ENG 310-(EQC 2001/468)

Methodology for assessing the seismic performance of unreinforced masonry single storey walls, parapets and free standing walls

E L Blaikie, Opus International Consultants Ltd

6 P01/468

Report Prepared for the EQC Research Foundation

ENG
310

OPUS
INTERNATIONAL
CONSULTANTS

Methodology for Assessing the Seismic Performance of Unreinforced Masonry Single Storey Walls, Parapets and Free Standing Walls

Opus: an accomplished work, a creation, an achievement

Report Prepared for the EQC Research Foundation

Methodology for Assessing the Seismic Performance of Unreinforced Masonry Single Storey Walls, Parapets and Free Standing Walls

Prepared By

E L Blaikie
Senior Design Engineer

Reviewed By

R A Davey
Principal Consultant, Structural

Opus International Consultants Limited
Wellington Office
Level 9, Majestic Centre
100 Willis Street, PO Box 12-003
Wellington, New Zealand

Telephone:	+64 4 471 7000
Facsimile:	+64 4 471 1397
Email	ted.blaikie@opus.co.nz

Date:	29 May 2002
Reference:	C0103.00
Status:	Final

Abstract – Non-Technical

In the early 1980's US researchers subjected full-scale unreinforced masonry (URM) face-loaded wall specimens to earthquake motions. These specimens representing a wall element spanning 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.

Subsequently this concept was used to develop the current New Zealand Society of Earthquake Engineering (NZSEE) Guidelines for the assessment of face-loaded walls. Previous research by the author indicated that these current Guidelines were often very non-conservative. This previous research also led to the development of a more reliable methodology for assessing the earthquake behaviour of face-loaded URM.

In this study the assessment methodology is extended to cover face-loaded single-storey walls, freestanding walls supported only by the ground, and parapets.

The methodology was extended and developed using the results obtained when these types of wall were modelled on a computer. The computer model was verified by comparing the walls predicted behaviour with the results obtained from a wall specimen recently tested on shake table by Australian researchers.

Design charts are provided to enable rapid design office assessment of face-loaded wall elements.

Abstract –Technical

An assessment methodology, developed in previous research that can be used to predict the seismic stability of a cracked face-loaded unreinforced masonry (URM) multi-storey wall was extended in this study to cover single-storey walls, freestanding cantilever walls and parapets.

The methodology makes use of both the acceleration and displacement response spectra for an earthquake motion. The acceleration spectrum is used to predict the earthquake intensity that will just open the joint cracks in the wall element. 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. Modification factors are applied to allow for the effect of the wall element boundary conditions and to allow for amplification of the earthquake motion due to flexibility in any building structure or diaphragms that support the wall element.

The methodology was principally developed from the results of inelastic dynamic analyses of computer models of a face-loaded URM walls supported by flexible shear walls and flexible or yielding floor and roof diaphragms. Parameters examined included interaction between the parapet and its supporting wall and the effect of building and/or diaphragm flexibility. The effects of long acceleration pulses in the ground motion, which may occur in the near-fault zone during an earthquake, were also evaluated.

The analyses indicated that the earthquake intensity required to collapse a face-loaded wall element, as indicated by the computer modelling, is generally predicted conservatively by the proposed methodology. However, when the earthquake motion contains a near-fault long duration pulse, the methodology is not conservative and tends to predict the mean earthquake intensity required to collapse the wall element. The methodology also becomes non-conservative for near-fault EQ motions if the building and/or diaphragms have more than moderate flexibility.

Design charts are provided to enable rapid design office assessment of face-loaded wall elements in terms of the current proposed revision to the NZS4203 Loading Standard Basic Seismic Hazard Spectra (i.e. DR 1170.4/PPC3). Similar design charts could be prepared for other earthquake records or code design spectral intensities using the proposed methodology.

Laboratory testing of pre-cracked face-loaded wall specimens under simulated seismic loading has recently been carried out in Australia. Good agreement was obtained when inelastic dynamic modelling was used to predict the time-history of the mid-height displacement of one of the Australian test specimens. This agreement should improve confidence in the assessment methodology that was derived using similar computer models.

Contents

1 Introduction

In the 1980's a consortium of Californian engineers, called the ABK Joint Venture (ABK, 1982), carried out a pioneering investigation into the post-cracking seismic resistance of unreinforced masonry (URM). This investigation included the testing of full-scale specimens representing a face loaded wall element spanning between two adjacent floor diaphragms.

When the test specimens were subjected to earthquake motions imposed at the supporting floor diaphragm levels, ABK 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. These 2 cracks acted as fuses and no further intermediate cracking was observed. During the test, the 2 cracked joints opened up and allowed the centre of the wall to undergo large displacements, comparable with the wall thickness. This ability to withstand large displacements without collapse resulted in the walls having a significant post cracking seismic resistance. ABK used the term "dynamic stability" to distinguish this type of behaviour from the behaviour that might have been expected from static force calculations where "failure" is assumed to occur when the wall cracks.

Not listed on pg 67

Subsequent to the ABK investigations, Priestley (1985) developed an~~d~~ equal energy procedure for the assessment of face-loaded masonry walls. This Methodology was incorporated into draft Guidelines for Assessing and Strengthening Earthquake Risk Buildings that have been published by the New Zealand National Society of Earthquake Engineering (NZSEE, 1995).

The author (Blaikie, 1992 and 2000) developed an alternative methodology that uses acceleration and displacement response spectra to predict the earthquake intensity that will cause a face-loaded wall element to collapse. This methodology was shown to give a more accurate prediction of a walls seismic capacity than the NZSEE Guidelines. where?

In this current study the parameters proposed previously by the author for single-storey walls are developed and refined and the methodology is extended to cover parapets, freestanding walls and single-storey walls without effective top support.

The scope of the current study encompasses:

1. Derivation of the equations that describe the behaviour of cracked face-loaded URM wall elements. These equations are used in the proposed assessment methodology.
2. Development of inelastic dynamic computer models for the type of the wall elements evaluated in this study.
3. Verification of the computer modelling by comparing the wall response predicted by the model with the results of a laboratory wall specimen that had been subjected to simulated seismic face-loading.
4. Analysis using the computer models to determine the collapse earthquake intensity of a range of single-storey and parapet wall configurations. These analyses

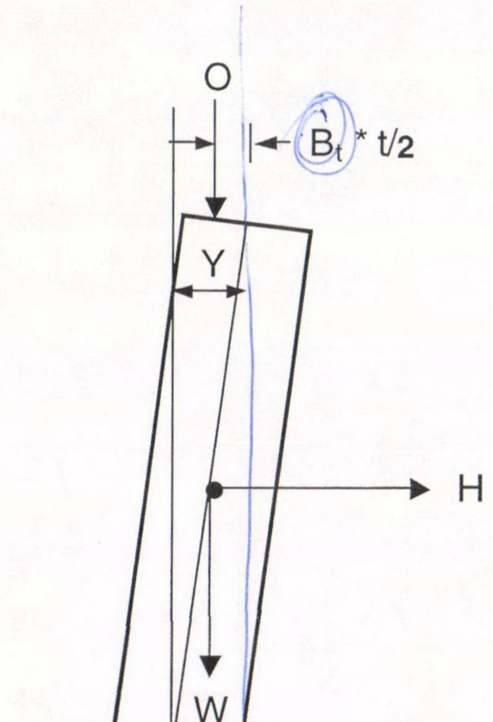
included variations in the flexibility of the buildings and diaphragms supporting the wall elements.

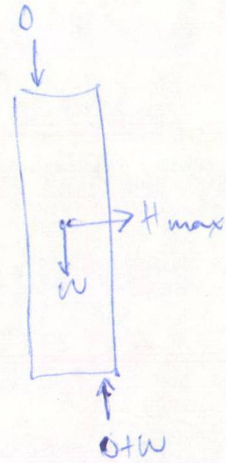
5. Development of the methodology based on the computer modelling and the equations describing the behaviour of cracked wall elements. This methodology uses the acceleration and displacement spectra for an earthquake motion to predict the earthquake intensity expected to cause a wall element to collapse.
6. Assessment of the effects of long duration pulses expected in some near-fault earthquake motions and their implications for the methodology.
7. Development of charts, based on the methodology, which can be used for the rapid seismic assessment of face-loaded wall elements.

2 Behaviour of Cracked Face Loaded URM Wall Elements

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

Figure 1 shows the forces assumed to act on a parapet or freestanding wall when a horizontal force H at the mid-height of the wall displaces the top of the wall element laterally. The wall element has a height, h, total weight, W, and effective thickness, t, and is considered here as a rigid body. The overburden load, O, represents, for example, the weight of a roof structure that is ineffective as a roof diaphragm. It is assumed to act at an eccentricity of B_t * t/2 relative to the centre of the wall element, where the ratio B_t is considered to be positive when the overburden improves the stability of the wall element. At the base of the wall element the vertical reaction, O + W, is assumed to act near the face of the wall at a point that is B_b* t/2 from the wall centreline. As a small compression zone depth would be required to develop the reaction 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}.





$$H_{max}\frac{h}{2} + (O+W)\frac{t}{2}(1-B_b) - O(1+B_t)\frac{t}{2} - \frac{Wt}{2} = 0$$

$$\Rightarrow H_{max}\,h = t\left[O(1+B_t) - (O+W)(1-B_b)\right.$$

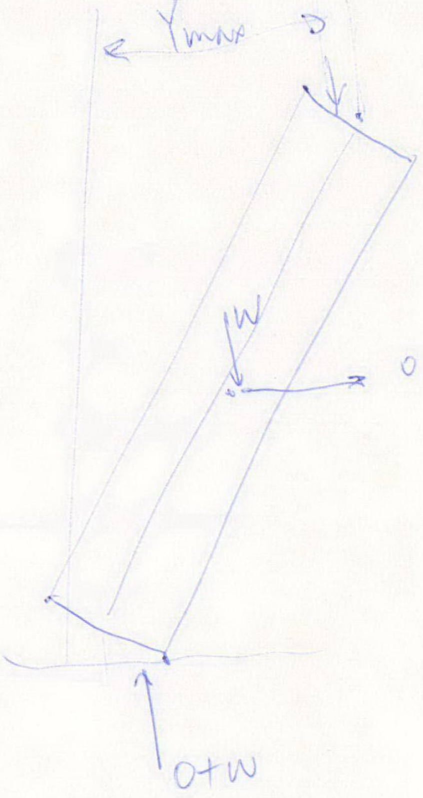
Figure 1: Behaviour of face loaded Cantilever or Free Standing Wall under Static Loading

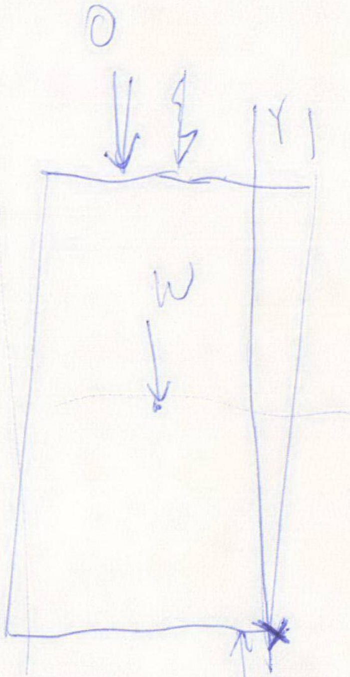
$$\Rightarrow H_{max} = \frac{t}{h}\left[\cancel{O} + OB_t - \cancel{O} + OB_b + W - WB_b\right]$$

$$= \frac{t}{h}\left[OB_t + WB_b + OB_b\right]$$

$$= \frac{tW}{h}\left[B_b + \frac{O}{W}(B_b + B_t)\right]$$

$$Y_{max} = \frac{1}{2} t \frac{\left[B_b + \frac{O}{W}(B_b + B_t)\right]}{\left(1 + \frac{2O}{W}\right)}$$





$$W\left(\frac{t}{2} - \frac{Y_{max}}{2}\right) + O\left(\frac{t}{2} + B_t\frac{t}{2} - Y_{max}\right) - (W+O)(1-B_b)\frac{t}{2} = 0$$

$$W\frac{t}{\cancel{2}} - W\frac{Y_{max}}{\cancel{2}} + O\frac{t}{\cancel{2}} + OB_t\frac{t}{\cancel{2}} - 2O\,Y_{max}\,\frac{t}{\cancel{2}}\left[W - WB_b + O - OB_b\right] = 0$$

$$Y_{max}(+W+2O) = Wt + Ot + OB_t t - t\left[W - WB_b + O - OB_b\right]$$

$$= \cancel{Wt} + \cancel{Ot} + OB_t t - \cancel{tW} + WB_b t - \cancel{Ot} + OB_b t$$

$$= t\left[\cancel{\quad}\right] = t\left[WB_b + O(B_b + B_t)\right]$$

$$= t\left[\; O(B_t + B_b)\right.$$

Neglecting elastic displacements

The horizontal force, H, will have a maximum value, H_{max} when a crack just starts to form at the base of the wall element (i.e. displacement at the top of the wall element Y = 0.0). By equating moments about the pivot point at the base of the wall when Y = 0.0 it can be easily shown that:

How much of this is original?

$$H_{max} = \frac{tW}{h}\left(B_b + \frac{O}{W}(B_b + B_t)\right) \qquad \textbf{Eqn 1}$$

static

The wall element will become unstable when H = 0.0. At this point the wall element displacement will have its maximum value = Y_{max}. By equating moments about the pivot point at the base of the wall it can also easily be shown that:

$$Y_{max} = t\,\frac{\left(B_b + \frac{O}{W}(B_b + B_t)\right)}{\left(1 + \frac{2O}{W}\right)} \qquad \textbf{Eqn 2}$$

It is convenient to express H_{max} and Y_{max} in terms of a reference wall element. For the reference wall element the reaction at the base of the wall acts at an eccentricity of t/2 (i.e. B_b = 1.0) and the overburden is applied at the centre of the wall element (i.e. B_t = 0.0). In this case:

$$H_{max} = \frac{tW}{h}\left(1 + \frac{O}{W}\right) * F_{fixity} \qquad \textbf{Eqn 3}$$

And:

$$Y_{max} = t\,\frac{\left(1 + \frac{O}{W}\right)}{\left(1 + \frac{2O}{W}\right)} * F_{fixity} \qquad \textbf{Eqn 4}$$

Where the top and bottom fixity factor, F_{fixity} = 1.0 for the reference wall element and for other cases is given by:

$$F_{fixity} = \frac{\left(B_b + (B_b + B_t)\frac{O}{W}\right)}{\left(1 + \frac{O}{W}\right)} \qquad \textbf{Eqn 5}$$

The horizontal load shown in Figure 1 may be considered to be the resultant of an equivalent static seismic load that is uniformly distributed over the height of the wall

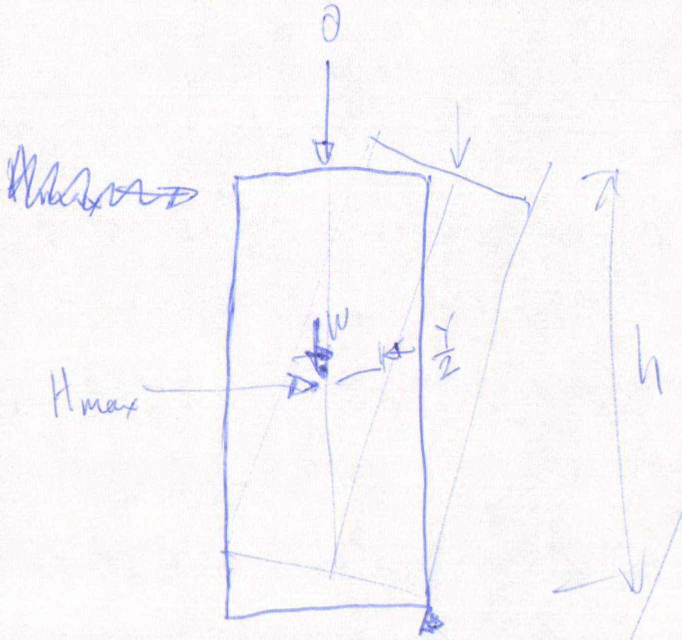
Call $J = t\left(1+\frac{O}{W}\right) * \text{Fixity}$

$\Rightarrow \quad H_{max} = \frac{W}{h} J$

$$\frac{t}{2}(O+W) = \left(O+\frac{W}{2}\right) y_{max}$$

$$\boxed{y_{max} = \frac{t}{2}\frac{(O+W)}{\left(O+\frac{W}{2}\right)}}$$

$$Y_{max} = \frac{J}{\left(1+\frac{2O}{W}\right)}$$



$$H = H_{max}\left(1 - \frac{\frac{2}{t}\left(O+\frac{W}{2}\right)y}{(O+W)}\right)$$

$$= H_{max}\left(1 - \frac{y}{y_{max}}\right)$$

$$H_{max}\frac{h}{2} = O\frac{t}{2} + W\frac{t}{2}$$

or $H_{max} = \frac{2t}{h\,2}(O+W) = \frac{t}{h}(O+W)$

when displaced,

$$H_{max}\frac{h}{2} = O\left(\frac{t}{2} - y_{max}\right) + W\left(\frac{t}{2} - \frac{y_{max}}{2}\right)$$

$$H\frac{h}{2} = O\frac{t}{2} - O y_{max} + \frac{Wt}{2} - \frac{Wy}{2} = \frac{t}{2}(O+W) - \left(O+\frac{W}{2}\right)y$$

$$H = \frac{t}{h}(O+W) - \frac{2}{h}\left(O+\frac{W}{2}\right)y$$

element. In this case the seismic coefficient that corresponds to a crack just starting to open at the base of the wall, C_d is given by:

$$C_d = \frac{H_{max}}{W} = \frac{t}{h}\left(1 + \frac{O}{W}\right) * F_{fixity} \qquad \textbf{Eqn 6}$$

The relationship between the mid-height horizontal load (H) and the displacement at the mid-height of the wall (Y/2) is shown graphically in Figure 2. In practice the wall element would not be rigid and would have some elastic response as indicated.

How is it shown that this is linear

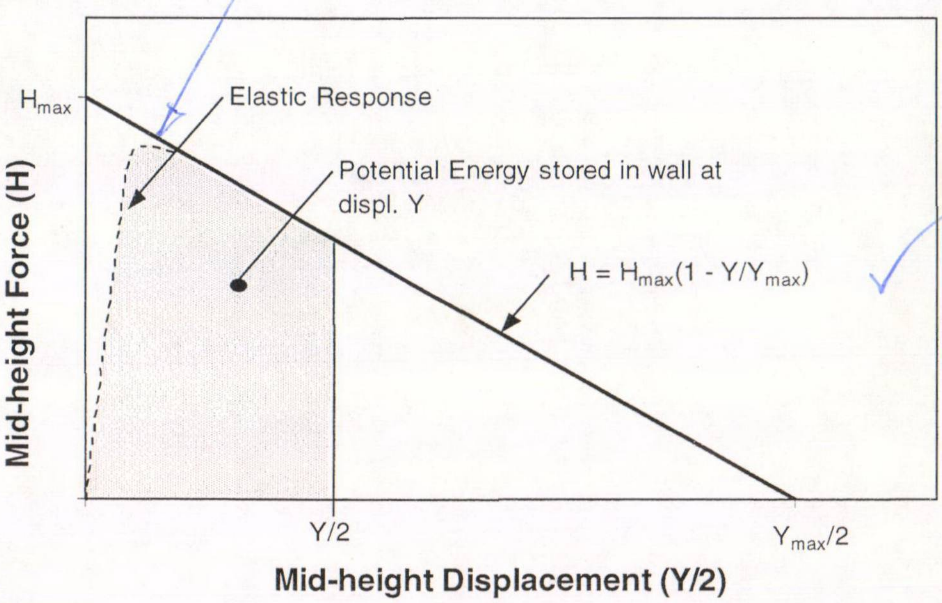
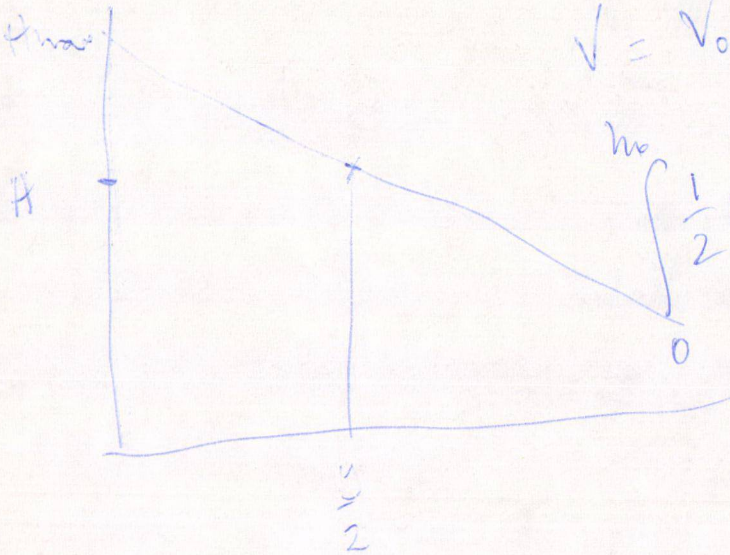


Figure 2: Mid-Height Load and Displacement Relationship for Parapet or Cantilever Wall

2.2 Period of Free Vibration Response

If the wall element indicated in Figure 1 was released from its displaced position, the free damped response of the wall will be similar to that shown in Figure 3 (Blaikie, 2000).



$$v = v_0 \frac{h}{h_0}$$

$$\int_0^{h_0} \frac{1}{2} m v_0^2 \frac{h^2}{h_0^2}\, dh$$

$$\frac{1}{2} m \frac{v_0^2}{h_0^2} \left[\frac{h^3}{3}\right]$$

$$\frac{1}{2} m \frac{v_0^2}{h_0^2} \frac{h_0^3}{3}$$

$$E = \int H y = \frac{1}{2}(H + H_{max}) \frac{y}{2} =$$

$$E_k = \int_0^{h_0} \frac{1}{2} m v^2\, dh$$

where $v = v_0 \left(1 - \frac{h}{h_0}\right) = \left(v_0 - \frac{v_0 h}{h_0}\right)$

$$= \frac{1}{2} m \int_0^{h_0} \left[v_0 \left(1 - \frac{h}{h_0}\right)\right]^2 dh$$

$$= \frac{1}{2} m \int_0^{h_0} \left[v_0^2 - 2 v_0^2 \frac{h}{h_0} + v_0^2 \left(\frac{h}{h_0}\right)^2\right] dh$$

$$= \frac{1}{2} m v_0^2 \int_0^{h_0} \left(1 - 2\left(\frac{h}{h_0}\right) + \left(\frac{h}{h_0}\right)^2\right) dh = \frac{1}{2} m v_0^2 \left[h - 2\frac{h^2}{2h_0} + \frac{h^3}{3h_0^2}\right]_0^{h_0}$$

$$= \frac{1}{2} m v_0^2 \left[\cancel{\quad} h_0 - h_0 + \frac{h_0}{3}\right]$$

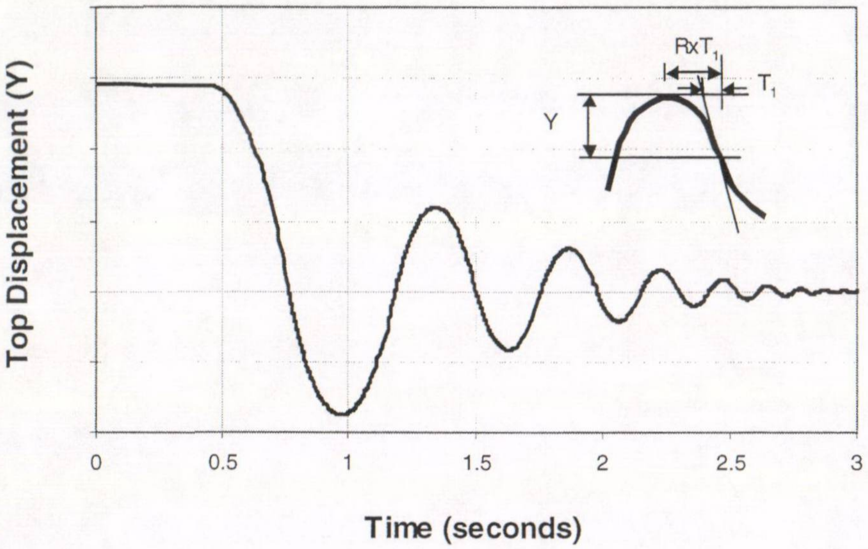


Figure 3: Free Damped Response of Parapet or Cantilever Wall

It can be seen that the free vibration period of the wall element decreases with decreasing lateral wall displacement. The peak potential energy, E_o, stored in the wall at a displacement Y can be calculated with reference to Figure 2:

Subtle change in definition of Y

$$E_o = (H + H_{max})\frac{Y}{4}$$

$$= H_{max}\left(2 - \frac{Y}{Y_{MAX}}\right)\frac{Y}{4} \qquad \text{Eqn 7}$$

Is this not the difference in kinetic energy

This peak potential energy will be equal to the peak kinetic energy, E_k, stored in the wall at zero displacement (if losses and second order terms are ignored).

$$E_k = \frac{MV_o^2}{6} \quad \text{and therefore}$$

$$V_o^2 = \frac{3\,Y\,H_{max}}{2\,M}\left(2 - \frac{Y}{Y_{max}}\right) \qquad \text{Eqn 8}$$

Where V_o is the maximum velocity at the top of the wall element at zero displacement, Y is the peak displacement at the top of the wall element and M is the total mass of the wall element. The velocity V_o can be evaluated before or after impact by using the peak displacement, Y, in the corresponding half cycles before or after impact respectively.

It has been shown previously (Blaikie, Spurr, 1992), that the shape of each of the half cycles in the response of a face-loaded single storey masonry wall is practically independent of the peak displacement and other parameters such as wall element thickness.

$$\frac{M V_0^2}{6} = H_{max}\left(2 - \frac{Y}{Y_{max}}\right)\frac{Y}{4}$$

$$V_0^2 = \frac{3\cancel{6}}{2\cancel{4}} \frac{H_{max}\, Y}{m}\left(2 - \frac{Y}{Y_{max}}\right)$$

$$T = 4RT_1 = 4R\frac{V_0}{Y}$$

$$= \frac{4R}{Y}\sqrt{\frac{3 Y H_{max}}{2M}\left(2 - \frac{Y}{Y_{max}}\right)}$$

$$= \frac{4R}{Y}\sqrt{\frac{3Yt}{h}\frac{(O+W)}{M}\left(2 - \frac{2Y\left(O+\frac{W}{2}\right)}{t/h\,(O+W)}\right)}$$

$$= \frac{4R}{Y}\sqrt{\frac{3Yt}{h}\left(\frac{O+W}{M}\right)2\left(1 + \frac{Y}{t}\frac{\left(O+\frac{W}{2}\right)}{(O+W)}\right)}$$

This property of the half cycle response is illustrated in the inset diagram in Figure 3. From the diagram it can be seen that the full cycle period is given by:

$$T = 4 R T_1 \qquad \textbf{Eqn 9}$$

where $T_1 = \frac{V_o}{Y}$ and the Period Shape Factor, R, is assumed to have a constant value.

Substituting for T_1 and then V_o in Eqn 9 and then substituting for H_{max} and Y_{max}, the period of the wall free vibration is given by:

$$T = \sqrt{\frac{K\,h}{\left(1 + 2\frac{O}{W}\right)}} \qquad \textbf{Eqn 10}$$

What is K in the evaluation?

Not a proper sentence

when the peak displacement is $0.6Y_{max}$. Where K is a constant and the units for the wall element height, h is in meters.
Previous researchers (Yim, Chopra and Penzien, 1980) have developed a similar equation for the period of a rigid rocking body of width, t, and height, h. Re-arranging their equation the constant, K, in Eqn 10 above, would have the value:

$$K = 2.65\sqrt{1 + \left(\frac{t}{h}\right)^2} \qquad \textbf{Eqn 11}$$

Not true

In this project a value of K= 2.8 has been adopted. This value makes an allowance for the effect that squat wall slenderness ratios will have on the value of K in Eqn 10 and for the effect that elastic deflections of slender wall elements will have on the period.

2.3 Behaviour of Face-Loaded URM Single-storey Walls

The behaviour of cracked face loaded single storey URM walls is described in previous research reports, (Blaikie, Spurr, 1992 and Blaikie, 2000). It is summarized here so that the current report can be read without reference to the earlier work.

Face loaded walls in URM buildings typically span vertically between floor framing. They may also be supported by roof framing or by the ground. When subjected to sufficient lateral load, a multi-storey URM wall can be expected to crack at the level of the supports and near the mid-height of the wall elements that span between the supports, providing the supports do not fail. Reference

Figure 4 shows the forces assumed to act on a cracked wall element spanning, h, between supports and subjected to a uniformly distributed lateral static load H. The wall has a total weight, W, and effective thickness, t. The overburden load, O, represents the weight of a parapet or the weight of any upper storey walls. It is assumed to be applied at an eccentricity $B_t * t/2$ from the wall centreline where B_t is considered to be positive when the overburden improves the stability of the wall element.

$$T = 4RT_1$$

$$= 4R\frac{V_0}{Y}$$

$$= \frac{4R}{Y}\sqrt{\frac{3Y\,H_{max}}{2M}\left(2 - \frac{Y}{Y_{max}}\right)}$$

$$= \frac{4R}{Y}\sqrt{\frac{3Y}{2M}\,\frac{H}{(1 - Y/Y_{max})}\left(2 - Y/Y_{max}\right)}$$

and if $\frac{Y}{Y_{max}} = 0{\cdot}6$ then $\frac{2 - Y/Y_{max}}{1 - Y/Y_{max}}$

$$= \frac{2 - 0{\cdot}6}{1 - 0{\cdot}6}$$

$$= \underbrace{\frac{1.4}{0{\cdot}4}} = \frac{14}{4} = \frac{7}{2} = 3{\cdot}5$$

$$= \frac{4R}{Y}\sqrt{\frac{3}{2} \times \frac{3.5}{m}\, h\, g}$$

At the base of the wall element the vertical reaction, O + W, is assumed to act near the face of the wall at a point that is $B_b * t/2$ from the wall centreline. As a small compression zone depth would be required to develop the reaction 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}.

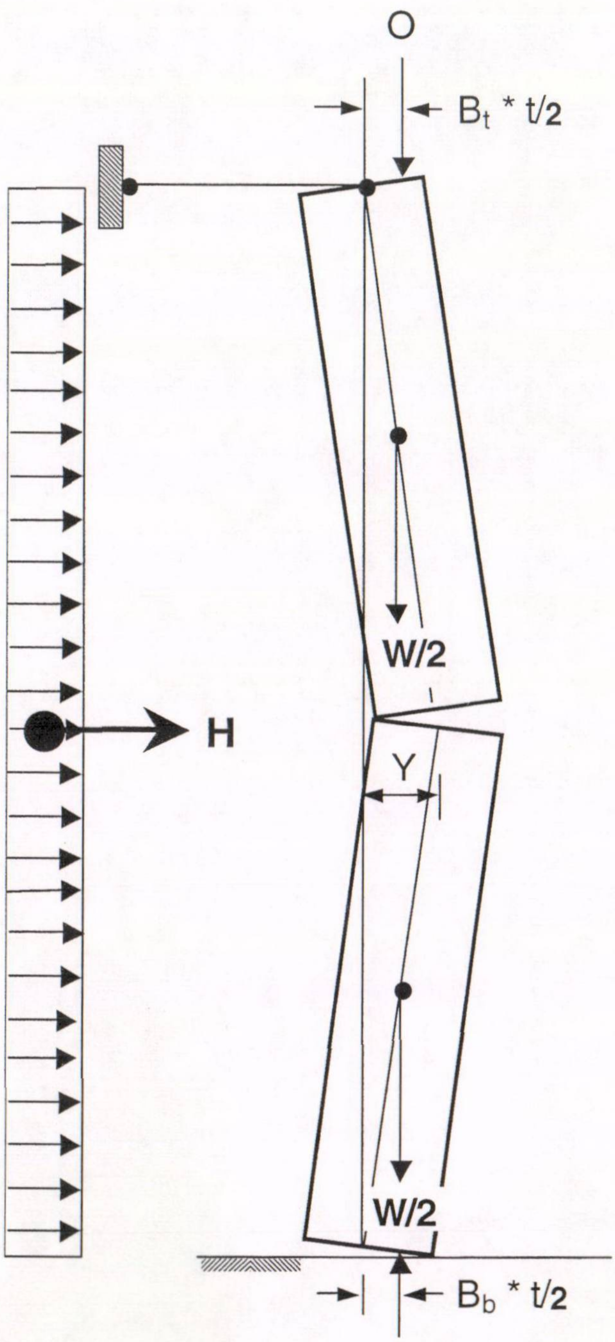


Figure 4: Behaviour of Face Loaded Single-Storey Wall Element under Static Loading

The equations describing the behaviour of a face loaded single storey wall under static loading and its free vibration period are remarkably similar to those derived in the previous two sections of this report for a parapet or cantilever wall. The equations describing the behaviour of a face loaded single storey wall under static loading are:

$$H_{max} = \frac{4\,t\,W}{h}\left(1 + 1.5\frac{O}{W}\right) * F_{fixity} \qquad \textbf{Eqn 12}$$

Where H_{max} is the maximum value of H that occurs when the wall cracks just start to open (i.e. Y=0.0) and;

$$Y_{max} = t\,\frac{\left(1+1.5\frac{O}{W}\right)}{\left(1+\frac{2O}{W}\right)} * F_{fixity} \qquad \textbf{Eqn 13}$$

Where Y_{max} is the mid-height wall displacement at which the wall becomes unstable. The top and bottom fixity factor, F_{fixity} = 1.0 for a reference wall where the reaction at the base of the wall acts at an eccentricity of t/2 (i.e. B_b = 1.0) and the overburden is applied at the centre of the wall element (i.e. B_t = 0.0). For other cases F_{fixity} is given by:

$$F_{fixity} = \frac{\left((1+B_b) + (2+B_b+B_t)\frac{O}{W}\right)}{2\left(1+1.5\frac{O}{W}\right)} \qquad \textbf{Eqn 14}$$

When the uniformly distributed horizontal load, H, shown in Figure 4 is considered to be an equivalent static seismic load the seismic coefficient, C_d that corresponds to the 2 cracks in the wall just starting to open is given by:

$$C_d = \frac{H_{max}}{W} = \frac{4\,t}{h}\left(1 + 1.5\frac{O}{W}\right) * F_{fixity} \qquad \textbf{Eqn 15}$$

The maximum velocity at the mid-height of the wall element, V_o, occurs when the joint cracks close and the mid-height wall displacement is zero and is given by:

$$V_o^2 = \frac{3\,Y\,H_{max}}{2\,M}\left(2 - \frac{Y}{Y_{max}}\right) \qquad \textbf{Eqn 16}$$

Where, Y is the peak displacement at the mid-height of the wall and M is the total mass of the wall element. The velocity V_o can be evaluated before or after impact (i.e. at zero displacement) by using the peak displacement, Y, in the corresponding half cycles before or after impact respectively.

The period of the wall free vibration in the cracked condition is given by:

What happens to the parapet.

$$T = \sqrt{\frac{0.7\,h}{\left(1 + 2\frac{O}{W}\right)}} \qquad \textbf{Eqn 17}$$

when the peak displacement is $0.6Y_{max}$ and where the wall element height, h is in meters.

When Eqn 10 and Eqn 17 are compared it can be seen that a parapet wall will have the same cracked fundamental period of response as a single storey wall that is 4 times as high if it has the same overburden to weight ratio. Robinson (2001) has evaluated the equations of motion for both a rocking rigid cantilever block and a cracked single-storey wall element and shown that this relationship is to be expected. The constant in Eqn 10 (K=2.8 = 4 x 0.7) was selected so that this relationship would be retained.

When Eqn 6 and Eqn 15 are compared it can been seen that the seismic coefficient, C_d corresponding to first crack opening at the base of a parapet wall (without overburden) is also the same as that expected for opening the cracks in a single storey wall 4 times as high. Australian researchers (Lam N, Wilson J L, Hutchinson G L , 2001) have recently made the observation that ***a cracked parapet cantilever wall can be simulated by a single storey wall 4 times as high***. It will be shown that this relationship only holds where the cracked cantilever wall or parapet can be considered as rigid blocks (i.e. squat wall elements). Also it will not apply when the wall element supports an overburden load (e.g. cantilever wall supporting an ineffective roof diaphragm or parapet with an eccentric corbel).

Who is this person?

3 Computer Modelling of Australian Shaking Table Test Specimen

3.1 Introduction

Blaikie (2000) used inelastic dynamic analysis methods to model the seismic behaviour of a face-loaded URM single-storey wall element similar to that shown in Figure 4. The wall was modelled as uncracked except for cracks at the foundation level and at mid-storey height. These cracks were free to open and close as the wall deflected under lateral loads. This computer model was used to predict the behaviour of a nominally 3.0m high, 230mm thick laboratory test specimen. The middle of the test specimen was displaced horizontally to open the wall cracks and then released so that the walls free damped response could be recorded. Agreement between the predicted and observed horizontal displacements of the mid-height of the wall specimen was excellent.

Blaikie and Spurr (1992) used a similar computer model to predict the behaviour of a laboratory test specimen that was part of the pioneering USA investigation into the behaviour of URM (ABK, 1981). However, the input motion used in the test was uncertain and the detailed response predicted by the computer model was sensitive to both the damping assumed and small changes in the earthquake motion. The agreement obtained between the predicted and observed horizontal displacements of the mid-height of the wall specimen was good given that the earthquake motion used in the test could not be accurately modelled.

Australia researchers (Doherty, 2000) have tested a number of laboratory test specimens under face loading conditions and the time-history input and output data recorded during the shaking-table tests of the face-loaded wall test specimens was made available to the author. In this research project, computer modelling is used to develop an assessment methodology for face-loaded URM. These data were used to verify the computer models used in this research project.

The Australian tests also included a number of free damped vibration tests of face-loaded wall specimens. These tests were reanalysed to provide data on the appropriate damping to be used in the computer modelling.

3.2 Computer Modelling of Test Specimen

Figure 5 shows diagrammatically the computer model used to analysis the Australian Test Specimen. The test specimen (specimen13) was 1530 high and nominally 110mm thick with the top support at the 1485mm level. The model was analysed using the inelastic dynamic analysis program Ram Xlinea which incorporates DRAIN-2DX Version 1.1. The program uses time step-by-step numeric integration to perform the inelastic dynamic analysis.

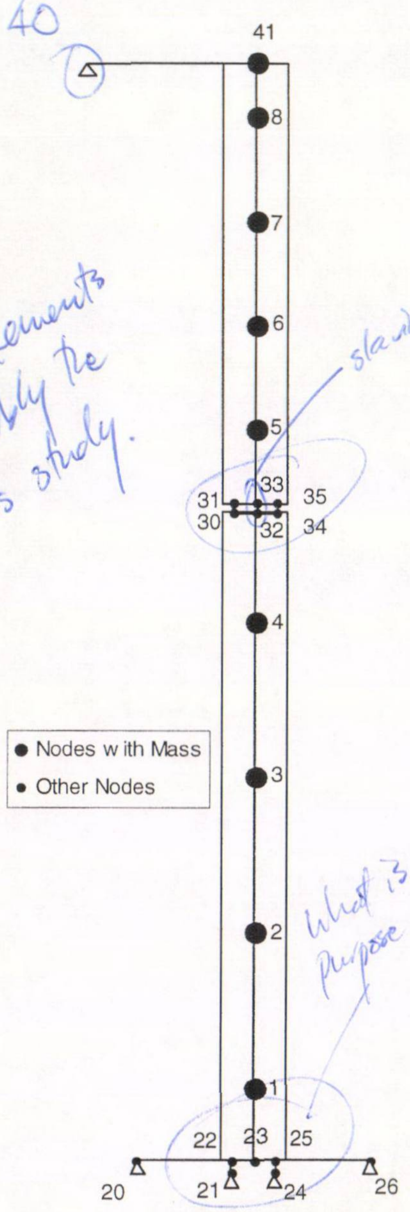


Figure 5: Computer Model used to Simulate Australian Test Specimen

40

Link elements are probably the key to this study.

What is the purpose of these

The DRAIN-2DX input file is shown in Appendix A. The input file gives details of the wall geometry and properties of the elements used to model the test specimen. The numbers given in the diagram correspond to the node numbers in the input file.

The model allows the wall to deform as indicated in Figure 4. Link members, that can only carry compressive forces, accommodate opening of the cracks at the mid-height and base of the wall. These are shown at an exaggerated vertical scale in the diagram for clarity and were actually modelled as only 2 mm long.

1mm long

The model is similar to that used by Blaikie (2000), and includes rotational mass inertia elements at each of the mass nodes. The horizontal support conditions at the base of the wall were modelled using link members that were only able to carry compressive forces.

The horizontal support member at the top of the wall was used to model the stiff rubber spacers that were used either side of the brick wall to separate the wall and test frame.

The position of the mid-height crack was not recorded but Doherty (in private correspondence) advised that the crack formed either one or 2 bricks depths above the mid-height position. The crack was, therefore, modelled 1½ bricks above the mid-height position. The centreline of the test specimen was assumed to lie on a vertical line and the wall was assumed to be symmetric about the centreline.

3.3 Comparison of Computer Model and Test Specimen

But your analytical expression only appears to address shaking of ground surface

Figure 6(a) shows the mid-height wall displacement recorded for the Australian test specimen (specimen 13) when the shake table was subjected to 80% of Pacoima Earthquake record (downstream component, Northridge EQ, 1994). The response predicted by the computer model, when the shake table displacement time histories recorded during the test were used as input motion for the computer model, is also shown for comparison.

The mid-height displacements of the test specimen were recorded relative to the frame that supported the wall specimen on the shake table. The computer model displacements, used for the comparison, were evaluated as the mid-height displacements of the wall relative to the top and bottom supports. The displacement time histories, used as input motions for the model, were the motions recorded at the actuator and at the top of the test frame.

Figure 6(b) shows a similar comparison between the recorded mid-height wall displacements and those predicted by the computer model when accelerations recorded during the test were used as input motions for the model. In this case the acceleration recorded on the shake table (i.e. at base of wall) and at the top of the wall support frame (i.e. at top of wall) were averaged and used as input motions at the top and bottom supports of the model.

When the displacements or accelerations at the top and bottom of the wall test specimen are plotted and compared, very little difference is evident. Therefore, the difference in the predicted responses shown in Figure 6(a) and Figure 6(b) indicates that the detailed response of the wall is very sensitive to small changes in input motion. Given this sensitivity, the agreement between the wall response recorded in the test and that predicted by the model is excellent.

The intent of this sentence is a little difficult to comprehend

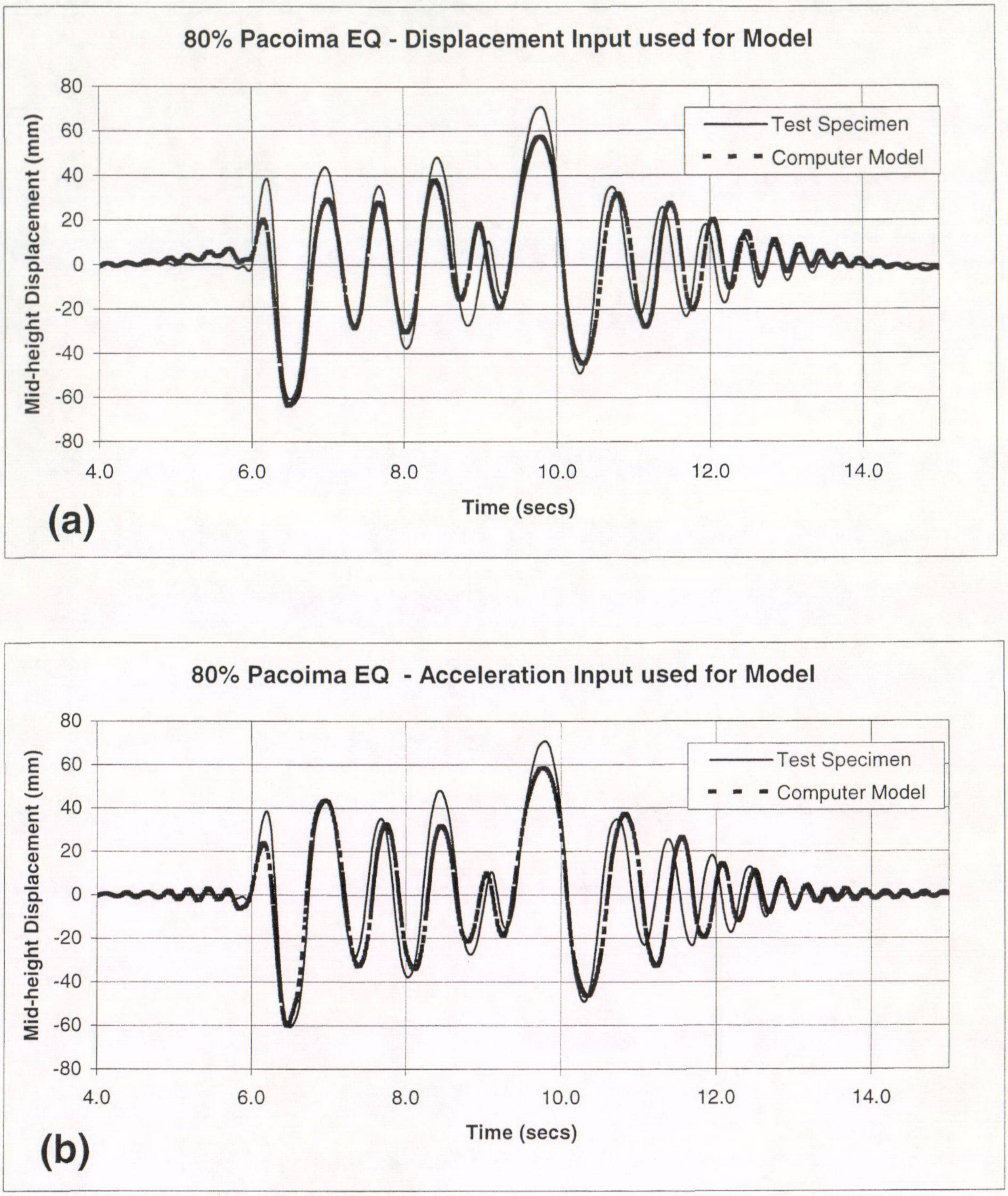


Figure 6: Mid-height Wall Displacement of Test Specimen Subjected to 80% of Pacoima EQ Record and Comparison with Response Predicted by Computer Model, (a) When Measured Displacement Time Histories and (b) When Measured Acceleration Time Histories used as input for Computer Model

3.4 Parameters Used in Computer Model

The agreement between the test results and those predicted by the computer model shown in Figure 6 was only achieved after an iterative process. This process involved adjusting the flexural stiffness, effective thickness, t, and damping used to model the wall until an acceptable agreement was achieved. The detailed response, and hence the agreement obtained, was quite sensitive to small changes in any of these parameters because they all affect the frequency and amplitude of the walls response.

Figure 7 shows the results obtained from a push over analysis obtained using the computer model that gave the best agreement with the Australian face-loaded wall test specimen data. The point load used for the push-over analysis was applied at the mid-storey crack (just above the wall mid-height). Horizontal wall displacements at the same location are shown.

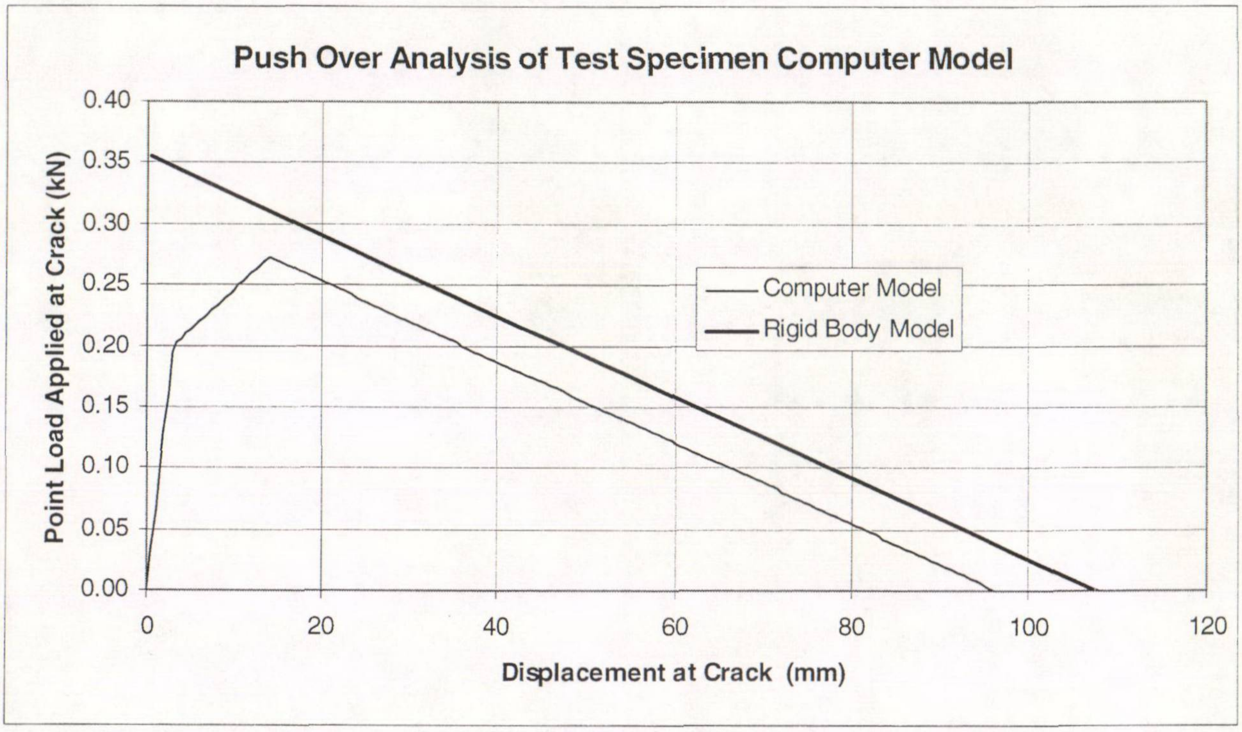


Figure 7: Push Over Analysis Results Obtained From the Computer Model used to Obtain the Best Agreement with the Australian Face-Loaded Wall Test Specimen

The results of a push-over analysis that would be obtained using the computer model if the wall behaved as 2 rigid blocks and if the wall's effective thickness t was equal to the nominal thickness of 110mm is also shown in Figure 7 (i.e. Rigid Body Model).

Prior to shake table testing the wall specimen to 80% of the Pacoima EQ record, the same test specimen had been subjected to a push-over test to a 90mm mid-height displacement, a further series of push-over tests with mid-height displacements between 10 and 90mm and a shake table test using 66% of the Pacoima record producing several mid-storey peak displacements in the 60 to 70 mm range (as advised by K Doherty in private correspondence). So what.

what does this replicate?

Previous research by the author (Blaikie, 2000) using a 3.0m high x 230mm thick wall specimen indicated that reasonable agreement between the test specimen and push over test results would be obtained by modelling the wall with an effective elastic modulus of 1.0GPa (with half this wall flexibility provided in the link members). This specimen had only been subjected to an initial low displacement test cycle to open the cracks in the wall specimen. A large displacement push-over test using the specimen indicated that the effective thickness of the wall, t, would be close to the nominal wall thickness t_{nom} given by the relationship:

Is it correct that this expression is based on a single test unit?

$$\boxed{t = t_{nom}(0.975 - 0.025\frac{O}{W})} \quad \textbf{Eqn 18}$$

The computer model used to predict the behaviour of the Australian test specimen had a wall stiffness based on an effective flexural elastic modulus of only 0.02GPa (with half this wall flexibility provided in the link members). Also, the effective wall thickness, t, used in the model was only 98mm compared with a value of 107mm given by Eqn 18.

Obviously.

Compression tests on wall samples similar to the materials used in the wall specimen indicated that the elastic modulus of the wall masonry (in compression) was between 3.3 and 16GPa (average 9.4GPa). The effective wall flexural stiffness indicated by the model was only approximately 0.2% of the value indicated by the compression tests. ***Therefore, it is evident that most of the wall flexibility was the result of rotations on the opening mortar joints.***

It would appear that the initial tests that were applied to the Australian test specimen prior to the shake table testing resulted in significant rounding and deterioration of the mortar joints at the opening cracks in the wall. This conclusion is supported by the results obtained from the initial pushover tests applied to the Australian test specimen. These tests (Doherty, 2000) indicated that, even at this early stage of loading, the effective flexural elastic modulus was only approximately 0.05GPa (Note: push-over tests results plotted in Doherty's thesis are for mid-height loads and displacements not for crack height loads and displacements and are, therefore, not directly comparable with those given in Figure 7).

In this project wall elements are generally modelled with an effective flexural elastic modulus of 1.0GPa (with half this wall flexibility provided in the link members) and with an effective wall thickness given by Eqn 18. These parameters are considered to be more appropriate for wall elements that have not been subjected to significant wall crack opening. ***However, the Australian tests indicate that significant deterioration in the effective flexural stiffness and effective thickness of a wall element may take place during the walls response to an earthquake.***

The sensitivity of the computer model results to the position assumed for the mid-storey crack in the model was also investigated. It was found that with the mid-storey crack modelled at the mid-height of the wall, instead of 1½ bricks above mid-height, only a small adjustment in the effective wall thickness (t increased from 98 to100mm) and reduction in wall damping (mass damping coefficient reduced from 0.48 to 0.42) was required to give almost as good agreement with the test results as that shown in Figure 6. It was concluded that the exact location of the mid-storey crack was not an important parameter.

4 Modelling Energy Loss and Damping

4.1 Previous Research

For a face loaded URM wall the main source of damping will be the energy loss that occurs on impact as the opening joint cracks close. Drain2dx attempts to ensure that no energy loss occurs during each time step during the analysis except for the work done by viscous damping. Therefore the energy losses in a Drain 2dx model occur throughout the response and not just at impact.

Some of the problems associated with modelling the damping of an URM wall have been discussed as part of previous research (Blaikie, Spurr 1992). These included:

- Stiffness damping of the link elements must be set to zero or the opening joints will lock up as stiffness damping in Drain2dx is based on the high initial stiffness of these elements not their secant stiffness.

- Additional damping can occur in the model because geometric stiffness in Drain2dx is based on the static loads acting on the wall and this stiffness is not updated to allow for the effects of vertical impact forces. Varying the proportion of the vertical inertia modelled can control the amount of this "numeric damping" as the amount of vertical inertia determines the magnitude of the impact forces.

- The logarithmic decrement method of defining the amount of damping in the system is not applicable to inelastic systems especially where strength declines rather than increases with displacement.

- Mass damping is dependent on nodal mass and velocity. It is most effective for larger amplitude cycles when the nodal velocity is higher.

- Inclusion of stiffness damping has more effect on the damping of the model response than would be expected from considering an equivalent elastic system. This seems to be related to the high frequency vibrations generated by impact in the model. The energy associated with these high frequency vibrations is mainly stored in the wall elements and is affected by the flexural stiffness used to model the wall elements. The magnitude of the impact forces is primarily determined by the axial stiffness of the link elements and by the proportion of vertical inertia included in the model. If the effect of varying these parameters on the free vibration response of the wall is studied, the amount of damping in the model can be adjusted to the required amount.

If the middle of a cracked single-storey wall is displaced horizontally as shown in Figure 4 and released, a free damped response similar to that shown in Figure 3 will occur. During the response the cracks will close each time the mid-height wall displacement returns to zero. Closing of the cracks will cause impact forces to be generated in the wall components.

$$\Delta E = \frac{1}{2} m (v_1^2 - v_2^2)$$

$$e = \frac{v_2}{v_1}$$

$$v_2 = e v_1$$

$$= \frac{1}{2} m \, e^2 v_1^2 - v_1^2$$

$$= \frac{1}{2} m v_1^2 (e^2 - 1)$$

By definition the coefficient of restitution, e, is the ratio of the walls angular velocity after impact compared with that before impact. If the elastic displacements of the wall are ignored (i.e. rigid body rotations are assumed) e will also equal the ratio of mid-height wall velocity after impact compared with that before impact.

The velocity before and after impact can be calculated using Eqn 16. In some more recent research (Blaikie, 2000) the resulting values for e, computed for each impact, was shown to be reasonably constant. As the energy loss on each impact = $1-e^2$, this indicates that the energy lost on each impact is a constant proportion of the energy stored in the wall before impact.

½ in proportional to

4.2 Australian Free Vibration Test Results on Single-Storey Walls

For rocking bodies the coefficient of restitution is not only dependent on the properties of the impacting surfaces. It can be easily demonstrated by simple desktop tests that the motion of a squat rocking body damps out more quickly than that of a similar slender body. This characteristic of rocking bodies is a result of only the vertical component of a wall momentum changing at impact. In squat walls this component makes up a larger proportion of the total momentum (and energy) stored in the wall. Energy losses, and therefore damping, are expected to be greater for squat walls.

To model damping for the computer modelling, a method of adjusting the expected coefficient of restitution to allow for the effects of wall slenderness is, therefore, required.

Testing of thin unreinforced masonry single-storey walls with a range of slenderness ratios has recently been carried out in Australia (Doherty, 2000). Results of some of the free vibration tests carried out as part of this research were supplied to the author in private correspondence.

The response of a 1500mm high by 110mm nominal thickness single-storey test specimen (no. 13) subjected to 3 shake table displacement pulses is shown in Figure 8. After the first impact (i.e. crossing of the zero displacement line) the wall has a free damped response. Eqn 16 was used to calculate the mid-height wall velocity before and after each impact of the free damped response and hence the coefficient of restitution, e, applicable each time the wall cracks closed. For these calculations an effective wall thickness, t, of 107mm was assumed. The results of the calculations are shown in Table 1.

Two sets of results for the coefficient of restitution calculation are shown in the table, one with and the other without a baseline correction. The results without a baseline correction indicate that damping varied for the 2 displacement directions about the wall mid-thickness centreline. If a very small shift is made in the reference point from which the wall displacements were measured (i.e. a baseline correction is applied), the results are more uniform as indicated in the table. This may indicate that a small amount of impact damage to the mortar in the joints is effectively re-zeroing the initial geometry of the test specimen after each impact.

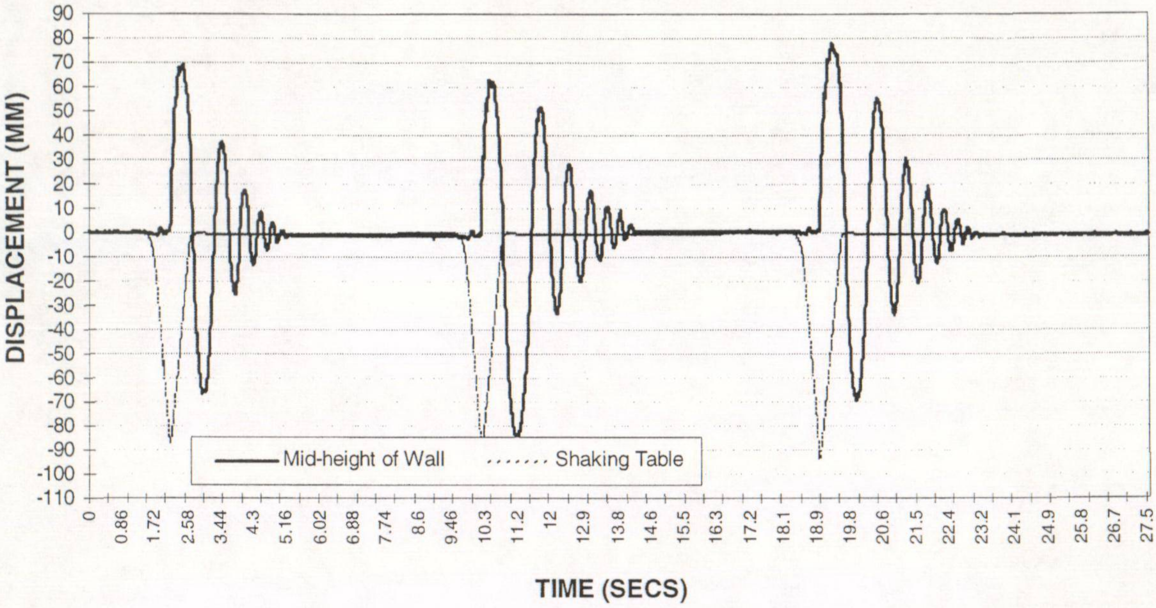


Figure 8: Response of 1500x110mm Single-Storey Test Specimen Subjected to 3 Shake Table Displacement Pulses

Table 1: Calculation of Coefficient of Restitution for Free Vibration Response of 1500x110mm Single Storey Wall Test Specimen

1500 high x 110 mm nominal thickness wall Test Specimen								
first pulse			2nd pulse			3rd pulse		
Peak Displ* (mm)	coefficient Restitution with baseline correction: 0.0 mm	-0.5 mm	Peak Displ* (mm)	coefficient Restitution with baseline correction: 0.0 mm	1.2 mm	Peak Displ* (mm)	coefficient Restitution with baseline correction: 0.0 mm	1.0 mm
-66.41	0.817	0.823	-85.38	0.875	0.865	-70.02	0.935	0.926
37.64	0.857	0.846	50.20	0.848	0.868	54.00	0.837	0.852
-25.12	0.843	0.862	-34.48	0.934	0.904	-35.03	0.954	0.930
17.97	0.882	0.855	27.08	0.858	0.897	29.38	0.852	0.882
-12.81	0.820	0.858	-21.12	0.942	0.889	-21.97	0.970	0.927
9.26	0.907	0.853	16.27	0.819	0.887	18.57	0.815	0.865
-6.66	0.777	0.849	-12.56	0.966	0.870	-13.56	0.883	0.809
4.76	0.952	0.843	9.36	0.769	0.888	8.66	0.875	0.980
Average**=	0.843	0.849		0.892	0.883		0.892	0.884
* Peak before impact - after non zero baseline correction								
** average of impacts 1 to 7								
Overall average for impacts 1 to 7 =			0.87	after base correction				

Similar response time histories and coefficients of restitution calculations are shown in Figure 9 and Table 2 for a 1500mm high specimen with a nominal thickness of 50mm.

The same test specimen (no. 10) was also subjected to a series of release tests, which produced similar results as shown in Figure 10 and Table 3. For each release test the centre of wall was displaced to a value close to the instability displacement (Y_{max} = t which was assumed in the model to equal 48.8mm). As can be seen in Figure 10 the first peak in each of the release tests was rather poorly defined and may have exceeded the walls actual instability displacement. Therefore, the peak value used for the calculation of the coefficient of restitution given in Table 3, was the average displacement value for first 0.3 seconds after the maximum value had been reached and the wall displacement had stabilised to approximately the targeted average value. As the first peak displacement was ill defined, average results for the coefficient of restitution calculations are presented in Table 3 either including or excluding the first impact in each release test. When the first impact is ignored the average coefficient of restitution (0.91) is the same as that given in Table 2 for the pulse tests of the same wall. When the first impact value is included a lower average coefficient of restitution (0.90) is obtained. A smaller coefficient of restitution than the average for the first impact is supported by the data in the other tests presented in Table 1 and Table 2 and this pattern was targeted when modelling the damping in the computer models.

So what?

4.3 Australian Free Vibration Test Results on Parapet Walls

One of the objectives of this research project is to extend the methodology developed for seismic assessment of face-loaded masonry walls supported by diaphragms, to parapets and cantilever walls. Some free vibration tests on a 1000mm high by 110mm thick parapet wall have also been carried out in Australia. However, peak displacements for each of the half cycles of the response ~~w~~here only reported (Lam, et. al., 1995) in a normalised graphical form. The maximum wall displacements used to normalise the peak displacements were obtained from the researchers (private email from N Lam) and the peak half-cycle displacement scaled from the graphs in the report. These peak half-cycle displacements were used to calculate the coefficient of restitution for each impact in the parapet response. The results are shown in Table 4 for three of the free vibration tests carried out on the test specimen.

4.4 Damping Adopted for Computer Modelling

Figure 11 shows the variation in the coefficient of restitution expected when impacts occur as the cracks close in a rocking wall element. Test specimen values shown are average results extracted from the proceeding sections of this report and, in the case of the 3000 x 230 mm specimen, from previous research (Blaikie, 2000).

For the parapet specimen, the slenderness has been expressed in terms of 4h/t. With this adjustment, the average parapet value is consistent with the general trend of increasing coefficient of restitution (i.e. reducing energy loss) with increasing slenderness as exhibited by the single storey specimen values.

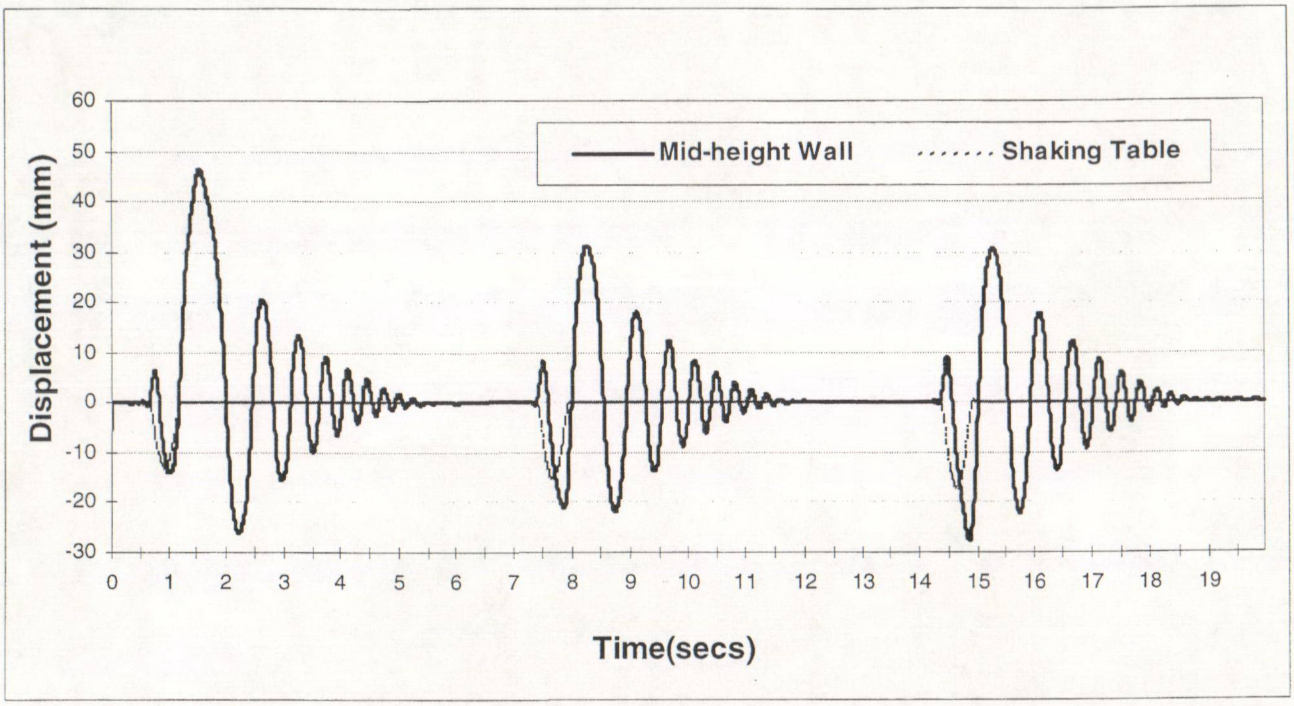


Figure 9: Response of 1500 x 50mm Single-Storey test Specimen Subjected to 3 Shake Table Displacement Pulses

Table 2: Calculation of Coefficient of Restitution for Free Vibration Response of 1500x110mm Single Storey Wall Test Specimen – Pulse Tests

1500 high x 50 mm Nominal Thickness wall Test Specimen								
first pulse			2nd pulse			3rd pulse		
Peak Displ* (mm)	coefficient Restitution with baseline correction: 0.0 mm	-0.5 mm	Peak Displ* (mm)	coefficient Restitution with baseline correction: 0.0 mm	-0.6 mm	Peak Displ* (mm)	coefficient Restitution with baseline correction: 0.0 mm	-0.4 mm
45.74	0.888	0.894	30.52	0.892	0.905	30.07	0.903	0.912
-26.73	0.917	0.903	-22.26	0.935	0.914	-22.47	0.924	0.910
19.88	0.902	0.922	17.49	0.889	0.917	17.45	0.892	0.911
-16.13	0.943	0.917	-14.11	0.963	0.926	-13.86	0.964	0.939
13.07	0.874	0.907	11.77	0.868	0.913	11.95	0.873	0.903
-10.43	0.964	0.920	-9.56	0.973	0.914	-9.47	0.992	0.953
8.65	0.865	0.917	7.83	0.853	0.922	8.50	0.823	0.867
-7.15	1.005	0.938	-6.57	1.014	0.924	-6.23	1.024	0.962
Average**=	0.908	0.911		0.910	0.916		0.910	0.914
* Peak before impact - after non zero baseline correction								
** average of impacts 1 to 7								
Overall average for impacts 1 to 7 =			0.91	after base correction				

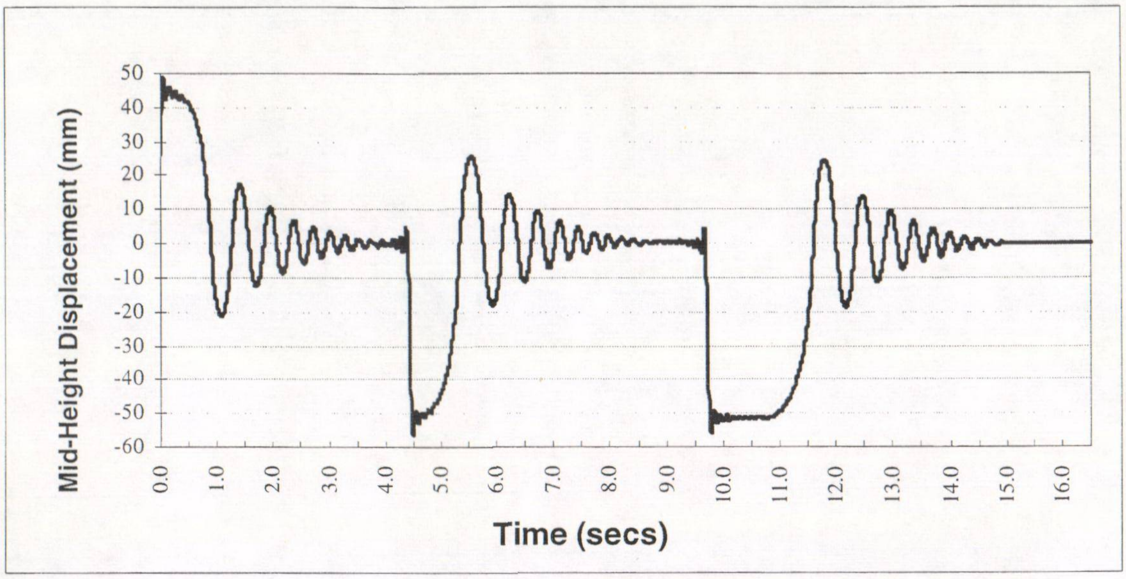


Figure 10: Response of 1500x110mm Single-Storey test Specimen Subjected to 3 Mid-height Displacements and then Release

Table 3: Calculation of Coefficient of Restitution for Free Vibration Response of 1500 x 50mm Single Storey Wall Test Specimen – Release Tests

1500 high x 50 mm nominal thickness wall Test Specimen								
first pulse			2nd pulse			3rd pulse		
Peak Displ* (mm)	coefficient Restitution with baseline correction: 0.0	0.0	Peak Displ* (mm)	coefficient Restitution with baseline correction: 0.0	0.4	Peak Displ* (mm)	coefficient Restitution with baseline correction: 0.0	0.0
44.02	0.830	0.830	-51.28	0.878	0.873	-51.21	0.867	0.867
-21.25	0.920	0.920	25.33	0.891	0.903	24.36	0.910	0.910
17.04	0.883	0.883	-18.33	0.904	0.886	-18.72	0.886	0.886
-12.59	0.925	0.925	14.25	0.898	0.922	13.86	0.916	0.916
10.51	0.915	0.915	-11.06	0.933	0.901	-11.27	0.921	0.921
-8.61	0.903	0.903	9.45	0.876	0.916	9.35	0.908	0.908
6.89	0.932	0.932	-7.06	0.977	0.926	-7.56	0.928	0.928
-5.92	0.910	0.910	6.72	0.858	0.917	6.43	0.908	0.908
Average**=	0.901	0.901		0.908	0.904		0.905	0.905
impacts 2-8	0.913	0.913		0.905	0.910		0.911	0.911

* Peak before impact - after non zero baseline correction

** average of impacts 1 to 7

Overall average for impacts 1 to 7 = 0.90 after base correction

Overall average for impacts 2 to 8 = 0.91 after base correction

Table 4: Calculation of Coefficient of Restitution for Free Vibration Response of 1000 x 110mm Parapet Test Specimen

1000 high x 110 mm Nominal Thickness Parapet Test Specimen								
first test			2nd test			3rd test		
Peak Displ* (mm)	coefficent Restitution with baseline correction: 0 mm	-3 mm	Peak Displ* (mm)	coefficent Restitution with baseline correction: 0 mm	+1 mm	Peak Displ* (mm)	coefficent Restitution with baseline correction: 0 mm	-3.5 mm
-52.00	0.826	0.872	-49.00	0.916	0.900	48.50	0.891	0.944
36.00	1.048	0.978	37.00	0.936	0.958	41.50	1.051	0.984
-34.10	0.859	0.932	-33.26	0.972	0.946	39.76	0.863	0.933
28.74	1.037	0.943	29.10	0.917	0.946	33.41	1.066	0.975
-25.04	0.896	0.995	-25.58	1.001	0.966	31.44	0.860	0.951
24.78	1.017	0.902	23.62	0.904	0.941	27.92	1.085	0.971
-19.65	0.883	1.012	-20.58	1.004	0.959	26.04	0.844	0.956
20.16	0.950	0.799	18.76	0.892	0.940	23.49	1.095	0.953
Average=**	0.938	0.948		0.950	0.945		0.951	0.959

* Peak before impact - after base correction

** average of 1 to 7 impacts

Overall average for impacts 1 to 7 = **0.95** after base correction

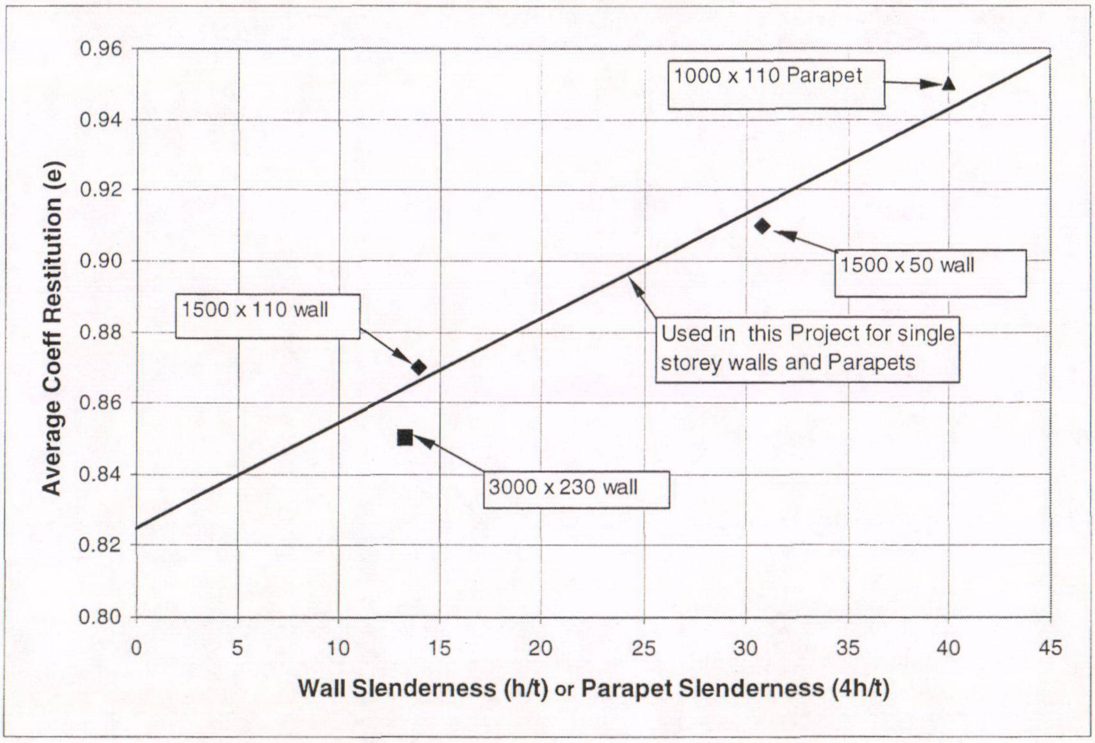


Figure 11: Variation of Coefficient of Restitution for Impacts When Cracks Close in a Wall Element. Test Specimen Values are Average Results. Values of Coefficient of Restitution used to Model Damping in Single-storey and Parapet Computer Models in this Project also shown.

The treatment of parapet (or cantilever) walls as a single-storey wall of 4 times the height is consistent with the fundamental equations describing the behaviour of the 2 types of wall element as outlined in section 2.3.

The linear variation of coefficient of restitution with wall element slenderness used to model damping in the single storey and parapet computer models used in this project is also shown in Figure 11.

An iterative procedure was used to adjust the damping in the computer model. The average e value was calculated for the free vibration response of the wall, as predicted by the computer model. To obtain a free vibration response for the wall computer model, the model was subjected to a short acceleration pulse that was sufficient to displaced the mid-height (or top of parapet) to approximately 60% of its instability displacement. The wall response was then allowed to decay freely except for the influence of damping.

Adjustments were made to the modelled mass and stiffness damping coefficients, the vertical inertia, and in some cases the wall flexural stiffness and the link axial stiffness until the target e value was approached for the main impacts associated with crack closing. Generally only impacts preceeded by mid-height displacements of the wall model that were greater than 5% of the wall thickness were considered. Damping of the wall expected for the elastic response prior to crack opening was also checked to ensure it was not excessive (i.e. less than 10%).

Parameters, other than slenderness, may effect the coefficient of restitution. When the 3000 x 230mm specimen refered to in Figure 11 was retested with weak mortar in the opening joints, the calculated values of e for the first 2 impacts of the computer modeled response were 0.81 and 0.83. These are only a modest reduction on the corresponding values of 0.84 and 0.85 obtained when stronger mortar was used.

It can be seen from Figure 11 that the 1500 x 110mm Australian wall test specimen was only slightly more slender than the 3000 x 110mm specimen and had a moderately lower average e value. As the Australian specimen was constructed using softer mortar (1:1: 6 mix with effectively ¼ the cement replaced with lime) a higher level of damping (i.e. lower e value) would have been expected. This indicates that thicker walls of the same slenderness may have marginally higher damping.

It is not clear to the author what effect a higher overburden load would have on damping. It may increase the proportion of vertical momentum in the system and hence increase the amount of damping. The recent Australian. tests included walls with overburden (applied using prestressing rods and springs). The method used to report the test results (Doherty, 2000) makes it difficult to determine the likely effect of overburden on the coefficient of restitution. However the results sugest that any increased damping is likely to be modest.

The increased damping expected from the presence of weaker mortar or of overburden will be offset by the detremental effect that these parameters will have on the mortar in the opening and closing joints. The smaller effective wall thickness, t resulting from the increased damage to the mortar joints is expected to at least cancel out the beneficial effects of the higher damping.

5 Prediction of Face-Loaded Wall Stability Using Response Spectra

5.1 Development of Response Spectra Used for Computer Modelling

A methodology to assess the seismic stability of face-loaded URM single storey wall elements was developed in previous research (Blaikie,2000). This methodology uses acceleration and displacement spectra. The spectra for an actual earthquake record or spectra given by design codes can be used. Where only an acceleration spectra is available a pseudo displacement response spectrum can easily be derived from an acceleration design spectra using the relationship:

$$Y = (T / 2\pi)^2 A \qquad \textbf{Eqn 19}$$

where A is the spectral acceleration (in m/sec^2) for a SDOF elastic oscillator with period T and Y is the pseudo spectral displacement.

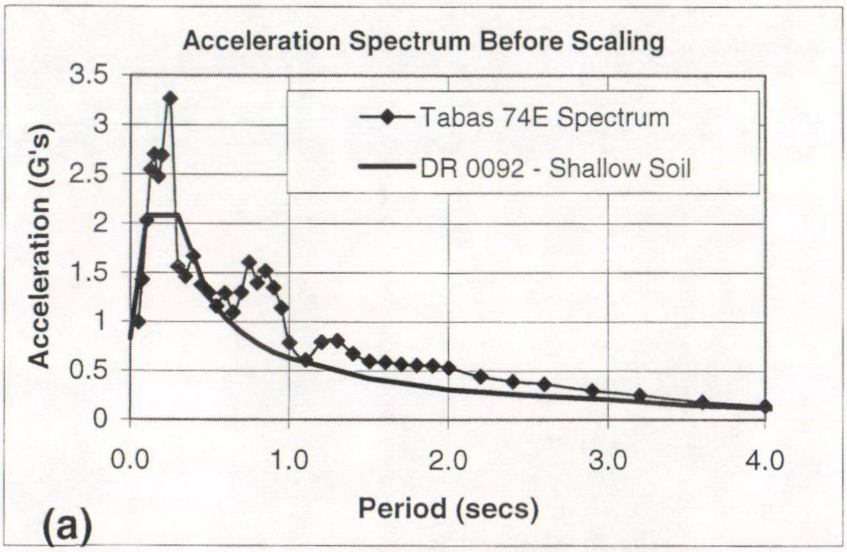
Figure 12 shows the 5% damped acceleration response spectra for the E74 component of the Tabas earthquake record before and after the record was scaled using the Wave-1 computer program (Opus, 1989).

The Institute of Geological and Nuclear Sciences (GNS) kindly supplied the unscaled record. This record had been filtered to remove the near-fault pulse and was selected by GNS because, in this form, it closely matched the shallow soil spectrum (Site Subsoil Class C) proposed in the draft New Zealand Loadings Standard (SANZ, DR 0092, 2000). The code spectrum is also shown in Figure 12 for comparison. The first 5 seconds of the record as supplied by GNS was ignored (as it was of very low intensity) and only the next 20 seconds of the record was used to produce the spectra.

The Wave-1 computer program scales the earthquake record so that their acceleration spectrum more closely matches a target spectrum. It can be seen in Figure 12 that after scaling, the response spectrum for the Tabas earthquake record more closely matches the target code spectrum. The scaling of the earthquake record using the Wave-1 program alters the frequency content of the record but does not change the records overall pattern as indicated by Figure 13.

Figure 14 shows the displacement spectra for the Tabas earthquake motion before and after scaling using the Wave-1 program. The pseudo response spectrum for the draft code spectra derived using Eqn 19 is also shown for comparison.

The scaled Tabas earthquake record is referred to as the DR 0092 earthquake record in this report and was one of the earthquake input motions used for the computer modelling. A more recent update of the draft code acceleration and displacement spectra were used to develop the design charts in Appendix B.



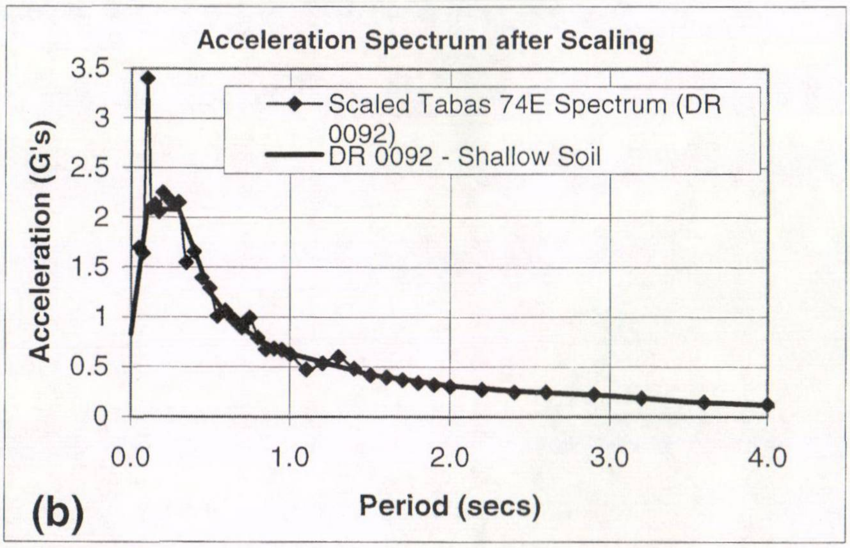
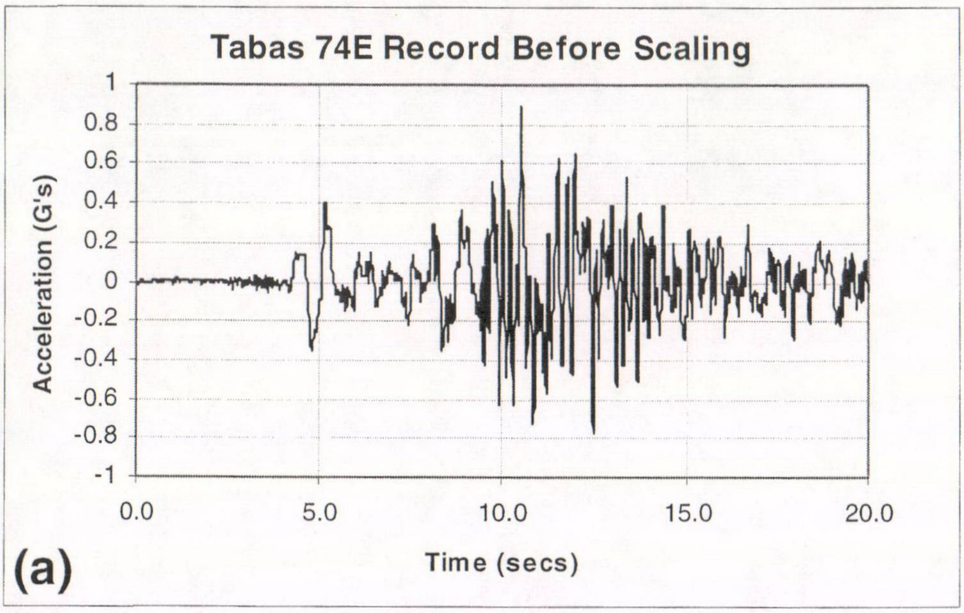


Figure 12: 5% Damped Acceleration Response Spectra For The E74 Component of the Tabas Earthquake Record (a) Before and (b) After The Record Was Scaled Using The Wave-1 Computer Program. The Draft NZ Loading Standard Spectrum (DR 0092) is also shown.



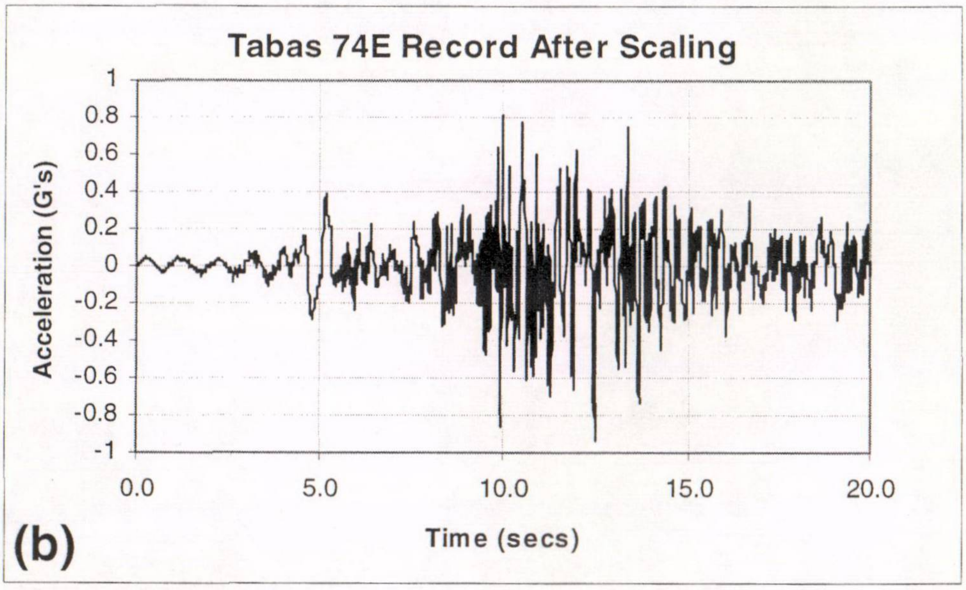
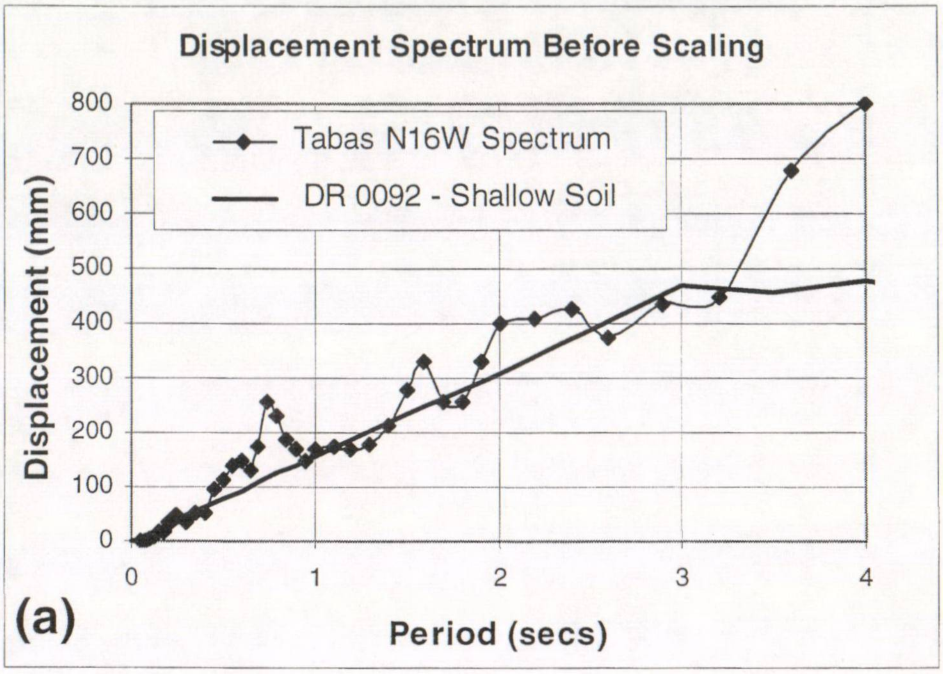


Figure 13: Ground Motion Record For The E74 Component Of The Tabas Earthquake Record (a) Before and (b) After The Record was scaled using the Wave-1 Computer Program.



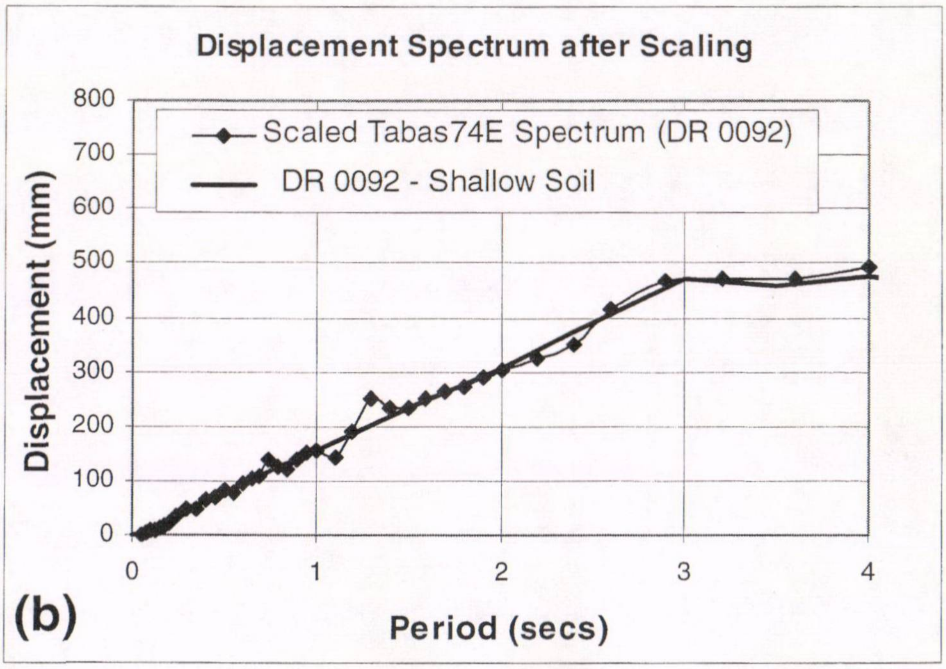


Figure 14: 5% Damped Displacement Response Spectra For The E74 Component of the Tabas Earthquake Record (a) Before and (b) After the Record was scaled using the Wave-1 Computer Program. The Pseudo Displacement Spectrum Derived form the Draft NZ Loading Standard Acceleration Spectrum is also shown.

5.2 Methodology used to Predict Face-loaded Wall Element Stability

If a URM wall element is subjected to gradually increasing earthquake intensity eventually the wall element will collapse.

Blaikie (2000) has shown that the earthquake intensity scaling factor, $I_{collapse}$, that must be applied to a given earthquake motion to cause a wall element to collapse can be predicted using the following relationships:

$$I_{collapse} = I_{sp} \qquad \text{when } I_{sp} \geq 2.5I_{cr}$$

or **Eqn 20**

$$I_{collapse} = \frac{I_{sp} + 2.5I_{cr}}{2} \qquad \text{when } I_{sp} < 2.5I_{cr}$$

where: I_{sp} is 1.2 times the earthquake scaling factor that must be applied to the earthquake motion so that the mid-storey wall displacement, predicted using a displacement spectrum procedure, is 60% of the collapse displacement, Y_{max} and,

I_{cr} is the earthquake scaling factor, predicted using an acceleration spectrum procedure, that must be applied to the earthquake motion so that the joint cracks in the wall element will just begin to open.

The methodology was developed for single-storey wall elements supported top and bottom by floor diaphragms or the ground. It is proposed that the same methodology be extended to include parapets or free standing cantilever wall elements.

The following steps are required to calculate the earthquake scaling factor, I_{sp}, that the procedure predicts must be applied to an earthquake motion to cause the wall element to collapse:

1. Evaluate the period, T, of the rocking motion of the wall element when the peak displacement is 60% of the instability displacement, Y_{max} (use Eqn 17 for single-storey wall elements and Eqn 10 for parapet type elements).

2. Use a displacement response spectrum for the earthquake motion (e.g. Figure 14(b)) to evaluate the maximum displacement expected for a SDOF structure with the period, T calculated in Step 1. The maximum displacement expected at the mid-height of a single-storey wall element (or top of a parapet) for the earthquake motion is 1.5 times this SDOF displacement.

   The 1.5 multiplier assumes a normalised modal participation factor of 1.5, which scales up the SDOF displacement expected to that expected for the multi-degree-of-freedom wall structure. The 1.5 modal participation factor is the theoretical value that applies if the cracked wall components behave as rigid blocks and respond elastically. This value is expected to be conservative for wall elements responding inelasticaly.

When evaluating the expected displacement from the displacement response spectrum, the spectral displacement is assumed to increase with increasing period, T. Effectively this means that any local "dips" in the displacement response spectra are ignored. This seems reasonable, as the wall period increases with increasing displacement and will, therefore, pass through the local early peak displacement period range prior to reaching 60% of the collapse displacement.

3. The mid-height wall displacement (or top displacement of a parapet) predicted in the previous step is proportional to any scaling factor applied to the earthquake motion used to derive the displacement spectra. Therefore the scaling factor that must be applied to the earthquake motion to obtain a predicted maximum wall displacement of 0.6 Y_{max} can be calculated.

4. The predicted earthquake scaling factor, I_{sp} that will cause the wall element to collapse is 1.2 times the earthquake scaling factor corresponding 60% of the collapse displacement, Y_{max} calculated in the proceeding step.

The following steps are required to calculate I_{cr}, the earthquake scaling factor that must be applied to an earthquake motion so that the joint cracks in the wall element just begin to open:

1. Evaluate the initial elastic period, T_o, of the wall element. For single storey wall elements, supported top and bottom, the wall is assumed to act as a propped cantilever. Parapet or freestanding cantilever walls are assumed to respond as fixed base cantilevers.
2. Use an acceleration response spectrum for the earthquake motion (e.g. Figure 12(b)) to evaluate the maximum response expected for a SDOF structure with a period, T_o.
3. Calculate the seismic coefficient, C_d, corresponding to the UDL lateral load that would be just sufficient to open the cracks in the wall (use Eqn 6 for single-storey wall elements and Eqn 15 for parapet type elements).
4. By comparing the response values evaluated in the previous 2 steps the earthquake scaling factor, Icr, which must be applied to the EQ motion so that it would be just sufficient to open the wall element joints can be evaluated.

The initial elastic period, T_o, of the wall element is evaluated in step 1 assuming an effective elastic modulus of 1.0GPa for the masonry. In practice there is likely to be considerable scatter in the effective values of the elastic modulus of the masonry. When preparing the design charts as part of previous research (Blaikie, 2000) an increase in the modulus value up to 4.0GPa was allowed for. In light of the Australian test data (see Sect 3.4) this allowance is no longer considered appropriate, as the effective modulus is likely to be less than 1.0GPa once significant crack opening has occurred during the wall response. When developing the new design charts given in Appendix B the possibility that the effective elastic modulus of the wall elements could be less than 1.0GPa was allowed for. This allowance, for possible variation in the effective elastic modulus, has been applied conservatively. When the initial elastic period, T_o, indicated that the wall response was on the rising branch of the acceleration spectrum, the rising branch was ignored and the peak response value was assumed. However, the rising branch of the acceleration spectrum was not ignored when comparing the collapse earthquake intensities predicted by the computer modelling and the proposed assessment methodology in the body of this report.

When a single-storey wall element, supported top and bottom, has sufficient top fixity it would be more appropriate to evaluate the initial elastic period assuming the wall element has top and bottom fixed end supports instead of as a propped cantilever. This refinement was not applied when developing the design charts given as part of the design office assessment procedure given in Appendix B. As both C_d and Y_{max} are proportional to the fixity factor, F_{fixity}, the collapse intensity predicted by Eqn 19 is also proportional to F_{fixity} providing that the effect of fixity on initial elastic period is ignored. The approach used in Appendix B was to develop design charts for a fixity factor, F_{fixity} = 1.0 and scale the resulting wall seismic capacity by the actual fixity factor applicable to the wall element. Ignoring the effect of fixity on initial elastic period allowed this very simple approach to be used for the design office procedure.

6 Computer Model for Face-Loaded Walls

6.1 Model Description

Figure 15 illustrates diagrammatically the components of the 2D computer model that was used in this study to analyse a face-loaded masonry single-storey wall with a parapet. The model of the masonry wall on the right of the diagram is linked through a roof diaphragm element to the model of a shear wall.

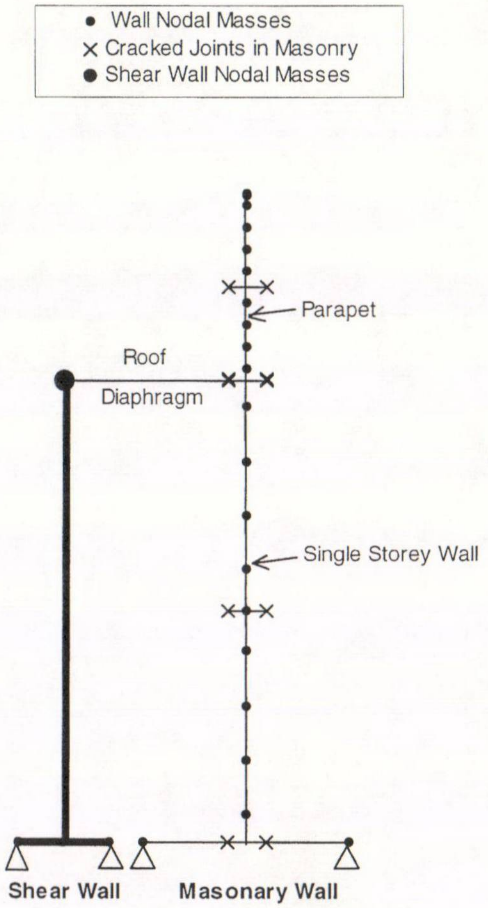


Figure 15: Diagram of Computer Model used to Analyse Face-loaded Masonry Single-Storey Walls with a Parapet

The complete model, as shown in Figure 15, was only used to evaluate the stability of a parapet supported by a single-storey building. To evaluate the stability of the single-storey wall as a separate wall element, the parapet was replaced by a central surcharge load (O/W ratio = 0.1) so that failure would occur in the first storey and not in the parapet. The parapet and wall elements were also analysed as freestanding cantilever walls. When the wall elements were used to model parapets and freestanding cantilever walls the mid-height crack was generally made inactive. However, some freestanding cantilever wall analyses were carried out with the mid-height crack able to open to evaluate the effect of intermediate crack opening on wall stability.

The wall element dimensions and model components used for the various series of computer analyses are shown in Table 5.

Table 5: Wall Element Dimensions and Model Components used for the Various Analysis Series

Analysis Series	Model Component		
	Single Storey Masonry Wall -- $h \times t_{nom}$ --	Masonry Parapet	Shear Wall Modelled
	(m)	(m)	
Single Storey Model with Surcharge:			
Model 1	4.5 x 0.23	O/W = 0.1	Yes
Model 2	2.1 x 0.23	O/W = 0.1	Yes
Free Standing Cantilever Wall Model:			
Model 1	4.5 x 0.23	NA	No
Model 2	2.1 x 0.23	NA	No
Model 3	1.0 x 0.23	NA	No
Model 4	0.5 x 0.23	NA	No
Single Storey Model with Parapet:			
Model 1	4.5 x 0.23	1.0 x 0.23	Yes
Model 2	2.1 x 0.23	0.5 x 0.23	Yes
Note: The 4.5 x 0.23m and 2.1 x 0.23m wall elements were modelled with a length of 0.49m. The 1.0 x 0.23 and 0.5 x 0.23m wall elements were modelled with a length of 1.0m			

6.1.1 Masonry wall Modelling

The masonry wall element models were similar to those used to model the Australian test specimen previously described in Figure 5 and Sect. 3.2.

Generally the beam elements that were used to model the masonry wall were modelled with an elastic modulus of elasticity of 2.0GPa. However, significant flexibility was also provided in the link members used to model the opening and closing joints. The assessment methodology, used to predict the computer model results, uses an effective uniformly distributed elastic modulus of 1.0GPa to evaluate the initial uncracked fundamental period of the wall response. The flexibility of the wall element models was, therefore, similar to that used in the assessment methodology. However, the 0.5 x 0.23m wall element modelled with these parameters exhibited significant high frequency vertical bounce in its response. The elastic modulus of the beam elements used to model the masonry wall were halved and the stiffness of the link members was increased by a factor of 4. These adjustments eliminated the bounce in the modelled response and gave an effective uniformly distributed elastic modulus close to the 1.0GPa value used in the assessment methodology (as determined from the 1st mode frequency of the model prior to crack opening).

Damping for each single-storey and freestanding cantilever wall model was adjusted as described in Sect 4.4.

6.1.2 Shear wall Modelling

An additional nodal mass was modelled at the roof level of the shear wall. This mass had a magnitude equal to approximately twice the tributary mass of the adjacent masonry wall (i.e. half the weight of first storey wall plus the mass of any parapet modelled x 2).

Almost all the shear walls flexibility was modelled as foundation level rotation (i.e. straight line deflected shape). The shear wall was modelled as having a fundamental period of 0.0 (i.e. rigid), 0.25, 0.5 or 1.0 seconds. Target damping for the wall was 10% of critical.

Drain2dx only permits damping of the wall to be fixed for 2 response periods. The 10% damping was fixed at response periods of 0.05 and 0.3 seconds for the wall modelled with a fundamental period of 0.25 seconds; at response periods of 0.1 and 0.6 seconds for the wall modelled with a fundamental period of 0.5 seconds; and at response periods of 0.2 and 0.9 for the wall modelled with a 1.0 second fundamental.

The shear wall periods and damping were selected to make some allowance for soil-structure interaction and to include an allowance for minor hysteric damping. The 1.0 second shear wall period is probably unrealistic for a single storey structure but was included for completeness.

6.1.3 Roof Diaphragm Modelling

The roof diaphragm members were modelled as elastic members with a fundamental response period of 0.0 (i.e. rigid), 0.25, 0.5 or 1.0 seconds. Damping of the diaphragms was modelled using stiffness damping only and fixed at 5% for the fundamental period.

The fundamental periods were calculated assuming the full tributary mass of the wall could be considered as being concentrated at the masonry wall end of the diaphragm. This would be reasonably accurate while the face-loaded wall remains elastic. However, previous research by the author (Blaikie and Spurr, 1992) has shown that a higher mode response can develop when the masonry wall cracks open and large mid-storey wall displacements occur. This higher mode can be visualised as the ends of the wall, which are fixed to the diaphragms, oscillating while the mid-height segment of the masonry wall remains stationary. Under these conditions the effective mass acting with the diaphragm is only approximately ¼ of the tributary masonry wall mass so that the period of this mode is half the fundamental period. As stiffness damping for a mode is proportional to the mode period, damping of the higher mode would be approximately 10%. However, as the parapet and the first storey wall element will tend not to respond in phase, this higher mode may not be as active for single storey walls modelled with parapets.

6.2 Analysis Procedure

Generally, for each set of parameters modelled, the earthquake intensity required to cause collapse was required. Therefore, a scaling factor was applied to the earthquake motion used for the analysis and this was incremented until the analysis indicated collapse had occurred. A 2.5% increment in the earthquake scaling factor was used (except where noted otherwise) and the last increment for which the wall remained stable was used to determine the collapse earthquake intensity.

Previous research (Blaikie, 1992 & 2000) has indicated that the vertical component of the earthquake does not significantly affect the stability of face-loaded URM walls. The frequency of the vertical component of the EQ motion tends to be high compared with the frequency of a cracked wall approaching collapse. Also, during any short time interval the additional vertical forces associated with the high frequency vertical component of the EQ has an equal probability of either improving or reducing the stability of the wall. Therefore, on balance, inclusion of the vertical component of the EQ motion in the computer analysis has little effect on the predicted collapse intensity and a vertical EQ motion component was not included in the analysis.

6.3 Analysis using Single Storey Wall Model with a Surcharge

6.3.1 Analysis Results for DR 0092 Earthquake Motion

The stability of the single-storey wall element was evaluated with the parapet replaced by a central surcharge load (O/W ratio = 0.1) so that failure would occur in the first-storey wall element and not in the parapet. Details of the computer models used for the analyses are given in Figure 15 and Table 5.

The effect of building flexibility on the earthquake intensity that would cause a single-storey wall to collapse was examined by including a range of shear wall and diaphragm flexibilities in the model as described in sections 6.1.2 and 6.1.3. The earthquake motion used as input for the analyses was the DR 9002 earthquake motion described in section 5.1.

Results of the analyses for the range of shear wall and diaphragm fundamental periods considered are summarised in Table 6 for the 4.5 x 0.23m and 2.1 x 0.23m single-storey wall elements modelled.

As the earthquake-scaling factor had to be incremented gradually to determine the lowest collapse intensity, the time history analysis was repeated on average 50 times for each set of parameters. Therefore, Table 5 summarises the results obtained from over 800 analyses using the computer models.

Table 6: Effect of Building Flexibility on Predicted Collapse EQ intensity for Single-storey Wall Model with Surcharge. Also Maximum Roof Diaphragm Forces Prior to Collapse – DR 0092 EQ Motion.

DR 0092 EQ Motion Shear Wall & Diaphragm Properties		EQ Scaling Factor, $I_{collapse}$ at Collapse & Implied Amplification Factor, A				Max. Roof Diaphragm Force: Recorded Value = kN & Normalised to 1.0 x DR 0092 EQ intensity & given as Seismic Coefficient = G's			
		Wall Dimensions – h x t_{nom}				Wall Dimensions – h x t_{nom}			
		4.5 x 0.23m		**2.1 x 0.23 m**		**4.5 x 0.23m**		**2.1 x 0.23 m**	
		$I_{collapse}$	A	$I_{collapse}$	A	kN	G's	kN	G's
		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
• **Shear wall period 0.0 sec (i.e. rigid shear wall)**									
- diaphragm Period	0.00 sec	**0.55**	**1.00**	**0.97**	**1.00**	-	-	-	-
ditto	0.25 sec	0.65	0.85	0.95	1.02	2.31	0.70	2.10	0.95
ditto	0.50 sec	0.57	0.96	1.02	0.95	2.08	0.71	1.98	0.83
ditto	1.00 sec	0.67	0.81	1.05	0.92	1.75	0.51	1.01	0.41
• **Shear wall period 0.25 sec**									
- diaphragm Period	0.00 sec	0.50	1.10	0.87	1.11	-	-	-	-
ditto	0.25 sec	0.52	1.05	0.90	1.08	2.50	0.94	2.5	1.19
ditto	0.50 sec	0.55	1.00	1.02	0.95	3.8	1.36	3.19	1.34
ditto	1.00 sec	0.62	0.88	1.00	0.97	1.76	0.56	1.02	0.44
• **Shear wall period 0.5 sec**									
- diaphragm Period	0.00 sec	0.52	1.05	0.92	1.05	-	-	-	-
ditto	0.25 sec	0.55	1.00	0.82	1.18	2.34	0.84	1.91	1.00
ditto	0.50 sec	0.52	1.05	0.75	1.29	2.06	0.77	1.99	1.14
ditto	1.00 sec	0.5	1.10	0.72	1.35	1.68	0.66	1.01	0.60
• **Shear wall period 1.0 sec**									
- diaphragm Period	0.00 sec	0.62	0.88	0.87	1.11	-	-	-	-
ditto	0.25 sec	0.57	0.96	1.10	0.88	1.69	0.58	1.64	0.64
ditto	0.50 sec	0.82	0.67	1.50	0.65	1.98	0.47	1.90	0.54
ditto	1.00 sec	0.70	0.79	1.42	0.68	1.50	0.42	1.01	0.31

Notes: Cols (2) to (4) – The implied building amplification factors, A are the collapse EQ intensity, $I_{collapse}$ given for the case of a rigid diaphragm & shear wall divided by the value of $I_{collapse}$ given for the various cases of diaphragm & shear wall flexibility.

Cols (5) to (8):- 1st value is maximum diaphragm force occurring prior to collapse of the 1st storey wall element given in kN. 2nd figure is diaphragm force expressed in terms of a seismic coefficient and normalised for 1.0 x the DR 0092 EQ intensity.

The earthquake scaling factors corresponding to collapse, when the shear wall and diaphragm are modelled as rigid, are shown in the first row of data in the table (in columns 1 and 3). It can be seen from the other data in columns 1 and 3 that the earthquake scaling factor required to cause the wall elements to collapse generally decreases when shear wall and/or diaphragm flexibility is introduced into the model. This reduction in seismic resistance implies that building flexibility results in an amplification of the earthquake motion imposed on the face-loaded wall elements. The implied amplification factors resulting from building flexibility are shown in columns 2 and 4 of the table.

The maximum diaphragm forces that occur for EQ scaling factors, up to the collapse intensity, are also given in the table (columns (5) to (8)) for the two single-storey wall elements modelled. For each height of wall element two values are given. The first is the peak diaphragm force in kN. The second value is the same force normalised. Normalisation was carried out by dividing the peak diaphragm force by the tributary weight of the adjacent masonry wall and by the EQ scale factor at collapse. The resulting normalised value is the seismic coefficient that the diaphragm would need to be designed for if the diaphragm was to remain elastic for an earthquake intensity of 1.0 x the DR 9002 EQ motion (although this would only be strictly true if the system was linearly elastic).

It can be seen from the table that the normalised diaphragm forces are relatively high, particularly where the diaphragm and shear wall are stiff.

No results are included for the rigid diaphragm cases as diaphragm forces obtained from the computer analyses are expected to be inaccurate under these conditions. A comparison between the peak diaphragm forces measured in a test specimen and the diaphragm forces predicted by the computer model showed that the peak forces associated with crack closing were greatly overestimated by the computer modelling (Blaikie, 2000). This over-estimate becomes less significant with increased diaphragm flexibility because increased diaphragm flexibility reduces the sharp peaks in the response associated with the impacts that occurs as masonry wall cracks close.

By comparing the diaphragm forces in columns 5 and 6 in Table 6 it can be seen that the maximum roof diaphragm force is not very sensitive to storey height. This could have been anticipated by considering Eqn 12 which indicates that the uniformly distributed load required to just open the cracks in the wall, H_{max} is independent of wall height (note, the wall weight, W is proportional to wall height, h so that the term W/h in Eqn 12 is independent of h). As opening of the wall cracks acts like a fuse and therefore limits the diaphragm forces, it could have been anticipated that the maximum diaphragm forces would also be independent of wall height.

The relatively high normalised seismic coefficients given in columns 6 and 7 of the table for the roof diaphragm, indicate that a relatively strong diaphragm would be required to prevent yielding (or failure). However, modelling of a 3 storey URM masonry wall (Blaikie, 2000) indicated that diaphragm yielding only reduced the seismic resistance of a face-loaded wall when both the wall and diaphragms were very flexible (both modelled with period of 1.0 secs) or, under some conditions, when the earthquake motion included near-fault pulse effects. Therefore, it is not expected that inclusion of diaphragm yielding in the model would result in an increase in the implied amplification factors given in Table 6.

6.3.2 Analysis Results for NZ4203 Earthquake Motion and Tokatori EQ Record

The single-storey wall computer models with the parapet replaced with a surcharge were also analysed using 2 other earthquake motions, an "NZ4203 earthquake motion" and the Tokatori earthquake record.

The NZS4203 earthquake motion used as one of the input ground motion for the computer modelling was produced by scaling the first 15 seconds of the 1940 El Centro NS earthquake record so that it more closely matched the Basic Seismic Hazard Spectra given in New Zealand's current Loading Standard, NZS4203, for intermediate soil profiles. The process used to scale the El Centro record was similar to that used to produce the DR 0092 earthquake motion as described in section 5.11. The acceleration and displacement spectra for the scaled earthquake motion are shown in Figure 16. The Loading Standard spectra are also show to indicate the degree of matching with the target spectra that was achieved by the scaling process.

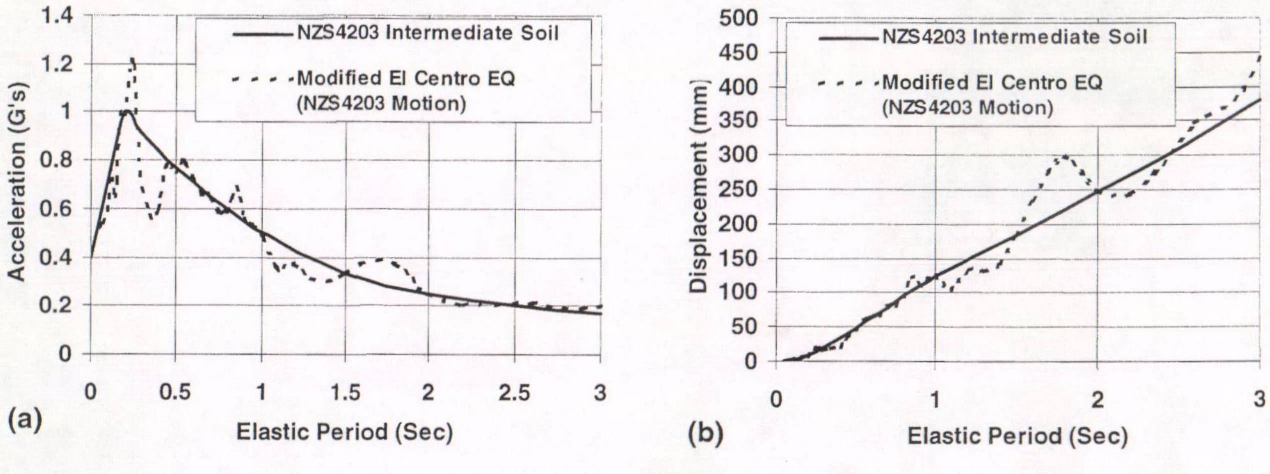


Figure 16: Elastic Response Spectra for NZS4203 Design Earthquake (Intermediate Soils) and for the El Centro EQ Record (5% damping) modified to match the NZS4203 spectral intensity (a) Acceleration spectra *and* (b) Displacement spectra

The Tokatori record is a near-fault EQ ground motion recorded during the 1995 Kobe (Great Hanshin) earthquake in the zone of greatest damage. The ground motion used as input for the computer model analyses was the Tokatori record between 1 and 16 seconds for the direction of maximum ground velocity. This part of the record includes the primary velocity pulse, which occurs between 5 and 7 seconds from the start of the record. This pulse has a period approaching 2 seconds and is typical of type of pulse found in the component of the ground motion normal to the fault in some near-fault earthquake records.

Figure 17 shows the acceleration and displacement response spectra for the Tokatori earthquake record. The basic DR 0092 design spectra for type C shallow sub soil profiles are also shown to enable a comparison to be made between the two earthquake spectral intensities. It can be seen that the Tokatori spectra have quite a different shape to the DR 0092 spectra. The proposed assessment methodology uses response spectra and could be sensitive to spectral shape. Use of the Tokatori record, as one of the input motions for the analyses, enables the applicability of the assessment methodology for a wide range of spectral shapes to be evaluated.

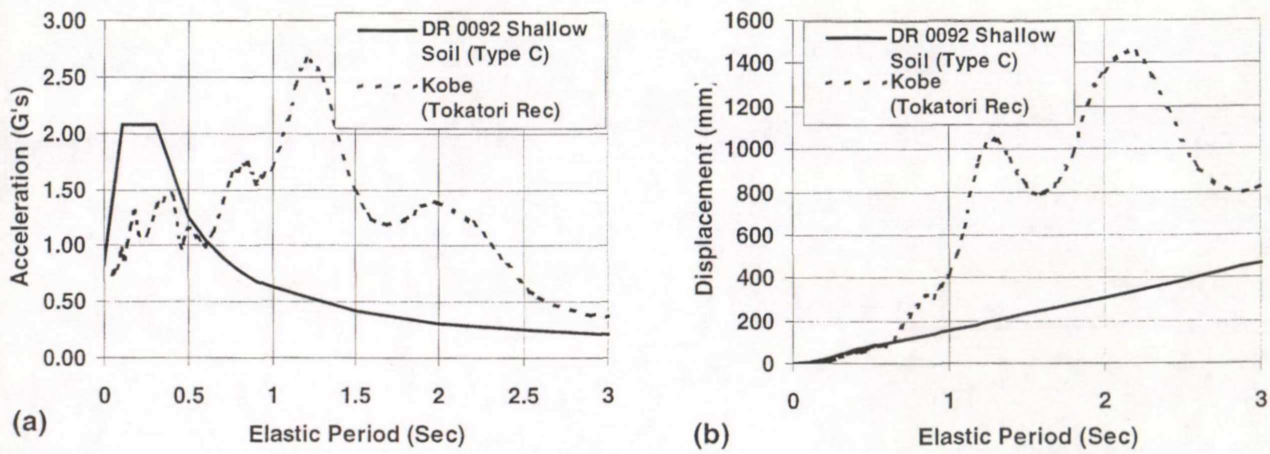


Figure 17: Elastic Response Spectra for Kobe Near Fault EQ Record (5% damping) and the DR 0092 Design Spectra for Type C Sub soils (a) Acceleration spectra *and* (b) Displacement spectra

Results of the analyses of the single-storey wall computer models with the parapet replaced with a surcharge are summarised in Table 7 and Table 8 for the NZ4203 earthquake motion and the Tokatori earthquake record respectively.

The results obtained for the NZ4203 earthquake motion, presented in Table 7, are similar to those obtained using the DR 0092 earthquake motion (Table 6).

The results obtained using the Tokatori record (Table 8) do, however, exhibit some significant differences. If the implied amplification factors, A, obtained when the building shear wall and/or diaphragm is modelled with a period of 1.0 seconds are set aside, the amplification factors for the 2.1x0.23m wall elements are modest (<1.28) and the values for the 4.5x0.23 m wall are all less than 1.0. These values are comparable with those obtained using the other code type spectra EQ motions.

Table 7: Effect of Building Flexibility on Predicted Collapse EQ intensity for Single Storey Wall Model with Surcharge - NZS4203 EQ Motion. Also Maximum Roof Diaphragm Forces Prior to Collapse

NZS4203 EQ Motion **Shear Wall & Diaphragm Properties**	EQ Scaling Factor, $I_{collapse}$ at Collapse & Implied Amplification Factor, A				Max. Roof Diaphragm Force: Recorded Value = kN & Normalised to 1.0 x NZS4203 EQ intensity & given as Seismic Coefficient = G's			
	Wall Dimensions – h x t_{nom}				Wall Dimensions – h x t_{nom}			
	4.5 x 0.23m		**2.1 x 0.23 m**		**4.5 x 0.23m**		**2.1 x 0.23 m**	
	$I_{collapse}$	A	$I_{collapse}$	A	kN	G's	kN	G's
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
• **Shear wall period 0.0 sec (i.e. rigid shear wall)**								
- diaphragm Period 0.00 sec	**1.00**	**1.00**	**1.825**	**1.00**	-	-	-	-
ditto 0.25 sec	1.025	0.98	1.80	1.01	2.07	0.40	2.17	0.52
ditto 0.50 sec	0.90	1.11	1.425	1.28	2.74	0.60	1.64	0.49
ditto 1.00 sec	0.90	1.11	1.6	1.14	1.63	0.36	1.00	0.27
• **Shear wall period 0.25 sec**								
- diaphragm Period 0.00 sec	0.975	1.03	2.00	0.91	-	-	-	-
ditto 0.25 sec	0.85	1.18	1.825	1.00	2.07	0.48	2.43	0.57
ditto 0.50 sec	0.80	1.25	1.225	1.49	1.96	0.48	1.65	0.58
ditto 1.00 sec	0.85	1.18	1.55	1.18	1.60	0.37	1.01	0.28
• **Shear wall period 0.5 sec**								
- diaphragm Period 0.00 sec	0.725	1.38	1.3	1.40	-	-	-	-
ditto 0.25 sec	0.725	1.38	1.475	1.24	1.72	0.47	1.98	0.58
ditto 0.50 sec	0.775	1.29	1.225	1.49	1.79	0.46	1.83	0.64
ditto 1.00 sec	0.95	1.05	1.325	1.38	1.93	0.40	1.01	0.33
• **Shear wall period 1.0 sec**								
- diaphragm Period 0.00 sec	0.85	1.18	1.2	1.52	-	-	-	-
ditto 0.25 sec	0.875	1.14	1.4	1.30	1.80	0.41	1.88	0.58
ditto 0.50 sec	0.90	1.11	1.55	1.18	1.89	0.41	1.73	0.48
ditto 1.00 sec	0.80	1.25	1.95	0.94	1.57	0.39	1.32	0.29

Notes: Cols (2) to (4) – The implied building amplification factors, A are the collapse EQ intensity, $I_{collapse}$ given for the case of a rigid diaphragm & shear wall divided by the value of $I_{collapse}$ given for the various cases of diaphragm & shear wall flexibility.

Cols (5) to (8):- 1st value is maximum diaphragm force occurring prior to collapse of the 1st storey wall element given in kN. 2nd figure is diaphragm force expressed in terms of a seismic coefficient and normalised for 1.0 x the NZS4203 EQ intensity.

Table 8: Effect of Building Flexibility on Predicted Collapse EQ intensity for Single Storey Wall Model with Surcharge – Near Fault Tokatori EQ Motion. Also Maximum Roof Diaphragm Forces Prior to Collapse

Tokatori EQ **Shear Wall & Diaphragm Properties**	EQ Scaling Factor, $I_{collapse}$ at Collapse & Implied Amplification Factor, A				Max. Roof Diaphragm Force: Recorded Value = kN & Normalised to 1.0 x Tokatori EQ intensity & given as Seismic Coefficient = G's			
	Wall Dimensions – h x t_{nom}				Wall Dimensions – h x t_{nom}			
	4.5 x 0.23m		**2.1 x 0.23 m**		**4.5 x 0.23m**		**2.1 x 0.23 m**	
	$I_{collapse}$	**A**	**$I_{collapse}$**	**A**	KN	G's	kN	G's
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
• **Shear wall period 0.0 sec (i.e. rigid shear wall)**								
- diaphragm Period 0.00 sec	**0.28**	**1.00**	**0.80**	**1.00**	-	-	-	-
ditto 0.25 sec	0.34	0.82	0.775	1.03	1.59	0.92	1.62	0.90
ditto 0.50 sec	0.34	0.82	0.65	1.23	1.58	0.92	1.45	0.96
ditto 1.00 sec	0.28	1.00	0.55	1.45	0.96	0.68	1.00	0.78
• **Shear wall period 0.25 sec**								
- diaphragm Period 0.00 sec	0.34	0.82	0.75	1.07	-	-	-	-
ditto 0.25 sec	0.34	0.82	0.70	1.14	1.70	0.99	1.37	0.84
ditto 0.50 sec	0.36	0.78	0.70	1.14	1.79	0.98	1.55	0.95
ditto 1.00 sec	0.28	1.00	0.525	1.52	1.42	1.00	1.00	0.82
• **Shear wall period 0.5 sec**								
- diaphragm Period 0.00 sec	0.32	0.88	0.625	1.28	-	-	-	-
ditto 0.25 sec	0.30	0.93	0.65	1.23	1.28	0.84	1.24	0.82
ditto 0.50 sec	0.32	0.88	0.65	1.23	1.44	0.89	1.43	0.94
ditto 1.00 sec	0.24	1.17	0.425	1.88	1.13	0.93	1.00	1.01
• **Shear wall period 1.0 sec**								
- diaphragm Period 0.00 sec	0.24	1.17	0.50	1.60	-	-	-	-
ditto 0.25 sec	0.26	1.08	0.50	1.60	1.67	1.27	1.52	1.30
ditto 0.50 sec	0.28	1.00	0.50	1.60	1.66	1.17	1.41	1.21
ditto 1.00 sec	0.20	1.40	0.375	2.13	1.25	1.23	1.01	1.16

Notes: Cols (2) to (4) – The implied building amplification factors, A are the collapse EQ intensity, $I_{collapse}$ given for the case of a rigid diaphragm & shear wall divided by the value of $I_{collapse}$ given for the various cases of diaphragm & shear wall flexibility.

Cols (5) to (8):- 1st value is maximum diaphragm force occurring prior to collapse of the 1st storey wall element given in kN. 2nd figure is diaphragm force expressed in terms of a seismic coefficient and normalised for 1.0 x the Tokatori EQ intensity.

However, much higher implied amplification factors were obtained for the Tokatori EQ motion when the computer model included more building flexibility.

For the Tokatori record the maximum roof diaphragm forces, for the same building component flexibility, are similar but generally less than those obtained using the other 2 earthquake motions. However, when the diaphragm forces are normalised they are almost twice as large for the Tokatori record. The diaphragm forces are limited by opening of wall joint cracks, which act as fuses. Under static UDL conditions the maximum top diaphragm force would be 0.88kN when the wall cracks just start to open. This force is independent of the 2 storey heights considered and the earthquake motion used in the analysis. The higher roof diaphragm force for the Tokatori record when the results are normalised, reflects the higher intensity of this earthquake motion.

6.4 Amplification Factors for Single-storey Walls

As part of a previous research project (Blaikie, 2000) a 3-storey masonry wall model, supported by a flexible shear wall and flexible diaphragms, was analysed to examine the effects that building flexibility would have on the performance of face-loaded masonry.

This research indicated that, when a buildings shear walls (and their foundations) can be considered rigid but the diaphragms are flexible, a storey level amplification factor, A = 1.2 can be expected for the wall element in the first storey of a multi-storey building.

When the building shear walls are more flexible it was proposed that:

$$A = 0.7(1 + 3\frac{h_i}{h_r}) \qquad \textbf{Eqn 21}$$

when the shear wall period is expected to be < 0.5 seconds

or

$$A = 0.7(1 + 2\frac{h_i}{h_r}) \qquad \textbf{Eqn 22}$$

when the shear wall period is expected to be > 1.0 seconds and with linear interpolation used when the shear wall period is between 0.5 and 1.0 seconds.

Where: h_i is the mid-storey height of the face loaded wall in the storey being assessed *and:*

h_r is the elevation of the building roof.

These amplification factors were considered to be applicable to earthquake motions that have a code type spectral shape similar to that used in NZS4203.

For a single-storey wall, equations Eqn 21 and Eqn 22 predict amplification factors of 1.75 when the shear wall period is < 0.5 seconds and 1.4 when the shear wall period is >1.0 seconds. When the buildings shear walls (or frame) can be considered as being rigid (i.e. say period < 0.1 seconds) an amplification factor of 1.2 would apply for a single-storey wall if it were considered to be the same as the first storey of multi-storey wall.

When the implied amplifications factors in Table 6 and Table 7 for the DR 0092 and NZS4203 type motions are inspected it can be seen that the results indicate that an amplification factor of 1.2 is applicable to single-storey structures when the shear wall is essentially rigid (i.e. say period < 0.1 seconds). However, when the shear wall is more flexible, the amplification factor predicted by Eqn 21 of 1.75 can be seen to be too conservative and a blanket amplification factor of 1.4 would be more appropriate.

The amplification factors given in Table 8 indicate that amplification factors of 1.2 for essentially rigid shear walls and 1.4 for more flexible diaphragms also appear to be generally applicable for the near-fault Tokatori EQ motion. However, when the sum of the shear wall and diaphragm periods exceeds 1.0 seconds the amplification factors given in column 4 of the table, for the more squat 2.1 x 0.23m wall element, indicate that a higher amplification factor than 1.4 would be more appropriate.

6.5 Effect of Reduced Damping on predicted capacity.

The 3-storey masonry wall computer model, analysed as part of a previous research project (Blaikie, 2000), included a 2.1 x 0.23m third storey floor element similar to the 2.1 x 0.23 wall element used to model one of the single-storey wall elements in this project. The method used in the previous research to model the damping resulted in the damping for the wall element being based on an average coefficient of restitution of 0.834 for the impacts that occur when the wall cracks close. This is smaller than the 0.854 coefficient of restitution indicated as appropriate for a 2.1 x 0.23 wall element by the test data presented in Figure 11 and used to model damping for the wall elements in this project.

As a smaller coefficient of restitution corresponds to a larger energy loss on impact, the smaller coefficient of restitution would result in higher modelled damping. Therefore, an increased collapse capacity, as predicted by the computer modelling, would be expected.

The 2.1 x 0.23m single-storey wall model with the parapet replaced by a surcharge was reanalysed with damping based on the smaller coefficient of restitution. Analyses were carried out using both the NZS4203 and Tokatori EQ motions.

The collapse intensity, $I_{collapse}$ predicted by the computer model for the 2 levels of damping is compared in Figure 18. The results obtained for the lower level of damping are the same as those given in column 4 of Table 7 and Table 8. When the Tokatori earthquake record was used for the analyses the increased damping did not have a significant effect. When the NZS4203 motion was used the effect was more significant and generally the higher damping resulted in a higher predicted collapse capacity.

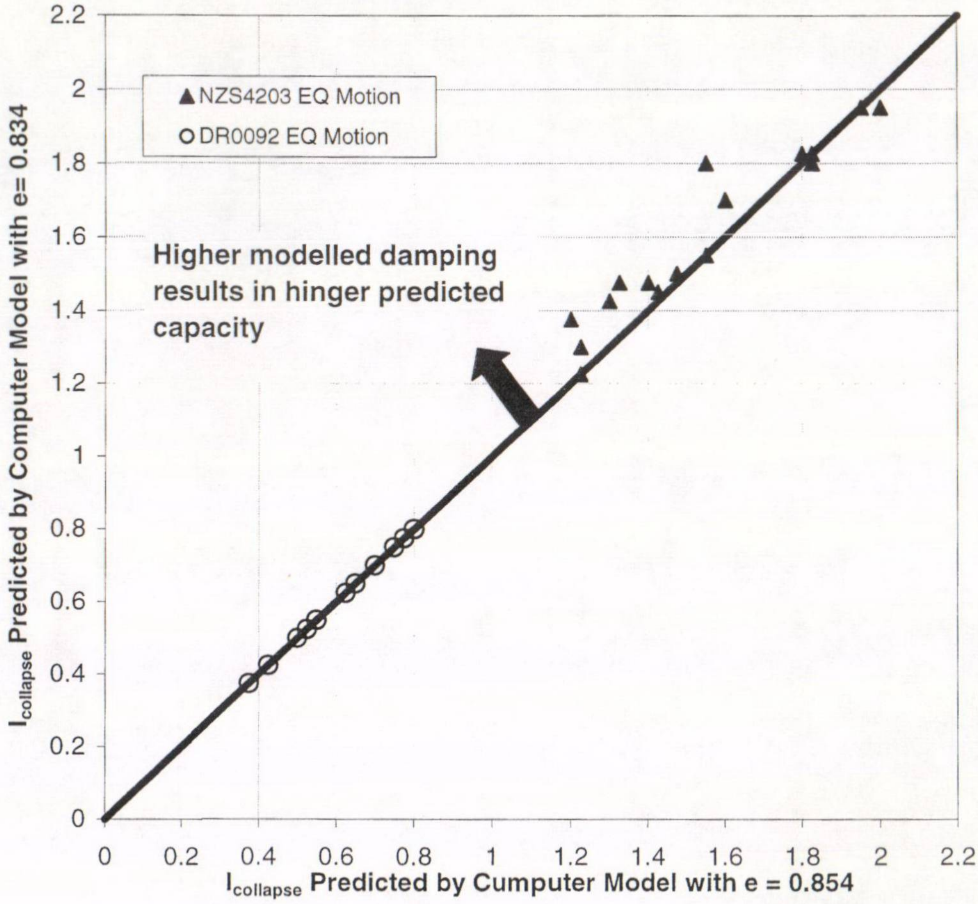


Figure 18: Collapse intensity, $I_{collapse}$ predicted by the Computer Model of a 2.1 x 0.23 Single-storey Wall Element for Two Levels of Damping

6.6 Analyses using Free Standing Cantilever Wall Models

The seismic stability of freestanding (or parapet) wall elements was evaluated using a series of freestanding computer wall models with the wall height varying between 0.5 and 4.5m. The nominal thickness, t_{nom}, of all the wall elements modelled was 0.23m. Details of the computer models used for the analyses were given in Table 5. The models used for the freestanding wall elements included a mid-height crack that was able to open and close. This crack was made either active or inactive so that the effect of an intermediate mid-height crack on the seismic resistance of the wall element could be evaluated.

EQ scaling factors predicted to cause the wall elements to collapse, $I_{collapse}$, as predicted by computer modelling and by the proposed methodology are shown in Table 9. Results are shown for the three earthquake motions used for seismic analyses in this study.

Table 9: Seismic Capacity of Freestanding or Parapet Wall Elements Predicted by Computer Modelling and by the Proposed Methodology

Procedure used To Predict Capacity	EQ Scaling Factor Predicted to Cause Wall Element to Collapse, $I_{collapse}$ — Height of Freestanding Wall Element (m)			
	0.5	**1.0**	**2.1**	**4.5**
• **DR 0092 EQ Motion**				
- Computer Model (no Mid-height crack)	0.93	0.6	0.31	0.24
- Computer Model (with Mid-height crack)	>1.88	0.6	0.32	0.19
- Parapet Methodology *	**0.79**	**0.42**	**0.3**	**0.23**
- Wall Methodology (wall = 4 x parapet ht)**	0.67	0.42	0.3	0.39
• **NZS4203 EQ Motion**				
- Computer Model (no Mid-height crack)	2.25	0.80	0.40	0.20
- Computer Model (with Mid-height crack)	2.25	0.98	0.45	0.175
- Parapet Methodology *	**1.62**	**0.76**	**0.36**	**0.19**
- Wall Methodology (wall = 4 x parapet ht)**	1.53	0.60	0.37	0.41
• **Tokatori EQ Record**				
- Computer Model (no Mid-height crack)	0.79	0.38	0.19	0.065
- Computer Model (with Mid-height crack)	0.67	0.38	0.19	0.065
- Parapet Methodology *	**1.1**	**0.47**	**0.14**	**0.08**
- Wall Methodology (wall = 4 x parapet ht)**	0.91	0.32	0.16	0.19

Notes:

* Parapet Methodology is procedure given in section 5.2 for parapet type elements.

** Wall Methodology is procedure given in section 5.2 for single-storey wall type elements but using a wall height that is 4 x the actual height of the parapet.

The increments in earthquake scaling factor, used to determine the collapse EQ intensity for the Tokatori EQ record, were 2.5%, 2%, 1% and 0.5% for the 0.5m, 1.0m, 2.1m and 4.5m high wall elements respectively instead of the standard 2.5% increment referred to in section 6.2. For the DR 0092 EQ motion a 1% increment was used for all but the 0.5m high wall element. These reduced increment values reflected the low wall element collapse EQ intensity anticipated in these cases.

For each earthquake motion considered, analysis results are given with the mid-height crack in the computer model either inactive or active. The EQ scaling factors required to cause the wall elements to collapse, as predicted using the parapet and single-storey wall methodologies described in section 5.2, are also given. In this case the wall methodology used was the procedure given in section 5.2 for single-storey wall type elements but using a wall height that is 4 x the actual height of the freestanding wall element.

Generally the inclusions of an active mid-height crack in the computer model of the wall element results in a higher or only marginally lower predicted seismic resistance. However, for the slender 4.5m high wall elements and the DR 0092 and NZS4203 EQ motions, inclusion of the mid-height crack reduced the collapse intensity below that predicted by the parapet methodology.

When Eqn 20 is used to predict the seismic resistance of a wall element for either the parapet or single-storey wall methodology, 2 components must be evaluated. One is, I_{sp}, the collapse capacity of the wall element predicted using a displacement spectrum the other, $2.5I_{cr}$, is the collapse capacity predicted using an acceleration spectrum.

For a given EQ displacement spectrum, I_{sp} depends primarily on the effective fundamental period of the cracked wall element. When Eqn 10 and Eqn 17 are compared it can be seen that a parapet wall (without overburden) will have the same cracked fundamental period of response as a single storey wall that is 4 times as high. Therefore, the value of I_{sp} calculated using the parapet methodology would be the same as that calculated using the wall methodology with the wall height taken as 4 x the actual parapet height.

When Eqn 6 and Eqn 15 are compared it can been seen that the seismic coefficient, C_d corresponding to first cracks opening in a parapet wall (without overburden) is also the same as that expected for a single storey wall 4 time as high as the parapet. If uncracked walls elements were considered as rigid blocks, the period of the wall element would be zero and the value of $2.5I_{cr}$ evaluated from the acceleration spectra using the parapet or single-storey wall methodologies would, therefore, also be the same. Hence, using Eqn 20 and either the parapet or single-storey methodologies would result in the same predicted seismic capacity of a freestanding type of wall element.

However, the uncracked wall elements are not rigid and the initial wall element fundamental period prior to cracking, calculated assuming the wall behaves as a cantilever, (as assumed for parapet methodology) is shorter than that calculated for a wall element behaving as a propped cantilever of 4 x the height (as assumed for the single-storey wall methodology). Hence the initial uncracked period of the wall element and the resulting component, $2.5I_{cr}$, evaluated using the acceleration spectrum also varies for the 2 methodologies.

It can be seen from Table 9 that, *for the squatter 0.5, 1.0 and 2.1m freestanding wall elements, use of the wall methodology instead of the parapet methodology results in a more conservative, or marginally less conservative, prediction of wall element seismic* *resistance. For the more slender 4.5m high wall element, the wall procedure results in a non-conservative prediction of seismic resistance when compared with both the parapet methodology and the computer modelling.*

Generally it can be seen from Table 9 that the seismic resistance of the freestanding wall elements, as expected from the computer modelling, is conservatively predicted by the parapet methodology. However, for the near-fault Tokatori EQ record the prediction is non-conservative except for the 2.1m high wall element. The prediction would have been improved by using $2.0I_{cr}$ instead of $2.5I_{cr}$ in Eqn 20. However, the prediction would still have been significantly non-conservative for the 0.5m high wall element.

6.7 Analyses using Single-storey Models with Parapet

6.7.1 Analysis Results for DR 0092 Earthquake Motion

The seismic stability of parapets supported on a single-storey building was evaluated using 2 computer models. One model had a 1.0 x 0.23m parapet supported on a 4.5 x 0.23 single-storey wall element the other had a 0.5 x 0.23m parapet supported on a 2.1 x 0.23m single-storey wall element. Details of the computer models used for the analyses are given in Table 5. The relative dimensions of the single-storey and parapet wall elements were selected so that there would be a high probability that the parapet would collapse before the single-storey wall element. The models used for the parapet and single-storey wall elements included a mid-height crack that was able to open and close. This crack was made inactive in the parapet wall elements.

EQ scaling factor that must be applied to the DR 0092 EQ motion to cause the parapets to collapse, $I_{collapse}$, as predicted by computer modelling, is given columns 1 and 5 of Table 10 for the two computer models analysed.

The earthquake scaling factors corresponding to collapse, when the shear wall and diaphragm are modelled as rigid, are shown in the first row of data in the table. It can be seen from the other data in columns 1 and 5 that the earthquake scaling factor predicted to cause the parapets to collapse generally decreases when shear wall and/or diaphragm flexibility is introduced into the model. This reduction in seismic resistance implies that introduction of building flexibility into the model results in an amplification of the earthquake motion imposed on the face-loaded wall elements. The implied amplification factors resulting from building flexibility are shown in columns 2 and 6 of the table. An upper limit for the amplification factors of approximately 2 is indicated by the analyses for the DR 0092 EQ motion.

Table 10: Effect of Building Flexibility on Predicted Collapse EQ intensity for Parapets on Single-storey Walls – DR 0092 EQ Motion.

DR 0092 EQ	Single-Story Wall Dimensions: 4.5 x 0.23m Parapet Dimensions: 1.0 x 0.23m				Single-storey Wall Dimensions: 2.1 x 0.23m Parapet Dimensions: 0.5 x 0.23m			
Shear Wall & Diaphragm Properties	Parapet		Single-storey Wall		Parapet		Single-storey Wall	
	$I_{collapse}$	Implied A	$I_{collapse}$	Horiz Displ Top/Mid-ht	$I_{collapse}$	Implied A	$I_{collapse}$	Horiz Displ Top/Mid-ht
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
• **Shear wall period 0.0 sec (i.e. rigid shear wall)**								
- diaphragm Period 0.00 sec	**0.66**	**1.00**	0.78	0.0/119	**0.93**	**1.00**	1.38	0/24
ditto 0.25 sec	0.67	0.99	0.77	9/123	0.95	0.98	1.18	13/25
ditto 0.50 sec	0.63	1.05	0.85	25/82	0.75	1.24	NA	34/52
ditto 1.00 sec	0.53	1.25	NA	49/32	0.53	1.75	NA	61/31
• **Shear wall period 0.25 sec**								
- diaphragm Period 0.00 sec	0.59	1.12	0.73	15/81	0.73	1.27	NA	24/26
ditto 0.25 sec	0.65	1.02	0.71	22/80	0.65	1.43	NA	28/17
ditto 0.50 sec	0.55	1.20	0.63	29/84	0.63	1.48	NA	36/39
ditto 1.00 sec	0.51	1.29	0.89	50/29	0.65	1.43	NA	83/40
• **Shear wall period 0.5 sec**								
- diaphragm Period 0.00 sec	0.39	1.69	0.71	25/20	0.53	1.75	1.23	38/21
ditto 0.25 sec	0.41	1.61	0.57	28/16	0.55	1.69	NA	45/23
ditto 0.50 sec	0.37	1.78	0.89	33/16	0.45	2.07	NA	46/23
ditto 1.00 sec	0.37	1.78	0.85	48/29	0.55	1.69	0.93	91/45
• **Shear wall period 1.0 sec**								
- diaphragm Period 0.00 sec	0.47	1.40	0.89	53/27	0.78	1.19	NA	96/47
ditto 0.25 sec	0.47	1.40	NA	55/31	0.78	1.19	NA	104/53
ditto 0.50 sec	0.47	1.40	NA	59/30	0.78	1.19	NA	106/53
ditto 1.00 sec	0.37	1.78	NA	73/35	0.60	1.55	NA	160/79

Notes: Cols (2) & (6) – The implied building amplification factors, A are the parapet collapse EQ intensity, $I_{collapse}$ given for the case of a rigid diaphragm & shear wall divided by the value of $I_{collapse}$ given for the various cases of diaphragm & shear wall flexibility.

Cols (5) & (8):- Horizontal displacement of computer modelled wall at top of single-storey wall (i.e. junction with parapet) and at mid-height crack in first storey. Maximum recorded up to failure of parapet.

During the analysis of the single-storey wall with a parapet model, the EQ scaling factor was gradually increased until either the wall element or parapet collapsed. At this time in the earthquake record the analysis stopped. In all the analyses carried out, the lowest EQ scaling factor required to cause a collapse corresponded to a parapet collapse rather than a wall element collapse. The EQ scaling factor for the increment immediately before the parapet collapsed is the value of $I_{collapse}$ given in columns 1 and 5 in the table. However, as the EQ scaling factor was increased above this value the parapet sometimes survived the higher intensity earthquakes and the single-storey wall element collapsed instead. The lowest EQ scaling factor that corresponded to a single-storey wall element collapse (reduced by one scaling increment) is given in columns 3 and 4 of the table. **It is important to note that this scaling factor is only an upper limit for the EQ scaling factor for collapse of the single-storey wall element as failure may have occurred at a lower EQ scaling factor if the analysis had not been stopped by a parapet collapse.** An "NA" in the table indicates that no collapses of the single-storey wall element occurred for the range of EQ scaling factors considered. The maximum EQ scaling factors considered were 1.03 and 1.67 for the 4.5 and 2.1 m high single-storey wall elements models respectively.

The maximum horizontal displacements recorded at the top (i.e. roof diaphragm level in Figure 15) and mid-height of the single-storey wall element are given in columns 4 and 8 of the table. These are the maximum horizontal displacements recorded at these locations for all EQ scaling factor increments considered prior to the collapse of the parapet. When the displacement at the mid-height crack location is only approximately half the value given for the top of the wall element (as it would be in a rigid wall rotating about its base) this is a good indicator that very little mid-height crack opening has taken place prior to the parapet collapsing. It can be seen that the cases that resulted in the lowest EQ scaling factor for the parapet to collapse, had relatively little mid-height crack opening in the single-storey element. The displacement at the top of the single-storey wall element also gives an indication of the total combined structural displacements taking place in the modelled shear wall and roof diaphragm.

Table 11 presents, for the DR 0092 EQ motion, the seismic capacity of the parapet and single-storey wall elements predicted by the computer modelling and as predicted by the proposed methodology.

For the parapet, two computer model analysis results are presented for the case of no building flexibility. The first is for the parapet modelled as a freestanding cantilever (as per Table 9) and the 2nd where the parapet is part of a single-storey model with a parapet (as per Table 10). The agreement between the predictions using the 2 models is close. The parapet seismic capacities predicted by the computer models can be compared with the capacities predicted by the parapet methodology for a rigid building. The methodology predictions are also given in the table. The parapet methodology was formulated so that, at the predicted capacity, the parapet would have a low probability of collapse. It can be seen that the capacity predicted by the methodology has the desired degree of conservatism.

When the single-storey building supporting the parapet is flexible, the amplified input at the base of the parapet is expected to reduce the seismic capacity of the parapet. If an amplification factor of A = 2.0 is assumed the capacities predicted by the parapet methodology are halved as indicated in the table. These predicted capacities can be compared with the minimum predicted capacities obtained from the analyses of the single-storey model with a parapet, given in Table 10, when the building is modelled with flexibility.

Table 11: Seismic Capacity of Parapet and Single-storey Wall Elements Predicted by Computer Modelling and by the Proposed Methodology – DR 0092 EQ Motion

Procedure used To Predict Seismic Resistance *for* **DR 0092 EQ Motion**	**EQ Scaling Factor Predicted to Cause Wall Element to Collapse, $I_{collapse}$**	
	Parapet Dimensions $h \times t_{nom}$	
	1.0 x 0.23m	0.5 x 0.23m
• **Parapet**		
- Computer Model (as freestanding wall element – Table 9)	0.60	0.93
- Computer Model (rigid single-storey model with parapet - Table 10)	0.66	0.93
- Computer Model (flexible single-storey model with parapet – Min value in Table 10 for flexible shear wall and/or diaphragm)	**0.37**	**0.45**
- Parapet Methodology (for rigid building = no amplification, A =1.0) *	0.42	0.79
- Parapet Methodology (flexible building; amplification factor, A =2.0)	**0.21**	**0.40**
	Single-storey Wall Dimensions - $h \times t_{nom}$	
	4.5 x 0.23m	2.1 x 0.23m
• **Single-storey Wall**		
- Computer Model (rigid single-storey model with parapet - Table 10)	> 0.66 but < 0.78	>0.93 but <1.38
- Computer Model (flexible single-storey model with parapet - Min value for parapet and wall elements in Table 10 for flexible shear wall and/or diaphragm)	**>0.37 but < 0.57**	**>0 .45 but < 0.93**
- Single-storey Wall Methodology: **		
• With full fixity at top (Bt = 1.0) and no Amplification, A = 1.0	0.45	1.11
• With full fixity at top (Bt = 1.0) and Amplification, A= 1.4	**0.32**	**0.79**

Notes:

* Parapet Methodology is procedure given in section 5.2 for parapet type elements.

** Wall Methodology is procedure given in section 5.2 for single-storey wall type elements, O/W= 0.45 and 0.47 for 4.5 and 2.1m high elements respectively.

OPUS

It can be seen that the parapet methodology is conservative particularly for the 1.0 x 0.23m parapet.

Similar comparison between the seismic capacities of the single-storey wall elements predicted by the computer modelling and by the single-storey wall methodology can also be made using the data in Table 11. The comparisons can be made for buildings with and without flexibility. In this case the computer modelling only indicates a range for the wall element capacity as collapse of the parapet stops the computer time-history analysis from proceeding. Therefore, had the parapet collapse not stopped the analysis, the single-storey wall element may have collapsed anywhere in the range indicated. As the EQ intensity was increased no single-storey wall elements collapsed prior the parapet collapsing. Therefore, the parapet capacity sets the lower limit for the capacity of the single-storey wall element.

It can be seen from the table that the capacities predicted by the single-wall methodology lies within the range indicted by the computer modelling for the 2.1 x 0.23m element and is conservative for the 4.5 x 0.23m element. Note that an amplification factor of 1.4 was assumed when applying the single-storey wall methodology to allow for building flexibility.

6.7.2 Analysis Results for NZS4203 Earthquake Motion

Analysis results similar to those described above for the DR 0092 EQ motion were obtained when the single-storey model with a parapet was analysed using the NZS4203 EQ motion. Results of the analyses performed with the NZS4203 EQ motion are presented in Table 12 and the seismic capacity of the parapet and single-storey wall elements predicted by the computer modelling and by the proposed methodology are compared in Table 13.

Some aspects of the data presented in Table 13 are as follows:

- The first 2 rows of data indicate that the agreement between the seismic capacities predicted for the parapet using the freestanding wall model and using the single-storey model with a parapet (when the building structure is rigid) are not as good as that obtained when the DR 0092 EQ motion was used in the analyses. However, modelling the parapet as a stand-alone freestanding cantilever wall element instead of modelling it as a parapet supported by a single-storey building is more conservative for the 1.0m high parapet but less conservative for the 0.5m parapet.

- The parapet capacity predicted using the parapet methodology for a rigid building is conservative as intended, and is in good agreement with the minimum capacity predicted by the computer modelling (compare the data in rows 1 and 2 with that in row 4).

Table 12 Effect of Building Flexibility on Predicted Collapse EQ intensity for Parapets on Single-storey Walls – NZS4203 EQ Motion.

NZS4203 EQ	Single-Story Wall Dimensions: 4.5 x 0.23m Parapet Dimensions: 1.0 x 0.23m				Single-storey Wall Dimensions: 2.1 x 0.23m Parapet Dimensions: 0.5 x 0.23m			
Shear Wall & Diaphragm Properties	Parapet		Single-storey Wall		Parapet		Single-storey Wall	
	$I_{collapse}$	Implied A	$I_{collapse}$	Horiz Displ Top/Mid-ht	$I_{collapse}$	Implied A	$I_{collapse}$	Horiz Displ Top/Mid-ht
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
• **Shear wall period 0.0 sec (i.e. rigid shear wall)**								
- Diaphragm Period 0.00 sec	1.05	1.00	1.28	0.0/26	1.78	1	NA	0.0/2
ditto 0.25 sec	1.0	1.05	1.30	7/33	1.53	1.16	NA	13/13
ditto 0.50 sec	1.1	0.95	1.13	24/114	1.08	1.65	NA	25/14
ditto 1.00 sec	0.8	1.31	1.00	46/30	0.90	1.98	NA	83/42
• **Shear wall period 0.25 sec**								
- Diaphragm Period 0.00 sec	0.83	1.27	0.93	11/28	1.23	1.45	NA	16/11
ditto 0.25 sec	0.88	1.19	1.10	14/44	0.95	1.87	NA	18/9
ditto 0.50 sec	0.93	1.13	1.03	33/46	0.85	2.09	NA	32/18
ditto 1.00 sec	0.75	1.40	1.15	47/31	0.90	1.98	NA	91/46
• **Shear wall period 0.5 sec**								
- Diaphragm Period 0.00 sec	0.80	1.31	0.93	39/31	0.68	2.62	NA	43/20
ditto 0.25 sec	0.80	1.31	0.98	43/42	0.83	2.14	NA	51/28
ditto 0.50 sec	0.70	1.50	0.98	48/36	0.68	2.62	NA	32/27
ditto 1.00 sec	0.65	1.62	0.78	51/28	0.73	2.44	NA	91/46
• **Shear wall period 1.0 sec**								
- diaphragm Period 0.00 sec	0.60	1.75	1.03	53/28	1.20	1.48	NA	121/60
ditto 0.25 sec	0.70	1.50	>1.43	65/31	1.20	1.48	NA	126/63
ditto 0.50 sec	0.70	1.50	1.23	76/39	1.10	1.62	NA	127/63
ditto 1.00 sec	0.63	1.67	>1.43	86/44	0.95	1.87	NA	147/73

Notes: Cols (2) & (6) – The implied building amplification factors, A are the parapet collapse EQ intensity, $I_{collapse}$ given for the case of a rigid diaphragm & shear wall divided by the value of $I_{collapse}$ given for the various cases of diaphragm & shear wall flexibility.

Cols (5) & (8):- Horizontal displacement of computer modelled wall at top of single-storey wall (i.e. junction with parapet) and at mid-height crack in first storey. Maximum recorded up to failure of parapet.

Table 13: Seismic Capacity of Parapet and Single-storey Wall Elements Predicted by Computer Modelling and by the Proposed Methodology – NZS4203 EQ Motion

Procedure used To Predict Seismic Resistance *for* NZS4203 EQ Motion	**EQ Scaling Factor Predicted to Cause Wall Element to Collapse, $I_{collapse}$**	
	Parapet Dimensions h x t_{nom}	
	1.0 x 0.23m	0.5 x 0.23m
• **Parapet**		
- Computer Model (as freestanding wall element – Table 9)	0.80	2.25
- Computer Model (rigid single-storey model with parapet - Table 12)	1.05	1.78
- Computer Model (flexible single-storey model with parapet – Min value in Table 10 for flexible shear wall and/or diaphragm)	**0.60**	**0.68**
- Parapet Methodology (for rigid building = no amplification, A =1.0) *	0.76	1.62
- Parapet Methodology (flexible building; amplification factor, A =2.0)	**0.38**	**0.81**
	Single-storey Wall Dimensions - h x t_{nom}	
	4.5 x 0.23m	2.1 x 0.23m
• **Single-storey Wall**		
- Computer Model (rigid single-storey model with parapet - Table 12)	>1.05 but <1.28	NA
- Computer Model (flexible single-storey model with parapet – Min values for parapet and wall elements in Table 12 for flexible shear wall and/or diaphragm)	**>0.60 but < 0.78**	NA
- Single-storey Wall Methodology: **		
• With full fixity at top (Bt = 1.0) and no Amplification, A = 1.0	1.10	2.50
• With full fixity at top (Bt = 1.0) and Amplification, A= 1.4	**0.78**	**1.78**

Notes:

* Parapet Methodology is procedure given in section 5.2 for parapet type elements.

** Wall Methodology is procedure given in section 5.2 for single-storey wall type elements, O/W= 0.45 and 0.47 for 4.5 and 2.1m high elements respectively.

- When the single-storey model with a parapet is modelled with flexible shear walls and/or diaphragms, the minimum EQ scaling factor required to cause collapse of the 1.0m high parapet (0.6) is surprising close to that required to collapse the 0.5m high parapet (0.68). When the parapet methodology with an amplification factor of A=2.0 to allow for building flexibility is used, the predicted EQ scaling factors are 0.38 and 0.81 for the 1.0 and 0.5 m high parapets respectively. It can be seen from the data presented in column 5 of Table 12 that the 0.81 EQ scaling factor predicted by the methodology to cause the 0.5m high parapet to collapse is only non-conservative when the shear wall is modelled with a 0.5 second period. In this case the implied amplification factors given in column 6 of the table are greater than the value of 2.0 proposed for use in the methodology.

- For the analyses both with and without amplification, it can be seen from Table 13 that the seismic capacities predicted by the single-storey wall methodology for the 4.5m high wall element lies at the upper limit of the range indicated by the computer modelling. No collapses of the 2.1m high single-storey wall elements occurred for the range of EQ intensities considered in the computer model analyses. This in not surprising given the relatively high predicted capacities given in Table 13 for the 2.1m high wall element compared with those predicted for the 0.5m high parapet.

6.7.3 Analysis Results for Tokatori Earthquake Record

Analysis results similar to those described above for the DR 0092 and NZS4203 EQ motions were obtained when the single-storey model with a parapet was analysed using the Tokatori EQ record. Results of the analyses performed with the Tokatori EQ record are presented in Table 14 and the seismic capacity of the parapet and single-storey wall elements predicted by the computer modelling and by the proposed methodology are compared in Table 15.

Some aspects of the data presented in Table 15 are as follows:

- The first 2 rows of data indicate that the agreement between the seismic capacities predicted for the parapet using the freestanding wall model and using the single-storey model with a parapet and a rigid building structure is very good. However, the predictions made using the parapet methodology for parapets supported on a rigid building structure can be seen to be non-conservative for this near-fault EQ record (compare the data in rows 1 and 2 with that 5th row of data).

- When the single-storey model with a parapet is modelled with flexible shear walls and/or diaphragms, the minimum EQ scaling factors predicted to cause collapse of the parapets using the parapet methodology with an amplification factor of 2.0 can be seen to be highly non –conservative if the full range of building flexibility is considered. However, if the building structure flexibility is limited (sum of shear wall and diaphragm period < 1.0 seconds) the use of an amplification factor of 2.0 in the parapet methodology gives a reasonable prediction of the minimum capacity indicated by the computer modelling. For buildings with more flexible shear walls and diaphragms it can be seen from the data in columns 2 and 6 of Table 14 that the amplification implied by the computer modelling can be very much higher than 2.0.

Table 14: Effect of Building Flexibility on Predicted Collapse EQ intensity for Parapets on Single-storey Walls – Tokatori Near-fault EQ Record.

Tokatori EQ Shear Wall & Diaphragm Properties	Single-Story Wall Dimensions: 4.5 x 0.23m Parapet Dimensions: 1.0 x 0.23m				Single-storey Wall Dimensions: 2.1 x 0.23m Parapet Dimensions: 0.5 x 0.23m			
	Parapet		Single-storey Wall		Parapet		Single-storey Wall	
	$I_{collapse}$	Implied A	$I_{collapse}$	Horiz Displ Top/Mid-ht	$I_{collapse}$	Implied A	$I_{collapse}$	Horiz Displ Top/Mid-ht
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
• **Shear wall period 0.0 sec (i.e. rigid shear wall)**								
- diaphragm Period 0.00 sec	0.38	1.00	0.48	0/2	0.73	1	2.25	0/1
ditto 0.25 sec	0.35	1.09	0.63	5/6	0.65	1.12	1.70	8/4
ditto 0.50 sec	0.33	1.15	0.59	16/11	0.58	1.26	1.65	35/18
ditto 1.00 sec	0.19	2.00	0.51	46/23	0.28	2.61	NA	76/38
• **Shear wall period 0.25 sec**								
- diaphragm Period 0.00 sec	0.31	1.23	0.47	8/5	0.43	1.70	1.50	11/5
ditto 0.25 sec	0.25	1.52	0.47	8/4	0.53	1.38	1.40	20/11
ditto 0.50 sec	0.25	1.52	0.57	18/9	0.45	1.62	1.38	33/11
ditto 1.00 sec	0.21	1.81	0.47	51/27	0.30	2.43	NA	92/46
• **Shear wall period 0.5 sec**								
- diaphragm Period 0.00 sec	0.25	1.52	0.51	19/9	0.48	1.52	0.83	36/18
ditto 0.25 sec	0.25	1.52	0.55	21/10	0.48	1.52	NA	43/22
ditto 0.50 sec	0.21	1.81	0.51	28/13	0.40	1.83	NA	56/28
ditto 1.00 sec	0.15	2.53	0.41	49/23	0.25	2.92	1.55	102/51
• **Shear wall period 1.0 sec**								
- diaphragm Period 0.00 sec	0.11	3.45	0.27	59/30	0.20	3.65	1.55	128/63
ditto 0.25 sec	0.13	2.92	0.27	75/39	0.18	4.06	NA	111/55
ditto 0.50 sec	0.09	4.22	0.41	56/29	0.18	4.06	1.6	126/63
ditto 1.00 sec	0.07	5.43	0.35	64/32	0.18	4.06	1.23	203/102

Notes: Cols (2) & (6) – The implied building amplification factors, A are the parapet collapse EQ intensity, $I_{collapse}$ given for the case of a rigid diaphragm & shear wall divided by the value of $I_{collapse}$ given for the various cases of diaphragm & shear wall flexibility.

Cols (5) & (8):- Horizontal displacement of computer modelled wall at top of single-storey wall (i.e. junction with parapet) and at mid-height crack in first storey. Maximum recorded up to failure of parapet.

Table 15: Seismic Capacity of Parapet and Single-storey Wall Elements Predicted by Computer Modelling and by the Proposed Methodology – Tokatori EQ Record

Procedure used To Predict Seismic Resistance *for* Tokatori EQ Record	EQ Scaling Factor Predicted to Cause Wall Element to Collapse, $I_{collapse}$	
	Parapet Dimensions h x t_{nom}	
	1.0 x 0.23m	0.5 x 0.23m
• **Parapet**		
- Computer Model (as freestanding wall element – Table 9)	0.38	0.79
- Computer Model (rigid single-storey model with parapet - Table 14)	0.38	0.73
- Computer Model (flexible single-storey model with parapet – Min value in Table 14 for flexible shear wall and/or diaphragm)	0.07	0.18
- Computer Model (as above but Min value in Table 14 when sum of the shear wall and diaphragm periods < 1.0 secs)	**0.25**	**0.43**
- Parapet Methodology (for rigid building = no amplification, A =1.0) *	0.47	1.10
- Parapet Methodology (flexible building; amplification factor, A =2.0)	**0.23**	**0.55**
	Single-storey Wall Dimensions - h x t_{nom}	
	4.5 x 0.23m	2.1 x 0.23m
• **Single-storey Wall**		
- Computer Model (rigid single-storey model with parapet - Table 14)	>0.38 but <0.48	>0.73 but < 2.25
- Computer Model (flexible single-storey model with parapet – Min values for parapet and wall elements in Table 14 for flexible shear wall and/or diaphragm when sum of the shear wall and diaphragm periods < 1.0 secs)	**> 0.25 but < 0.47**	**> 0.43 but < 1.4**
- Single-storey Wall Methodology: **		
• With full fixity at top (Bt = 1.0) and no Amplification, A = 1.0	0.46	1.77
• With full fixity at top (Bt = 1.0) and Amplification, A= 1.4	**0.33**	**1.26**

Notes:

* Parapet Methodology is procedure given in section 5.2 for parapet type elements.

** Wall Methodology is procedure given in section 5.2 for single-storey wall type elements, O/W= 0.45 and 0.47 for 4.5m and 2.1m high elements respectively.

- For the analyses both with and without amplification due to building flexibility, it can be seen from Table 13 that the seismic capacities predicted by the single-storey wall methodology lie within the range indicated by the computer modelling. However, the 0.43 to 1.4 range indicated for the 2.1m high wall element by the computer modelling is relatively wide. It should also be noted that, in setting the range, the upper limit for the range indicated by the value of 0.83 given in column 7 of Table 14 for the case where the shear wall period was 0.5 seconds and the diaphragm was rigid was ignored. In this case the model was analysed for 50 EQ scaling factors incremented in the range 0.4 and 1.63 and collapse of the 2.1m high single-storey wall element only occurred when the EQ scaling factor was 0.83. Therefore, the result of this analysis was treated as the type of anomaly that can be expected from time to time in dynamic analysis.

7 Comparison Between Computer Modelling and Proposed Methodology

The earthquake scaling factors that are expected to cause a range of face-loaded freestanding walls and parapets to collapse, as predicted by the computer modelling and as predicted by the proposed methodology, are compared in Figure 19.

The data used for plots was extracted from the various tables of this report. Plots constructed using the data obtained when the DR0092 and NZS4203 EQ motions were used in the analysis are shown in Figure 19(a). Plots constructed using the data obtained when the Tokatori EQ motion was used in the analysis are shown separately in Figure 19(b).

For the Tokatori near-fault EQ motion only the data obtained when the sum of the shear wall and diaphragm periods in the computer model was less than 1.0 seconds was used for the plots. Even with this restriction it can be seen that the methodology tends to predict only the mean seismic resistance indicated by the computer modelling. Similar results were obtained previously (Blaikie, 2000) for single-storey wall elements in multi-storey walls.

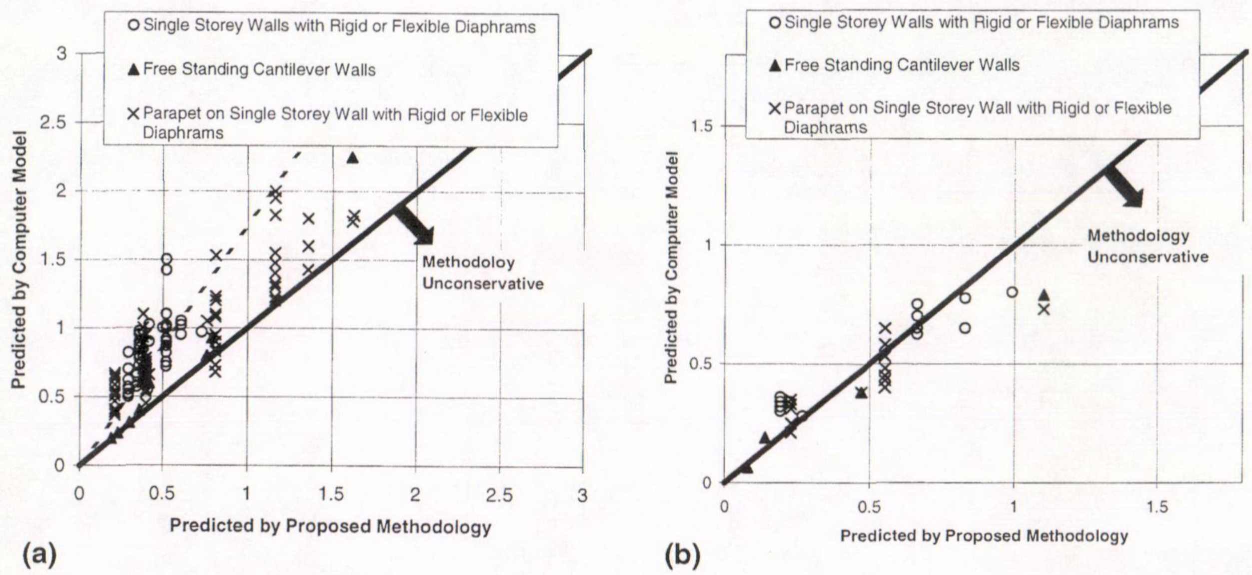


Figure 19: Earthquake Scaling Factors Required to Cause Collapse of Face-Loaded Freestanding Walls and Parapets – Comparison of the Values Predicted using the Computer Modelling and those Predicted using the Proposed Methodology (a) when Modelling using DR0092 and NZS4203 EQ Motions and (b) when Modelling using Near-fault Tokatori EQ motion

However, for the code type EQ motions (DR0092 and NZS203), Figure 19(a) indicates that the proposed methodology can be used to predict the seismic capacity of face-loaded URM freestanding walls and parapets with a low probability of failure. Therefore, when the methodology is used to predict collapse intensities with a low probability of failure, no additional safety factors need to be applied.

Figure 19(a) also shows a straight (dotted) line plotted so that 50% of the results lie above and below the line. ***This plot indicates that if a mean or expected collapse intensity is required for the seismic assessment of a face-loaded single storey, freestanding wall or parapet, the value predicted using the proposed methodology should be increased by approximately 75%.*** If the freestanding walls and parapets data were considered separately the increase seismic resistance would reduce to 65%.

8 Summary and Conclusions

The following is a summary of the work undertaken in this study and the conclusions reached.

8.1 Behaviour of Cracked Face Loaded URM Wall Elements

The equations describing the behaviour of a cracked parapet or cantilever wall under static loading and the equation describing its free vibration period are remarkably similar to those derived in previous research for a face loaded single storey wall element supported horizontally at the top and bottom of the wall element.

The equations indicate that a parapet type wall element, cracked at its base and responding as a rigid block, will have the same fundamental period of response as a single storey wall element that is 4 times as high if the 2 wall elements have the same overburden to weight ratio.

The equations also indicate that the seismic coefficient, C_d corresponding to opening of a crack at the base of a parapet (without overburden) is also the same as that expected for first opening of the joint cracks in a single storey wall 4 time as high. Therefore, as the fundamental period of response and the seismic coefficient corresponding to crack opening are thought to control the seismic stability of a face-loaded wall element, it was concluded that:

- **A cracked parapet or cantilever type wall element can be simulated by a single storey wall element 4 times as high.**

However, computer modelling indicated that this relationship would only hold when the cracked cantilever wall or parapet can be considered as a rigid block (i.e. squat wall elements). Also the relationship will not apply when the wall elements support an overburden load (e.g. a cantilever wall supporting an ineffective roof diaphragm or a parapet with an eccentric corbel).

8.2 Australian Laboratory Test Specimen Modelling and Analysis

The measured response of a single-storey URM wall test specimen, recently tested on a shake table in Australia, was obtained. The specimen had been subjected to seismic forces normal to the face of the wall so that the mid-height of the wall under went large displacements similar to the wall thickness and large cracks opened at joints near the mid-height and at the base of the wall. Inelastic dynamic modelling was then used to predict the time-history of the mid-height displacement of the test specimen. Agreement between the response predicted by the test and computer model was excellent and established that:

- **An inelastic computer model can be used to model the earthquake displacement response, and hence the seismic stability, of a URM face-loaded wall element.**

Compression tests on wall samples similar to the materials used in the wall specimen indicated that the elastic modulus of the wall masonry (in compression) averaged 9.4GPa. The effective wall flexural stiffness indicated by the modelling of the test specimen was only approximately 0.2% of this value. Prior to the shake table test that was modelled, the test specimen had been subjected to a number of other tests that resulted in significant crack opening and closing. It was concluded that:

- **After significant crack opening and closing, most of the flexibility of a face-loaded wall element will be the result of rotations at the opening mortar joints. The Australian tests also indicate that significant deterioration in the wall flexural stiffness and effective thickness may take place during a walls response to an earthquake.**

When a cracked face-loaded wall element responds to an EQ motion, impact occurs when the joint cracks open and close and this results in energy loss or damping of the walls response. Previous research on face loaded single-storey wall elements indicated that a relatively constant proportion of the energy that is stored in a wall element immediately prior to the cracks closing was lost each time they closed. This feature of the dynamic response of the wall can be used to model damping in a computer model of a face-loaded wall element when parameters such as the wall height or thickness are varied from that tested. A coefficient of restitution can be calculated for each of the impacts that occur when the cracks close in a test specimen and is related to the energy loss and damping in the wall element. Analysis of a NZ test specimen results and some of the Australian test specimen results indicated that:

- **The average coefficient of restitution increases in a linear manner with increased wall element slenderness. This indicates reduced energy loss and damping with increasing slenderness. The test results also indicated that cantilever/parapet type wall elements have damping that is consistent with a single storey wall element, supported horizontally at the top and bottom of the element, which is 4 times as high and of the same thickness. This relationship between the 2 types of wall element is consistent with the relationship between the fundamental equations that govern the behaviour of the 2 types of wall element.**

8.3 Assessment Methodology

An assessment methodology, developed in pervious research that can be used to predict the seismic stability of a cracked face-loaded URM multi-storey wall was extended in this study to cover single-storey walls, freestanding cantilever walls and parapets.

The methodology makes use of both the acceleration and displacement response spectra for an earthquake motion. The acceleration spectrum is used to predict the earthquake intensity that will just open the joint cracks in the wall element. 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. Modification factors are applied to allow for the effect of the wall element boundary conditions and to allow for amplification of the earthquake motion due to flexibility in any building structure or diaphragms that support the wall element.

The methodology was principally developed using inelastic dynamic analysis of a 3-storey computer model of a face-loaded URM wall supported by flexible shear walls and flexible or yielding floor and roof diaphragms. In this study, inelastic dynamic analysis of computer models was used to extend the assessment methodology to cover face-loaded single-storey walls, freestanding cantilever walls and parapets. Parameters examined included interaction between the parapet and its supporting wall, effect of building and/or diaphragm flexibility and the presence of long acceleration pulses in the ground motion which may occur in the near-fault zone during an earthquake. The analyses indicated that:

- **The earthquake intensity required to collapse a face-loaded wall element, as indicated by the computer modelling, is generally predicted conservatively by the proposed methodology. However, when the earthquake motion contains a near-fault long duration pulse, the methodology is not conservative and tends to predict the mean earthquake intensity required to collapse the wall element. The methodology also becomes non-conservative for near-fault EQ motions if the building and/or diaphragms have more than moderate flexibility.**

8.4 Design Charts

Design charts to enable rapid design office assessment of face-loaded wall elements in terms of the current proposed revision to the NZS4203 Loading Standard Basic Seismic Hazard Spectra (i.e. DR 1170.4/PPC3) are provided in Appendix B. Similar design charts could be prepared for other earthquake records or code design spectral intensities using the proposed methodology.

Acknowledgements

The author gratefully acknowledges the funding of this research project by the EQC Research Foundation. Thanks also to Australian researchers Kevin Doherty, John Wilson and Nelson Lam who generously made available their test data in electronic form for reanalysis by the author. Thanks to Graeme McVerry of GNS who made available copies of EQ records that were suitable for scaling to match the proposed code spectra and Andrew King of BRANZ who approved their release to the author before their official publication. Finally I would like to acknowledge Lou Robinson of Hadley & Robinson Ltd for his helpful suggestions and the mathematical formulations he derived for rocking wall elements.

9 References

ABK Joint Venture (1982): "Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings – Report ABK-TR-08 Interpretation of Wall Tests: Out- of-plane" El Segundo, CA Agbabian Associates

Blaikie E L (2000): "Methodology for the Assessment of Face Loaded Unreinforced Masonry Walls under Seismic Loading". Research report sponsored by the New Zealand Earthquake and War Damage Commission, Opus International Consultants, May 2000.

Blaikie E L and Davey R A (2000): "Seismic Behaviour of Face Loaded Unreinforced Masonry Walls". 12th World Conference on Earthquake Engineering, Auckland New Zealand.

Blaikie E L and Spurr D D (1992): "Earthquake Vulnerability of Existing Unreinforced Masonry Buildings". Research report sponsored by the New Zealand Earthquake and War Damage Commission, Works Consultancy Services, December 1992.

Doherty KT, Rodolico B, Lam TK, Wilson JL, Griffith C (2000): "The Modelling of Earthquake Induced Collapse of Unreinforced Masonry Walls Combining Force and Displacement Principals" ". 12th World Conference on Earthquake Engineering, Auckland New Zealand.

Doherty KT (2000) "An Investigation of the Weak Links in the Seismic Load Path of Unreinforced Masonry Buildings" PhD thesis submitted to Faculty of Engineering at the University of Adelaide.

Lam N, Wilson J L, Hutchinson G L (2001): "Seismic Assessment of Geometric Unreinforced Masonry Wall sections based on Displacement" 6th Australian Masonry Conference, Adelaide Australia.

Lam N T K, Wilson J L, Hutchinson G L (1995): "Modelling of an Unreinforced Masonry parapet wall for seismic performance evaluation based on dynamic testing", Dept. of Civil & Environmental Engineering, The University of Melbourne, Report no. RR/Strurct/03/95

NZNSEE (1995): "Draft Guidelines for Assessing And Strengthening Earthquake Risk Buildings" New Zealand National Society for Earthquake Engineering, 10 February 1995.

Opus International Consultants (1989) "Wave-1 Computer Programme for Scaling Earthquake Records" Opus Civil Engineering Information Document, CEI 28 issue 09.1989.

Robinson, L (2001): Private correspondence with the author.

Standards New Zealand (2000) "DR 0092: Structural Design – General Requirements and design actions Part 4: Earthquake actions." Draft Australian/New Zealand standard for comment.

Standards New Zealand (2002) "DR 1170.4/PPC3: Structural Design Actions – Part 4 Earthquake Actions" Draft Joint Australian/New Zealand Standard - Draft Revision to AS1170.4 and NZS4203: 1992, March 2002

M Priestley J N (1985): "Seismic Behaviour of Unreinforced Masonry Walls" Bulletin of the NZNSEE, Vol. 18 (2) and Discussion Vol. 19 (1), 1986.

Yim C , Chopra K and Penzien J (1980): "Rocking Response of Rigid Blocks to Earthquakes". Earthquake Engineering and Structural Dynamics, Vol 8.

Appendix A – Drain2dx Input File for Modelling Australian Test Specimen 13

```
!UNITS L m F kn    [Inserted by RAM XLinea]
*STARTXX              ! wall, t=98/h=1.53M, effective E=1/50
  SP13B50R          0 0 1 1            URM wall - mid-height crack above mid-height
*NODECOORDS                            ! O/W = 0.0 Kevin Doherty test spec 13
C        41     0.00        1.485
C         8     0.00        1.415
C         7     0.00        1.275
C         6     0.00        1.135
C         5     0.00        0.995
C         4     0.00        0.733
C         3     0.00        0.524
C         2     0.00        0.314
C         1     0.00        0.105
!
C        20    -1.049       0.0
C        21    -0.049      -0.001
C        22    -0.049       0.0
C        23     0.0         0.0
C        24     0.049      -0.001
C        25     0.049       0.0
C        26     1.049       0.0
!
C        30    -0.049       0.889
C        31    -0.049       0.891
C        32     0.00        0.889
C        33     0.00        0.891     +.002
C        34     0.049       0.889
C        35     0.049       0.891
C        40    -2.000       1.500
*RESTRAINTS
S 221        20        26        6                 ! floor supports for Displ inputs
!S 111        40                                    ! rigid shear wall if used
S 211        40                                    ! rigid shear wall if used  for Displ
inputs
S 011        21        24        3
*SLAVING
S 100        22        21
S 100        25        24
S 100        32        33
*MASSES
S 110     0.258         1        4        1                   9.806       .60
S 110     0.325         5        8        1                   9.806       .60
S 110     0.097        32
S 110     0.065        33
!
S 001    0.00180        1        4        1                   9.806       .60
S 001    0.00012       32
S 001    0.00068        5        8        1                   9.806       .60
S 001    0.00007       33
*ELEMENTGROUP
    2     0     1        0.0040             1 ** WALL ELEMENTS
    1     0     1
```

```
     1     4.0E4      0.01      .104     .00011     4    4    2                 0.1
     1    1    1000.0     1000.0
     1         23         1              1          1    1
     2          1         2         1    1          1    1
     4          3         4              1          1    1
     5          4        32              1          1    1
     6         33         5              1          1    1
     7          5         6         1    1          1    1
     9          7         8              1          1    1
    10          8        41              1          1    1
*ELEMENTGROUP
     9    1    0            0.0                    2 ** LIFTOFF ELEMENTS
     1
     1   -2        0.8        0.9    0.34E4     0.34E4     0.34E4               0.1
     1         21        22         1    1
     2         24        25         1    1
     3         30        31         1    1
     4         34        35         1    1

*ELEMENTGROUP
     2    0    0         0.0040                    3 ** END BLOCK ELEMENTS
     1    0    1
     1     1.0E6      0.01      10.0      3.00      4    4    2                 0.1
     1    1   10000.0   10000.0
     1         22        23              1          1    1
     2         23        25              1          1    1
     3         30        32              1          1    1
     4         32        34              1          1    1
     5         31        33              1          1    1
     6         33        35              1          1    1
*ELEMENTGROUP
     9    1    0         0.0040                    4 ** LATERAL BASE SUPPORTS
     1
     1   -1        0.8        0.9    10.0E6     10.0E6     10.0E6               0.1
     1         20        22         1    1
     2         25        26         1    1

*ELEMENTGROUP
! Damping for rigid diaphragm
     1    0    0         0.0040                  5 ** TOP SUPPORT
     1                                                          1
! Stiffness for rigid diaphragm
     1   200.0E6      0.01   3.00E-5     1.0e7      1.0e7     0     0.010
     1         40        41              1
*RESULTS
NSD     10                                       ! all nodes for Post Processor
NSV     10                                       ! ditto nodal velocities
NSA     10                                       ! ditto nodal accelerations
NSD     11         32                            ! nodes also for .out file
E       10                                       ! all elements for Post Processor
!E       11    1    5    6    1                   ! elements also for .out file
*NODALOAD
  D+S                                  DEAD LOADS + O/W = 0.0
S       0.0     -0.083                41                     ! surcharge above support
S       0.0     -0.386                 1          4          1
S       0.0     -0.097                32
```

```
S       0.0    -0.258                    5         8         1
S       0.0    -0.065                   33
*ACCNREC
 PULS     PULSE2.DAT            (10f8.0)1 G PULSE
 1001   10    0    2                9.806      0.02       0.0
*ACCNREC
 AP80      PAC80.TXT    (f10.0,50x,f10.0) 80% PAC DAM -Kevin's Fig F50
 1501    1    1    2                9.806                 0.0
*DISPREC
 DTP8      PAC80.TXT        (f10.0,f10.0) 80% PAC DAM top Frame Displ -Kevin's Fig F50
 1501    1    1    1                0.001                 0.0
*DISPREC
 DBP8      PAC80.TXT    (f10.0,10x,f10.0) 80% PAC DAM Bott Table Displ -Kevin's Fig F50
 1501    1    1    1                0.001                 0.0
*PARAMETERS
VS                0.80       0.80          !Damping scaling
C                             1.0          !Collapse  Y Displ
F                       1.0                !Overshoot scaling
OS                   1                     !GRAV and STAT to .Out file every step
OD        0                   3      0.02    3      0.02              9999
DC  1               10                     !Event Calc/Max substeps
DT                0.01       0.01          !Initial/Max Var Time step
DA      .05                                !Force Tolerances For Var time Step
*GRAV                                      Gravity Load
N       D+S        1.0
*STAT
N      PULL       1.00
!L      0.01
D        32         23     1     0.0005     0.100     ! Disp control -1st 10mm
!*MODE                                      First 5 mode shapes & periods
!    5               1
! ************************* ACCEL OPTION ***********************************
!*ACCN
! 15.0      99999    2                      !Anal Time/Steps + Step Type
!1      PULS      0.300
!       AP80      1.000
! *********************** DISPL OPTION ***********************************
!*DISN
! 15.0      99999    2                        !Anal Time/Steps + Step Type
!R      DTP8                  1.00       0.0   ' Displ record No 1 at top of wall
!R      DBP8                  1.00       0.0   ' Displ record No 2 at bott of wall
!D   1    1       1.0         40                     ' horiz for nodes 40 at top wall
!D   2    1       1.0         20         26         6  ' horiz for nodes 20 & 26 at bott wall
*STOP
```

Appendix B – Methodology for The Assessment of Cracked Face-Loaded Walls and Parapets

B1 Description of Assessment Methodology

This appendix sets out a methodology that can be used to assess the seismic stability of a face-loaded URM wall element using the procedures developed in the body of this report and in previous research carried out by the author on face-loaded URM.

Design charts are provided for use where the assessment is being carried out in terms of the Basic Seismic Hazard Spectral intensity earthquake included in the current draft revision of the NZS4203 Loading Standard.

The assessment is carried out on face-loaded wall elements, which may be a single storey wall element supported top and bottom by a floor or roof diaphragms or horizontal strengthening members, a freestanding cantilever wall element or a parapet. Separate sets of design charts are given for single storey wall elements and for freestanding cantilever and parapet wall elements.

One of the ground motions used as input for the computer models, that were used to evaluate the behaviour of the face loaded wall elements in this study, was scaled to match the Basic Seismic Hazard Spectra for shallow soils given in the 2000 draft revision of NZS4203 (i.e. DR 0092: 2000). However, as this report was approaching final draft status, another draft revision of NZS4203 was released (i.e. DR 1170.4/PPC3: 2002). The Basic Seismic Hazard Spectra given in this latest draft revision are significantly different from those proposed in the 2000 draft as can be seen from Figure B1. The design charts given in this appendix were developed using the spectra extracted from the latter 2002 draft revision of NZS4203 and are referred to as the DR1170 spectral intensity earthquake in this appendix.

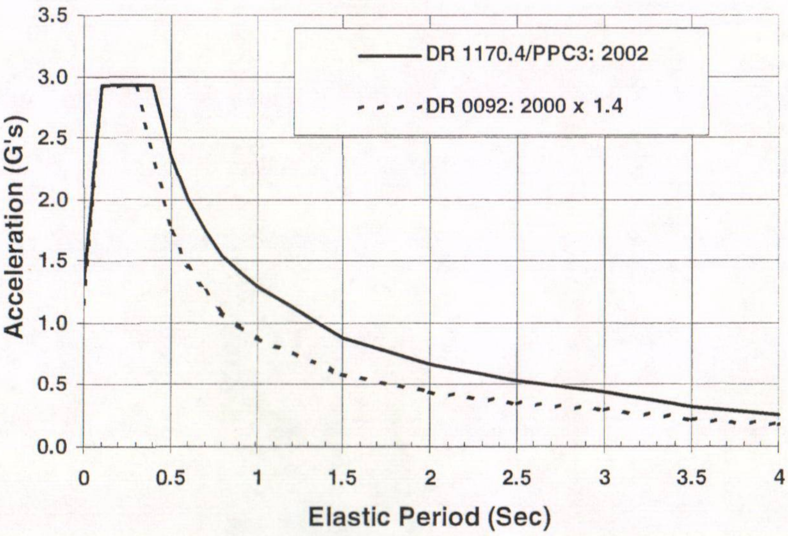


Figure B1: Basic Seismic Hazard Spectra Given by two Recent Draft Revisions to NZS4203 for Shallow Soil (Type C) Sites

The design charts give the EQ scaling factor, $I_{collapse}$, which must be applied to the basic DR1170 spectral intensity earthquake to cause collapse of a face-loaded wall element supported on the ground or supported by a rigid structure with rigid diaphragms. (Note that the methodology can be adapted for other earthquake records or code design spectral intensities by using equation Eqn 20 to calculate $I_{collapse}$).

The charts were derived using Eqn 20 and the DR1170 Basic Hazard Spectra for 3 soil foundation types (i.e. rock -Type A and B, shallow - Type C, and soft or deep - Type D). The acceleration spectrum for a Type C soil foundation is shown in Figure B1. The corresponding displacement spectra for the 3 soil types, that is also required for application of the methodology, were derived from the acceleration spectra using the procedure given in section 5.1.

The effective thickness of the wall, t, was assumed to be given by: $t = t_{nom}(0.975 - 0.025\frac{O}{W})$

where t_{nom} is the nominal thickness of the wall. Other assumptions regarding the elastic modulus and density of the masonry were the same as given in section 5.2 of this report.
To allow for the effect of top and bottom fixity, a modification factor is applied to the basic EQ scaling factor, $I_{collapse}$, obtained from the charts. The resulting DR1170 EQ scaling factor, $I_{capacity}$, represents the seismic collapse capacity of the face-loaded wall element expressed as a proportion of the DR1170 basic hazard spectral intensity.

$I_{capacity}$ can then be compared with the scaling factor, I_{demand}, which must be exceeded to meet the assessment criteria. When the assessment is being carried out in terms the of DR1170, I_{demand} will be dependant on the seismic zone factor Z, the structural performance factor, S_p and an appropriate risk factor, R. It will also need to include a modification factor to allow for amplification of the earthquake motion due to the building response and diaphragm flexibility or yielding. If an allowance for near fault effects is to made in the assessment, the near-fault factor N(T, D) given in DR1170 would need to be adjusted. The analyses on face loaded wall elements in this study indicate that the assessment methodology is non-conservative for near-fault earthquake motions particularly when the combined building and diaphragm periods exceeds 1.0 seconds. The adjustment to the DR1170 near-fault factor N(T,D) would, therefore, need to allow for both the higher spectral intensity expected for longer period structures and the less conservative nature of the methodology when the earthquake motion may contain long duration pulses.

B2 Assessment Methodology

The formulation that follows applies to assessments of face-loaded URM walls elements where the assessment is carried out in terms of DR 1170/PPC3. The assessment can be carried out for one of the 3 principal types of site soil foundation conditions covered by the proposed Standard.

The seismic collapse capacity of a face-loaded wall element, expressed as a proportion of the DR1170 basic hazard spectral intensity, is given by:

$$\mathbf{I}_{\text{capacity}} = \mathbf{F}_{\text{fixity}}\,\mathbf{I}_{\text{collapse}} \qquad \textbf{Eqn B1}$$

Where: $\mathbf{I}_{\text{collapse}}$ = the EQ scaling factor that, when applied to the basic DR1170 spectral intensity earthquake, will result in an EQ intensity that has a low probability of causing a face-loaded wall element to collapse. Single-storey wall elements and parapets are assumed at this stage of the methodology to be supported by a rigid structure with rigid diaphragms and to have the reference conditions of fixity at the top and bottom of the wall element. I_{collapse} may be read from design charts provided in this appendix or calculated using Eqn 20. A set of design charts is provided for single storey wall elements (supported top and bottom by floor or roof diaphragms, the ground or by horizontal strengthening members). A separate set of charts is provided for freestanding cantilever and parapet wall elements.

$\mathbf{F}_{\text{fixity}}$ = top and bottom fixity modifier, equals 1.0 when the reference conditions of fixity used to construct the design charts are satisfied (i.e. full fixity at bottom of the wall element, ($B_b = 1.0$), and no fixity at the top of the wall element ($B_t = 0.0$)).

Otherwise:

$$\mathbf{F}_{\text{fixity}} = \frac{\left((1+\mathbf{B}_b)+(2+\mathbf{B}_b+\mathbf{B}_t)\dfrac{\mathbf{O}}{\mathbf{W}}\right)}{2\left(1+1.5\dfrac{\mathbf{O}}{\mathbf{W}}\right)} \quad \text{for Single-Storey wall elements} \qquad \textbf{Eqn B2(a)}$$

or

$$\mathbf{F}_{\text{fixity}} = \frac{\left(\mathbf{B}_b+(\mathbf{B}_b+\mathbf{B}_t)\dfrac{\mathbf{O}}{\mathbf{W}}\right)}{\left(1+\dfrac{\mathbf{O}}{\mathbf{W}}\right)} \quad \text{for freestanding walls \& parapets} \qquad \textbf{Eqn B2(b)}$$

OPUS

Where: **O** = the overburden weight acting on the wall at the top of the wall element being assessed.

W = the weight of the wall element being assessed.

B_t and B_b = Top and bottom fixity factors for the wall element as defined in Figure 1 and Figure 4 in the body of this report.

Where the top and/or bottom wall fixity is asymmetric about the centreline of the wall element, the fixity factor computed will depend on whether the mid-height (or top) of the wall element is displaced towards the interior or towards the exterior of the building. In this case it is recommended that the lowest fixity factor computed for the two directions should be used for the assessment.

$I_{capacity}$ = the assessed seismic collapse capacity of the face-loaded wall element expressed as a proportion of the DR1170 basic hazard spectral intensity where the wall element has only a low probability of having a lower capacity. If a collapse capacity with a 50% probability of exceedance is required, a higher value of 1.2 x $I_{capacity}$ is recommended for making the assessment. Modelling carried out as part of this research project indicates that the 1.2 factor may be conservative in some cases and that a value of 1.9 for single-storey walls and a value of 1.65 for freestanding cantilevers and parapets may be more appropriate.

The assessed face-loaded wall element is satisfactory if:

$$I_{capacity} > A \; I_{demand} \qquad \textbf{Eqn B3}$$

Where: I_{demand} = $S_p RZ$ where the assessment is carried out in terms of. DR 1170/PPC3 without consideration of near-fault effects. I_{demand} is the minimum seismic capacity required for the assessment, expressed as a proportion of the DR1170 basic hazard spectral intensity.

(Note: S_p is the structural performance factor; **R** is the risk factor and **Z** is the Zone factor as given in the proposed Loading Standard. However, a value of S_p = 1.0 is probably more appropriate as the methodology has been formulated to predict the collapse limit state instead of the ultimate limit state. The risk factor, R, will also need to be adjusted so that it is appropriate for the assessment of an existing building.

A = the storey elevation amplification factor which allows for the effects of building and diaphragm flexibility and is given in Table B1.

Table B1: Storey Elevation Amplification Factors for Wall Element within one Storey and Parapets

Wall Element Type	Storey Elevation Amplification Factor (A)	
	Single Storey Building	Multi-Storey Building
• **Wall Element within one Storey**		
(floor/roof diaphragms rigid, flexible or yielding)		
- Building considered rigid* – first storey	1.2	1.2
- Building considered rigid* – other storey	NA	1.4
- Building period < 0.5 seconds **	1.4	$A = 0.7(1+3\frac{h_i}{h_r})$
- Building period >1.0 seconds **	1.4	$A = 0.7(1+2\frac{h_i}{h_r})$
• **Parapets**		
(floor/roof diaphragms rigid flexible or yielding)		
- Building period < 0.5 seconds **	2.0	3.0
- Building period >1.0 seconds **	2.0	2.0

Notes:

* Buildings where the shear walls (and their foundations) can be considered rigid (e.g. long boundary walls on firm ground with period < 0.1 seconds and expected to respond elastically)

** Period should allow for inelastic deformations and ignore diaphragm flexibility – linear interpolation is proposed for building periods between 0.5 and 1.0 seconds.

h_i is the mid-storey height of the face loaded wall in storey being assessed and:

h_r is the elevation of the building roof (parapets are assumed to be located near roof level)

Design Charts

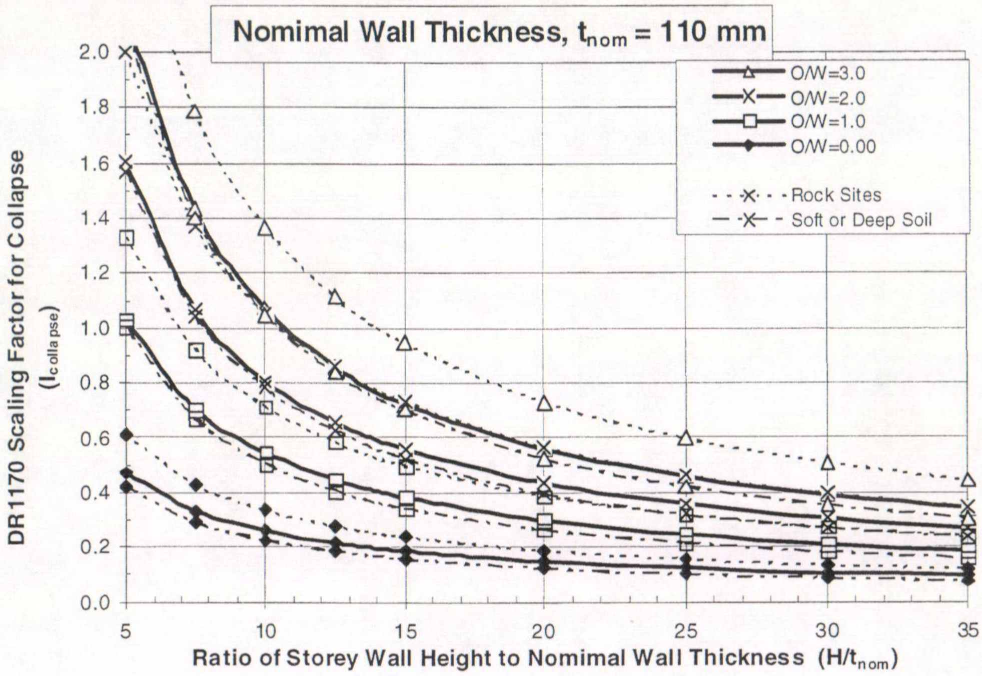
For

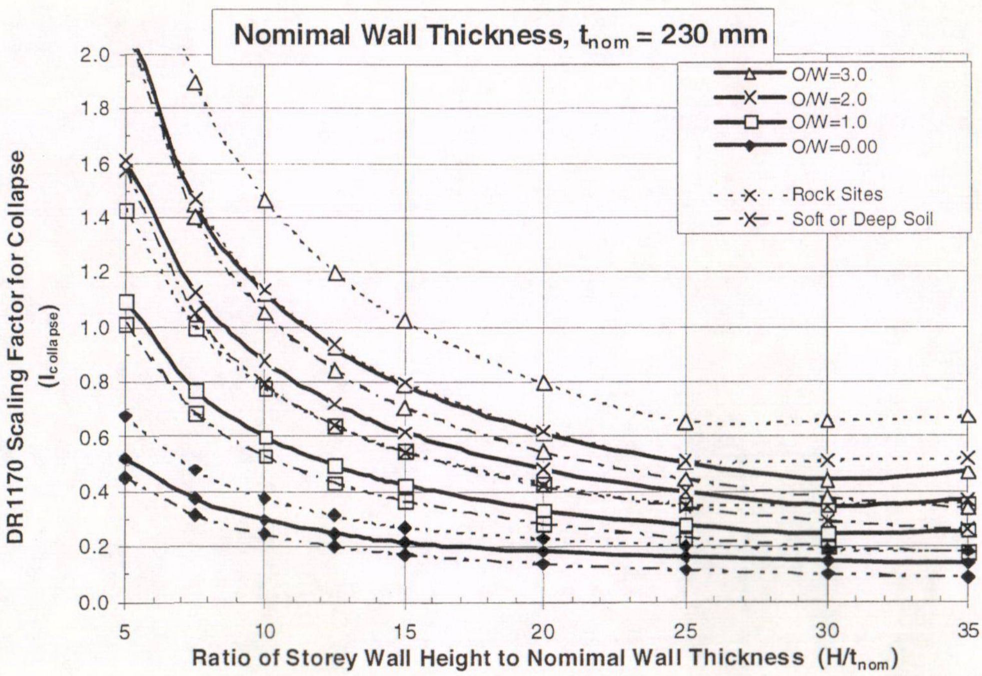
Single-storey Wall Elements

(Wall elements supported top and bottom by rigid floor diaphragms and with full fixity at base of element (i.e. $B_b = 1.0$) and no fixity at top (i.e. $B_t = 0.0$))

EQ Scaling Factor, $I_{collapse}$, to be Applied to DR1170/PPC3 Spectral EQ Intensity to Cause Collapse of Face Loaded Single-storey Walls - Rigid Blding & Diaphragms

(Solid Line Plots are for Shallow Soil (type C) sites - Comparative Capacities for Rock (Type A or B) and Soft or Deep Soil Sites (Type D) Shown dotted and dashed respectively)





Note: W = weight (per m) of the wall within the storey under consideration
O = Load (per m) imposed on the wall at the top of the storey under consideration
Plots apply for Bottom fixity, B_b = 1.0 and Top Fixity B_t = 0.0

EQ Scaling Factor, $I_{collapse}$, to be Applied to DR1170/PPC3 Spectral EQ Intensity to Cause Collapse of Face Loaded Single-storey Walls - Rigid Blding & Diaphragms

Nominal URM Wall Thickness, t_{nom} = 110 mm

$\frac{O}{W}$	DR1170 Soil Spectrum	Ratio Storey Wall Height to Nominal Wall Thickness (h / t_{nom})								
		5	7.5	10	12.5	15	20	25	30	35
0.00	Rock	0.61	0.43	0.34	0.28	0.24	0.19	0.16	0.13	0.12
	Shallow	0.47	0.33	0.26	0.22	0.19	0.15	0.12	0.11	0.10
	Soft/Deep	0.42	0.29	0.23	0.18	0.16	0.12	0.10	0.09	0.08
1.00	Rock	1.33	0.92	0.71	0.58	0.50	0.39	0.32	0.27	0.24
	Shallow	1.03	0.70	0.55	0.45	0.38	0.30	0.25	0.21	0.19
	Soft/Deep	1.01	0.67	0.51	0.41	0.34	0.26	0.22	0.18	0.16
2.00	Rock	2.00	1.37	1.05	0.86	0.73	0.57	0.46	0.40	0.35
	Shallow	1.60	1.07	0.80	0.66	0.56	0.43	0.36	0.31	0.27
	Soft/Deep	1.57	1.04	0.79	0.63	0.52	0.40	0.32	0.27	0.24
3.00	Rock	2.66	1.79	1.37	1.11	0.95	0.73	0.60	0.51	0.44
	Shallow	2.14	1.43	1.07	0.86	0.72	0.56	0.46	0.39	0.34
	Soft/Deep	2.09	1.40	1.05	0.84	0.70	0.53	0.42	0.35	0.31

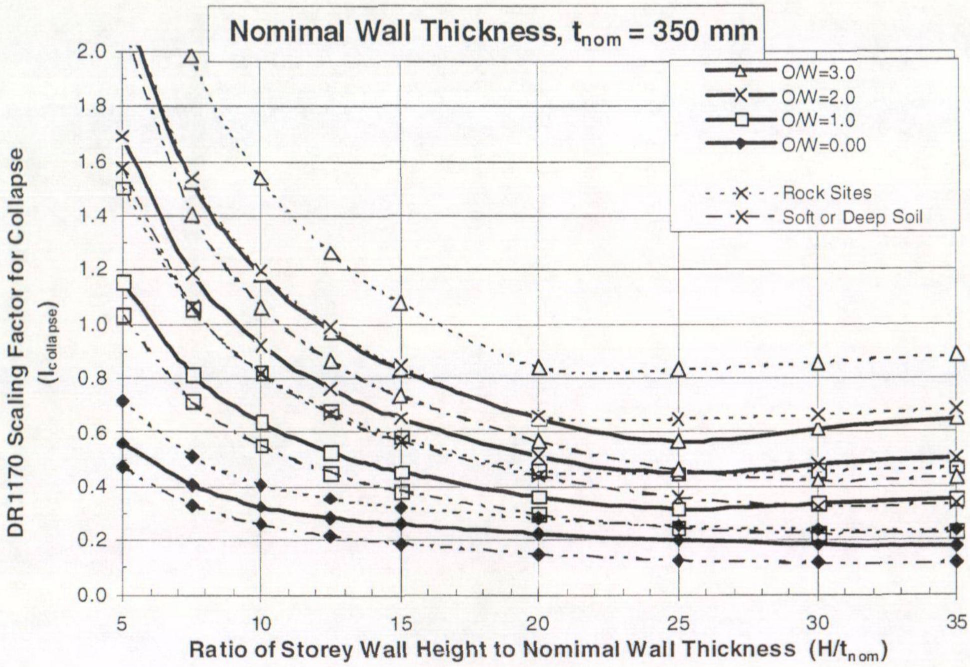
Nominal URM Wall Thickness, t_{nom} = 230 mm

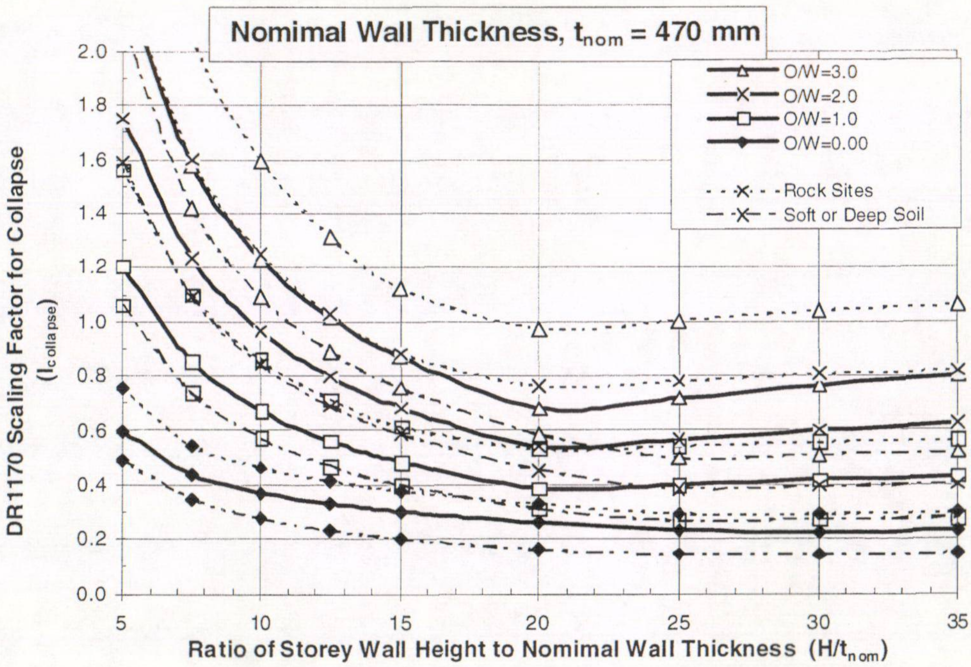
$\frac{O}{W}$	DR1170 Soil Spectrum	Ratio Storey Wall Height to Nominal Wall Thickness (h / t_{nom})								
		5	7.5	10	12.5	15	20	25	30	35
0.00	Rock	0.68	0.48	0.38	0.31	0.27	0.23	0.20	0.19	0.18
	Shallow	0.52	0.37	0.30	0.25	0.22	0.18	0.16	0.15	0.14
	Soft/Deep	0.45	0.32	0.25	0.20	0.17	0.14	0.12	0.10	0.09
1.00	Rock	1.43	1.00	0.78	0.64	0.55	0.43	0.36	0.35	0.36
	Shallow	1.10	0.77	0.60	0.50	0.42	0.33	0.28	0.25	0.26
	Soft/Deep	1.01	0.69	0.53	0.43	0.37	0.28	0.23	0.20	0.18
2.00	Rock	2.12	1.47	1.14	0.93	0.80	0.62	0.51	0.51	0.52
	Shallow	1.61	1.13	0.87	0.72	0.62	0.48	0.40	0.35	0.37
	Soft/Deep	1.57	1.05	0.79	0.64	0.54	0.42	0.34	0.29	0.26
3.00	Rock	2.74	1.90	1.47	1.20	1.02	0.80	0.66	0.66	0.67
	Shallow	2.15	1.45	1.12	0.92	0.79	0.61	0.51	0.44	0.47
	Soft/Deep	2.10	1.40	1.05	0.85	0.71	0.54	0.44	0.38	0.34

Note: W = weight (per m) of the wall within the storey under consideration
O = Load (per m) imposed on the wall at the top of the storey under consideration
Charts apply for Bottom fixity, B_b = 1.0 and Top Fixity B_t = 0.0

EQ Scaling Factor, $I_{collapse}$, to be Applied to DR1170/PPC3 Spectral EQ Intensity to Cause Collapse of Face Loaded Single-storey Walls - Rigid Blding & Diaphragms

(Solid Line Plots are for Shallow Soil (type C) sites - Comparative Capacities for Rock (Type A or B) and Soft or Deep Soil Sites (Type D) Shown dotted and dashed respectively)





Note: W = weight (per m) of the wall within the storey under consideration
O = Load (per m) imposed on the wall at the top of the storey under consideration
Plots apply for Bottom fixity, B_b = 1.0 and Top Fixity B_t = 0.0

EQ Scaling Factor, $I_{collapse}$, to be Applied to DR1170/PPC3 Spectral EQ Intensity to Cause Collapse of Face Loaded Single-storey Walls - Rigid Blding & Diaphragms

Nominal URM Wall Thickness, t_{nom} = 350 mm

O/W	DR1170 Soil Spectrum	Ratio Storey Wall Height to Nominal Wall Thickness (h / t_{nom})								
		5	7.5	10	12.5	15	20	25	30	35
0.00	Rock	0.72	0.51	0.41	0.36	0.32	0.28	0.25	0.24	0.24
	Shallow	0.56	0.41	0.33	0.29	0.26	0.23	0.20	0.18	0.18
	Soft/Deep	0.47	0.33	0.26	0.22	0.19	0.15	0.13	0.12	0.12
1.00	Rock	1.50	1.06	0.82	0.68	0.58	0.46	0.45	0.46	0.47
	Shallow	1.16	0.82	0.64	0.53	0.46	0.36	0.32	0.34	0.35
	Soft/Deep	1.03	0.72	0.55	0.45	0.39	0.30	0.25	0.22	0.23
2.00	Rock	2.21	1.54	1.20	0.99	0.84	0.66	0.65	0.66	0.69
	Shallow	1.69	1.19	0.92	0.76	0.65	0.51	0.45	0.48	0.51
	Soft/Deep	1.57	1.06	0.82	0.67	0.57	0.44	0.36	0.33	0.34
3.00	Rock	2.85	1.98	1.54	1.26	1.08	0.84	0.83	0.86	0.89
	Shallow	2.18	1.52	1.18	0.97	0.83	0.65	0.57	0.61	0.65
	Soft/Deep	2.10	1.41	1.06	0.87	0.73	0.57	0.46	0.42	0.44

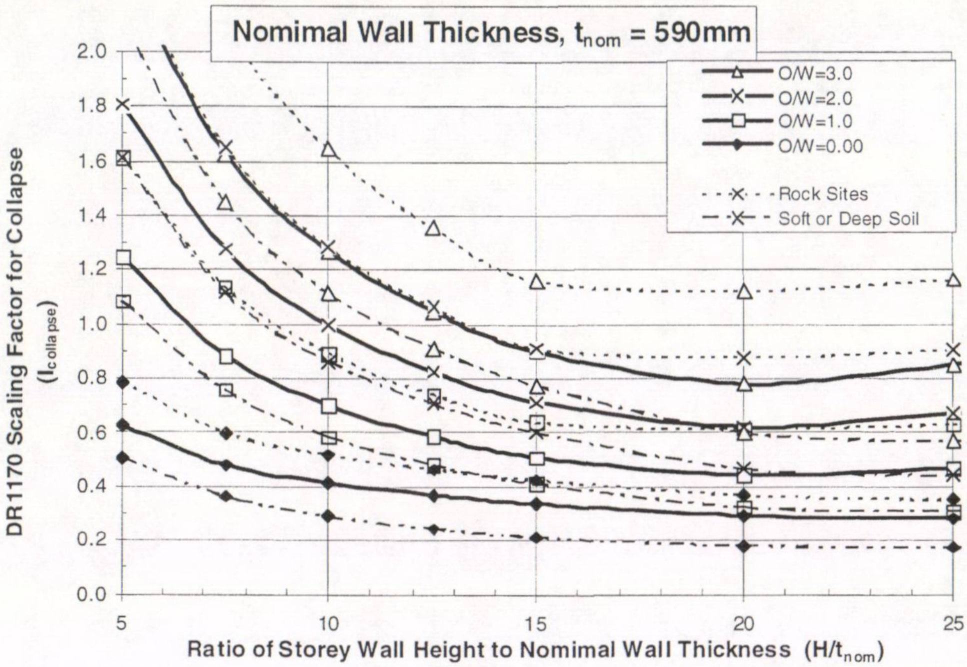
Nominal URM Wall Thickness, t_{nom} = 470 mm

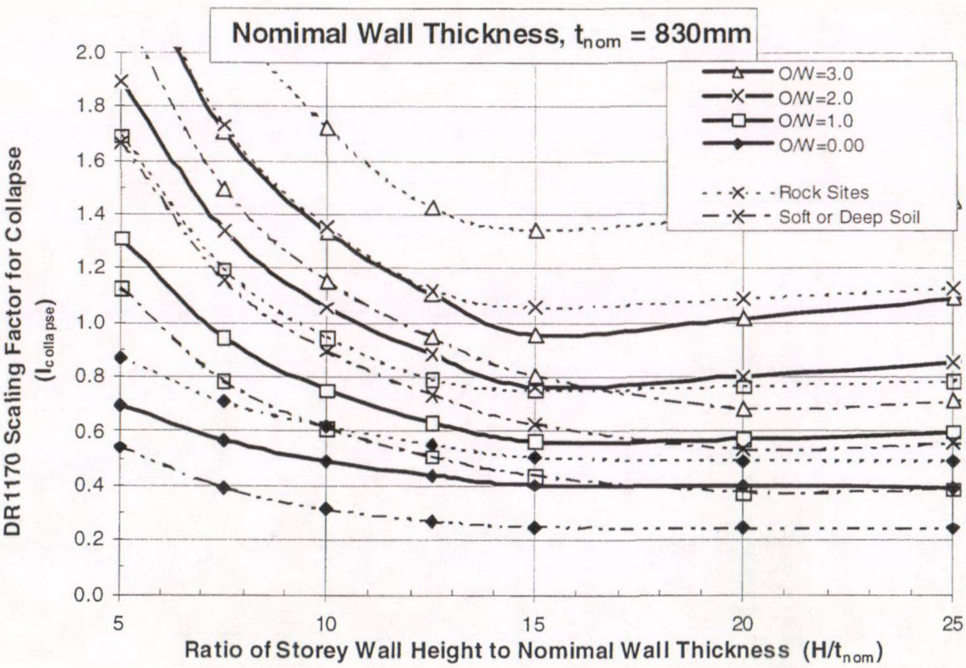
O/W	DR1170 Soil Spectrum	Ratio Storey Wall Height to Nominal Wall Thickness (h / t_{nom})								
		5	7.5	10	12.5	15	20	25	30	35
0.00	Rock	0.75	0.54	0.46	0.41	0.38	0.33	0.29	0.29	0.30
	Shallow	0.60	0.44	0.37	0.33	0.30	0.26	0.23	0.22	0.23
	Soft/Deep	0.49	0.35	0.28	0.23	0.20	0.16	0.14	0.14	0.15
1.00	Rock	1.56	1.10	0.86	0.71	0.61	0.53	0.54	0.56	0.56
	Shallow	1.21	0.85	0.67	0.56	0.48	0.39	0.40	0.42	0.43
	Soft/Deep	1.06	0.74	0.57	0.47	0.40	0.31	0.27	0.27	0.28
2.00	Rock	2.28	1.60	1.25	1.03	0.88	0.76	0.78	0.81	0.82
	Shallow	1.75	1.23	0.96	0.80	0.68	0.54	0.57	0.60	0.62
	Soft/Deep	1.59	1.09	0.84	0.69	0.59	0.45	0.39	0.40	0.40
3.00	Rock	2.94	2.05	1.60	1.31	1.12	0.97	1.00	1.04	1.06
	Shallow	2.25	1.58	1.23	1.02	0.87	0.68	0.72	0.77	0.80
	Soft/Deep	2.10	1.42	1.09	0.89	0.76	0.59	0.50	0.51	0.52

Note: W = weight (per m) of the wall within the storey under consideration
O = Load (per m) imposed on the wall at the top of the storey under consideration
Charts apply for Bottom fixity, B_b = 1.0 and Top Fixity B_t = 0.0

EQ Scaling Factor, $I_{collapse}$, to be Applied to DR1170/PPC3 Spectral EQ Intensity to Cause Collapse of Face Loaded Single-storey Walls - Rigid Blding & Diaphragms

(Solid Line Plots are for Shallow Soil (type C) sites - Comparative Capacities for Rock (Type A or B) and Soft or Deep Soil Sites (Type D) Shown dotted and dashed respectively)





Note: W = weight (per m) of the wall within the storey under consideration
O = Load (per m) imposed on the wall at the top of the storey under consideration
Plots apply for Bottom fixity, B_b = 1.0 and Top Fixity B_t = 0.0

EQ Scaling Factor, $I_{collapse}$, to be Applied to DR1170/PPC3 Spectral EQ Intensity to Cause Collapse of Face Loaded Single-storey Walls - Rigid Blding & Diaphragms

Nominal URM Wall Thickness, t_{nom} = 590 mm

O / W	DR1170 Soil Spectrum	Ratio Storey Wall Height to Nominal Wall Thickness (H / t_{nom}) 5	7.5	10	12.5	15	20	25
0.00	Rock	0.79	0.60	0.52	0.46	0.43	0.37	0.35
	Shallow	0.63	0.48	0.41	0.37	0.34	0.29	0.28
	Soft/Deep	0.51	0.36	0.29	0.24	0.21	0.18	0.17
1.00	Rock	1.61	1.14	0.89	0.74	0.64	0.61	0.63
	Shallow	1.25	0.88	0.70	0.59	0.51	0.44	0.47
	Soft/Deep	1.09	0.76	0.59	0.48	0.41	0.32	0.31
2.00	Rock	2.34	1.65	1.29	1.06	0.91	0.88	0.91
	Shallow	1.81	1.27	1.00	0.83	0.71	0.62	0.67
	Soft/Deep	1.61	1.12	0.86	0.71	0.60	0.47	0.44
3.00	Rock	3.01	2.11	1.64	1.36	1.16	1.12	1.16
	Shallow	2.31	1.63	1.27	1.05	0.90	0.78	0.85
	Soft/Deep	2.11	1.45	1.11	0.91	0.78	0.60	0.57

Nominal URM Wall Thickness, t_{nom} = 830 mm

O / W	DR1170 Soil Spectrum	Ratio Storey Wall Height to Nominal Wall Thickness (H / t_{nom}) 5	7.5	10	12.5	15	20	25
0.00	Rock	0.87	0.71	0.62	0.55	0.50	0.49	0.49
	Shallow	0.70	0.57	0.49	0.44	0.40	0.40	0.39
	Soft/Deep	0.54	0.39	0.31	0.27	0.25	0.25	0.24
1.00	Rock	1.69	1.20	0.95	0.79	0.75	0.77	0.79
	Shallow	1.31	0.94	0.75	0.63	0.56	0.57	0.60
	Soft/Deep	1.13	0.79	0.61	0.51	0.44	0.38	0.39
2.00	Rock	2.45	1.73	1.35	1.12	1.06	1.09	1.13
	Shallow	1.89	1.34	1.06	0.88	0.76	0.80	0.85
	Soft/Deep	1.66	1.16	0.90	0.73	0.63	0.53	0.56
3.00	Rock	3.14	2.21	1.72	1.43	1.34	1.39	1.45
	Shallow	2.42	1.71	1.33	1.11	0.95	1.02	1.09
	Soft/Deep	2.16	1.49	1.15	0.95	0.80	0.68	0.71

Note: W = weight (per m) of the wall within the storey under consideration
O = Load (per m) imposed on the wall at the top of the storey under consideration
Charts apply for Bottom fixity, $B_b = 1.0$ and Top Fixity $B_t = 0.0$

Design Charts

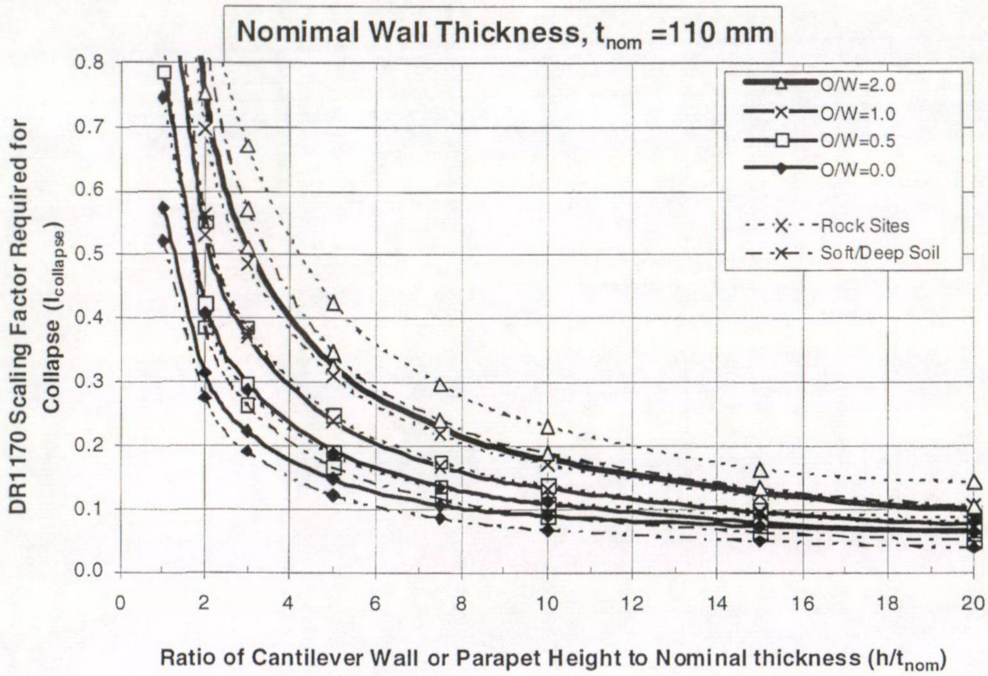
For

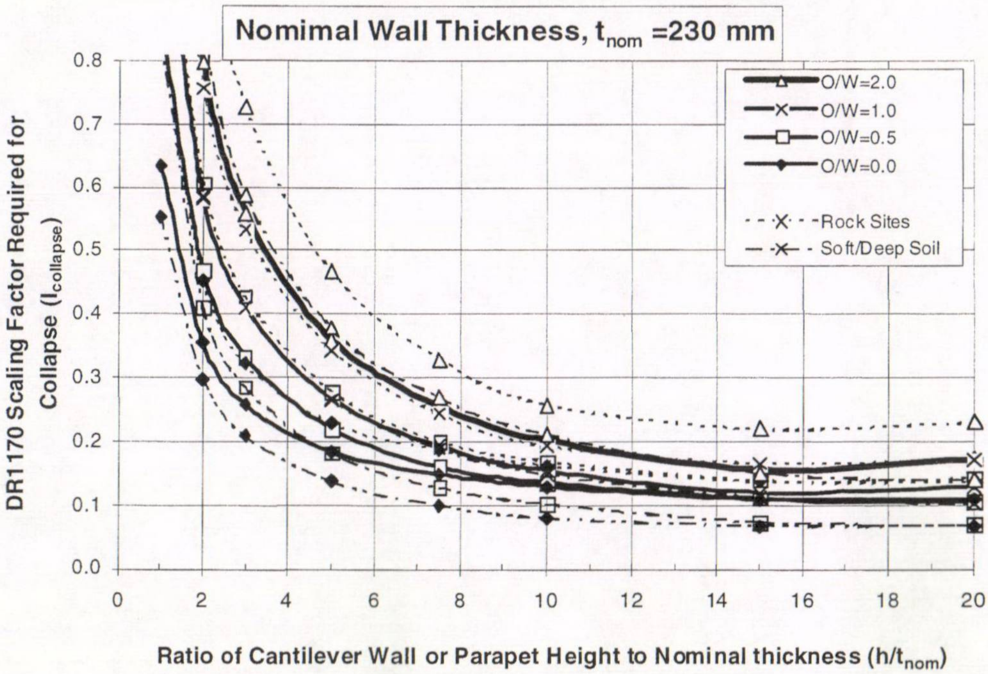
Freestanding Cantilever Wall Elements and Parapets

(Wall elements supported bottom and free to displace at the top with full fixity at base of element (i.e. B_b = 1.0) and no fixity at top (i.e. B_t = 0.0))

EQ Scaling Factor, $I_{collapse}$, to be Applied to DR1170/PPC3 Spectral EQ Intensity to Cause Collapse of Cantilever Walls & Parapets Supported on a Rigid Building

(Solid Line Plots are for Shallow Soil (type C) sites - Comparative Capacities for Rock (Type A or B) and Soft or Deep Soil Sites (Type D) Shown dotted and dashed respectively)





Note: W = weight (per m) of the cantilever wall or parapet
O = Load (per m) imposed at the top of the cantilever wall or parapet under consideration
Plots apply for full Bottom fixity, $B_b = 1.0$ and no Top Fixity, $B_t = 0.0$

EQ Scaling Factor, $I_{collapse}$, to be Applied to DR1170/PPC3 Spectral EQ Intensity to Cause Collapse of Cantilever Walls & Parapets Supported on a Rigid Building

Nominal URM Wall Thickness, t_{nom} = 110 mm

$\frac{O}{W}$	DR1170 Soil Spectrum	Ratio Storey Wall Height to Nominal Wall Thickness (h / t_{nom})								
		1.0	2.0	3.0	5.0	7.5	10.0	15.0	20.0	25.0
0.0	Rock	0.75	0.41	0.29	0.19	0.13	0.11	0.09	0.08	0.07
	Shallow	0.57	0.31	0.22	0.15	0.11	0.09	0.07	0.06	0.06
	Soft/Deep	0.52	0.28	0.19	0.12	0.09	0.07	0.05	0.04	0.03
0.5	Rock	1.03	0.55	0.39	0.25	0.18	0.14	0.10	0.09	0.09
	Shallow	0.79	0.42	0.30	0.19	0.14	0.11	0.08	0.07	0.07
	Soft/Deep	0.82	0.39	0.26	0.17	0.12	0.09	0.06	0.05	0.04
1.0	Rock	1.30	0.70	0.48	0.31	0.22	0.17	0.12	0.11	0.11
	Shallow	1.03	0.53	0.37	0.24	0.17	0.13	0.10	0.08	0.08
	Soft/Deep	1.12	0.56	0.38	0.24	0.17	0.13	0.09	0.07	0.06
2.0	Rock	1.87	0.97	0.67	0.42	0.30	0.23	0.16	0.14	0.15
	Shallow	1.50	0.75	0.51	0.33	0.23	0.18	0.13	0.10	0.11
	Soft/Deep	1.70	0.85	0.57	0.35	0.24	0.19	0.13	0.10	0.09

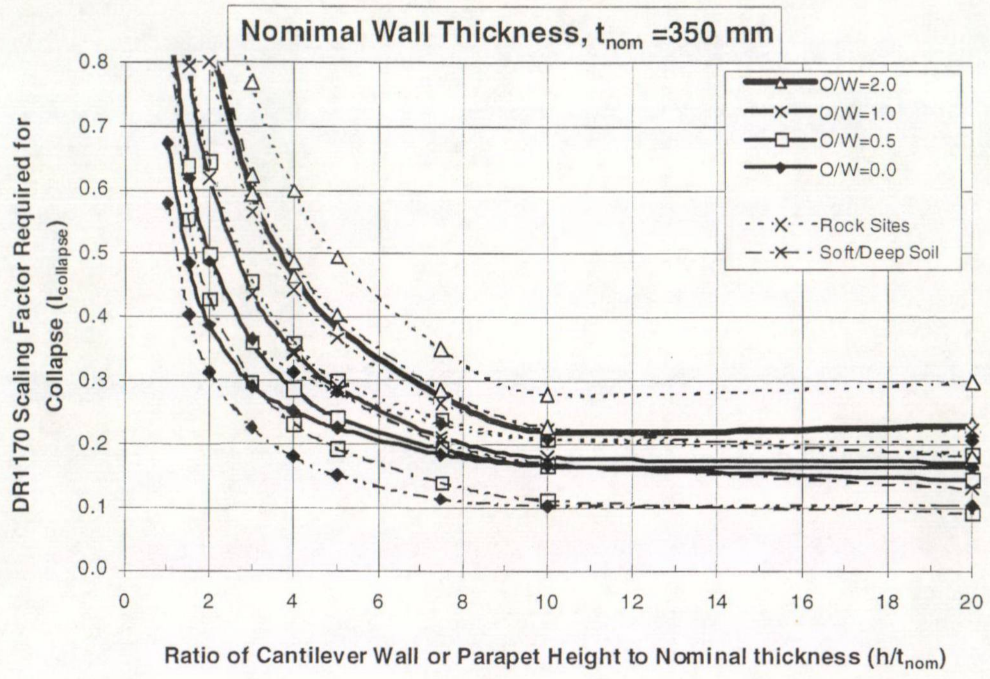
Nominal URM Wall Thickness, t_{nom} = 230 mm

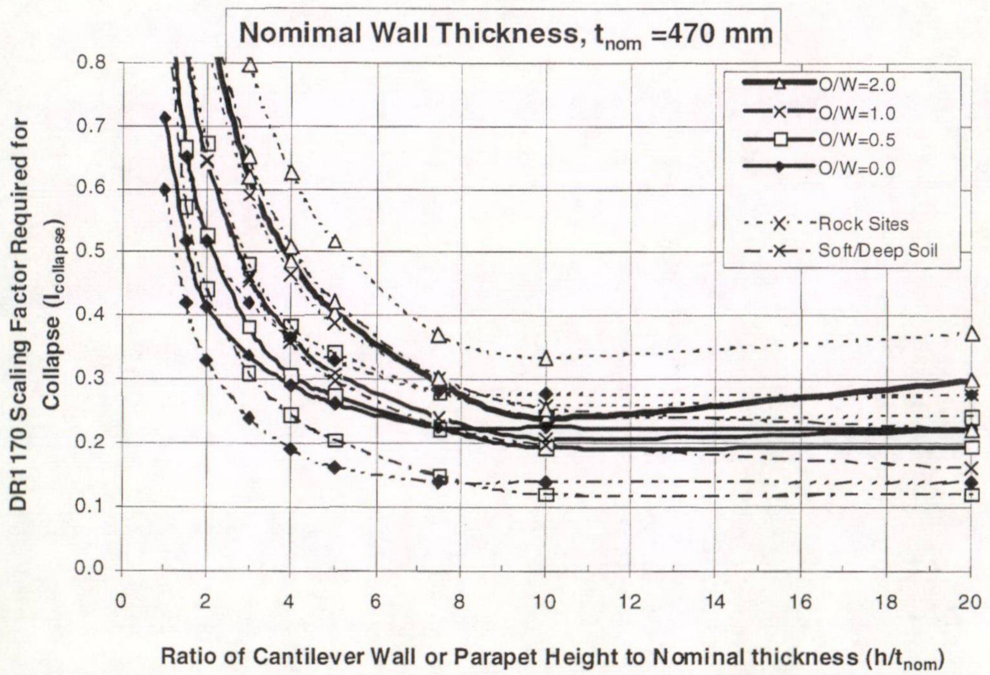
$\frac{O}{W}$	DR1170 Soil Spectrum	Ratio Storey Wall Height to Nominal Wall Thickness (h / t_{nom})								
		1.0	2.0	3.0	5.0	7.5	10.0	15.0	20.0	25.0
0.00	Rock	0.82	0.45	0.32	0.23	0.19	0.16	0.14	0.14	0.14
	Shallow	0.63	0.35	0.26	0.18	0.15	0.13	0.11	0.11	0.11
	Soft/Deep	0.55	0.30	0.21	0.14	0.10	0.08	0.07	0.07	0.07
0.50	Rock	1.11	0.61	0.43	0.28	0.20	0.17	0.14	0.14	0.14
	Shallow	0.85	0.47	0.33	0.22	0.16	0.14	0.11	0.11	0.11
	Soft/Deep	0.83	0.41	0.29	0.18	0.13	0.10	0.07	0.07	0.07
1.00	Rock	1.40	0.76	0.53	0.34	0.24	0.19	0.17	0.17	0.17
	Shallow	1.07	0.58	0.41	0.27	0.19	0.15	0.12	0.13	0.13
	Soft/Deep	1.12	0.59	0.41	0.27	0.19	0.15	0.11	0.10	0.10
2.00	Rock	1.94	1.04	0.73	0.47	0.33	0.26	0.22	0.23	0.24
	Shallow	1.50	0.80	0.56	0.36	0.26	0.20	0.16	0.17	0.18
	Soft/Deep	1.71	0.86	0.59	0.38	0.27	0.21	0.15	0.14	0.14

Note: W = weight (per m) of the cantilever wall or parapet
O = Load (per m) imposed at the top of the cantilever wall or parapet under consideration
Charts apply for full Bottom fixity, B_b = 1.0 and no Top Fixity, B_t = 0.0

EQ Scaling Factor, $I_{collapse}$, to be Applied to DR1170/PPC3 Spectral EQ Intensity to Cause Collapse of Cantilever Walls & Parapets Supported on a Rigid Building

(Solid Line Plots are for Shallow Soil (type C) sites - Comparative Capacities for Rock (Type A or B) and Soft or Deep Soil Sites (Type D) Shown dotted and dashed respectively)





Note: W = weight (per m) of the cantilever wall or parapet
O = Load (per m) imposed at the top of the cantilever wall or parapet under consideration
Plots apply for full Bottom fixity, B_b = 1.0 and no Top Fixity, B_t = 0.0

EQ Scaling Factor, $I_{collapse}$, to be Applied to DR1170/PPC3 Spectral EQ Intensity to Cause Collapse of Cantilever Walls & Parapets Supported on a Rigid Building

Nominal URM Wall Thickness, t_{nom} = 350 mm

O / W	DR1170 Soil Spectrum	Ratio Storey Wall Height to Nominal Wall Thickness (h / t_{nom}) 1	1.5	2	3	4	5	7.5	10	20
0.00	Rock	0.87	0.62	0.49	0.36	0.31	0.28	0.23	0.21	0.21
	Shallow	0.67	0.49	0.39	0.29	0.25	0.23	0.18	0.17	0.16
	Soft/Deep	0.58	0.40	0.31	0.23	0.18	0.15	0.11	0.10	0.10
0.50	Rock	1.17	0.82	0.64	0.46	0.36	0.30	0.24	0.21	0.19
	Shallow	0.90	0.64	0.50	0.36	0.29	0.24	0.19	0.17	0.14
	Soft/Deep	0.87	0.55	0.43	0.30	0.23	0.19	0.14	0.11	0.09
1.00	Rock	1.46	1.03	0.80	0.57	0.44	0.37	0.26	0.22	0.22
	Shallow	1.12	0.79	0.62	0.44	0.35	0.29	0.21	0.18	0.17
	Soft/Deep	1.14	0.79	0.62	0.44	0.34	0.28	0.20	0.17	0.13
2.00	Rock	2.02	1.41	1.09	0.77	0.60	0.50	0.35	0.28	0.30
	Shallow	1.54	1.08	0.84	0.59	0.46	0.38	0.28	0.22	0.23
	Soft/Deep	1.71	1.15	0.88	0.62	0.49	0.40	0.29	0.23	0.18

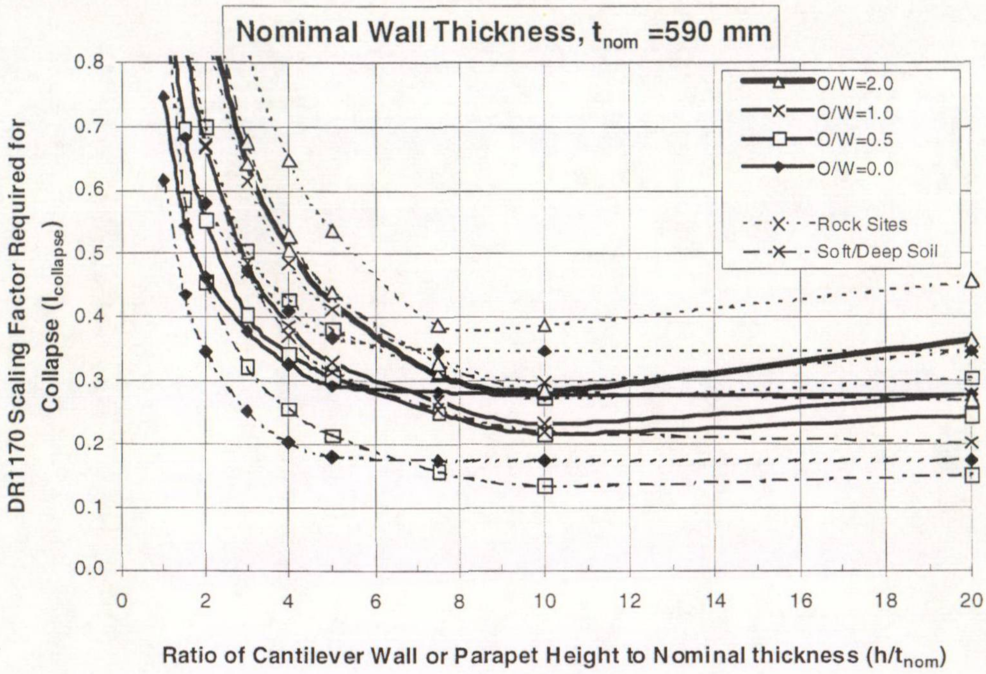
Nominal URM Wall Thickness, t_{nom} = 470 mm

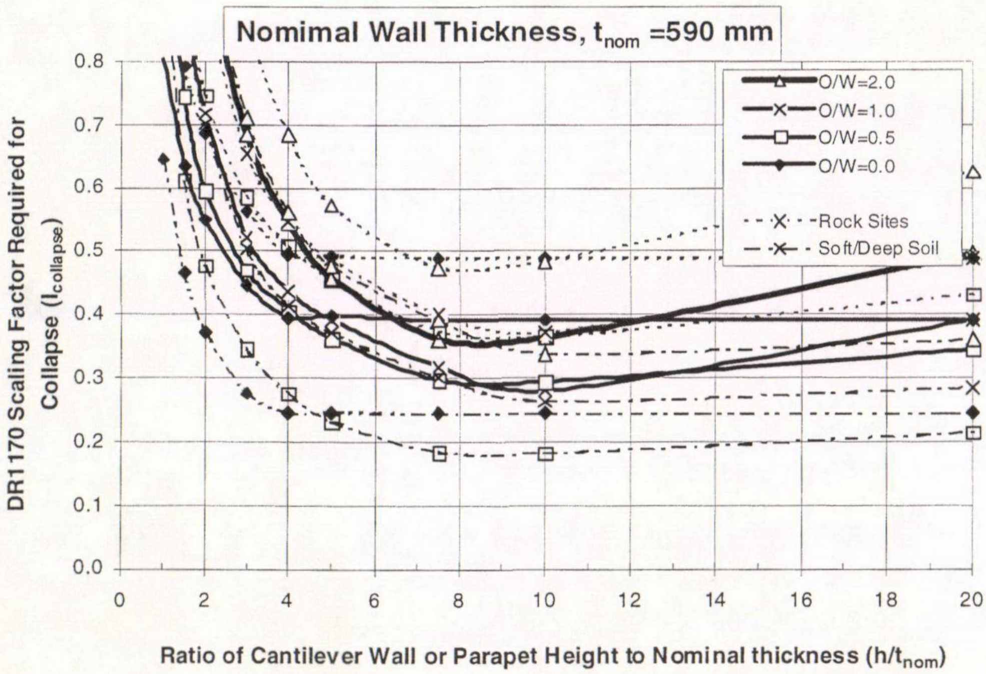
O / W	DR1170 Soil Spectrum	Ratio Storey Wall Height to Nominal Wall Thickness (h / t_{nom}) 1	1.5	2	3	4	5	7.5	10	20
0.00	Rock	0.91	0.65	0.52	0.42	0.37	0.33	0.28	0.28	0.28
	Shallow	0.71	0.52	0.41	0.34	0.29	0.26	0.22	0.22	0.22
	Soft/Deep	0.60	0.42	0.33	0.24	0.19	0.16	0.14	0.14	0.14
0.50	Rock	1.22	0.86	0.67	0.48	0.38	0.34	0.28	0.24	0.24
	Shallow	0.94	0.67	0.53	0.38	0.31	0.27	0.22	0.19	0.19
	Soft/Deep	0.90	0.57	0.44	0.31	0.24	0.20	0.15	0.12	0.12
1.00	Rock	1.52	1.07	0.83	0.59	0.46	0.39	0.30	0.26	0.28
	Shallow	1.17	0.83	0.65	0.46	0.37	0.31	0.24	0.21	0.22
	Soft/Deep	1.17	0.82	0.64	0.46	0.36	0.30	0.23	0.20	0.16
2.00	Rock	2.09	1.46	1.13	0.80	0.63	0.52	0.37	0.34	0.37
	Shallow	1.60	1.12	0.88	0.62	0.49	0.41	0.29	0.24	0.30
	Soft/Deep	1.72	1.18	0.92	0.65	0.51	0.42	0.30	0.25	0.22

Note: W = weight (per m) of the cantilever wall or parapet
O = Load (per m) imposed at the top of the cantilever wall or parapet under consideration
Charts apply for full Bottom fixity, B_b = 1.0 and no Top Fixity, B_t = 0.0

EQ Scaling Factor, $I_{collapse}$, to be Applied to DR1170/PPC3 Spectral EQ Intensity to Cause Collapse of Cantilever Walls & Parapets Supported on a Rigid Building

(Solid Line Plots are for Shallow Soil (type C) sites - Comparative Capacities for Rock (Type A or B) and Soft or Deep Soil Sites (Type D) Shown dotted and dashed respectively)





Note: W = weight (per m) of the cantilever wall or parapet
O = Load (per m) imposed at the top of the cantilever wall or parapet under consideration
Plots apply for full Bottom fixity, B_b = 1.0 and no Top Fixity, B_t = 0.0

EQ Scaling Factor, $I_{collapse}$, to be Applied to DR1170/PPC3 Spectral EQ Intensity to Cause Collapse of Cantilever Walls & Parapets Supported on a Rigid Building

Nominal URM Wall Thickness, t_{nom} = 590 mm

$\frac{O}{W}$	DR1170 Soil Spectrum	Ratio Storey Wall Height to Nomimal Wall Thickness (h / t_{nom})								
		1.0	1.5	2.0	3.0	4.0	5.0	7.5	10.0	20.0
0.0	Rock	0.94	0.68	0.58	0.47	0.41	0.37	0.35	0.35	0.35
	Shallow	0.75	0.54	0.46	0.38	0.33	0.29	0.28	0.28	0.28
	Soft/Deep	0.61	0.44	0.35	0.25	0.20	0.18	0.17	0.17	0.17
0.5	Rock	1.26	0.89	0.70	0.50	0.43	0.38	0.31	0.27	0.31
	Shallow	0.97	0.70	0.55	0.40	0.34	0.31	0.25	0.22	0.24
	Soft/Deep	0.93	0.59	0.46	0.32	0.26	0.21	0.16	0.13	0.15
1.0	Rock	1.56	1.10	0.86	0.61	0.48	0.41	0.34	0.30	0.35
	Shallow	1.21	0.85	0.67	0.48	0.39	0.33	0.27	0.23	0.28
	Soft/Deep	1.21	0.85	0.67	0.47	0.37	0.31	0.26	0.22	0.20
2.0	Rock	2.14	1.50	1.17	0.83	0.65	0.54	0.39	0.39	0.46
	Shallow	1.64	1.16	0.90	0.64	0.51	0.42	0.31	0.28	0.36
	Soft/Deep	1.74	1.22	0.95	0.67	0.53	0.44	0.33	0.29	0.27

Nominal URM Wall Thickness, t_{nom} = 830 mm

$\frac{O}{W}$	DR1170 Soil Spectrum	Ratio Storey Wall Height to Nomimal Wall Thickness (h / t_{nom})								
		1.0	1.5	2.0	3.0	4.0	5.0	7.5	10.0	20.0
0.00	Rock	1.01	0.79	0.69	0.56	0.50	0.49	0.49	0.49	0.49
	Shallow	0.80	0.63	0.55	0.45	0.40	0.40	0.39	0.39	0.39
	Soft/Deep	0.65	0.47	0.37	0.28	0.24	0.25	0.24	0.24	0.24
0.50	Rock	1.32	0.94	0.75	0.59	0.51	0.46	0.37	0.36	0.43
	Shallow	1.03	0.74	0.60	0.47	0.41	0.36	0.30	0.29	0.34
	Soft/Deep	0.97	0.61	0.48	0.35	0.28	0.23	0.18	0.18	0.21
1.00	Rock	1.64	1.16	0.91	0.65	0.55	0.49	0.40	0.37	0.49
	Shallow	1.27	0.90	0.72	0.52	0.44	0.39	0.32	0.28	0.39
	Soft/Deep	1.27	0.89	0.70	0.50	0.41	0.37	0.30	0.26	0.29
2.00	Rock	2.23	1.57	1.23	0.87	0.69	0.57	0.47	0.48	0.62
	Shallow	1.72	1.22	0.95	0.68	0.54	0.46	0.36	0.36	0.50
	Soft/Deep	1.81	1.28	1.00	0.71	0.56	0.48	0.39	0.34	0.36

Note: W = weight (per m) of the cantilever wall or parapet
O = Load (per m) imposed at the top of the cantilever wall or parapet under consideration
Charts apply for full Bottom fixity, B_b = 1.0 and no Top Fixity, B_t = 0.0

OPUS

OPUS

INTERNATIONAL
CONSULTANTS