# DETAILED ENGINEERING EVALUATION PROCEDURE – DEVELOPMENT DRAFT

## DISCUSSION DOCUMENT FOR CONSIDERATION BY CSG

## 1 INTRODUCTION

The post-disaster building safety evaluation process endorsed by DBH involves three levels of assessment, as follows:

- Initial
- Rapid (Level 1 and Level 2)
- Detailed

The first two have a clearly defined process[1] 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 Evaluations 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 make it safe. It simply identifies that no significant damage has been identified, that is, it is not known to be unsafe. 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.

Public attention has been drawn to this repeatedly, but it is clear that this has not always been recognized, despite the building collapses that have happened in previously green placarded buildings. It is not yet known in those cases whether a detailed evaluation was completed.

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. There are now many damaged buildings of all forms, raising the question of how we assess their residual capacity. 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 following such an event 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.

## 2 OBJECTIVES

The overarching primary objective of the Detailed Engineering Evaluation procedure is to ensure confidence in our remaining building stock in order to assist the recovery from the Canterbury Earthquake. The measure of success will be 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 the process is applied to all buildings that could have suffered damage
- Auditable – by requiring a consistent quality of information to be lodged with the BCA's
- Able to be understood by lay people – by describing a process that is transparent and well communicated

Secondary objectives include:

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

## 3 SCOPE

It is strongly recommend that affected BCA's request DEE's 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. 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 BCA's in the Canterbury area – Christchurch, Waimakiriri and Selwyn, but in practice this must be confirmed by CERA (under S51).

Suggestions have been made to exclude smaller buildings, for example buildings of three storeys and below. However it is noted that many of the buildings that collapsed into the street during the February 22 earthquake, causing death and injury, were one or two storeys only.

In addition to the structural engineering aspects of the buildings, there are a number of non-structural matters that should be checked prior to occupation, most of which are subject to standard compliance schedule review. These additional inspections will not require structural engineering review, but for the sake of completion, should be completed and submitted at the same time as the structural report.

# 4 THE PROCEDURE

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 should be reserved for those buildings that are the most complex and which generally may be yellow placarded.

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 Basic Safety Evaluation (BSE) 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 1 below. Both the qualitative and the quantitative procedures are described separately below.

Following the qualitative assessment procedure, those buildings requiring no further action 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.

Following the lodging of the Detailed Engineering Evaluation Reports 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.

## 4.1 Qualitative Assessment Procedure

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 strength will be made in terms of %NBS (New Building Standard), either in accordance with the NZSEE Initial Evaluation Procedure, or by a simple comparison with current code according to the original design.

It is considered that the Qualitative procedure will be as follows:

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

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.

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 appropriate for the loads 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 buidlngs 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. 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 1 below. Note this list is given for guidance and is not necessarily comprehensive.

6. A thorough investigation of possible collapse hazards or critical structural weaknesses (CSW) should be made. Note that it is not adequate to assume that a detail formed from a ductile material will behave in an acceptable fashion. For example:

   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.

7. An assessment must be made of both the original and the residual strength of the building, taking into account the damage it has suffered. This may be achieved in a number of ways:

   a. An IEP may be performed, in accordance with the NZSEE procedures[3] (noting that the IEP is not designed to accommodate post-76 buildings, also that the IEP may be unduly conservative).

   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 strength could be expressed as:

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

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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 strength of damaged elements

8. The collapse hazard or critical structural weaknesses must be addressed. This may be either by reduction of the %NBS assessment to allow for the margin required for collapse over ULS strength, or by simply mitigating the weakness.[i]

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 required**. Although strengthening is also recommended for any building with %NBS<67%.

3. For buildings with significant damage, a quantitative assessment is required.

The qualitative assessment process is presented graphically in Figure 2 below.

On completion of the qualitative assessment, the engineer should have a comprehensive understanding of the building's performance, and the reasons why it has behaved as it has. 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

Only when the qualitative assessment has been completed should a quantitative assessment be considered. The extent of quantitative assessment will have been informed by the outputs of the qualitative 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



[i] The former approach could be achieved by dividing the strength of the CSW by 1.8, ie the generally accepted margin of MCE over DBE. This would achieve the normal relativity of Collapse Limit State to Ultimate Limit State. Therefore the %NBS would be the lesser of the calculated %NBS at ULS of the main structure, or %NBS/1.8 of the Collapse mechanism.
Another possibility would be to adopt a determininstic approach, ie check the collapse hazard %NBS against a load level determined from an investigation of the likely seismic events under which collapse may be unacceptable.

or CSW's 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]), but will require modification in order to accommodate observed damage.

It is recognised that earthquake damage to existing building elements may reduce strength 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.

The quantitative procedure is intended initially to assess the residual strength of the building in its damaged state, and then to assess the efficacy of proposed repairs. Analysis may be generally in accordance with NZS1170.5[8] and the NZSEE guidelines[3]. 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 be an assessment of the %NBS of the building in its damaged state, leading to an assessment of the required repairs. It is assumed that the repairs will comply with the appropriate EPB policy of the local BCA.

### 4.3 Reporting

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. 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.

5. A full summary of damage sustained (plans and elevations if necessary), both structural and non-structural damage as it relates to building movement.

6. 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.

7. A full 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.

8. Some form of reference to any generic building/material/configuration issues that are known to occur, with verification of whether these have/have not occurred.

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

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

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

12. A clear 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.

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

14. A completed table of Compliance Schedule items (refer Table 3 below)

All of the above would of course 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.

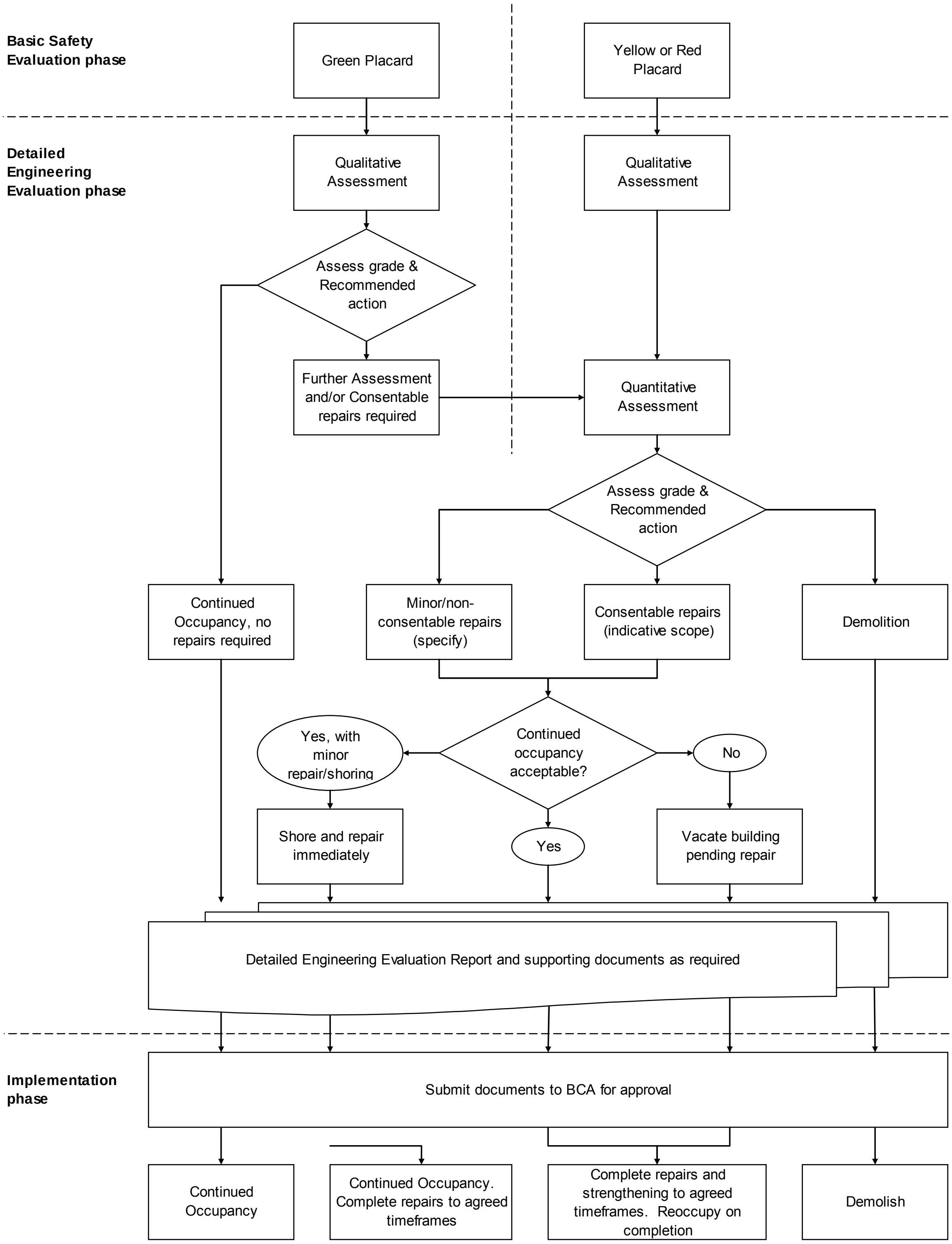


Figure 1: Detailed Engineering Evaluation - Overall Procedure Outline

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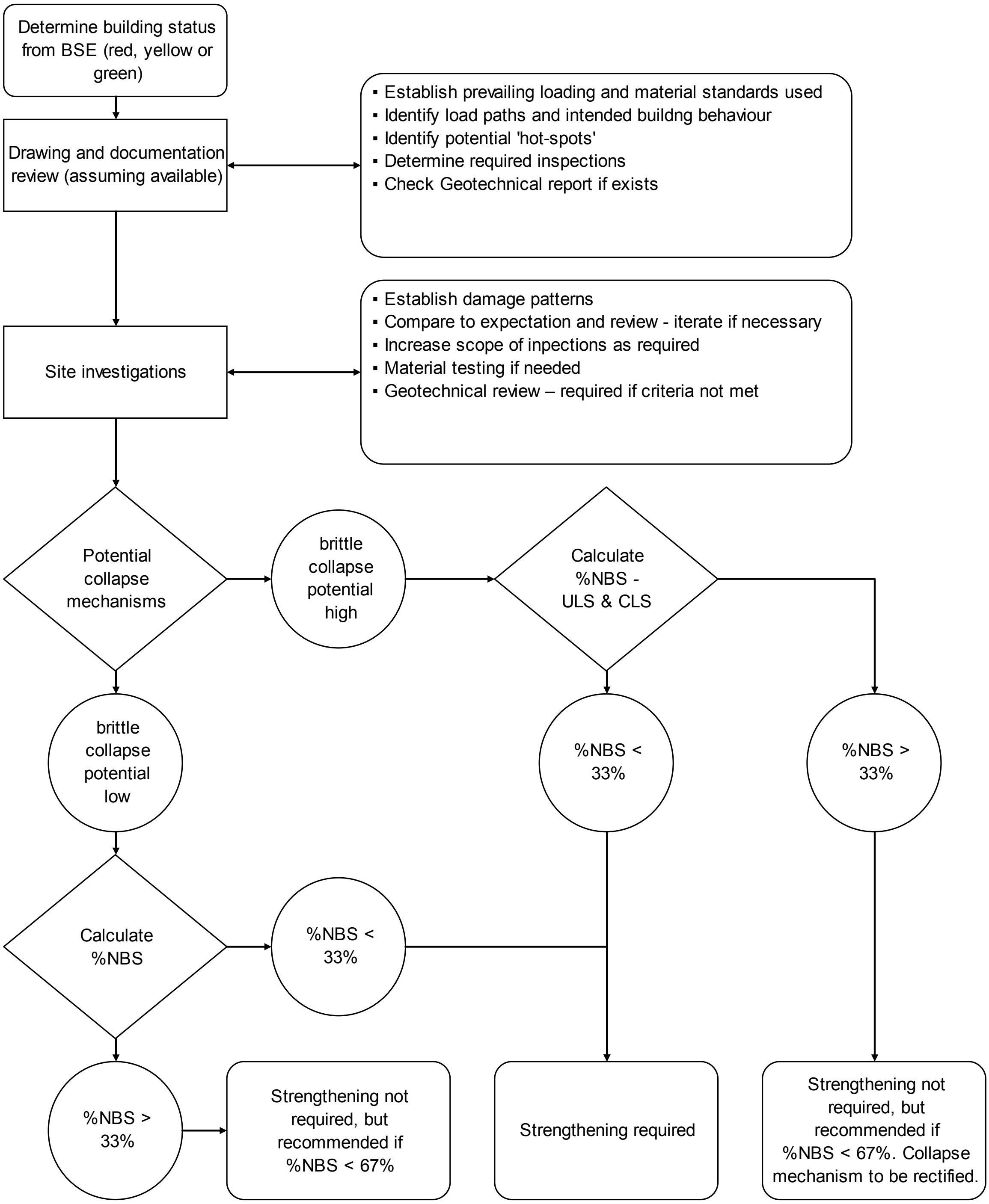


Figure 2: Qualitative Assessment Procedure

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Table 1: Schedule of Recommended Inspections

| Area | Element | Notes |
|---|---|---|
| Foundations | Ground conditions | • Verify whether liquefaction has occurred at or near the site[ii]<br>• Verify whether lateral spread has occurred at or near the site<br>• Check whether geotechnical information is available for the site<br>• Look for signs of obvious settlement |
| | Foundations | • Investigate possible movement, lateral and vertical<br>• If piled and lateral movement is observed, expose a pile or piles in order |
| Exterior | Roof | • Check for movement at flashings<br>• Check parapets and other roof level appendages<br>• 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?<br>• Beams – investigate potential plastic hinges and beam elongation<br>• Beam-column joints – crack patterns<br>• Possible fracture in steel frame joints |
| | Shear walls | • Crack patterns |

[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 fo review may be applicable.

| Area | Element | Notes |
| --- | --- | --- |
| | | • Possible base hinging or shear failure? |
| | Bracing systems | • Extension in braces<br>• Shear or flexural yielding in links of EBF's<br>• Lateral buckling of brace elements<br>• Yielding or damage to connections |
| | Diaphragms | • Transfer or inertial?<br>• Floor type?<br>• Precast floors – investigate seatings (above and below), crack patterns in topping, review ties at perimeter, saddle bars, topping reinforcement integrity |
| Secondary structure | Stairs | • Review seating and connections<br>• Review intermediate landings – compression or tension failure |
| | Cladding | • Check whether cladding may have modified structural behaviour<br>• Identify areas where structural interference has occurred due to drift<br>• Investigate connections |
| | Ceilings | • Review fixing of grid (if applicable)<br>• Fixing/support of lights, a/c grilles etc.<br>• Damage to/at sprinkler systems |
| | Building services | • All plant items connected and restrained suitably |
| Non-structural elements | Compliance Schedule items | • Refer Table 2 over. |
| | Electrical | • Electrician to inspect wiring. |

Table 2: Soil Damage Assessment Criteria

| Parameter | Desk study | Geotechnical investigations and Testing of footings | Limited Exposure of Critical Ground connections | Full exposure of Typical Foundation elements | Intrusive Investigations of footings/piles |
|---|---|---|---|---|---|
| Settlement (mm) | >25 | >50 | >100 | >200 | >400 |
| Differential Settlements | >1:750 | >1:500 | >1:250 | >1:150 | >1:100 |
| Liquefaction ($m^3/100m^2$) | >1 | >2 | >5 | >10 | >20 |
| Lateral Spreading (mm) | >50 | >100 | >250 | >500 | >1000 |
| Damage to superstructure | Cosmetic | Minor Structural | Significant structural | Severe structural | Major structural |
| Damage in Area | Slight | Moderate (1in 10) | Substantial (1in 5) | Widespread (1in 3) | Major (Most) |
| Function (Occupancy) | 3 | 5 | 10 | 20 | >20 |

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Table 3: Complianance Schedule Items

| | |
|---|---|
| 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) | ☐ |

# 5 REFERENCES

[1] New Zealand Society for Earthquake Engineering *Building Safety Evaluation*. August 2009
[2] Canterbury Earthquake Recovery Bill, 2011
[3] New Zealand Society for Earthquake Engineering *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, June 2006
[4] New Zealand Society for Earthquake Engineering *Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance*, Draft 2011
[5] Federal Emergency Management Agency, FEMA 306 *Evaluation of Earthquake Damaged Concrete and Masonry Buildings – Basic Procedures Manual*, 1998
[6] Consortium of Universities for Research in Earthquake Engineering (CUREE), EDA-02 *General Guidelines For The Assessment And Repair Of Earthquake Damage In Residential Woodframe Buildings,* February 2010
[7] Steel Advisory Council, *SAC95-02 Interim Guidelines (FEMA 267B)*, 1995
[8] Standards New Zealand *NZS1170.5:2004 Structural Design Actions Part 5: Earthquake Actions, New Zealand,* SANZ

# 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.

# 1 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: NZS 3101: 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 NZS 3101:1995, 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 ealier 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;sdetailing 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 lowr 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.<br><br>If an exterior column, a very intrusive solution.<br><br>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. |

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| 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. |
| 5. Torsional behaviour through secondary structures (walls, stairs or ramps) which are incompatible with | 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. |

| Problem | Fix | Impact |
|---|---|---|
| displacements of the moment resisting frame structures. | 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.<br><br>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.<br><br>Likely to be very intrusive |
| | b. Selective weakening of elements to reduce impact of irregularity | Not always able to achieve desired effect. |

# 2 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 1965, these buildings were subject to increased seismic loads which are closer to current standards, particularly for the taller more flexible frames.

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.<br><br>Loss of lateral strength of the building, new structures need to be added to compensate<br><br>Can introduce weatherproofing issues in boundary walls. |
| 2. nadequate 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. |
| 5. Infills falling out of the | a. Strengthen the connections of the infill | a. Moderately intrusive |

| Problem | Fix | Impact |
|---|---|---|
| frames. | panels to the frame.<br><br>b. Provide supplemental support to the infill panel (cast-in-place concrete or shotcrete or steel frames) | <br><br>b. Very intrusive. |
| 6. Structural irregularity or discontinuity | a Introduce strengthening in areas of high demand | Very extensive work will be required.<br><br>Likely to be very intrusive |
| | b Rationalise structural system | |

# 3 CONCRETE SHEARWALL STRUCTURES

Concrete structural walls, “shearwalls”, 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 fleural 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 reinformcement focused at relatively few wide cracks (as opposed ot 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 splilces, 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 expoxying in the reinforcement or strips. | Moderately intrusive |
| | d. Selective weakening, by cutting some or all of the vertical bars in the wall. | Moderately intrusive.<br><br>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.<br><br>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.<br><br>Concrete overlay thickness makes stairs etc a problem due to height rise of the floor<br><br>Steel straps difficult to fix appropriately. |
| | | |
| | | |
| Structural irregularity or discontinuity | Introduce strengthening in areas of high demand | Very extensive work will be required.<br><br>Likely to be very intrusive |

# 4 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. n addition to the panel strength, many of the roof diaphragms are inadequate, particularly early tension bracing systems and there connections. .

| Problem | Fix | Impact |
|---|---|---|
| 1. Brittle panel connections and/or cracked panels at | a. Retrofit supplementary ductile connections. | Minimal, provided connections are accessible |

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| Problem | Fix | Impact |
|---|---|---|
| the connection. | Epoxy cracks where weatherproofing compromised. | (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 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 |
| Structural irregularity or discontinuity | Introduce strengthening in areas of high demand | Very extensive work will be required.<br><br>Likely to be very intrusive |

# 6 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 discontiuity

- Inadequate quality control during construction lead to 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. Expoxying the strips to the wall. | Moderately intrusive. |
| | d. Selective weakening, by cutting some or all of the vertical bars in the wall. | Moderately intrusive.<br><br>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.<br><br>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.<br><br>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.<br><br>Concrete overlay thickness makes stairs etc a problem due to height rise.<br><br>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.<br><br>Likely to be very intrusive |

# 7 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 lead to 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. Expoxying the strips to the wall. | Moderately intrusive. |
| | d. Selective weakening, by cutting some or all of the vertical bars in the wall. | Moderately intrusive.<br><br>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.<br><br>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.<br><br>Likely to be very intrusive |

# 8 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 boiler makers 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<br>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.<br>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.<br><br>Likely to be very intrusive |

# 9 STEEL BRACED FRAMES

To come.

| Problem | Fix | Impact |
|---|---|---|
| | | |
| | | |
| | | |

# 10 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 EPB's, 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<br><br>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.<br><br>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. |

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| 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.<br><br>Likely to be very intrusive |

# 11 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 fail 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 frames. | a. Strengthen the connections of the infill | a. Moderately intrusive |

| Problem | Fix | Impact |
|---|---|---|
| | panels to the frame.<br>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.<br><br>Likely to be very intrusive |
| 6. Inadequate seismic separation | a. Increase width of seimic separation | Very extensive work will be required.<br><br>Likely to be very intrusive |

# 12 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 EPB's, 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.<br><br>Likely to be very intrusive |

# 13 SHALLOW FOUNDATIONS

To come.

| Problem | Fix | Impact |
|---|---|---|
| | | |
| | | |
| | | |

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# 14 DEEP FOUNDATIONS

To come.

| Problem | Fix | Impact |
|---|---|---|
| | | |
| | | |
| | | |

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# 15 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.<br><br>Medium cost |
| | c. Install catch frames of steel beams or trusses under the floors. | Highly intrusive solution.<br>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<br>Rebar or flat plate - moderate to highly intrusive<br>Steel members underneath - very highly intrusive. |

# 16 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 insit concrete slabe 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<br>Rebar or flat plate - moderate to highly intrusive<br>Steel members underneath - very highly intrusive. |

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# 17 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. |

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# 18 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. |

Steel braced systems
We should include buildings, often only industrial, that rely on tension bracing

In the inspections we made of industrial buildins one of the noticeable features was the low capacity or absence of steel cross bracing in the longitudinal direction of portal framed structures

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