift, which may be the lesser of the full drift calculated by a full detailed analysis, or the simplified drift check times K_c.

- In the case of displacement controlled elements which may lose support in the case of excessive movement, provide additional support for the element to the full inelastic design drift, times K_c.

6.5 EXAMPLES:

1. Simplified method of dealing with stair, without full analysis.

Consider a ten-storey moment frame structure with precast stairs. Stairs are full flight, seated on steel hangers at each end, with one end welded, the other seated within the depth of the landing using 80s style detail, with 50mm seating.

No analysis or calculations available for building. Floor heights are 3650mm floor to floor. The stair is a displacement-controlled system.

Assume that ULS drift is at or close to limit of 2.5% of storey height, say 2%. Therefore:

$$\Delta_{demand} = 0.02 \times 3600 = 72mm$$

Assuming that a residual seating of 20mm is required,

$$\Delta_{capacity} = 50 - 20 = 30mm$$

From Table 6-2, $K_d = 2$

Therefore, $\%NBS = \dfrac{\Delta_{capacity}}{K_d \Delta_{demand}} = \dfrac{30mm}{2 \times 72mm} = 21\%, < 33\%$

So the stair is earthquake prone and must be retrofitted. The required seating, S is:

$$S \geq K_c \times \Delta_{demand} + residual = 2 \times 72 + 20 = 164mm$$

2. Collector for shear wall structure – simplified analysis.

Consider a six storey building, with offset shear wall system, with three equal walls orthogonal to floor plate, as below. Torsional resistance is supplied by the orthogonal system.

Floors are precast double tees or similar, floor loading is office. Cladding is lightweight.

Basic plan dimension 30m x20m and the building is otherwise regular over height.

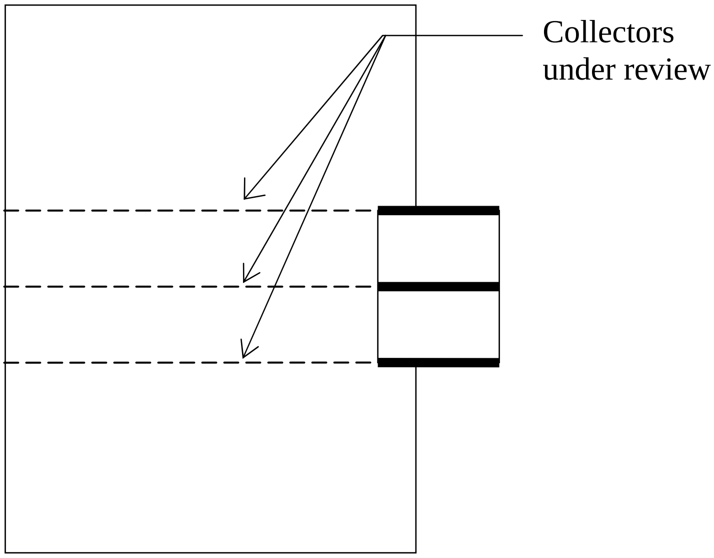

Figure 6-2: Example building floor plan

Estimate average floor load $p_{G+\psi Q} = 8.5kPa$

So single floor load = $P_{G+\psi Q} = 20 \times 30 \times 8.5 = 5100kN$

No calculations are available. Detailing may satisfy ductility provisions on complete review, but no overstrength calculations are available. Therefore, for evaluation of the collector, treat as μ=1.25 to current code.

Therefore, if assume that:

$T = 0.15 \times 6 = 0.9\sec$, with Class D subsoils,

then $C_h(T) = 2.09$

So, with Z=0.3

Structural Performance factor, S_p=0.9 (NZS3101:2006, cl. 2.6.2.2.1

Building Importance IL2, therefore Return Period Factor, R=1

No near fault condition, therefore $N(T,D)$=1

Therefore, $C(T) = C_h(T)ZRN(T,D) = 2.09 \times 0.3 \times 1 \times 1 = 0.627$

So, with $k_\mu = \mu$, k_m=1.25

Giving: $$C_d(T_1) = \frac{C(T_1)S_p}{k_\mu} = \frac{0.627 \times 0.9}{1.25} = 0.451$$

So, for a regular (over height) structure, max floor load through collectors at DBE

Therefore, demand, $N_{demand} = 2 \times 0.451 \times 5100 = 4600kN \equiv 1533kN / wall$

If the existing collectors are 3-HD24 bars per wall, analyse according to NZSEE guidelines:

Assume average steel yield stress, $F_y = 1.1 * 380 = 418MPa$
Strength reduction factor, $\phi = 1$

Gives capacity: $\varphi N_{capacity} = 1 \times 3 \times 452.4 \times 418 = 567kN / wall$

From Table 6-2, $K_d = 2$

and so $\%NBS = \frac{567}{2 \times 1533} = 18\%$

Therefore collectors are earthquake prone, and must be retrofitted to

Design load $N^* = K_c \times N_{demand} = 2 * 1533 = 3066kN / wall$

Alternatively, in the quantitative assessment phase a capacity design process may be followed to determine a more appropriate value.

7 REPORTING

On completion of the Qualitative and Quantitative (if required) Procedures, a report shall be prepared summarising the findings. This report is likely to be required by the BCA in considering the buildings ongoing occupancy and use and will form a basis for assessing the future repair and strengthening strategy.

Note that reports will be publicly available as they will form part of the record for the buildings to which they attach. **It is important that all parties to the process understand this and it is recommended that engineers advise their clients accordingly**. The report should be able to be relied upon by current building owners and the BCAs, but not necessarily future owners or other interested parties. It is recommended that report authors use appropriate disclaimers and seek separate legal advice if necessary.

It is generally assumed in the procedure and flow charts that the report and other related documentation will be lodged concurrently, but it may be advantageous to lodge the report separately and to use the report as a basis for which to discuss proposed repair and strengthening strategies with owners and their insurers, and with the BCA.

Following the lodging of the report and supporting documentation, buildings may be occupied if their existing condition allows it, with or without temporary repairs and/or shoring. Building Safety Ratings may be awarded, and timeframes may be agreed for future strengthening, assuming required.

7.1 REPORT OUTLINE

The Detailed Engineering Evaluation report should include but not be limited to the following:

1. Building Address – noting that where more than one building is located on a particular site, this should be clearly noted.

2. A full description of the building including plan dimensions, number of storeys, total plan area, occupancy and importance classification.

3. A full description of the structural system - both lateral and gravity, including materials and noting proprietary systems where applicable. It is expected that this would be drawn from a review of existing plans, where available. If no plans are available, it will be necessary to complete more intensive investigation on site in order to verify the structure.

4. A full description of the foundation system and ground conditions, noting the extent of geotechnical investigation completed.

5. Whether drawings are available or not, a prediction of the likely 'hot-spots' should be made in order to prioritise the required inspections. This may be informed by a set of generic building types and behaviours that is included in Appendix A.

6. A summary of damage sustained (plans and elevations if necessary), both structural and non-structural damage as it relates to building movement. This will include an assessment of the severity of the damage, including noting whether the damage is substantial as defined in Section 5.

7. A record of intrusive investigation of key elements and connection details. Include foundations and secondary structural elements as well as primary structure. This should be fully documented, with the required inspections identified during the plan review in steps 1&2 of the qualitative assessment procedure.

8. A consideration of the implications of and reasons for the damage. All failures must be addressed, with a conclusion drawn as to the reasons for the damage and the impact on both gravity and lateral structure.

9. Reference to generic building/material/configuration issues that are known to occur (from Appendix A); with verification of whether these have/have not occurred.

10. A statement must be made as to what elements have been specifically reviewed and what have been simply inferred. Mark areas of uncertainty on plans.

11. An estimate of the original lateral load resistance as %NBS, and post damage capacity, if significantly damaged. This must include consideration of the failure mechanism, clearly identifying whether the failure is brittle or ductile.

12. A list of items that are to be repaired or further investigations required, with prioritization if this work is to be staged in any way.

13. A statement (Design Features Report) describing the new load paths and load levels used in design (if changes are to be made), or otherwise detailing the existing load path.

14. Sketch (at least) plans for any proposed retrofit.

15. A completed table of Compliance Schedule items (refer Table 4-1 below)

All of the above would form part of any Building Consent for a repair, whereas only the first 10 may be required where no repairs are necessary i.e. no damage has been observed.

8 REFERENCES

[1] New Zealand Society for Earthquake Engineering *Building Safety Evaluation*. August 2009
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APPENDIX A

GENERIC BUILDING TYPES AND EXPECTED DAMAGE

Contents

The following outlines the generic performance and damage expected of a variety of building forms, constructed at different periods of New Zealand's construction history.

1A DUCTILE CONCRETE MOMENT RESISTING FRAMES

Ductile Concrete Moment Resisting Frames (DCMRFs) are buildings that have some to full modern detailing and are designed with practices that account for seismic attack. Largely restricted to the CBDs of the main cities, DCMRFs were constructed from about 1975 to the present.

In terms of New Zealand Standards for Concrete Structures: NZS3101: in 1982, the first version, there was an enormous leap in design and detailing practices for seismic performance of buildings. In 1995, there were significant improvements in detailing for robustness of structures; in 2006, further improvements were made. The sections of the Ministry of Works and a few leading structural engineers were developing and employing what was to become the accepted modern seismic engineering principles from 1975 onwards.

The lateral load resisting mechanism is typically frame action on all sides.

The seismic performance should be acceptable in most cases as detailing for ductility was employed and, through "capacity design", acceptable plastic mechanisms should have been selected.

Frame action should result in the preferred weak beam-strong column mechanism. In a limited number of cases, for buildings three storeys or less, ductile column sidesway mechanisms, may be acceptable.

Prior to NZS3101:1995, the design of interior columns was not up to full ductility detailing. If the columns are in buildings with high lateral drift then these columns may have insufficient ductility and gravity capacity in a major seismic event.

Lift shafts had evolved away from reinforced concrete cores to sheathed timber partitions. These partitions have little lateral capacity. The stairs and lift guides in these cores, can be significantly damaged due to the relatively large interstorey drifts expected in these MRFs. The presence of heavy reinforced concrete stairs can alter the behaviour of the building, acting as stiff props between floors (as do ramps). Many earlier versions of these stairs have sliding details where the stair slides within the plane of the supporting floors. These details have been found in many cases to have had the sliding joints compromised when maintenance personnel have filled the gaps to prevent failure of floor finishes and damage to heels. These stairs are prone to collapse due to jamming between floors.

Subsequently, from the mid-to-late '90s; detailing of these stairs with sliding of the lower landing over the supporting slab became the accepted feature. This detail offers less chance of being compromised, but also may have greater seating available. Also in the mid-90s, research at the University of Canterbury demonstrated that contiguous mid-height landings could be prone to damage due to tension failure at the junction to the lower flight. Standard detailing has since been changed to mitigate this form of failure.

Early floors and roofs are usually cast insitu concrete flat slabs, though at this time precast concrete floors with cast-in-place concrete toppings were emerging. By the early 1980s, most floors and roofs in commercial buildings were prestressed precast concrete units with concrete topping. Issues with precast concrete floors are highlighted in a section specifically written on these systems.

Problem	Fix	Impact
1. Columns (typically interior) have insufficient ductility and shear capacity.	a. Wrap the columns with steel plates or reinforced concrete or FRP jackets.	Intrusive, with disruption to the fit-out of each floor affected. If an exterior column, a very intrusive solution. May be impractical in many cases, where cladding impedes access, or where beam-column joints are inaccessible due to concrete floors or two-way frames.
	b. Supplementary columns added, to carry a portion of the gravity load.	Very intrusive on fit-out and architecture. No enhancement of the lateral capacity of the building, typically.
2. Column sidesway mechanism, ***not specifically designed for***, results in excessive ductility and shear demand on columns.	a. Add separate stiffer lateral load resisting system to reduce displacement.	Very intrusive solution. New system requires new load path, so that diaphragm and collectors need to be reassessed, and new foundations will be required.
	b. Introduce supplemental damping into the structure to reduce demand on frames	Dampers tend to be very expensive although less intrusive than complete new supplemental structure. If using hysteretic dampers, load to foundations increase significantly requiring upgrade.

Problem	Fix	Impact
	c. Strengthen columns and beam-column joints to force beam mechanisms	Very intrusive particularly on external frames. May be impractical in many cases, where cladding impedes access or where joints are inaccessible due to concrete floors or two-way frames.
3. Inadequate connections of floor and roof diaphragms to MRFs – common where the MRFs are adjacent to lifts and stair and hence separated from main diaphragm support	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that out of building load support to MRFs is still provided.
	b. Strengthen diaphragm in areas affected with steel straps, concrete or FRP overlay.	FRP least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise. Steel straps difficult to fix appropriately.
4. Inadequate stiffness of the structure as a whole meaning that the building exceeds drift limits.	a. Add separate stiffer lateral load resisting system to reduce displacement.	Very intrusive solution. New system requires new load path, so that diaphragm and collectors need to be reassessed, and new foundations will be required.
	b. Introduce supplemental damping into the structure to reduce displacement.	Dampers tend to be very expensive although less intrusive than complete new supplemental structure. If using hysteretic dampers, load to foundations increase significantly requiring upgrade.

Problem	Fix	Impact
5. Torsional behaviour through secondary structures (walls, stairs or ramps) which are incompatible with displacements of the moment resisting frame structures.	a. Modify structure that is inducing the torsional response (stairs or ramps or concrete stair).	Moderate work may be required. Cutting one end of stairs/ramps, possibly providing additional gravity support structure.
	b. Introduce stiffer load elements in parallel frames such as braced frames to reduce eccentricity	Significant intrusion into the existing space. May increase foundation loads to affected frames requiring expensive foundation work.
	c. Remove the concrete cores	Very extensive work will be required. If the core was part of the exterior fabric, can introduce weatherproofing issues in boundary walls.
6. Structural irregularity or discontinuity	a. Introduce strengthening in areas of high demand	Very extensive work will be required. Likely to be very intrusive
	b. Selective weakening of elements to reduce impact of irregularity	Not always able to achieve desired effect.
7. Inadequate seismic separation	a. Increase width of seismic separation	Very extensive work will be required. Likely to be very intrusive
	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures Structures must have compatible strength and stiffness and/or require strengthening to achieve this.
8. Low cycle fatigue	a. Detailed assessment required, with either strengthening or replacement of yielded elements required	Very intrusive if steel has to be cut out and replaced, or strengthening is required.

2A NON-DUCTILE CONCRETE MOMENT RESISTING FRAMES

Non-ductile Concrete Moment Resisting Frames (CMRFs) are buildings that lack the modern detailing and design practices that account for seismic attack. Concrete non-ductile MRFs are relatively common throughout New Zealand main metropolitan centres.

New Zealand-wide they were constructed from the early 1900s to around 1975. After this, the Ministry of Works required that public buildings have defined and acceptable mechanisms: "capacity design" and detailing for ductility. From here emerged better design practice from the structural engineers in general, producing buildings of the better expected performance.

From 1935 to 1965 these buildings were designed to a uniform load of 8-10% of gravity, applied uniformly at each level. From 1965, the seismic loading was amended to include reference to the building's period and for different seismic zones. In the longer period range, loads were in fact reduced. Refer to the NZSEE bulletin Vol 42, 2009, Fenwick and McRae *Comparison of New Zealand standards used for seismic design of concrete buildings.*

Often these buildings were constructed with concrete or masonry wall elements that were not seismically separated from the frames. Lateral load resisting mechanisms are often a mixture of wall action, particularly on boundaries through infills, with frame action on the open faces. Infill walls are less likely to exist from the 1960's on, leaving the buildings primarily reliant on pure frame action. Early provision for seismic separation was inadequate to maintain separation. Frame action may result in column sidesway mechanisms, particularly for the earlier frames.

The poor seismic performance, largely due to a lack of ductility and shear capacity in beams columns and beam column joints of these buildings, is due to insufficient transverse reinforcement (quantity and anchorage), poor design detailing of longitudinal reinforcement and lack of design control over where the plastic hinge zones will form (lacking "capacity design")

- Beam, column and beam-column joint shear failure
    - Column and beam-column joint shear failure will lead to collapse.

- Buckling of column bars, due to inadequate restraint of widely spaced transverse reinforcement

    - Develops a collapse failure almost immediately.

- Inadequate tensile capacity of longitudinal reinforcement, bar lapping and termination

    - Lower flexural strengths with rapid degradation of strength.

    - This poor performance is amplified where the main bars were plain round bars, used up until the mid-1960s.

- Local overstressing of sections of beams and columns and foundations, in part through the detailing issues noted above and from not ensuring that a desirable plastic mechanism is constrained to form

  - Loss of gravity capacity, particularly in columns and partial collapse or soft-storey mechanisms will occur.

- Indeterminate behaviour of the CMRFs result from the presence of non-structural elements such as infill walls, built-in staircases, ramps and concrete facades that are rigidly connected to the frames.

Floors and roof are usually cast insitu concrete flat slabs.

Problem	Fix	Impact
1. Torsional behaviour through infill boundary walls which are incompatible with the moment resisting frame structures.	a. Softening of walls through selective weakening to reduce eccentric behaviour	Extensive work may be required. Can introduce weatherproofing issues in boundary walls.
	b. Introduce stiffer load elements in parallel frames such as braced frames to reduce eccentricity	Significant intrusion into the existing space. May increase foundation loads to affected frames requiring expensive foundation work.
	c. Remove the infills	Very extensive work will be required. Loss of lateral strength of the building, new structures need to be added to compensate Can introduce weatherproofing issues in boundary walls.
2. Inadequate stiffness of the structure as a whole meaning that the building exceeds drift limits.	a. Add separate stiffer lateral load resisting system to reduce displacement.	Very intrusive solution. New system requires new load path, so that diaphragm and collectors need to be reassessed, and new foundations will be required.

Problem	Fix	Impact
	b. Introduce supplemental damping into the structure to reduce displacement.	Dampers tend to be very expensive although less intrusive than complete new supplemental structure. If using hysteretic dampers, load to foundations increase significantly requiring upgrade.
3. Column sidesway mechanism results in excessive ductility and shear demand on columns.	a. Add separate stiffer lateral load resisting system to reduce displacement.	Very intrusive solution. New system requires new load path, so that diaphragm and collectors need to be reassessed, and new foundations will be required.
	b. Introduce supplemental damping into the structure to reduce demand on frames	Dampers tend to be very expensive although less intrusive than complete new supplemental structure. If using hysteretic dampers, load to foundations increase significantly requiring upgrade.
	c. Strengthen columns and beam-column joints to force beam mechanisms	Very intrusive particularly on external frames. May be impractical in many cases, where joints are inaccessible due to concrete floors or two-way frames.
4. Inadequate connections of floor and roof diaphragms to infilled frames – common where boundary infilled frames are adjacent to lifts and stair and hence separated from main diaphragm support	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that face load support to walls is still provided.
	b. Strengthen diaphragm in areas affected with steel straps, concrete or FRP overlay.	FRP least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise. Steel straps difficult to fix appropriately.

Problem	Fix	Impact
5. Infills falling out of the frames.	a. Strengthen the connections of the infill panels to the frame. b. Provide supplemental support to the infill panel (cast-in-place concrete or shotcrete or steel frames)	a. Moderately intrusive b. Very intrusive.
6. Structural irregularity or discontinuity	a Introduce strengthening in areas of high demand	Very extensive work will be required. Likely to be very intrusive
	b Rationalise structural system	
7. Inadequate seismic separation	a. Increase width of seismic separation	Very extensive work will be required. Likely to be very intrusive
	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures Structures must have compatible strength and stiffness and/or require strengthening to achieve this.

3A CONCRETE SHEAR WALL STRUCTURES

Concrete structural walls, “shear walls”, started to be used from about 1925. Before the late 1970s, walls were not detailed for ductile behaviour during a major earthquake. The Concrete Standard, NZS3101:1982 was the first formal requirements for seismic design and detailing of structural walls, improvements were made in the 1995 and 2006 versions of the Standard. Poor performance of building with structural walls can be attributed to:

- Inadequate flexural strength
- Inadequate shear strength.
- Inadequate foundations, not sized for forces and displacements that are expected for a major earthquake.
- The connections of concrete floor diaphragms to walls may be compromised because of:
  - Stair and lift penetrations through the adjacent floor
  - Inadequate design of reinforcement across the floors and in to the walls
  - Displacements of the walls (such as by rocking, by design or by inadequate foundations) can damage the floor to wall connections. The structure being restrained by the walls can disconnect from the walls and collapse, as observed in large seismic events.
- Inadequate confinement to prevent brittle failure
- Under-reinforced walls, leading to non-ductile failure of flexural steel
- Poor detailing of flexural steel splices, leading to necking of steel, loss of confinement, or non-ductile failure

Walls constructed prior to the late 1970s are expected to have low to moderate damage. Observations in major earthquakes overseas indicate that most walls are unlikely to collapse. However, lightly reinforced walls have been observed to behave poorly, with damage to reinforcement focused at relatively few wide cracks (as opposed to the traditional fan-shaped crack patterns that are expected from testing). Singularly reinforced walls of less than 200 mm in thickness are more prone to overload as compared to doubly reinforced walls (typically thicker and with wider boundary elements at the ends of the walls). Lap lengths and locations in these walls are also problematic, often being placed in potential plastic hinge locations.

Heavily reinforced structural walls with well-confined boundary elements (constructed generally after the late 1970s) are expected to perform adequately in a major event. Use of precast panels as shear walls has in many cases resulted in compromise to the detailing in order to allow efficient precasting. Use of grouted ducts and splices has not always resulted in good behaviour – there has been incidence of ungrouted splices, and some welded details have exhibited brittle behaviour. In many cases the overall wall area is much greater than required, resulting in under-reinforced walls with low ductility demand. These walls have behaved poorly, resulting in the worst case observed, in fracture of the reinforcement with little obvious cracking. Buckling of steel at the splices due to lack of confinement is also a problem.

Connection details for diaphragms to walls have varied over the years. Early insitu floor systems generally have a significant area of concrete both in bearing and in shear, resulting in low stresses. This low stress may often compensate for poor detailing (lack of anchorage, plain bars), but overall ductility demand may still result in failure.

The introduction of precast floor systems has brought many more issues, including:

- Lack of room for collector elements in the floor
- Increased shear stresses in the topping

Even now, there is relatively little guidance in the standards for diaphragm design, but it was not until 1995 that strut-and-tie modelling was formally introduced into NZS3101, giving more flexibility to designers.

Further issues with precast concrete floors are highlighted in a section specifically written on these systems.

Problem	Fix	Impact
1. Inadequate flexural strength	a. Provide tension capacity by FRP, reinforcing rods or flat steel plate cut in to the wall (epoxied and bolted).	Moderately intrusive
	b. Build new boundary elements attached to the wall, reinforced vertically and transversely.	Highly intrusive
	c. Typically will require new foundations as a result of 4.a. and 4.b.	Very highly intrusive

Problem	Fix	Impact
2. Inadequate shear strength	a. Build a new reinforced wall or skin against the existing wall – New concrete and reinforcement needs to be placed.	Highly intrusive
	b. Apply a new skin – FRP typically, though steel plates can be used.	Moderately intrusive
	c. Embed in to walls reinforcing bars or steel strips strapped to the walls. Chasing out grooves and epoxying in the reinforcement or strips.	Moderately intrusive
	d. Selective weakening, by cutting some or all of the vertical bars in the wall.	Moderately intrusive. Limited use: usually requires addition main structure to be added elsewhere.
3. Inadequate foundations	a. Build new foundations, possibly including piles	Very highly intrusive
	b. Selective weakening, by cutting some or all of the vertical bars in the wall.	Moderately intrusive. Limited use: usually requires addition main structure to be added elsewhere.
4. Inadequate connections of floor and roof diaphragms to the walls.	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that face load support to walls is still provided.

Problem	Fix	Impact
	b. Strengthen diaphragm in areas affected with steel straps, concrete or FRP overlay.	FRP least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise of the floor Steel straps difficult to fix appropriately.
5. Structural irregularity or discontinuity	Introduce strengthening in areas of high demand	Very extensive work will be required. Likely to be very intrusive
6. Inadequate seismic separation	a. Increase width of seismic separation	Very extensive work will be required. Likely to be very intrusive
	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures Structures must have compatible strength and stiffness and/or require strengthening to achieve this.
7. Low cycle fatigue	a. Detailed assessment required, with either strengthening or replacement of yielded elements required	Very intrusive if steel has to be cut out and replaced, or strengthening is required.

4A SINGLE LEVEL TILT PANEL

These buildings are very common in Christchurch. Tilt panel construction was introduced into New Zealand during the late 1950's and quickly became a popular choice for industrial buildings, in conjunction with steel portal frames. This building type spread to commercial use, being very common for large supermarkets and shopping centres. Generally these buildings have lightweight metal roofs, supported on steel portal frames.

From 1965, these buildings were designed to increased seismic loads, which were then increased again in 1976 to a level that is approximately the same as current load levels.

Connections of panels have changed markedly since first introduced. Initially the panels were regarded as secondary structure, and lateral load resistance in the plane of the panels was often provided by (more flexible) steel cross-bracing. When the panel strength and stiffness was recognised, the panels were used as bracing, generally through welded connections, although site drilled and cast-in bolts were also used.

Fire is an issue also in many of these structures, both for spread of fire, where collapse of the steel frames may cause issues, or in the after-fire case, when the panel must maintain structural integrity. The former was recognised from the mid-90's, while the latter was recognised from the mid 60's, although neither has been consistently well dealt with.

Many STP's have potential seismic issues, for several reasons:

- Many of the connections details used are stiff and brittle and fail to address the long-term shrinkage and thermal action that the panels are subject to. Consequently, many panels crack at connection points, and the residual connection is non-ductile, so prone to failure in the event of movement. Assessing the strength of these connections is now difficult, but retrofitting is relatively simple. .

- More recent details include ducted splices, which may result in non-ductile failure where stresses are concentrated by the confining effect of the duct. In some cases, ducts have been found to not be grouted.

- A more important issue in many cases is the use of hard-drawn mesh reinforcement. The mesh has very low ductility, to the extent that a crack in the panel may be sufficient to fracture the mesh. These panels have the potential to fail dramatically under face loading. .

- During the 80's and 90's, panel thicknesses were reduced and panel spans increased, to the extent that many panels have the possibility of buckling in diagonal compression induced during earthquake, particularly when considering concurrency with face loading. In addition to the panel strength, many of the roof diaphragms are inadequate, particularly early tension bracing systems and there connections. .

Problem	Fix	Impact

Problem	Fix	Impact
1. Brittle panel connections and/or cracked panels at the connection.	a. Retrofit supplementary ductile connections. Epoxy cracks where weatherproofing compromised.	Minimal, provided connections are accessible (usually the case).
2. Hard–drawn wire mesh reinforcing or inadequate reinforcing contents making panels prone to non-ductile face loading failure.	a. Strengthen panels with externally applied fibre-reinforced polymer (FRP) sheets or strips.	Expensive solution, but non-intrusive. Must be strong enough to remain elastic as FRP has minimal ductility.
	b. Introduce secondary steel or reinforced concrete members to reduce spans and strengthen panels.	Possibly less expensive than FRP, but more intrusive, and may require supplementary foundations.
	c. Replace affected panels.	Expensive option in most cases, but may be practical where other changes are proposed.
3. Panel span/thickness ratio too high, leading to panel buckling concerns (particularly in panels with minimal edge restraint)	a. Add intermediate steel or reinforced concrete elements to reduce spans and decrease span/thickness ratio.	Very intrusive solution and new foundations may be required.
	b. Replace affected panels	Expensive option in most cases, but may be practical where other changes are proposed
4. Steel bracing inadequate	a. Retrofit new bracing or upgrade existing members and/or connections.	Relatively simple fix, although may be extensive.
5. Inadequate seismic separation	a. Increase width of seismic separation	Very extensive work will be required. Likely to be very intrusive
	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures Structures must have compatible strength and stiffness and/or require strengthening to achieve this.

5A MULTI-STOREY TILT PANEL

These buildings are quite common in Christchurch. As tilt panel construction became more popular and as crane capacity increased, engineers and architects looked for more innovative ways to use the technology. This heightened during the precast boom of the late 70's through the 80's

Uses extended to light commercial two-storey units (common in the industrial areas), tourist accommodation of 2-3 storeys, and to apartments (from the 80's). Similar technology was extended to larger multi-unit apartments and institutional accommodation of up to 6 stories and beyond, often using grouted splices to joint together multiple lifts of precast panels.

Floors and roofs of these buildings vary considerably. Many of the older units have timber floors with timber or steel roof structures. Many of the more cellular units have precast concrete topping-less floor systems, secured with weld-plates or small concrete/grout closing pours. Others use conventional precast topped floor systems, some with proprietary hanger systems to support the floors, where panels are continuous through joints.

From 1976 seismic loads were increased to approximately current load levels. Most of these buildings (particularly the taller ones) will have been built since that time.

A few MTP's have potential seismic issues, for several reasons:

- Many of the connections details used are stiff and brittle and fail to address the long-term shrinkage and thermal action that the panels are subject to. Consequently, many panels crack at connection points, and the residual connection is non-ductile, so prone to failure in the event of movement. Assessing the strength of these connections is now difficult, but retrofitting is relatively simple. .

- More recent details include ducted splices, which may result in non-ductile failure where stresses are concentrated by the confining effect of the duct. In some cases, ducts have been found to not be grouted.

- Some of these buildings may have hard-drawn mesh reinforcement. The mesh has very low ductility, to the extent that a crack in the panel may be sufficient to fracture the mesh. These panels have the potential to fail dramatically under face loading. .

- Many MTPs have little or no seating for precast flooring systems. In the some cases, there are very small (20mm) rebates in the panels to receive precast flooring elements, and cast-in sockets for topping steel to connect to. In the worst case, these units may lose seating and delaminate from the toppings. Other types include proprietary connection details that may initiate a break in the flooring units at a distance from the support.

- In addition to the panel strength, many of the roof and floor diaphragms may be inadequate, in the case of flexible metal or timber diaphragms. Connections may be poor and/or diaphragms weak. .

Problem	Fix	Impact
1. Brittle panel connections and/or cracked panels at the connection.	a. Retrofit supplementary ductile connections. Epoxy cracks where required for weatherproofing.	Minimal, provided connections are accessible (usually the case).
2. Hard–drawn wire mesh reinforcing or inadequate reinforcing contents making panels prone to non-ductile face loading failure.	a. Strengthen panels with externally applied fibre-reinforced polymer (FRP) sheets or strips.	Expensive solution, but non-intrusive. Must be strong enough to remain elastic as FRP has minimal ductility.
	b. Introduce secondary steel or reinforced concrete members to reduce spans and strengthen panels.	Possibly less expensive than FRP, but more intrusive, and may require supplementary foundations.
	c. Replace affected panels.	Expensive option in most cases, but may be practical where other changes are proposed.
3. Poor seating connections for concrete floor systems	a. Provide adequate seating	
4. Steel and timber bracing inadequate connections	a. Retrofit new connections.	Relatively simple fix in light commercial structures, although may require removal of linings. More difficult in residential or institutional structures where more intrusive
5. Structural irregularity or discontinuity	Introduce strengthening in areas of high demand	Very extensive work will be required. Likely to be very intrusive
6. Inadequate seismic separation	a. Increase width of seismic separation	Very extensive work will be required. Likely to be very intrusive
	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures. Structures must have compatible strength and stiffness and/or require strengthening to achieve this.

6A FULLY FILLED REINFORCED CONCRETE MASONRY

Fully (solid) filled reinforced concrete masonry was used from the mid-1970s. As the cells or the flues are fully filled with concrete grout, these walls are stronger that the lightly reinforced partially filled concrete masonry walls and behave similarly to a reinforced cast-in-place wall of the same dimensions.

Fully filled reinforced masonry walls are an alternative way of building structural walls. Therefore the performance issues of structural concrete walls will apply to these concrete masonry walls.

Poor performance of buildings with fully filled reinforced concrete masonry walls can be attributed to:

- Inadequate flexural strength
- Inadequate shear strength.
- Inadequate foundations, not sized for forces and displacements that are expected for a major earthquake.
- The connections of concrete floor diaphragms to walls may be compromised because of:
    - Stair and lift penetrations through the adjacent floor
    - Inadequate design of reinforcement across the floors and in to the walls
    - Displacements of the walls (such as by rocking, by design or by inadequate foundations) can damage the floor to wall connections. The structure being restrained by the walls can disconnect from the walls and collapse.
    - Floors disconnecting from the walls due to inadequate connection hardware or the face shells of the blocks separating from the grouted flues.
    - Structural irregularity or discontinuity
- Inadequate quality control during construction has resulted in poor grout take, particularly at the base of walls and in lap zones. In the worst cases, some cores were unfilled. Both of these have resulted in poor behaviour of the walls.
- Fully filled reinforced concrete masonry walls, constructed from the mid-1990s, are not expected to have major damage. However, a remaining issue will be the integrity of the connections of the floors to the walls (though improved over that used for earlier walls).

Problem	Fix	Impact
1. Inadequate shear strength	a. Build a new reinforced wall or skin against the existing wall – New concrete and reinforcement needs to be placed.	Highly intrusive solution.
	b. Apply a new skin – FRP typically, though steel plates can be used.	Moderately intrusive.
	c. FRP or steel strips strapped to the walls. Epoxying the strips to the wall.	Moderately intrusive.
	d. Selective weakening, by cutting some or all of the vertical bars in the wall.	Moderately intrusive. Limited use: usually requires addition main structure to be added elsewhere.
2. Inadequate foundations	a. Build new foundations, possibly including piles	Very highly intrusive
	b. Selective weakening, by cutting some or all of the vertical bars in the wall.	Moderately intrusive. Limited use: usually requires addition main structure to be added elsewhere.
3. Inadequate connections of floor and roof diaphragms to the walls.	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that face load support to walls is still provided.

Problem	Fix	Impact
	b. Strengthen diaphragm in areas affected with steel straps, concrete or FRP overlay. Plywood overlay on timber floors also.	FRP and ply wood least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise. Steel straps difficult to fix appropriately.
4. Inadequate flexural strength	a. Provide tension capacity by FRP, reinforcing rods or flat steel plate bonded to the wall (epoxied and bolted).	Moderately intrusive
	b. Build new boundary elements attached to the wall, reinforced vertically and transversely.	Highly intrusive
	c. Typically will require new foundations as a result of 4.a. and 4.b.	Very highly intrusive
5. Structural irregularity or discontinuity	a. Introduce strengthening in areas of high demand	Very extensive work will be required. Likely to be very intrusive
6. Inadequate seismic separation	a. Increase width of seismic separation	Very extensive work will be required. Likely to be very intrusive
	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures Structures must have compatible strength and stiffness and/or require strengthening to achieve this.

7A PARTIALLY FILLED CONCRETE MASONRY

Lightly reinforced partially filled concrete masonry was used from the mid-1940s. In order to save costs, only the main cells or flues, containing reinforcement, where filled with concrete grout. This meant that significant sections (panels of rectangular shape) where made up of empty blocks mortared together. Such voids produce a weaker wall than completely filled ("solid") concrete masonry wall or much weaker wall than a reinforced cast-in-place wall of the same dimensions.

Poor performance of building with LRPF concrete masonry walls can be attributed to:

- Inadequate flexural strength
- Inadequate shear strength.
- Inadequate foundations, not sized for forces and displacements that are expected for a major earthquake.
- The connections of concrete floor diaphragms to walls may be compromised because of:
    - Stair and lift penetrations through the adjacent floor
    - Inadequate design of reinforcement across the floors and in to the walls
    - Displacements of the walls (such as by rocking, by design or by inadequate foundations) can damage the floor to wall connections. The structure being restrained by the walls can disconnect from the walls and collapse.
    - Floors disconnecting from the walls – inadequate connection hardware or the face shells of the blocks separating from the grouted flues.
- Inadequate quality control during construction has resulted in poor grout take, particularly at the base of walls and in lap zones. In the worst cases, some cores were unfilled. Both of these have resulted in poor behaviour of the walls.
- Structural discontinuity or irregularity

LRPF concrete masonry walls, prior to the mid-1990s, are expected to have moderate damage. After that period, the walls are expected to have low damage. However, a remaining issue will be the integrity of the connections of the floors to the walls (though improved over that used for earlier walls).

Masonry walls are an alternative way of building structural walls and tilt panel walls. Therefore the performance issues of structural walls and tilt up panels will apply to LRPF concrete masonry walls.

Problem	Fix	Impact
1. Inadequate shear strength	a. Build a new reinforced wall or skin against the existing wall – New concrete and reinforcement needs to be placed.	Highly intrusive solution.
	b. Apply a new skin – FRP typically, though steel plates can be used.	Moderately intrusive.
	c. FRP or steel strips strapped to the walls. Epoxying the strips to the wall.	Moderately intrusive.
	d. Selective weakening, by cutting some or all of the vertical bars in the wall.	Moderately intrusive. Very limited use: usually requires addition main structure to be added elsewhere.
2. Inadequate foundations	a. Build new foundations, possibly including piles	Very highly intrusive
	b. Selective weakening, by cutting some or all of the vertical bars in the wall.	Moderately intrusive. Very limited use: usually requires addition main structure to be added elsewhere.
3. Inadequate connections of floor and roof diaphragms to the walls.	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that face load support to walls is still provided.
	b. Strengthen diaphragm in areas affected with steel straps, concrete or FRP overlay. Plywood overlay on timber floors also.	FRP and ply wood least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise. Steel straps difficult to fix appropriately.

Problem	Fix	Impact
4. Inadequate flexural strength	a. Provide tension capacity by FRP, reinforcing rods or flat steel plate bonded to the wall (epoxied and bolted).	Moderately intrusive
	b. Build new boundary elements attached to the wall, reinforced vertically and transversely.	Highly intrusive
	c. Typically will require new foundations as a result of 4.a. and 4.b.	Very highly intrusive
5. Structural irregularity or discontinuity	a. Introduce strengthening in areas of high demand	Very extensive work will be required. Likely to be very intrusive
6. Inadequate seismic separation	a. Increase width of seismic separation	Very extensive work will be required. Likely to be very intrusive
	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures Structures must have compatible strength and stiffness and/or require strengthening to achieve this.

8A WELDED AND BOLTED STEEL MOMENT FRAMES

These buildings are relatively uncommon in Christchurch. New Zealand-wide they were constructed any time from the 1950's to date. In practice, steel has suffered behind concrete for many years from cost, and also the impact of the boilermakers' union difficulties of the 70's. Not until the 90's did steel become more common again for anything other than low-rise construction.

The earlier versions of these buildings are similar in construction to the riveted frames that they replaced, with insitu concrete stair and lift enclosures and concrete infill walls. Later versions used spray-on or boarded fire protection.

From 1965, these buildings were subject to increased seismic loads which are closer to current standards, particularly for the taller more flexible frames.

Floors and roof are usually cast insitu concrete flat slabs for the earlier buildings. Later buildings may have precast floor systems (from the 70s) or composite metal tray floor systems (from the late 80s).

Lateral load resisting mechanisms are often a mixture of wall action, particularly on boundaries through infills, with frame action on the open faces. Infill walls are less likely to exist from the 1960's on, leaving the buildings primarily reliant on pure frame action. Frame action may result in column sidesway mechanisms, particularly for the earlier frames.

These buildings are generally quite flexible, although this may not be an issue provided that there is sufficient clearance to the adjacent buildings. Where there is not, pounding may be a problem, particularly if adjacent floor levels do not match. In addition, P-delta effects need to be considered.

Problem	Fix	Impact
1. Torsional behaviour through infill boundary walls or lift and stair enclosures which are incompatible with the steel frame structures.	a. Softening of walls through selective weakening to reduce eccentric behaviour	Extensive work may be required. Can introduce weatherproofing issues in boundary walls.
	b. Introduce stiffer load elements in parallel frames such as braced frames to reduce eccentricity	Significant intrusion into the existing space. May increase foundation loads to affected frames requiring expensive foundation work.
2. Inadequate stiffness of the structure as a whole meaning that the building exceeds drift limits.	a. Add separate stiffer lateral load resisting system to reduce displacement.	Very intrusive solution. New system requires new load path, so that diaphragm and collectors need to be reassessed, and new foundations will be required.

Problem	Fix	Impact
	b. Introduce supplemental damping into the structure to reduce displacement.	Dampers tend to be very expensive although less intrusive than complete new supplemental structure. If using hysteretic dampers, load to foundations increase significantly requiring upgrade.
4. Column sidesway mechanism results in excessive ductility demand on columns.	a. Add separate stiffer lateral load resisting system to reduce displacement.	Very intrusive solution. New system requires new load path, so that diaphragm and collectors need to be reassessed, and new foundations will be required.
	b. Introduce supplemental damping into the structure to reduce demand on frames	Dampers tend to be very expensive although less intrusive than complete new supplemental structure. If using hysteretic dampers, load to foundations increase significantly requiring upgrade.
	c. Strengthen columns to force beam mechanisms	Very intrusive particularly on external frames. May be impractical in many cases, where joints are inaccessible due to concrete floors or two-way frames.
5. Inadequate connections of floor and roof diaphragms to walls – common where boundary walls are adjacent to lifts and stair and hence separated from main diaphragm support	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that face load support to walls is still provided.
	b. Strengthen diaphragm in areas affected with steel straps, concrete or FRP overlay.	FRP least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise. Steel straps difficult to fix appropriately.
6. Structural irregularity or discontinuity	a. Introduce strengthening in areas of high demand	Very extensive work will be required. Likely to be very intrusive
7. Inadequate seismic	a. Increase width of seismic	Very extensive work will be

Problem	Fix	Impact
separation	separation	required. Likely to be very intrusive
	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures Structures must have compatible strength and stiffness and/or require strengthening to achieve this.
8. Low cycle fatigue	a. Detailed assessment required, with either strengthening or replacement of yielded elements required	Very intrusive if steel has to be cut out and replaced, or strengthening is required.

9A RIVETED STEEL MOMENT FRAMES

These buildings are relatively uncommon in Christchurch. New Zealand-wide they were constructed any time from the early 1900's through to the 1950's, when bolting and welding became prevalent.

The steel frames are generally concrete encased for fire protection. Often boundary walls are infill concrete insitu walls, again for fire resistance. Stair and lift enclosures are also typically insitu concrete.

Floors and roof are usually cast insitu concrete flat slabs, with varying forms of reinforcement. Early versions may have vaulted or arched supports, with later versions being plain round bar reinforcement.

Lateral load resisting mechanisms are often a mixture of wall action, particularly on boundaries through infills, with frame action on the open faces. Frame action may result in column sidesway mechanisms, particularly for the earlier frames.

Some RSMFs are expected to be EPBs, particularly in cases where one or more adjacent sides have concrete infill walls. Another common hazard is from the cladding which may include substantial areas of insitu concrete or heavy masonry stiff, brittle cladding. These buildings are generally quite flexible, although this may not be an issue provided that there is sufficient clearance to the adjacent buildings. Where there is not, pounding may be a problem, particularly if adjacent floor levels do not match. In addition, P-delta effects need to be considered.

Problem	Fix	Impact
1. Torsional behaviour through infill boundary walls or lift and stair enclosures which are incompatible with the steel frame structures.	a. Softening of walls through selective weakening to reduce eccentric behaviour b. Introduce stiffer load elements in parallel frames such as braced frames to reduce eccentricity	Extensive work may be required. Can introduce weatherproofing issues in boundary walls. Significant intrusion into the existing space. May increase foundation loads to affected frames requiring expensive foundation work.
2. Inadequate stiffness of the structure as a whole meaning that the building exceeds drift limits.	a. Add separate stiffer lateral load resisting system to reduce displacement.	Very intrusive solution. New system requires new load path, so that diaphragm and collectors need to be reassessed, and new foundations will be required.

Problem	Fix	Impact
	b. Introduce supplemental damping into the structure to reduce displacement.	Dampers tend to be very expensive although less intrusive than complete new supplemental structure. If using hysteretic dampers, load to foundations increase significantly requiring upgrade.
3. Riveted joints lack strength, either with discontinuous flange plates, or through lack of rivets.	a. Add separate stiffer lateral load resisting system to reduce load to joints	Very intrusive solution. New system requires new load path, so that diaphragm and collectors need to be reassessed, and new foundations will be required.
	b. Introduce supplemental damping into the structure to reduce demand on frames	Dampers tend to be very expensive although less intrusive than complete new supplemental structure. If using hysteretic dampers, load to foundations increase significantly requiring upgrade.
	c. Strengthen joint areas by removing concrete to upgrade joint, or by adding external reinforcing.	Difficult and messy work, potentially affecting exterior of building also. Joint by joint is relatively expensive work.
4. Column sidesway mechanism results in excessive ductility demand on columns.	a. Add separate stiffer lateral load resisting system to reduce displacement.	Very intrusive solution. New system requires new load path, so that diaphragm and collectors need to be reassessed, and new foundations will be required.
	b. Introduce supplemental damping into the structure to reduce demand on frames	Dampers tend to be very expensive although less intrusive than complete new supplemental structure. If using hysteretic dampers, load to foundations increase significantly requiring upgrade.

Problem	Fix	Impact
	c. Strengthen columns to force beam mechanisms	Very intrusive particularly on external frames. May be impractical in many cases, where joints are inaccessible due to concrete floors or two-way frames.
5. Inadequate connections of floor and roof diaphragms to walls – common where boundary walls are adjacent to lifts and stair and hence separated from main diaphragm support	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that face load support to walls is still provided.
	b. Strengthen diaphragm in areas affected with steel straps, concrete or FRP overlay.	FRP least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise. Steel straps difficult to fix appropriately.
6. Structural irregularity or discontinuity	a. Introduce strengthening in areas of high demand	Very extensive work will be required. Likely to be very intrusive
7. Inadequate seismic separation	a. Increase width of seismic separation	Very extensive work will be required. Likely to be very intrusive
	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures Structures must have compatible strength and stiffness and/or require strengthening to achieve this.

10A STEEL CONCENTRIC BRACED FRAMES

There are a number of such buildings in Christchurch, frequently industrial or other low-height structures. New Zealand-wide they were constructed any time from the early 1900's through to date, although there may be few prior to the 40s and 50s. Many such buildings will have mixed systems with SMRFs (portal frames) in the transverse direction.

In addition to being used for low-height structures of one or two storeys, this form of construction was frequently used for lateral support of upper lightweight storeys, penthouses or plantrooms in multi-storey construction. Equally, concentric bracing is commonly used for roof bracing in lightweight structures.

Prior to1992, there was little consideration of the true ductility of this system, with the possible assumption that steel, as a ductile material, would ensure a ductile system. This has been exposed in observations of performance from previous earthquakes where these systems have behaved poorly, often with failure of braces or connections.

For most lightweight structures, it is possible that wind load may govern the design, but for penthouse structures and heavier buildings, this is unlikely. In these cases, inelastic demand on the braces may result in failure at connections.

From 1992, issues with CBFs were recognised in the Standard, with the requirement that notched braces be used to ensure an element of capacity design, or that the system be designed for elastic or nominally ductile response with a suitable magnification (Cf) factor to provide reliable performance over a greater range.

In industrial buildings, maintenance has often been an issue, as has alteration. It has been common to find braces with connections that have either corroded to the point of losing significant capacity, or where braces have been removed or had connections weakened. In some such buildings, the cladding may have taken a significant load, but with resulting added displacement.

In more recent years, proprietary bracing systems have been used that have cast or fabricated connectors and threaded tension members. Although these have been tested within their theoretical elastic limits, they have not been tested as a system, and cannot be relied upon in any situation where there may be some inelastic demand. As such, they may only form part of a capcity design-protected secondary system, at least until further testing has been completed. There were several incidences of premature failure of these systems in the September 4 earthquake, and possibly more in February 22.

Problem	Fix	Impact
1. Braces inadequately sized to meet current loads and/or ductility requirements	a. Replace or strengthen braces, taking care to follow the load path throughout rest of structure to ensure capacity of remainder of system adequate	Minor, provided accessible. Ensure any fire proofing to gravity structure maintained or replaced.

Problem	Fix	Impact
2. Braces lack notches	a. Notch members in order to achieve sufficient member ductility and also to protect secondary (non-yielding) elements	Minor, provided accessible. Ensure any fire proofing to gravity structure maintained or replaced.
3. Connections inadequate for capacity of braces	a. Upgrade connections for capacity of brace.	Minor provided accessible.
4. Inadequate seismic separation	a. Increase width of seismic separation	Very extensive work will be required. Likely to be very intrusive
	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures Structures must have compatible strength and stiffness and/or require strengthening to achieve this.

11A STEEL ECCENTRIC BRACED FRAMES

This form of construction was introduced from the US in the late ‘80s/early ‘90s and there are relatively few examples in Christchurch. Providing (generally) a stiff but ductile yielding mechanism, these frames are suitable for structures from low to high-rise. Although less economic than CBFs to fabricate, they should provide superior behaviour under lateral load.

All EBF systems should have been designed to modern standards as the earliest versions will have been generally design using the HERA Seismic Design Manual, which was a precursor to the updated seismic design provisions of NZS3404:1992.

Common issues for EBFs may include:

- Lack of restraint to yielding portion of link and/or to braces
- Inadequate collector axial capacity
- Poor connectivity to the structural diaphragm,

All of these characteristics have been identified through observations of seismic performance in the Christchurch or Gisborne earthquakes.

A likely concern with EBFs is with the likely future performance, due to the effects of low cycle fatigue on the link. In practice this depends on the number of cycles of yield that the frame has been through, and the ductility demand.

Problem	Fix	Impact
1. Inadequate lateral or torsional restraint to links and braces	a. Provide additional restraint	Minor, provided accessible. Ensure any fire proofing to gravity structure maintained or replaced.
2. Inadequate collector capacity	b. Plate collector in order to provide added capacity, or reduce effective length if practical	Minor, provided accessible. Ensure any fire proofing to gravity structure maintained or replaced.
3. Inadequate connection of frame to diaphragm	a. Provide additional collector elements to the frames, taking care not to affect actions on link	Minor to very intrusive, depending on access. Ensure any fire proofing to gravity structure maintained or replaced.
4. Inadequate seismic separation	a. Increase width of seismic separation	Very extensive work will be required. Likely to be very intrusive

Problem	Fix	Impact
	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures Structures must have compatible strength and stiffness and/or require strengthening to achieve this.
8. Low cycle fatigue	a. Detailed assessment required, with possible replacement of link required	Very intrusive if steel has to be cut out and replaced.

12A CONCRETE OR STEEL FRAME WITH INFILL

These buildings are relatively common throughout New Zealand main metropolitan centres. New Zealand-wide they were constructed from the early 1900s to the mid 1960s. After this, pure frame action of Moment Resisting Frames (MRFs) was relied upon.

Early styles of CSFI involved unreinforced masonry infills between the beams and columns. Lightly reinforced concrete walls were a rare option in the later period. On boundaries to other buildings, these walls usually had few windows. On street fronts, these walls can have extensive penetrations.

These unfilled frames behave much like wall structures. Typically the concrete frames where not designed to act as a moment resisting frame. The columns tended to perform as tension and compression boundary elements in the wall-like structure.

Concrete columns and beams are relatively lightly reinforced as compared to modern MRFs. Steel frames were typically riveted frames encased in concrete. Floors and roof are usually cast insitu concrete flat slabs for the frames with integral infills.

The performance of these infilled frames, in Christchurch with relative significant seismicity:

- The infills are involved in the action of the frame, with either destruction of the infill which fails in horizontal shear; this results in flexure-shear failure of the adjacent columns. For the building, a soft-storey sway mechanism is quite likely, particularly for the earlier frames.
    - This is the main risk and is aggravated by the presence of windows.
    - The presence of windows can introduce a short column shear failure
- The infills are sufficient strong to work with the frame, as a wall element.
    - It is suspected that there are a limited number of such cases.

Awareness of earthquakes and changes in architecture after 1965 resulted in MRFs with infills that were not supposed to interfere with frame action. This was achieved by having gaps between the infill wall, now acting simply as cladding, and the columns and beam above. The infills were often reinforced concrete block masonry. However, up until mid 1980s, these gaps were not large enough to accommodate the distortion of the frame relative to the wall infills. These infills would interfere with the frame action, leading to any of the possible column failure mode described above.

Lateral load resisting mechanisms are often a mixture of wall action, particularly on boundaries through infills, with frame action on the open faces.

Problem	Fix	Impact
1. Torsional behaviour through infill boundary walls which are incompatible with the moment resisting frame structures.	a. Softening of walls through selective weakening to reduce eccentric behaviour	Extensive work may be required. Can introduce weatherproofing issues in boundary walls.
	b. Introduce stiffer load elements in parallel frames such as braced frames to reduce eccentricity	Significant intrusion into the existing space. May increase foundation loads to affected frames requiring expensive foundation work.
2. Column sidesway mechanism results in excessive ductility and shear demand on columns.	a. Strengthen the infill panels and connection of these to the frames to ensure wall action.	Reasonably intrusive requiring either shotcrete or cast-in-place walls to be cast against the existing infilled frames. Connections from each new wall – skin must be made through each floor and to each of the infilled wall sections. And new foundations will be required.
	b. Add separate stiffer lateral load resisting system (concrete walls typically) to reduce lateral displacement.	Very intrusive solution. New system requires new load path, so that diaphragm and collectors need to be reassessed, and new foundations will be required.
	c. Retro fit with base isolation to reduce demand on the building; suited to the squatter wall-like buildings	Post-installed base isolation will be very expensive. New substructures and foundations will be built under the existing building.
3. Inadequate connections of floor and roof diaphragms to infilled frames – common where boundary infilled frames are adjacent to lifts and stair and hence separated from main diaphragm support	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that face load support to walls is still provided.
	b. Strengthen diaphragm in areas affected with steel straps, concrete or FRP overlay.	FRP least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise. Steel straps difficult to fix appropriately.
4. Infills falling out of the	a. Strengthen the	a. Moderately intrusive

Problem	Fix	Impact
frames.	connections of the infill panels to the frame. b. Provide supplemental support to the infill panel (cast-in-place concrete or shotcrete or steel frames)	b. Very intrusive.
5. Structural irregularity or discontinuity	a. Introduce strengthening in areas of high demand	Very extensive work will be required. Likely to be very intrusive
6. Inadequate seismic separation	a. Increase width of seismic separation	Very extensive work will be required. Likely to be very intrusive
	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures Structures must have compatible strength and stiffness and/or require strengthening to achieve this.

13A UNREINFORCED MASONRY BEARING WALLS

Prevalent from the 1850's through to the mid-1930's, although some may have persisted after that time in industrial and residential use.

Floors and roof generally light timber framed. Some are known to have concrete floors which may be constructed over brick or stone vaulting.

Most UMB buildings are expected to be EPBs, including many which have been secured or strengthened prior to the Building Act.

Problem	Fix	Impact
1. Lack of shear capacity	a. Enhancement of existing shear strength through concrete or FRP overlays	May require increase in foundation strength. Will need to have existing linings removed and reinstated.
	b. New concrete or steel lateral load resisting structure.	Significant intrusion into the existing space. May compromise any heritage fabric more than less intrusive methods. Difficult to make new system compatible with old.
2. Rocking resistance of walls or piers is too low	a. Extend wall or foundation length to increase resistance	Extensive excavation and opening of ground floor required.
3. Inadequate connections of floor and roof diaphragms to walls	a. Open up floors and/or ceilings to provide added connections.	Extensive reinstatement to ceilings and or floors required. Damaging to heritage fabric
4. Diaphragms lacking sufficient strength to transfer shear to supporting elements	a. Plywood overlay diaphragm or ceiling diaphragm may be added	
5. Structural irregularity or discontinuity	a. Introduce strengthening in areas of high demand	Very extensive work will be required. Likely to be very intrusive
6. Inadequate seismic separation	a. Increase width of seismic separation	Very extensive work will be required. Likely to be very intrusive

	b. Tie adjacent structures together to prevent pounding	Requires common ownership or complex legal structures Structures must have compatible strength and stiffness and/or require strengthening to achieve this.

14A SHALLOW FOUNDATIONS

Foundation elements are considered to be shallow when the depth to breadth ratio is less than 5 (D/B <5), generally including the following:

- Isolated pads
  Isolated pads are seldom appropriate for building foundations subject to seismic actions, especially in Christchurch where the ground conditions are known to be variable and mostly unsuitable. These could well have suffered from differential settlement and differential lateral movement, especially in areas of liquefaction.

- Strip/beam footings
  Continuity of foundation elements is important to ensure integrity of a structure subject to differential ground movements. Where differential movements are excessive, the footings should be checked for structural damage.

- Pad and Tie Beam foundations
  Similar to above

- 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.

- Raft foundations
  Raft foundations are similar to mat foundations but have sufficient strength and stiffness to behave essentially as a rigid body when accommodating differential ground movements. True rafts are rare as the required levels of strength and stiffness are prohibitive.

A key generic issue relevant to all types of shallow foundations is to decide whether or not shallow foundations remain appropriate for the structure or whether underpinning with deep foundations is required. This decision should not be based solely on the performance of the foundation to date, but on the risks of damaging settlement from future events, based on proper analysis of the ground conditions. While differential settlements as measured post February 2011 may be within tolerable limits for the structure, another earthquake could produce similar or greater differential movement, cumulative to the first, which could then lead to severe structural damage or failure.

Settled footings may be the result of liquefaction or soil response at depth, or 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”, which is much higher than factors typically used for other load combinations, resulting in a high risk that the ultimate capacity of the footing will be exceeded at the design load. In reality, the bearing capacity of shallow foundations is reduced by inertial effects during shaking as well as from increased pore water stresses, which in combination with high seismic loading from the structure can induce large deformation.

Some foundations have suffered from non-uniform aspects such as basements under only parts of the building, irregular footprints with differential movements in plan, or piles installed to provide tension capacity under parts of a shallow foundation only. Particular attention should be given to the areas around such features in looking for damage, differential movement etc. A number of buildings have suffered differential movement due to uplift of basements under part of the ground floor.

Basements can be exposed to high uplift pressures generated in liquefied sands or in loose gravels. This can result in vertical displacement as well as damage to the basement floor, depending on the construction as a raft or slab between footings or piles. Uplifted basements, particularly those on gravels rather than liquefied sands, may have large voids below them. Basement walls may have been subjected to lateral earth pressures much higher than normal static loading. Many basements were partially flooded after the earthquake, as the result of damage to walls, floor or tanking.

Where gapping has occurred adjacent to footings, the gaps should be filled with sand-bentonite grout to restore the full passive resistance of the soil.

Where rocking of foundations has occurred (or suspected to have occurred) gaps may exist underneath foundation elements or under the edges of elements. Locate and fill such gaps.

Problem	Fix	Impact
1. Excessive settlement	a. If settlement tolerable but structure at risk if similar settlement occurred in future earthquake, and bearing capacity of ground suitable for shallow foundations, widen foundations	Difficult with boundary walls – may require offset foundation and crossbeams to take out eccentricity.
	b. underpin with piles. This may also allow re-levelling	May not have sufficient access for piling rig, both to perimeter and internal foundations. Consider the type of pile carefully and check compatibility with existing foundations for both vertical and lateral actions.
	c. Compaction grouting can relevel foundations and stiffen soils to reduce settlement in a future earthquake	Not suitable in all soils, may require drilling through floor.

Problem	Fix	Impact
2. Lateral spread	a. Consider external damming or buttressing of soils in order to restrain future spread	Not likely to be practical in many cases – only applicable when there is sufficient access and work can be achieved on some site.
	b. external sheetpile wall or piles, with ground anchors to restrain lateral load	Requires access for plant and suitable ground conditions for toe of sheetpiling and anchorage.
	c. underpinning may be installed to perimeter foundations. May also need addition of foundation ties across the building to counteract future spread.	Relatively simple to install, provided clear access available. May still be vulnerable to future damage if lateral spread not addressed externally.
3. Basement with uplift	Grout under the floor to fill any voids	May compromise any tanking; uncertainty as to how effective grouting may have been.

Note that foundation-related problems for shallow footings may have a 'binary' aspect, i.e. if there has been excessive movement, there may be no effective repair solution even if the super structure is relatively undamaged.

15A DEEP FOUNDATIONS

Foundation elements are considered to be deep when the depth to breadth ratio is greater than 5 (D/B >5), generally, in Christchurch, the following deep foundation types are in use:

- Driven concrete piles
  Typically these are 6 m to 15 m long, with some as short as 2 – 3m and rare buildings with piles in excess of 20m and are driven to found onto a dense gravel stratum. Few buildings in Christchurch have been founded on driven piles larger than 150mm square section in the last 15 years due to resource consent issues to do with noise and vibration during driving. They are typically designed as end bearing, although a contribution from side friction may be included. Both compression and uplift capacity from side resistance may be lost with liquefaction. Lateral capacity may also be affected if adequate embedment has not been achieved into the dense soils.

- Driven steel piles
  Not widely used in Christchurch but may be driven to found onto the more dense gravel strata at depth. Uplift capacity from friction may be lost with liquefaction unless adequate embedment has been achieved into a dense (non liquefiable) soil.

- Driven timber piles
  Typically these tend to be shallower than other pile types and may be vulnerable to both bearing and lateral capacity strength loss within or underneath the bearing stratum. Not common for commercial buildings

- Bored cast-in-place piles
  Usually 6 – 15m deep and 0.6 to 1.2m diameter, occasionally up to 1.5m diameter and up to 20m deep. Typically excavated in water filled steel casing which is withdrawn during concreting. Although often designed as end bearing with some contribution from side resistance, in reality, for many of them, the gravity loads will have been carried since construction by the side resistance mechanism. Loss of side resistance from pore water pressure effects during shaking may lead to settlement from gravity loads, (see discussion below).

  Uplift in bored piles in Christchurch is resisted by side resistance. There is no knowledge of belling or underreaming of any piles in Christchurch, where the cohesionless sands and gravels below the water table do not allow undercutting or even any excavation outside a fully cased hole without bentonite slurry support.

- Bulb (Franki) piles
  Common on many buildings between about 1970 and late 1980s. Steel casings were bottom driven to depth, a cement-gravel plug driven out to form the bulb, and then casing withdrawn as shaft concreted. Typically 450mm – 600mm diameter shafts on nominal 1m diameter bulbs and less than 10 – 12m depth. The bulbs are below the reinforcing cage and thus there is no reliable uplift capacity except on the shaft unless there is a second bulb driven out through the reinforcing cage above the compression bulb. Piles may have limited fixity at the base affecting lateral capacity.

- Screw piles
  Typically these are 10 m to 20 m long and are screwed into a dense stratum. Capacity

comes from end bearing onto the screw flanges. Uplift capacity comes from "upside down" bearing which may fail if the overlying materials liquefy. There is minimal side resistance along the stem.

- Continuous Flight Auger (CFA) piles.
  This is a relatively new technology in Canterbury so is included here for completeness only, as there are not known to be many in use yet. CFA piles are essentially bored piles installed without casing, so most of the notes relating to bored piles will apply. The maximum length and diameter is limited by available equipment but is in the order of 600mm diameter and 15m length. Using specially adapted equipment, an auger is screwed into the ground and then withdrawn as concrete is pumped down the centre of the flight under pressure, displacing the soil. Once withdrawn, a reinforcing cage is placed into the concrete. This technique is relatively quick, but is technically challenging and requires good QA procedures and experienced operators.

There are several key generic issues for deep foundations that need to be considered:

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 percent of the pile diameter). This can potentially be exacerbated if poor construction has left a zone of disturbed material at the base of the piles.

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

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, resulting in a high risk that the pile capacity will be exceeded at the design load . Strength reduction factors for pile design, including earthquake load cases, should be selected based on a proper risk assessment procedure such as that given in AS2159-2009.

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.

Damage to foundations may not always be evident from the surface, particularly where a large area has been subject to lateral displacements. Where there is evidence of relative motion between the structure and the ground, pile heads and the connection to the structure should be checked for overload in shear. Shear transfer from the ground to the building is typically assumed to be carried by friction underneath the building and by passive resistance of the soil against buried foundation beams and walls etc. The friction mechanism will typically fail quickly with any settlement of the ground and the passive mechanism degrades rapidly with development of gapping. For this reason, and because the earthquake shaking was stronger than design levels, it is likely that the piles may have carried far more shear than the designer ever intended.

Kinematic interactions between the ground and the piles need to be carefully considered. Ground deformations are known to have been significant around many parts of Christchurch, including both dynamic and permanent deformations. These ground deformations may impose significant strains within piles resulting in pile damage and permanent deformation well below the ground surface. Physical investigation of such damage is difficult and expensive and may be impractical. Analytical procedures are available as a first step to try and estimate the pile strain levels and therefore likelihood of damage. Guidance for selecting the appropriate level of investigation is given in the Table 1.

Problem	Fix	Impact
1. Excessive pile settlement	a. May be possible to cut piles and re-level building (However, this will not increase the pile capacity which may be inadequate)	Careful consideration must be given to temporary stability, or the building may be vulnerable to even small earthquakes during implementation
	b. May be possible to use compaction grouting below the pile tips and either lift the piles themselves, or the whole soil block in which the piles are embedded.	Requires access for drilling in grout pipes; probably requires offshore expertise
2. structural damage	In many situations it should be possible to access and repair flexural damage if it is close to the pile caps. Damage here signals the possibility of damage at depth; this would need to be checked; possibly by drilling down the centre of the pile if not under wall or column, or by angle borehole from alongside.	Difficulties in determining whether additional damage at depth exists may mean pile integrity cannot be relied on. An indirect approach is to assess pile damage at depth by analysis of the pile-soil kinematic interactions.

16A PRECAST CONCRETE FLOOR SYSTEMS

Early floors and roofs are usually cast insitu concrete flat slabs, though at this time precast concrete floors with cast-in-place concrete toppings were emerging. By the late 1970s, most floors and roofs in commercial buildings were prestressed precast concrete units with concrete topping.

Floors and roofs must act as large flat elements (diaphragms) that tie the vertical parts of the building together and transfer forces generated by the earthquake or wind across the building to the vertical lateral force resisting structures.

A precast concrete floor system may be a slab, a hollowcore unit, “rib and timber” infill, or single or double tee units. All the variations will have reinforced cast-in-place topping (50 – 70 mm thick, and on occasions, up to 150 mm thick).

Precast concrete floors started in around 1965; these where typically short spans (6 m) and conventional reinforced. From the early 1970s, prestressing of the precast floor units started, permitting longer spans.

Prior to 1995, the minimum seating for precast floors was typically 50 mm. Post-1995, the seatings are specified as a minimum of 75 mm. Observation in the field shows that the seatings were less than these specified minima, in each time period, mainly due to construction tolerances and poor design.

From the mid 1970s through to 1995, for flat units (slab and hollowcore), the provided seating on site ranged between 25 to 50 mm. For stem supported Tees, the seatings ranged between 75 and 150 mm. For rib and timber infill the seating range from 25 to 75 mm.

Each floor type has some common structural performance traits:

- Typically supported on the unreinforced cover concrete. Though reinforced ledges (armoured and unarmoured) have been used to support relatively long and/or heavily loaded floors.

- Lack of alternative load paths (redundancy) should local overload/collapse occur.

  Loss of support through spalling of the units and supports, and pulling off the support by neighbouring beams undergoing plastic elongation.

- Catastrophic failure of the floor when deformations are imposed on the floor (unaccounted for in the design of the floors) by the neighbouring parts of the structure (warping of the floor, rocking walls, prising apart of the units or the topping off the units and significant bending causing tension on the top of the floor).

- Some precast flooring systems rely on unreinforced concrete for shear capacity. Brittle failure of the unreinforced concrete can result if total failure of the floor system

Concrete and steel Moment Resisting Frames are expected to displace laterally at or exceeding the Loading Code limits (those design from mid 1970s onwards). If theses frames form plastic hinges that undergo plastic elongation, this elongation stresses the floor diaphragm frame interface and sections of floor can become unsupported. Sections of floors drop on to the floor below. If one unit falls, it is unlikely to overload the floor below. Should a significant section of floor fall, then it is likely that the lower floor below will fail and fall with the first floor on to the next causing a cascading collapse of all floors below.

The elongation of beams and associated reduction of seating is a function of the lateral drift of the MRFs. Further or compounding causes of loss of support, in all structures, is the distortion of the supports. Each building should be assessed for critical weaknesses and performance features including what was the as-built seating available to support the floors.

Floors and roofs need to act a “diaphragms”. To date, the design of diaphragms has been simplistic and do not cover all the critical behaviour (maintaining load paths, detailing the floor to structure connections and dealing with large penetrations through the diaphragms, for stairs and lifts). Older cast-in-place conventionally reinforced slabs are expected to perform better than the topped precast concrete floors. This is due to the brittle nature of hollowcore and some tee units and the relatively narrow ledges supporting floor units. The reinforcement in the topping, up until 2004, was typically a non-ductile cold-drawn wire mesh. After 2004, the reinforcement was required to be ductile. (Though under very limited circumstances, the non-ductile mesh could be used).

Up until recently many diaphragms were modelled as rigid elements. Actual deformations can be sufficient to increase the demand on gravity resisting structural elements.

Load paths across the floors were not visualised well up until 2000. The additional reinforcement needed along these load paths was not sized or placed correctly or not consider at all. Though improved, this design feature is still being done inadequately in modern structures.

Some diaphragms are required to act as load distribution elements, the performance of which are critical to overall building performance

Problem	Fix	Impact
1. Inadequate support: seating length and unreinforced cover concrete	a. Build an additional ledge (steel angle, typically) or hanger (structural steel cleat or “U” shaped support).	Low to medium intrusive solution. Depends on access to the plenum space below each floor. Lowest cost of the three options here.
	b. Install vertical reinforcement, “hangers”, through the critical areas of the floor. Steel rods, bolts or FRP.	Medium intrusive solution. Medium cost
	c. Install catch frames of steel beams or trusses under the floors.	Highly intrusive solution. Relatively high cost

Problem	Fix	Impact
2. Moment resisting frames – inadequate stiffness of the structure meaning that the building exceeds drift limits, causing loss of support.	Refer to the section on Ductile Concrete Moment Resisting Frames	
3. Inadequate connections of floor and roof diaphragms to the vertical structure.	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that face load support to walls is still provided.
	b. Strengthen diaphragm in areas affected with steel straps, concrete or FRP overlay.	FRP least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise. Steel straps difficult to fix appropriately.
4. Inadequate tension capacity across zones of the floors.	a. provide tension bands or "collectors: FRP, reinforcing rods or flat steel; plate cut in to the floor (epoxied and bolted). Steel members fixed in place under the floors.	FRP - moderately intrusive Rebar or flat plate - moderate to highly intrusive Steel members underneath - very highly intrusive.

17A INSITU CONCRETE FLOOR SYSTEMS

Early floors and roofs are usually cast insitu concrete flat slabs, though at this time precast concrete floors with cast-in-place concrete toppings were emerging. By the late 1970s, most floors and roofs in commercial buildings were prestressed precast concrete units with concrete topping.

Floors and roofs must act as large flat elements (diaphragms) that tie the vertical parts of the building together and transfer forces generated by the earthquake or wind across the building to the vertical lateral force resisting structures.

Floors and roofs need to act a "diaphragms". To date, the design of diaphragms has been simplistic and do not cover all the critical behaviour (maintaining load paths, detailing the floor to structure connections ,dealing with large penetrations through the diaphragms, for stairs and lifts) and deformation compatibility during the post elastic range.. Older cast-in-place conventionally reinforced slabs are expected to perform better than the topped precast concrete floors. . The reinforcement in the insitu concrete slabs was typically mild steel

Load paths across the floors were not visualised well up until 2000. Generally insitu concrete floors have sufficient reinforcement along these load paths.

Problem	Fix	Impact
1. Inadequate connections of floor and roof diaphragms to the vertical structure.	a. Disconnect diaphragm altogether if alternative load paths exist.	Only possible in a limited number of cases. Care needs to be taken to ensure that face load support to walls is still provided.
	b. Strengthen diaphragm in areas affected with steel straps, concrete or FRP overlay.	FRP least intrusive if possible. Concrete overlay thickness makes stairs etc a problem due to height rise. Steel straps difficult to fix appropriately.
2. Inadequate tension capacity across zones of the floors	a. provide tension bands or "collectors: FRP, reinforcing rods or flat steel; plate cut in to the floor (epoxied and bolted). Steel members fixed in place under the floors.	FRP - moderately intrusive Rebar or flat plate - moderate to highly intrusive Steel members underneath - very highly intrusive.

18A PRECAST CLADDING SYSTEMS

Precast cladding became common with the advent of ready-mix concrete, and larger cranes, at which time architects began experimenting with precast concrete as an alternative to cast-in-place or built-up cladding systems. Early examples date from the early 60's.

Although seismic loadings and design techniques became more formalised with the 1965 code, it was not really until 1976 that the considerations of parts and portions seismic loading was more clearly articulated, along with the need to provide adequate clearances to structural members to allow for the deformation of the main building frames. Coupled with this was the understanding of the significant forces that the connection may be subject to.

Another significant issue affecting early precast cladding systems is corrosion. This manifests in two ways – firstly in the lack of cover concrete leading to corrosion of the reinforcement, leading in turn to spalling and cracking of the units. Secondly in corrosion of the connections, many of which are simple drilled-in or cast-in mild steel anchors, in positions that were not as waterproof as may have been anticipated.

Although these systems may not impact on the performance of the structure as a whole, there are in some cases life safety implications from these elements that could or should be addressed. Notwithstanding, failure of the panels will not generally cause failure of the main structure. The only exception would be if the panels engage with the main structure and modify its behaviour enough to cause failure.

For the sake of completeness, some issues and fixes are listed below:

Problem	Fix	Impact
1. Corrosion or reinforcing or metal embedded items have weakened panels to the extent that large pieces are able to fall in event of earthquake.	a. Break out and repair affected areas of panels	Expensive and difficult, as extent of damage is difficult to determine.
	b. Remove panels and reclad building	Very expensive solution and very intrusive as will involve linings also.
2. Connections are weak and/or corroded.	a. Replace connections.	May be difficult if connections are inaccessible, and/or expensive if it requires removal of linings.
	b. Remove panels and reclad building	Very expensive solution and very intrusive as will involve linings also.
3. Panels have inadequate clearance to structure	a. Cut back or replace panels to ensure no impact can occur	Very expensive and/or intrusive as likely to impact internal linings.

19A HEAVY MASONRY OR PLASTER CLADDING SYSTEMS

These systems were in general use from the development of multi-storey buildings (other than UMBs) to around the 60's when they were gradually phased out in favour of precast and curtain wall systems (although the latter technology had been available and in sporadic use for some time).

These systems were not generally subject to specific seismic design, and have a number of potential issues, including:

- Lack of clearance to the main structure, causing modification of the main structure behaviour and/or significant failure of the cladding itself.

- Lack of connection of the cladding to the main structure.

- Inadequate out-of-plane capacity of the cladding system.

Although these systems may not impact on the performance of the structure as a whole, there are in some cases life safety implications from these elements that could or should be addressed. If the panels engage with the main structure and modify its behaviour enough they may cause failure of the main structure.

For the sake of completeness, some issues and fixes are listed below:

Problem	Fix	Impact
1. Lack of capacity of cladding systems in face loading.	a. Add supplementary structural support such as steel or reinforced concrete mullions	Often quite intrusive and may require removal and reinstatement of internal linings.
	b. Remove panels and reclad building	Very expensive solution and very intrusive as will involve linings also.
2. Connections are weak and/or corroded.	a. Replace connections.	May be difficult if connections are inaccessible, and/or expensive if it requires removal of linings.
	b. Remove panels and reclad building	Very expensive solution and very intrusive as will involve linings also.
3. Panels have inadequate clearance to structure	a. Cut back or replace panels to ensure no impact can occur	Very expensive and/or intrusive as likely to impact internal linings.

APPENDIX B

CHRISTCHURCH CITY COUNCIL COMPLIANCE SCHEDULE

1. Automatic systems for fire suppression (for example, sprinkler systems)	☐
2. Automatic or manual emergency warning systems for fire or other dangers (other than a warning system for fire that is entirely within a household unit and serves only that unit).	☐
3. Electromagnetic or automatic doors or windows (for example, ones that close on fire alarm activation)	
3.1 Automatic Doors	☐
3.2 Access controlled doors	☐
3.3 Interfaced fire or smoke doors or windows	☐
4. Emergency lighting systems	☐
5. Escape route pressurisation systems	☐
6. Riser mains for fire service use	☐
7. Automatic back-flow preventers connected to a potable water supply	☐
8. Lifts, escalators, travelators, or other systems for moving people or goods within buildings	
8.1 Passenger-carrying lifts	☐
8.2 Service lifts including dumb waiters	☐
8.3 Escalators and moving walks	☐
9. Mechanical ventilation or air conditioning systems	☐
9a. Cooling tower as part of an air conditioning system	☐
9b. Cooling tower as part of a processing plant [not a specified system]	☐
10. Building maintenance units for providing access to the exterior and interior walls of buildings	☐
11. Laboratory fume cupboards	☐
12. Audio loops or other assistive listening systems	☐
13. Smoke control systems	
13.1 Mechanical smoke control	☐
13.2Natural smoke control	☐
13.3Smoke curtains	☐
14. Emergency power systems for, or signs relating to, a system or feature specified in any of the clauses 1 to 13	☐
14.1Emergency power systems	☐
14.2Signs	☐
15. Other fire safety systems or features	☐
15.1Systems for communicating spoken information intended to facilitate evacuation	☐
15.2Final exit (as defined by A2 of the Building Code; and	☐
15.3Fire separations	☐
15.4Signs for communicating information intended to facilitate evacuation	☐
15.5Smoke separations	☐
16. Cable Car (including to individual dwellings)	☐