SEISMIC BEHAVIOUR OF FACE LOADED UNREINFORCED MASONRY WALLS

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SUMMARY

Inelastic dynamic analysis methods were used to model the seismic behaviour of a face-loaded URM wall. The wall was modelled as uncracked except for cracks at the diaphragm levels and mid-storey height. These cracks were free to open and close as the wall deflected under lateral loads. Parameters considered included, wall thickness, over-burden load, and the intensity and type of earthquake motion.

The model was used to develop a methodology that can be used to assess the stability of face-loaded URM walls. In this methodology, the effective period of the face loaded wall motion is computed using semi-empirical formulae. The effective period is then used in conjunction with an elastic displacement response spectrum to predict the earthquake magnitude that will cause the wall to collapse.

INTRODUCTION

In the past, many engineers assessed the seismic resistance of URM buildings by evaluating the earthquake intensity required to cause cracking. However, the tensile strength of masonry is highly variable and URM buildings are often already cracked due to stresses induced by settlement, temperature and moisture changes or due to previous earthquakes.

It has also been observed that many masonry buildings that are severely cracked are often still standing after earthquakes and that they are still able to resist strong aftershocks.

These observations led a consortium of Californian engineers, called the ABK Joint Venture (ABK, 1982), to carry out a pioneering investigation into the post-cracking seismic resistance of 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 near its base. During the test these cracks 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.

Draft guidelines for assessing and strengthening earthquake risk buildings have been published by the New Zealand National Society of Earthquake Engineering (NZNSEE, 1995). These include guidelines for the assessment of face loaded walls which are based on the equal energy method developed by Priestley (Priestley, 1985). This method depends on the initial elastic stiffness of the wall. For example, if the assumed elastic modulus of the masonry is increased by 100%, an increase in seismic resistance of approximately 40% is

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predicted. It appears that this increase is unlikely as the initial elastic deflection of the wall, prior to cracks opening in the wall, is very small compared with the wall displacement at which the wall becomes unstable.

Recent research in Australia on the behaviour of rocking masonry cantilever walls (Lam et al, 1995) has shown that the equal energy method predicted a seismic resistance that was 3.5 times greater than that indicated by shake table testing.

This paper describes the development of an alternative methodology that can be used to assess the seismic resistance of a face loaded URM wall. The methodology was developed as part of study funded by the EQC Research Foundation. The Methodology is described in more detail in the study report (Blaikie, Spurr, 1992).

BEHAVIOUR OF CRACKED URM FACE LOADED WALLS

Behaviour of Face-Loaded URM walls subjected to Static Loads

Face loaded walls in URM buildings normally span vertically between floor diaphragms. They may also be supported by roof diaphragms or by the ground. In many instances the walls also span horizontally between walls or returns. For the current investigation only vertically spanning walls are considered.

When subjected to sufficient lateral load, a multi-storey URM wall can be expected to crack at the level of the diaphragm supports and near the mid-height of the wall elements that span between the supports. Figure 1(a) shows the forces assumed to act on a cracked wall element spanning, H, between supports and subjected to a static lateral load V. 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 and is assumed to act at the wall centre line.

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 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 slightly less than the nominal wall thickness.

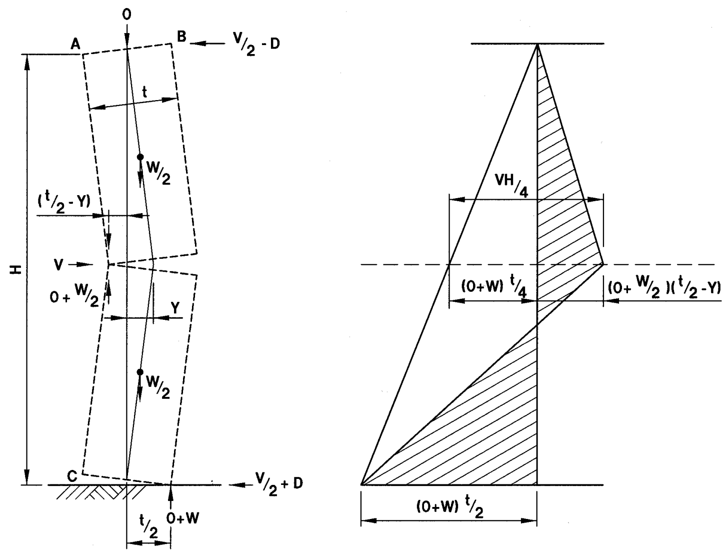

Figure 1: Behaviour of face loaded wall under static loading - a) forces assumed to act on the wall and - b) wall bending moment distribution.

At the mid-height crack, the reaction between the upper and lower halves of the wall is also assumed to be located t/2 from the deflected centre line of the wall or (t/2 - Y) relative to the undeflected wall centre line as indicated. Figure 1(b) shows the bending moments developed in the wall when the wall is subjected to a point load, V, acting laterally at the mid-height crack. The bending moments (shown shaded) are relative to the undeflected centre line.

Equating the simply supported bending moment to the wall moments at the mid-height crack:

$$\frac{VH}{4} = (O+W)\frac{t}{4} + (O+\frac{W}{2})(\frac{t}{2}-Y) \tag{1}$$

$$\therefore V = \frac{2}{H}\left(W(t-Y) + O(\frac{3t}{2}-2Y) \right) \tag{2}$$

V will have a maximum value, V_{max} when Y = 0.0

$$\therefore V_{max} = \frac{t}{H}[2W + 3O] \tag{3}$$

The wall will become unstable when the applied load,V, reduces to zero and the wall displacement, Y, will then have its maximum static value, Y_{max}.as shown in Figure 2. Therefore rearranging equation 2 and substituting V = 0.0:

$$Y_{max} = \left(\frac{W+1.5O}{W+2O} \right) t \tag{4}$$

Using equation 3 and 4 to replace O and W in equation 2 and rearranging equation 2 it can also be shown that:

$$V = V_{max}\ (1 - Y/Y_{max}) \tag{5}$$

This equation is shown graphically below:

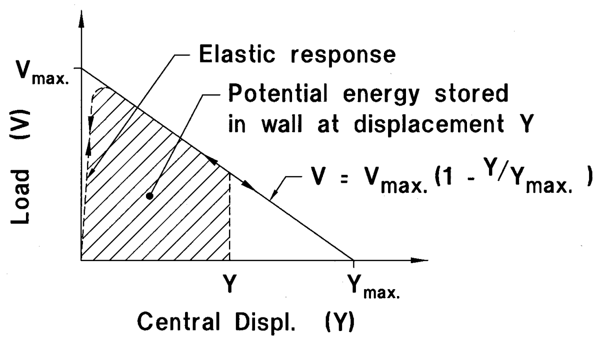

Figure 2: Load Deformation Relationship for Point Load Acting at Mid-Height Crack

The difference in reactions at the top and bottom support levels, D, indicated in Figure 1(a), can be obtained by taking moments about the undeflected wall centre line at the mid-height crack. This results in the expression:

$$D = [O + W]\frac{t}{2H} - \frac{WY}{2H} \tag{6}$$

If a uniformly distributed load, wH replaces the point load, V, in Figure 1(a), the simply supported bending moment will be $wH^2/8$. Therefore the load, V, in equations 1 to 5 would need to be replaced by wH/2 if the load applied to the wall was uniformly distributed,

$$\text{ie, } wH = 2V \tag{7}$$

Therefore, if the load is uniformly distributed, twice the load is required to produce the same wall displacement.

The equations used to calculate Y_{max} and D (equations 4 and 6) are not affected by the load distribution and remain the same for a uniformly distributed load.

Equations 3 and 7 can also be used to find the seismic lateral load coefficient, Cd, at which the cracks in the wall will start to open:

$$C_d = \frac{wH}{W} = \frac{2V_{max}}{W} = \frac{2t}{H}(2 + \frac{3O}{W}) \qquad (8)$$

Computer model

A computer model was developed that could be used to evaluate the inelastic dynamic behaviour of a face-loaded URM wall.

The model allows the wall to deform as indicated in Figure 1(a). Opening of cracks at mid-height and at the base of the wall is accommodated by the link members that buckle when subjected to any compressive load.

The “full scale” wall modelled had a height, H, of 4.8m and an effective thickness, t, of 330mm.

Half scale and 2 x scale walls were also evaluated. These walls had the same slenderness ratio, H/t, as the full-scale (4.8 / 0.33m) walls but had half or double the effective wall thickness and height respectively.

The model was analysed using the inelastic dynamic analysis program DRAIN-2DX Version 1.01. The program uses time step-by-step numeric integration to perform inelastic dynamic analysis.

Comparison between Computer Modelling and ABK Test Results

A comparison between the wall displacements predicted by the computer model and those measured during the testing of one of ABK's wall specimens is shown in Figure 3. The actual diaphragm level motion used in the test was unknown and had to be approximated to the response of SDOF elastic oscillator with a 0.5 second period.

Agreement between the predicted and observed displacements of the wall specimens was good given the uncertain nature of the input motion used in the test and the sensitivity of the response to small changes in the input motion. This demonstrated that the model could be used to predict the behaviour of face-loaded URM walls. It also indicated, with significant uncertainty, the “normal” level of damping to be expected.

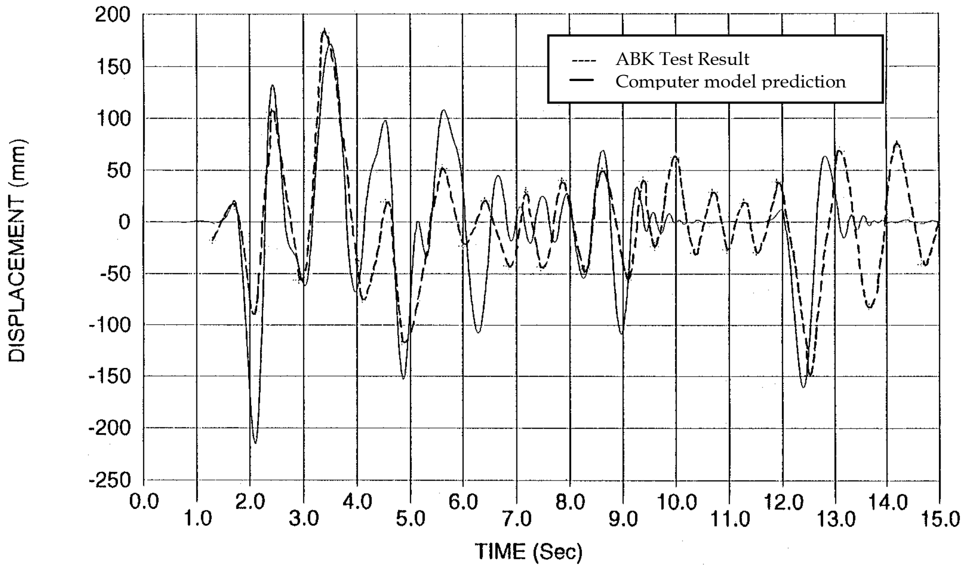

Figure 3: Comparison Between Face Loaded Wall Displacements Predicted by Computer Modelling and those Measured During Testing of ABK Wall Specimen (No.1)

Wall Response to Earthquake motion

A computer program (WAVE) was used to modify the acceleration time history of the 1940 El Centro NS earthquake motion so that its 5% damped spectra more closely matched the Basic Seismic Hazard Spectra given in New Zealand's loading code for intermediate soils. This modified earthquake is referred to in this paper as the NZS4203 earthquake motion.

To evaluate the effect of earthquake intensity on the stability of face loaded walls the intensity of the input motion used for the computer analysis was gradually increased by scaling until the wall "collapsed". The relationship between the displacements at the mid-height crack in the wall and the NZS4203 earthquake intensity is shown in Figure 4 for the full scale wall model. The wall displacements shown have been normalised using the displacement at which the wall becomes unstable, Y_{max} (equation 4).

It can be seen that the results for the lower (reduced) level of damping considered fall into two zones. Broken lines show the upper and lower envelopes of these zones. It can also be seen that the results that lie near the upper envelope correspond to narrow bands or windows of the earthquake intensity scaling factor (e.g. shaded window for scale factor between 0.9 and 1.0).

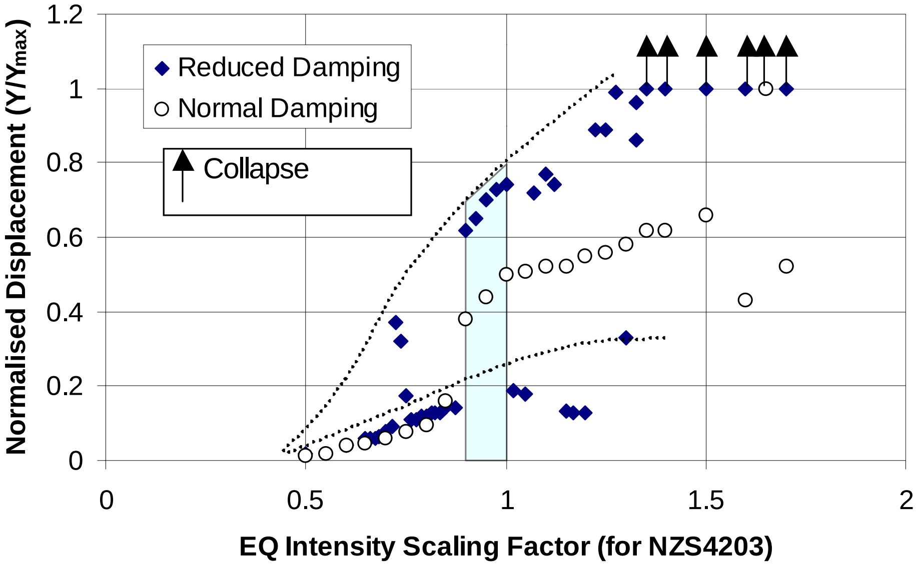

Figure 4: Normalised Displacements of wall at the Mid-height crack level vrs the Scaling Factor applied to the NZS4203 Earthquake Motion For full-scale (4.8 x .33 m) wall

This behaviour demonstrates the sensitivity of the wall response to small changes in the earthquake motion. As the wall is responding inelastically, a change in the earthquake intensity generates a phase shift between the earthquake loading and the wall displacements. Hence, a small increase in earthquake intensity can result in a pulse that was causing wall instability to act at a different time in the response and stabilise the wall instead. With increasing earthquake intensity, the average wall displacement tends to increase and a smaller adverse pulse is required to cause collapse. When the wall is displaced beyond about $0.6Y_{max}$ only a small additional adverse acceleration pulse is required to make the wall unstable. Therefore the wall tends to become less stable.

It can also be seen that a large number of analyses (or tests) with a <u>single</u> earthquake record are required to establish any pattern to the results.

Similar behaviour was observed for other earthquake records and for the full and half scale walls. However, it was found that the analysis results were insensitive to the initial elastic stiffness of the wall prior to the cracks opening. Sensisitivity to this stiffness would have been anticipated if the equal energy assessment method given in the NZNSEE guidelines was applied to the wall.

Period of Free Vibration Response

The face loaded wall model was subjected to a short duration (0.2 sec) acceleration pulse applied to the top and bottom of the wall. This was a convenient method of applying an initial displacement to the centre of the wall. The free damped response of the wall that follows the initial displacement is shown in Figure 5.

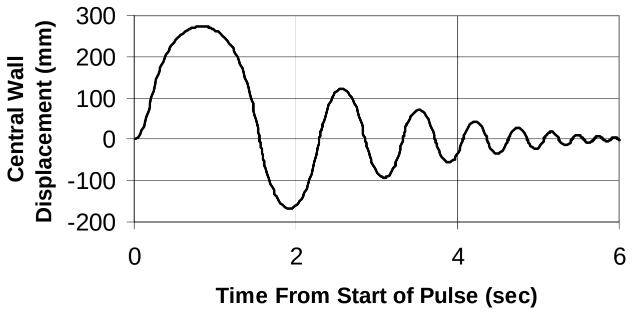

Figure 5: Horizontal Displacement at Centre of 4.8 x .33m Wall (after inital 0.64g Pulse for 0.2 Seconds)

It can be seen that the free vibration period of the wall decreases with decreasing lateral wall displacement.

The peak potential energy stored in the wall at a displacement Y can be calculated with reference to Figure 2. This peak potential energy will be equal to the peak kinetic energy stored in the wall at zero displacement (if losses are ignored). It was also established that the shape of each of the half cycles in the response was practically independent of the peak displacement and thickness of the wall for walls of the constant height to thickness ratio examined. Using these observations it can be shown that the period of the wall free vibration, when the peak displacement is $0.6Y_{max}$, is given by:

$$T=\sqrt{\frac{0.0014Y_{max}W}{V_{max}}} \tag{9}$$

where V_{max} and Y_{max} are given by equations 3 and 4 and the units are kN and mm.

This period of the wall response can be used in conjunction with a displacement spectra to predict the seismic stability of face loaded URM walls.

Prediction of Face Loaded Wall Stability Using Displacement Response Spectra

The stability of a face loaded wall depends mainly on the displacement at the mid-height of the wall so that it seems logical to use a displacement response spectra to predict the earthquake intensity required to cause collapse.

A pseudo displacement response spectrum can easily be derived from an acceleration design spectrum using the relationship:

$$Y=(T/2\pi)^2 A \tag{10}$$

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.

The resulting pseudo displacement spectrum for the NZS4203 EQ motion is shown in Figure 6

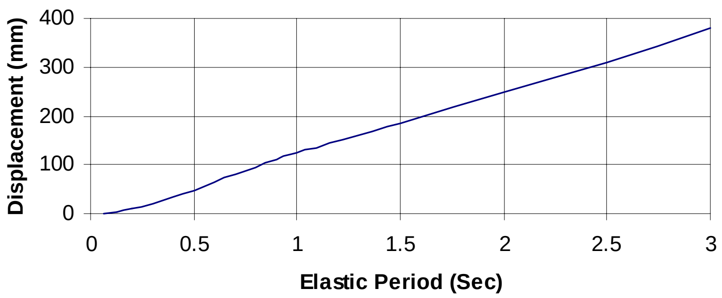

Figure 6: Elastic Displacement Response Spectra for NZS4203 Design Earthquake (Intermediate Soils)

It was noted above that the face loaded wall stability becomes somewhat erratic once the peak wall displacement exceeds approximately $0.6Y_{max}$. Therefore, the displacement response spectra was used to predict the earthquake intensity required to generate a peak wall displacement of $0.6Y_{max}$. Collapse was then assumed to occur at about 20% greater earthquake intensity as indicated by the computer analysis results.

If the cracked wall was responding <u>elastically</u> to an earthquake motion, the peak displacement at the centre of the wall would be 1.5 times that predicted using a SDOF displacement response spectrum (i.e. a modal participation factor of 1.5). However, a scaling factor, S, of 1.4 was selected to scale the 5% damped spectral displacements. This gave results that were conservative relative to the envelope of results predicted using the computer model for the range of wall damping considered during the modelling.

A summary of the calculations required to estimate the stability of a face loaded wall using the proposed methodology is given in Table 1 for a 330mm thick wall subjected to the NZS4203 earthquake motion. The calculations apply to a wall with a wall slenderness ratio (H/t) of 14.5 and a range of overburden to wall weight ratios (O/W) are considered. Results of similar computations for a range of wall slenderness ratios are plotted in Figure 7.

Table 1

H/t	O/W	t	W	O	Y_{max}	V_{max}	T	1.4 x Spectral Displ for period T	0.60 x Y_{max}	EQ Scaling Factor Corresponding to:	
		(mm)	(kN)	(kN)	(mm)	(kN)	(Sec)		=0.6x(6) (mm)	$0.6Y_{max}$= (10)/(9)	Collapse= (11)x1.2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
14.5	0	330	36	0	330	5.0	1.83	317	198	0.62	0.75
	1	330	36	36	275	12.4	1.06	184	165	0.90	1.08
	2	330	36	72	264	20	.82	137	158	1.15	1.38

Notes:

Col (1)	wall height H, to effective thickness, t, ratio
Col (2)	overburden weight, O, to wall weight, W, ratio.
Cols (4) & (5)	wall weight and overburden weight.
Col (6)	displacement at which wall becomes statically unstable: - eq (4)
Col (7)	point load at wall mid-height to open cracks: - eq (3)
Col (8)	free vibration period of wall when peak displacement = 0.6Ymax : - eq (9)
Col (9)	spectral displacement read from Figure 6 for 5%damping and scaled by a factor S = 1.4

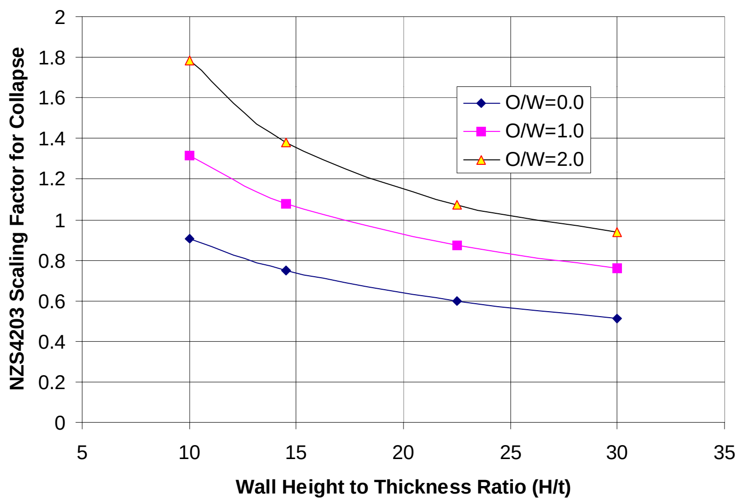

Figure 7: Predicted Seismic Resistance of a 330mm Thick URM Wall Using Proposed Methodology and NZS4203 (Intermediate soil) EQ Intensity

Comparison between Proposed Methodology and Computer Model

The Earthquake Intensity Scaling Factors corresponding to a wall displacement of $0.6Y_{max}$, as predicted by the proposed formulae and as predicted by the computer model, are compared in Table 2 for a range of earthquake motions.

The scaling factor predicted by the computer model, column (5), corresponds to the lower envelope of the wall responses obtained for the range of damping considered.

Table 2

EQ MOTION	WALL SCALE FACTOR	PERIOD T when $Y=0.6Y_{max}$ (sec's)	Earthquake Intensity Scaling Factors Required For Central Wall Displacement Of $0.6Y_{max}$		
			As Predicted by Proposed Formulae	As Predicted by Computer Model	Predicted by Formula / Predicted Computer Model
(1)	(2)	(3)	(4)	(5)	(6)
TABAS (Iran, 16 Sept. 1978 - Transverse	1/2 1 2	1.13 1.6 2.26	0.38 (0.35)** 0.43 0.53	0.45 0.48 0.67	0.85 (0.78)** 0.9 0.79
WEBER (NZ, 13 May ,1990 – N67E component)	1/2 1 2	1.13 1.6 2.26	0.6 (1.03)** 1.07 2.95 (1.91)*	1.25 1.46 2.0	0.48 (0.82) 0.73 1.47 (0.95)
NZS4203 (see text)	1/2 1 2	1.13 1.6 2.26	0.5(.78)** 0.63 0.83	0.95 0.85 1.13	0.52(0.82)** 0.74 0.73
ELCENTRO NS x 1.3	1	1.6	1.07	1.08	0.99
DIAPHRAGM (0.5 second period)	1	1.6	0.71 (0.5)*	0.5	1.42(1.0)*
Notes:	Col (4) & (6)	** Scaling factor corresponds to 120% of EQ intensity required to open cracks. * Peak spectral displacement occurring at period less than T used in calculation.			

For the half scale walls, two values are given in column (4) for the earthquake intensity scaling factors predicted by the proposed formulae. The first was derived using the period given in column (3) and displacement spectra for the earthquake motions. However, for the WEBER and NZS4203 motions, the predicted earthquake intensity using this procedure is only just sufficient to open the cracks in the half scale walls and can be seen to be too conservative when compared with the computer model prediction. The second value given in column (4) was calculated using the 5% damped acceleration spectra assuming that the earthquake intensity required to generate a wall displacement of $0.6Y_{max}$ would be 175% greater than the earthquake intensity required to just open the cracks. In this case, the wall's peak response was assumed to be equivalent to a uniformly distributed static load and the elastic period of the wall motion was taken as 0.1 seconds. This alternative procedure can be seen to improve the correlation with values predicted by the computer modelling for the 1/2 x scale modelled walls.

For the 2 x scale walls and the Weber and Diaphragm earthquake motions, the correlation with the computer model predictions is also improved if the alternative second value given in the table is used for the comparison. This value was computed using the "early" peak in the displacement response spectrum that occurs for these two earthquake motions. As the inelastic face loaded wall period varies with displacement, it appears reasonable to use this early peak spectral displacement if it occurs for a wall period (and displacement) less than that corresponding to a wall displacement of $0.6\ Y_{max}$.

The 3 scaled models of wall analysed had the same H/t ratio. However, the computer model predicts that the earthquake intensity required to cause collapse will increase by 40 to 100% as the effective wall thickness is increased from 166 to 660mm. The same trend is evident in Table 2 in the earthquake intensity scaling factors corresponding to a central wall displacement of $0.6Y_{max,}$ as predicted by the computer model and the proposed formulae. This indicates that wall stability is dependent on wall thickness as well as the height to thickness ratio.

The influence of diaphragm flexibility on the seismic stability of face loaded walls and on diaphragm anchorage forces was also examined as part of the original study on which this paper was based (Blaikie, Spurr, 1992) but is beyond the scope of this paper.

CONCLUSIONS

A computer model was used to predict the seismic behaviour of a face-loaded laboratory wall test specimen. Agreement between the predicted and observed displacements of the wall specimen demonstrated that the model could be used to predict the behaviour of face-loaded URM walls.

The results of analyses using the computer model demonstrated that the seismic stability of a face-loaded URM wall is significantly dependent on the wall thickness and is not only dependant on the height to thickness ratio of the wall as assumed in some current assessment procedures.

It was demonstrated that an alternative assessment procedure using the effective period of a face-loaded URM wall and an elastic displacement response spectrum could be used to predict the earthquake magnitude that will cause the wall to collapse.

ACKNOWLEDGEMENTS

This paper is based on a study funded by the New Zealand Earthquake Commission Research Foundation. This financial assistance is gratefully acknowledged.

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