Methodology for the seismic assessment of face-loaded unreinforced masonry walls and parapets

E.L. Blaikie & R.A. Davey

Opus International Consultants Ltd, Wellington, New Zealand

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ABSTRACT: This paper outlines the development of a methodology that can be used to predict the seismic stability of a cracked, face-loaded unreinforced masonry (URM) wall.

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 the results of inelastic dynamic analyses of computer models of face-loaded URM wall elements. Good agreement was obtained when this type of modelling was used to predict the displacement time-history of a test specimen.

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.

1 INTRODUCTION

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

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

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

This methodology has been incorporated, in modified form, into the New Zealand Society of Earthquake Engineering draft procedures for assessing and improving the structural performance of earthquake risk buildings (NZSEE 2002).

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

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

This paper outlines the derivation of the assessment methodology, summarises the results of the calibration analyses and outlines how the methodology can be used to develop charts which can be used for the rapid seismic assessment of face-loaded URM wall elements.

2 STATIC BEHAVIOUR OF FACE-LOADED URM WALLS

Face loaded walls in URM buildings typically span vertically between floor framing. They may also be supported by roof framing, the ground or horizontal members provided to strengthen the wall. 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.

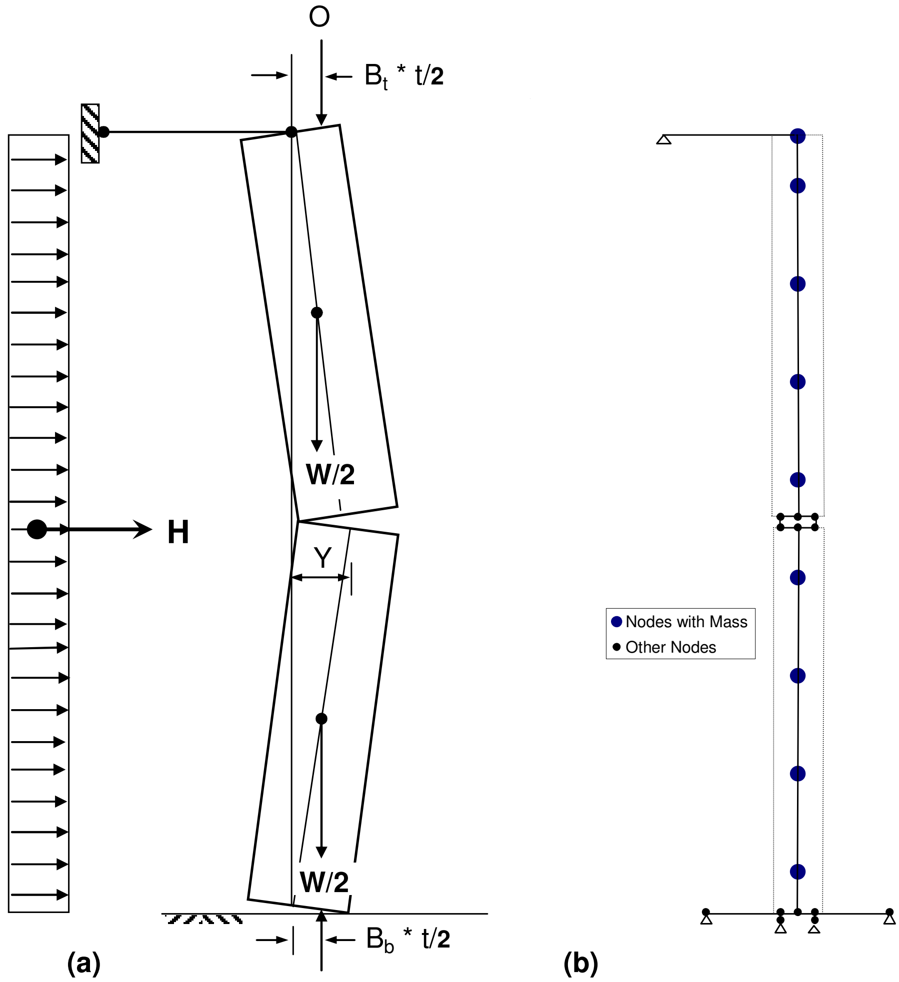

Figure 1: (a) Behaviour of Face Loaded Single-Storey Wall Element under Static Loading and (b) Diagram of the Computer Model used to Analyse a Similar Wall Element (B_t = 0.0)

Figure 1a shows the forces assumed to act on a cracked single storey 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. This overburden 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.

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.

The lower half of Figure 1a may also be thought of as describing a parapet or cantilever wall. In this case an overburden load, O, acting at the top of the wall element with a positive sign would act on the left hand side of the wall centreline, the wall weight would be W instead of W/2, and the wall height, h, would be the total height of the parapet.

A computer model that could be used to analyse the cracked single-storey wall element is also show in Figure 1b and is discussed further in section 4.1.

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

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

Where t_{nom} = the nominal wall thickness.

The equations describing the behaviour of face-loaded wall elements under static loading can be derived by considering equilibrium of the wall elements assuming they are rigid and are given as follows:

$$Y_{max} = t \frac{\left(1 + K_1 \frac{O}{W}\right)}{\left(1 + \frac{2O}{W}\right)} * F_{fixity} \tag{2}$$

Where Y_{max} is the wall displacement at which the wall element becomes unstable and K_1 is given in Table 1. The top and bottom fixity factor, F_{fixity} = 1.0 for a 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:

$$F_{fixity} = \frac{\left((K_2 + B_b) + (K_3 + B_b + B_t)\frac{O}{W}\right)}{K_4\left(1 + K_1\frac{O}{W}\right)} \tag{3}$$

Where K_1 to K_5 are given in Table 1.

Table 1 Equation constants for the two wall element types

Type of Wall Element	K_1	K_2	K_3	K_4	K_5
Single storey wall element	1.5	1.0	2.0	2.0	1.0
Parapet of free standing wall	1.0	0.0	0.0	1.0	4.0

When the uniformly distributed horizontal load, H, shown in Figure 1 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:

$$C_d = \frac{H_{max}}{W} = \frac{4t}{K_5 h}\left(1 + K_1 \frac{O}{W}\right) * F_{fixity} \tag{4}$$

where H_{max} = the maximum value of H that occurs when the wall cracks just start to open (i.e. when Y = 0.0); K_1 and K_5 are given in Table 1.

If the wall element was released from the displaced position shown in Figure 1 the period of the wall free vibration expected, when the peak displacement is $0.6Y_{max}$, would be given by:

$$T = \sqrt{\frac{0.7 K_5 h}{\left(1 + 2\frac{O}{W}\right)}} \tag{5}$$

where h = the wall element height in meters.

This relationship was developed by considering the conservation of the potential and kinetic energy of the vibrating wall element and studying the free vibration response of wall element computer models (Blaikie Spurr 1992). The free vibration period increases towards infinity as the wall displacement approaches Y_{max} and the period when the peak displacement is $0.6Y_{max}$, given by equation (5), is taken as representative of a wall approaching collapse.

3 METHODOLOGY USED TO PREDICT FACE-LOADED WALL ELEMENT STABILITY

A methodology was developed to predict the seismic stability of face-loaded wall elements by observing the behaviour a wide range of URM wall elements subjected to time-history dynamic analyses.

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

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

$$Y = (T / 2\pi)^2 A \tag{6}$$

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

Figure 2 shows the acceleration response spectrum given by draft design code AS/NZS 1170.4 and the pseudo displacement spectrum derived from this acceleration spectrum.

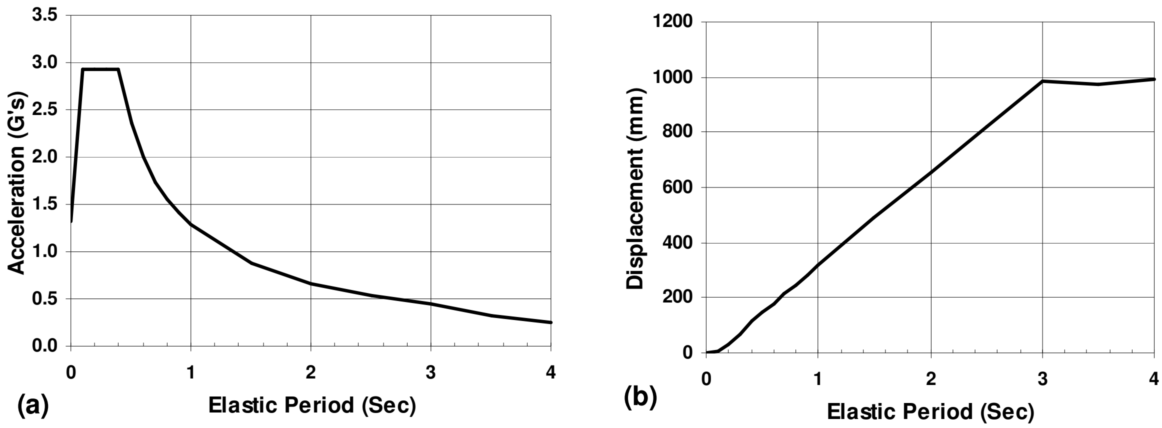

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Figure 2. (a) Acceleration response spectrum given by the draft design code AS/NZS 1170.4. (b) Pseudo displacement spectrum derived from this acceleration spectrum.

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

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

$$\textit{either} \quad I_{capacity} = I_{sp} \quad \text{when } I_{sp} \geq 2.5 I_{cr} \tag{7}$$

$$\textit{or} \quad I_{capacity} = \frac{I_{sp} + 2.5 I_{cr}}{2} \quad \text{when } I_{sp} < 2.5 I_{cr} \tag{8}$$

where I_{sp} corresponds to the earthquake intensity that will generate wall displacements equal to the displacement at which the wall element becomes unstable and is calculated using the displacement spectrum procedure given in section 3.1; I_{cr} corresponds to the earthquake intensity that will just open the joint cracks in the wall element and is calculated using the acceleration spectrum procedure given in section 3.2.

When evaluating $I_{capacity}$, single-storey wall elements and parapets are assumed to be supported by a rigid structure with rigid diaphragms. When the wall element is supported on a flexible structure the shaking intensity that the wall element will be subjected to will be amplified . Amplification factors, A, to be used in the methodology are given in Table 2.

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

The methodology detailed above has been calibrated so that it results in an assessed earthquake spectral intensity with a low probability of causing of causing collapse rather than the expected collapse intensity. This makes it more useful as an assessment/design tool (see Section 4.2). This has been achieved by adjusting the constants such as the 2.5 factor in equation (7), the 0.6 and 1.2 factors in equation (9). These factors were adjusted so that the methodology predicted the results of the time history computer modelling of wall elements conservatively and so that the scatter in the difference between the seismic capacity predicted by the methodology and the computer modelling would be reduced to a minimum. The amplification factors given in Table 2 were deduced from computer modelling with parameters changed so that the effects of building amplification on face loaded wall element stability could be evaluated.

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

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

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

3.1 Evaluation of I_{sp} using a Displacement Spectrum

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

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 equation (2)).

2. Use a displacement response spectrum for the earthquake motion (e.g. Figure 2(b)) to evaluate the maximum displacement expected for a SDOF structure with the period, T calculated in

Step 1. The maximum displacement, Y_{sp}, 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 is the normalised modal participation factor of 1.5, applicable if the cracked wall components behave as rigid blocks, are slender and respond elastically.

When evaluating the expected displacement from the displacement response spectrum, the spectral displacement is assumed to generally increase with increasing period, T and any local "dips" in the displacement response spectra are ignored.

3. The wall element will become unstable when the maximum wall displacement is Y_{max}. The earthquake intensity required to generate a maximum wall displacement of Y_{max} is taken as 1.2 times the earthquake intensity required to generate a wall displacement of $0.6Y_{max}$. Therefore , the earthquake spectral intensity scaling factor, I_{sp} that will cause the wall element to collapse, predicted using the displacement spectrum, is given by:

$$I_{sp} = 1.2\left(\frac{0.6\ Y_{max}}{Y_{sp}}\right) \quad (9)$$

where Y_{max} is given by equation (2); Y_{sp} is evaluated as in step 2 above

3.2 Evaluation of I_{cr} using an Acceleration Spectrum

The following steps are required to calculate the scaling factor, I_{cr} using an acceleration spectrum. When the earthquake intensity, represented by the spectrum is scaled by this factor, the procedure predicts that the joint cracks in the 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. An effective elastic modulus of 1.0GPa should be used for the masonry. As tests (Blaikie, 2002) indicate that a large amount of wall curvature occurs adjacent to the cracks in the wall, this value of the elastic modulus may not be conservative so that any initial rising branch of the spectra should be ignored.
2. Use an acceleration response spectrum for the earthquake motion (e.g. Figure 2b) to evaluate the maximum response , C_{sp}, 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 (using equation (4)).
4. The earthquake scaling factor, I_{cr}, which must be applied to the EQ motion so that it would be just sufficient to open the wall element joints is then given by.

$$I_{cr} = C_d / C_{sp} \quad (10)$$

3.3 Using Design Charts to Evaluate $I_{capacity}$

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. AS/NZS 1170.4/PPC8) have been prepared (Blaikie 2002, Appendix B). These charts allow the $I_{capacity}$ scale factor (equations (7) & (8)) to be rapidly evaluated for a wide range of wall thickness, wall element types and boundary conditions. Similar design charts could be prepared for other code design spectral intensities using the proposed methodology outlined above.

4 COMPARISON BETWEEN COMPUTER MODELLING AND METHODOLOGY

4.1 Computer Modelling of Face-loaded Wall Elements

Figure 1b shows diagrammatically an example of the type of computer model that was used to analysis face-loaded wall elements. The model allows the wall to deform as indicated in Figure 1a with link members, that can only carry compressive forces, accommodating opening of the cracks at the mid-height and base of the wall.

Models analysed included a 3 storey wall, single storey walls with and without parapets and free-standing walls. Diaphragms and shear walls supporting the face-loaded wall elements were included in some models to evaluate building amplification effects. The models were analysed using the inelastic dynamic analysis program Ram Xlinea which incorporates DRAIN-2DX. The program uses time step-by-step numeric integration to perform the inelastic time-history analysis.

4.2 Comparison between Computer Modelling and Methodology

The earthquake scaling factors that are expected to cause a range of face-loaded wall elements to collapse, as predicted by the computer modelling and as predicted by the proposed methodology, are compared in Figure 3.

In this case earthquake spectra derived from the range of input time-histories used for the computer models were used in the methodology.

The comparison indicates that the proposed methodology has been calibrated so that it predicts the seismic capacity of face-loaded URM wall elements corresponding to a low probability of failure.

Figure 3 also shows straight (dotted) lines plotted so that 50% of the results lie above and below the lines. These plots indicate that the mean or expected collapse intensity is significantly greater than that predicted using the methodology.

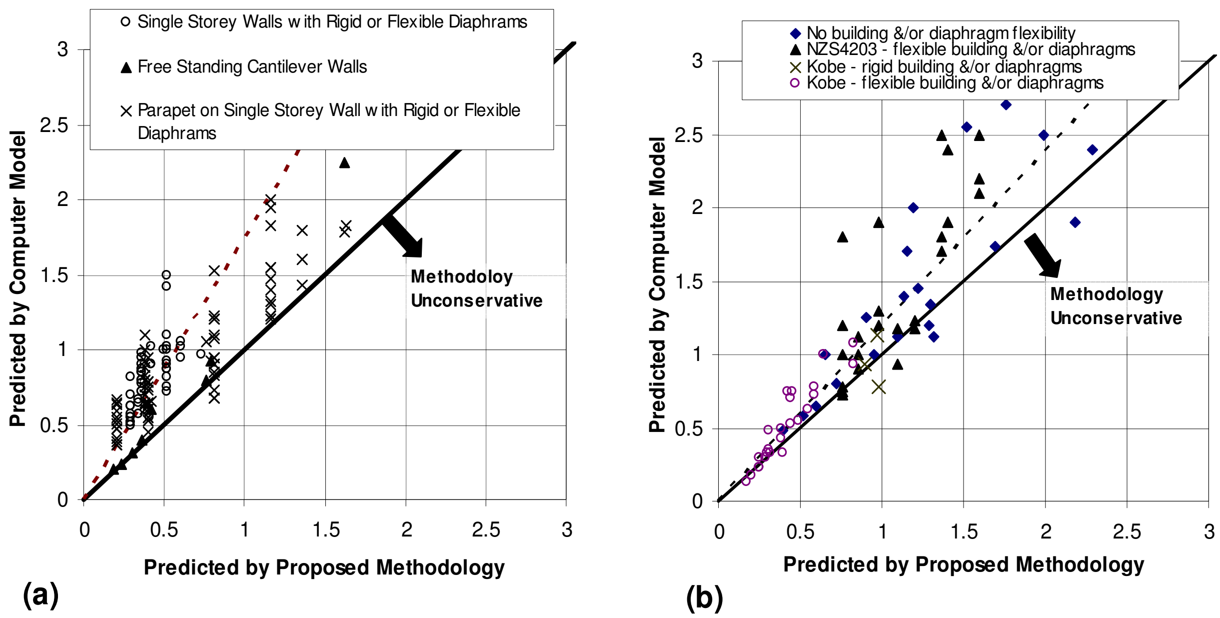

Figure 3. Earthquake Scaling Factors Required to Cause Collapse of Face-Loaded wall elements – Comparison of the Values Predicted using the Computer Modelling and those Predicted using the Proposed Methodology (a) (Blaikie 2002) (b) Single storey type wall elements only (Blaikie 2000)

However, the data in Figure 3a excludes the results obtained when the input earthquake motion used in the computer modelling had a long acceleration pulse that is characteristic of some near fault earthquake motions. In this case, the methodology tends to predict the mean seismic resistance indicated by the computer modelling. These analyses indicate that the methodology is less conservative for this type of earthquake motion and that, for near fault type earthquake motions, an additional safety factor of 1.5 would be required for the methodology to predict the seismic capacity of face-loaded URM wall elements corresponding to a low probability of failure.

5 COMPUTER MODELLING OF AUSTRALIAN SHAKING TABLE TEST SPECIMEN

5.1 Comparison of Computer Model and Test Specimen

Figure 4 shows the mid-height wall displacement recorded when a face-loaded single storey wall element was tested on a shake table (Doherty 2000). The shake table was subjected to 80% of the 1994 Pacoima Earthquake record.

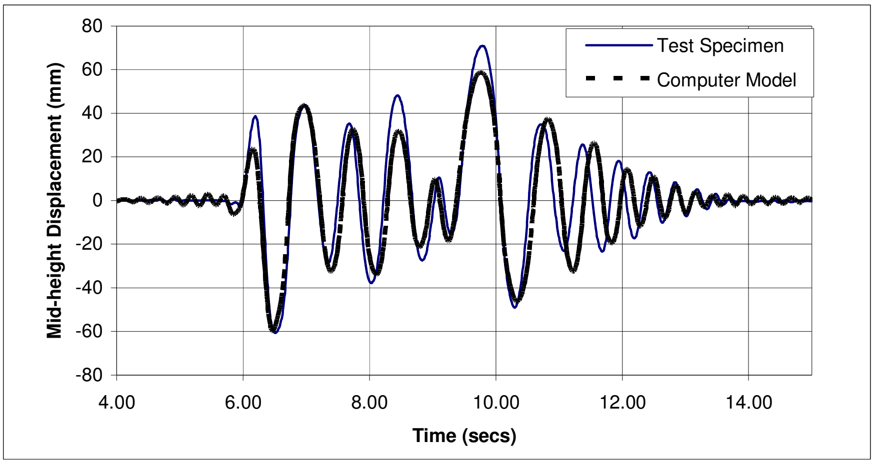

Figure 4. Mid-height Wall Displacement of Test Specimen Subjected to 80% of Pacoima EQ Record and Comparison with Response Predicted by Computer Model

The response predicted by a 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 (Blaikie 2002). Agreement between the test specimen response and that predicted by the computer model can be seen to be excellent given that the analysis results were quite sensitive to small changes in the analysis input parameters.

As the methodology was developed and calibrated using computer models, the agreement demonstrated in Figure 4 should help to provide increased confidence in the proposed methodology.

6 CONCLUSIONS

This paper outlined the development of a methodology that can be used to predict the seismic stability of a cracked face-loaded URM multi-storey wall, single-storey walls, parapets or freestanding cantilever walls.

The earthquake spectral 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 a wall element.

Computer modelling was used to predict the behaviour of a pre-cracked face-loaded wall specimen tested under simulated seismic loading. Good agreement between the predicted and measured mid-height displacement of the test specimen was obtained which should give some confidence in the assessment methodology which was developed using similar computer models.

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