

Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential buildings

Part 3 Technical Guidance

Section 9 – Reinforced Concrete Wall Buildings

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

Revision 6, 24 April, 2013

**This draft document is NOT for distribution.
The contents do not represent government policy**

Document Status

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

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

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

Part 1	Background
Part 2	Evaluation Procedure
Part 3	Technical Guidance

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

Where errors or 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 5, Developed Draft, General Issue 21 December 2011
Revision 6, Minor amendments, plus addition of Appendix C

CONTENTS

9	REINFORCED CONCRETE WALL BUILDINGS	1
9.1	Introduction.....	1
9.2	Notation	2
9.3	Description	2
9.4	Seismic Response Characteristics and Common Deficiencies	3
9.4.1	Flexure	4
9.4.2	Rocking	5
9.4.3	General Performance Issues	5
9.5	Assessment and Analysis	6
9.5.1	Displacement based assessment.....	7
9.5.2	Failure Mode and Repair Assessment.....	15
9.6	Repair and Strengthening Strategies	21
9.7	References	23
Appendix 9A	Worked Examples	
Appendix 9B	Spectral Acceleration and Displacement Data	
Appendix 9C	Commercial Low-Rise Panel Structures	

TABLES

Table 9-1: Wall Element Failure Modes	6
Table 9-2: Failure Modes and Repair Assessment – Flexure	17
Table 9-3: Failure Modes and Repair Assessment – Shear	18
Table 9-4: Failure Modes and Repair Assessment – Crushing.....	19
Table 9-5: Failure Modes and Repair Assessment – Sliding	20
Table 9-6: Failure Modes and Repair Assessment – Rocking	21
Table 9-7: Repair and Strengthening methods	22

FIGURES

Figure 9-1: Typical Wall Systems	3
Figure 9-2: Maximum structural ductility factors from NZS3101:2006	4
Figure 9-3: Summary of Displacement Based assessment procedure	9
Figure 9-4: Idealised response of wall system – all walls reach yield	11
Figure 9-5: Idealised Response of Wall system – all walls do not reach yield	12
Figure 9-6: Equivalent Viscous Damping vs. Ductility	13
Figure 9-7: Feb 22nd Average Displacement Spectra for differing Damping Levels	15

9 REINFORCED CONCRETE WALL BUILDINGS

9.1 INTRODUCTION

Concrete shear walls have been commonly used throughout Christchurch, in various forms from low-rise to medium-rise construction. Although in many cases concrete wall buildings have performed satisfactorily (at least in terms of life safety), many suffered more damage than was expected. In particular it is of note that the two major building collapses were both in concrete wall buildings, although the reasons are yet to be determined.

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

- the significance of damage to walls, with emphasis on the reduction in capacity (stiffness and strength);
- assessment of remaining life, acknowledging that low cycle fatigue may have reduced the available inelastic strain capacity of the reinforcement;
- the effectiveness of repairs to wall systems.

One of the more significant concerns to emerge from assessments that have been completed to date is that concrete walls have apparently failed to perform as may have been expected from years of research. Instead of well developed plastic hinges displaying fan-cracking patterns, there have been single wide cracks formed, with fractured flexural steel in the worst cases. Factors that are thought to influence this include:

- Low reinforcement ratios.
- High compressive strength (in older concrete) relative to the nominated value.
- (Assumed) high tensile strength of the concrete.
- The high dynamic load rate caused by the Feb 22 earthquake imposing high drifts over relatively few cycles of load.

There are a variety of existing documents available internationally that address some or all of these aspects. In particular:

- the NZSEE Red Book¹ offers guidance on the assessment of existing walls and possible strengthening solutions, but it does not address damage.
- FEMA 306² is entirely dedicated to evaluation of earthquake damage, but it requires careful consideration for adaptation to New Zealand conditions. It does not appear to address low cycle fatigue, which has been found to be significant in cases where testing has been completed to date.

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

² Federal Emergency Management Agency, *FEMA 306 Evaluation of Earthquake Damaged Concrete and Masonry Buildings – Basic Procedures Manual*, 1998

- NZSEE and more recently SESOC³ (in downloadable form for members) have published a paper providing guidance on the performance of rocking systems

The methodology presented in this section provides guidance generally in accordance with the Red Book, with some updating to reflect more recent research and development. However, users who wish to consider alternative methodologies, or who require more background knowledge may wish to refer to the Red Book.

9.2 NOTATION

A_{re}	Aspect ratio of wall to effective height, h_{eff}
h_{eff}	Height to effective centre of seismic load, typically $2/3 h_w$
h_w	Height of wall, assumed to be the full height of the building
H	Height of building, or height to uppermost seismic mass
L_p	Length of plastic hinge in wall
l_w	length of structural wall
U	Displacement at h_{eff} . Note this is NOT the total displacement, which occurs at uppermost seismic mass
δ	Interstorey drift ratio
ϵ_y	Yield strain of reinforcement
ϕ	Curvature
ξ	Damping ratio

Subscript notation: Subscripts are used to denote different stages of performance and different components, as follows (unless otherwise noted in the text):

X_{wy}	performance of an individual wall, at yield
X_{sy}	performance of the whole system, at yield
X_{wu}	performance of an individual wall, at ULS
X_{su}	performance of the whole system, at ULS
X_{wc}	capacity of an individual wall
X_{sc}	capacity of the system as a whole
X_{wp}	post-yield performance of an individual wall, equal to $X_{wu} - X_{wy}$
X_{sp}	post-yield performance of the whole system, equal to $X_{su} - X_{sy}$
X_{sL}	Lyttelton earthquake actions on the whole system
X_{wL}	Lyttelton earthquake actions on an individual wall
X_{Lp}	Lyttelton earthquake post-yield component of actions, equal to $X_{sL} - X_y$

9.3 DESCRIPTION

Concrete structural walls, “shear walls”, started to be used from about the mid 1920’s. Earlier walls were generally thin and lightly reinforced by current standards, although some were well detailed.

The Concrete Standard, NZS3101:1982 presented the first formal requirements for seismic design and detailing of structural walls, with subsequent improvements being

³ Kelly, TE, *Tentative Seismic Design Guidelines For Rocking Structures*, SESOC Journal Vol 24 No. 1, 2011

made in the 1995 and 2006 versions of the Standard. Notwithstanding this, some earlier walls incorporated detailing that is close to current standards.

Wall systems may take a variety of forms according to the building configuration and designers' intentions. Many walls may also form part of a mixed system (comprising combinations of walls and lateral force resisting frames). This can be by intention, or in many earlier cases, where stair, lift or boundary walls that were not necessarily intended to form part of the lateral load resisting system, in fact, contribute significantly to the overall behaviour.

General forms of walls are described below in Figure 9-1 below. Most wall configurations will conform to one or the other of these general arrangements, noting that often the behaviour will be significantly modified by the foundation system. In particular, rocking foundation systems will often act to preclude some of the less desirable failure modes, and will substantially influence the overall building response.

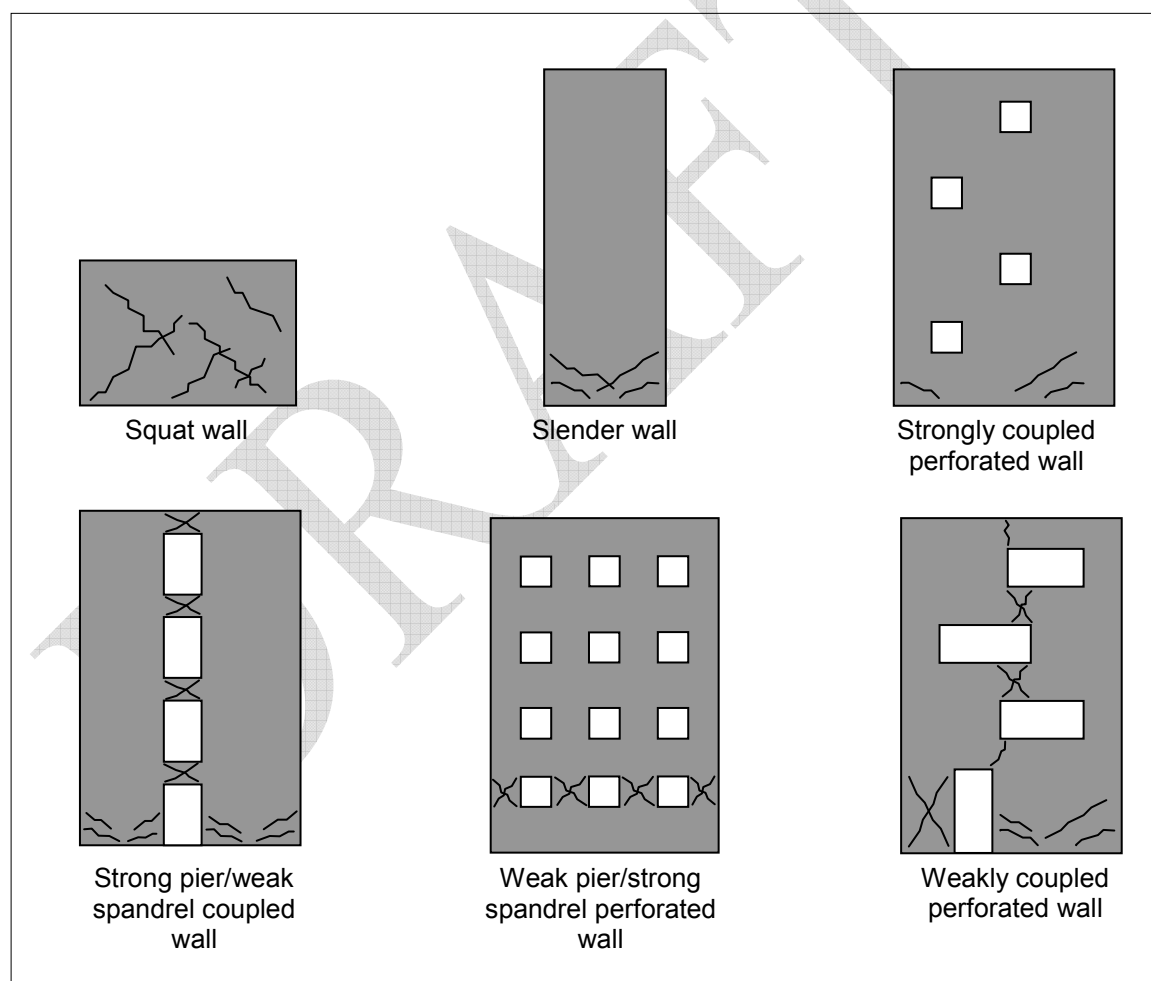


Figure 9-1: Typical Wall Systems

9.4 SEISMIC RESPONSE CHARACTERISTICS AND COMMON DEFICIENCIES

Walls are generally considered as flexural elements with potential plastic hinges at the base, although there are many walls with small height to length ratios, known as “squat walls”, or squat elements within wall systems, that may work primarily in shear.

Another significant factor in the response of many walls is the possibility of rocking; primarily in walls with shallow foundations. The potential for rocking should be identified prior to considering any more detailed analysis, as it immediately changes the expected behaviour.

9.4.1 Flexure

Along with rocking, flexure is the preferred mode of behaviour for wall systems. This may take the form of simple cantilever walls, or coupled wall systems. In most such cases, the main flexural mode will be at the base of the wall, but reductions in wall length or thickness may in some cases introduce a further potential hinge location at higher levels.

Type of structure	Reinforced concrete	Prestressed concrete with bonded non-prestressed reinforcement
1. Nominally ductile structures	1.25	1.25
2. Structures of limited ductility		
(a) Moment resisting frame	3	3
(b) Walls	3	3
(c) Cantilever face loaded walls (single storey only)	2	2
3. Ductile structures		
(a) Moment resisting frame	6	5
(b) Wall		
(i) Two or more cantilevered	$\frac{5}{\beta_a}$	As for reinforced concrete
(ii) Two or more coupled	$\frac{5}{\beta_a} \leq \frac{3A+4}{\beta_a} \leq \frac{6}{\beta_a}$	As for reinforced concrete
(iii) Single cantilever	$\frac{4}{\beta_a}$	As for reinforced concrete
<p>NOTE –</p> <p>(1) The ductility factor is a measure of the anticipated overall structural ductility demand which is a function of the appropriate magnitude of earthquake design forces.</p> <p>(2) In the above table</p> <p>$1.0 < \beta_a = 2.5 - 0.5A_r < 2.0$</p> <p>and</p> <p>$\frac{1}{3} \leq A = \frac{T_w L'}{M_{ow}} \leq \frac{2}{3}$</p>		

Figure 9-2: Maximum structural ductility factors from NZS3101:2006

Flexural behaviour is generally ductile, but the available ductility must be checked. NZS3101:2006 sets maximum ductility limits for wall systems based on the aspect (height to length) ratio, with a maximum of $\mu = 4$ for single cantilever walls and $\mu = 5$ for multiple cantilever wall systems (refer Figure 9-2 above). Earlier blanket assumptions of ductility of up to $\mu = 5$ may be unconservative, i.e. these levels of ductility may not be able to be met.

In general, it should be assumed that wall systems that fail in shear of gravity load bearing elements are allocated low ductility, or alternative gravity load systems must be inserted into the structure to preclude collapse.

9.4.2 Rocking

Rocking may well be the mechanism that has saved a number of walls from further damage, that may otherwise have performed poorly. In particular, earlier non-ductile walls that may have been designed for lower levels of seismic load may have rocked prior to a less desirable wall failure mode developing.

Rocking is a conceptually simple, but analytically more difficult to verify, for a number of reasons, including:

- The period is displacement dependent at in excess of the rocking displacement.
- The level of damping developed under rocking is poorly understood.
- The influence of vertical accelerations may be more critical than in other systems.
- The interaction with the soil is critical, considering both bearing capacity and deformation within the soil.

Notwithstanding those, rocking is an acceptable system, or even desirable for squatter wall systems. In broad terms, provided that the following are met, rocking walls are a suitable system:

- The onset of rocking should be at a level greater than the serviceability level earthquake (taken at $Z=0.3$, $R=0.33$).
- At the rocking load, the soil pressure should be less than ϕq_u , and settlement should be at less than the acceptable limits agreed with a geotechnical engineer.

9.4.3 General Performance Issues

Some of the major issues with the performance of walls are as follows:

1. Few large cracks as opposed to well distributed fine cracks. It has been postulated that this may be due to a number of factors, including:
 - a. The loading sequence, commencing with a large pulse which caused the most significant movement.
 - b. The tensile strength of the concrete being significantly higher than expected, meaning that the reinforcement did not have the capacity to distribute the cracks.
 - c. The low reinforcement content.
2. Buckling failures, most pronounced in L- and T-shaped walls. In these cases, when the outstand element went into compression, the combination of the axial load (possibly increased under seismic load) and the flanges in tension, forced a compression buckling failure in many cases.

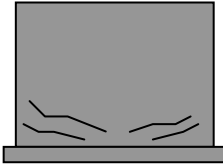
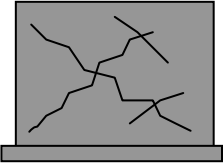
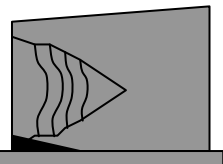

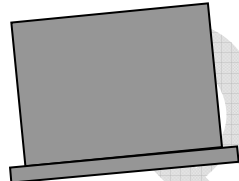
3. Failure of ducted splices. These failures took several forms:
 - a. Lack of grout causing complete failure of the splice.
 - b. Over-confinement of the duct preventing yield penetration and hence focusing the inelastic demand over too short a length, resulting in snapping of the reinforcement.
 - c. Buckling of the splice caused when the cover concrete has been lost, resulting in the unconfined horizontal steel being lost from the wall.
 - d. Insufficient transverse reinforcement around the ducted splice to maintain the integrity of the surrounding concrete and hence leading to failure of the splice between the duct with the grouted starter bar and the longitudinal steel in the wall.
4. Shear failures in walls where flexural inelastic behaviour may have been expected. It is of note that shear failures have been observed over the full height of the walls.
5. Snapping of flexural steel at wall bases. Similar to the ducted splices, where the walls have had relatively low reinforcement, a single crack has formed at the base of the walls, accompanied by low yield penetration along the main bars.
6. Failure of walls at construction joints, particularly in older walls which may have been constructed using site-batched concrete. In such cases, the construction joints have opened up, with sliding movement at the joints.
7. Bursting of horizontal bars. A number of walls have partially failed with the horizontal reinforcement yielding and buckling in the potential plastic hinge regions. Once yielded, the bars have been vulnerable to buckling.
8. Inadequate connection to the floor diaphragms – not strictly a problem with the wall itself, and will be dealt with separately in the floor section (to come).

9.5 ASSESSMENT AND ANALYSIS

Within the overall wall systems described in Figure 9-1 above, the walls may be broken down into individual elements, with the behaviour categorised according to the individual failure modes. These failure modes are described in Table 9-1 below.

Table 9-1: Wall Element Failure Modes

Type		Mode		Notes
------	--	------	--	-------

RCW1		Flexure	✓✓	Flexure is the ideal wall behaviour, as generally derived from NZS3101. Issues to be considered are whether the remaining reinforcement capacity is sufficient and what repair may be required.
RCW2		Shear (tension)	×	Shear is generally not the desired mechanism, as vertical load-bearing capacity may be lost at relatively low strains. This is generally considered a non-ductile mechanism and if a wall is shear controlled, it should be evaluated at $\mu=1$, $S_p=1$
RCW3		Crushing	××	Crushing may occur where there is inadequate confinement of the compression zone, or axial load in excess of the calculated demand. Excess axial load may result from elongation due to flexural actions, and from greater than expected overstrength actions, particularly in L- and T- shaped walls
RCW4		Sliding	✓	Sliding is not generally a design failure mode, but appears to have happened widely, particularly at poorly formed and compacted construction joints. However, because it maintains the gravity load bearing capacity, it is not inherently an unsafe mechanism.
RCW5		Rocking	✓✓	Rocking has probably saved many walls that would otherwise have failed if they had rigid foundations. Although inherently a simple mechanism, rocking is dynamically complex. Issues to be considered are the impact of rocking on the foundations and soil, with consideration to settlement and rounding of the soil profile, reducing the level of future performance.

In order to determine the hierarchy of failure and to evaluate the significance of wall damage, it is first of all critical to develop an understanding of the performance of the walls at a detailed level. The Red Book offers several methods of evaluating walls, although some methods may be more directed at improving the performance of walls, i.e. assessing against a target load, rather than determining capacity.

For the sake of simplicity, one method of assessing the key performance characteristics of concrete wall structures is presented herein, which will allow a full assessment to be made. It should be noted that other methods may also be used, but full consideration must be made of the actual ductility capacity and displacement demand.

9.5.1 Displacement based assessment

The most suitable form of analysis of shear wall systems is direct displacement based assessment, although a force based assessment can also be used. The method describe in this section is generally in accordance with that used in the Red Book, section 7.4.3, with some minor amendments. Refer to Figure 9-3 below for a flowchart describing the procedure.

In more detail, the process is described in the steps below:

Step 1: Probable Strength of the Building

Calculate the probable flexural strength M_{wprob} of each of the walls, noting the depth to neutral axis, c . In the absence of testing data, the following may be assumed:

- The expected mean yield stress is 1.08 times the lower characteristic yield stress
- The concrete compressive strength is 1.5 times the nominal concrete strength
- Strength reduction factors ϕ , may be taken as 1.0 for flexure, 0.85 for shear

Calculate the probable shear strength of each of the walls, V_{wprob} , using the process outlined in the Red Book, section 7.4.2.

Step 2: Base Shear Capacity

Calculate the base shear capacity V_{prob} of the building for the associated with the wall flexural capacity:

$$V_{prob} = \frac{\sum M_{wprob}}{h_{eff}}, \quad \dots 9(1)$$

where h_{eff} = the effective height of the walls, assumed to be $2/3h_w$ for cantilever wall systems

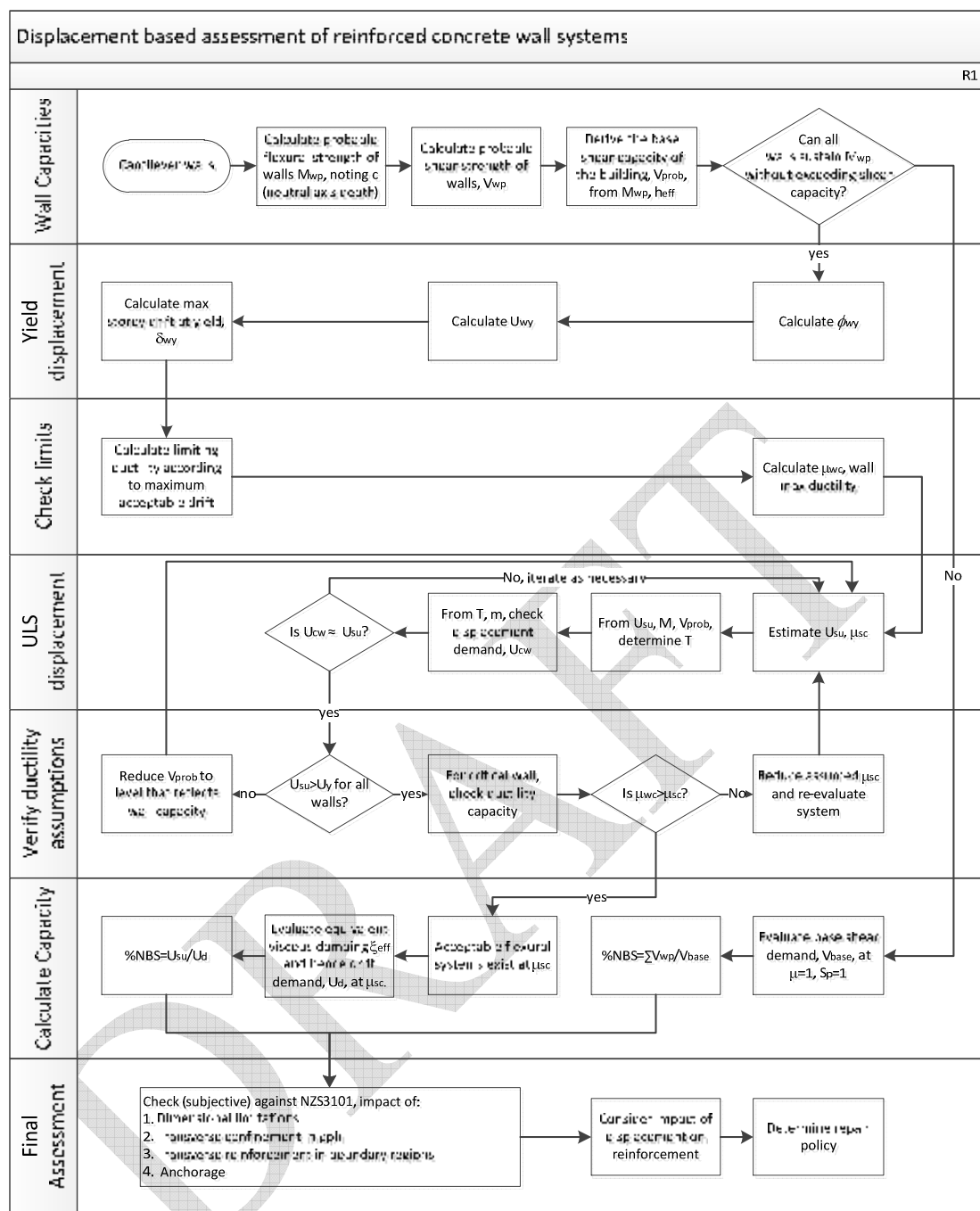


Figure 9-3: Summary of Displacement Based assessment procedure

Step 3: Check that suitable mechanisms can develop

Check that the shear associated with the development of the wall flexural capacity may be achieved without premature shear failure, i.e.

$$\frac{M_{wprob}}{h_{eff}} < V_{wprob} \quad \dots 9(2)$$

If this test fails, go to step 9, for non-ductile systems

Step 4: Check yield displacement of walls

The yield curvature and displacement of a wall is determined purely by its geometric properties. For a plain rectangular wall of length l_w , the nominal yield curvature is:

$$\phi_{wy} = \frac{1.8\varepsilon_y}{l_w} \quad \dots 9(3)$$

The corresponding yield displacement (at h_{eff}) is:

$$U_{wy} \approx \frac{\phi_{wy} h_{eff}^2}{3} \approx 0.6\varepsilon_y A_{re} h_{eff}, \quad \dots 9(4)$$

where A_{re} is the effective aspect ratio, $A_{re} = \frac{h_{eff}}{l_w}$

and the interstorey drift ratio at levels above h_{eff} is approximately:

$$\delta_{wy} \approx \frac{\phi_{wy} h_{eff}}{2} \approx 0.9\varepsilon_y A_{re} \quad \dots 9(5)$$

Step 5: Check limiting ductility according to drift limits

The overall drift limit for the walls is going to be the lesser of either $\mu_{wc} = 3$ to 5 (from NZS3101), or that which keeps the drift within acceptable limits to NZS1170.5 (generally a maximum of 2.5%). Drift criteria may limit the ductility of flexible walls, with the maximum drift at or above the effective height of the wall being:

$$\delta_{w,max} = \delta_{wy} + \delta_{wp} \quad \dots 9(6)$$

Given that the full post-yield drift, δ_{wp} of the wall is:

$$\delta_{wp} = \frac{U_{wp}}{(h_{eff} - 0.5L_p)} \quad \dots 9(7)$$

Where U_{wp} is the inelastic portion of the total drift,

$$U_{wp} = (\mu_{wc} - 1)U_{wy} \quad \dots 9(8)$$

and the plastic hinge length of the wall may be considered to be:

$$L_p \approx 0.5l_w \quad \dots 9(9)$$

Hence in order to satisfy the 2.5% drift limitation, it is found that the limiting ductility capacity:

$$\mu_{wc} = 0.025(A_{re} - 0.25)\left(\frac{l_w}{U_{wy}}\right) + 1 \approx 0.04 \frac{(A_{re} - 0.25)}{(\varepsilon_y A_{re}^2)} + 1 \quad \dots 9(10)$$

Step 6: Determine Maximum drift according to available damping

Estimate (for the system) the total drift U_{su} and ductility capacity μ_{su} , and from them, determine the effective period, T_{eff} from the acceleration spectra for the site; and then from T_{eff} , back-check the implied displacement U_{sc} . Iterate as necessary to achieve agreement between the assumptions of displacement and the implied ductility demand, using:

$$\mu_{su} = \frac{U_{sc}}{U_{sy}} \quad \dots 9(11)$$

Note that U_{sy} in this relationship is the idealised yield drift using the initial stiffness, k_i , of the system. This is shown in general terms in Figure 9-4 below.

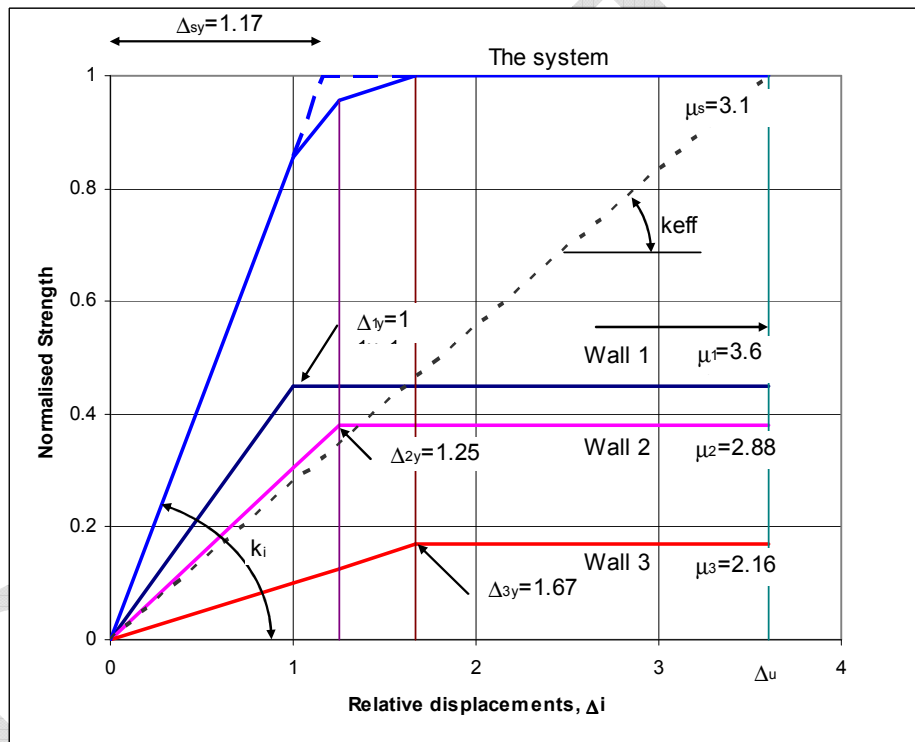


Figure 9-4: Idealised response of wall system – all walls reach yield

On completion of this step, the drift must be checked to ensure that all walls have exceeded their full yield displacement at this drift. With reference to Figure 9-4 above, for a three wall system, it can be seen in this case that all walls reach their yield drift before the limiting wall reaches its capacity.

In the event that the limiting wall has less ductility, or the other walls are very flexible, all walls may not reach their yield drift before the limiting walls reaches its capacity. This is illustrated below in Figure 9-5.

In this event, the capacity(ies) of any non-yielding walls must be reduced to allow for the limiting drift. If the system stiffness is reduced significantly, it may be necessary to further iterate to verify the overall system performance.

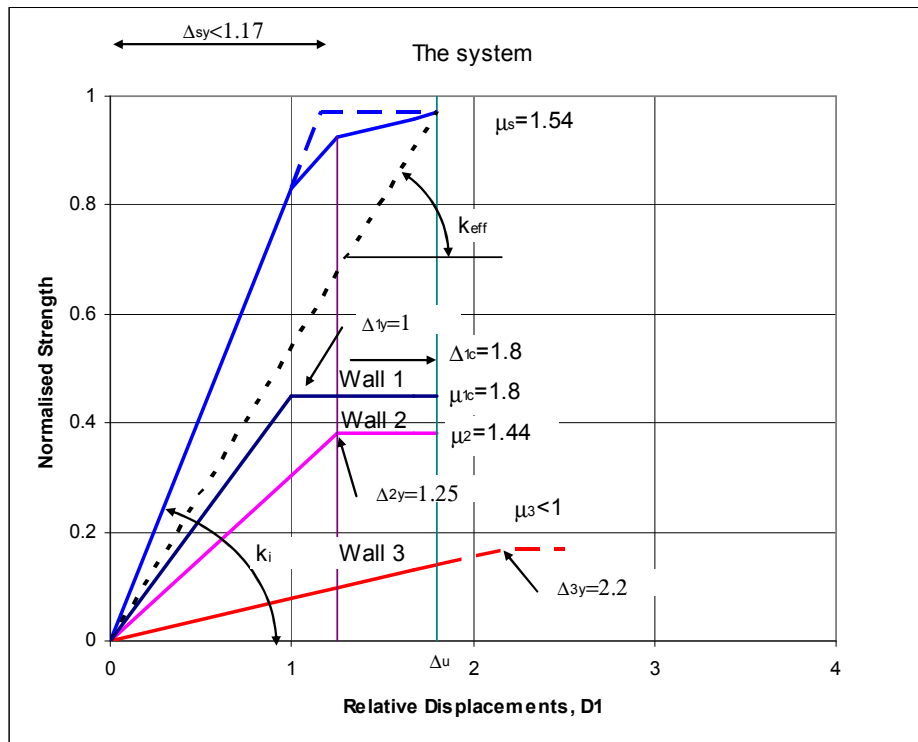


Figure 9-5: Idealised Response of Wall system – all walls do not reach yield

Step 7: Calculate the Equivalent Viscous Damping

The equivalent viscous damping ξ_{eff} is a function of the ductility. Generally it is taken that this is a function of:

- ξ_0 – the inherent damping (typically taken as 5%)
- ξ_{hv} – the hysteretic damping
- ξ_d – added damping due to supplemental damping (if present)

There are several published relationships for damping to ductility. For the purposes of assessment of concrete walls, the relationship below is recommended, either from the formula or as presented in Figure 9-6 below.

For walls,

$$\xi_{eff} = 5\% + 95\left(\frac{1 - \mu^{-0.5}}{\pi}\right)\% \quad \dots 9(12)$$

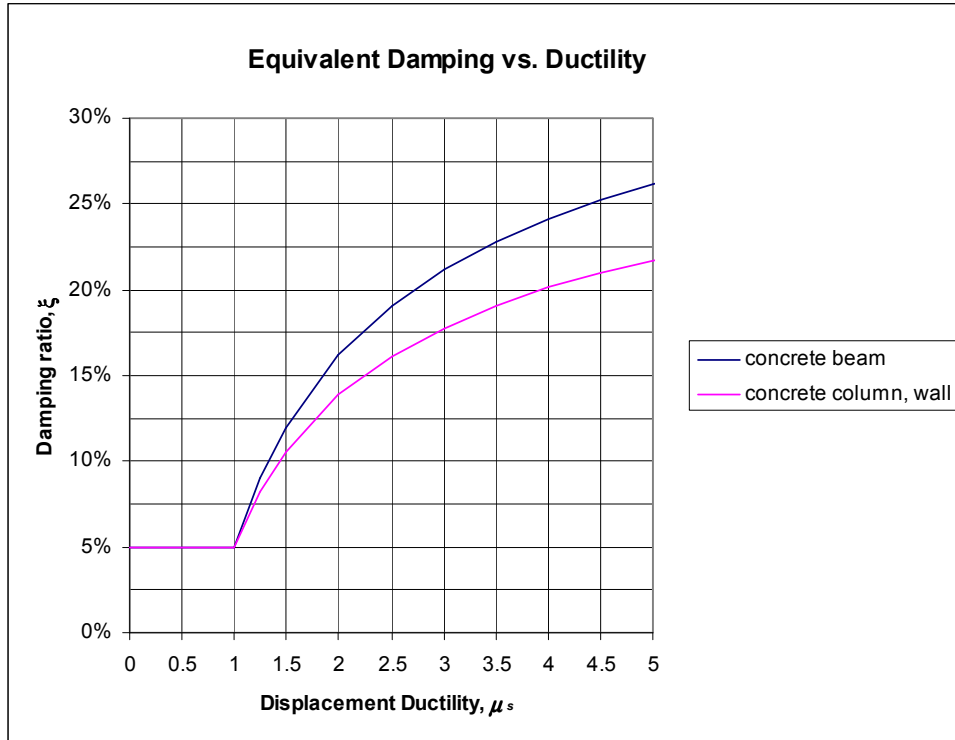


Figure 9-6: Equivalent Viscous Damping vs. Ductility

Step 8: Calculate the Drift demand

From the equivalent viscous damping and the effective period, the drift demand U_D can be calculated. This may be done using the combined acceleration and displacement spectra in Appendix 9B, for the calculated equivalent viscous damping ratio and the calculated effective period T_{eff} , taken from the effective stiffness, using

$$T_{eff} = 2\pi \sqrt{\frac{M_e}{g \cdot k_{eff}}} \quad \dots 9(13)$$

Where M_e is the effective mass, calculated from:

$$M_e = \frac{\sum m_i h_i}{h_e} \quad \dots 9(14)$$

And k_{eff} is the effective stiffness, calculated from

$$k_{eff} = \frac{\sum M_{wp}}{h_{eff} U_{sc}} \quad \dots 9(15)$$

Step 9: Calculate the overall %NBS

The %NBS capacity of the system can now be calculated simply for ductile systems as:

$$\%NBS = \frac{U_{su}}{U_D} \quad \dots 9(16a)$$

For non-ductile systems, the %NBS may be calculated from the base shear at $\mu = 1$, $S_p = 1$, as:

$$\%NBS = \frac{\Sigma V_{wpro b}}{V_{base}} \quad \dots 9(16b)$$

Step 10: Complete subjective checks against critical Code requirements

Although the process outlined above will determine the overall capacity of the system in the direction being reviewed, there is still a need to confirm that structural detailing provided will support the level of ductility demand that may be expected. Particular aspects that should be considered include:

1. Dimensional limitations
2. Transverse confinement in potential plastic hinge regions
3. Transverse reinforcement in boundary regions
4. Anchorage

Note that deficiencies in all of these elements have impacted on the performance of buildings in the earthquake.

Step 11: Calculate flexural crack widths

It is now possible to calculate the theoretical crack widths at the plastic hinge regions, according to the total drift demand from the actual earthquake records, and the pattern of observed cracking.

Utilising the Lyttleton earthquake actions, using the effective period and calculated equivalent viscous damping from above, the displacement demand, U_{sL} can be determined from the averaged displacement spectra in Figure 9-7 below.

From the displacement, calculate the effective post-yield rotation of the walls from:

$$\theta_{wLp} = \frac{U_{sL} - U_{wy}}{h_{eff}} \quad \dots 9(17)$$

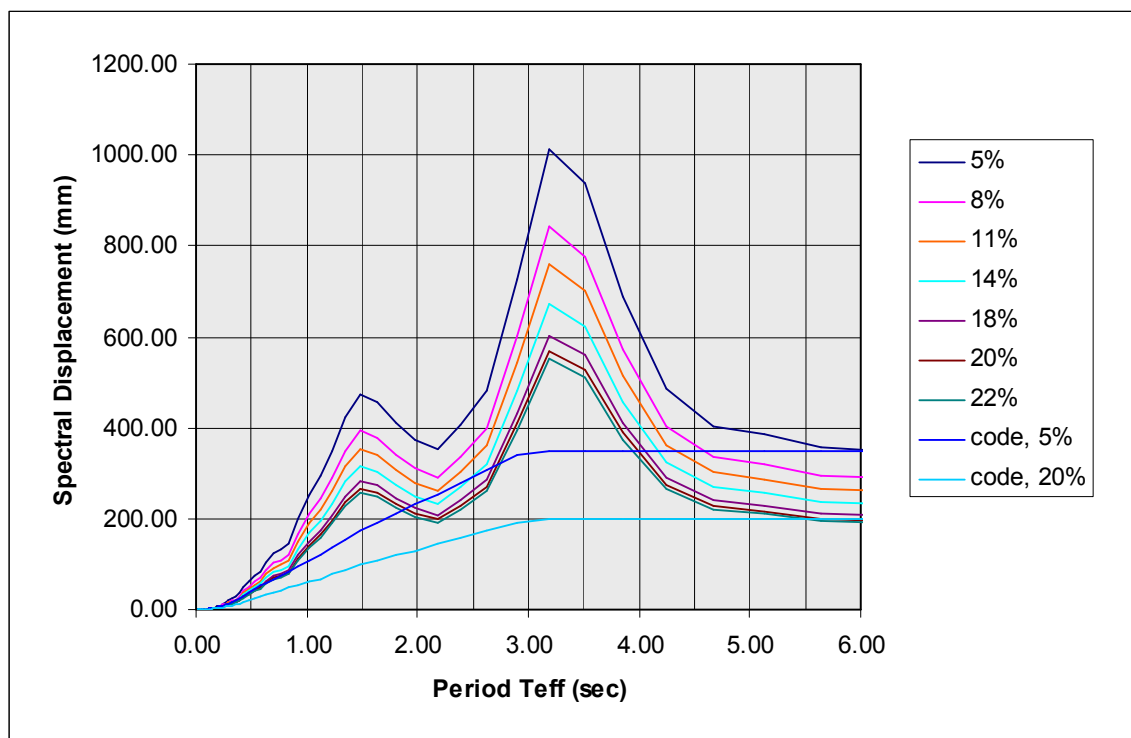


Figure 9-7: Feb 22nd Average Displacement Spectra for differing Damping Levels

Note: Code spectra are for $R=1$, Soil Class D, $Z=0.3$, for comparison purposes only

The maximum total peak crack width, $w_{wLp, tot}$, at the plastic hinge region, can now be calculated:

$$w_{wLp, tot} = \theta_{wLp} \times (l_w - c) \quad \dots 9(18)$$

The impact of this will depend on the distribution of cracks within the plastic hinge zone. Clearly, where only one crack has formed, all of this strain will have been focused in a single location. Where there are multiple cracks, clearly the strain will be distributed more evenly.

It is recommended that the number of cracks, n_c over the potential plastic hinge length is counted and the crack width, w_L is distributed according to:

$$w_{wLp, max} = w_{wLp, tot} \times \frac{2}{(n_c + 1)}, \text{ for all } n_c \quad \dots 9(19)$$

Step 12: Complete assessment of the impact of the cracks

With reference to Table 9-2 to Table 9-6 below, complete the assessment and repair process for each wall being evaluated.

9.5.2 Failure Mode and Repair Assessment

The following tables are intended to provide guidance for assessment and repair, according to the failure modes and the assessment (from above) of the likely reasons for the observed performance, and its implications for future performance.

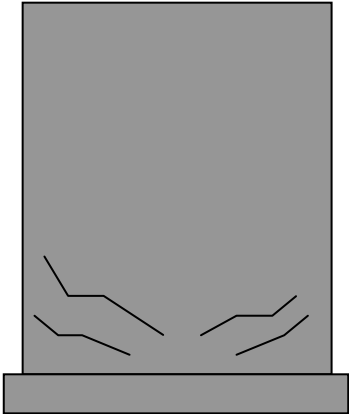
Note that the repair methodologies are discussed further in section 9.6 following.

The tables below introduce the k' factor, in column 1. This is an assessment of the reduction in ULS capacity that has occurred because of the damage observed, assuming a force-based assessment (as might be used for other aspects of the assessment). In general terms:

$$\phi R' = k' \phi R \quad \dots 9(20)$$

Where ϕR is the capacity prior to the damage, and $\phi R'$ is the capacity post-damage

Table 9-2: Failure Modes and Repair Assessment – Flexure

RCW1 – Flexure mode				
		Assumptions/ verification required		Ductility: $\mu=1$ to 5 Capacity design required if $\mu \geq 1.25$
	Theoretical crack width in accordance with Step 11 above	Investigation required	Repair methodology	Post-repair capacity
Minor damage $k'=0.9k$	<0.5mm		Epoxy injection	100%
Moderate Damage $k'=0.5k$	0.5mm – 2mm	Expose reinforcement in high strain areas and test for strain hardening across cracks	Epoxy injection, strain hardening assessed <15%	100%
			Epoxy injection, strain hardening assessed at 15% to 30%	60% May still need to enhance structure or change load path
			Strain hardening assessed at greater than 30% - Demolish and rebuild affected areas	100%
Severe Damage $k'=0k$	>2mm		Demolish and rebuild, OR	100%
			Remove and use alternative structure	0%

Note: The % strain hardening in the table is assessed as the percentage of the overall elongation capacity of the bar, where 100% is the maximum capacity, nominally a 20% elongation. Hence 15% strain hardening equates to 3% elongation.

Table 9-3: Failure Modes and Repair Assessment – Shear

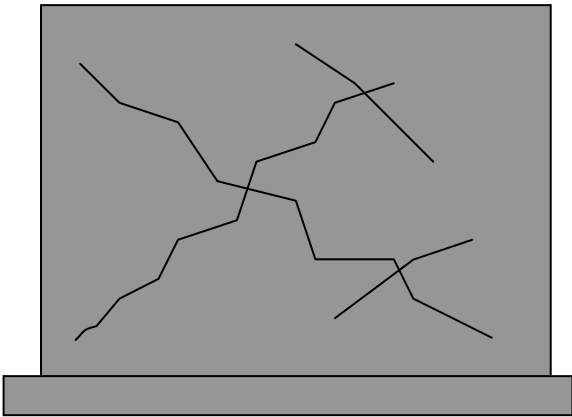
RCW2 – Shear mode				
			Assumptions/ verification required	Ductility: $\mu=1$, $S_p=1$ Capacity design not possible
	Actual crack widths	Investigation required	Repair methodology	Post-repair capacity
Minor damage $k'=0.9k$	<0.5mm		Epoxy injection	100%
Moderate Damage $k'=0.5k$	0.5mm – 5mm	Expose reinforcement in high strain areas and test for strain hardening across cracks	Epoxy injection, strain hardening assessed <15%	100%
			Epoxy injection, strain hardening assessed at 15% to 30%	60% May still need to enhance structure or change load path
			Strain hardening assessed at greater than 30% - Demolish and rebuild affected areas	100%
Severe Damage $k'=0k$	>5mm, single crack		Demolish and rebuild, OR	100%
			Remove and use alternative structure	0%

Table 9-4: Failure Modes and Repair Assessment – Crushing

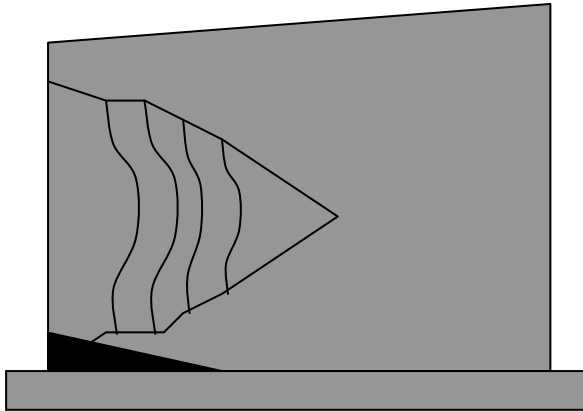
RCW3 – Crushing mode				
			Assumptions/ verification required	Ductility: $\mu=1$. $S_p=1$ Capacity design not possible
	Total vertical displacement	Investigation required	Repair methodology	Post-repair capacity
Moderate Damage $k'=0.5k$	<2mm, no spalling over height of wall		Epoxy injection,	75% Will still need to enhance structure or change load path
			OR Demolish and rebuild affected areas	100%
Severe Damage $k'=0k$	>2mm		Demolish and rebuild, OR	100%
			Remove and use alternative structure	0%

Table 9-5: Failure Modes and Repair Assessment – Sliding


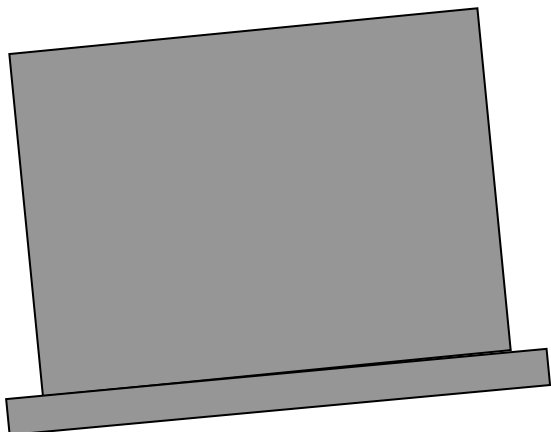
RCW4 – Sliding mode				
			Assumptions/ verification required	Ductility: $\mu=1.25$ (Capacity design required if $\mu \geq 1.25$)
	Horizontal offset	Investigation required	Repair methodology	Post-repair capacity
Minor damage $k'=0.9k$	<0.5mm		Epoxy injection	100%
Moderate Damage $k'=0.5k$	0.5mm – 2mm	Expose reinforcement in some areas and test for strain hardening	Epoxy injection, strain hardening assessed <15%	100%
			Epoxy injection, strain hardening assessed at 15% to 30%	60% May still need to enhance structure or change load path
			Strain hardening assessed at greater than 30% - Demolish and rebuild affected areas	100%
Severe Damage $k'=0k$	>2mm, single crack		Demolish and rebuild, OR	100%
			Remove and use alternative structure	0%

Table 9-6: Failure Modes and Repair Assessment – Rocking

RCW5 – Rocking mode				
			Assumptions/ verification required	Ductility: $\mu=1$ to 5 Capacity design required Note that at $\mu>3$, may exceed SLS displacement limits
	Theoretical wall uplift	Investigation required	Repair methodology	Post-repair capacity
Minor damage $k'=0.9k$	<5mm		Epoxy injection to damaged concrete	100%
Moderate Damage $k'=0.5k$	5mm – 20mm	Expose reinforcement in high strain areas, such as the adjacent slab and tops of piles* (if necessary) and test for strain hardening	Reinstate levels in the walls, repair foundations, wall, and misc structures (floors beams, foundations) connected to the wall(s)	75% Depending on the damage type and the repair methods.
			OR Demolish and rebuild affected areas	100%
Severe Damage $k'=0k$	>20mm		Reinstate levels in the walls, repair foundations, wall, and misc structures (floors beams, foundations) connected to the wall(s)	75% - 100% Depending on the damage type and the repair methods. Will still need to enhance structure or change load path
			Demolish and rebuild, OR	100%
			Remove and use alternative structure	0%

* Rocking of piled foundations will generally be assessed as a primary failure in the foundation, and should be assessed with a geotechnical engineer, separately to this, unless there is no tension connection to the pile

9.6 REPAIR AND STRENGTHENING STRATEGIES

Repair techniques for reinforced concrete shear walls are well documented. 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 9-7: 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, 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.	Has significant noise and vibration, 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 wall with reinforce concrete or structural steel	No demolition of wall 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.	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 wall	No demolition of wall required, therefore minimises	Can be difficult to achieve required bond if multiple	Will restore some of initial (pre-cracked) stiffness,	Provided that there are no significant issues with strain

Repair type	Advantages	Disadvantages	SLS performance	ULS Performance
with FRP	propping and shoring requirements. Bond generally easier to achieve than RC overlay or structural steel. No appreciable change to elastic stiffness of wall.	layers required. Only applicable to shear strength enhancement, not suitable for flexural enhancement. May require epoxy injection first.	but not all, to extent of epoxy repairs.	hardening, will restore most of initial strength to extent achievable through epoxy. Note that this can be used mainly only to increase shear capacity, to ensure capacity design objectives achieved.
Selective weakening e.g. 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.

9.7 REFERENCES

New Zealand Society for Earthquake Engineering, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, June 2006
 Federal Emergency Management Agency, *FEMA 306 Evaluation of Earthquake Damaged Concrete and Masonry Buildings – Basic Procedures Manual*, 1998
 Kelly, TE, *Tentative Seismic Design Guidelines For Rocking Structures*, SESOC Journal Vol 24 No. 1, 2011

APPENDIX 9A – WORKED EXAMPLES

9A.1 CANTILEVER CONCRETE WALL BUILDING

CONTENTS

9A.1	CANTILEVER CONCRETE WALL BUILDING	1
9A.1.1	INTRODUCTION	2
9A.1.2	DESCRIPTION	2
9A.1.3	DISPLACEMENT BASED ASSESSMENT	2
9A.1.4	BASE SHEAR & LOAD DISTRIBUTION TO AS/NZS1170.5.....	10

TABLES

Table 1: Wall Flexural and Shear Capacities	3
Table 2: Yield Curvature, Yield Displacement and Interstorey Drift by Wall	4
Table 3: Drift Calculation according to assumed wall ductility	5
Table 4: %NBS Calculations	8
Table 5: Wall Rotations and Crack Widths from Feb 22nd	9
Table 6: Base Shear and Static Load Distribution.....	10
Table 7: Wall Relative Stiffnesses and Locations	12
Table 8: Wall Load Translational and Torsional Distribution Factors	13
Table 9: Wall Load distribution factors for zero eccentricity	14
Table 10: Total Load Distribution.....	14

FIGURES

Figure 1: Typical Floor Plan.....	2
Figure 2: Force-displacement relationships for x direction.....	7
Figure 3: Force-displacement relationships for y direction.....	7
Figure 4: Wall Load distribution factors for zero eccentricity.....	14
Figure 5: Total Load Distribution	14

9A.1.1 INTRODUCTION

This section presents an example based on a real building, illustrating the use of the guidelines to analyse a reinforced concrete cantilever wall building.

9A.1.2 DESCRIPTION

The building is a reinforced concrete shear wall building of 13 levels. Refer to the plan below for the typical floor layout and shear wall location.

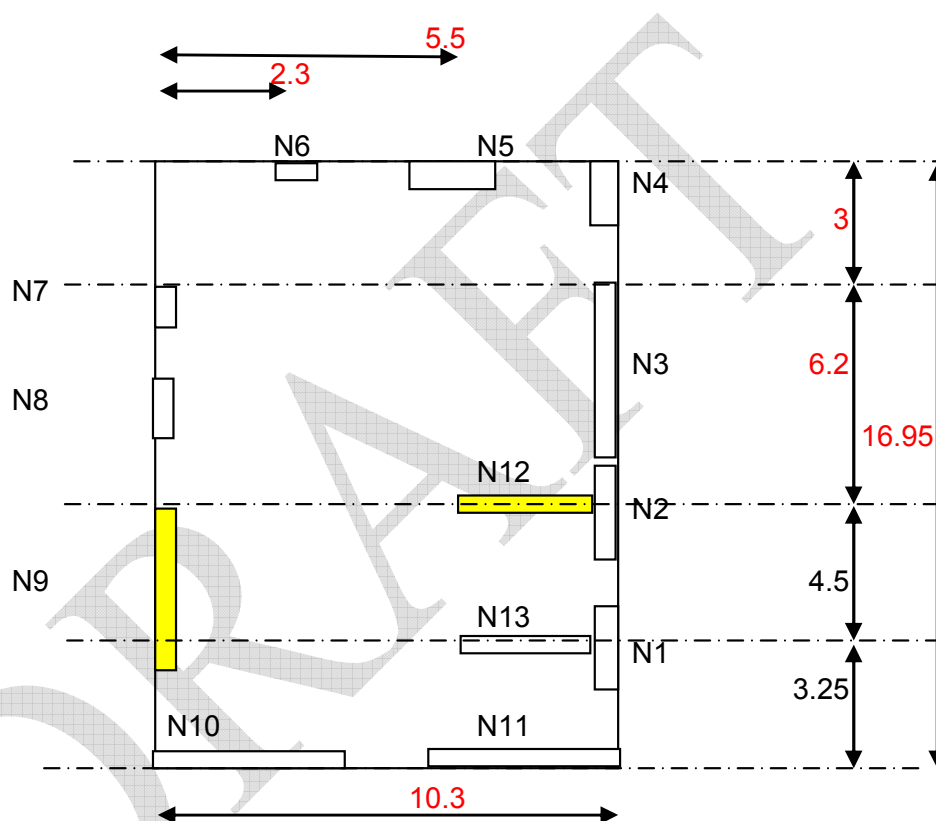


Figure 1: Typical Floor Plan

Notes:

1. All dimensions in metres
2. Walls highlighted in yellow are walls that have had visible damage to reinforcement.
3. Floor area = 174.6m^2 . Floors are concrete rib and timber infill with concrete topping.
4. Cladding otherwise lightweight.
5. Floor to floor typically 3m (except lower level, ignored for this example)

9A.1.3 DISPLACEMENT BASED ASSESSMENT

This assessment follows the steps set out in section 9.5.1 of the Reinforced Concrete Wall section. Note that this analysis has been completed mainly using a spreadsheet, i.e. it lends itself to automation.

Step 1: Probable Strength of Building

The probable strengths of the walls have been calculated, assuming:

- The expected mean yield stress is 1.08 times the lower characteristic yield stress,
- The concrete compressive strength is 1.5 times the nominal concrete strength
- Strength reduction factors ϕ , may be taken as 1.0 for flexure, 0.85 for shear

These strengths have been calculated approximately using the axial loads to the walls and assuming that the total area of longitudinal steel in the wall is lumped at the middle of the wall. From this analysis, the following strengths were calculated:

Table 1: Wall Flexural and Shear Capacities

1	2	3	4	5	6	7	8
Wall	Direction	Vw,prob	Mw,prob	Mw,prob,x	Mw,prob,y	Mw,prob/heff	
N1	y	585	1951		1951	75	ok
N2	y	2003	3835		3835	147	ok
N3	y	1471	8306		8306	319	ok
N4	y	473	771		771	30	ok
N5	x	849	2540	2540		98	ok
N6	o		0				
N7	o		0				
N8	y	582	2634		2634	101	ok
N9	y	1478	8877		8877	341	ok
N10	x	1305	7774	7774		299	ok
N11	x	1193	6424	6424		247	ok
N12	x	1121	6560	6560		252	ok
N13	x	1121	6560	6560		252	ok
			$\Sigma Mw,prob$	29858	26374		

Note that walls N6 and N7 have been excluded as their relative stiffness and hence contribution is too little.

Step 2: Base Shear Capacity

From equation 9(1), $V_{prob} = \frac{\Sigma M_{wprob}}{h_{eff}}$

Therefore, total base shear capacity of the building:

Vprobx	Vproby
1148	1014

Note that these values are, in the worst case, approximately 25% of the calculated static base shear capacity in accordance with AS/NZS1170.5 – refer below.

Step 3: Check that suitable mechanisms can develop

Check equation 9(2), for each wall, require: $\frac{M_{wprob}}{h_{eff}} < V_{wprob}$,

Where V_{wprob} is the capacity calculated in Step 1, from the reinforcement in the wall and the assumed concrete strength.

In this case, refer to column 8 of Table 1, showing that all walls satisfy this inequality. This verifies that a flexural mechanism can develop. If this were not the case, this displacement based assessment would be invalid and the building would need to be evaluated against static load distribution of AS/NZS1170.5, using $\mu=1$, $S_p=1$.

Step 4: Check yield displacement of walls

This is a simple geometrical relationship based in the yield strain of the reinforcement and the length of the walls.

Applying equations 9(3), for yield curvature ϕ_{wy} , 9(4), for yield displacement (at height h_{eff}) & 9(5) for interstorey drift δ_{wy} above the effective height, gives:

Table 2: Yield Curvature, Yield Displacement and Interstorey Drift by Wall

1	2	3	4	5	6	7	8
	Lw	dirn	ϕ_{wy} =1.8 ϵ_y /lw	Are =heff/lw	Uwy =0.6 ϵ_y .Are.heff	δ_{wy} =0.9 ϵ_y Are	$\delta_{wy}<2.5\%$?
N1	2.6	y	0.001731	10.00	0.390	0.023	ok
N2	3.5	y	0.001286	7.43	0.290	0.017	ok
N3	4.3	y	0.001047	6.05	0.236	0.014	ok
N4	1.2	y	0.00375	21.67	0.845	0.049	ng
N5	1.45	x	0.003103	17.93	0.699	0.040	ng
N6		o					
N7		o					
N8	2	y	0.00225	13.00	0.507	0.029	ng
N9	4.55	y	0.000989	5.71	0.223	0.013	ok
N10	4.35	x	0.001034	5.98	0.233	0.013	ok
N11	3.95	x	0.001139	6.58	0.257	0.015	ok
N12	3.68	x	0.001223	7.07	0.276	0.016	ok
N13	3.68	x	0.001223	7.07	0.276	0.016	ok

Note that in this case, three of the walls exceed the maximum drift of 2.5% before reaching yield. This may in itself be enough to either exclude these walls from the analysis, or to conclude that the wall system has insufficient capacity, but in this case, the analysis has been continued.

Step 5: Check limiting ductility according to drift limits

In this step, the limiting ductility of each wall is calculated.

The inelastic portion of the drift is calculated from an assumption of the system ductility, μ_{wc} . In this case the assumed value was $\mu_{wc} = 1.25$. This value can be assumed initially and revised with iteration though the process noted in the following steps. The plastic hinge length is using equation 9(9). From this the maximum drifts can be calculated and compared to the limiting values (column 4, 5 below).

Table 3: Drift Calculation according to assumed wall ductility

1	2	3	4	5	6	7
Wall	U_{wp} $=(\mu_{wc}-1)U_{wy}$	L_p $=0.5L_w$	δw_p $U_{wp}/(\mu_{wc}-0.5L_p)$	δw_{max}		μ_{wc}
N1	0.0975	1.3	0.0038	0.0263	ng	2.56
N2	0.0724	1.75	0.0029	0.0196	ok	3.08
N3	0.0590	2.15	0.0024	0.0160	ok	3.54
N4	0.2113	0.6	0.0082	0.0570	ng	1.73
N5	0.1748	0.725	0.0068	0.0472	ng	1.88
N6						
N7						
N8	0.1268	1	0.0050	0.0342	ng	2.21
N9	0.0557	2.275	0.0022	0.0151	ok	3.68
N10	0.0583	2.175	0.0023	0.0158	ok	3.56
N11	0.0642	1.975	0.0026	0.0174	ok	3.34
N12	0.0689	1.84	0.0027	0.0186	ok	3.18
N13	0.0689	1.84	0.0027	0.0186	ok	3.18

Note that column 6 indicates limiting ductility according to equation 9(10). In this case, it is indicated that the ductility is limited by wall N5. As noted in the Red Book, it is only necessary to complete this check for the longest wall in any one direction, but as this work was completed in a spreadsheet, it was trivial to extend to all walls. This represents the maximum ductility demand for any one wall, given the overall assumption of system ductility that was used to derive the inelastic drifts.

Step 6: Determine Maximum drift according to available damping

In building x direction:

Est. Usc	340.4652	μ		1.25	Usy	272.4	Goal seek, μ_s to μ , by changing Usc	
		T=		2.90	μ_s	1.25		
col offset	9	8	7	6	5	4	3	2
μ	1	1.25	1.5	2	3	4	5	6.000
T	2	2.5	4.5	4.5	4.5	4.5	4.5	4.5
D(T)	318.7362	296.2087	273.4578	168.900387	112.60026	84.450193	67.56015468	56.30013
row	13	14	18	18	18	18	18	18
row+1	14	15	19	19	19	19	19	19
T(row+1)	2.5	3	5	5	5	5	5	5
	3	4	5	6	7	8	9	10
D(T+)	400.282	352.1439	0	0	0	0	0	0
Period (T)	2.133232	2.895605	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

And in building y direction:

Est Usc	331.6511							
		μ		1.25		U _{sy}	265.3	
		T=		2.82		μ_s	1.25	Goal seek, μ_s to μ , by changing Usc
col offset	9	8	7	6	5	4	3	2
μ	1	1.25	1.5	2	3	4	5	6.000
T	2	2.5	4.5	4.5	4.5	4.5	4.5	4.5
D(T)	318.7362	296.2087	273.4578	168.900387	112.60026	84.450193	67.56015468	56.30013
row	13	14	18	18	18	18	18	18
row+1	14	15	19	19	19	19	19	19
T(row+1)	2.5	3	5	5	5	5	5	5
	3	4	5	6	7	8	9	10
D(T+)	400.282	352.1439	0	0	0	0	0	0
Period (T)	2.079188	2.816817	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A

Note that these calculations have been completed using displacement spectra taken from the acceleration spectra for the site, as follows:

1. The displacement spectra can be developed for different ductilities from the relationship:

$$\delta(T) = 9800 \frac{C(T) \times T^2}{4\pi^2}$$

2. A displacement U_{sc} is initially assumed, for which a lookup function can be used to ascertain the related building period to achieve that displacement, for different levels of ductility. In the calculation above, the function looks up the discrete period above and below the assumed displacement and interpolates to calculate the period for the displacement assumed.
3. A goalseek function can then be used to determine the displacement at a particular ductility

μ_s is the overall system ductility that can be achieved, corresponding to U_{sc} .

From this information, the force-displacement relationships for all walls can be plotted, corresponding to the different directions of loading. It can be seen from both Figure 2 and Figure 3 below that the most flexible walls do not develop their full capacity at this displacement, but it also clear that the contribution of these walls is negligible to the overall capacity.

Step 7: Calculate the Equivalent Viscous Damping

The equivalent damping is then calculated from equation 9(12). In this case, the damping equating to $\mu=1.25$, is:

$$\xi_{eff} = 8.19\%$$

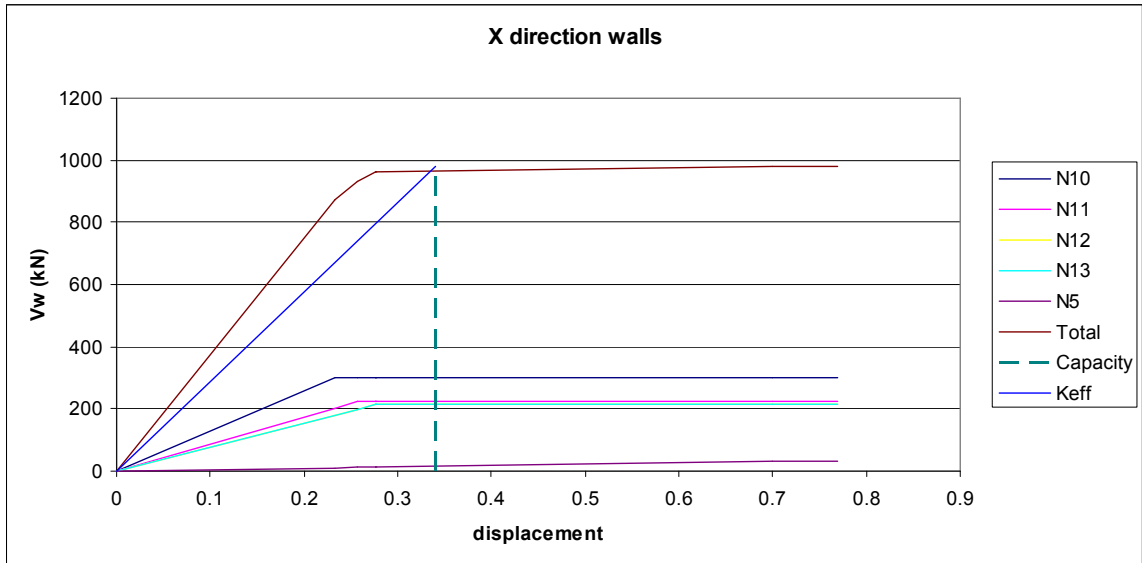


Figure 2: Force-displacement relationships for x direction

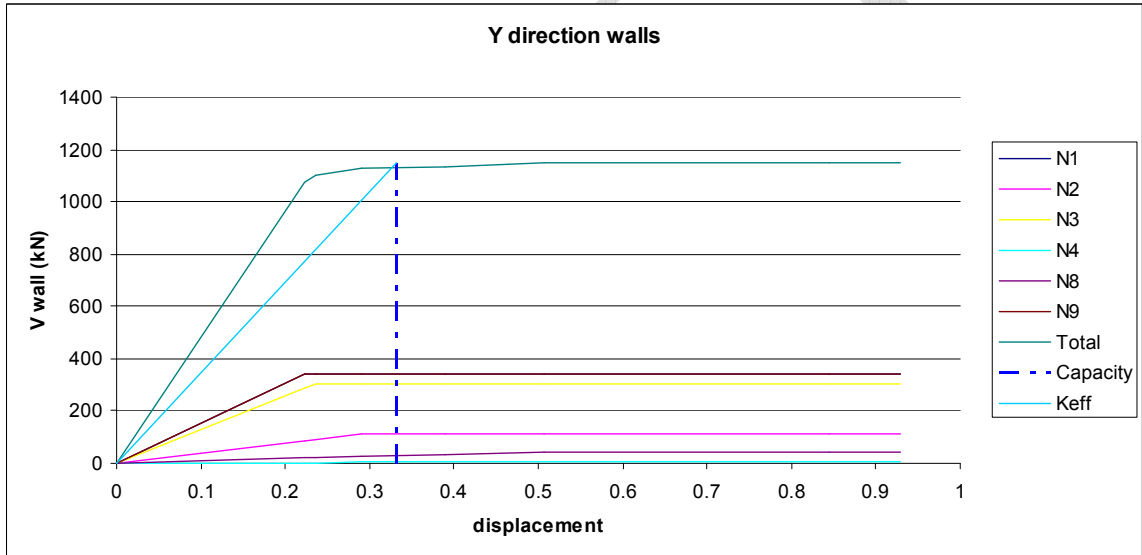


Figure 3: Force-displacement relationships for y direction

Step 8: Calculate the Drift demand

The effective period is calculated from the effective stiffness, which is a function of the Effective Mass and the effective stiffness, calculated from equations 9(14) and 9(15) respectively.

From equation 9(14), effective mass, $M_e = \frac{\sum m_i h_i}{h_e} = \frac{435599}{26} = 17754kN$

(Note value for $\sum m_i h_i$ from Table 6 below.)

And, from equation 9(15) $k_{eff,x} = 2887kN/m$
 $k_{eff,y} = 3248kN/m$

Hence the effective period from equation 9(13), $T_{eff} = 2\pi \sqrt{\frac{M_e}{g.k_{eff}}}$

Gives: $T_{eff,x} = 4.83 \text{ sec}$
 $T_{eff,y} = 4.56 \text{ sec}$

For each of these, the spectral displacement is 400mm at the target ductility.

Step 9: Calculate the overall %NBS

Now, the capacity expressed as %NBS comes from equations 9(16)a, $\%NBS = \frac{U_{sh}}{U_D}$

Hence the capacity in this case is the lesser of the two directions, as below:

Table 4: %NBS Calculations

	U_{sc}	U_D	%NBS
x	340.4652	400	85%
y	331.6511	400	83%

Note however that this figure is adjusted later by consideration of the extent of damage – refer Step 11 below.

Step 10: Complete subjective checks against critical Code requirements

Notwithstanding the above, the building needs also to be assessed for detailing against the code, specific to the level of ductility assumed, including for:

1. Dimensional limitations
2. Transverse confinement in potential plastic hinge regions
3. Transverse reinforcement in boundary regions
4. Anchorage

Step 11: Calculate flexural crack widths

From equation 9(17), the effective crack widths can be calculated.

The rotation must first be calculated by considering the effective post-yield rotation of the walls, to equation 9(16), using the yield displacement from Step 4 above. Note that the displacement demand in this case is the demand from the Average Displacement Spectra from Feb 22, using the effective damping calculated in Step 7. From this rotation, the maximum total peak crack width can be calculated for each wall, using equation 9(17).

The number of cracks over the plastic hinge length is then counted, and the total crack width is distributed according to equation 9(18). This is all tabulated for the example in Table 5 below.

Step 12: Complete assessment of the impact of the cracks

With reference to Table 9.2 – *Failure Modes and Repair Assessment – Flexure*, damage levels are categorised according to crack widths. Depending on whether the crack widths are then categorised as *minor*, *moderate* or *severe*, alternative capacity reduction and/or repair measures are recommended. In this case, column 10 shows the crack widths, with the colour coding indicating the severity of the damage, in this case with green indicating minor damage, orange indicating moderate damage.

Note that walls N6 and N& are not included in the analysis, as they provide no meaningful contribution to the overall building capacity. Also that in this case, walls N4 and N5 are not assessed to have any significant yielding in the reinforcement, as the calculated building displacement at the effective height is less than the wall yield displacement.

Table 5: Wall Rotations and Crack Widths from Feb 22nd

1	2	3	4	5	6	7	8	9	10
Wall	Lw (m)	direction	c (m)	U _{sL} (mm)	U _{wy} (m)	θ _{wlp}	W _{tot}	n _c	W _{lp,max}
N1	2.6	y	0.124	402.4	0.3900	0.00048	0.19	1	0.2
N2	3.5	y	0.138	402.4	0.2897	0.00434	1.29	1	1.3
N3	4.3	y	0.247	402.4	0.2358	0.00641	1.58	1	1.6
N4	1.2	y	0.083	402.4	0.8450	0.00000	0.00		0.0
N5	1.45	x	0.152	334.5	0.6993	0.00000	0.00		0.0
N6	0.8	o	0.000			0.00000	0.00		0.0
N7	1	o	0.000			0.00000	0.00		0.0
N8	2	y	0.173	402.4	0.5070	0.00000	0.00		0.0
N9	4.55	y	0.249	402.4	0.2229	0.00691	1.61	2	1.1
N10	4.35	x	0.300	334.5	0.2331	0.00390	0.96	1	1.0
N11	3.95	x	0.273	334.5	0.2567	0.00299	0.81	1	0.8
N12	3.68	x	0.303	334.5	0.2755	0.00227	0.67	1	0.7
N13	3.68	x	0.303	334.5	0.2755	0.00227	0.67	1	0.7

From this information, the future demolition or repair strategy can be considered (subject to other damage and assessment. For example in this case, due to the flexibility of the system, the floors and secondary elements should clearly be reviewed more closely).

With respect to the walls in this example, it would be considered that walls N1, N4, N5, N6, N7 and N8 are unlikely to require further assessment, subject to confirmation that the cracks are no wider than analysis would indicate. These walls are assessed to have 90% of the capacity that they had prior to the earthquakes, but may be restored to 100% by full epoxy grouting.

Walls N2, N3, N9, N10, N11, N12 and N13 require further review. The recommended review is to expose reinforcement at the crack and perform testing to confirm the extent of strain hardening that has occurred (either through destructive testing or Liebh hardness testing or similar NDT, provided that some calibration of the equipment has been completed). These walls are assessed as having only 50% of the capacity that they had prior to the earthquake, but subject to the findings of the testing, may be restored to up to 100% of that capacity by epoxy grouting. Alternatively, a separate load path should be sought, with supplementary structure.

Note that, with reference to Figure 1 above, walls N9 and N12 suffered the most obvious damage on site, with fracture of the reinforcement at a single crack. This

analysis confirms that this could be the case. Note that for a 16mm bar (as used for flexural steel in W12, if the yield penetration was limited to 0.5db either side of the crack, a 0.7mm crack equates to 4% total elongation, nominally 20% of capacity. This indicates an even higher inelastic drift for this structure. But for the largest crack width of 1.6mm, this equates to 10% elongation, i.e. 50% of the total nominal capacity lost.

9A.1.4 BASE SHEAR & LOAD DISTRIBUTION TO AS/NZS1170.5

(For comparative purposes, and for insertion at the appropriate points above)

i) Building Weigh-up and Seismic load distribution

Calculated floor loads are:

$$\begin{aligned} p_g &= 8.39 \text{ kPa} \\ \psi_s p_q &= 0.75 \text{ kPa} \\ p_{g\&qr} &= 9.14 \text{ kPa} \end{aligned}$$

Therefore, total seismic mass:

$$W_t = 1596 \text{ kN per floor}$$

Seismic load

$$\text{For } T(\text{est}) = 2.9 \text{ sec, } \mu = 1.25, \text{ Type D subsoils, } \therefore C(\mu, T) = 0.191$$

(Note that period was ascertained by iteration for effective stiffness)

Seismic Mass distribution over building height (for $C=0.191$):

Table 6: Base Shear and Static Load Distribution

Level	W (kN)	delta h (m)	h (m)	Wh	Wh ²	Fx (kN)	Sum F (kN)	M (kNm)	mihi
13	1596	3	39	62228	2426909	1069	1069	3206	62228.439
12	1596	3	36	57442	2067899	641	1709	8334	57441.636
11	1596	3	33	52655	1737609	538	2248	15076	52654.833
10	1596	3	30	47868	1436041	445	2692	23153	47868.03
9	1596	3	27	43081	1163193	360	3053	32311	43081.227
8	1596	3	24	38294	919066	285	3337	42324	38294.424
7	1596	3	21	33508	703660	218	3555	52990	33507.621
6	1596	3	18	28721	516975	160	3716	64136	28720.818
5	1596	3	15	23934	359010	111	3827	75617	23934.015
4	1596	3	12	19147	229767	71	3898	87310	19147.212
3	1596	3	9	14360	129244	40	3938	99124	14360.409
2	1596	3	6	9574	57442	18	3956	110992	9573.606
1	1596	3	3	4787	14360	4	3960	122872	4786.803
Sum	20743		0	435599	11761175				435599.07

Further, the height to the effective centre of gravity $h_{\text{eff}} = \frac{2}{3} h = 26\text{m}$

ii) Load Distribution to walls

Distribute load to walls in accordance with elastic stiffness (initially), based on I^2 , adjusting also for width, t .

- Refer to Table 7: Wall Relative Stiffnesses and Locations for a spreadsheet locating the wall centres of reaction and the directions of action. Note that a 'o' in the Direction column indicates that the wall has been discarded due to lack of effective contribution ; and
- Table 8: Wall Load Translational and Torsional Distribution Factors for distribution factors for the loads to the walls. These factors are calculated using a rivet group analogy according to the eccentricity of the walls from the centre of rigidity, and;
- Table 9: Wall Load distribution factors for zero eccentricity. This table combines the factors from Table 8, calculated using the eccentricity from the centre of mass to the centre of rigidity

Table 7: Wall Relative Stiffnesses and Locations

Wall	l	t	w	x	y	wx	wy	Direction	t.L ²
N1	2.6	0.25	16.25	10.3	3.25	167.4	52.8	y	1.690
N2	3.5	0.325	28.44	10.3	7.75	292.9	220.4	y	3.981
N3	4.3	0.325	34.94	10.3	11.8	359.9	412.3	y	6.009
N4	1.2	0.325	9.75	10.3	16.35	100.4	159.4	y	0.468
N5	1.45	0.5	18.13	5.5	16.95	99.7	307.2	x	1.051
N6	0.8	0.6	12.00	2.3	16.95	27.6	203.4	o	0.384
N7	1	0.325	8.13	0	13.45	0.0	109.3	o	0.325
N8	2	0.325	16.25	0	9.85	0.0	160.1	y	1.300
N9	4.55	0.325	36.97	0	5.225	0.0	193.2	y	6.728
N10	4.35	0.25	27.19	2.18	0	59.1	0.0	x	4.731
N11	3.95	0.25	24.69	8.33	0	205.5	0.0	x	3.901
N12	3.68	0.25	23.00	8.46	7.75	194.6	178.3	x	3.386
N13	3.68	0.25	23.00	8.46	3.25	194.6	74.8	x	3.386
			278.7			1701.7	2071.0		
		x,mean	6.11						
		y,mean	7.43						

Table 8: Wall Load Translational and Torsional Distribution Factors

Wall	x			$(t/L^2).ry^2$	y			$(t/L^2).rx^2$				
	(t/L^2)	$(t/L^2).y$	ry		(t/L^2)	$(t/L^2).rx$	ry		transx	transy	rotx	roty
N1					1.69	17.407	4.098349	28.38602		0.08376		0.007964
N2					3.98125	41.00688	4.098349	66.87092		0.197318		0.018761
N3					6.00925	61.89528	4.098349	100.9342		0.29783		0.028318
N4					0.468	4.8204	4.098349	7.860745		0.023195		0.002205
N5	1.05125	17.81869	13.60362	194.5428					0.063891		0.016443	
N6												
N7												
N8					1.3	0	-6.201651	49.99862		0.06443		-0.00927
N9					6.728313	0	-6.201651	258.7741		0.333468		-0.047978
N10	4.730625		0 -3.346377	52.97468					0.287511		-0.018202	
N11	3.900625		0 -3.346377	43.68014					0.237067		-0.015008	
N12	3.3856	26.2384	4.403623	65.65319					0.205765		0.017142	
N13	3.3856	11.0032	-0.096377	0.031447					0.205765		-0.000375	
	16.4537	55.06029		356.8823	20.17681	125.1296		512.8246	1	1	-3.74E-18	0

r.y

3.346

r.x

6.202

Table 9: Wall Load distribution factors for zero eccentricity

Wall	on cr	
	x=	y=
	5.65	7.93
	x	y
N1	0.036484	0.079371
N2	0.085948	0.18698
N3	0.129729	0.282226
N4	0.010103	0.02198
N5	0.139222	-0.009061
N6		
N7		
N8	-0.042468	0.069538
N9	-0.219798	0.359905
N10	0.204123	0.01003
N11	0.168309	0.00827
N12	0.284299	-0.009446
N13	0.204046	0.000207
	1	0
		2.11E-18
		1

Figure 4: Wall Load distribution factors for zero eccentricity

Hence for the Base shear load of $V=3960$ kN, the overall load distribution is:

Table 10: Total Load Distribution

Wall	x		y	
	Vxx	Vxy	Vyx	Vyy
N1		144		314
N2		340		740
N3		514		1118
N4		40		87
N5	551		-36	
N6				
N7				
N8		-168		275
N9		-870		1425
N10	808		40	
N11	667		33	
N12	1126		-37	
N13	808		1	
	3960	0	0	3960

Figure 5: Total Load Distribution

Note that the distribution of loads to the walls is now quite different, partly due to the Torsional resistance of the walls, and partly due to the different effective stiffness assumptions. Trying to include torsional actions would significantly complicate the displacement based analysis, but is felt not to be required in most cases, given the intent of the analysis (to verify damage).

Appendix 9B – Spectral Acceleration and Displacement Data

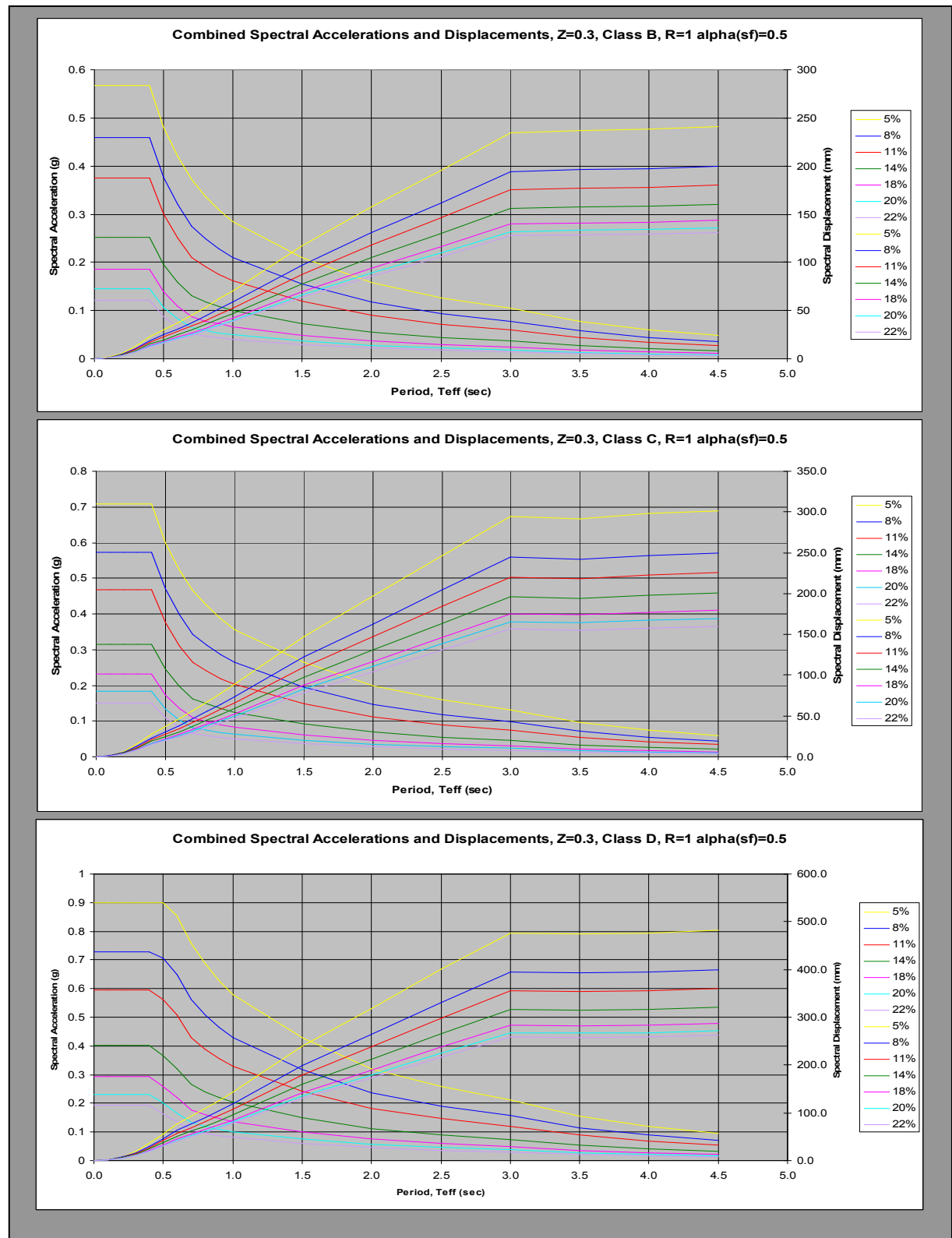


Figure 1: Acceleration and Displacement Spectra for R=1.0

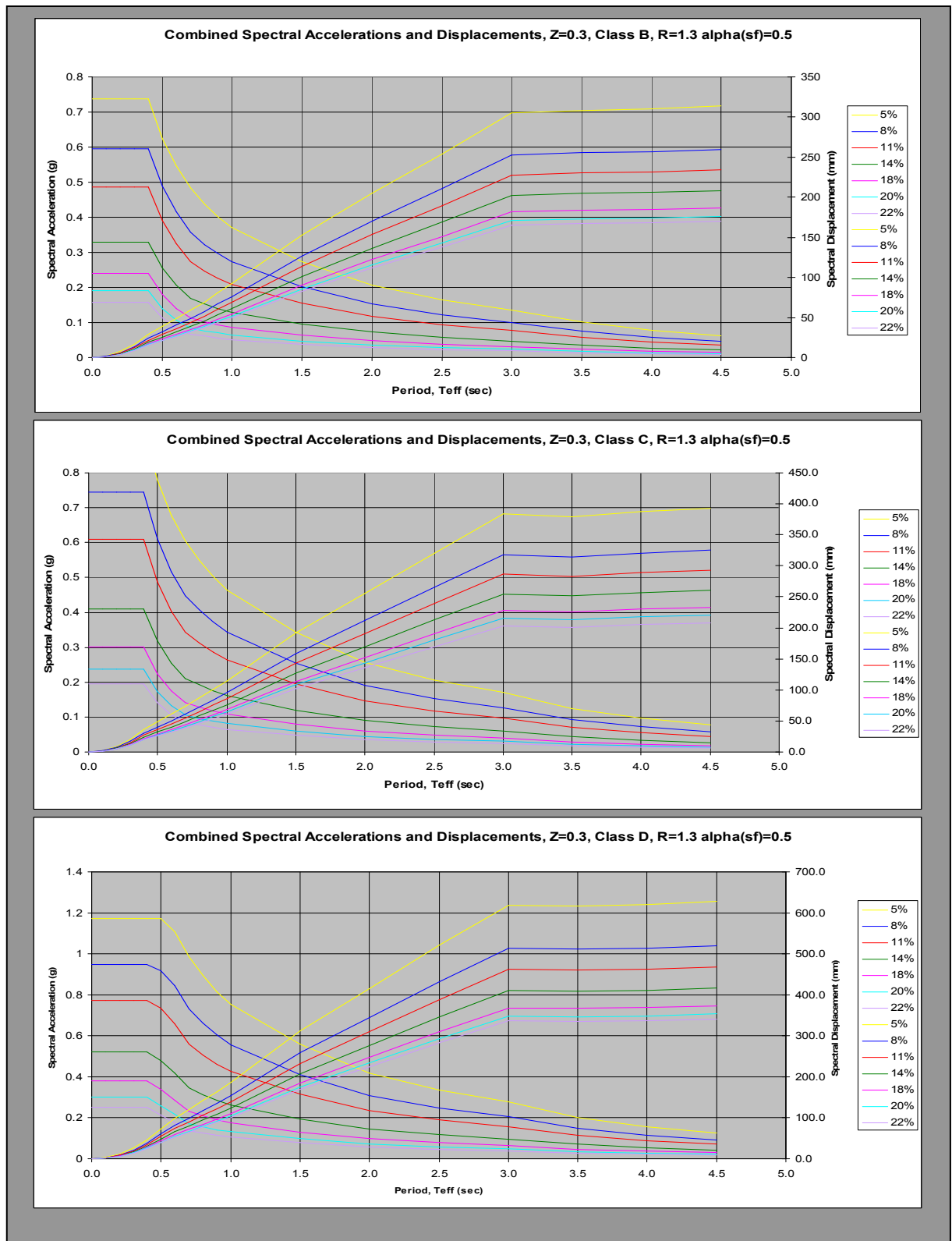


Figure 2: Acceleration and Displacement Spectra for $R=1.3$

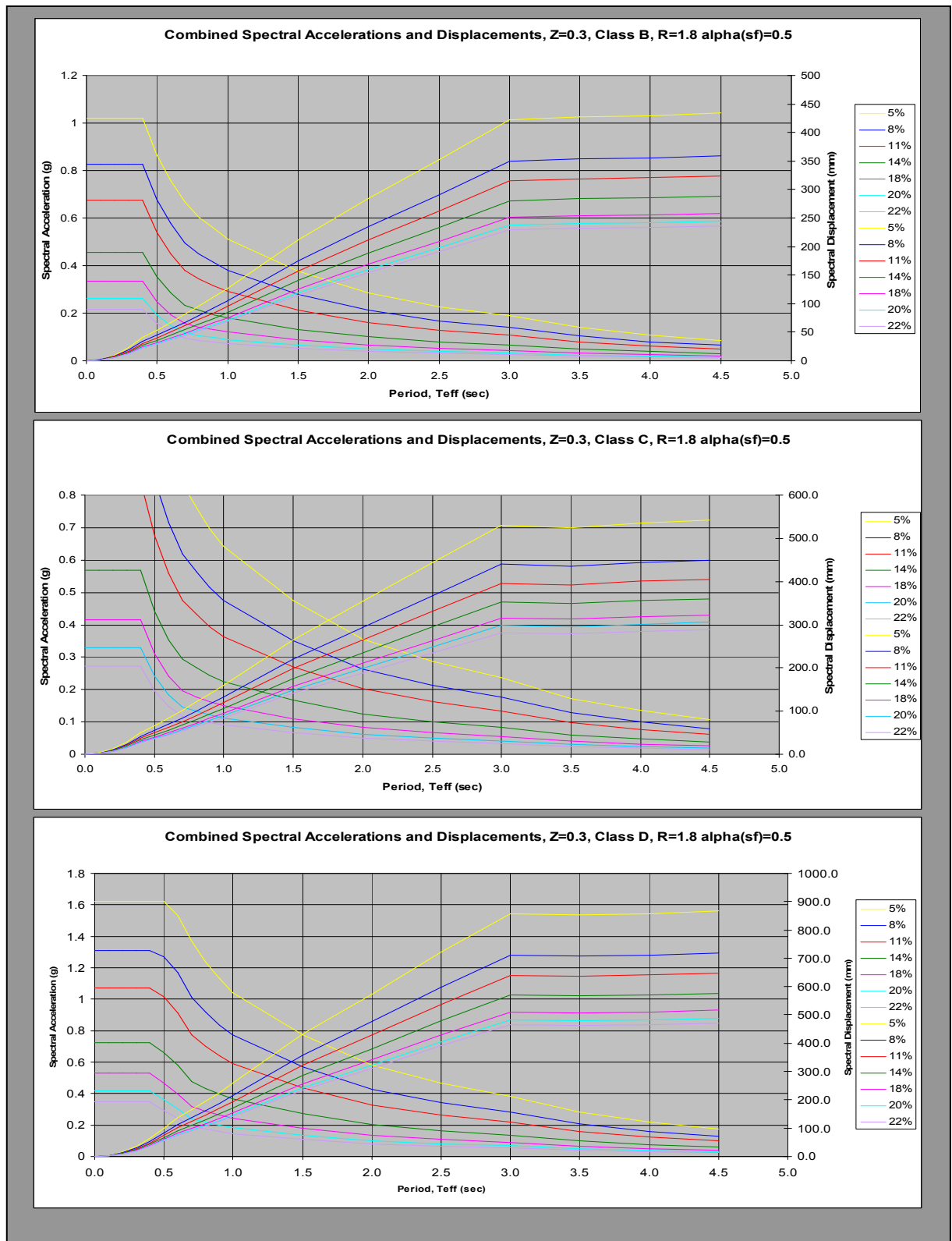


Figure 3: Acceleration and Displacement Spectra for $R=1.8$

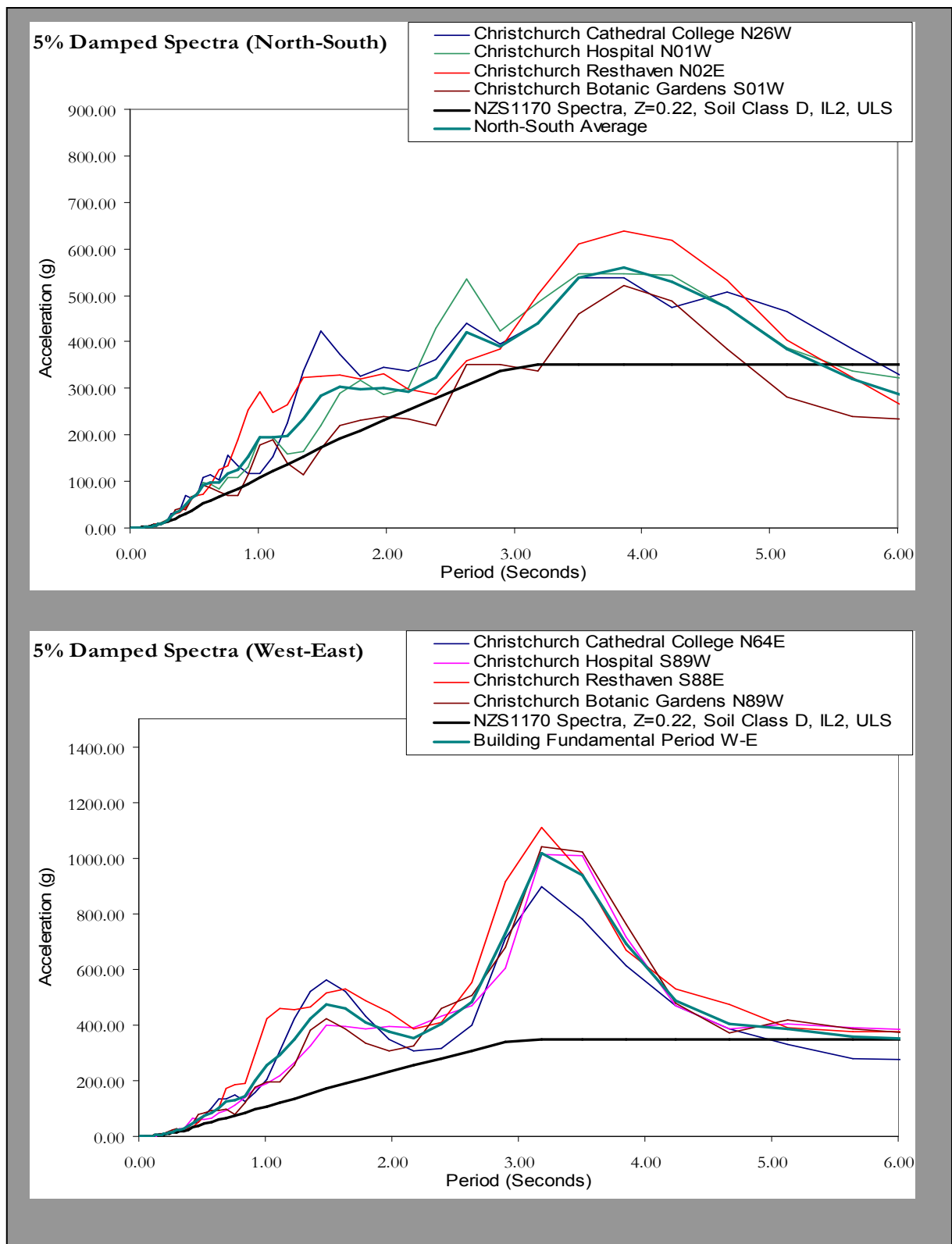


Figure 4: 5% Damped Displacement Spectra

APPENDIX 9C – SPECIFIC BUILDING TYPES

9B.1 COMMERCIAL LOW-RISE PANEL STRUCTURES

CONTENTS

9B.1	COMMERCIAL LOW-RISE PANEL STRUCTURES	1
9B.1.1	Introduction	2
9B.1.2	Seismic Response Characteristics And Common Deficiencies	2
9B.1.2.1	Precast Panel System behaviour	2
9B.1.2.2	Single-storey commercial applications	4
9B.1.3	Failure Mode And Repair Assessment	5
9B.1.3.1	In-plane and Out-of-plane loading	5
9B.1.3.2	Connection Capacity	7
9B.1.4	Reporting of Results	8

FIGURES

Figure 1: Vertical panel joints	3
Figure 2: Free body analysis of panel behaviour	4
Figure 3: Representative yield line pattern	5
Figure 4: Possible arrangement of repair elements	6
Figure 5: Tributary area to connections	8

9B.1.1 INTRODUCTION

A special form of concrete structural walls is precast panels, either connected to behave as composite wall sections, or as individual panels. Precast panels may be cast on-site or off-site according to the space available, the complexity of the panels, transportation considerations and contractor preference. Subject to the particular form of the structure, precast panels may behave in a similar fashion to insitu concrete wall systems, provided that the connections are detailed accordingly. There are however specific detailing considerations that must be met in order to achieve this.

A specific form of precast panel structure that is included in this section is tilt panel structures, which generally (but not always) are attached to steel portal frames.

9B.1.2 SEISMIC RESPONSE CHARACTERISTICS AND COMMON DEFICIENCIES

9B.1.2.1 Precast Panel System behaviour

As noted above, precast panels may behave in similar fashion to insitu concrete walls systems, provided that the detailing supports that. Reviewers should refer to NZS3101¹ section 18, for further guidance on the requirements for composite action of precast elements. It is essential, as the first stage of any assessment of concrete panels being used as shear walls, that there is a determination made as to whether the panels are able to achieve composite action, and if not, what configuration of walls should be assumed for analysis.

The primary determining factor in assessing composite performance is connection detailing. The connections between panels must not be the limiting factor in assessing the strength of the system if full composite action is to be assumed.

Issues to consider include:

- Base connections. Assuming a mechanical connection, it is noted that the maximum allowable ductility is $\mu=1.25$. Although it is possible that splices may be apparently staggered by using different lengths of ducting and splices, the necking effect created by confining a bar in a duct with high-strength grout will tend to cause all bars to yield at the same level. If present, this detail should be carefully inspected, noting that several instances of the failure of bars have been observed in such cases.
- Vertical joints between panels. In cases where the joints run over the full thickness and height of the panels, with all of the horizontal steel lapping through the joint and the ends of the panels adequately roughened; it is probably reasonable to assume full composite behaviour. However, in many cases, the exterior of the panels have been precast with a rebated joint on the

¹ Standards New Zealand NZS3101:2006 *Concrete Structures Standard: Part 1 – The Design of Concrete Structures*, SANZ

interior face of the panel, for architectural reasons. Refer to Figure 1: Vertical panel jointsFigure 1 below for further graphical representation of these situations. Case 1 is the former case, where the full section of the panel is jointed. Case 2 is a reduced section to provide a full precast surface face.

In case 2, even if the effective steel area lapping through the joint is greater than or equal to the area of horizontal reinforcement required in the panel, there is still a need to consider the shear stress on the reduced section. If the degree of necking is too great, the shear stress may be too high, leading to separation of the panels at the joints.

If the area of steel is less than the area of horizontal steel required, full composite connection is not ensured, leading to separation of the panels as above. In such a case, the behaviour may be bounded by considering the wall as either a single composite unit, or as a series of vertical cantilevering elements.

In all cases, the horizontal steel should be capable of transferring the shear force as in shear friction across the joint.

- Foundation compliance may be critical to the behaviour of the panels. The wall starters may have anchorages capable of developing the full tensile capacity of the bars, but this may be limited by the flexural or shear capacity of the foundation. Particularly when the panels are considered separately, this needs to be carefully considered, noting that it may be necessary to consider the panels and foundations as a series of free bodies with capacity limited by rocking. A diagrammatic example is given below in Figure 2. Note that the capacity of the foundation may be a limiting factor, depending on whether the foundation has sufficient strength to develop the capacity of the vertical reinforcement in the wall panels or (more likely) the foundation capacity in shear or flexure will limit the overall system capacity.

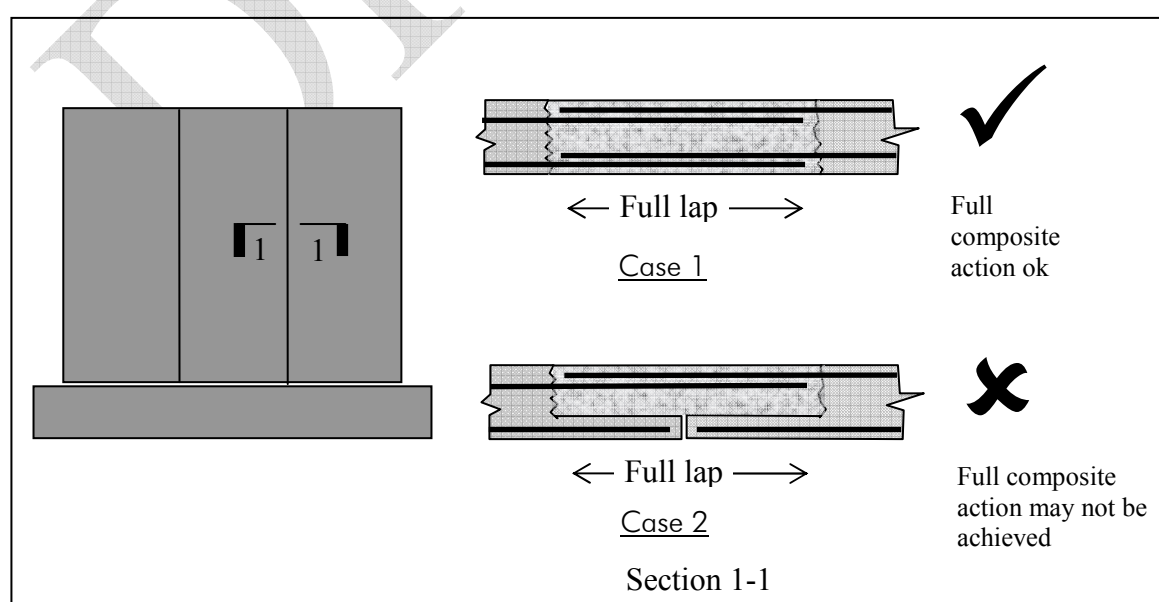


Figure 1: Vertical panel joints

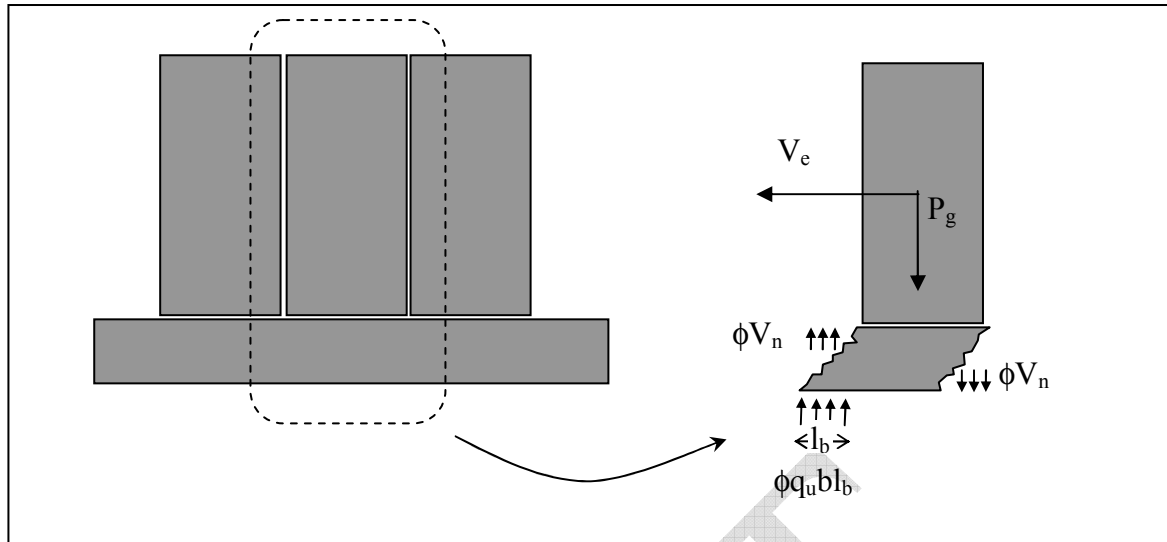


Figure 2: Free body analysis of panel behaviour

9B.1.2.2 Single-storey commercial applications

This section addresses primarily uses such as commercial warehouses and factories where the panels are used as a combination of cladding and in-plane structural walls, often fabricated as tilt panels. Although these are a subset of precast panels, there are a number of unique behavioural characteristics of low-height panel structures.

In general, these are thin panels, designed for elastic or nominally ductile response. The behavioural characteristics of these panels are determined by a number of factors, including:

- **Reinforcement.** Many earlier panels are reinforced with hard-drawn wire mesh. This steel is generally not capable of developing any significant strain beyond 'yield'. Furthermore, this reinforcement is often relatively light, reflecting the low demand at the time of design, which was often governed by lifting considerations for tilt-panels. However, as the assessed demand under face-loading may now be significantly greater than when the panel was designed, it is likely in many cases that the panels have inadequate reinforcement to resist even 33% of current code demand in order to satisfy Earthquake Prone Building criteria. In many cases, it is possible that the flexural capacity of the panel reinforcement may be less than the cracking moment of the panels.
- **Connections.** Many early panel structures used weld-plate connectors. These are often brittle, and have no allowance for shrinkage over the length of the structure. Where multi-bay structures contain panels over a significant length, it was common even prior to the earthquakes to see cracked connections at reasonably frequent intervals, as a result of shrinkage.

Consideration also needs to be given to the behaviour of panel connections in fire, according to factors such as the proximity to the boundary and the spread of fire requirements to adjacent structures.

- Foundations. There are a variety of different foundation conditions for tilt-panels. A common condition is to have the panels erected onto the portal foundation pads, (with intermediate levelling pads if required) and to then cast the floor slab against the panels. Thickened foundations are often used to satisfy after-fire considerations.
- Out-of-plane support. Many tilt-panel structures have no additional support at the eaves junction of the panels, apart from connections at the edges of the panels at the portal knee. Therefore, the connection detail may determine the critical failure mechanism of the panel.
- Stiffness compatibility. At the ends of portal frame structures, the last frame may be a braced frame, a full portal, or a series of panels. The stiffness of the end frame may have a significant impact on the behaviour of the last panel. If there is a significant stiffness differential between the end wall and the first portal, there will be warping actions on the panel as well as regular loading. This may also result in increased stresses on the connections.

9B.1.3 FAILURE MODE AND REPAIR ASSESSMENT

Several common failure characteristics have been observed in tilt panel structures which require addressing.

9B.1.3.1 In-plane and Out-of-plane loading

Failure mode 1: Full yield line development under face loading. Note that the yield line pattern will be dependent on the aspect ratio and support conditions of the panels.

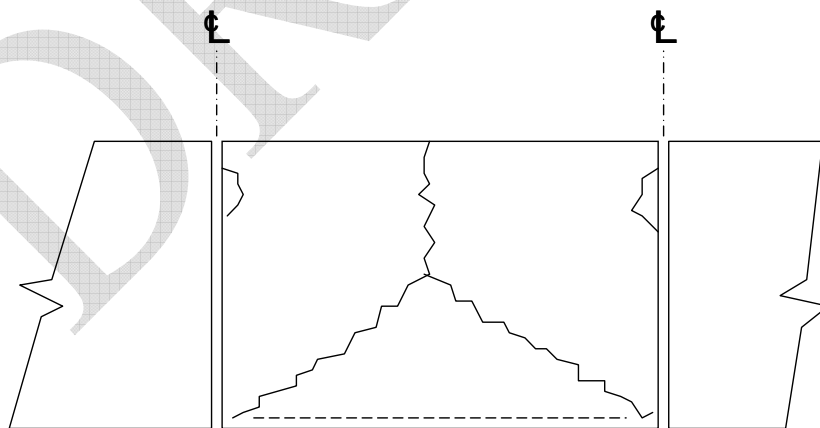


Figure 3 Representative yield line pattern

There are two possible variations to consider, according to the reinforcement in the panel.

- 1a). Firstly, if the panel is reinforced with mesh (excluding trimmers). In this case, it is possible that the mesh may be fractured, or on the point of fracture. Non-destructive bar testing, as has been completed for many buildings in

Christchurch, is ineffective, as cold-drawn mesh has no particular yield plateau, and hence no predictable point of failure. Therefore in such cases, an alternative load path is required for out-of plane loading. This may require the addition of one or more horizontal or vertical support members, for example at eaves level and at mid-height (as shown in Figure 4 above). Care needs to be taken to ensure that fire requirements are satisfied.

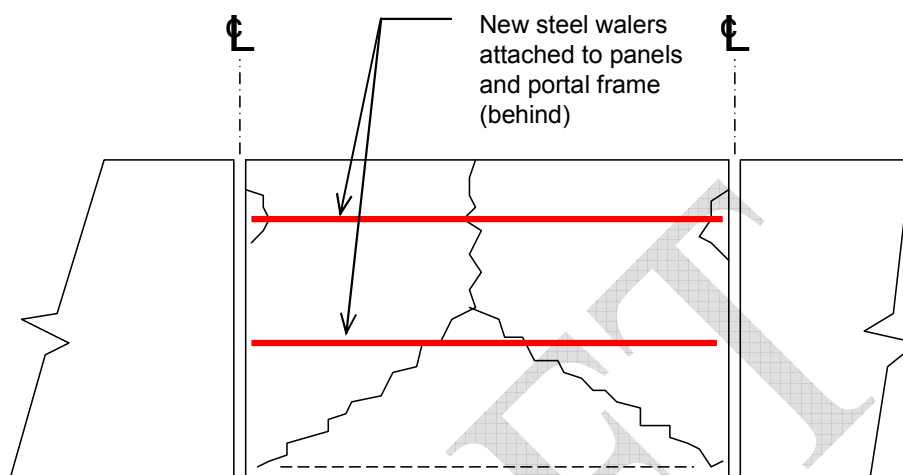


Figure 4: Possible arrangement of repair elements

The design of these elements must be in accordance with the Building Code. Connection of the panels to the new elements should reflect both the demand caused by the face loading and the required shear transfer to satisfy in-plane loading. This must also continue through to the consideration of the connection of the introduced elements back to the supporting members.

The repair to the panel itself is dependent on the in-plane loading demand and taking into account the connections of any introduced strengthening members, which may change the load path. Assuming that the panels are assessed for nominally ductile ($\mu=1.25$) loading, provided that the concrete strength is greater than or equal to the demand,

i.e. $\phi V_c \geq V_e$, (assuming $f'c = 25$ MPa, unless the original specification is available or testing is conducted to show otherwise.)

then the reinforcement is not required to resist seismic loading, and the panel may be repaired for in-plane loading by epoxy injection and/or coating of the panel (in order to restore weatherproofing) if the cracks are not wide enough to epoxy.

If the concrete strength is less than the demand, then the capacity of the panels can be calculated as:

$$\%NBS = \frac{\phi V_c}{V_e}, \text{ excluding the reinforcement.}$$

If required, the panels may be replaced or strengthened. If the panel is not to be replaced, the panel may be repaired as above, using epoxy injection or coating.

OR

- 1b). Secondly, if the panel is reinforced with conventional mild steel. In this case, the reinforcement should be able to continue to sustain the panel flexural and shear capacity, provided that the rotation is not excessive. Therefore the capacity of the panel can be calculated as normal, using conventional yield line theory, and compared to the demand, based on the support conditions and the required face loading according to NZS1170.5, section 8 (Requirements for Parts and Components).

The in-plane loading demand may be treated as above, but the mild steel reinforcement may be assumed to contribute fully to the capacity of the panel. Assuming that the panels are assessed for nominally ductile ($\mu=1.25$) loading, the capacity of the panels may be calculated as:

$$\%NBS = \frac{\phi(V_c + V_s)}{V_e},$$

As above, the panel may be repaired for in-plane loading by epoxy injection and/or coating of the panel (in order to restore weatherproofing).

The limiting capacity of the panels should be reported as the lesser of the in-plane or out-of-plane capacity.

Failure mode 2: Only partial yield line development under face loading – less than 50% of the full panel yield line mechanism in evidence. In this case, although it is possible that there is undetectable cracking, it is reasonable to assume that the reinforcement has not been subjected to excessive strain. The capacity of the panel in both in-plane and out-of-plane loading may be calculated conventionally, including the contribution of all reinforcement, whether it is mild steel or hard-drawn wire mesh.

9B.1.3.2 Connection Capacity

Demand on the connections for face-loading may be calculated using the appropriate coefficient from the Parts and Connections section of NZS1170.5. Assuming that the panels are simply supported at the base, the tributary area of panel contributing to the loading may be calculated as

$$h_e = \frac{h_p^2}{2h_f}, \text{ where}$$

h_p = the height of the panel

h_f = the height to the fixing (assumed to be at eaves level)

h_e = the effective height of the panel tributary to the fixing

Refer to Figure 5 below for graphic representation of the definitions above for a portal frame structure. In this case, the tributary weight of the panels on either side of the building, W_p , is

$$W_p = \gamma_{conc} h_e s t, \text{ where}$$

γ_{conc} = concrete density
 s = portal frame spacing,
 t = panel thickness

And the connection design load, F_{ph} , is

$$F_{ph} = C_p (T_p) C_{ph} R_p W_p, \text{ per equation 8.5(1) of NZS1170.5}$$

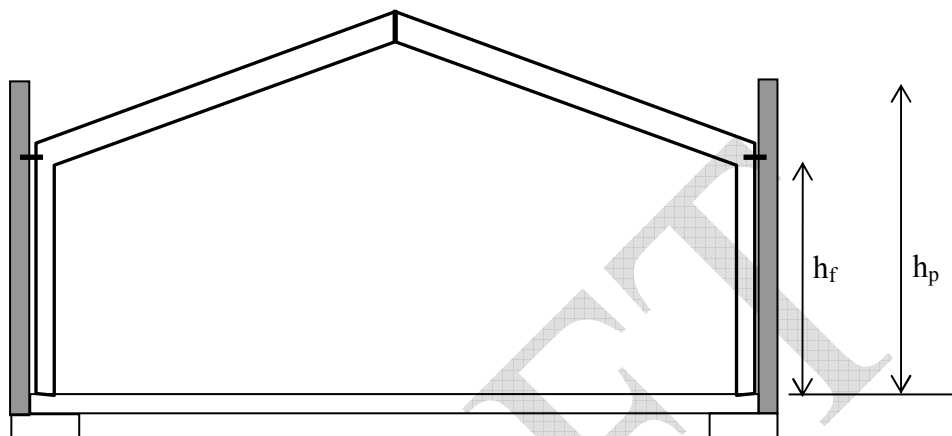


Figure 5: Tributary area to connections

Note that if new fixings are required, the connections must comply with the fire rating requirements of the Building Code, in full. Guidance is provided in clause 4.8 of NZS3101:2006. Attention should be paid to the type of connection used as well as to the required design loads, noting that adhesive anchors should only be used if they have achieved the appropriate fire resistance ratings through test.

9B.1.4 REPORTING OF RESULTS

The results of the evaluation should be reported according to the Part 2 DEE Guidelines. The capacity of the panels is the lesser of the %NBS scores calculated for the in-plane or out-of-plane loading