Report Prepared for the New Zealand Earthquake Commission

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Predicted & Actual Performance of Masonry Parapets in the 2007 Gisborne Earthquake

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Abstract – Non-Technical

The author previously developed a methodology that can be used to predict the earthquake performance of the walls and parapets of masonry buildings. This procedure has been incorporated, in modified form, into the New Zealand Society of Earthquake Engineering procedures for assessing and improving the structural performance of earthquake risk buildings.

On the 20th of December 2007 a Mi 6.9 earthquake occurred 47 km south-east of Gisborne. This earthquake caused damage to unreinforced masonry buildings in the Gisborne CBD that included the collapse of 22 parapets. Fortunately a record of the ground motion in the Gisborne CBD was obtained. This record could be used to evaluate whether the risk of parapet collapse in the earthquake was consistent with that predicted by the procedure developed by the Author and as modified by the NZSEE.

The NZSEE procedure for evaluating parapets makes a number of modifications to the original assessment procedure developed by the Author. This study shows that these modifications significantly alter the predicted performance of the parapets exposed to the Gisborne earthquake.

To enable the assessment procedures to be used to compare the predicted and actual behaviour of the parapets in the Gisborne CBD details were obtained for 19 of the 22 parapets that collapsed in the 2007 Gisborne earthquake. A further 82 parapets were also surveyed so that they could be evaluated using the assessment procedures.

It was found that the best prediction of the actual parapet behaviour was given by a modified version of the Author's procedure. This modified procedure is actually an amalgam of the Author's original procedure and the NZSEE procedure.

The parapets in the Gisborne CBD had been exposed to an earlier 1993 Ormond earthquake. A comparison between the earthquake records of the 1993 and 2007 earthquakes indicated that the 1993 earthquake was significantly weaker, particularly in the direction parallel to the main street. Almost all the parapets that collapsed in the 2007 earthquake were subjected to this component of the earthquake. This relatively greater damage was consistent with what would be expected from using the assessment procedures

In the other direction, where the parapets are parallel to the main street, the spectral displacement intensities of the two earthquakes were similar and only one parapet collapsed in the 2007 earthquake and none collapsed in the 1993 earthquake. The relative amount of damage was, therefore, consistent with the relative spectral intensities of the two earthquakes as would be expected from the assessment procedures.

However in this direction, more collapses would have been predicted in both earthquakes using the assessment procedures. It is quite possible that these parapets had been subjected to significantly higher earthquake motions during the even earlier 1966 Gisborne earthquake which would have "culled" the weaker parapets. This "culling" may help to explain why fewer of the parapets orientated parallel to the main street collapsed in the 2007 than would have been predicted by the Author's modified assessment procedure.

Only approximately 6% of the total number of parapets exposed to the earthquake actually collapsed during the 2007 Gisborne earthquake. This study indicated that the Author's modified procedure would have predicted a greater probability of parapet collapse. It is suggested that a significant proportion of this difference in performance can be explained by the manner in which the external walls of the masonry buildings responded to the earthquake. It is believed that many of the external walls of the Gisborne CBD buildings responding, at least in part, as free standing vertical cantilever walls rocking about a horizontal "crack" opening at foundation level. This type of behaviour is to be expected when the floor and roof diaphragm of a building is not strong or stiff enough to significantly affect the response of the walls. Vertical cracks at the corners of some of the Gisborne masonry buildings where the external walls had separated, indicates that the external walls may have responded, at least in part, as vertical cantilevers.

When the external walls respond to an earthquake as vertical cantilevers the parapets are not subjected to as highly amplified ground motion and are not as likely to collapse. Paradoxically this suggests that if roof diaphragms are stiffened and strengthened the parapets will be subjected to amplified ground motions and be more likely to collapse in a moderate earthquake. The Author's modified procedure may then more accurately predict the actual risk of collapse.

Abstract –Technical

The author previously developed a methodology that uses acceleration and displacement response spectra to predict the earthquake spectral intensity that will cause a face-loaded unreinforced masonry wall element to collapse. This procedure has been incorporated, in modified form, into the New Zealand Society of Earthquake Engineering procedures for assessing and improving the structural performance of earthquake risk buildings.

On the 20[th] of December 2007 a Mi 6.9 earthquake occurred 47 km south-east of Gisborne. This earthquake caused damage to unreinforced masonry buildings in the Gisborne CBD that included the collapse of 22 parapets. Fortunately a free-field record of the ground motion in the Gisborne CBD was obtained adjacent to the 2ZG radio station. This record could be used as a measure of the spectral intensity of the earthquake motion that had been imposed on the collapsed parapets and presented a golden opportunity to evaluate whether the risk of parapet collapse in the earthquake was consistent with that predicted by the procedure developed by the Author and as modified by the NZSEE.

The NZSEE procedure for evaluating parapets and other cracked face-loaded URM wall elements makes a number of modifications to the original face-loaded wall assessment procedure developed by the Author. These are principally; use of only a displacement spectrum instead of both displacement and acceleration spectra; use of floor spectra based only on the peak ground acceleration instead of a floor response spectra equal to an amplification of the ground spectra; different factors for the effect of building amplification; and formulae for effective period and participation factor that include the effect of wall element slenderness. This study shows that these modifications significantly alter the predicted performance of the CBD parapets during the Gisborne earthquake.

The study indicated that the main difference between the performance predicted by the Author's and the NZSEE procedures is due to the differences in the shape of the floor response spectra adopted for the two procedures. The shapes of these spectra diverge most on soft ground and where the ground response spectral displacements start to reduce at a relatively short spectral response period. Both these conditions were satisfied for the ground motion recorded during the 2007 Gisborne earthquake. For earthquake motions that have displacement spectral shapes closer to those given by design codes the differences in the two procedures would be less pronounced.

To enable the assessment procedures to be used to compare the predicted and actual behaviour of the parapets in the Gisborne CBD, details were obtained for 19 of the 22 parapets that collapsed in the 2007 Gisborne earthquake. These parapets were surveyed in sufficient detail to enable them assessed using both the Author's and the NZSEE procedures. A further 82 parapets were also surveyed so that they could be evaluated

using the assessment procedures.

It was found that the best prediction of the actual parapet behaviour was given by a modified version of the Author's procedure for assessing parapets.

The Modified Author's Procedure requires two changes to the Author's original procedure. The first modification is to use the NZSEE equations for calculating the rocking period of the parapet and its participation factor. This modification takes into account the slenderness of the parapet and has most effect on squat parapets. The second modification requires that only the displacement spectrum be used in the procedure (as in the NZSEE procedure), instead of using both acceleration and displacement spectra as used in the Author's original procedure. Again this mainly effects squat parapets but tends to negate the inclusion of parapet slenderness in the equations used to calculate the period and participation factor.

Similarly to alter the NZSEE procedure so that it is the same as the Modified Author's Procedure two modifications would be required. The first modification is to use an amplified version of the ground motion (as in the Author's procedures) as the input spectral intensity of the earthquake motion at the support level of the parapet rather than a floor spectra response based on the peak ground acceleration (as in the current NZSEE procedure). The second modification would be use the Author's height amplification factor instead of those effectively derived from NZS 1170.5 using 5% or 15% damping as currently used in the NZSEE procedure.

The survey indicated that only approximately 6% of the total number of parapets exposed to the earthquake and that actually collapsed during the 2007 Gisborne earthquake. Assuming that the Gisborne CBD parapets surveyed in detail and included in the assessment analyses were reasonably representative of the total population of the CBD parapets, the study would have predicted a greater probability of parapet collapse. It is suggested that a significant proportion of this difference in performance can be explained by the manner in which the external walls of the building may have responded to the earthquake. It is believed that many of the external walls of the Gisborne CBD buildings responded, at least in part, as free standing face-loaded vertical cantilever walls rocking about a horizontal "crack" opening at foundation level. This type of behaviour is to be expected when the floor and roof diaphragm is not strong or stiff enough to significantly affect the response of the walls. Under these conditions a horizontal crack is less likely to form at the parapet support level and, if it does form, the parapet is less likely to collapse. Paradoxically this suggests that if roof diaphragms are stiffened and strengthened the parapets will be subjected to more highly amplified ground motions and be more likely to collapse in a moderate earthquake. The Author's modified procedure may then more accurately predict the actual risk of collapse.

The displacement spectra for the 2007 Gisborne earthquake do not have a steadily increasing displacement demand with increasing period. Counter-intuitively, the assessment procedures predicts that, in a moderate earthquake motion that has this type of spectral shape, the slender external walls of masonry buildings may be more stable responding as vertical cantilevers than squat parapets responding the amplified ground motion delivered to the supporting level of parapets by relatively strong/stiff roof structures. This type of response is to be expected when the floor and roof diaphragms are not strong or stiff enough to significantly affect the response of external walls. Vertical cracks at the corners of some of the Gisborne masonry buildings where the external walls had separated, indicates that the external walls may have responded, at least in part, as vertical cantilevers.

The parapets in the Gisborne CBD had been subjected to the earlier 1993 Ormond earthquake. A comparison between the spectral displacement intensities of the earthquake records recorded in the 1993 and 2007 earthquakes indicated that the 1993 earthquake was significantly weaker in the critical spectral period range particularly parallel to the main street. All but one of the parapets that collapsed in the 2007 earthquake were subjected to this (N51W) component of the earthquake.

In the other direction, where the parapets are parallel to the main street, the spectral displacement intensities of the two earthquakes were similar and only one parapet collapsed in the 2007 earthquake and none collapsed in the 1993 earthquake. The relative amount of damage was, therefore, consistent with the relative spectral intensities of the two earthquakes as would be expected from the assessment procedures.

However in this direction, more collapses would have been predicted in both earthquakes using the assessment procedures. It is quite possible that these parapets had been subjected to significantly higher spectral displacement intensity motions during the even earlier 1966 Gisborne earthquake which would have "culled" the weaker parapets. This "culling" may help to explain why fewer of the parapets orientated parallel to the main street collapsed in the 2007 than would have been predicted by the Author's modified assessment procedure.

Contents

1 Introduction

In the early 1980's US researchers (ABK 1982) subjected full-scale unreinforced masonry (URM) face-loaded wall specimens to earthquake motions. These specimens represented a wall element spanning vertically between two adjacent floors. They found that a single horizontal crack tended to form near mid-height of the test specimens and another crack formed at the test bed floor, and that the walls were able to sustain large displacements normal to the face of the wall, comparable with the wall thickness. This ability to withstand large displacements without collapse resulted in the walls having a significant post cracking seismic resistance. The term "dynamic stability" was used to distinguish this type of behaviour from the behaviour that might have been expected from static force calculations.

The author (Blaikie, 1992 and 2000) developed a methodology that uses acceleration and displacement response spectra to predict the earthquake spectral intensity that will cause a face-loaded wall element to collapse or lose its "dynamic stability".

In a later study (Blaikie 2002), the assessment procedure was extended to cover face-loaded single-storey walls, parapets and freestanding walls supported only by the ground.

This procedure has been incorporated, in modified form, into the New Zealand Society of Earthquake Engineering procedures for assessing and improving the structural performance of earthquake risk buildings (NZSEE 2006).

The procedure was calibrated using computer analyses of face-loaded wall elements. These analyses indicated that the earthquake intensity required to collapse a face-loaded wall element is generally conservatively predicted by the procedure.

The use of computer modelling was, in turn, validated by comparing the wall response predicted by the model with the results of a wall test specimen that had been subjected to simulated seismic face-loading.

On the 20^{th} of December 2007 a Mi 6.9 earthquake occurred 47 km south-east of Gisborne at a depth of 44km. This earthquake caused damage to unreinforced masonry buildings in the Gisborne CBD that included the collapse of a significant number of parapets. Fortunately a free-field record of the ground motion in the Gisborne CBD was obtained adjacent to the 2ZG radio station. This record could be used as a measure of the spectral intensity of the earthquake motion imposed on the collapsed parapets and presented a golden opportunity to evaluate whether the risk of parapet collapse in the earthquake was consistent with that predicted by the procedure developed by the Author and as modified by the NZSEE.

The scope of the current study encompasses:

1. Refinement of the equations that describe the behaviour of cracked face-loaded URM parapets as used in the assessment procedure developed by the Author and as modified by the NZSEE.
2. Evaluating the risk of the Gisborne parapets collapsing as predicted by the Author's procedure and the modified NZEE procedure.
3. Using the Author's and the NZEE procedures to evaluate whether the Gisborne parapets would be considered to be earthquake-prone and, therefore, potentially require strengthening under New Zealand's building legislation.
4. Investigating whether the Gisborne earthquake had "proof loaded" the Gisborne CBD parapets so that they would no longer be considered to be earthquake-prone.
5. Investigating the reasons for the differences in the parapet performance predicted by the Author's procedure and modified by the NZSEE.
6. Surveying the collapsed, damaged and undamaged parapets in the Gisborne CBD so that the actual risk of parapet collapse in the earthquake could be compared with the risk of collapse predicted by the assessment procedures.

2 Behaviour of Face-Loaded URM Parapets and Free standing walls

2.1 Rocking of Face-Loaded URM Parapets and Free-standing walls

A Face loaded parapet or free standing wall may, conservatively, be assumed to be cracked at its base where it is supported on the remainder of the wall (i.e. on a parapet) or on a foundation (i.e. a free-standing wall). This crack may form when the element is subjected to sufficient lateral load, or may have formed prior to an earthquake due to differential thermal or moisture effects.

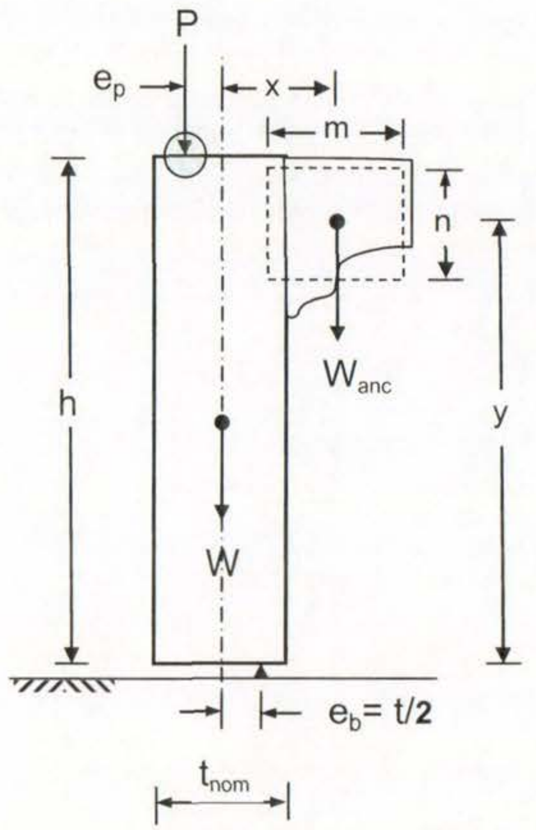


Figure 1: Face Loaded Parapet or Free Standing Wall

Figure 1 shows a parapet or single storey wall element about to rotate clock-wise about a pivot point at the base of the element and close to the element face. This clockwise rotation would be part of the motion of the wall element rocking back and forward during an earthquake.

The wall has a total weight, W; height, h, and nominal thickness, t_{nom}. The wall element also has an ancillary mass (corbel or similar eccentric mass) on one face, of weight W_{anc} and with its centroid located at x and y relative to the main element centreline and the pivot point respectively. Note that x is considered positive where it reduces the stability of the wall element as in Figure 1. A rectangular block is also shown in Figure 1 with dimension m and n which has the same rotational mass moment of inertia as the actual ancillary mass. The overburden load (and/or mass), P, represents the weight of a roof or similar mass and would normally be zero for a parapet. This overburden is assumed to be applied at an eccentricity e_p from the wall centreline and is considered to be positive when the overburden improves the stability of the wall element as in this case.

As a compression zone depth would be required to develop the compressive forces acting at the wall cracks and as the mortar may not extend to the outside face of the wall, the effective wall thickness, t, will be less than the nominal wall thickness, t_{nom}. The effective wall thickness, t, may be evaluated using the relationship (Blaikie 2002):

$$t = t_{nom}(0.975 - 0.025\frac{P}{W}) \tag{1}$$

Equations describing the motion of a rocking wall element such as that shown in Figure 1 have been developed for simple cases in previous studies by the Author (Blaikie, 2000, 2005) and for the more general conditions like that shown in Figure 1 with an eccentric mass in the New Zealand Society of Earthquake Engineering Guidelines (NZSEE, 2006, Appendix 10A). The Guidelines describe the derivation of the equations of motion for the general case in detail and only the results for the conditions given in Figure 1 are presented here.

The equation of motion of the rocking wall element for free vibration takes the form:

$$-aA + b = -JA \tag{2}$$

Where: a & b are derived using virtual work to obtain equilibrium of the rotating wall element:

$$a = PF_H h + W\frac{h}{2} + W_{anc} y \tag{3}$$

$$b = P(e_p + e_b) + We_b - W_{anc}(x - e_b) \tag{4}$$

$$J = \left[\frac{W}{12g}(h^2 + t_{nom}^{\ 2})\right] + \left[\frac{W}{g}\left(\left(\frac{h}{2}\right)^2 + e_b^{\ 2}\right)\right] + \left[\frac{PF_H}{g}(h^2) + \frac{PF_v}{g}(e_p + e_b)^2\right]$$
$$+ \left[\frac{W_{anc}}{12g}(m^2 + n^2) + \frac{W_{anc}}{g}\left(y^2 + (x - e_b)^2\right)\right] \tag{5}$$

Where: F_H and F_v are the proportion of the mass associated with the overburden load P active in the horizontal and vertical directions respectively (for discussion of these parameters see section 2.5).

The equation of motion, equation (2), is only strictly valid for the wall element when the parameters a, b, and J evaluated for a anticlockwise rotation of the wall element are the same as for the clockwise rotation conditions. However, providing the wall element is assessed for stability in both rotational directions, and the minimum stability case is adopted, the assessment is assumed in the procedure to be conservative.

Once the three parameters a, b, and J have been evaluated the following features of the rocking wall element motion can be computed:

The instability displacement at the top of the wall element, at which the element becomes unstable,

$$\Delta_i = \frac{bh}{a} \tag{6}$$

The period of the wall free vibration expected when the peak displacement is $0.6\ \Delta_i$. The free vibration period increases towards infinity as the wall displacement approaches Δ_i and the period when the peak displacement is $0.6\Delta_i$, given by equation (7), is taken as representative of a wall element approaching collapse.

$$T_p = 6.7\sqrt{\frac{J}{a}} \tag{7}$$

Where: J and a are evaluated with dimensions are in meters, forces in kN and g = 9.8m/sec2.

The participation factor which gives the ratio of the displacement expected at the top of the wall element to that expected for a single-degree-of-freedom (SDOF) oscillator. This factor can be used to predict the displacement expected at the top of a wall element for a SDOF displacement response spectrum.

$$\gamma = \frac{\left[W + W_{anc}\frac{2y}{h}\right]h^2}{2Jg} \tag{8}$$

The seismic load coefficient that corresponds to the conditions when the crack at the base of the wall element just starts to open when a uniform horizontal static seismic load is applied:

$$C = \frac{2b}{Wh} \tag{9}$$

Note that the NZSEE Guidelines has used a modified parameter notation compared with that used in previous studies by the Author. These modifications, as detailed in Table 1, were also adopted for this study.

Table 1: Difference in parameter notion used in this study compared with previous studies

Study	Comparison of Parameter Notation				
This study & NZSEE Guidelines	C	T_p	Δ_i	P	C_{Hi}
Previous Studies	C_d	T	Y_{max}	O	A

2.2 Author's Procedure Used to Predict Performance of Face-loaded Parapets

A procedure was developed in previous studies to predict the seismic stability of face-loaded wall elements by observing the behaviour a wide range of URM wall elements subjected to inelastic time-history analyses.

The procedure makes use of both the acceleration and displacement response spectra for an earthquake motion and simplified versions of the equations given in the proceeding section.

The spectra for an actual earthquake record or spectra given by design codes can be used in the procedure. Where only an acceleration spectrum is available a pseudo displacement response spectrum can easily be derived from an acceleration design spectra using the relationship:

$$Y = \frac{T^2}{(2\pi)^2} C(T) \qquad \textbf{(10)}$$

Where: $C(T)$ = the spectral acceleration (in m/sec^2) for a SDOF elastic oscillator with period T; Y = the pseudo spectral displacement.

Figure 2 shows the acceleration response design spectrum given by NZS 1170.5 for the Gisborne CBD and the pseudo displacement spectrum derived from this acceleration spectrum. The spectrum is for a return period of 500 year on deep or soft soil sites (Class D, zone factor Z = 0.36)

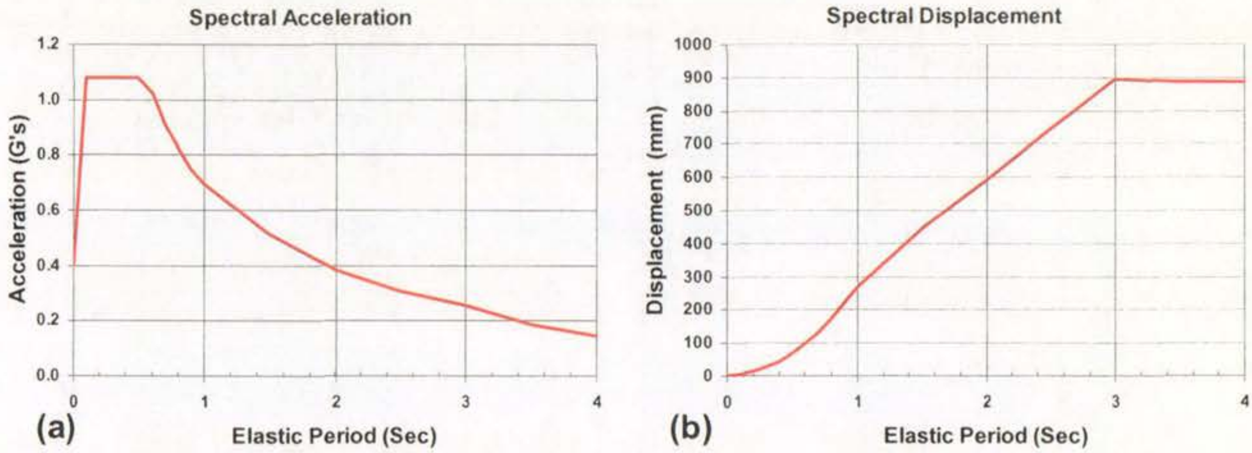


Figure 2: (a) Acceleration response design spectrum given by the NZS 1170.5. for Gisborne CBD (b) Pseudo displacement spectrum derived from this acceleration spectrum.

If a URM wall element is subjected to gradually increasing earthquake intensity eventually the wall element will collapse. The spectra shown in Figure 2, for example, represent a certain earthquake ground shaking intensity. In the procedure the displacement spectrum is used to predict the earthquake intensity that will generate wall displacements equal to the displacement at which the wall element becomes unstable. However, in some cases, this displacement procedure may predict a collapse earthquake intensity that is not significantly greater than that required to open the cracks in the wall element. Therefore, an acceleration spectrum is also used to predict the earthquake intensity that will just open the joint cracks in the wall element.

The procedure predicts that the scaling factor, $I_{capacity}$ that must be applied to an earthquake intensity to cause a wall element to collapse is:

either $$I_{capacity} = I_{sp} \quad \text{when } I_{sp} \geq 2.5I_{cr} \qquad \textbf{(11)}$$

or $$I_{capacity} = \frac{I_{sp} + 2.5I_{cr}}{2} \quad \text{when } I_{sp} < 2.5I_{cr} \qquad \textbf{(12)}$$

Where: I_{sp} corresponds to the earthquake intensity that will generate displacements equal to the displacement at which the wall element becomes unstable and is calculated using the displacement spectrum procedure given in section 2.3;

I_{cr} corresponds to the earthquake intensity that will just open the joint cracks in the wall element and is calculated using the acceleration spectrum procedure given in section 2.4.

When evaluating $I_{capacity}$, the parapet is assumed to be supported by a rigid structure with rigid diaphragms. When the wall element is supported on a flexible structure the parapet will be subjected to an amplified shaking intensity. Amplification factors, C_{Hi}, to be used in the Authors procedure are given in Table 2.

For example, a scaling factor $I_{capacity}$ of 0.6 would indicate that parapet collapse would be predicted at 60% of the full earthquake ground shaking intensity represented by the spectra used for the analysis assuming no amplification of the ground motion by the supporting structure. However, a parapet would be subjected to twice the earthquake intensity shaking (i.e. C_{Hi} = 2.0 from Table 2) and the earthquake intensity predicted to correspond to parapet collapse would be halved (i.e. to 30% of the spectral intensity).

The procedure summarised above has been calibrated so that it results in an assessed earthquake spectral intensity with a low probability of causing collapse rather than the "expected" collapse intensity. This makes it more useful as an assessment/design tool. This has been achieved by adjusting the constants such as the 2.5 factor in equation (11) and the 0.6 and 1.2 factors in equation (13). These factors were adjusted so that the procedure predicted the results of the time-history computer modelling of wall elements conservatively and so that the scatter in the difference between the seismic capacity predicted by the procedure and the computer modelling would be reduced to a minimum.

The amplification factors given in Table 2 were deduced from the computer modelling which varied the model parameters so that the effects of building amplification on face-loaded wall element stability could be evaluated.

Table 2. Storey elevation amplification factors for wall element within one storey and parapets

Parapets (floor/roof diaphragms rigid, flexible or yielding)	**Storey Elevation Amplification Factor (C_H)**	
	Single Storey Building	**Multi-Storey Building**
- Building period < 0.5 seconds	2.0	3.0
- Building period >1.0 seconds	2.0	2.0

2.3 Evaluation of I_{sp} for a Parapet using a Displacement Spectrum

The following steps are required to calculate the scaling factor, I_{sp} using a displacement spectrum. When the earthquake intensity, represented by the spectrum is scaled by this factor, the procedure predicts the parapet would reach a top displacement at which it becomes unstable.

Step1: Evaluate the period, T_p, of the rocking motion of the wall element when the peak displacement is 60% of the instability displacement, Δ_i. (i.e. use equation (7)).

Step2: Use a displacement response spectrum for the earthquake motion (e.g. Figure 3(b)) to evaluate the maximum displacement expected for a SDOF structure with the period, T_p calculated in Step 1. The maximum displacement, Y_{sp}, expected at the top of the parapet for the earthquake motion is 1.5 times this SDOF displacement.

The 1.5 multiplier is the value of the normalised modal participation factor evaluated using equation (8) assuming $(t/h) = 0$.

When evaluating the expected displacement from the displacement response spectrum, the spectral displacement is assumed to generally increase with increasing period, T and any local "dips" in the displacement response spectra are ignored (i.e. a smoothed spectrum is used in the procedure).

Step 3: The parapet will become unstable when the maximum wall displacement is Δ_i. The earthquake intensity required to generate a maximum wall displacement of Δ_i is taken as 1.2 times the earthquake intensity required to generate a wall displacement of $0.6\Delta_i$ (where Δ_i is evaluated in step 1).

Therefore, the earthquake spectral intensity scaling factor, I_{sp} that will cause the wall element to collapse, predicted using the displacement spectrum, is given by:

$$I_{sp} = 1.2 \left(\frac{0.6\ \Delta_i}{Y_{sp}} \right) \qquad \textbf{(13)}$$

where Δ_i is evaluated in step 1; Y_{sp} is evaluated as in step 2 above

2.4 Evaluation of I_{cr} for a Parapet using an Acceleration Spectrum

The following steps are required to calculate the scaling factor, I_{cr} using an acceleration spectrum. When the earthquake intensity, represented by the spectrum is scaled by this factor, the procedure predicts that the joint cracks in the base of the parapet just begin to open:

Step 1: Evaluate the initial elastic period, T_o, of the wall element. A parapet or freestanding cantilever wall is assumed to respond as fixed base cantilever. An effective elastic modulus of 1.0GPa is be used for the masonry. As tests (Blaikie, 2002) indicate that a large amount of wall curvature occurs adjacent to the cracks in the wall, this value of the elastic modulus may not be conservative so that any initial rising branch of the spectra is normally ignored.

Step 2: Use an acceleration response spectrum for the earthquake motion (e.g. Figure 3(b) to evaluate the maximum response, C_{sp}, expected for a SDOF structure with a period, T_o.

Step 3: Calculate the seismic coefficient, C, corresponding to the UDL lateral load that would be just sufficient to open the cracks at the base of the parapet (using equation (4)).

Step 4: The earthquake scaling factor, I_{cr}, which must be applied to the EQ motion so that it would be just sufficient to open the wall element joints is then given by.

$$I_{cr} = C/C_{sp} \qquad \textbf{(14)}$$

Design charts to enable rapid design office assessment of face-loaded wall elements in terms a late draft of the Loading Standard Basic Seismic Hazard Spectra (i.e. AS/NZS 1170.4/PPC8) which were unchanged in the final version of NZS1170.5 have been prepared (Blaikie 2002, Appendix B). These charts allow the $I_{capacity}$ scale factor (equations (11) & (12)) to be rapidly evaluated for a wide range of wall thickness, wall element types and boundary conditions.

2.5 Procedure Used by NZSEE Guidelines to Predict Face-loaded Parapets

The NZSEE Guidelines include a procedure for the assessment of face-loaded URM walls and parapets that is similar to that developed by the author in previous studies. However, there are some significant differences between the two methodologies.

The main differences in the two methodologies are:

Scope of Applicability:
The Author's procedure only applies to wall elements of uniform thickness and mass distribution over their height. The Guideline procedure is more general making allowance for non-uniform mass distribution such as the corbel shown in Figure 1.

Mass associated with Overburden P:
The basic equations for evaluating the parameters a & J (i.e. equations (3)&(5)) include factors F_H and F_V. These factors allow for the possibility that not all the mass associated with the overburden load supported by the wall element may be activated during the dynamic response of free standing wall element. For example if it could be assumed that the roof structure of a single-storey building would carry the seismic loads generated by its mass the F_H factor would be zero. Similarly, if the roof structure is very flexible in the vertical direction not all the roof mass would be activated when the top of the wall element moves in the vertical direction during the rocking motion. In this case F_V would be less than 1.0. The factors F_H and F_V are not included in the NZSEE Guideline procedure and are, therefore, effectively assumed to be 1.0 in all cases. In the Author's procedure these factors were previously assumed to be zero.

However, as parapets generally do not have overburden loads or masses this difference is not significant for the parapets which are the main focus of this study.

Effect of wall slenderness on the effective period of the rocking response:
For uniform parapets with no overburden, equation (7), which is the same as that in the Guidelines, reduces to:

$$T_p = \sqrt{2.67\left(1+\left(\frac{t}{h}\right)^2\right)h} \qquad \textbf{(15)}$$

In the Author's procedure the term under the square root was taken as 2.8h so that the same period, T_p, would be obtained using equation (15) for a parapet with t/h of 4.5. For more squat parapets the effective period T_p would be longer and the predicted displacement using a displacement spectrum would, therefore, be greater leading to a lower predicted collapse capacity.

Effect of wall slenderness on the Participation Factor:
For uniform parapets with no overburden, equation (8), which is the same as that in the Guidelines, reduces to:

$$\gamma = \frac{3}{2\left(1+\left(\frac{t}{h}\right)^2\right)} \qquad \textbf{(16)}$$

In the Author's procedure the maximum value of the participation factor which is 1.5 was used. For more squat parapets the participation factor would be greater and the predicted displacement using a displacement spectrum would, therefore, be smaller leading to a higher predicted collapse capacity.

Amplification factor for height of parapet above ground level:
The Guidelines use the "Part and Portions" section of NZS1170.5 to obtain the expected variation in shaking intensity with height of the parapet above ground level.

$$C_{Hi} = (1+\frac{h_i}{6}) \text{ with a max value 3.0 for } h_i > 12\text{m} \qquad \textbf{(17)}$$

Where: h_i is the height of the parapet above ground level

This amplification factor varies between about 1.5 (for h_i = 3.0m) and 3.0 ((for h_i > 12.0m) with linear interpolation in between. However, for URM buildings the guidelines allow 15% damping to be assumed for the building response. This greater damping would reduce these height amplification factors to about 2/3 of the above values (i.e. to 1.0 & 2.0 respectively).

These height amplification factors can be compared with the storey elevation amplification factors used in the Author's procedure given in Table 2. Except for parapets on non URM multi-storey building higher than 12m, the Guidelines amplification factors would result in a higher predicted parapet collapse capacity.

Floor response spectral shape:
The Guidelines use the "Part and Portions" section of NZS1170.5 to obtain what is effectively a fixed floor response spectral acceleration shape. The shape of the basic floor response spectrum is assumed to only depend on the peak ground acceleration (i.e. C(T = 0) value) and varies between twice C(0) for periods less than 0.75 seconds and 0.5C(0) for periods greater than 1.5 seconds with linear interpolation in between.

In the Author's procedure the shape of the floor response spectra is assumed to be the same as the ground response.

The difference in the assumed shape of the floor response spectrum is the main difference between the two methodologies and results in significant differences in the predicted performance of face-loaded wall elements. For example the Author's procedure would generally predict the worst performance for wall elements on very soft soil while the Guideline procedure would generally predict the worst performance on shallow soil sites where the ground acceleration given by NZS1170.5 is the highest.

Use of only displacement spectra to predict parapet performance:
The Author's procedure, as summarised in equations (11) & (12) uses both acceleration and displacement spectrum to predict the shaking intensity required to collapse a parapet.
The Guidelines uses only the displacement spectrum part of the Authors procedure (i.e. equation (12)). This is conservative as the collapse earthquake intensity computed using the acceleration spectrum is only used in Author's procedure when it predicts a higher earthquake resistance for the parapet. Hence, using only the displacement spectra results in a lower predicted parapet collapse capacity. Generally this conservatism only effects squatter wall elements such as parapets but is most pronounced for thinner wall elements.

The effect of these differences in the two methodologies is discussed in the following sections of this report where the expected performance of parapets in the Gisborne earthquake are predicted and compared with the actual performance.

3 Predicted Gisborne Parapet Performance – Author's Procedure

3.1 2ZG Earthquake Record

Figure 3 and Figure 4 show the acceleration and pseudo displacement spectra for the two components of the free field earthquake record recorded at the 2ZG radio station site during the Gisborne CBD. The N51W component of this ground motion was approximately parallel to the main street (Gladstone Rd) and is the component that would have most effect on the side-wall parapets of the buildings in the main street.

For this study the two components of the 2ZG earthquake record were taken as representative of the spectral response intensity of the earthquake motion that the parapets in the Gisborne CBD were subjected to in the 2007 Gisborne Earthquake for the two principal directions parallel and normal to the main street.

To some extent the spectral response intensity will have varied locally throughout the CBD but the two earthquake ground motions recorded in the Gisborne CBD during the 1993 Ormond EQ (see details section 8.7) suggest that the variation in spectral response intensity may have only been moderate.

Figure 3 & Figure 4 also show smoothed spectra with the local dips in intensity removed. These spectra are the versions used in the Author's procedure as previous research indicated that this smoothing improved the correlation between the performance predicted by time-history analyses and the evaluation procedure.

3.2 Predicted Performance of Free Standing Walls in Gisborne Earthquake

Figure 5 shows the earthquake scaling factor that Author's procedure predicts would need to be applied to two components of the Gisborne earthquake motion to result in a low probability of collapse of free standing 220 mm thick walls of varying slenderness. The procedure uses both the acceleration and displacement spectra in Figure 3 & Figure 4 and the procedure given in section 2.2.

The plot for the S39W component of the earthquake indicates that a free standing 220mm wall with a h/t_{nom} thickness ratio of 4.5 (i.e. height of 990mm) would be the most vulnerable in the slenderness range considered and have a "capacity" to withstand, with a low probability of collapse, a shaking intensity 1.5 times that recorded at the 2ZG site. Counter-intuitively, a more slender wall with a h/t_{nom} ratio of 25 (i.e. height of 5.5m) is predicted to be more stable and have a capacity to safely withstand about 4 times the S39W recorded earthquake spectral response intensity.

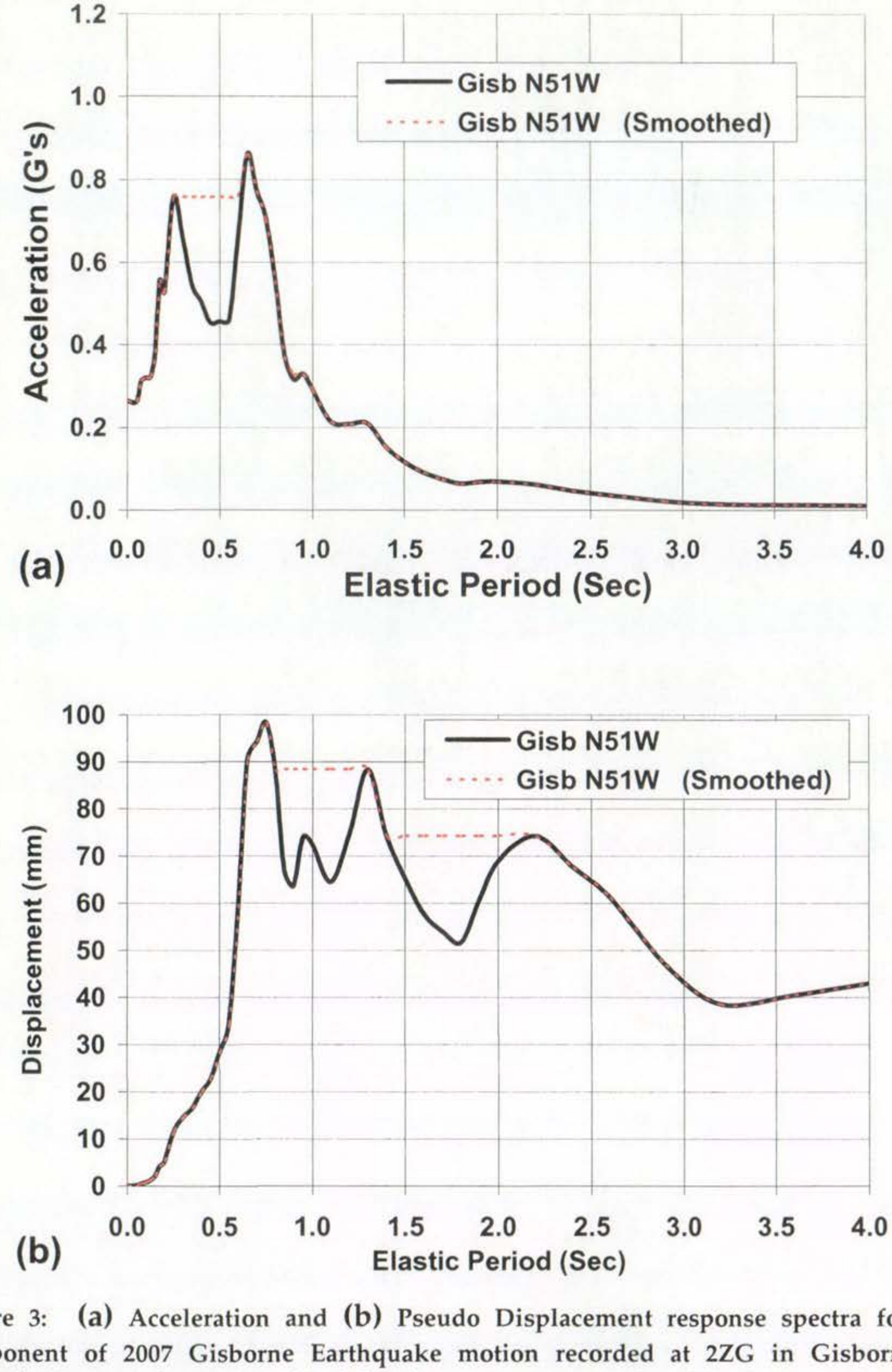
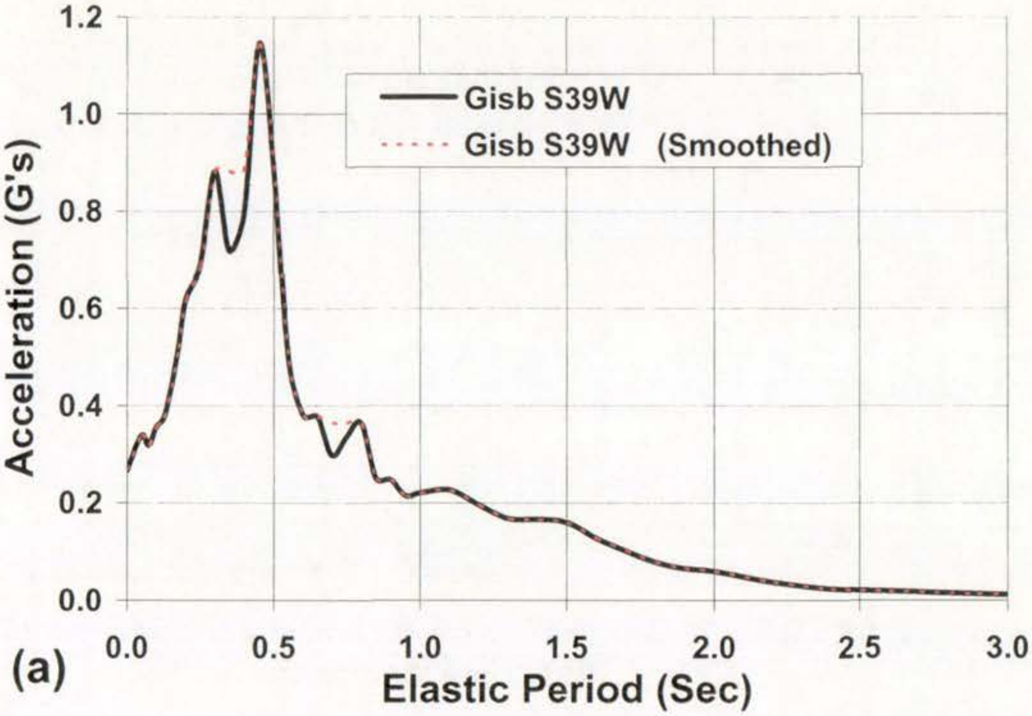


Figure 3: (a) Acceleration and (b) Pseudo Displacement response spectra for N51W component of 2007 Gisborne Earthquake motion recorded at 2ZG in Gisborne CBD. Smoothed spectra shown have local dips in response removed and are the spectra used in the Author's parapet assessment procedure.



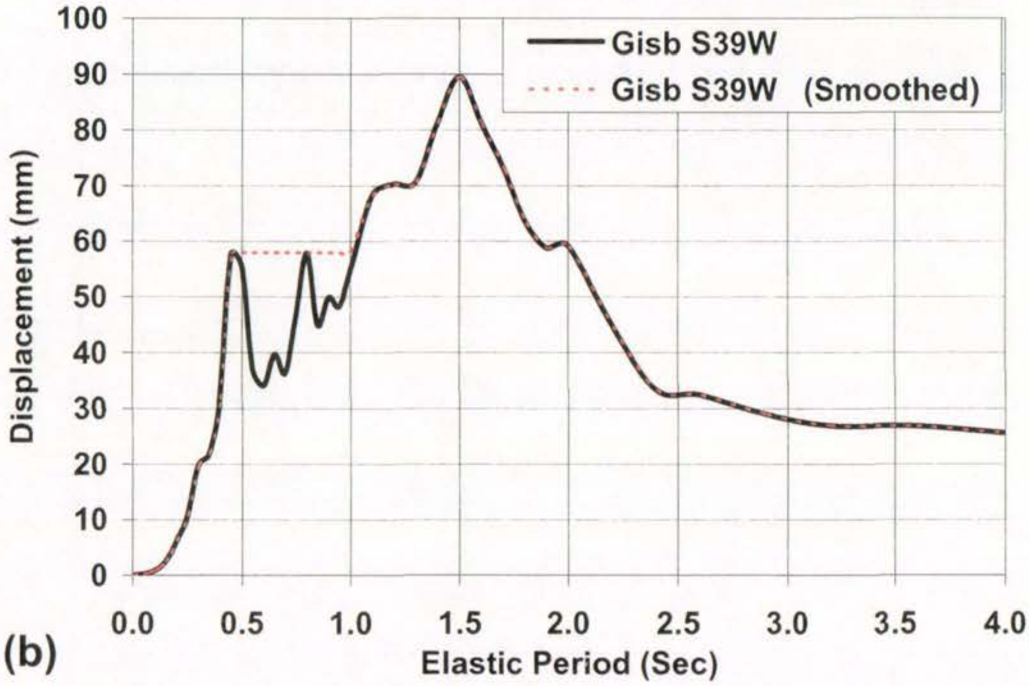


Figure 4: (a) Acceleration and (b) Pseudo Displacement response spectra for S39W component of 2007 Gisborne Earthquake motion recorded at 2ZG in Gisborne CBD. Smoothed spectra shown have local dips in response removed and are the spectra used in the Author's parapet assessment procedure.

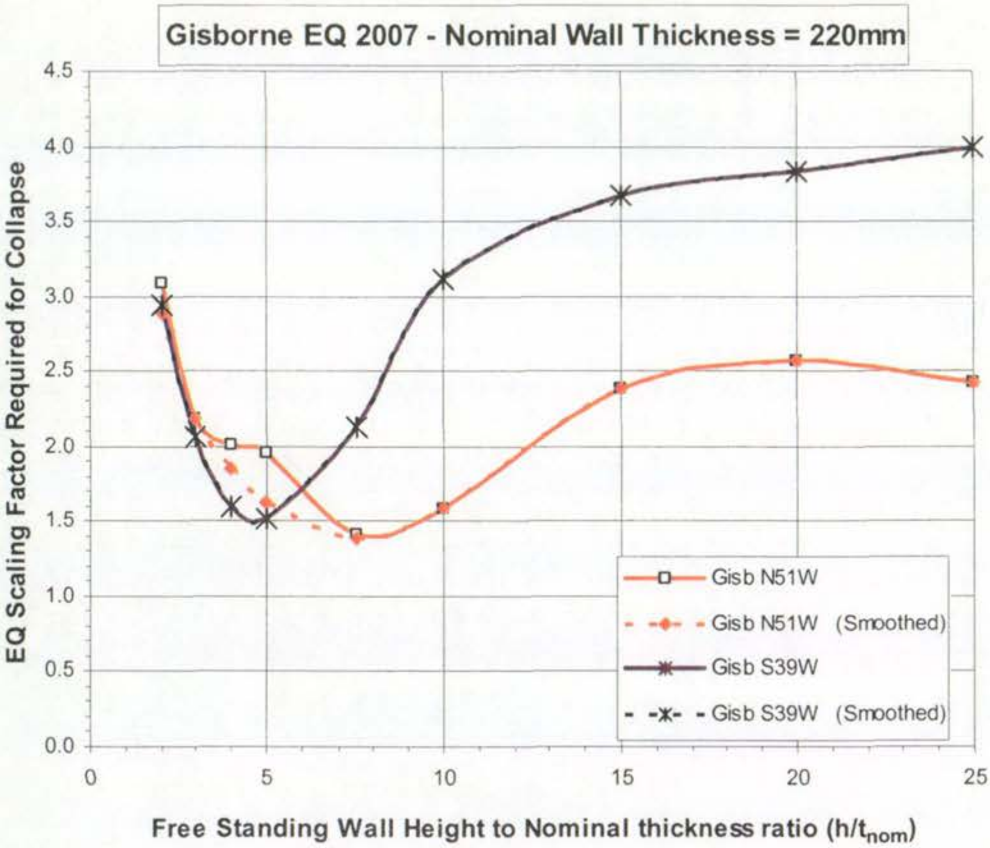


Figure 5: EQ Scaling factor that Author's procedure predicts would need to be applied to Gisborne EQ motion (2ZG record) to result in only a low probability of collapse of free standing 220 mm thick walls of varying slenderness.

The counter-intuitive shape of the plots in Figure 5 can be explained by the shape of the displacement spectrum used in the analysis. For squat wall with slenderness, h/t_{nom} less than about 5 to 6 the acceleration response required to open the crack at the base of the wall element is relatively high and increases the predicted collapse capacity predicted using equation (12).

For more slender walls, equation (11) governs and, therefore, only the displacement spectrum is used to calculate the wall collapse capacity and the acceleration spectrum has no influence. The displacement spectra for the Gisborne earthquake show peak displacements at periods of about 0.75 seconds and 1.5 seconds for the N51W and S39W components respectively. The effective period of a 220mm thick wall element with h/t_{nom} of 5, rocking about a base crack, is about 1.75 seconds (when calculated using equation (7)) so that, for more slender wall elements, the displacement spectra have already peaked. Hence the predicted collapse capacity (i.e. EQ scaling factor corresponding to a low probability of collapse) tends to increase with increasing slenderness as indicated in Figure 5.

Figure 5 also indicates that the typical 220mm thick (or thicker) walls of masonry buildings in the Gisborne CBD would have been predicted to be stable (i.e. have an earthquake scaling factor corresponding to a low probability of collapse greater than 1.0) if they had responded as free standing cantilever walls rocking about a crack opening and closing at the top of the foundation level. This type of response would be expected if the floor and roof diaphragms of a building were not strong or stiff enough to significantly influence to response of the walls.

A walk around of the CBD suggested to the Author that this is how many of the buildings in the CBD did respond to the earthquake. Evidence could be seen of vertical cracks that had formed at the exterior longitudinal and transverse wall junctions. These probably formed when the walls, only weakly restrained by flexible roof and/or floor diaphragms, bowed out under face loads. If a horizontal crack did not form at the support level of the parapets during this type of response, the parapets would have been stable along with the rest of the wall responding as a rocking cantilever. However, in other buildings, with stiffer and stronger roof and floor diaphragms, the earthquake motion at the support level of the parapets would have been amplified by the building response increasing the probability that the parapets would collapse.

3.3 Predicted Performance of Parapets in Gisborne Earthquake

Table 2 gives the amplification factors expected at the support level of parapets. They were derived by comparing the predictions made by time-history analyses and the Author's procedure and form part of the Author's parapet assessment procedure. These amplification factors may be thought of as the increase in "demand" imposed at the support level of a parapet due to the building response. This increased demand is relative to the ground motion intensity that would be imposed on a similar free-standing wall element resting on the ground. These increased demand values of 2, for parapets on single storey buildings, and 3 for multi-storey buildings, are plotted in Figure 6. The smoothed "capacity" curve for 220 thick walls elements subjected to the N51W component of Gisborne earthquake presented in Figure 5 is also reproduced in Figure 6.

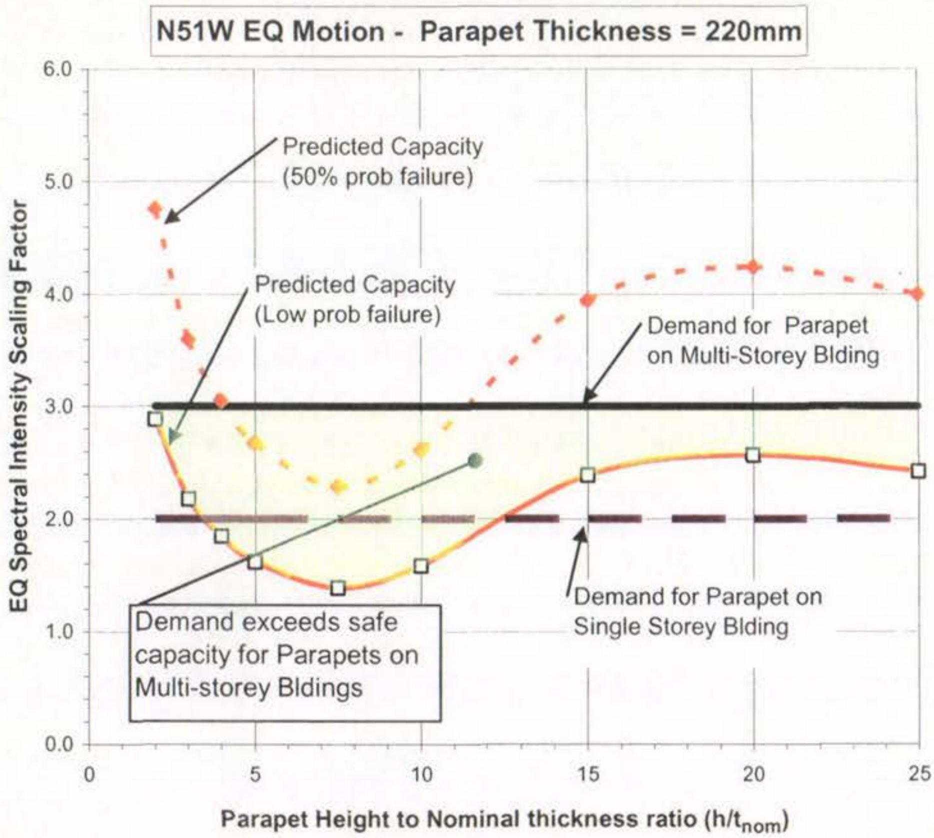


Figure 6: Capacity of 220mm thick parapets of varying slenderness compared with the amplified demand expected for parapets on single and multi-storey buildings. "Capacity" and "Demand" are as predicted by Author's Procedure and expressed as scaling factors for the N51W component of the Gisborne EQ (2ZG EQ record).

It can be seen from Figure 6 that a parapet wall element with an h/t_{nom} of 7.5 would be predicted to have a low probability of collapse if subjected to an earthquake motion with spectral response intensity about 1.5 times that of the N51W component of the Gisborne earthquake ground motion. However for a parapet supported by a reasonably stiff and strong roof diaphragm, the amplified demand at the parapet support level is expected to be 3 times the spectral response intensity of the ground motion. The shaded zone in the plot indicates that, for all parapets with an $h/t_{nom} > 2$ on multi-storey buildings, the demand is predicted to exceed the safe capacity for the N51W component of the earthquake. The plots also indicate that for a parapet h/t_{nom} between about 3.5 and 12.5 some parapets on single storey buildings would be predicted to collapse.

Inelastic time-history analyses carried out as part of previous research (Blaikie, 2005) indicated that when the earthquake spectral response intensity is increased about 65% above the level where there is a low probability of parapet collapse, approximately 50% of the parapets modelled collapse. This increase was used to plot the capacity curve in Figure 6 corresponding to a predicted 50% probability of parapet collapse. It can be seen that the "demand" scaling factor of 3 plotted for parapets on multi-storey buildings exceed this 50% probability capacity curve for parapet slenderness, h/t_{nom} between about 4 and 12. Hence the procedure predicts more than 50% of parapets in this slenderness range would collapse.

It may reasonably be assumed from the results of previous studies that, if the spectral response intensity of the earthquake motion was increased by a further 65% above the level resulting in 50% probability of collapse, all parapets would be predicted to collapse. In this case it can be seen that, for the most vulnerable parapets with an h/t_{nom} of 7.5 on multi-storey buildings, the predicted probability of collapse would have been close to 100% (i.e. almost all parapets of this slenderness are predicted to have collapsed). This procedure can be used to give an approximate prediction of the probability that a given parapet on a building in the Gisborne CBD would have collapsed during the Gisborne earthquake. This procedure was used to obtain an approximate predicted probability of collapse for the parapets surveyed in the CBD that were surveyed in detail and the results are presented in section 8.4.

Figure 7 presents the "capacity" and "demand" predictions for 220mm thick parapets orientated normal to the S39W component of the Gisborne earthquake (i.e. parapets parallel to the main street, Gladstone Rd). The results are similar to those presented for the orthogonal N51W component presented in Figure 6 for the slenderness range of interest for parapets which would typically be up to an h/t_{nom} of 10. However, the least stable parapets are predicted to be those with an h/t_{nom} of 5 compared with 7.5 for the orthogonal component. It can also be seen that the shaded zone in Figure 7, where the demand on parapets in multi-storey buildings exceeds the safe capacity, is more peaked than for the orthogonal component of the Gisborne earthquake.

Parapets with an h/t_{nom} of 5, are predicted in Figure 7 to be stable with a low probability of failure when subjected to approximately 1.5 times the spectral shaking response of the S39W component (i.e. have a 1.5 scaling factor "capacity"). This capacity can be compared with the amplified response of 2.0 and 3.0 times the ground level spectral response expected at the support level of the parapets on single and multi-storey buildings respectively (i.e. a 2 to 3 "demand" scaling factor). It can also be seen that, for these parapets, the demand on multi-storey buildings (scaling factor of 3) also exceeds the spectral response intensity where 50% of parapets are predicted to collapse (i.e. at capacity scaling factor of about 2.5). Comparing the relative positions of the curves when the h/t_{nom} is 5 indicates that approximately 75% of these Gisborne parapets parallel to the main street would be predicted by the Author's procedure to have collapsed on multi-storey buildings while 25% are predicted to collapse on single storey buildings.

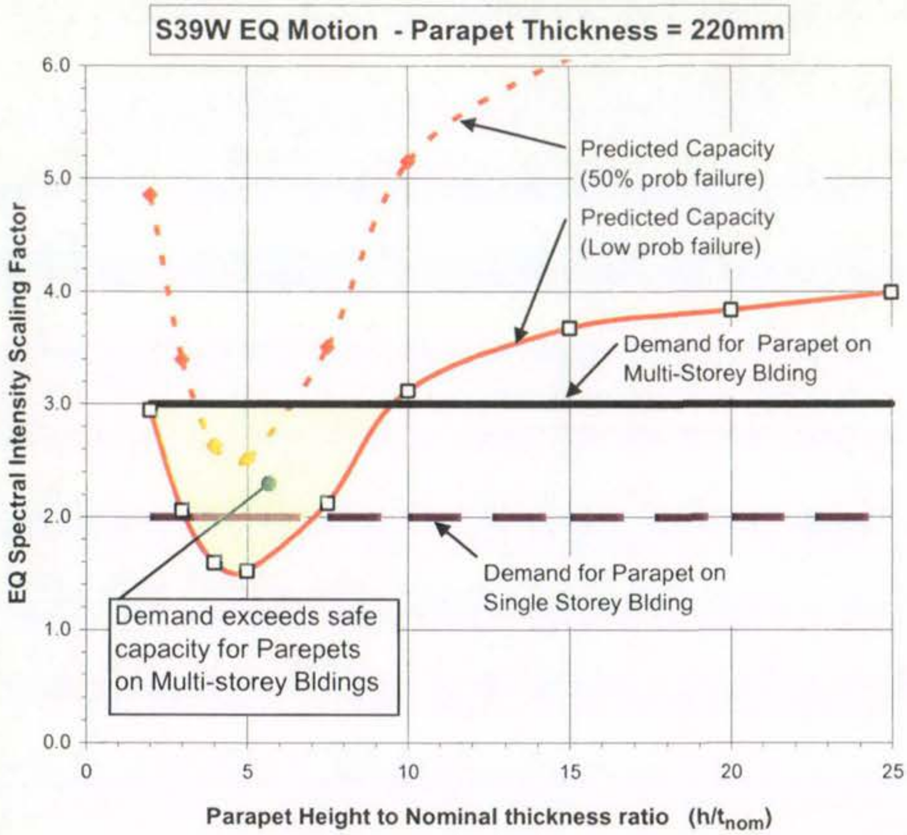


Figure 7: Capacity of 220mm thick parapets of varying slenderness compared with the amplified demand expected for parapets on single and multi-storey buildings. "Capacity" and "Demand" are as predicted by Author's Procedure are expressed as scaling factors for the S39W component of the Gisborne EQ (2ZG EQ record).

4 Predicted Gisborne Parapet Performance – NZSEE Procedure

4.1 Floor Response Spectra

As described in section 2.5, a major difference between the Author's procedure for assessing parapets and that adopted for the NZSEE Guidelines is the shape of the floor response spectra assumed in the analysis procedure. The Author assumes that the spectral response at the support level of a parapet (i.e. the floor response spectra) will be ground motion multiplied by a building amplification factor. However, the NZSEE procedure assumes that the floor response spectra depend only on the peak ground acceleration. It then applies a building amplification factor that is more closely dependent on the height of the parapet support level above the ground.

Figure 8 shows the smoothed acceleration and pseudo displacement response spectra for N51W component of 2007 Gisborne Earthquake ground motion recorded at the 2ZG site in Gisborne CBD. Floor response spectra derived from the peak ground acceleration of the N51W record, as used for the NZSEE procedure and before applying the building amplification factor, are also shown. Similar spectra for the S39W component of the earthquake are presented in Figure 9.

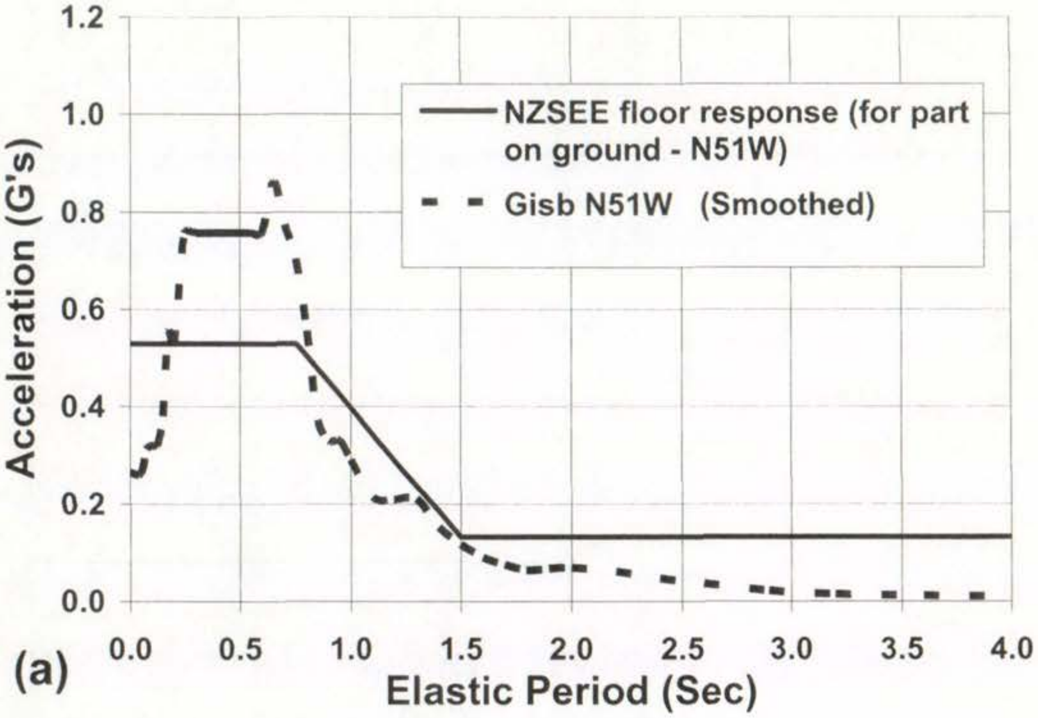
4.2 Predicted Performance of Parapets in Gisborne Earthquake

Figure 10 shows the capacity, with a low probability of collapse, of 220mm thick parapets of varying slenderness as predicted by the NZSEE procedure. As in the Author's procedure, the probability of collapse was assumed to increase from a low value to 50% when the scaling factor applied to the displacement spectra used for the "capacity" analysis is increased by 65%. The resulting capacity corresponding to a predicted 50% probability of collapse is also plotted in Figure 10.

The amplified demand expected for parapets on various height buildings are also plotted.

In the plots "capacity" and "demand" are expressed as spectral intensity scaling factors for the ground level floor displacement response spectrum given in Figure 8. As noted above, this spectrum was based on the peak ground acceleration in the N51W component of the 2ZG Gisborne EQ record.

The "demand" scaling factors plotted are the amplification factors that allow for the increasing floor response with increased elevation in a building as proposed by the NZSEE parapet assessment procedure and calculated using equation (17). Therefore, these amplification factors represent the increased "demand" in spectral intensity expected at the support level of the parapets when compared with the floor response spectra at ground level. The spectral intensity scaling factors that correspond to this amplified demand varies between 1.5 for parapets supported only 3m above ground to 3.0 for parapets supported 12m or more above ground level and are also plotted in Figure 10.



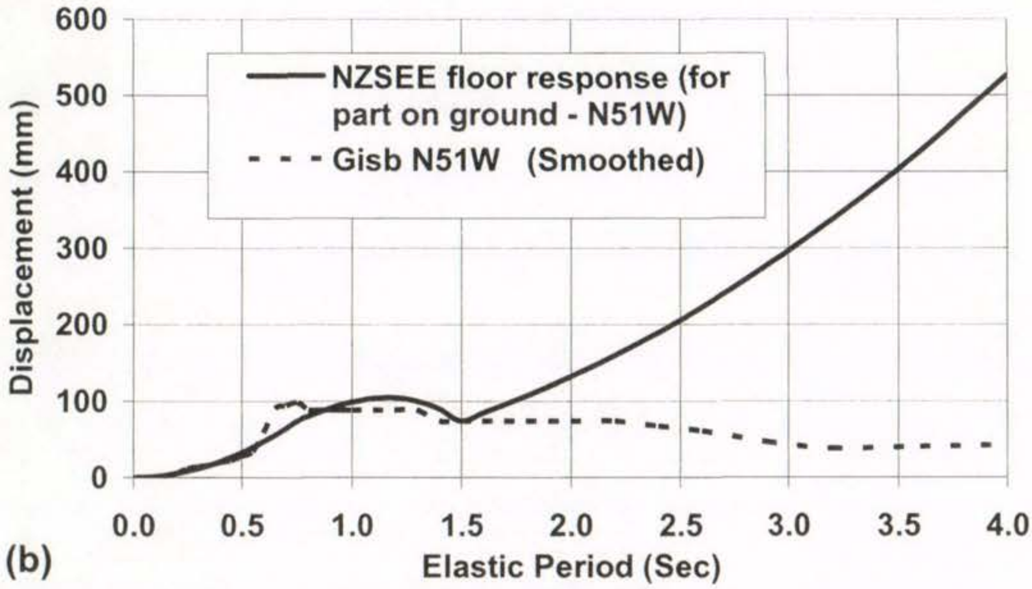
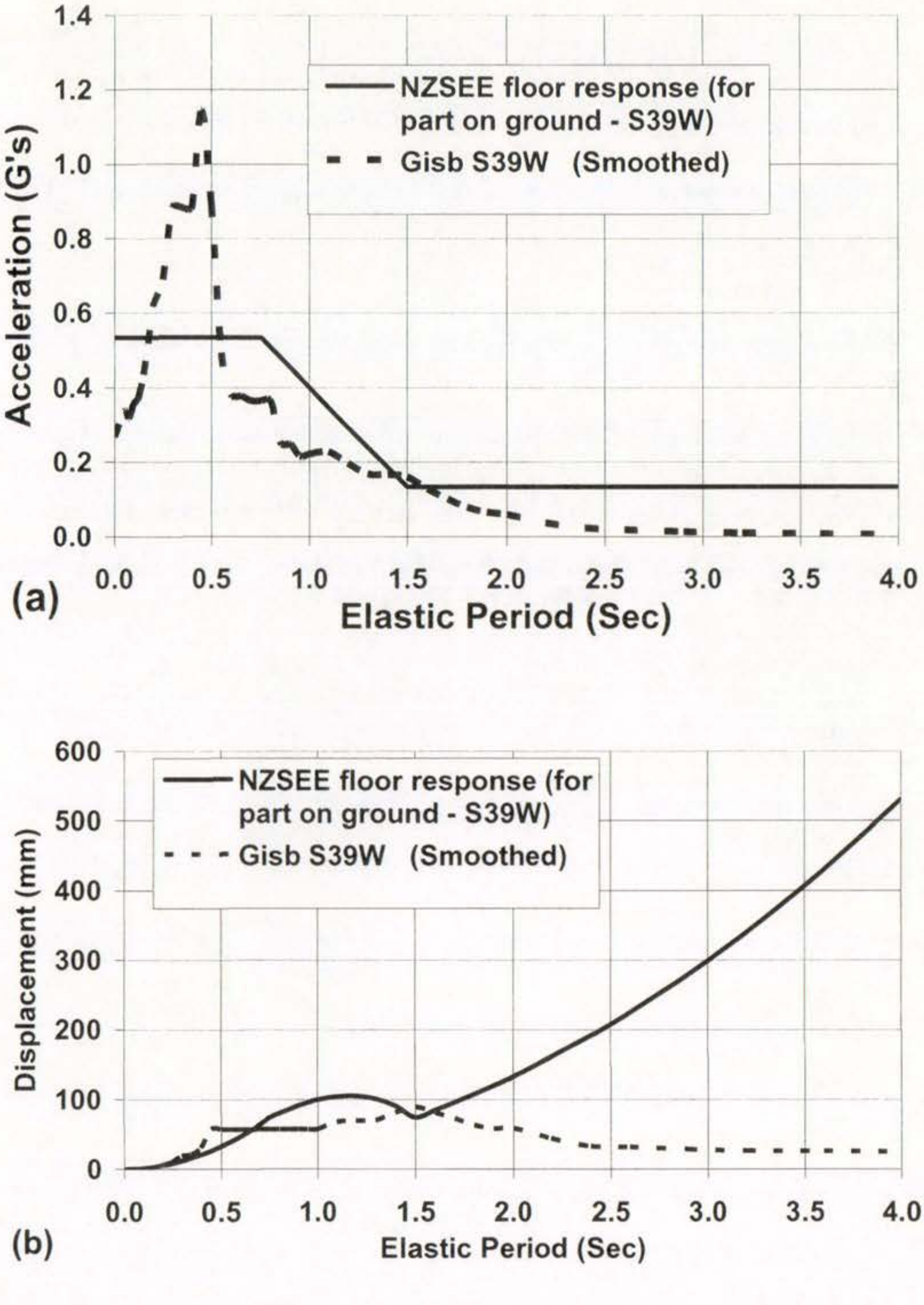


Figure 8: (a) Acceleration and (b) Pseudo displacement (smoothed) response spectra for the N51W component of the 2007 Gisborne Earthquake ground motion recorded at 2ZG in the Gisborne CBD. Floor response spectra at ground level, derived from the peak ground acceleration of the N51W record, as used in the NZSEE procedure, are also shown.



(a)

(b)

Figure 9: (a) Acceleration and (b) Pseudo displacement (smoothed) response spectra for the S39W component of the 2007 Gisborne Earthquake ground motion recorded at 2ZG in Gisborne CBD. Floor response spectra at ground level, derived from the peak ground acceleration of the S39W record as used in the NZSEE procedure, are also shown.

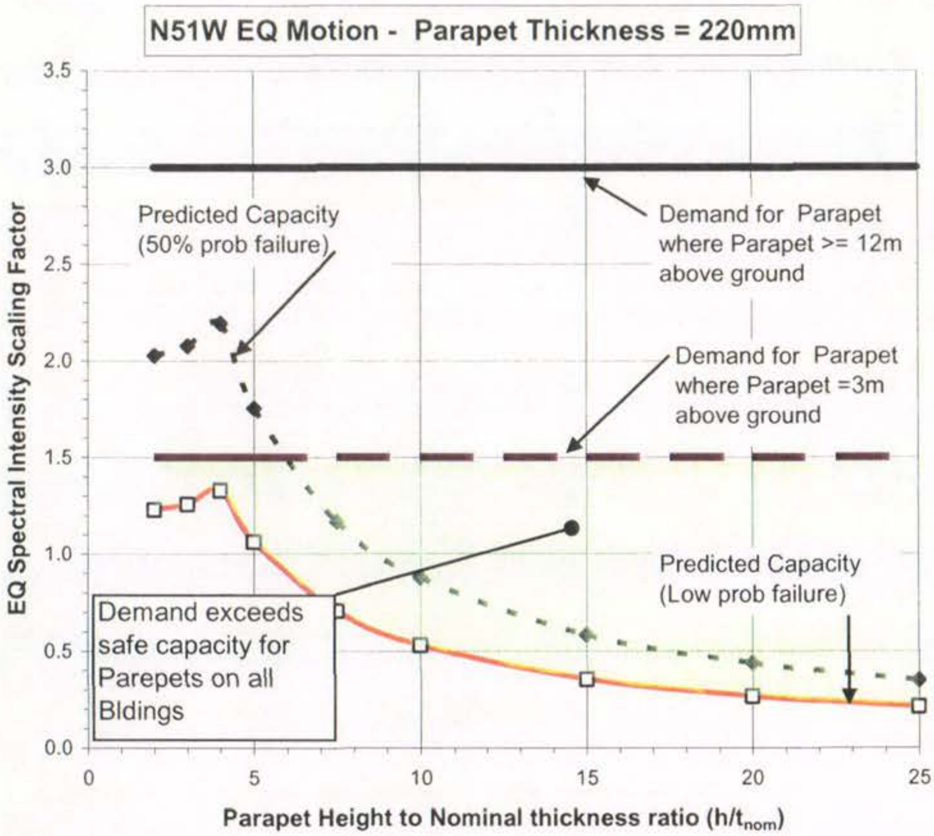


Figure 10: Capacity of 220mm thick parapets of varying slenderness compared with the amplified demand expected for parapets on various height buildings. "Capacity" and "demand" are as predicted by NZSEE Procedure and are expressed as scaling factors for the ground level floor response spectra. Amplification of "demand" due to building response was based on 5% damping.

It can be seen from Figure 10 that a parapet wall element with an h/t_{nom} of 5 would be predicted to have a low probability of collapse if subjected to an earthquake motion equal to the ground floor level displacement spectral intensity derived using the NZSEE procedure (i.e. EQ Spectral Intensity Scaling factor approx 1.0). However for a parapet supported by a roof diaphragm with significant strength and stiffness, the amplified demand at the parapet support level is expected to be 1.5 to 3 times the spectral response of the ground motion depending how high the parapet support level is above ground level. Therefore, for a parapet with this slenderness, the demand exceeds the capacity and a high probability of collapse is predicted, particularly when the support level for the parapet approaches the upper 12m limit above ground level.

The shaded zone in the plot indicates that the NZSEE procedure predicts that all 220mm thick parapets in the Gisborne CBD would have their safe capacity exceeded for the N51W component of the earthquake and that more slender parapets are predicted to be less stable.

The demand amplification factor plotted in Figure 10 varies between about 1.5 (for h_i = 3.0m) and 3.0 ((for h_i > 12.0m) with linear interpolation in between. However, for URM buildings the NZSEE Guidelines allow 15% damping to be assumed for the building response when assessing the face-loaded performance of URM walls. This greater damping would reduce these height amplification factors to about 2/3 of the above values (i.e. to 1.0 & 2.0 respectively). Figure 10 is re-plotted in Figure 11 with these reduced "demand" amplification factors.

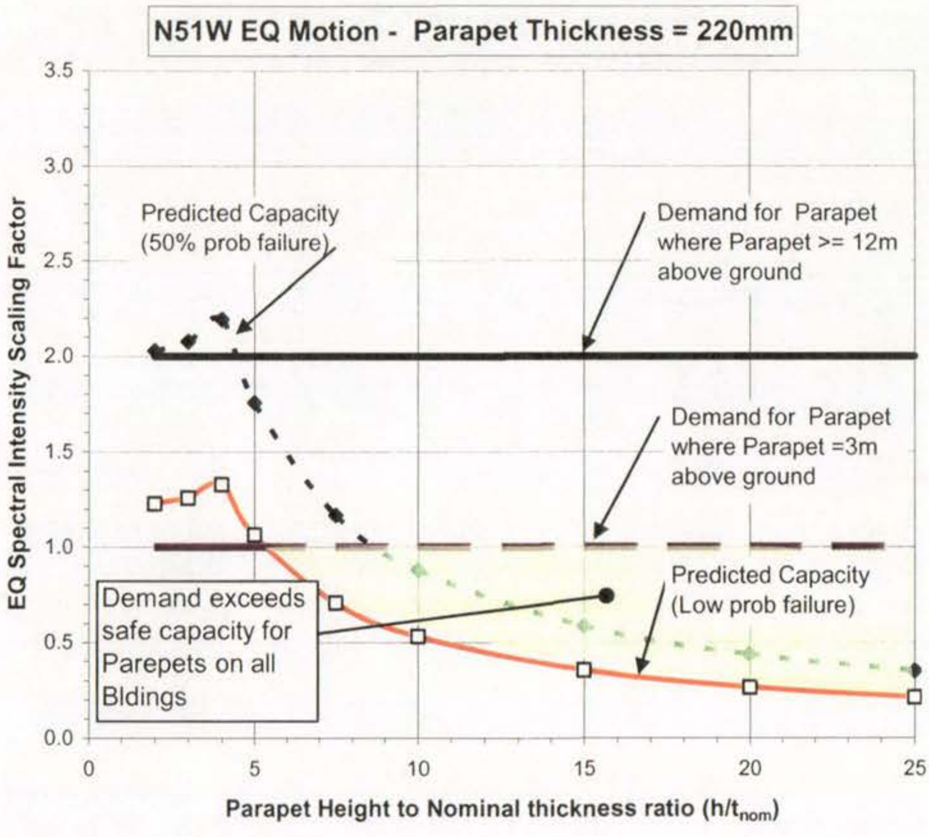


Figure 11: Capacity of 220mm thick parapets of varying slenderness compared with the amplified demand expected for parapets on single and multi-storey buildings as predicted by the NZSEE procedure. Amplification of "demand" due to building response based on 15% damping.

Figure 11 indicates that parapets with a h/t_{nom}< 5 would be predicted by the NZNSEE procedure to be stable if it was supported only about 3m above ground level but would have more than 50% probability of failure if supported 12m above ground level. The stability of more slender parapets with a $h/t_{nom} > 4$ are predicted to have rapidly declining stability up to the upper slenderness limit of about 10 applicable for typical 220mm thick parapets.

It is also of interest to note that Figure 11 indicates that free standing walls with $h/t_{nom} > 5$, such as exterior walls with little effective restraint from roof and floor diaphragms, are predicted not to be stable in the Gisborne earthquake (i.e. they have an EQ spectral intensity scaling factor less than 1.0)

5 EQ Prone Parapets in Gisborne CBD – Author's Procedure

5.1 Earthquake Prone Parapets

In New Zealand the Building Act defines buildings that are likely to collapse and cause loss of life or injury in a moderate earthquake as "earthquake-prone". Local Authorities in New Zealand can require earthquake-prone buildings to be strengthened. The regulations under the Act further define a moderate earthquake as an "earthquake that would generate shaking at the site of the building that is of the same duration as, but that is one-third as strong as the earthquake shaking (determined by normal measures of acceleration, velocity and displacements) that would be used to design a new building at the site".

The NZSEE has interpreted "likely to collapse" as meaning "could well occur". The "could well occur" risk level probably lies closer to the Author's "low probability of collapse" limit while the "likely to collapse" risk level tends to imply a "50% probability of collapse" risk level.

The Gisborne earthquake presents two interesting questions; were the parapets in Gisborne earthquake-prone and have those parapets that survived the earthquake been "proof loaded" to a higher earthquake load level than the moderate earthquake defined by the regulations?

In this context the requirement that the moderate earthquake has the same duration as the design earthquake, which is three times larger, is relevant as larger earthquakes tend to have a longer interval of strong shaking. Inelastic dynamic time-history analyses of face-loaded URM wall models have shown that the stability of cracked URM wall elements are primarily dependant on the spectral intensity of the ground motion used for the analysis. Using duration of shaking that is, say, twice as long in the analyses will have a similar effect as repeating the analysis with another earthquake record with the same spectral intensity. As the wall element may survive one of these analyses and collapse in the other, doubling the duration of the earthquake motion but not changing its spectral intensity is only likely to increase the risk of collapse but only by a moderate amount. However, when the uncertain meaning of "likely to collapse" is considered the duration of the moderate earthquake is not considered to be a significant issue.

In this study parapets are assumed to be earthquake-prone if they exceed the Author's "low probability of failure" risk level.

5.2 Gisborne Parapets - Earthquake-prone Prediction Using Author's Procedure

Figure 12 shows the acceleration and displacement (not smoothed) response spectra for N51W component of 2007 Gisborne Earthquake ground motion recorded at 2ZG in Gisborne CBD. Also shown are the spectra for 1/3 of the design earthquake that would be used for a new building in the Gisborne CBD which is a soft soil site (category D). These one-third design level response spectra are the spectral intensity that would be used to determine if an exiting building in the Gisborne CBD is earthquake-prone.

These two sets of spectra were used in the Author's procedure to obtain the scaling factors would need to be applied to the spectral intensities to give a predicted low probability of collapse of free standing 220 mm thick walls of varying slenderness. The results of these analyses are presented in Figure 13. Also shown are the similar predictions when 1/3 of the Design earthquake spectral intensity for rock and shallow soil sites are used in the procedure assuming such sites existed in the Gisborne CBD.

The analysis results presented in Figure 13 are for wall elements supported on the ground. However, in the Author's procedure it is assumed that the ground motion will be amplified by a factor of 2 and 3 at the support level of the parapets on single and multi-storey buildings respectively.

It can be seen from Figure 13 that a parapet wall element with an h/t_{nom} of about 3 would be predicted to have a low probability of collapse if supported on a multi-storey building where it would be exposed to 3 times the ground shaking intensity (i.e. corresponding to an EQ scaling factor of 3.0). Squatter parapets would, therefore, not be considered earthquake-prone. However most of the Gisborne parapets would be more slender than this and be classified as earthquake-prone using the Author's procedure. Similarly for parapets on single storey buildings, where the parapets are assumed to be exposed to twice the ground shaking intensity, parapets more slender than those with an h/t_{nom} of 4 would be classified as earthquake prone.

In the zone where the parapets are not considered to be earthquake-prone (i.e. $h/t_{nom} < 4$) it can be seen that a lower spectral scaling factor is required to cause collapse when the N51W component of the Gisborne earthquake is used in the analysis than when 1/3 the Gisborne design earthquake is used. This indicates that these parapets were not "proof loaded" to earthquake-prone level by this earthquake but they would still not be considered earthquake prone using the Authors procedure.

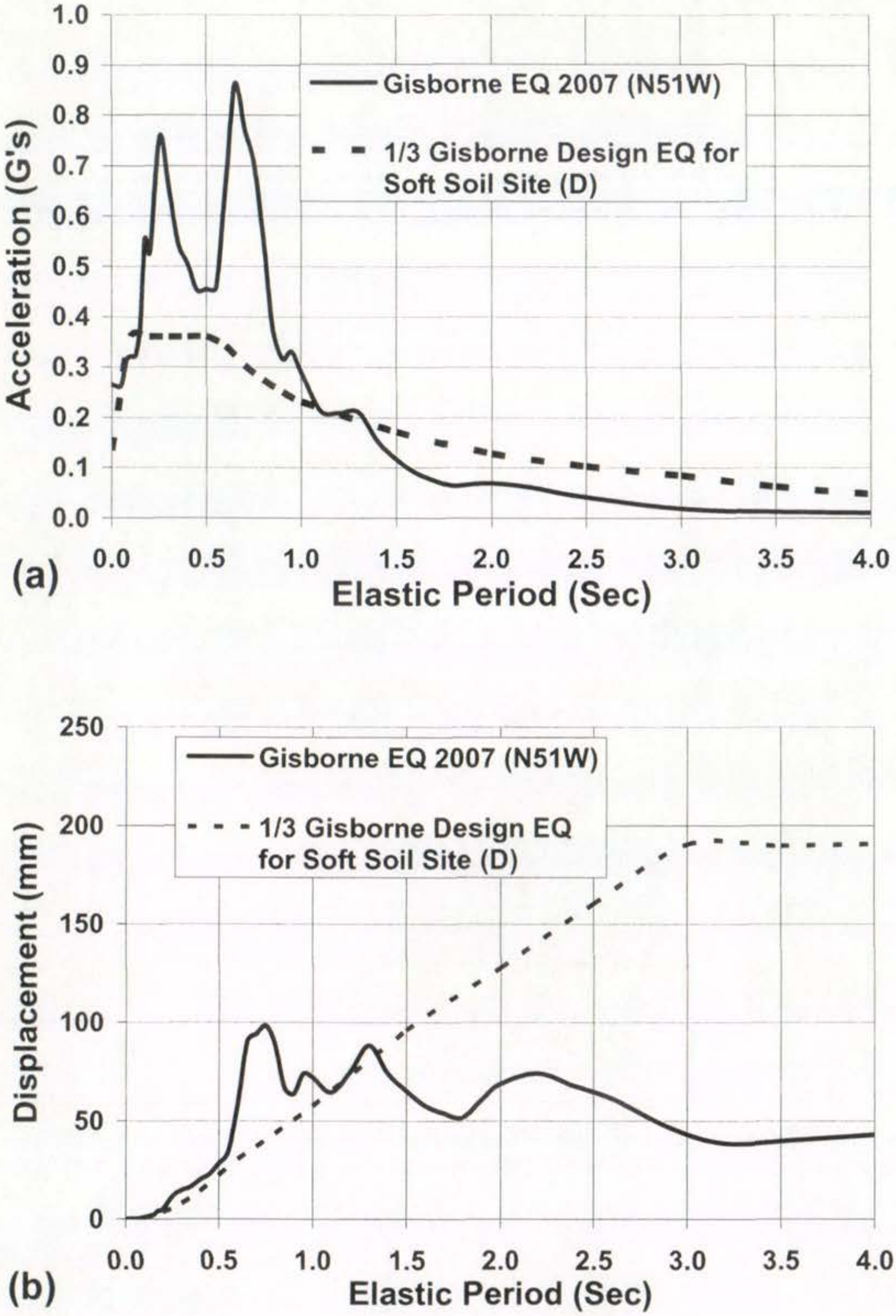


Figure 12: (a) Acceleration and (b) Pseudo displacement (not smoothed) response spectra for N51W component of 2007 Gisborne Earthquake ground motion recorded at 2ZG in Gisborne CBD. Also spectra for 1/3 of the design earthquake that would be used for new buildings in the Gisborne CBD that is used to determine if exiting buildings are EQ-Prone.

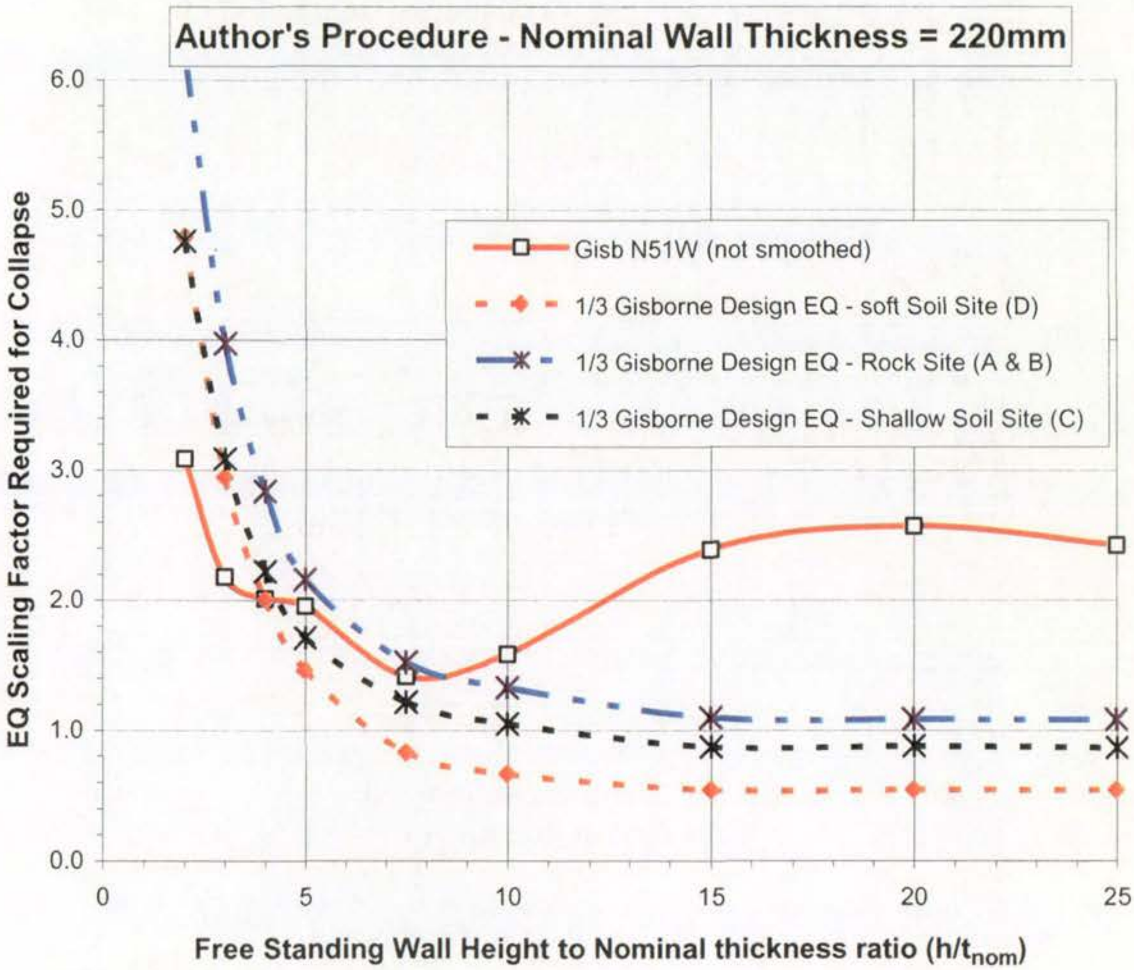


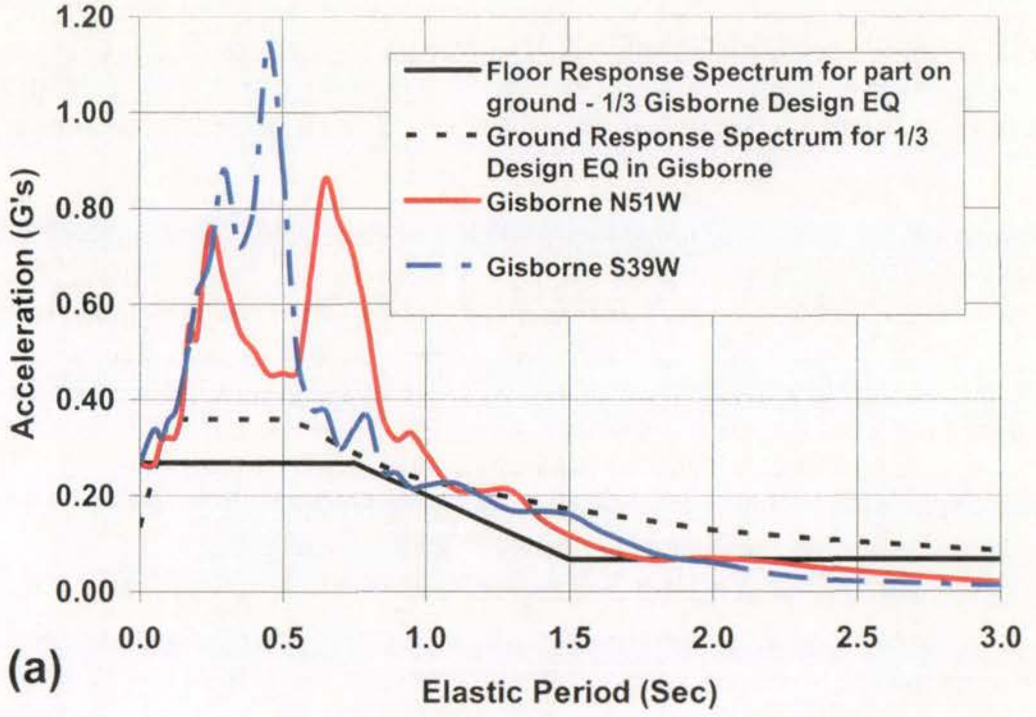
Figure 13: EQ Scaling factor that Author's procedure predicts would need to be applied to Gisborne EQ motion (N51W component of 2ZG record) to result in a low probability of collapse of free standing 220 mm thick walls of varying slenderness. Also shown are the similar predictions when 1/3 of the Design EQ spectral intensity for the Gisborne CBD is used in the procedure.

6 EQ Prone Parapets in Gisborne CBD – NZSEE Procedure

6.1 Gisborne Parapets - Earthquake-prone Prediction Using NZSEE Procedure

Figure 14 shows the acceleration and pseudo displacement response spectra (not smoothed) for the two component of 2007 Gisborne Earthquake ground motion recorded at 2ZG in Gisborne CBD. The spectra for 1/3 of the soft soil (Category D) design earthquake that would be used for a new building in the Gisborne CBD and the ground level floor response spectra derived from the peak ground acceleration value of the 1/3 design earthquake acceleration spectrum are also shown . The one-third design level response spectra would be the spectral intensity that would be used to determine if an exiting building in the Gisborne CBD is earthquake-prone. However when using the NZSEE procedure, only the floor response displacement spectrum for a "part on the ground" is used to access whether a parapet in the CBD is earthquake-prone.

The ground level floor displacement spectrum was used in the NZSEE procedure to obtain the scaling factors would need to be applied to the spectral intensity to give a predicted low probability of collapse of free standing 220 mm thick walls of varying slenderness. The results of these analyses are presented in Figure 15. Also shown is the amplification of shaking intensity demand expected when using the NZSEE procedure for parapets supported at various heights above ground level when 15% building damping is assumed.



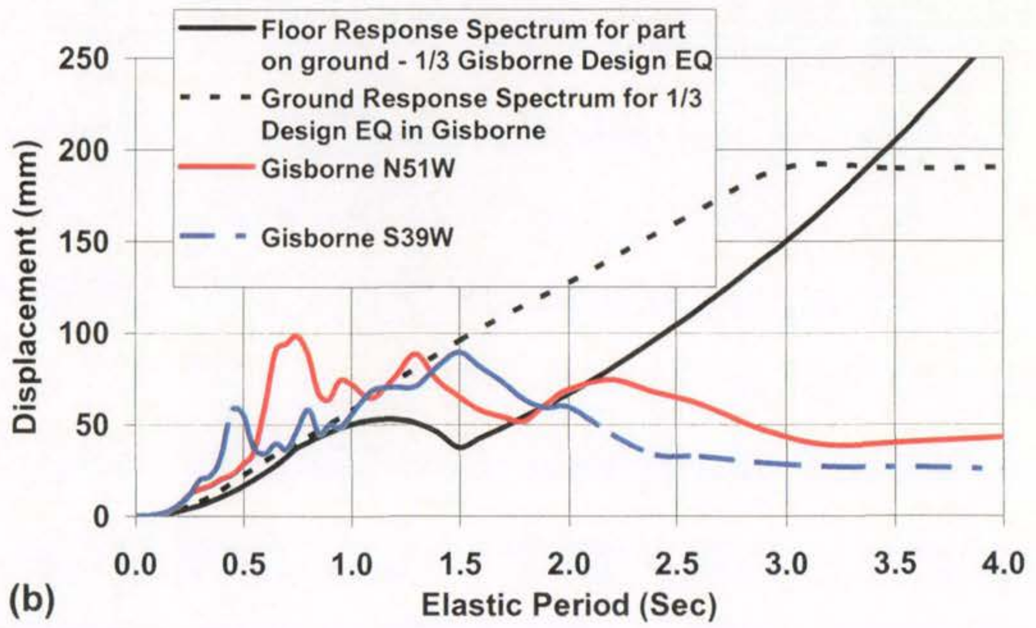


Figure 14: (a) Acceleration and (b) Pseudo displacement (not smoothed) response spectra for two components of 2007 Gisborne Earthquake ground motion recorded at 2ZG in Gisborne CBD. Also spectra for 1/3 of the design earthquake that would be used for new buildings in the Gisborne CBD. Only the floor displacement response spectra at ground level shown is used in the NZSEE procedure to determine if parapets are EQ-prone.

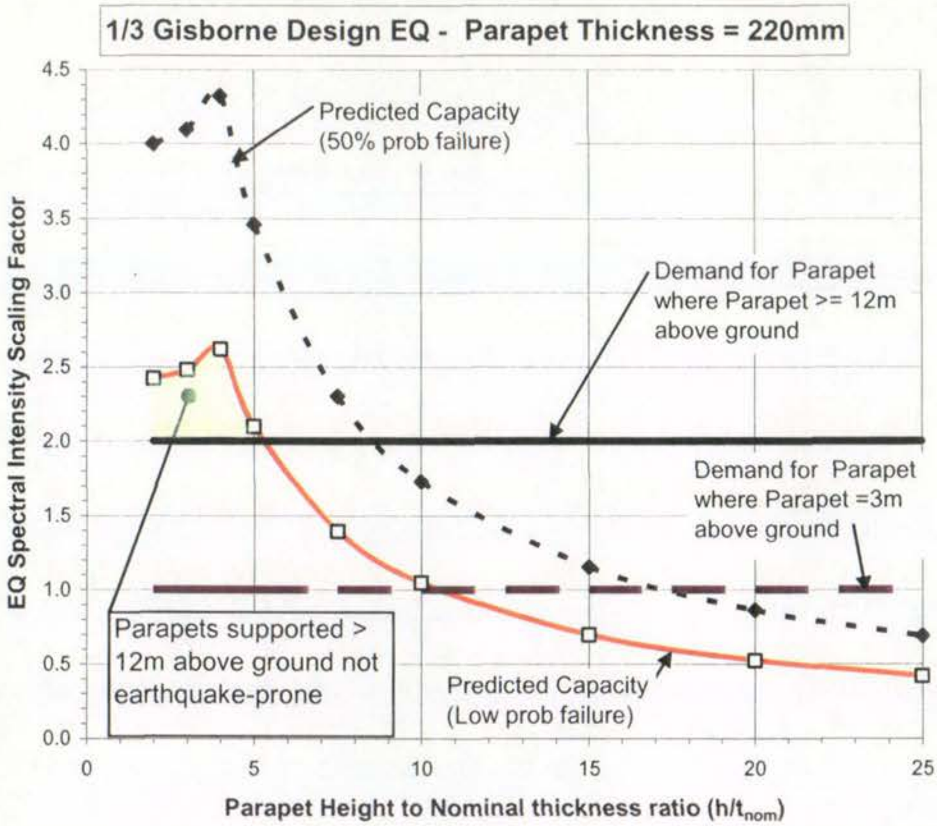


Figure 15: EQ Scaling factors that the NZSEE procedure predicts would need to be applied to the ground level floor displacement spectral intensity to result in a low probability, and a 50% probability, of collapse. Analysis was with 1/3 Gisborne Design EQ intensity for 220 mm thick parapets of varying slenderness. Also shown is the amplification of shaking intensity demand expected above ground level when using the NZSEE procedure and assuming 15% building damping.

In the shaded zone of Figure 15, the scaling factor that would need to be applied to the ground level floor displacement spectral intensity to have a low probability of collapse (i.e. the capacity) would be greater than the amplification of the same spectral intensity (i.e. the demand) predicted if the support level of a parapet is 12m or more above ground level. As the capacity exceeds the demand in this zone (i.e. where $h/t_{nom} < 5.5$), Gisborne CBD parapets in this zone would not be considered earthquake-prone when evaluated using the NZSEE procedure. Similarly parapets supported 3m above ground level,, would not be considered earthquake-prone when $h/t_{nom} < 10.5$.

It is also interesting to note that, in the NZSEE procedure with 15% building damping assumed, no amplification of the ground level spectral intensity is expected for parapets supported 3.0m above ground level (i.e. the "demand" EQ spectral intensity scaling factor = 1.0).

7 Differences in Author's and NZSEE Predicted Parapet Performance

7.1 Effect of Difference in Floor Response Spectra – Gisborne EQ Predictions

The performance of the Gisborne CBD parapets, normal to the main street, during the Gisborne 2007 earthquake as predicted using the Author's procedure is shown in Figure 6. If this is compared with the performance predicted using the NZSEE procedure, shown in Figure 11, it can be seen that the two predicted performances are significantly different.

Given that the NZSEE procedure is based on the Author's procedure, with what appears at first glance to be relatively superficial modifications, this difference in predicted performance is unexpected.

Unlike the Authors procedure, which uses both the acceleration and displacement spectra, the NZSEE procedure uses only the displacement spectra. The Author's procedure assumes that the spectral response at the support level of a parapet (i.e. the floor response spectra) will be scaled up version of the ground motion spectra with the scaling factor depending on whether the parapet is on a single or multi-storey building. However, the NZSEE procedure assumes that the floor response spectra at ground level depend only on the peak ground acceleration of the spectrum used in the analysis. It then applies a different scaling factor that is dependant on both the height of the parapet above ground level and on the assumed building damping.

It can be seen in Figure 8 and Figure 9 that the displacement spectra used in the two procedures diverge at a period of about 1.5 seconds which corresponds to the effective period of a 220mm thick parapet height of about 0.8 m (i.e. h/t_{nom} approx. 4). Higher parapets, with longer effective response periods calculated using equation (7), would therefore, be predicted to be less stable if the NZSEE floor response displacement spectrum is used in the analysis instead of the actual ground response displacement spectrum.

Differences in the earthquake scaling factor that would need to be applied to the floor and ground response spectra to result in a low probability of collapse of free standing walls as predicted by the NZSEE procedure are presented in Figure 16. Only the displacement spectra shown in Figure 8 and Figure 9 for the two components of the 2ZG record were used in the analyses to isolate only the effect of the difference in the shape of the two floor response displacement spectra assumed in the Author's and the NZSEE procedures. It can be seen that for, $h/t_{nom} > 4$, where the displacement spectra in Figure 8 and Figure 9 diverge, the predicted stability of the parapets also diverge as expected.

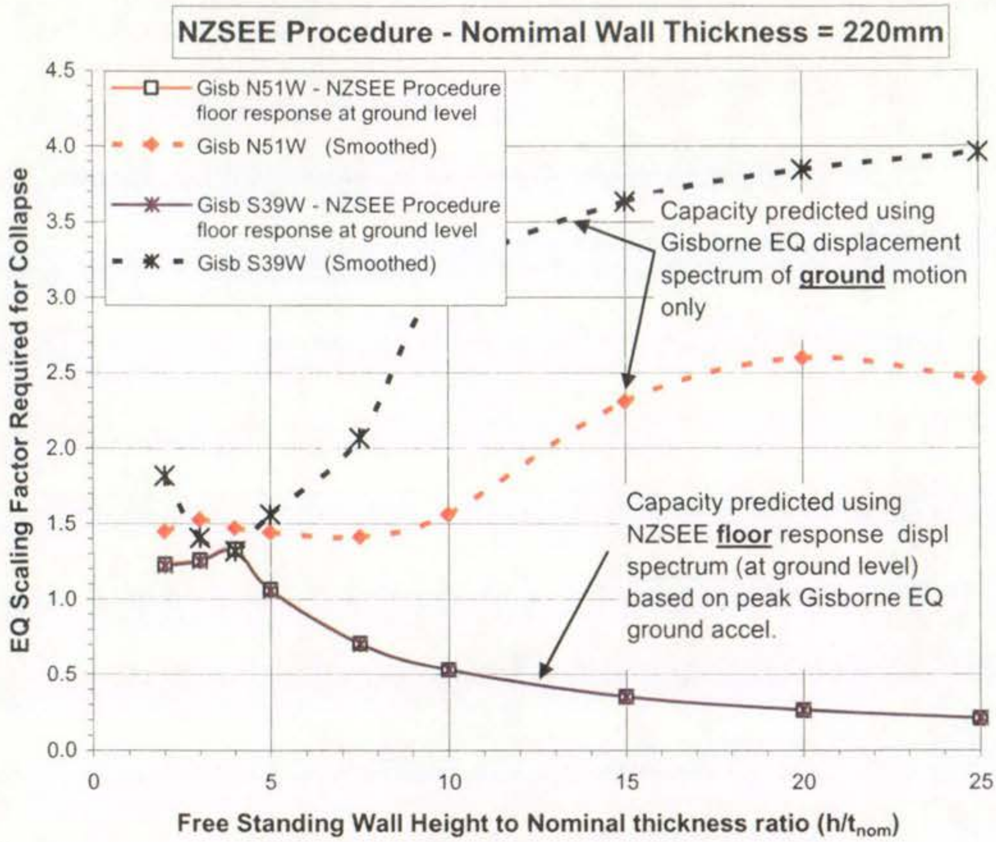


Figure 16: Differences in the EQ scaling factor that would need to be applied to the two components of the recorded Gisborne EQ to result in a low probability of collapse of free standing walls as predicted by the NZSEE procedure using either the displacement spectrum for the actual recorded ground motion or a displacement floor response spectrum based only on the peak ground acceleration of the recorded ground motion.

The peak recorded accelerations for the N51W and S39E components of Gisborne earthquake were very similar (0.265g and 0.267g respectively). Consequently the capacity of face-loaded wall elements evaluated using the NZSEE procedure, where the floor response is only dependant on the peak ground acceleration, was essentially the same for the two components of the Gisborne earthquake as shown in Figure 16.

7.2 Effect of Difference in Floor Response Spectra – Earthquake-prone Predictions

The predicted capacity curve (labelled "1/3 Gisborne Design EQ – soft soil site (D))" shown in Figure 13 was obtained using the Author's procedure that used both the acceleration and displacement spectra for 1/3 the design earthquake ground motion. This curve can be compared with the predicted capacity shown in Figure 15 using the corresponding floor response displacement spectra used in the NZSEE procedure and it can be seen that there are significant differences in the capacities predicted by the two procedures. These two capacity curves are reproduced in Figure 17 except that, in this case, the NZSEE "displacement spectra only" procedure was used for both curves, so that only the effect of the different displacement spectra used in the Author's procedure and in the NZSEE procedure could be isolated.

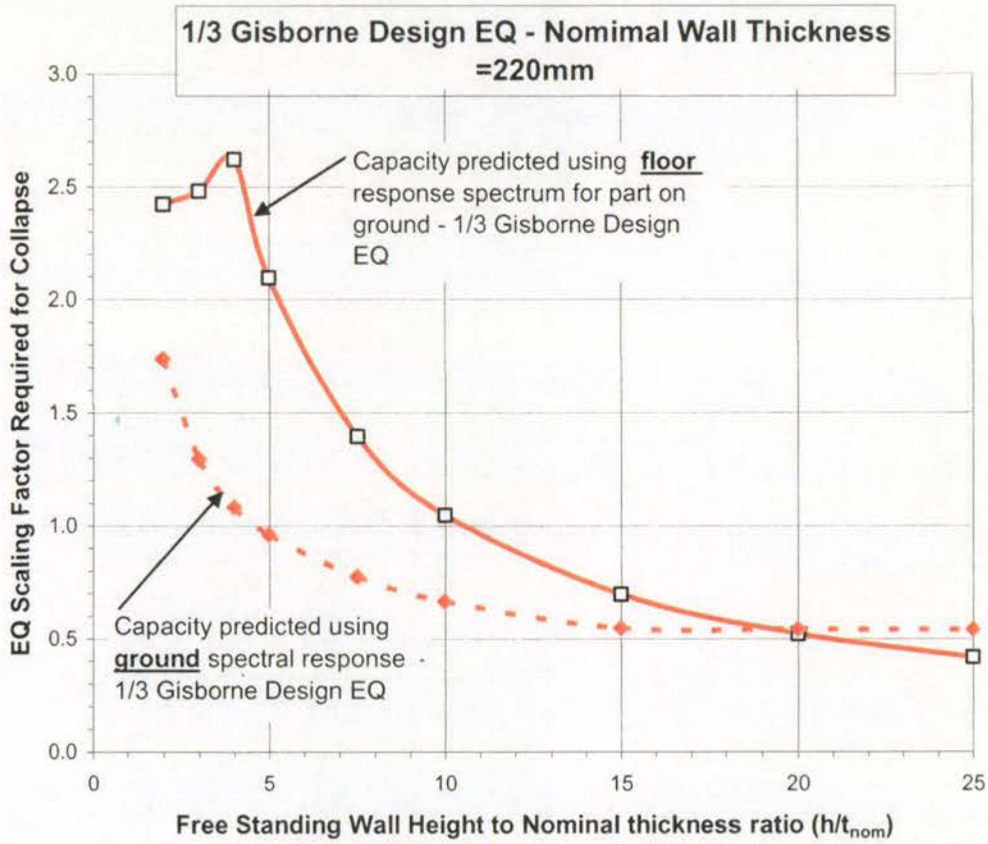


Figure 17: EQ Scaling factors that NZSEE "displacement spectra only" procedure predicts would need to be applied to the ground level floor displacement spectral intensity or ground displacement response spectral intensity, to result in a low probability of collapse. Analysis was with 1/3 Gisborne Design EQ intensity for 220 mm thick parapets of varying slenderness

The two displacement spectra used to produce Figure 17 are those given in Figure 14 (b) for "1/3 Gisborne Design EQ". It can be seen in Figure 14(b) that, for periods greater than 1.0 seconds, the ground response displacement spectrum (as used in the Author's procedure) has larger spectral displacements and is, therefore, more conservative than the ground level floor response spectra used in the NZSEE procedure. The effect of this conservatism on the scaling factor that needs to be applied to the spectral intensity to obtain a low probability of collapse for a wall element resting on the ground can be seen in Figure 17.

It is interesting to note that, if the Gisborne CBD had been founded on rock instead of soft soils, the divergence between the analyses results similar to those plotted in Figure 17 would have been minor compared with those shown in Figure 17 for a soft soil site. This is a direct consequence of the ground level floor and ground design displacement spectra shown Figure 18 being very similar in the period range 1.2 to 2.5 seconds which corresponds to the h/t_{nom} range of 2 to 10 of interest for 220mm thick parapets.

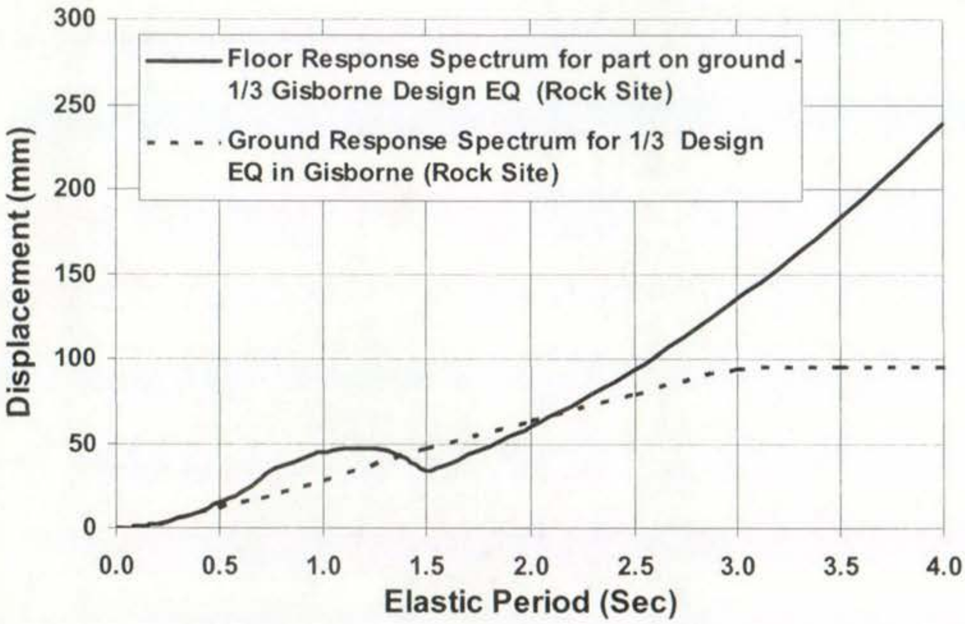


Figure 18: Displacement response spectra for 1/3 of the design earthquake that would be used for new buildings on a rock site in the Gisborne. Floor displacement response spectrum at ground level as used in the NZSEE procedure and the ground response displacement spectrum as used in the Author's procedure are shown.

From the above discussion, it may be concluded that the Author's procedure and the NZSEE procedure would give much more similar predictions of parapet stability for a rock site than for a soft soil site. This divergence in the predictions is a consequence of the NZSEE floor response spectra being only dependant on the peak ground acceleration which has less sensitivity to the ground conditions than the ground displacement spectra.

This is potentially a significant weakness in the modifications made to the Author's procedure for the NSEE Guidelines.

7.3 Effect of Using only the Displacement Spectra & formulae Modifications on Gisborne EQ Predictions

Two other significant modifications were made to the Author's procedure when developing the NZSEE procedure. The NZSEE procedure only uses the displacement spectrum instead of the acceleration and displacement spectra, and it modifies the formulae used to calculate the participation factor for the parapet and its effective period, T_p (see section 2.5). To isolate the effect of these modifications, parapets with a range of wall thicknesses and slenderness were analysed using the two procedures. With these changes the Author's original procedure is referred to as the Author's Modified Procedure.

The results of the analyses with and without these modifications for the two recorded components of the 2007 Gisborne earthquake are shown in Figure 19.

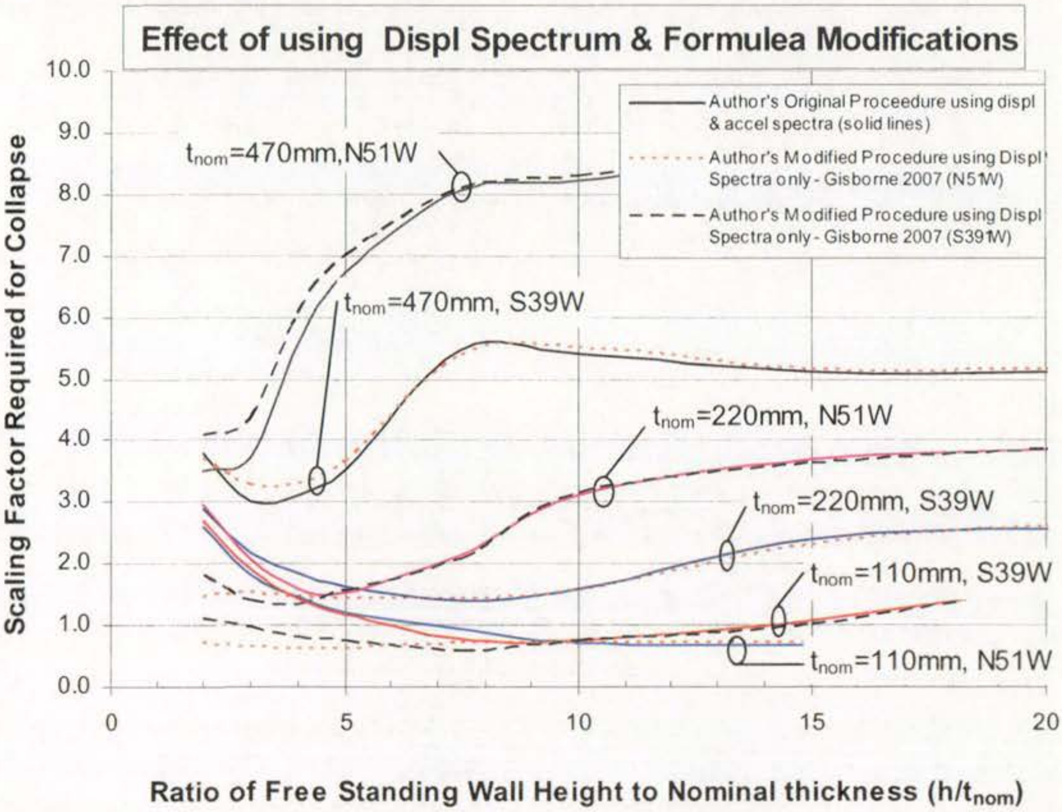


Figure 19: Effect of using the displacement spectra (as in the NZSEE procedure) instead of both the displacement and acceleration spectra combined with the effect of the NZSEE modifications made to the formulae used to calculate the participation factors and effective period T_p (i.e. Author's Modified Procedure). Analyses used smoothed spectra for the 2007 Gisborne Earthquake.

It can be seen from Figure 19 that the modifications will not have a significant effect on the assessed performance of most free-standing wall element resting on the ground (h/t_{nom}> 10). However, they will have a significant effect on the predicted performance of squat parapets (h/t_{nom}< 5). It can also be observed that the Author's original and modified procedures predicts that all 470mm thick Gisborne CBD parapets would be stable in the Gisborne earthquake as the maximum amplification of the ground motion expected at the support level of parapets would be 3.0 on multi-storey buildings which would just be exceeded for almost the full range of parapet slenderness.

Figure 19 also indicates that, for the a wall thickness of 470mm, the modifications result in a lower earthquake scaling factor required to produce a low probability of squat parapet collapse in the Gisborne earthquake and is, therefore, less conservative. This is because, for this wall thickness, the capacity predicted using the acceleration spectra in equation (11) does not govern in the Author's procedure and, therefore, only the displacement spectra is used as in the both the original and modified procedures. Consequently, the less conservative prediction of squat parapet performance in this case is due only to the revised formulae used in the modified procedure.

For the wall thickness of 110 and 220mm parapets, use of the acceleration spectra in the Author's original procedure enhances the predicted performance to a greater extent than the revised formulae used in the modified procedure. Therefore, for these wall thicknesses, the modifications are <u>more</u> conservative.

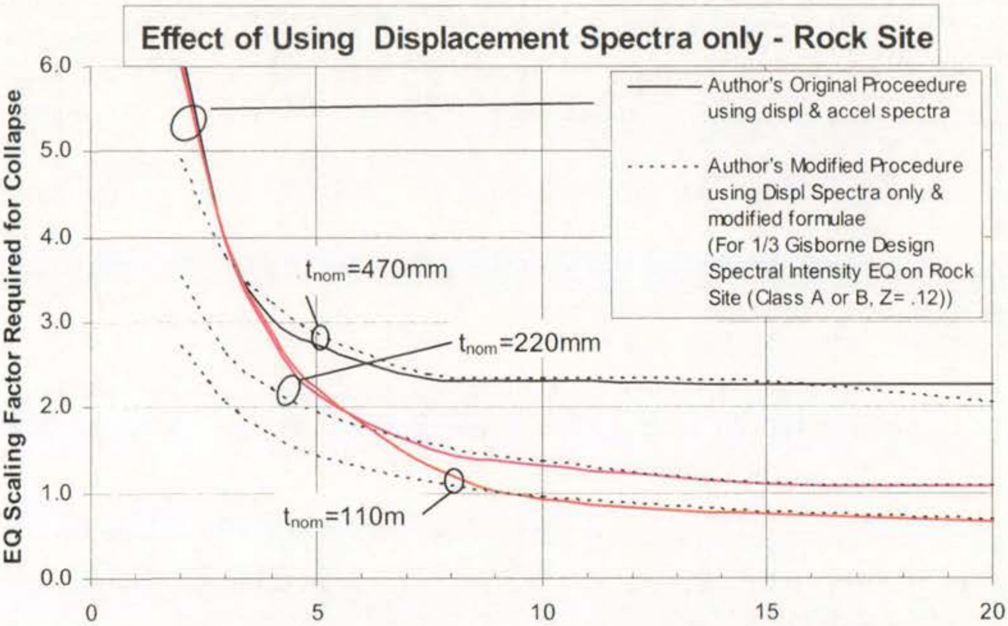
It may be concluded that, generally, including the parapet slenderness in the calculation of modal participation factor and effective parapet period in the Author's modified procedure reduces the conservatism of using only the displacement spectra in the modified procedure.

7.4 Effect of Using only a Displacement Spectra & Formulae Modifications on Earthquake-prone Predictions

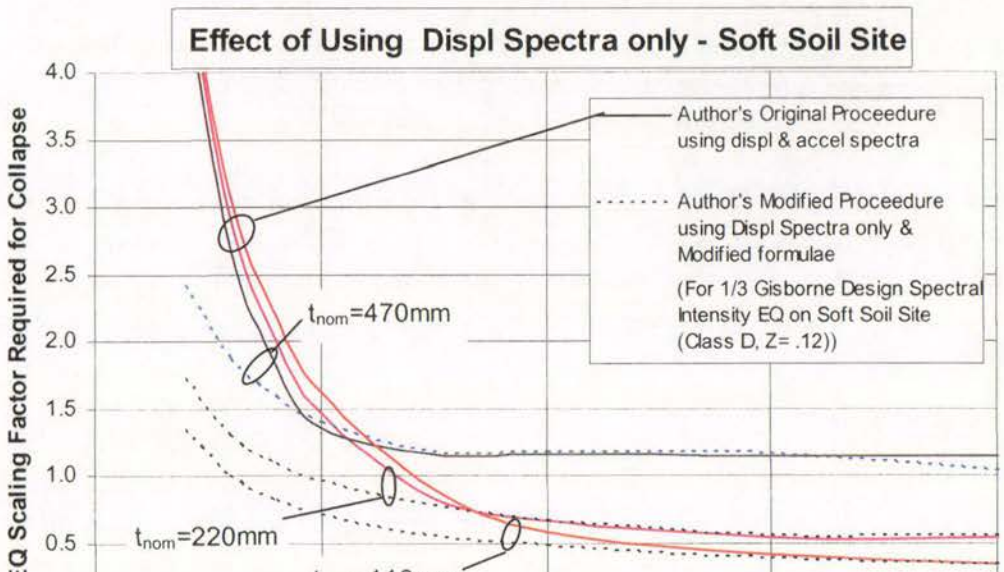
Similar analyses to those presented above in Figure 19 were carried out, except that the "1/3 Gisborne Design Earthquake" that would be used to determine if parapets are earthquake-prone, were used as the input spectra for the analyses with and without the modifications. Results of these analyses, using the ground response spectra shown in Figure 18 for a rock site, are shown in Figure 20(a). Similar results using the soft soil spectra given in Figure 14 are shown in Figure 20(b).

The analysis results indicate that, for Design Earthquake type spectral shapes, including the parapet slenderness in the calculation of the modal participation factor and effective wall element period combined with the effect of using only the displacement spectra, makes the Author's modified procedure more conservative than the Author's original procedure.

The results presented in Figure 20(a) for the Author's original procedure also indicates that, for a rock site in Gisborne (if there was one), parapets on multi-storey buildings would need to have $h/t_{nom} < 4$ if they were not to be considered earthquake-prone. These parapets would have the capacity to withstand more than 3 times the ground shaking intensity and would, therefore, be able to withstand the amplification of three times the ground motion expected at the support level of the parapets. On single storey buildings, where the ground motion amplification is expected to be 2, the 110 and 220mm thick parapets would need to have a h/t_{nom} < about 5.5 if they were not to be considered earthquake-prone. However the results indicate that all parapets more than about 300mm thick would not be considered earthquake-prone on single storey buildings.



(a) Ratio of Free Standing Wall Height to Nominal thickness (h/t_{nom})



(b) Ratio of Free Standing Wall Height to Nominal thickness (h/t_{nom})

Figure 20: Effect of using only the displacement spectra (as in the NZSEE procedure) instead of both the displacement and acceleration spectra. The effect the NZSEE modifications made to the formulae used to calculate the participation factors and effective period T_p were also included in the Modified Procedure. Analyses used 1/3 Gisborne Design Spectra for (a) a rock site and (b) a soft soil site.

Similarly for a soft soil site, Figure 20(b) indicates that using the Author's original procedure the parapets would need to have h/t_{nom} < about 3 on a multi-storey buildings and an h/t_{nom} < about 4 on a single-storey building not to be considered earthquake-prone.

Using the Author's modified procedure it can be seen from Figure 20(b) that only a few very squat parapets in the Gisborne CBD would be assessed as not being earthquake prone especially on multi-storey buildings where an implication of the ground motion of 3.0 is expected.

Figure 20(b) also indicates that freestanding walls, supported directly on the ground in the Gisborne CBD, with a thickness greater than about 400mm would have a corresponding "earthquake scaling factor" greater than 1.0. These walls would, therefore, have a low probability of not collapsing and not be considered earthquake-prone. This analysis would apply to perimeter walls of single storey buildings where the stabilizing effect of the roof diaphragm is only sufficient to compensate for the destabilising effect of the roof load being applied eccentrically to the inside face of wall.

Figure 19 indicates that similar walls only 100 thick would have been stable in the Gisborne earthquake, suggesting that such walls responding as vertical cantilevers were not proof-load tested to a high enough earthquake intensity to "prove" that they are not earthquake-prone.

8 Performance of Surveyed Parapets in 2007 Gisborne Earthquake

8.1 Parapet Survey

A survey was carried out of the parapets in the Gisborne CBD to determine the number of parapets exposed to the 2007 earthquake and the number that collapsed. The number with a visible horizontal crack near the support level of the parapet, which would indicate that the parapet had rocked back and forth significantly during the earthquake, were also estimated. Detailed data was then collected on a sample of the parapets with a primary emphasis on the parapets that had collapsed and a secondary emphasis on those that had had a visible horizontal crack. Detailed data was also obtained for other visibly undamaged parapets, often on the same building as the damaged parapets, which could be collected relatively easily and other buildings for which details were more readily available.

A summary of the best estimate of total number of parapets in the CBD in each category and the number for which detailed data was collected is shown in Table 3.

Table 3: Summary of Gisborne Parapets

Item	Total Number	No. Surveyed in Detail for this Study
Number of Parapets in Gisborne CBD	360	101
Number Partly or Totally Collapsed	22	19
Number of Parapets cracked horizontally*	80	39

* includes 2 parapets that partly collapsed

8.2 Detailed Surveyed Data for Gisborne Parapets

Sufficient detailed data was gathered for some of the parapets to enable them to be analysed using the assessment procedures so that their predicted earthquake performance could be compared with their actual performance in the Gisborne earthquake.

Table 5 (see Appendix A) shows the detailed data that was required for the analyses of the parapets and additional data gathered for some of the parapets is also shown in Table 6.

Aspects of the data in Table 5 to note are:

- The "parapet location" (e.g. "front", "right") is in relationship to an observer viewing the building from the street frontage and "central" indicates that the parapet is an intermediate parapet parallel to side walls
- "Normal Gisborne EQ comp" indicates the component of the 2007 earthquake that would have been acting approximately normal to the parapet.
- "Height to Parapet Support" is the level of the roof structure that supports the parapet. Where this varies the roof support is generally sloping. This height is used in the NZSEE procedure to derive the building height amplification factor.
- "Parapet Height above crack (or support)" is the parapet height, h, used to calculate the slenderness, h/t_{nom}, of the parapet. When two values are given the supporting roof is either sloping or the parapet is stepped in height. Where a visible horizontal crack formed or the parapet collapsed, the height given is the parapet height above the crack.
- If a greater "thickness", t_{nom}, is given for the "rib" than "wall" of the parapet the parapet has vertical ribs. The procedure used to evaluate parapets with ribs is given in section 8.3

- "Status" indicates whether the parapet has a visible horizontal crack near the support level of the parapet indicating significant rocking of the parapet and that the parapet was probably near to collapse. Often the cracks were visible from street level and other parapets may have had a visible crack if they were inspected closely. The status also indicates whether at least part of the parapet collapsed.

- "Valid parapet plot data" indicates whether sufficient details were obtained for the parapet to enable the stability to be evaluated and included in the plotted results of the analyses.

Aspects of the additional data in Table 6 to note are:

- The "Parapet location" and "side of street" data was used to determine which of the recorded components of the 2007 earthquake was approximately normal to the parapet.

- Data on spacing of ribs and end supports (i.e. return walls) was collected, where readily available, to help explain any potential anomalies in the analysis results.

- Presence of a "Recessed flashing at crack" was recorded, where known, as it had been suggested that the presence of a flashing had affected the stability of the parapets. The flashing was generally turned into a mortar joint and would have acted as a bond breaker that encouraged any horizontal failure crack to develop at the level of the flashing rather than at the roof support level. However, for the parapets with flashings, 7 collapsed outwards (away from the recess) and 6 inwards with one collapsing partially in both directions. This lack of preference in collapse direction suggests that the presence of the flashing recess did not affect the stability of the parapets significantly.

8.3 Effect of Vertical Ribs on Parapet Stability

As shown in Table 5 a significant number of the parapets surveyed in detail had vertical ribs. To evaluate the effect that these ribs are likely to have on the stability of the parapets the predicted effect of adding 110mm ribs over 20% of one face of a 220mm thick parapet was evaluated. The parapets were evaluated using the NZSEE procedure except that the S39W component of the Gisborne earthquake was used as the input motion at the support level of the parapet instead of a floor response spectra and no amplification was assumed (the same results would have been obtained using the Author's proposed modified procedure and the same support input motion). The parapet height was assumed to vary between 490 and 5000mm.

Table 4 indicates the predicted effect the 110mm vertical ribs added to 220mm thick parapet for a variety of methods of modelling the ribs. The predicted increase in capacity of the ribbed parapet relative to the 220mm thick non-ribbed parapet is given for a parapet of the same height. The increased capacity corresponds to the increase in spectral intensity predicted to cause parapet collapse.

Table 4: Effect of vertical ribs on assessed parapet stability

Item No.	Parapet analysed (i.e. modelling effect of rib on one face of parapet)	Face parapet Rotating towards	Increased capacity relative to 220mm uniform parapet	Paramenters used in analysis equations (see section 2.1)
1	220mm uniform parapet without ribs (reference parapet)	Either	0% (i.e. reference)	$t_{.nom}$=220 mm
2	220mm parapet with additional 110mm vertical ribs on 20% of one face	Non ribbed	14 – 18%	W_{anc}=+0.2W/2, x= -165mm, $t_{.nom}$=220 mm
3	330mm parapet with additionall 110mm recesses on 80% of one face (i.e. identical to item 2)	Non ribbed	ditto	W_{anc}=-0.8W/3, x= -110mm, $t_{.nom}$=330 mm
4	As for item 3	Ribbed	90 -120%	W_{anc}=0.8W/3, x= +110mm, $t_{.nom}$=330 mm
6	330 uniform thickness parapet (i.e. thickness = to rib thickness	Either	50 – 67%	$t_{.nom}$=330 mm
7	Parapet thickness equal to average of rib and wall thickness (i.e. 275mm)	Either	24 -34%	$t_{.nom}$=275 mm

Items 2 and 3 indicate that the ribs can be modelled in two ways when the parapet is rotating towards the non- ribbed face of the parapet. In item 2 the ribs are treated as a positive mass at a negative eccentricity while in item 3 the cutaway parapet thickness between the ribs can is treated as a negative mass at a negative eccentricity (i.e. W_{anc} and $t_{.nom}$ both negative). Both methods predict the same capacity for a ribbed parapet of the same height. In this case the increase in capacity is 14 to 18% for various height parapets. However, when the parapet rotates towards the ribbed face (item 4) the increased capacity is 90 to 125% which is greater than the increase of 50 to 67% expected if the parapet thickness is increased from 220mm to 330mm rib thickness (i.e. item 5).

These analyses indicate that the wall has the least resistance when it rotates towards the non-ribbed face. However, the analyses for items 2 and 3 assume that the wall has equal stability in both directions and some increase in the 14 to 18% increase can be expected in the unsymmetrical case towards the 90 to 125% in item 4. Item 6 indicates that if the average ribbed and non-ribbed parapet thickness is used to model the parapet the predicted increase in capacity is 24 to 34% which is an intermediate value.

Comparative inelastic time-history analyses would be required to evaluate the effect of unsymmetrical ribs on the performance of parapets but, for this study, averaging of the ribbed and non-ribbed parapet thicknesses was used to include the effect of the ribs in the analyses.

8.4 Predicted Performance of Gisborne Parapets – Modified Author's Procedure

The performance of the surveyed Gisborne parapets was evaluated using a modified version of the Author's procedure and the results compared with the actual behaviour of the parapets in the Gisborne earthquake. The reasons for the modifications to the procedure are discussed in subsequent sections of this report.

The Author's procedure for assessing parapets was modified to use the NZSEE equations for calculating the rocking period of the parapet and its participation factor. This modification includes the slenderness of the parapet in the equations and has most effect on squat parapets. Also only the displacement spectrum was used in the modified procedure (as in the NZSEE procedure), instead of using both acceleration and displacement spectra as used in the Author's original procedure. Again this mainly effects squat parapets but tends to negate the inclusion of parapet slenderness in the equations used to calculate the period and participation factor. The effect of these modifications to the Author's procedure was discussed previously in sections 7.2 and 7.3.

The resulting modified Author's parapet assessment procedure can also be thought of as a modified version of the current NZSEE procedure. In this case the input spectral intensity of the earthquake motion at the support level of the parapet that is used in the modified procedure is an amplified version of the ground motion (as in the Author's procedure) rather than a floor spectra response based on the peak ground acceleration. Also the building height amplification factor would be the values used in the Author's procedure and not those effectively derived from NZS 1170.5 using 5% or 15% damping..

The probability of the surveyed parapets collapsing, assessed using the Author's modified procedure, is presented in Figure 21 (a) and (b) for the parapets subjected to the N51W and S39W components of the 2007 Gisborne respectively. The probabilities were evaluated using the procedure given previously in section 3.3 using the demand/capacity ratios presented in Figure 22 for the same sets of parapets. The parapets were analysed using the maximum surveyed parapet height (i.e. slenderness).

As indicated in Figure 22 parapets with a demand/capacity ratio of less than 1.0 are assessed by the procedure as having a low probability of failure (i.e. corresponding to probability of failure of zero in Figure 21). A demand/capacity ratio of 1.0 corresponds to the condition where the spectral intensity of the earthquake motion is predicted to have a low probability of causing collapse (i.e. spectral demand and safe spectral capacity are equal).

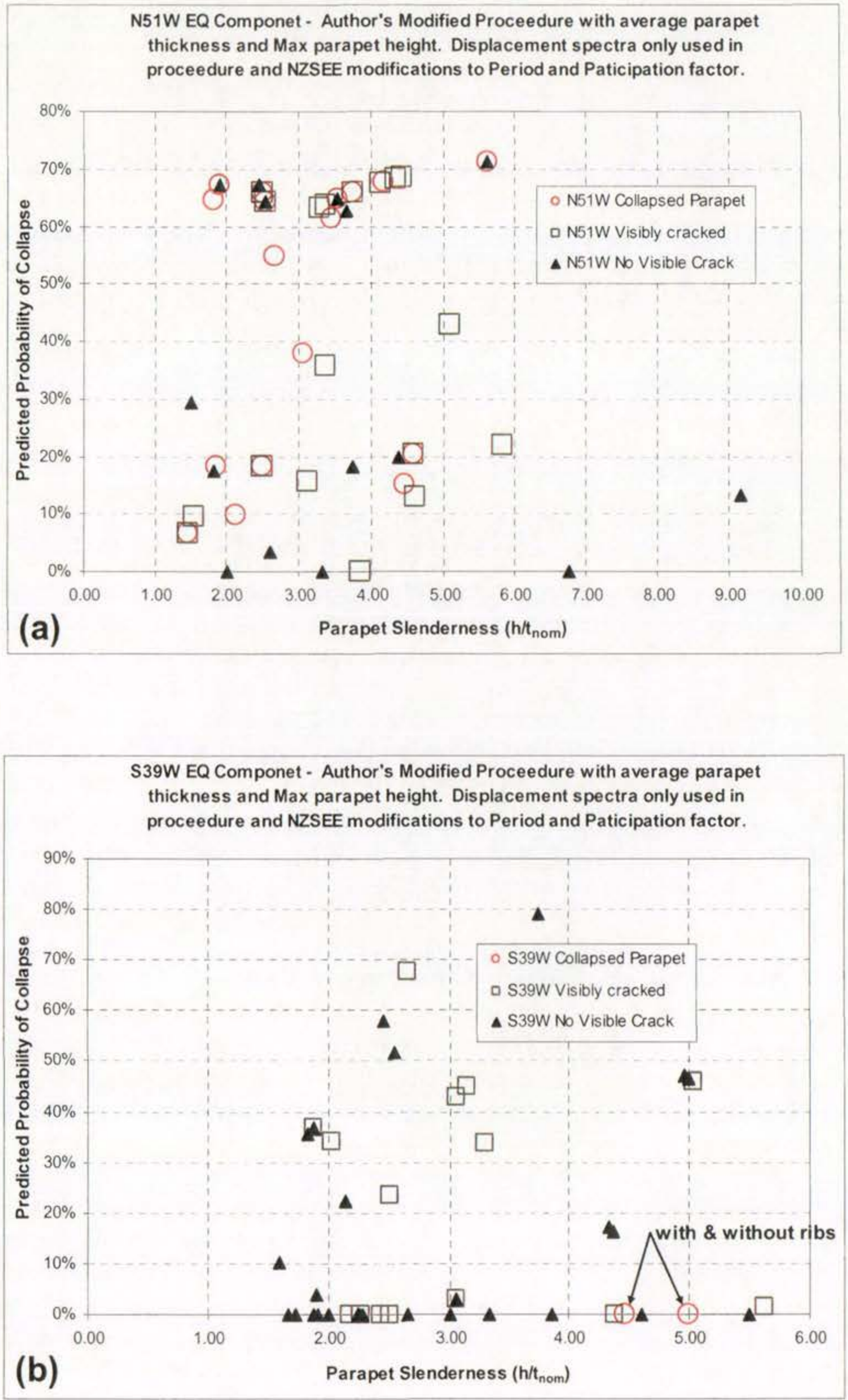
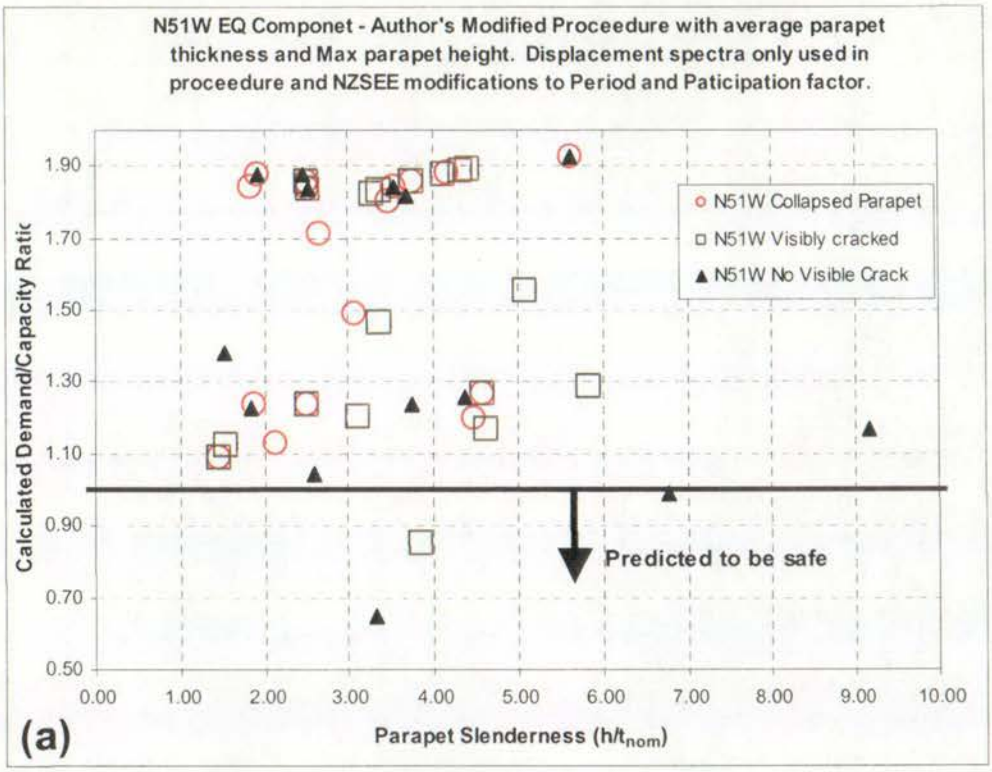


Figure 21: Predicted probability of surveyed parapets using Author's modified procedure and maximum surveyed parapet height. (a) Parapets subjected to N51W component of Gisborne EQ. (b) Parapets subjected to S39W component of Gisborne EQ.



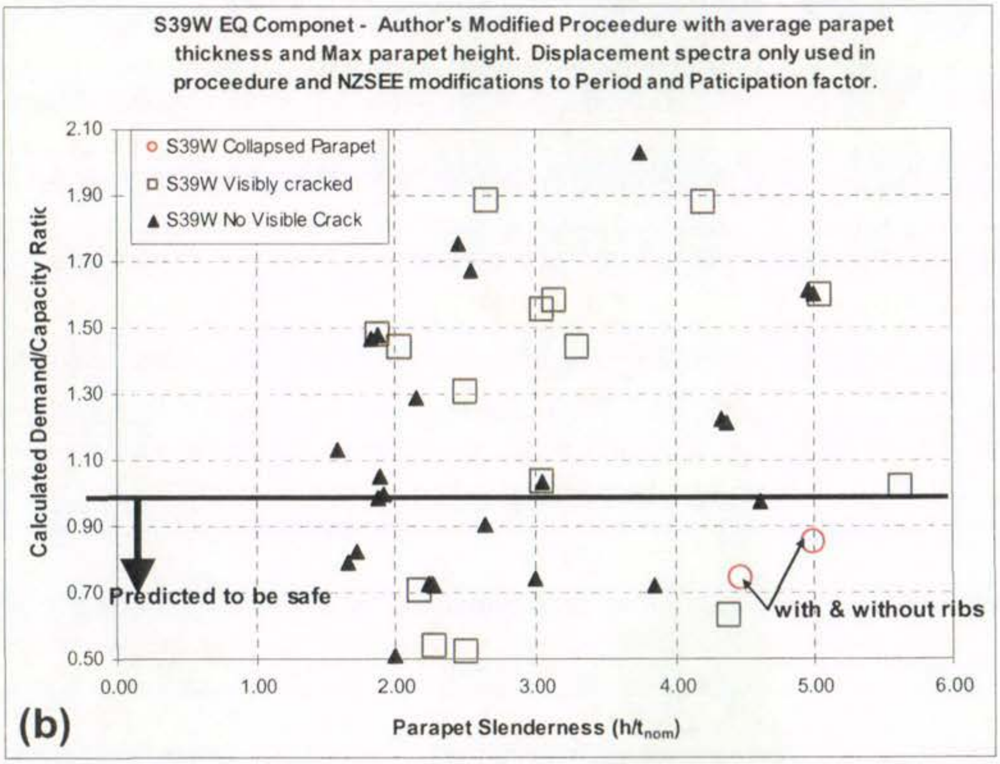


Figure 22: Predicted Demand/Capacity ratios of surveyed parapets using Author's modified procedure and maximum surveyed parapet height. (a) Parapets subjected to N51W component of Gisborne EQ. (b) Parapets subjected to S39W component of Gisborne EQ.

The veracity of the modified assessment procedure used to produce Figure 21 (a) using the N51W component of the Gisborne earthquake can be judged using the following criteria:

1. The predictions of parapet failures are not sensitive to parapet slenderness. This lack of sensitivity to parapet slenderness is supported by the actual parapet damage apparent in Figure 21 (a). Using this criterion the predictions are good.

2. No parapets assessed as being safe actually collapsed. Using this criterion the predictions are good.

3. It would be expected that the predicted probability of collapse would be somewhat higher for the collapsed parapets than for the visibly cracked parapets which, in turn would be somewhat higher than for the non-visibly cracked parapets. The average calculated probabilities of collapse were 46%, 35% and 34% for the collapsed, visibly and non-visibly cracked parapets respectively. Using this criterion the predictions do not strongly support the veracity of the modified procedure.

As indicated in Figure 22 (a) and (b) only one of the surveyed parapets subjected to the S39W component of the earthquake collapsed and this parapet was predicted by the Author's modified procedure to be have been safe with a zero probability of failure.

This collapsed parapet was the left hand parapet on the Albert Building (i.e. on building No. 15, Table 5, Appendix A). The collapse occurred at one end of the parapet where the parapet had its maximum height above the support level and had separated from the transverse supporting wall and between the widely spaced vertical ribs. Two data points are, therefore, plotted for this parapet, with and without the assessed effect of the vertical ribs. It can be seen that even without the ribs the parapet was predicted to be safe. This parapet collapse could be an unexplained anomaly. However, given the number of visibly cracked parapets that the modified procedure predicts have relatively a large probability of failure (see Figure 22 (a)) more failures of the parapets subjected to the S39W component of the earthquake would have been expected.

Part of the explanation of this lack of parapet collapse is likely to be that the roof diaphragms are so weak that they can not deliver an amplified building response to the support level of the parapet so that the face loaded wall of the buildings are responding essentially as tall cantilever walls rocking on their foundations. It is interesting to speculate that if the roof diaphragms of the Gisborne buildings are strengthened the parapets may become more vulnerable to collapse. The "culling" effect of earlier earthquakes which may have contributed to the relatively good performance of the parapets subjected to the S39W component of the earthquake are discussed in section 8.7.

The surveyed parapets were reanalysed using the minimum surveyed parapet height in the analysis. The results are presented in Figure 21 and Figure 22 which can be compared with the corresponding results in Figure 23 and Figure 24 respectively. The general conclusion reached above for the analysis results obtained using the maximum parapet heights remain unchanged.

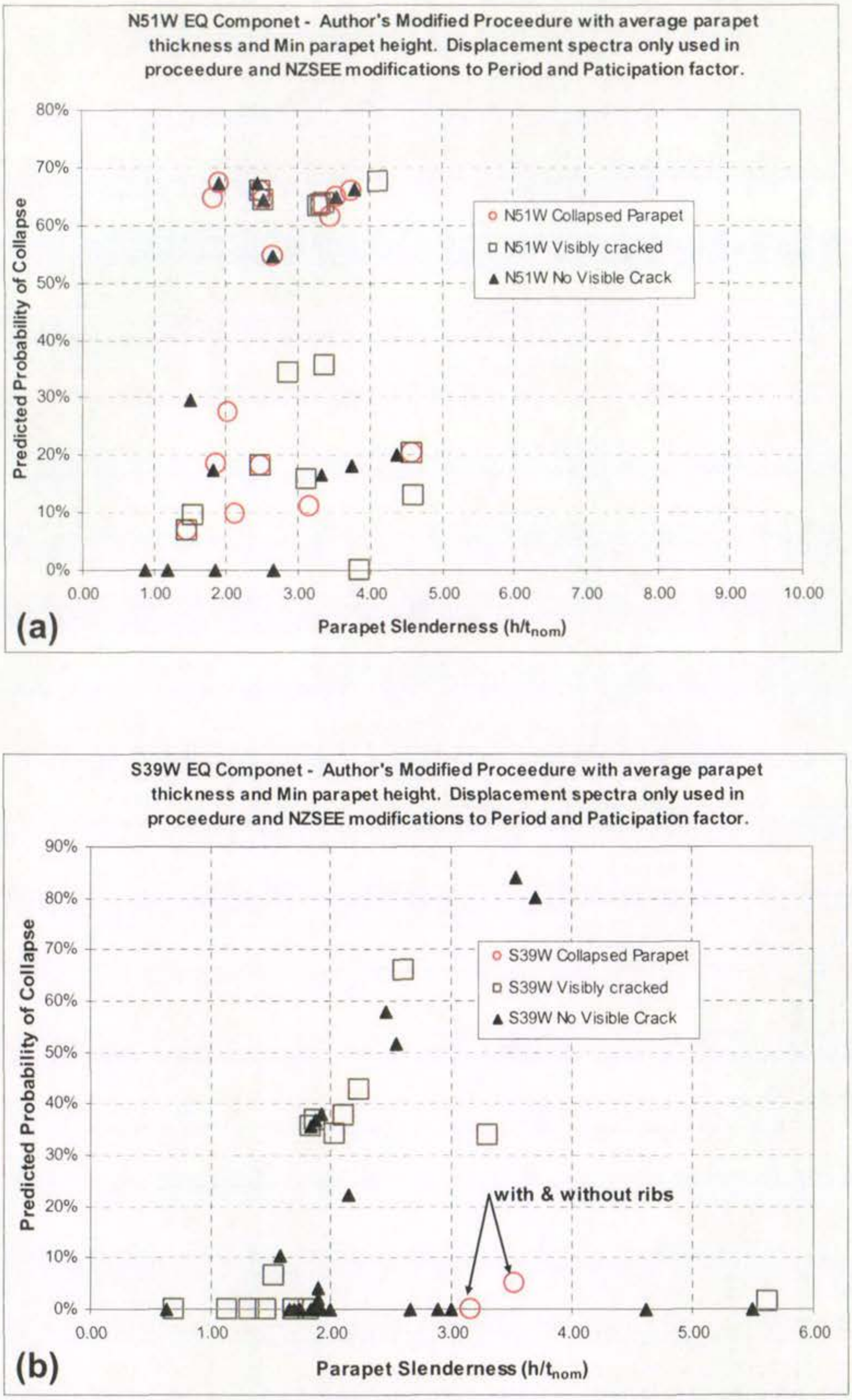
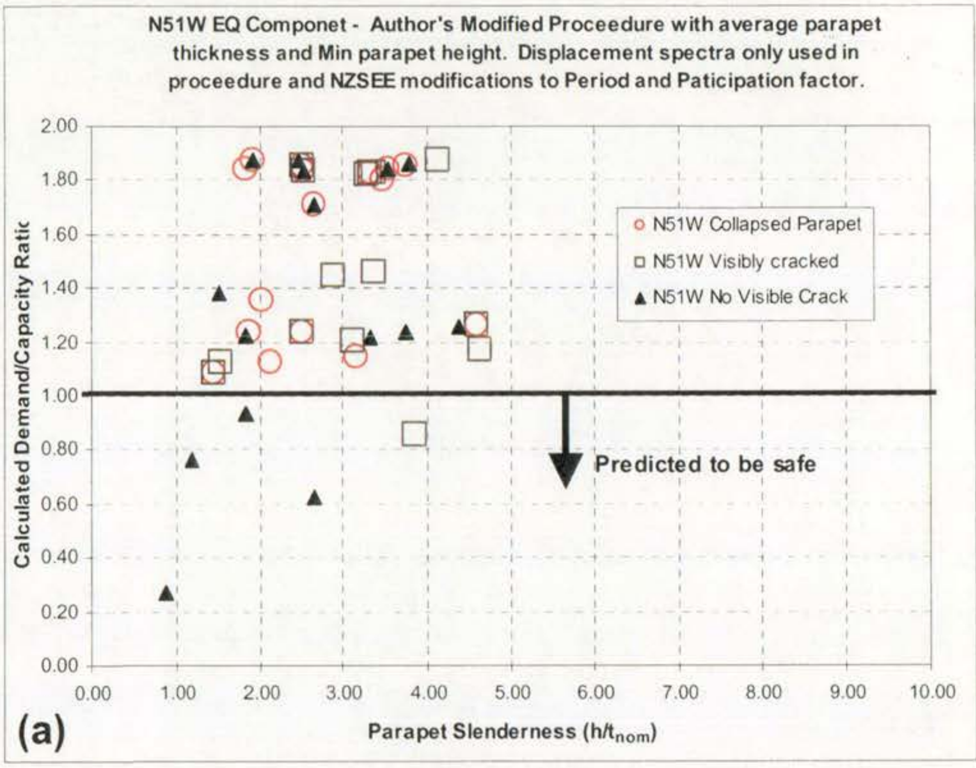


Figure 23: Predicted probability of collapse of surveyed parapets using Author's modified procedure and minimum surveyed parapet height. (a) Parapets subjected to N51W component of Gisborne EQ. (b) Parapets subjected to S39W component of Gisborne EQ.



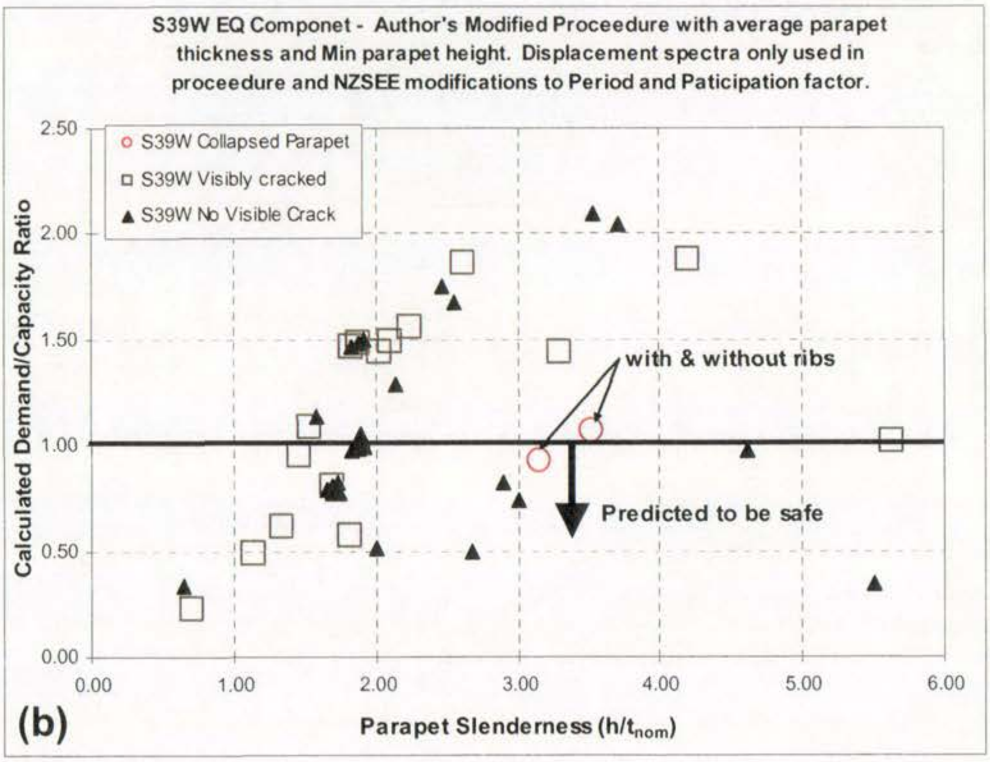


Figure 24: Predicted Demand/Capacity ratios of surveyed parapets using Author's modified procedure and minimum surveyed parapet height. (a) Parapets subjected to N51W component of Gisborne EQ. (b) Parapets subjected to S39W component of Gisborne EQ.

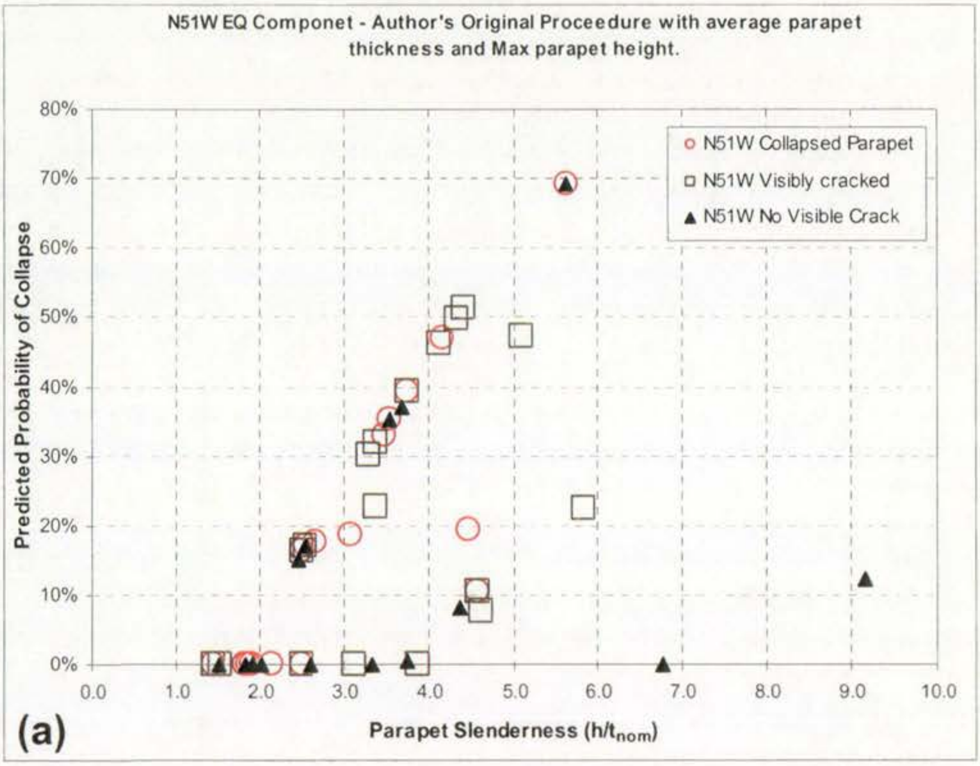
8.5 Reasons for Modification to Author's Parapet Assessment Procedure

The surveyed parapets were reanalysed using the Author's original parapet assessment procedure. The results of the analyses using the maximum surveyed parapet height in the analyses are presented in Figure 25 and Figure 26. These plots can be compared with the corresponding results obtained using the Author's modified procedure presented in Figure 21 and Figure 22 for predicted probability of collapse and demand/capacity ratios respectively.

When the results obtained using the Author's original procedure are evaluated using the three veracity criteria given above in section 8.4 the predicted risk of collapse is seen to be relatively poor compared with those predicted using the Author's modified procedure.

Figure 27 demonstrates the effect of using the displacement spectra only in the Author's modified procedure (as per the NZSEE procedure) instead of both acceleration and displacement spectra as in the Author's original procedure. The effect of this modification to the Author's original procedure on the predicted probability of failure can be seen by comparing Figure 27(a) with Figure 25(a). Also the effect of the modification on the calculated Demand/Capacity ratio can be seen by comparing Figure 27(b) with Figure 26 (b). When the effect of this modification is evaluated using the three veracity criteria given it can be seen that prediction of the risk of collapse is improved by the modification.

The other change made to the Author's original procedure was to include slenderness in formulae used to calculate the period and participation factor (as per the NZSEE procedure). The effect of this second modification on the predicted probability of failure can be seen by comparing Figure 27(a) with Figure 21(a). Also the effect of this modification on the calculated Demand/Capacity ratio can be seen by comparing Figure 27(b) with Figure 22(b). When effect of this modification is evaluated using the three veracity criteria given above in section 8.4 it can be seen that prediction of the risk of collapse is probably improved marginally.



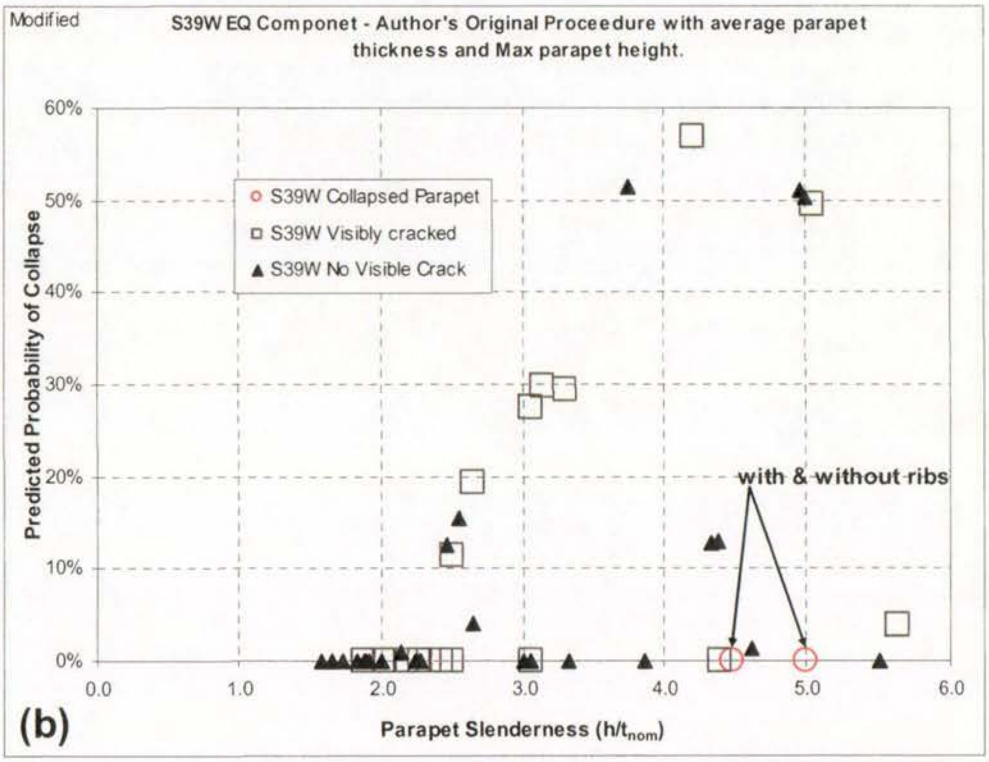
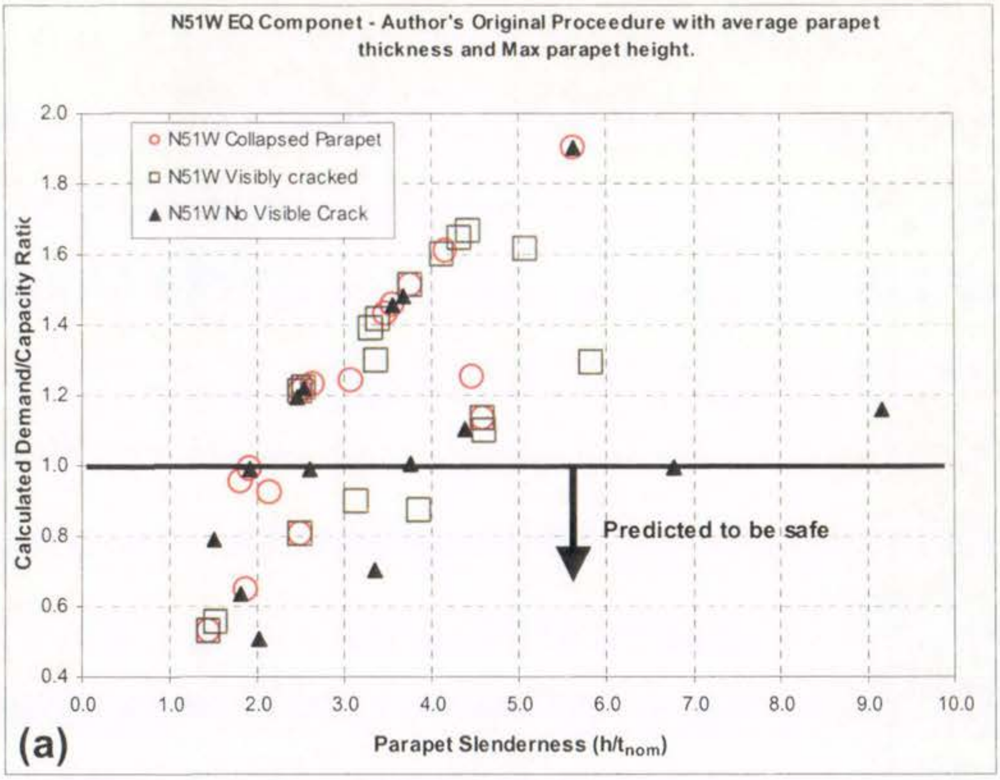


Figure 25: Predicted probability collapse of surveyed parapets using Author's original procedure and maximum surveyed parapet height. (a) Parapets subjected to N51W component of Gisborne EQ. (b) Parapets subjected to S39W component of Gisborne EQ.



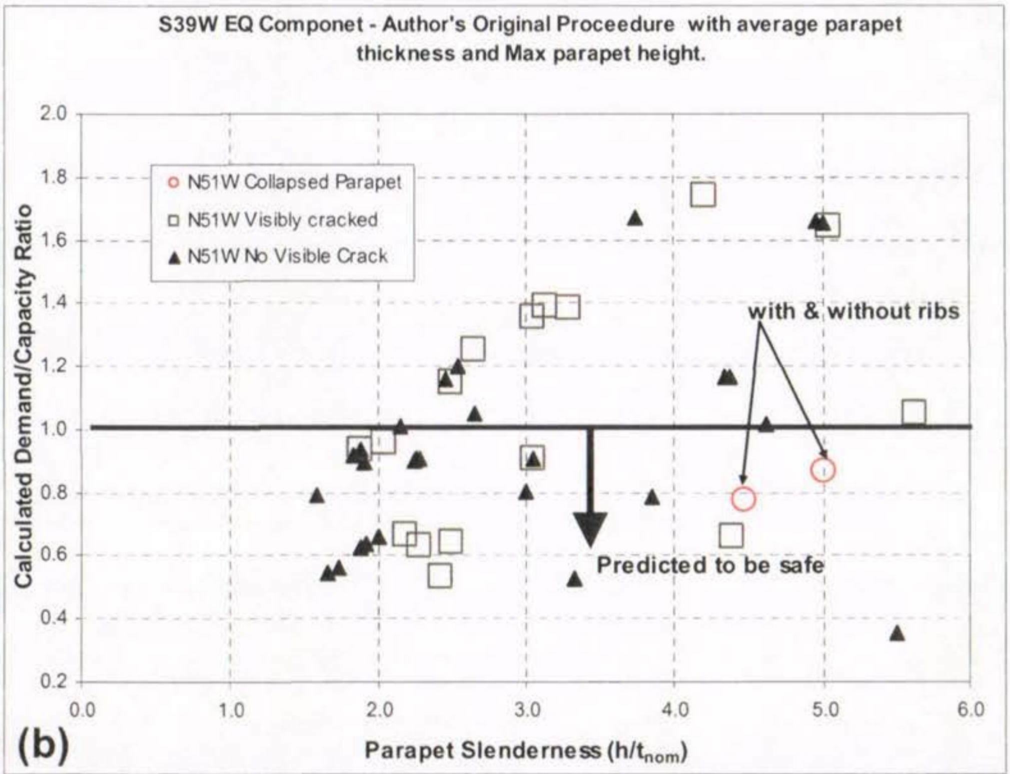
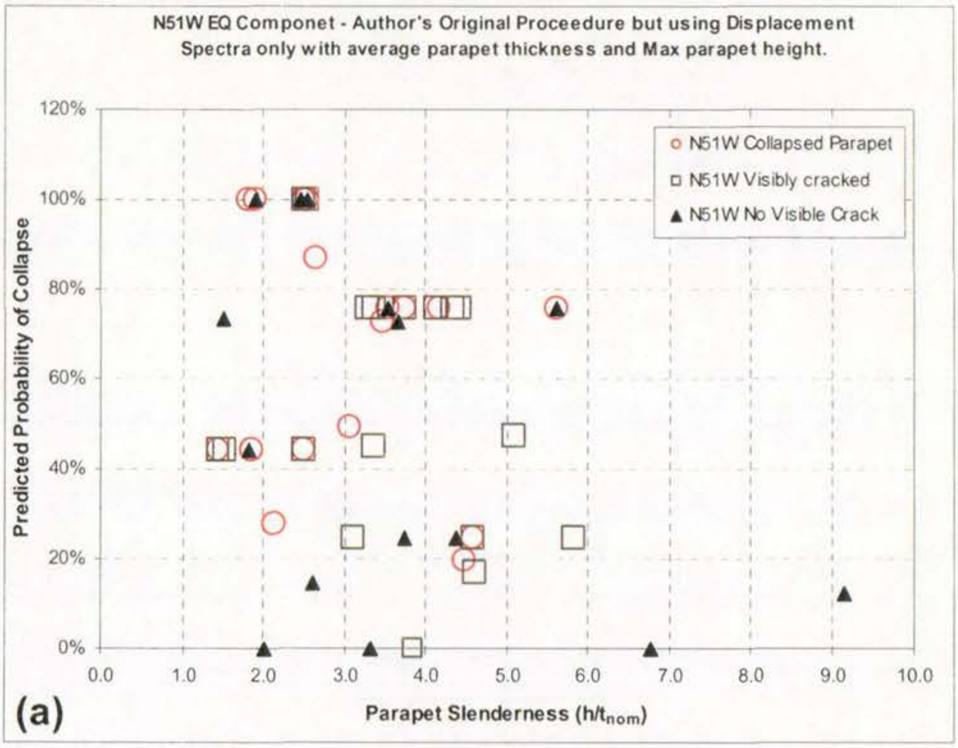


Figure 26: Predicted Demand/Capacity ratios of surveyed parapets using Author's original procedure and maximum surveyed parapet height. (a) Parapets subjected to N51W component of Gisborne EQ. (b) Parapets subjected to S39W component of Gisborne EQ.



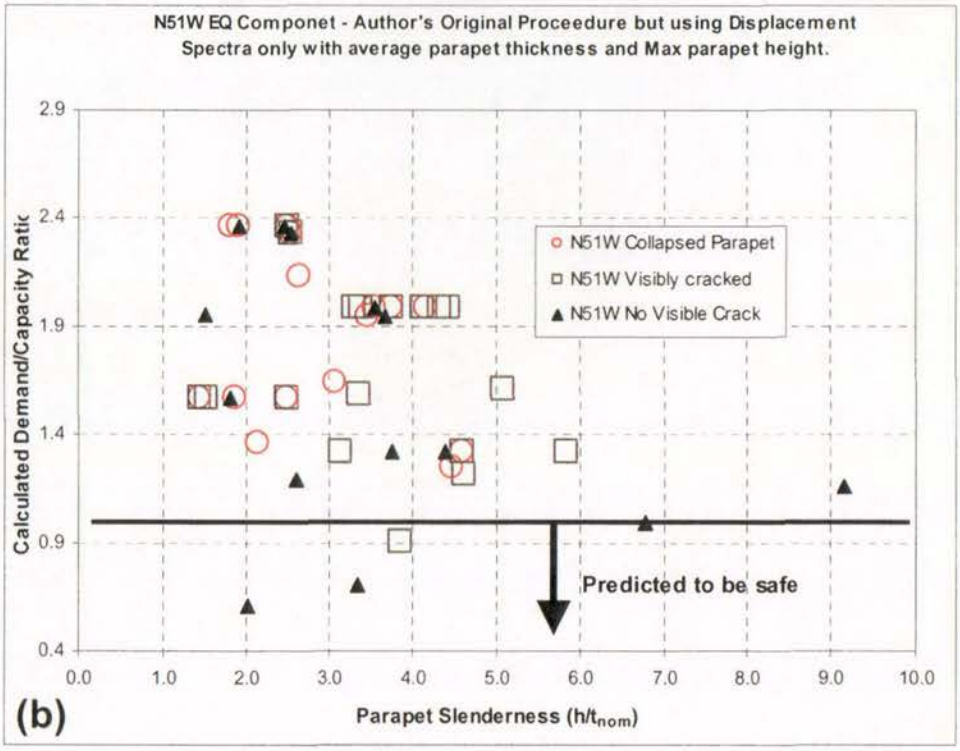


Figure 27: Effect of using only the displacement spectra in the Author's original assessment procedure (a) Effect on probability of failure (compare results with Figure 25(a)). (b) Effect on Demand/Capacity ratios (compare results with Figure 26 (b)).

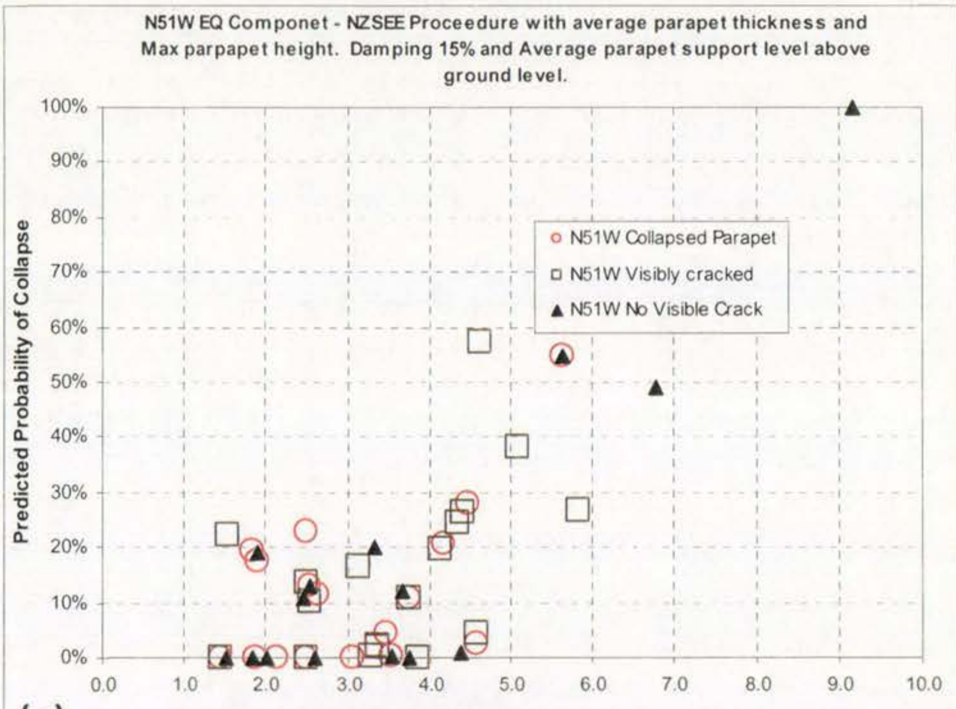
8.6 Reasons for Modification to NZSEE Parapet Assessment Procedure

The surveyed parapets were reanalysed using the NZSEE parapet assessment procedure. The results of the analyses using 15% damping when calculating the building amplification factor and using the maximum surveyed parapet height in the analyses are presented in Figure 28 and Figure 29 for the predicted probability of collapse and the demand/capacity ratios respectively.

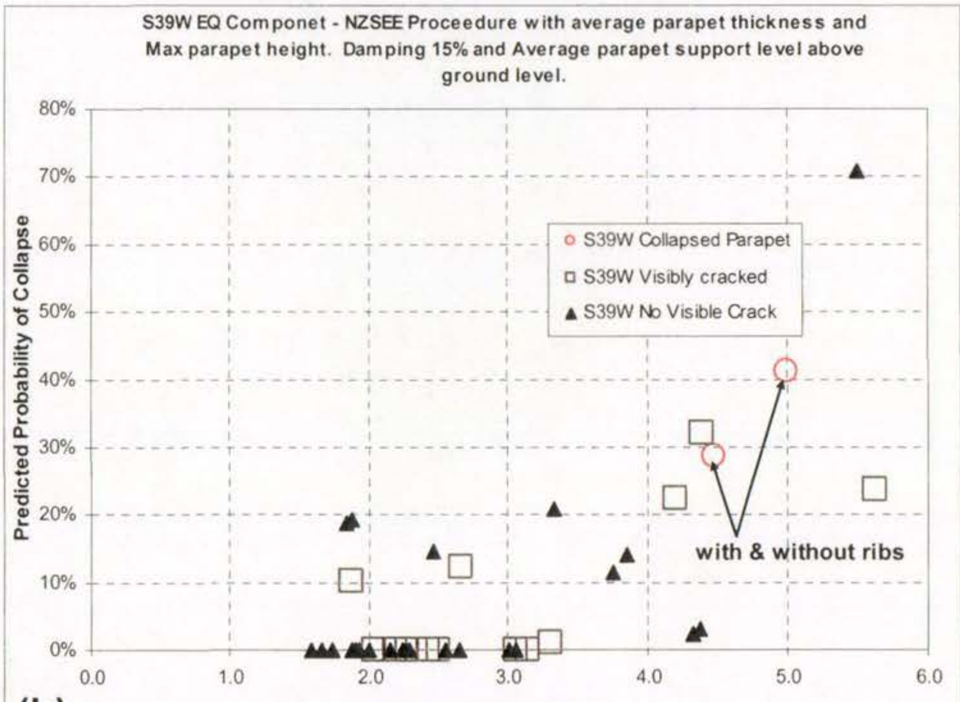
If the damping used to calculate the building amplification factor in the NZSEE procedure is reduced from 15% to 5% the demand/capacity ratio calculated is reduce by a factor of 0.67. The effect of this change in assumptions made for the NZSEE procedure on the predicted probability of failure can be seen by comparing Figure 28 with Figure 30. Also the effect on the calculated Demand/Capacity ratio of this change in assumptions can be seen by comparing Figure 29 with Figure 31. Generally it can be seen that assuming lower damping improves the procedures predictions of the actual parapet collapses

When the analyse results are evaluated using the three veracity criteria given in section 8.4 the predictions of the performance of the collapsed parapets made using the NZSEE procedure are seen to be relatively poor compared with the predictions made using the Author's modified procedure.

The exception to this conclusion is the performance of the single parapet that collapsed when subjected to the S39W component of the earthquake. However, this is a single data point and its collapse may be due to an unexplained factor.

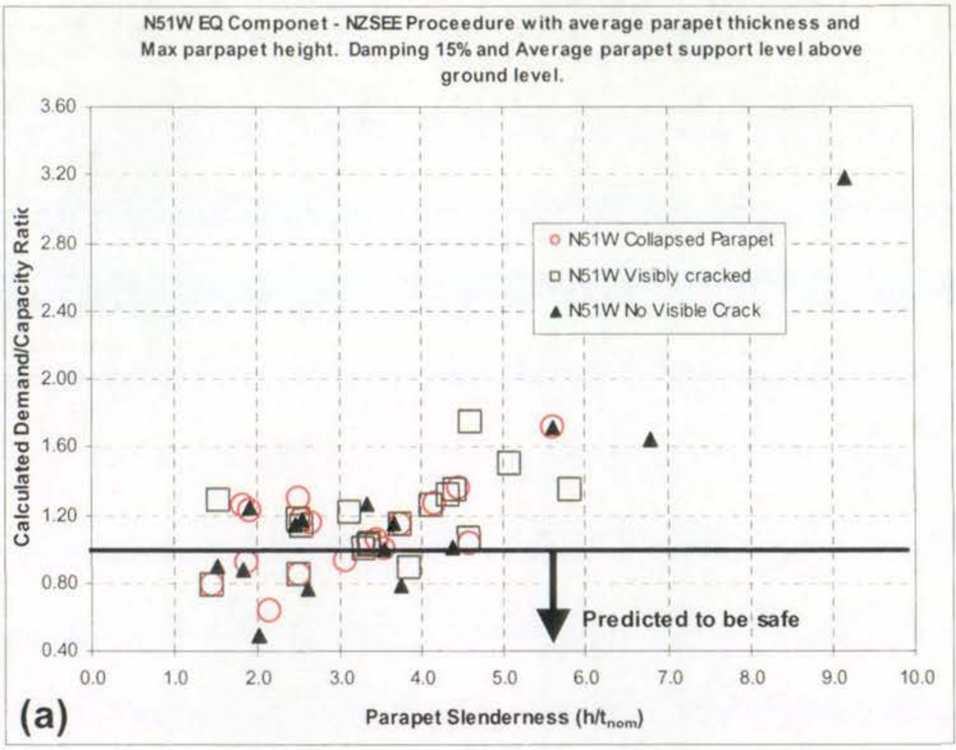


(a)



(b)

Figure 28: Predicted probability of collapse for the surveyed parapets using NZSEE procedure with 15% damping and maximum surveyed parapet height. (a) Parapets subjected to N51W component of Gisborne EQ. (b) Parapets subjected to S39W component of Gisborne EQ.



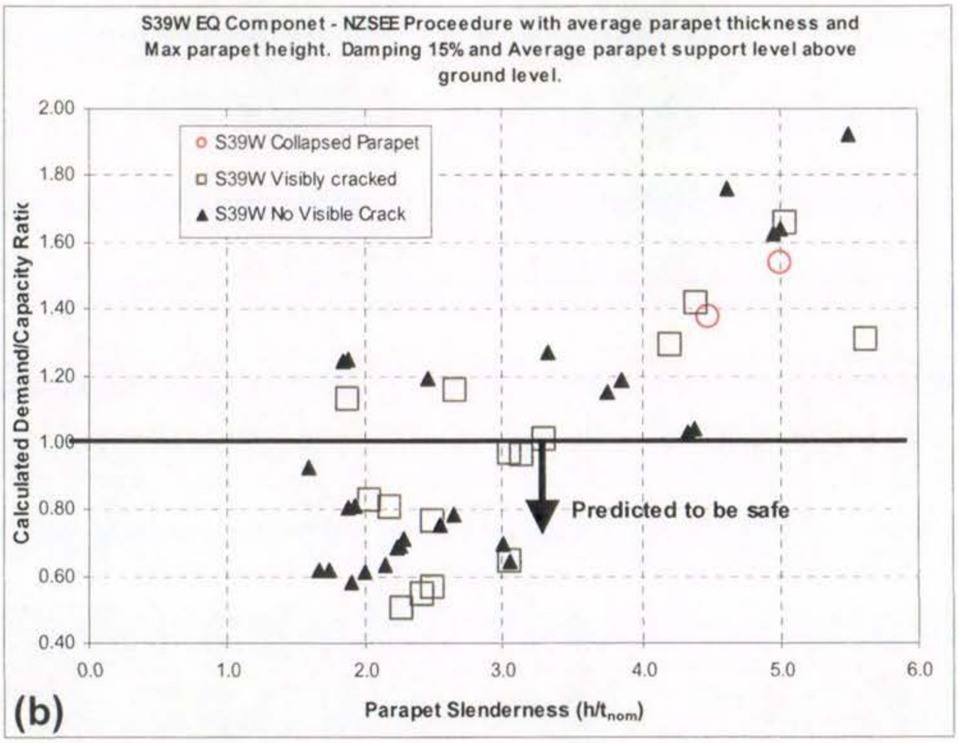
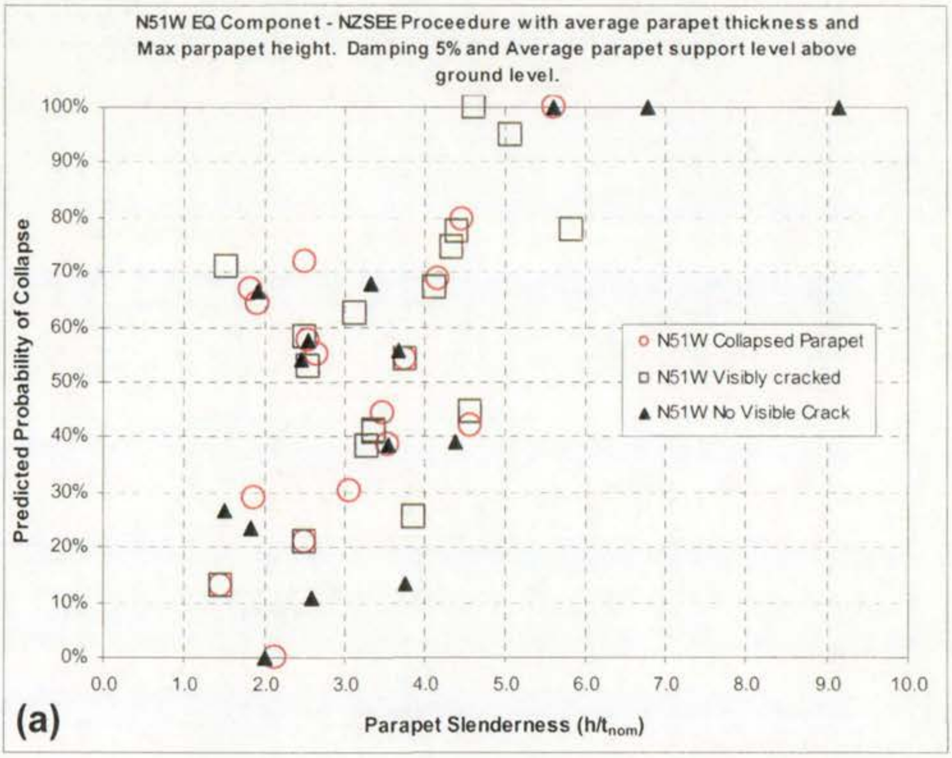


Figure 29: Predicted Demand/Capacity ratios of surveyed parapets using the NZSEE procedure with 15% damping and maximum surveyed parapet height. (a) Parapets subjected to N51W component of Gisborne EQ. (b) Parapets subjected to S39W component of Gisborne EQ.



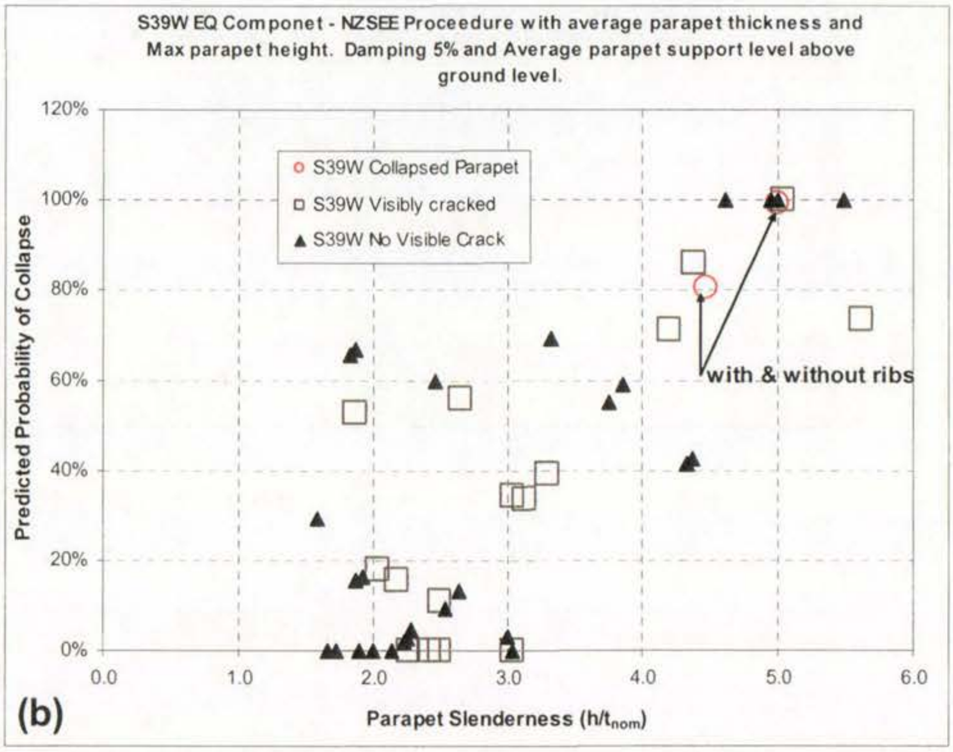
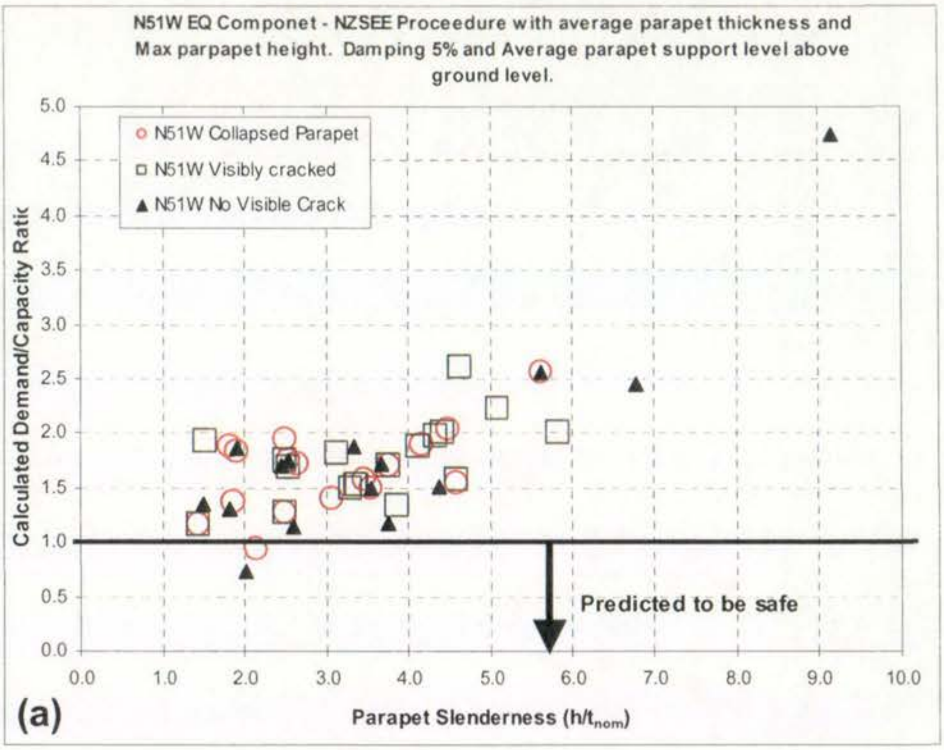


Figure 30: Predicted probability of collapse for surveyed parapets using NZSEE procedure with 5% damping and maximum surveyed parapet height. (a) Parapets subjected to N51W component of Gisborne EQ. (b) Parapets subjected to S39W component of Gisborne EQ.



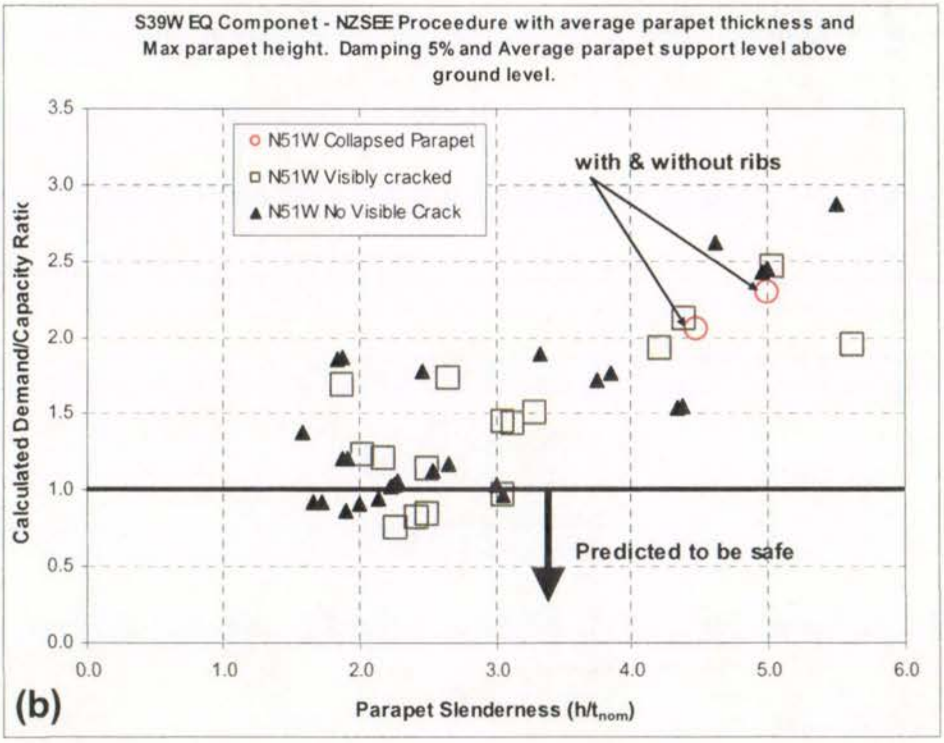


Figure 31: Predicted Demand/Capacity ratios of surveyed parapets using the NZSEE procedure with 5% damping and maximum surveyed parapet height. (a) Parapets subjected to N51W component of Gisborne EQ. (b) Parapets subjected to S39W component of Gisborne EQ.

8.7 Effect of Previous Earthquake Shaking on Gisborne Parapets

The number of parapets that failed in the 2007 Gisborne earthquake was relatively low (i.e. 22, or 6%, of the 360 total number of parapets in the CBD) and this was particularly true for the parapets subjected to the S39W component of the earthquake. This performance may be partially explained by the effect of previous earthquake shaking of the Gisborne parapets.

Figure 32 presents the acceleration and displacement response spectra derived for the 2007 Gisborne Earthquake ground motion recorded at 2ZG in Gisborne CBD. Spectra for the ground motion recorded at the same site and at the nearby CPO site during the 1993 Ormond EQ are also shown. These spectra are for the earthquake component that was approximately parallel to the Main St. in the Gisborne CBD (i.e. Gladestone Rd.). Similar spectra for the components approximately perpendicular to the main street are also shown in Figure 33.

The plots indicate that the spectral intensity of the ground shaking was generally greater in the 2007 Gisborne earthquake than in 1993, especially in the critical period range 1.2 to 2.5 seconds for typical 220mm thick parapets. The difference between the spectra for the two sites in the1993 earthquake also gives an indication of the variation in the ground shaking that may have occurred during the 2007 earthquake.

Very little parapet damage was reported for the 1993 earthquake. Wells (Wells JD, 1994) reports only "1 badly damaged parapet:" and "3 parapets rendered unstable and brick(s) dislodged" in the Gisborne CBD. The lack of damage to parapets orientated normal to the Main St. in the 1993 earthquake is not surprising given that the (smoothed) displacement spectra for the 2007 earthquake shown in Figure 32(b) is about twice that for the 1993 earthquake in the critical effective period range.

Also, the relatively minor damage to parapets orientated parallel to the Main St. in the 1993 earthquake is not surprising given that only one of these parapets collapsed in the 2007 earthquake. For these parapets, Figure 33(b) indicates that the spectral displacement intensities were more closely matched but were greater in the 2007 earthquakes than in the 1993 earthquake. Given that weaker parapets were more likely to have been "taken out" by the earlier 1993 earthquake the significantly greater damage in the 2007 earthquake would not have been expected.

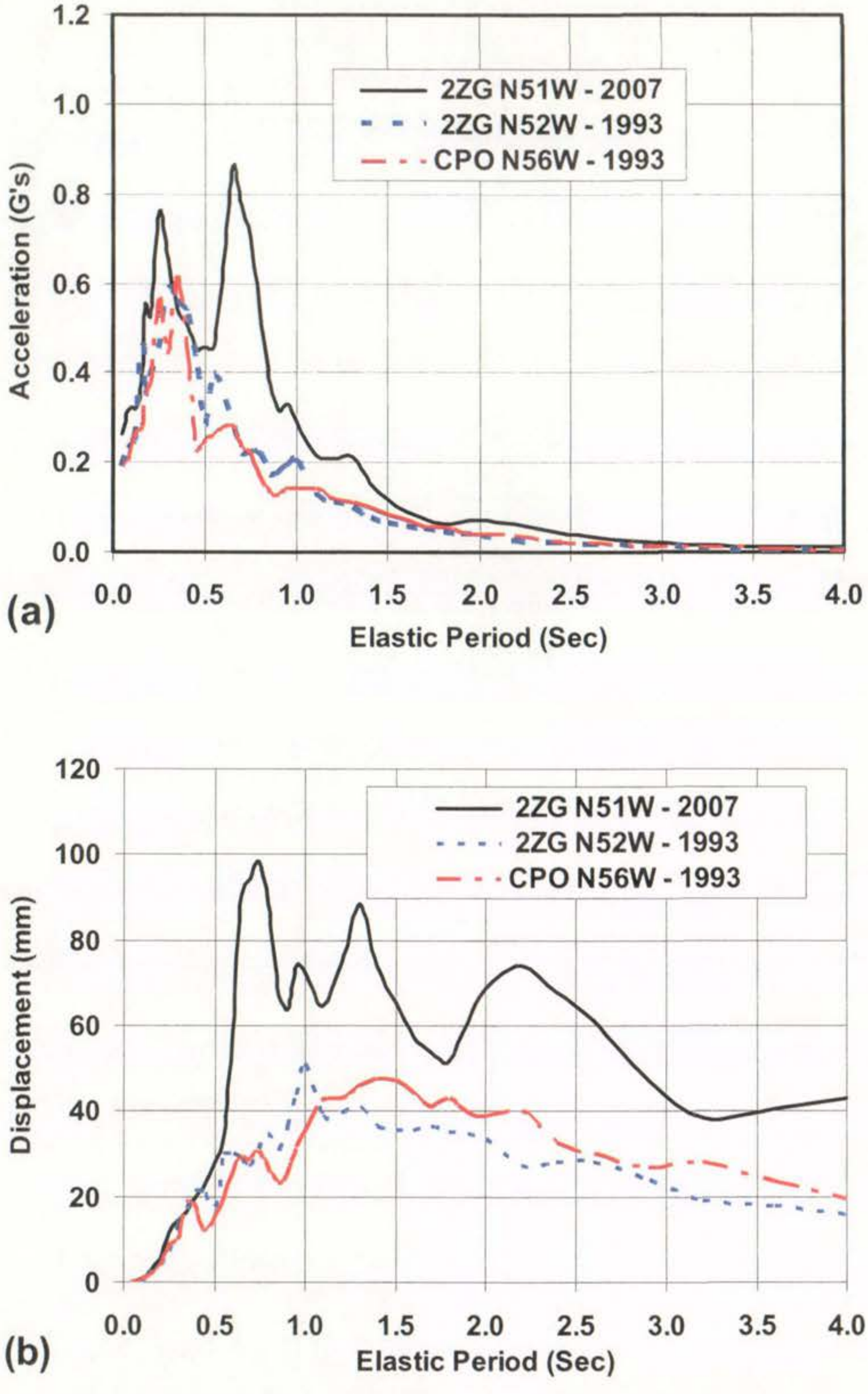


Figure 32: (a) Acceleration and (b) Pseudo displacement response spectra for the 2007 Gisborne Earthquake ground motion recorded at 2ZG in Gisborne CBD and those recorded at the same site and at the nearby CPO site during the 1993 Ormond EQ. Spectra are for the recorded component that was approximately parallel to the Main St in the Gisborne CBD.

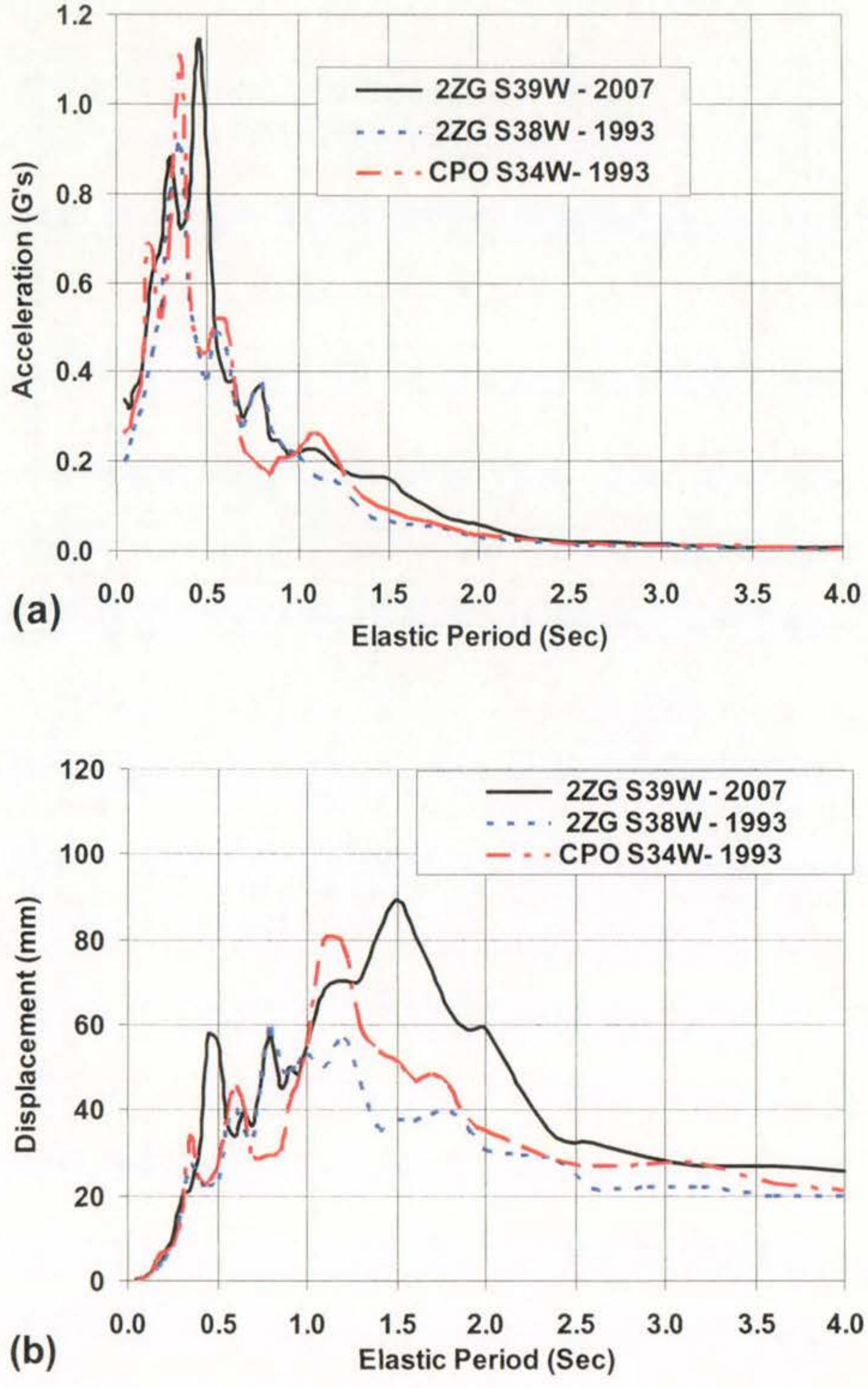


Figure 33: **(a)** Acceleration and **(b)** Pseudo displacement response spectra for the 2007 Gisborne Earthquake ground motion recorded at 2ZG in Gisborne CBD and those recorded at the same site and at the nearby CPO site during the 1993 Ormond EQ. Spectra are for the recorded component that was approximately perpendicular to the Main St in the Gisborne CBD.

The 1993 earthquake is reported (Wells J D, 1994) as having a felt intensity of MMV to MMVI in Gisborne City while an earlier March 1966 earthquake had a higher felt intensity of MMVI-VII. It was also reported that the March 1966 Gisborne earthquake had a significantly stronger component normal to the main street than parallel to it. It is possible that the most vulnerable parapets parallel to the main street were "taken out" by the 1966 earthquake, either by collapsing or being removed or replaced in horizontally spanning reinforced concrete after the earthquake. This would leave only the less vulnerable parapets exposed to the less intense 1993 and 2007 earthquake shaking. This may help to explain why only one of the parapets orientated parallel to the main street collapsed in the 2007 earthquake and suggests that this collapse could have been an anomaly. Unfortunately only a scratch plate record (at the CPO site) is available for the 1966 Gisborne earthquake so that its spectral intensity is unknown.

9 Conclusions and Recommendations

The current study lead to the following conclusions and recommendation:

- It is recommended that both the Author's previous formulae and the current NZSEE formulae that are used to predict the effects of overburden on the stability of a face-loaded freestanding wall element be modified. The formulae should be modified so that only the proportion of the mass associated with the overburden load that will act as an inertial load is included in the inertial loads. (The proposed modification is the addition of the F_H and F_v parameters in equation (5). These parameters are currently taken as 0.0 and 1.0 in the Author's original procedure and the NZSEE procedure respectively)

- The NZSEE procedure for evaluating parapets and other cracked face-loaded URM wall elements makes a number of apparently minor modifications to the original face-loaded wall assessment procedure developed by the Author. These are principally; use of only a displacement spectrum instead of both displacement and acceleration spectra; use of floor spectra based only on the peak ground acceleration instead of a floor spectra equal to amplified ground spectra; different factors for the effect of building amplification; and formulae for effective period and participation factor that include the effect of wall element slenderness. This study shows that the cumulative effect of these modifications is to significantly alter the predicted performance of face-loaded URM wall elements.

- The main difference between the performance predicted by the Author's and the NZSEE procedures occurs because of the differences in the shape of the floor response spectra adopted for the two procedures. The shapes of these spectra diverge most on soft ground and where the ground response spectral displacements start to reduce at a relatively short spectral response period. Both these conditions were satisfied for the ground motion recorded during the 2007 Gisborne earthquake. For earthquake motions that have displacement spectral shapes closer to those given by design codes the differences in the two procedures would be less pronounced.

- 19 of the 22 parapets that collapsed in the 2007 Gisborne earthquake and 82 other parapets were surveyed to collect sufficient data to enable the assessment procedures to be used to predict their performance. The best prediction of actual parapet behaviour was given by a modified version of the Author's procedure. This Modified Author's Procedure an amalgam of the Author's original procedure and the NZSEE procedure.

- Based on the comparison between the predicted and actual performance of the parapets in the Gisborne earthquake it is recommended that the Author's procedure for assessing parapets be modified to use the NZSEE equations for calculating the rocking period of the parapet and its participation factor. This modification takes into account the slenderness of the parapet and has most effect on squat parapets. It is also proposed that only the displacement spectrum be used in the modified procedure (as in the NZSEE procedure), instead of using both acceleration and displacement spectra as used in the Author's original procedure. Again this mainly effects squat parapets but tends to negate the inclusion of parapet slenderness in the equations used to calculate the period and participation factor.

- For the NZSEE procedure to be the same as the modified Author's parapet assessment procedure two modifications would be required. The input spectral intensity of the earthquake motion at the support level of the parapet would need to be an amplified version of the ground motion (as in the Author's procedures) rather than a floor spectra response based on the peak ground acceleration. Also the building height amplification factor would need to be the values used in the Author's procedures and not those effectively derived from NZS 1170.5 using 5% or 15% damping.

- The Author's modified procedure predicts that only a few very squat parapets in the Gisborne CBD would be able to resist 1/3rd the design earthquake for a new building in Gisborne with only a low probability of collapse. Therefore, in terms of the New Zealand Building Act and regulations, almost all parapets in the Gisborne CBD would be assessed as being "Earthquake Prone".

- This study indicated that the Author's modified procedure predicted a greater probability of parapet collapse than indicated by the actual number of collapses in the 2007 Gisborne earthquake. It is suggested that a significant proportion of this difference in performance can be explained by many of the external walls of the Gisborne CBD buildings responding as free standing face-loaded walls rocking about a horizontal "cack" opening at foundation level. This type of behaviour is to be expected when the floor and roof diaphragm is not strong or stiff enough to significantly affect the response of external walls. Under these conditions a horizontal crack is less likely to form at the parapet support level and if it forms the parapet is less likely to collapse. When roof diaphragms are stiffened and strengthened the performance of parapets in moderate earthquake may approach that predicted by the Author's modified procedure and the parapets may have a higher risk of collapse.

- The displacement spectra for the 2007 Gisborne earthquake do not have a steadily increasing displacement demand with increasing period. Counter-intuitively the Author's modified assessment procedure predicts that, in a moderate earthquake that has this type of displacement spectral shape, the slender external walls of masonry buildings may be more stable responding as vertical cantilevers than squat parapets responding the amplified ground motion delivered to the supporting level of parapets by relatively strong/stiff roof structures. The external walls of a buildings will only respond as vertical cantilevers if they relatively unaffected by weak floor and roof diaphragms. Vertical cracks at corners of some of the Gisborne masonry buildings where the external walls had separated, indicates that the external walls may have responded, at least in part, as vertical cantilevers.

- The relative damage to Gisborne CBD parapets in the 1993 and 2007 was consistent with the relative spectral displacement intensities of the two earthquake motions recorded in the CBD. It is likely that the parapets parallel to the main street and subjected to the S39W component of the 2007 earthquake were subjected to higher spectral displacement intensity motions during the 1966 Gisborne earthquake which would have "culled" the weaker parapets. This may help to explain why fewer of the parapets orientated parallel to the main street collapsed in the 2007 than would have been predicted by the Author's modified assessment procedure.

Acknowledgements

The author gratefully acknowledges the funding of this research project by the Earthquake Commission. The efforts made by John Wells (Opus, Gisborne) in surveying and collecting the data on the Gisborne parapets and sharing his extensive knowledge of the Gisborne building stock is also acknowledged.

10 References

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Appendix A: Survey Results for Gisborne Parapets

Table 5: Gisborne Parapet Survey Data - **(Data used in Analysis and Plots)**

Bldg No.	Building Description	Street Frontage	Parapet Location	Normal Gisborne EQ Comp	No. of Storeys	Height to parapet support		Parapet Height above crack (or support)		Thickness @		Status (Yes/No)		Valid Parapet Plot Data
						Max (m)	Min (m)	Min (mm)	Max (mm)	ribs (mm)	wall (mm)	visible crack	parapet collapsed	
1	AMI Building	229-233	Front	S39W	2	8.5	8.5	1200	1600	480	350	N	N	Valid
		Gladstone Road	Left	N51W	2	11	8.5	440	440	240	240	N	Y	Valid
			Right	N51W	2	10	8.5	460	460	240	240	N	Y	Valid
2	Moleta Bros -	167-173	Front	S39W	2	8.5	8.5	1000	1000	670	470	Y	N	
		Gladstone Road	Left	N51W	2	8.5	8.5	800	900	240	240	N	Y	Valid
			Right	N51W	2	8.5	8.5	800	900	240	240	Y	N	Valid
3	Assett Finance	78 Gladstone Rd	Front	S39W	2	10	8.5	600	1800	470	350	Y	N	Valid
			Left	N51W	2	10	10	600	600	240	240	N	Y	Valid
			Right		2							N	N	
4	Arthur Toye	73-75 Gladstone	Front	S39W	1	5	4.5	500	1100	440	440	Y	N	Valid
		Road	Left	N51W	1	5	5	1100	1100	240	240	Y	N	Valid
			central	N51W	1	5	5	600	1400	240	240	Y	N	Valid
			Right	N51W	1	5	4.5	1100	1100	240	240	N	Y	Valid
5	Grant Bros	69-71 Gladstone	Front	S39W	1	4.8	4.5	350	1200	550	440	Y	N	Valid
		Road	Left	N51W	1	4.5	4.5	600	600	240	240	N	Y	Valid
			Right	N51W	1	4.5	4.5	600	600	240	240	Y	N	Valid
6	Vitality Foods	67 Gladstone Rd	Front	S39W	1	4.5	4.5	800	1000	440	440	Y	N	Valid
			Left	N51W	1	5.5	4.5	350	350	240	240	Y	N	Valid
			Right	N51W	1	5.5	4.5	350	350	240	240	N	Y	Valid

Table 5 . Continued

Bldg No.	Building Description	Street Frontage	Parapet Location	Normal Gisborne EQ Comp	No. of Storeys	Height to parapet support		Parapet Height above crack (or support)		Thickness @		Status (Yes/No)		Valid Parapet Plot Data
						Max (m)	Min (m)	Min (mm)	Max (mm)	ribs (mm)	wall (mm)	visible crack	parapet collapsed	
7	Smiths City	Bright Street	Front	N51W	1	4	4	900	900	240	240	N	N	Valid
		cnr Childers Rd	Left	S39W	1	4	4	500	900	350	240	Y	N	Valid
			Right	S39W	1	4	4	500	900	350	240	N	N	Valid
8	Bates Building	2 Lowe Street	Front	N51W	2	8	8	850	1500	350	240	Y	N	Valid
			Left	S39W	2	8	8	750	750	350	240	N	N	Valid
			Right	S39W	2	8	8	450	450	240	240	Y	N	Valid
9	Geneva Finance	Gladstone Road	Front	S39W	2	8	8	760	1060	450	350	N	N	Valid
		cnr Derby	Left	N51W	2	8	8	740	1040	450	350	N	N	Valid
			Right	N51W	2	8	8	750	750	350	350	N	Y	Valid
			Rear	S39W	2	8	8	750	750	350	350	N	N	Valid
10	ex Adair's/ Pumpkin Patch	Gladstone Road cnr Grey St	Front	S39W	3	12.5	12.5	1200	1500	450	450	N	N	Valid
			Left	N51W	3	12.5	12.5	1200	1500	450	450	N	N	Valid
			Right	N51W	3	12.5	12.5	1200	1200	260	260	Y	N	Valid
			Rear	S39W	3	12.5	12.5	1200	1200	260	260	N	N	Valid
11	Melbourne Cash (Ian Penny)	118-122 Gladstone Road	Front	S39W	2	9	9	660	900	350	240	Y	N	Valid
			Left	N51W	2	9	9	650	900	250	240	N	N	Valid
			Right	N51W	2	9	9	650	650	250	240	N	Y	Valid
			Rear	S39W	2	9	9	640	650	250	240	Y	N	Valid
12	Ex Lyric Café (Ray Moleta) Old Food 4 Tht	124 Gladstone Rd	Front	S39W	2	8.5	8.5	520	850	440	240	Y	N	Valid
			Left	N51W	2	8.5	8.5	850	850	250	240	N	Y	Valid
			Right		2	8.5	8.5			240	240			
			Rear	S39W	2	10.5	8.5	850	450	240	240	N	N	Valid

Table 5 - Continued

Bldg No.	Building Description	Street Frontage	Parapet Location	Normal Gisborne EQ Comp	No. of Storeys	Height to parapet support		Parapet Height above crack (or support)		Thickness @		Status (Yes/No)		Valid Parapet Plot Data
						Max (m)	Min (m)	Min (mm)	Max (mm)	ribs (mm)	wall (mm)	visible crack	parapet collapsed	
13	T Adair's Building	126-134 Gladstone	Front	S39W	3	11.5	11.5	520	850	440	340	Y	N	Valid
	New Food 4 Tht	Road	Left	N51W	3	11.5	11.5	750	750	240	240	Y	N	Valid
			Right	N51W	3	11.5	11.5	370	370	240	240	Y	N	Valid
15	Albert Building	Peel Street	Front	N51W	2	8.5	8.5	1200	1700	420	340	Y	Y	Valid
	cnr Read's Quay	(without ribs)	Left	S39W	2	8.5	8.5	1200	1700	340	340	N	Y	Valid
		(with ribs)	Left	S39W	2	8.5	8.5	1200	1700	420	340	N	Y	Valid
			central	S39W	2	10.5	8.5	440	1210	240	240	Y	N	Valid
			central	S39W	2	10.5	8.5	460	1190	240	240	N	N	Valid
			Right	S39W	2	10.5	8.5	450	1200	240	240	N	N	Valid
16	Trades & Labour	Childers Road	Front	S39W	2	9.5	9.5	460	460	340	240	N	N	Valid
	Building	cnr Customhouse	Left	N51W	2	9.5	9.5	460	460	240	240	N	N	Valid
			Right	N51W	2	9.5	9.5	440	440	340	240	N	N	Valid
			Rear	S39W	2	9.5	9.5	440	440	240	240	N	N	Valid
17	USS Co	12-16 Childers Rd	Front	S39W	2	8.5	8.5	750	750	450	340	N	N	Valid
			Front	S39W	2	8.5	8.5	750	750	450	340	N	N	Valid
			Front	S39W	2	8.5	8.5	750	750	450	340	N	N	Valid
			Left	N51W	2	8.5	8.5	600	1000	240	240	N	Y	Valid
			central	N51W	2	8.5	8.5	610	610	240	240	N	N	Valid
			central	N51W	2	8.5	8.5	610	610	240	240	N	Y	Valid
			Rear	S39W	2	8.5	8.5	590	590	240	240	N	N	Valid
18	Mitchells Camera	Gladstone Road	Front	S39W	2	8.5	8.5	610	910	340	240	Y	N	Valid
	House	cnr Grey St	Left	N51W	2	8.5	8.5	600	600	310	310	N	N	
			Left	N51W	2	8.5	8.5	600	600	240	240	Y	N	Valid
			Right	N51W	2	8.5	8.5	590	890	340	240	Y	Y	Valid

Table 5 - Continued

Bldg No.	Building Description	Street Frontage	Parapet Location	Normal Gisborne EQ Comp	No. of Storeys	Height to parapet support		Parapet Height above crack (or support)		Thickness @		Status (Yes/No)		Valid Parapet Plot Data
						Max (m)	Min (m)	Min (mm)	Max (mm)	ribs (mm)	wall (mm)	visible crack	parapet collapsed	
19	Emmere	Peel Street	Front	N51W	2	8.5	8.5	1010	1010	360	240	Y	N	Valid
	& Hathaway/	cnr Childers	Left	S39W	2	8.5	8.5	1010	1010	240	240	Y	N	Valid
	Radioworks		Right	S39W	2	8.5	8.5	990	990	360	240	Y	N	Valid
			Rear	N51W	2	8.5	8.5	990	990	240	240	Y	N	Valid
20	Dominos	209 Gladstone Rd	Front	S39W	2	8	8	600	600	350	240	Y	N	Valid
			Left	N51W	2	8	8	610	610	240	240	Y	N	Valid
			Right	N51W	2	8	8	590	590	240	240	N	N	Valid
21	Gisborne Motors	Grey St	Front	N51W	1	6.5	4.5	350	2000	350	240	N	N	Valid
			Left	S39W	1	4.5	4.5	510	510	350	240	N	N	Valid
			Right	S39W	1	4.5	4.5	490	490	350	240	N	N	Valid
			Rear	N51W	1	6.5	4.5	450	450	240	240	N	Y	Valid
22	Sports Credentials	Derby St	Left	S39W	1	5	3	460	460	240	240	N	N	Valid
	(Tomo's)	(rear store)	Right	S39W	1	5	3	450	450	240	240	N	N	Valid
			Rear	N51W	1	5	5	440	440	240	240	N	N	Valid
23	Foon Bldg	161-167 Gladstone	Left	N51W	2	7.5	7.5	850	850	240	240	N	Y	Valid
	(rear bldg)	Road	Right	N51W	2	7.5	7.5	850	850	240	240	N	N	Valid

Table 5 - Continued

Bldg No.	Building Description	Street Frontage	Parapet Location	Normal Gisborne EQ Comp	No. of Storeys	Height to parapet support Max (m)	Height to parapet support Min (m)	Parapet Height above crack (or support) Min (mm)	Parapet Height above crack (or support) Max (mm)	Thickness @ ribs (mm)	Thickness @ wall (mm)	Status (Yes/No) visible crack	Status (Yes/No) parapet collapsed	Valid Parapet Plot Data
24	Ardoyne Bldg	161-167 Gladstone	Front	S39W	2	9	8.5	300	1060	530	400	N	N	Valid
	(front bldg)	Road	Front	S39W	2	8.5	8.5	810	1050	530	400	N	N	Valid
			Front	S39W	2	8.5	8.5	790	1040	530	400	N	N	Valid
			Left	N51W	2	8.5	8.5	810	1060	240	240	Y	N	Valid
			central	N51W	2	8.5	8.5	810	810	240	240	Y	N	Valid
			central	N51W	2	8.5	8.5	790	790	240	240	Y	N	Valid
			Right	N51W	2	8.5	8.5	790	1040	240	240	Y	N	Valid
25	Rosies Bldg	Gladstone	Front		2	8.5	8.5				240	Y	N	
		cnr bright	Left		2	8.5	8.5				240	Y	N	
			Right	N51W	2	8.5	8.5	610	610	240	240	N	N	Valid
			Rear	S39W	2	8.5	8.5	590	590	240	240	N	N	Valid
26	Farmers	Gladstone	Front	S39W	2	8.5	8.5	1200	1200	600	600	N	N	Valid
	(West front bldg)	cnr Bright	Left	N51W	2	8.5	8.5	900	1350	240	240	N	Y	Valid
			Right	N51W	2	8.5	8.5	910	1350	240	240	N	N	Valid
			Rear	S39W	2	8.5	8.5	890	900	240	240	N	N	Valid
27	Allen Trading	Customhouse St	Front	N51W	1	5.5	5.5	460	1060	600	450	N	N	Valid
			Left	S39W	1	5.5	5	450	1050	240	240	N	N	Valid
			Right	S39W	1	5.5	5	440	1040	240	240	N	N	Valid
28	Neil Walker Realty	66 Reads Quay	Front	S39W	1	5	5	1050	1050	350	350	N	N	Valid
			Left	N51W	1	5	5	1350	1350	350	350	Y	N	Valid
			Right	N51W	1	5	5	1050	1050	240	240	N	N	Valid
			Rear	S39W	1	5	5	1350	1350	240	240	Y	N	Valid

Table 5 - Continued

Bldg No.	Building Description	Street Frontage	Parapet Location	Normal Gisborne EQ Comp	No. of Storeys	Height to parapet support		Parapet Height above crack (or support)		Thickness @		Status (Yes/No)		Valid Parapet Plot Data
						Max (m)	Min (m)	Min (mm)	Max (mm)	ribs (mm)	wall (mm)	visible crack	parapet collapsed	
29	Lunken Bldg	58 Gladstone Rd	Front	S39W	3	10.5	10.5	2200	2200	450	350	N	N	Valid
			Left	N51W	3	10.5	10.5	800	2200	240	240	N	N	Valid
			Right	N51W	3	10.5	10.5	800	2200	240	240	N	N	Valid

Table 6: Additonal Gisborne Parapet Survey Data - (Data not used in Analysis and Plots)

Bldg No.	Building Description	Street Frontage	Parapet Location	Side of Street (N,S,E,W)	Length between vertical ribs (m)	Length between end supports (m)	End Support 1/Both	Mortar Type (Lime/Cement)	Recessed Flashing @ crack	Collapse In/ Outward	Comments
1	AMI Building	229-233	Front	N	3.5	23	Both	Lime/Cement	NA	NA	Stepped para, horiz gutter. Strengthened after 1966 EQ
		Gladstone Road	Left	N	NA	12	1	Lime	Yes	Both	Gable fell onto adj roof & thru own ceiling
			Right	N	NA	12	1	"	Yes	Out	Truncated gable; fell thru adj shop roof (1-storey) (Ridge-line parallel to road)
2	Moleta Bros -	167-173	Front	N	11	14	Both	Lime/Cement	Yes	NA	+ 450mm ornamental overhang (strengthened)
		Gladstone Road	Left	N	NA	25	1	Lime	Yes	In	Lay on own roof; removed after event
			Right	N	NA	25	1	"	Yes	NA	Horiz & vert cracks; held by Ty-glass
3	Assett Finance	78 Gladstone Rd	Front	S	4	5	Both	Lime	Yes	NA	Separated; frontage removed after event; mono-pitch roof
			Left	S	NA	19	1	"	Yes	Out	Fell thru adj shop roof (single-storey)
			Right	S							No wall; roof supported off adj. bldg
4	Arthur Toye	73-75 Gladstone Road	Front	N	NA	9	Both	Lime	Yes	NA	Front para. moved away from side parapet
			Left	N	NA	18	1	"	Yes	NA	Supported by Odeaon wall
			central	N	NA	18	1	"	Yes	NA	Parapet broken, tilted to East
			Right	N	NA	18	1	"	Yes	In	Broke off at adj (higher) gutter; fell to west (party-wall)
5	Grant Bros	69-71 Gladstone Road	Front	N	5	10	Both	Lime	Yes	NA	Gable roof; parapet moved out 10-50mm from side walls
			Left	N	NA	14	1	"	Yes	Out	Broke off at flashing (party-wall); fell on adj roof
			Right	N	NA	14	1	"	Yes	NA	Moved, supported by adj gable
6	Vitality Foods	67 Gladstone Rd	Front	N	5	10	Both	Lime	Yes	NA	Moved out 5-10mm at eastern end
			Left	N	NA	15	1	"	Yes	NA	Gable; partly supported by adj parapet
			Right	N	NA	15	1	"	Yes	Out	Gable (party-wall); partly collapsed
7	Smiths City	Bright Street	Front	E	NA	38	Both	Lime	NA	NA	No damage
		cnr Childers Rd	Left	E	NA	29	Both	"	Yes	NA	Cracked & moved out but stayed in place
			Right	E	4	58	Both	"	NA	NA	No damage

Table 6 - Continued

Bldg No.	Building Description	Street Frontage	Parapet Location	Side of Street (N,S,E,W)	Length between		End Support (1/Both)	Mortar Type (Lime/Cement)	Recessed Flashing @ crack	Collapse In/ Outward	Comments
					vertical ribs (m)	end supports (m)					
8	Bates Building	2 Lowe Street	Front	E	3.5	37	Both	Lime	Yes	NA	Bldg strengthened prior; some cracking RH end
			Left	E	4	12	Both	"	NA	NA	No damage
			Right	E	NA	11.5	1	"	Yes	NA	Gable end; held by roof framing; wall buttressed
9	Geneva Finance	Gladstone Road cnr Derby	Front	N	5	15	Both	Lime	NA	NA	Racked; moved West30mm
			Left	N	5	28	Both	"	NA	NA	Moved out 30mm at South end
			Right	N	NA	6	1	"	Y	Out	Fell on & thru adj bldg roof (AMI) (party-wall)
			Rear	N	NA	10	1	"	NA	NA	Stiffened by conc diaphragm 1st floor
10	ex Adair's/ Pumpkin Patch	Gladstone Road cnr Grey St	Front	N	NA	20	Both	Lime/Cement		NA	Building strengthened prior to EQ
			Left	N	NA	44	"	"		NA	
			Right	N	NA	44	"	"		NA	Vertical steel channel bracing externally
			Rear	N	NA	20	"	"		NA	
11	Melbourne Cash (Ian Penny)	118-122 Gladstone Road	Front	S	3.5	12	Both	Lime		NA	Full width crack
			Left	S		18	"	"		NA	Diag crack near front
			Right	S		14	"	"	Y	Out	Collapsed
			Rear	S		10	"	"		NA	Bowed-out
12	Ex Lyric Café (Ray Moleta) Old Food 4 Tht	124 Gladstone Rd	Front	S		7	Both	Lime		NA	Crack at gutter-line
			Left	S		15	"	"	Y	In	Collapsed down to ceiling level; double brick cavity wall
			Right	S		15	"	"			Party wall - not a parapet
			Rear	S		7	"	"		NA	Gable end; matches side paras at ends
13	T Adair's Building New Food 4 Tht	126-134 Gladstone Road	Front	S		20	Both	Lime/Cement	N		Lateral ridge-line;
			Left	S		14	"	"	N		Ground floor and stairwell strengthened prior to EQ
			Right	S		14	"	"	N		Diag crack at front; damaged at rear corner

Table 6 - Continued

Bldg No.	Building Description	Street Frontage	Parapet Location	Side of Street (N,S,E,W)	Length between vertical ribs (m)	Length between end supports (m)	End Support (1/Both)	Mortar Type (Lime/Cement)	Recessed Flashing @ crack	Collapse In/ Outward	Comments
15	Albert Building	Peel Street	Front	E	7.5	26+15+15	Both	Lime/Cement		Out	Vert and horiz cracking; 5% collapsed at Nth end
	cnr Read's Quay	(without ribs)	Left	E	7.5	16	Both			Out	Section between ribs that failed
		(with ribs)	Left	E	7.5	16	Both			Out	Vert and horiz cracking; 30% collapsed at West end
			central	E		15	1				Gable; horiz crack at apex
			central	E		15	1				Gable; horiz crack at apex
			Right	E		15	1				Gable against adjacent bldg
16	Trades & Labour	Childers Road	Front	S	4.5	13	Both	Lime			
	Building	cnr Customhouse	Left	S		14+6	"	"			Rear hammered by USSCo
			Right	S		20	"	"			
			Rear	S		9+4	"	"			Rear hammered by USSCo
17	USS Co	12-16 Childers Rd	Front	S		11	Both	Lime	N		
			Front	S		8	"	"	N		
			Front	S		9	"	"	N		
			Left	S		21	"	"	Y	Out	75% collapsed up to 1.0m from top
			central	S		21	"	"	N		
			central	S		21	"	"	Y	In	50% collapsed onto own roof
			Rear	S		9	"	"	N		
			Rear	S		8	"	"	N		
18	Mitchells Camera	Gladstone Road	Front	N	5	10	Both	Lime/cement	Y		Partially strengthened prior to EQ
	House	cnr Grey St	Left	N		15	"	Concrete	Y		Early reinf conc; craze-cracking over whole wall
			Left	N		13.5	1	Lime/cement			
			Right	N		30	1	"	Y	Out	30% collapsed onto verandah canopy

Table 6 - Continued

Bldg No.	Building Description	Street Frontage	Parapet Location	Side of Street (N,S,E,W)	Length between vertical ribs (m)	Length between end supports (m)	End Support (1/Both)	Mortar Type (Lime/Cement)	Recessed Flashing @ crack	Collapse In/ Outward	Comments
19	Emmere	Peel Street	Front	E	3	15	Both	Lime/Cement			Cracked
	& Hathaway/	cnr Childers	Left	E		12	"	"		Out	Small piece fallen-out
	Radioworks		Right	E	3	12	"	"		Out	Small piece fallen-out
			Rear	E	3	15	"	"		Out	Small piece fallen-out
20	Dominos	209 Gladstone Rd	Front	N	3	9	Both	Lime/Cement	N		Bldg strengthened to 1st Fl ceiling level prior to EQ
			Left	N		9	1	"	N		
			Right	N		9	1	"	N		
21	Gisborne Motors	Grey St	Front	W	7	20	Both	Lime/Cement			
			Left	W	3.6	48	"	"			Two internal trusses cracked
			Right	W	3.6	48	"	"			
			Rear	W		20	"	Lime	N	Out	Previously strengthened; top of gable end collapsed
22	Sports Credentials	Derby St	Left	E		6	Both	Lime	-		Roof slopes up to rear (East) wall
	(Tomo's)	(rear store)	Right	E		6	1	"	Y		
			Rear	E		13	Both	"	-		Sth end wall very dodgey
23	Foon Bldg	161-167 Gladstone	Left	N		NA	None	Lime	Y	In	West parapet fell on own roof
	(rear bldg)	Road	Right	N		NA	"	"	-	-	East parapet removed after event
24	Ardoyne Bldg	161-167 Gladstone	Front	N	6	8	Both	Lime	-		Old vert cracks; 500w x 300d o/hang; façade reinf conc retrofit
	(front bldg)	Road	Front	N	7	5	"	"			"
			Front	N	6	6	"	"			"
			Left	N		16	1	"	Y		Extensive cracking
			central	N		16	1	"	Y		Extensive cracking; held by pipe struts & sheet metal cladding
			Right	N		16	1	"	Y		Ditto - no struts; half length supported by adj bldg

Table 6 - Continued

Bldg No.	Building Description	Street Frontage	Parapet Location	Side of Street (N,S,E,W)	Length between vertical ribs (m)	Length between end supports (m)	End Support (1/Both)	Mortar Type (Lime/Cement)	Recessed Flashing @ crack	Collapse In/ Outward	Comments
25	Rosies Bldg	Gladstone	Front	N	5	11	Both	Concrete			Reinf conc retrofit 1966; horiz cracks at several levels
		cnr bright	Left	N	5	24	"	"			Ditto
			Right	N		24	-	Lime			Brick
			Rear	N		11	-	"			Brick
26	Farmers (West front bldg)	Gladstone cnr Bright	Front	S		10	Both	Lime/Cement	N		600 thick facade parapet
			Left	S		13	"		N	In	Collapsed to west on own roof
			Right	S		13	"	"	Y	In	Diag crack at front; rear 70% strengthened prior to EQ
			Rear	S		10	"	"	N		Strengthened prior to EQ
27	Allen Trading	Customhouse St	Front	E		20	Both	Lime/Cement	N		Front wall moved and bowed forward from roof flashing
			Left	E		12	1	"	N		Supported by adjacent bldg; roof slopes back from façade
			Right	E		12	1	"	N		Ditto
28	Neil Walker Realty	66 Reads Quay	Front	S		12	Both	Lime/Cement	N		
			Left	S		35	"	"	Y		Severe cracking at support level; pounded
			Right	S		40	"	"	N		by adjacent building
			Rear	S		11	"	"	N		Cracking at S-E cnr
29	Lunken Bldg	58 Gladstone Rd	Front	S		9	Both	Lime/Cement	N	-	Very high front parapet - poss. reinf. Conc; 500w x300d o/hang
			Left	S		12	1	"	N	-	Diag crack at front cnr
			Right	S		12	1	"	N	-	Ditto

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