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

Part 3
Technical Guidance

Section 8 – Reinforced Concrete Moment Resisting Frames

Draft Prepared by the Engineering Advisory Group
Read in Conjunction with Sections 1 to 3 (separate document)

Revision 4, April 24th, 2013

Document Status

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

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

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

Part 1	Background
Part 2	Evaluation Procedure
Part 3	Technical Guidance

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

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

Revision history:

Revision 1, 11 June 2012, internal to EAG
Revision 2, internal to EAG
Revision 3, EAG internal draft for comment
Revision 4, 24th April 2013, first draft for comment

Contents

TABLES

FIGURES

8 REINFORCED CONCRETE MOMENT FRAME BUILDINGS

8.1 INTRODUCTION

Concrete moment frame (CMRF) structures are commonly used throughout Christchurch, dating back to the early 1900's. The performance of CMRFs in the Canterbury earthquakes has generally been acceptable, in life safety terms. However, many of these structures have suffered damage that may not be repairable.

There are a number of concerns to be dealt with, including:

- the significance of frame damage.
- assessment of remaining life, acknowledging that low cycle fatigue may have reduced the available inelastic strain capacity of the reinforcement;
- the effectiveness of repairs.

In general, CMRFs have behaved as could be expected. More recent capacity designed CMRFs have formed mechanisms as expected, and there has been little or no deterioration noted in protected elements. Older non-ductile buildings have not performed as well, but have been within expectations for that type of structure.

Notwithstanding this, there have been some concerns expressed at the number of buildings being demolished, and the apparent lack of repairability of CMRFs, particularly the more recent ductile structures. Although not conclusive, testing of yielded reinforcement has indicated that significant strain hardening has occurred in many cases, without the development of further primary cracking within the concrete of the element. This indicates that low-cycle fatigue failure is a significant concern in consideration of buildings' remaining life.

The main reference document for assessment of CMRF structures is the NZSEE Guidelines[1]. This offers guidance on the assessment of existing CMRFs and possible strengthening solutions, but it does not address damage. The information presented in this section provides guidance to sit alongside the NZSEE Guidelines.

The EAG notes that an owner's insurance policy may entitle the owner to a level of reinstatement that is not practically achievable without full or partial replacement of the building structure. This document specifically addresses the technical evaluation of existing structures without regard to the wording of the insurance. It is generally recommended that a comprehensive assessment of damaged buildings should include a combination of testing and analysis in order to verify the buildings remaining capacity in order to make sure that the building behaviour is fully understood and that the implications of damage on future performance can be assessed.

[1] New Zealand Society for Earthquake Engineering *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, June 2006

8.2 NOTATION

Refer to Section 9 Reinforced Concrete Wall Buildings for notation.

8.3 DESCRIPTION

CMRFs can be broadly categorised as either ductile or non-ductile.

Non-ductile CMRFs are buildings that lack the modern detailing and design practices that account for seismic attack. Non-ductile CMRFs are relatively common throughout New Zealand main metropolitan centres.

New Zealand-wide non-ductile CMRFs 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 better expected performance.

From 1935 to 1965 non-ductile CMRFs were designed to a lateral force of 8-10% of gravity, applied uniformly at each level. From 1965, the seismic loading was amended to include reference to the building's period and for different seismic zones. In the longer period range, loads were in fact reduced. Refer to the NZSEE Bulletin Vol 42, 2009[2].

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.

Floors and roof are usually cast insitu concrete flat slabs.

Ductile CMRFs are buildings that have some to full modern detailing and are designed with practices that account for seismic attack. Largely restricted to the CBDs, ductile CMRFs were constructed from about 1975 to the present.

Research at the University of Canterbury (and Auckland, to a lesser extent) led to the development of capacity design techniques that were eventually codified. This work commenced in the 60's and these institutions continue to contribute to the international development of reinforced concrete design.

In terms of New Zealand Standards for Concrete Structures, NZS3101, significant developments include:

- 1975, a provisional Standard was issued, introducing capacity design and ductile detailing.

[2] New Zealand Society for Earthquake Engineering Bulletin Vol 42 2009, Fenwick and McRae *Comparison of New Zealand standards used for seismic design of concrete buildings.*

- 1982, the first official version was issued, largely continuing the provisional standard
- 1995, there were significant improvements in detailing for robustness of structures;
- 2006, further improvements were made.

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 1965 onwards.

The lateral load resisting mechanism is typically perimeter frame action, in either one or both principal directions. However, many of the earlier ductile CMRFs include full two-way frame action, using both exterior and interior frame lines.

Initially, construction was mainly cast insitu concrete for both frames and floors. In these structures, the floor diaphragms tended to be well integrated, with direct connections to the frame element, both columns and beams. However, use of precast was increasingly common, with the introduction of prestressed precast floor systems from the mid-70s allowing greater spans. Use of precast spread to the primary structure, particularly in the 80's with variations from insitu columns and precast beams with mid-span splices to integrated beam-column units with grouted ducted splices in the columns and a variety of different methods for the beams. At the same time, floor seatings for precast elements were minimal (by today's standards) and there was often minimal connection from the frame elements to the floors.

The form of the structure has in many cases also been influenced by the analysis methods adopted. In the earlier years of capacity design, hand methods of analysis may have been used, with static load distribution methods. The availability of computer systems allowed dynamic analysis to be adopted. In early cases in Christchurch, the University of Canterbury computer system was used and from the mid-80's the advent of PCs allowed offices to do their own analysis.

Early computer analysis typically modelled only the lateral load resisting elements of the structure, due to hardware limitations which have gradually been overcome. In some cases, this may have resulted in greater capacity than would now be provided, as the secondary structure would always add some resistance. However, until 1995, the secondary structure may not have had the ductility to deal with the implied drifts.

The seismic performance of ductile CMRFs 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, generally for buildings three storeys or less, ductile column sidesway mechanisms, may have been acceptable. These structures were categorised as limited ductility in the 1982 Concrete Standard[3], and later as nominally

[3] Standards New Zealand *NZS3101:1982 Concrete Structures,* SANZ

ductile in the 1995 Concrete Standard[4]. Such buildings have been required to be designed to lower ductility levels.

From the advent of CMRFs in the early 20th century, floors and roofs were usually cast insitu concrete flat slabs. However from the 1960's, precast concrete floor systems 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.

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 diagonal props between floors (as do ramps). These elements could bind in the event of earthquake, stiffening the building response and potentially causing torsional movements where the stairs or ramps are off-centre.

This may be exacerbated in buildings designed prior to 1995, when the requirements for allowance from cracking in concrete members were changed, resulting in much greater drift allowances being needed.

8.4 SEISMIC RESPONSE CHARACTERISTICS AND COMMON DEFICIENCIES

The behaviour of CMRFs is very dependent on the detailing and configuration of the frames. Some of the response characteristics are dependent on the level of ductility designed for, and some are common to all frames, whether ductile or not. This section is split accordingly.

8.4.1 Common Characteristics

Concrete. Older reinforced concrete (prior to the 1960s) was typically batched on site. Site batched concrete is typically more variable in strength and is frequently considerably stronger than the nominated f'_c. Site batched concrete may often be detected by the irregular location of construction joints, often appearing as 'cold' joints, where the poured stopped as the concrete ran out. These joints are frequently vulnerable to cracking, which is considerably more apparent in these locations than at floor level, where joints are typically located in more modern construction..

Since the 60s, ready-mix concrete has become prevalent. Because ready-mix concrete is batched off-site, there were less limitations on concrete supply volumes, meaning that locations of construction joints are more consistent, although cracking at floor level may be harder to see and measure after an earthquake. During the 60s and 70s, the cement content and covers were reduced in many cases, reflecting the (assumed) better quality control

---

[4] Standards New Zealand *NZS3101:1995 Concrete Structures Standard – The Design of Concrete Structures,* SANZ

standards. In reality, this has proved to be overly optimistic and many buildings from this era now suffer from corrosion of the reinforcement, which should be considered when assessments are being completed.

Guidance on default assumed concrete strengths is given in the NZSEE Guidelines[1]. However testing, in sufficient quantity to achieve statistical admissibility, should be employed in cases where there is doubt, or where concrete strength may be critical.

Reinforcement. The earliest reinforcement was often plain square or round bars. Deformations were gradually introduced, with variations including twisted squares, jagged or barbed squares, and various round bar deformations, typically from the rolling process. However, plain round bars were used well into the mid-60s.

Plain bars need to considered carefully, regardless of whether the structure is considered ductile or not. Unless well anchored with a 90° or 180° hook, these bars are rarely capable of developing yield. This is not necessarily a major problem, as it may increase displacement capacity and reduce shear demand in some critical elements. In cases where laps coincide with peak moments, sensitivity studies on the performance of the member should be made using zero tensile capacity in the reinforcement as well as the assessed yield capacity. This may not mean that the member capacity reduces to zero if for example there is significant axial load on the element.

Plain bar bond stress considerations may vary with location:

- In columns, the impact of a bond loss may be for the columns to rock at the base (assuming a base lap). This may have the effect of increasing demand in the beams or slab, as lateral displacements and hence element rotations will also increase. The integrity of the base of the column may need to be considered, as may the shear transfer at the base, which will be dependent on dowell action and axial load.
- In beams, the behaviour of the system may change completely, depending also on the column detailing. But excess rotations in the beam may lead to loss of shear capacity, depending on the location, as well as loss of flexural capacity. Note however that beams are more likely to have 180 hooks as anchorages.
- In joints, bar slip will lead to softening of the joint, increased lateral displacement and reduced capacity of the system .

The yield stress of older bars may generally be taken as 300MPa, but higher values may have been used in some cases. Since the 60s, yield strengths have been standardised, with the Grade 275 gradually giving way to Grade 300 (essentially the same steel).

Higher grade steel was also made available. Grade 380 was used until the late 80s. This steel was not readily weldable, and had a shorter yield plateau and higher overstrength. It was generally not used in ductile elements. Grade 380 gave way to Grade 430, which was both more ductile and weldable. This in turn was phased out in the early 2000s and replaced with Grade 500.

Further guidance on the yield strength of various grades of older reinforcement is given in the NZSEE Guidelines.

Configuration. Whether a building is ductile or non-ductile, overall configuration plays a significant part in the behaviour of the building. In general, regular buildings can be expected to perform better than irregular buildings, unless the irregularity has been specifically and carefully addressed.

Beam (or column) elongation. Whether a yielding member has been designed for ductility or not, the development of a plastic hinge must result in elongation of the member. The impact of elongation should be considered, particularly on precast floor elements, which may lose seating under excessive beam elongation. In the case of multiple bay frames, the elongation aggregates. The impact of this on columns should be considered, particularly at the level immediately above the base level. At this level, the aggregate elongation may cause column hinging in the end members, or a super-compression of the beam caused by the columns may significantly increase the beams' flexural capacity, causing possible shear failure or a full column mechanism.

Cladding and cladding connections. Rigid cladding systems (such as precast panels, should have connections that make allowance for the effects of interstorey drift. Prior to NZS1170.5, these drifts were typically under-estimated. This may affect the building behaviour in a number of ways, including:

- If the elements are embedded within the frame, they may cause locking up of the structure. In the event of an irregular distribution, this may force a torsional response for the structure as a whole, even though the frame may be regular.
- If elements are within the frame, but not fully continuous between floors, they could potentially cause a short-column condition in the main frame columns.
- If the connections are brittle (for example solely reliant on shallow embedded anchors), these may break out of the frame or cladding element, resulting in a significant falling hazard.

Conversely, if there is some ductility and/or redundancy in the connections, the elements may survive an event without falling, even if severely damaged.

Diaphragms. Floor diaphragms are a critical element of any lateral load resisting mechanism. In the past, diaphragms have been termed type I and type II, signifying respectively, diaphragms that carry only inertial effects, and transfer diaphragms, which act to redistribute seismic actions between separate vertical lateral force resisting mechanisms within the structure. In practice, pure inertial diaphragms are rare, only occurring in the most structurally regular of buildings. In other words, virtually all real diaphragms develop both inertia and transfer components of forces; transfer forces in diaphragms are not just limited to ground floors above basements or roof levels of podiums beneath taller towers.

Most CMRF structures have been designed assuming rigid diaphragms. This may be generally conservative, but flexibility should be considered where there are significant openings and/or diaphragm segments of aspect ratio greater than 6:1. Irregular shapes such as L-shaped floors or significant atrium openings may also be factors.

Diaphragm flexibility may be specifically modelled in most 3D analysis software such as ETABS, noting that it is also important to consider the effects of deformation, particularly when orthogonally opposed systems have markedly different response.

Where diaphragms are very flexible (more typically in the case of timber floor diaphragms), a tributary width assumption is more appropriate, often resulting in very different distribution of load between elements, compared to a rigid diaphragm assumption. Displacement compatibility should be reviewed by considering the implications of the individual support lines deflections, with respect to the diaphragm displacement capacity.

If the means to model the diaphragm flexibility are unavailable, or if the criticality of flexibility is uncertain, the problem can be bounded by considering both fully flexible and rigid options and investigating the impact on the remaining structure.

Along with the assumption of rigidity, ductility of diaphragms has seldom been considered. Until relatively recently, non-ductile welded wire mesh has been used in thin toppings for precast floors. The resulting diaphragms are lacking in ductility and have relatively low strength. The mesh may fracture in cracks as small as 2mm or less, often occurring at pre-existing cracks, typically caused by shrinkage of the floor topping during construction. These cracks may be even more pronounced where the effects of shrinkage over large floor plates are restrained by widely distributed primary structural elements.

A further consideration in slabs (particularly in toppings reinforced with non-ductile mesh), is whether the load paths to critical lateral load resisting elements are complete. CMRFs are usually not the most critical cases, but where frames are offset or at one end of a building, diaphagms should be checked for adequate collector elements. This is particularly important where there is significant reliance on non-ductile mesh.

Little guidance is available for either the assessment of design actions or the design of diaphragms and collectors. One possible source of information is a US document published by the National Institute of Standards and Technology (NIST), *Seismic Design of Cast-in-place Concrete Diaphragms, Chords and*

Collectors[5] . Care must be taken to account for the effects of using thin toppings when using this document. Also to note that this document primarily addresses inertial forces, so designers should make additional allowance for transfer forces where they exist.

Displacement compatibility. Until as late as 1995, lateral load resisting and gravity resisting structure were considered separately. Although the Concrete Standard as early as 1982 required some consideration of displacement compatibility, this was often neglected and/or complied at the much lower displacements that were calculated until the advent of NZS1170. In reality, all structure must go through the same lateral displacement and so the displacement ductility demand must be considered for all members regardless of designation.

Foundations. Local overstressing of foundations may occur, in part due to detailing deficiencies of the foundations, but also from not ensuring that a desirable plastic mechanism achieved in the superstructure. Piles, whether precast or insitu, may lack sufficient anchorage to adequately resist uplift actions generated by higher than expected actions.

The following matters may require consideration (noting that this list is not necessarily comprehensive, i.e. assessors may need to consider other matters not addressed here):

- Differential settlement. In some cases, the deformation of a frame may have been induced by foundation movement rather than shaking damage, particularly at sites affected by lateral spread. In these cases, the deformation may have been primarily a monotonic, rather than cyclic, as the ground deformation happens relatively slowly. This may be considered when evaluating the impact on the reinforcement.
- Lack of tie beams. Early reinforced concrete structures have tended to have foundation tie elements between columns at the base floor level, but this practice was not always adhered to, particularly in the 1980's. Where there are no tie beams, the effect of differential column movement at the base level should be considered.
- Piles may have yielded or failed in shear. Refer to Part 3 Section 5[6] for further guidance.
- Foundation flexibility may or may not have been considered in the design of the building. The significance of this should be considered in assessing the building, noting that although increased foundation flexibility may reduce demand, the effects of the additional implied deformation must be considered.

[5] Moehle, Jack P., Hooper, John D., Kelly, Dominic J., and Meyer, Thomas R. (2010). "*Seismic design of cast-in-place concrete diaphragms, chords, and collectors: a guide for practicing engineers*," NEHRP Seismic Design Technical Brief No. 3, National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 10-917-4.

[6] *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury Part 3 Technical Guidance, Section 5 - Foundations*, Draft prepared by the Engineering Advisory Group, Rev 3, 16 May 2012

Non-structural interference. Until the advent of NZS1170.5, calculated displacements of structures were underestimated. Elements such as stairs, ramps, lifts, cladding and ceilings may all lack the clearance required to prevent interference with the primary structure.

Many earlier versions of these stairs and ramps have sliding details where the stair or ramp 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 or ramps are prone to collapse due to jamming between floors during lateral displacement of the floors.

Weaker elements such as ceilings may not have significant impact on the overall structural performance, but elements such as precast panels or masonry infills may have a significant effect, depending on the deformation at which contact occurs, and the strength of the element or the connections of the element. Guidance is given on the analysis of frames with infills in the NZSEE Guidelines.

Pounding. As with non-structural interference, the under-estimation of seismic drifts (at least prior to NZS1170.5) may have resulted in inadequate boundary clearances or internal seismic joint allowances. This should be considered in evaluating structural damage and in considering what extent of retrofit may be required.

In general, where floors align reasonably well between adjacent structures, there may be increased actions related to the transfer of inertial forces between structures. This will be most pronounced where the structures are of different heights. Guidance is given on assessment of this in Appendix 4D of the NZSEE Guidelines.

Where the floors are misaligned, there is a possibility of localised damage to columns or walls on the boundary, through impact from the adjacent floor plate, in addition to any inertial force transfer as noted above. Potentially affected elements should be carefully assessed for such damage.

8.4.2 CMRFs of Reduced Ductility

Reduced ductility CMRFs may exhibit poor behaviour, due to a range of factors including:

- Inadequate confinement
- Inadequate or poorly located laps
- Inadequate shear steel (for the flexural capacity of the members)
- Inadequate joint reinforcement
- Inappropriate hierarchy of yielding leading to soft storey behaviour.

This is often compounded by the structures having been designed for relatively low seismic loads, particularly in pre-1965 designs, or low-rise structures.

Note however that this is not always the case. Further guidance is available in Fenwick and McRae[7].

In reality, there is in most cases a continuum of performance between ductile and non-ductile behaviour, although assessment against current standards tends to yield a binary pass/fail outcome, e.g. anti-buckling steel either meets the spacing provisions ($s \leq 6d_b$) or not, in which case the allowable ductility is downgraded significantly.

Inappropriate Mechanisms. Frame action in non-ductile CMRFs may often result in column sidesway mechanisms, particularly for the earlier frames. In these structures, member sizes were often dominated by gravity, resulting in deep strong beams and relatively small columns.

8.4.3 Ductile CMRFs

Displacement Compatibility. Prior to NZS3101:1995, the design of interior (designated gravity) columns was often 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. Such columns may have significant strength, due to an alternative approach of allowing reduced tie requirements if a lower strength reduction factor ($\phi = 0.7$) was adopted. This may manifest itself in column shear failure or reinforcement buckling at higher drifts.

Premature Yield. In structures designed to develop high levels of ductility, the onset of yield may occur at relatively low seismic loads. Although these structures may be detailed for this level of ductility, the damage to reinforcement at this level of demand may not be repairable.

8.5 ASSESSMENT AND ANALYSIS

It is recommended that the NZSEE Guidelines is used in the assessment of CMRFs, whether fully ductile or of reduced ductility. The NZSEE Guidelines provides guidance on both force-based and displacement-based methods. However, it is recommended that displacement-based assessment is used, to provide a better understanding of the likely behaviour of buildings.

The assessment of existing structures through use of design methods (such as NZS 3101:2006) is strongly discouraged, except as guidance.

The assessment of earthquake damaged buildings must take into account the damage sustained by the building, in considering its ability to resist future earthquakes. This requires first that a sufficiently comprehensive damage survey is completed and then that there is a means of correlating the observed damage to the expected performance. It should not be expected that it will be possible to predict by analysis the actual damage suffered, but the observed behaviour of the building should be generally

[7], Fenwick and McRae *Comparison of New Zealand standards used for seismic design of concrete buildings*. New Zealand Society for Earthquake Engineering Bulletin Vol 42, No 3, September 2009

aligned with the findings of the analysis. Iteration of the analysis may be required to achieve this, or more comprehensive analysis techniques may need to be used.

In the case of more complex structures, the analysis techniques used should be reflective of the form and materials of the building. For example, in torsionally irregular buildings, it is important that the methods used can assess the impact of torsion.

The impact of damage on future performance is dependent on the nature of the damage, and the extent to which it may be repaired or the structure modified. One of the difficulties of this assessment is that while the capacity of the building may be essentially unchanged by the damage, there is a possibility that the future performance may be reduced as a result of the number of cycles of loading sustained during the earthquakes. This applies mainly to potential strain hardening of the reinforcement, as discussed in section 8.5.1 below.

If testing of the reinforcement is considered, the testing programme must take into account the observed damage patterns and should be undertaken in sufficient locations to give a good overview of the variation of results across the full range of damage patterns. If non-destructive testing is used, it should at least be calibrated by limited destructive testing to ensure the outcomes are reflective of the actual steel used.

8.5.1 Impact of Strain Hardening

In all cases, the effect of repeated cycles of yield needs to be considered. Lower grade ductile steel generally has a longer yield plateau and is therefore generally better able to survive repeated cycles. However, it may have a reduced post-yield stiffness and be more prone to buckling between layers of lateral support.

In cases where the reinforcing content is relatively low, there is a significant risk that only a single, large crack may form when the reinforcement yields. This may result in bar fracture in extreme cases.

Strain hardening in itself is not a problem and in fact is required in order to force appropriate plastic hinge development. As the reinforcement strain hardens at the primary crack, secondary cracking will occur and the plastic hinge lengthens.

If the moment gradient is too high, it is possible that strain hardening of the yielded bar will not be enough to form further primary cracks. This means that all of the inelastic deformation will occur at the one location. Compounding this, if the concrete strength is particularly high, yield penetration along the reinforcing bars may be limited. The expected strain penetration length may be found in various references[8] and together with the expected plastic hinge length, is the basis of current ductile concrete design requirements

---

[8] Priestley, Seible and Calvi, Displacement-Based Seismic Design of Structures, IUSS Press, 2007

Detailed analysis may help to determine the possible strain history of the reinforcement, but testing may be the only way to determine the condition of the reinforcement. Destructive testing provides the most accurate information, provided that boundary conditions are considered, but it may not be practical for testing large numbers of locations. Non-destructive testing in conjunction with limited destructive testing (for calibration) may provide a more comprehensive coverage.

Testing in itself will only provide an estimate of what has happened. Analysis is then necessary to correlate the deterioration of the reinforcement with the likely future demand, in order to determine the available capacity. If there is significant strain hardening over short strain penetration lengths, and it is not possible to demonstrate that future cracking will occur elsewhere, it is considered that a reduction in assessed ductility capacity is appropriate.

Older non-ductile CMRFs frequently have little or no documentation available. Reinforcement may be detected using non-destructive scanning equipment, but care must be taken to identify lap locations, and it is necessary to verify whether bars are plain or deformed.

Suggested method for assessing impact of strain hardening:

Testing being done in Christchurch is typically reporting the extent of strain hardening expressed as the percentage of peak strain at uniform elongation. Under AS/NZS4761:2002, for Grade 300E or 500E, this is required to be a minimum of 15% or 10% respectively.

In consideration of damage with respect to insurance, it may be considered that any strain hardening represents a reduction in total pre-existing capacity, but that is outside the scope of this document, which is intended to provide a methods for assessment of remaining capacity only. Therefore this assessment method is based on the minimum limits imposed by AS/NZS4671 and as they relate to NZS3101.

For reinforced concrete, material strain limits in potential plastic hinge regions are defined in NZS3101, clause 2.6.1.3.4. Under this the limiting curvatures are given as:

$\phi_{max} = K_d \phi_y$, where $\phi_y = \dfrac{2 f_y}{E_s h}$ and K_d is taken from Table 2.4(a).

for $f_y \leq 425$ MPa.

Table 2.4 (a) – K_d factor for limiting curvatures in flexural plastic hinge regions in beams and columns (from NZS3101:2006)

Classification of Plastic region	Type of plastic hinge	Limiting curvature K_d
Limited ductile plastic	Unidirectional	22

region	Reversing	11
Ductile region	Unidirectional	38
	Reversing	19

This implies for reversing hinges, usable inelastic strain $10\varepsilon_y$ and $18\varepsilon_y$ for limited ductile and ductile regions respectively, noting the limitation on f_y. Although calculated at the ULS condition, these material strain limits are calculated in order to ensure that reasonable capacity exists for the larger rotations required under an event of greater intensity, in accordance with the loading standard. *And presumably make suitable allowance for the greater number of cycles that could be required in a much larger event.*

Note that the material strain limits in NZS 3101:2006 imply much higher direct axial strains than the reinforcement is likely to experience, but these limits should include allowance for kinking and flexural strains, which may be greater than the direct axial component of strain resulting from plastic hinge rotation only.

The draft recommendations are that:

1. Provided that testing shows that the reduction in strain capacity leaves no less than the required minimum strain capacity of AS/NZS4671, the building capacity may be considered to be undiminished, even though there has been some minor reduction in the total usable strain capacity of the reinforcement.

2. Given the requirement for $10\varepsilon_y$ minimum in limited ductile hinge regions, a linear reduction is proposed for the maximum allowable ductility from μ=6 at ≥100% of the limits of AS/NZS4671, down to μ=1.25 at 10/18≈ 50% remaining capacity.

UNLESS, it can be shown that the strain hardening increase at the first primary crack is sufficient to force a second primary crack in later cycles of load. This requires the moment gradient over the plastic hinge length to be sufficiently flat, i.e.:

$$\Delta M \leq \frac{f_{su}}{f_y}$$

8.6 REPAIR AND STRENGTHENING STRATEGIES

Repair techniques for CMRFs must be considered carefully in the context of future performance expectations. In particular, epoxy injection of cracked frame elements should only be considered after a comprehensive evaluation of the condition of the reinforcement.

Although the list below is by no means comprehensive, it discussed some of the more common methods with some comments as to their limitations:

Table 8-1: Repair and Strengthening methods

Repair type	Advantages	Disadvantages	SLS performance	ULS Performance
Epoxy injection	Simple to implement, relatively unintrusive May be carried out with limited occupation, provided screening for fumes is possible	Epoxy fumes may be H&SE hazard Need to access all sides of structure to get full coverage Limited penetration for narrow cracks and/or wide elements	Will restore some of initial (pre-cracked) stiffness, but not all.	Provided that there are no significant issues with strain hardening or bond degradation between reinforcement and concrete, will restore most of initial strength
Partial break-out and rebuild	Restores more of strength in damaged areas than epoxy; less intrusive than full demolition and rebuild	Still has significant noise and vibration, creation of debris for disposal, may not be possible to even partially occupy, or must be done out of hours. May force failure into adjacent sections if not matching strength and stiffness	Will restore close to full stiffness, therefore SLS performance as good as prior to earthquakes	Provided that damaged reinforcement is replaced, will restore to capacity that existed prior to earthquakes
Full break-out and rebuild	Allows strength to be added with no increase in dimensions	Highly intrusive	Restores SLS condition to at least that which existed pre-earthquake, subject to consideration of foundation capacity	Allows improvement to ULS performance to whatever level is required, subject to consideration of foundation capacity
External reinforcing of frame with reinforced concrete or structural steel	No/minimal demolition of frame required, therefore minimises propping and shoring requirements	Can be difficult to achieve required bond of old to new. May change relative stiffness of elements, changing load distributions; requiring consequent consideration of diaphragm actions May require epoxy injection first May change behaviour of frame or hierarchy of failure.	Restores SLS condition to at least that which existed pre-earthquake, subject to consideration of foundation capacity	Allows improvement to ULS performance to whatever level is required, subject to consideration of foundation capacity
External reinforcing	No demolition of frame required,	Can be difficult to achieve required	Will restore some of the initial (pre-	Provided that there are no significant

Repair type	Advantages	Disadvantages	SLS performance	ULS Performance
of frame with FRP	therefore minimises propping and shoring requirements Bond generally easier to achieve than RC overlay or structural steel No appreciable change to elastic stiffness of frame	bond if multiple layers required. Only applicable to shear strength and confinement enhancement, not suitable for flexural enhancement, except in the case of protected members in a capacity designed CMRF. May require epoxy injection first.	cracked) stiffness of the structure, but not all and possibly not to the same extent as epoxy repairs (assuming no epoxy injection prior to wrapping).	issues with strain hardening or bond degradation between reinforcement and concrete, will restore most of initial strength to extent achievable through epoxy injection. Note that this can be used mainly only to increase shear capacity and confinement, to ensure capacity design objectives achieved
Selective weakening eg through vertical slots, cutting for flexural steel	May improve global performance of building	Requires careful consideration of load paths and available ductility. Will affect overall load distribution. May require higher order of analysis.	Requires full re-evaluation	Requires full re-evaluation. Assuming that capacity design approach followed, should improve performance significantly.

8.7 REFERENCES

Moehle, Jack P., Hooper, John D., Kelly, Dominic J., and Meyer, Thomas R. (2010). "*Seismic design of cast-in-place concrete diaphragms, chords, and collectors: a guide for practicing engineers*", NEHRP Seismic Design Technical Brief No. 3, National Institute of Standards and Technology, Gaithersburg, MD, NIST GCR 10-917-

New Zealand Society for Earthquake Engineering, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, June 2006.

New Zealand Society for Earthquake Engineering Bulletin Vol 42, Fenwick and McRae *Comparison of New Zealand standards used for seismic design of concrete buildings*. NZSEE 2009

Priestley, Seible and Calvi, *Displacement-Based Seismic Design of Structures*, IUSS Press, 2007

Standards New Zealand, *Concrete Structures NZS3101:1982*, SANZ 1982.

Standards New Zealand, *Concrete Structures Standard – The Design of Concrete Structures NZS3101:1995*, SANZ 1995

Standards New Zealand, *Concrete Structures Standard Part 1 – The Design of Concrete Structures , NZS 3101:2006*, SANZ 2006.