Out-of-Plane Assessment of an Unreinforced Masonry Wall: Comparison with NZSEE Recommendations

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ABSTRACT: Out-of-plane seismic assessment of unreinforced masonry (URM) walls is an important step in the assessment of a URM building. In this paper, a parametric study is performed on the assessment methodology proposed by the New Zealand Society for Earthquake Engineering, NZSEE, (NZSEE 2006). A single-degree-of-freedom (SDOF) model is then used in time-history analysis (THA), and an alternative assessment method is proposed based on the obtained results. The results are next compared with the NZSEE recommendations. To perform the THA, a well-known commercially available finite element (FE) program is first correlated with a special THA computer program written by researchers in Australia. The commercial program is then used to predict the behaviour of a single-storey 2-leaf URM wall subjected to several earthquake records. The selected wall, having dimensions of 4100 mm high by 220 mm thick, is one of the most common configurations of URM walls found in New Zealand. Earthquake records are selected based on New Zealand seismicity, and the analysis is repeated to account for several soil conditions.

1 INTRODUCTION

Out-of-plane failure of URM walls and parapets has been reported as the major component of financial loss in several earthquakes. In order to reduce the risk from future earthquakes, the NZ Building Act (DBH 2004) requires all buildings to be assessed for earthquake movements that are at least “one-third as strong” as the earthquake shaking used for the design of new buildings. Any building not complying with this legislation is termed an “Earthquake Prone Building”, and needs to be demolished or strengthened. This necessitates every component of URM buildings, including all out-of-plane walls and parapets, to be assessed. NZSEE (2006) recommends to select a stronger earthquake level (two-thirds) for assessment, as the NZSEE view is that any building below this two-third level should be regarded as a questionable earthquake risk and the building is termed an “Earthquake Risk Building”. Percentage of New Building Standard (%NBS) strength is calculated, and out-of-plane walls are categorised as “Unacceptable” (%NBS<33), “Low Hazard” (%NBS equal or greater than 67), or otherwise “Moderate Hazard” walls. A %NBS equal to 33 corresponds to the aforementioned “one-third as strong” level.

The proposed procedure for out-of-plane assessment in the NZSEE (2006) includes both “one-way” and “two-way” evaluations. Assessment of one-way walls is based on “stability” instead of "strength" concepts. As a result, a URM wall is still accepted if it is predicted to crack but remain stable during a certain earthquake. The assessment procedure is mainly based on research by Blaikie (1999, 2002) and Blaikie and Spurr (1992), which utilise bilinear multi-degree-of-freedom (MDOF) models to predict the behaviour of walls. While being simple and suitable for performing fast analyses, a bilinear model may not fully capture the effects of the wall elastic deformation and the effects of material crushing at the cracked joint. A tri-linear model was developed by Doherty et al. (2002) which represented more accurately the real curvilinear force-displacement relationship. The model was further investigated by Griffith et al. (2003) and was used in THA by Lam et al. (2003). A series of experiments was recently performed by the authors (Derakhshan and Ingham 2008) and tri-linear models were characterised for use with URM walls in New Zealand. The tri-linear models can ideally be used with a SDOF system representing out-of-plane URM walls connected to rigid diaphragms, or as part of a two-degree-of-freedom (2DOF) or MDOF systems to study the effects of flexible diaphragms on wall behaviour.

In the following sections, results of wall assessment based on the NZSEE guidelines are first presented. The outcome is then compared to the results from THA performed using tri-linear force-displacement models.

2 NZSEE ASSESSMENT

The NZSEE (2006) assessment procedure involves using the New Zealand Standard for Structural Design Actions – Part 5, NZS 1170.5:2004 (NZS 2004). The available "elastic site spectra" in this standard is used to determine the seismic coefficient, $C_p(T_p)$, for out-of-plane walls, with T_P calculated as the instantaneous period of the study wall cracked at mid-height and displaced to 60% of the instability displacement. The instability displacement, Δ_{ins}, can be calculated from static equilibrium of the cracked wall. The rocking displacement demand of the wall is then calculated by multiplication of pseudo-displacement obtained from this "elastic" acceleration coefficient and a participation factor, γ. The participation factor aims to relate the displacement demand of a rocking wall to the displacement of an elastic body with a natural period equal to T_P.

The NZSEE assessment procedure was applied to walls located in both levels of a two-storey building and to a single-storey wall. Analyses were repeated for different soil types. This resulted in 9 walls listed in Table 1 being analysed. In Table 1, "t_n" and "H" are nominal wall thickness and wall clear height, respectively. Length of wall was not an important factor in one-way out-of-plane analysis, and a unit length was adopted for all walls. The assumed wall height and thickness reflected one of the walls previously tested to obtain tri-linear force-displacement model data (Derakhshan and Ingham, 2009). Walls were assumed to have a density of 1800 kg/m^3 and be located in Wellington City.

Table 1: Wall specification

Wall	t_n, mm	H, mm	H/t	Overburden, kPa	Storey	Soil
1						A/B
2	220	4100	18.6	0	Single-storey	C
3						D/E
4					Top-storey of a	A/B
5	220	4100	18.6	0	two-storey	C
6					building	D/E
7					1st storey of a	A/B
8	220	4100	18.6	75	two-storey	C
9					building	D/E

All of the calculations involved in the assessment procedure for the walls listed in Table 1 are summarised in Tables 2 and 3. According to Table 2, all walls except for Wall 5 satisfy the criteria of "Moderate Hazard", although Walls 4, 6 and 8 are on the threshold of the "Unacceptable" criteria. Soil type C and the location of Wall 5 in the building (second level) contributed to its low %NBS.

3 IN-DEPTH ANALYSIS OF THE NZSEE ASSESSMENT PROCEDURE

The slenderness ratio (S) of out-of-plane walls has long been considered as an appropriate parameter to judge wall stability. For instance, Seismic Rehabilitation of Existing Buildings, ASCE/SEI 41-06 (ASCE 2007), suggests parameter S to be the sole criterion for assessment of out-of-plane URM walls. Although assuming wall slenderness ratio as the sole criteria for wall assessment is not correct for walls with different thicknesses (Lam et al. 1995), analyses performed by the authors in a separate research showed that this assumption is correct for walls with similar thickness.

Calculations presented in Tables 2 and 3 were done for a single wall with a specific slenderness ratio (18.6). In order to investigate the effects of the wall slenderness ratio on the above results similar analyses were performed on Wall 1 using the NZSEE procedure, while changing the wall height and keeping its thickness constant. This enabled the critical values of slenderness ratio, responsible for making the value of %NBS higher or lower than 33 or 67, to be identified. The calculated critical slenderness ratios are highlighted in Table 4 for Wall 1 and Soil Type A/B category. Table 4 also lists range of slenderness ratios calculated for different levels of seismic hazard using the NZSEE (2006) procedure for other single-storey and two-storey walls located in different seismic zones. As an example, one can obtain appropriate critical slenderness ratios from Table 4 for walls with geographical and elevation properties similar to Walls 6 and 8. The maximum value for S for a "Moderate Hazard" wall, which is also equal to the threshold for an "unacceptable" wall, is 19 for both cases. This has been highlighted in Table 4. This means that having a slenderness ratio of 18.6, Walls 6 and 8 are only marginally acceptable in the "Moderate Hazard" category. This example could have also been viewed from Table 2, where the %NBS were calculated as 33 and 34 for Walls 6 and 8, respectively. Table 4 also suggest a maximum slenderness ratio of 4 and 11 for "Low Hazard" level walls falling in the same seismic, soil, and storey category as Walls 6 and 8.

Table 2: Assessment of walls based on NZSEE guidelines*

Wall	t, m	P, kN	b, kN.m	a, kN.m	Δ_m, m	J, kN.m.sec^2	T_P, sec	γ	$C_P(T_P)$	D_{ph}, m	%NBS	R**
1-6	0.215	0	3.48	33.28	0.129	2.37	1.675	1.437	0.268	0.274	56	Mod
									0.357	0.365	42	Mod
									0.301	0.307	50	Mod
									0.405	0.414	37	Mod
									0.539	0.550	28	U
									0.454	0.463	33	Mod
7-9	0.209	16.2	8.48	99.85	0.105	2.53	0.998	1.349	0.807	0.275	46	Mod
									1.074	0.366	34	Mod
									0.904	0.308	41	Mod

* P in the above table is the total axial load produced by overburden specified in Table 1. $C_P(T_P)$ is obtained from Table 3. The other parameters are defined in the NZSEE (2006) guidelines and are calculated according to equations available in those guidelines. A few unlisted parameters involved in the above calculations are J_{b0} and J_{t0}, which were calculated as 0.29 kN.m.sec^2, and W_t and W_b, which were calculated as 8.12 kN. e_o,e_b, and e_t are assumed to be equal to t/2. e_p is assumed to be zero.
**Results: "Mod" refers to a "Moderate Hazard" wall, and "U" refers to an "Unacceptable" wall.

Table 3: Calculation of $C_P(T_P)$ based on the NZS 1170.5:2004 *

Wall	T_P, sec	$C_h(T)$	R	N(T,D)	C(0)	h_n, m	h_i, m	C_{Hi}	$C_i(T_P)$	$C_P(T_P)$
1	1.675	1	1	1	0.4	4.1	2.05	1.34	0.50	0.27
2	1.675	1.33	1	1	0.532	4.1	2.05	1.34	0.50	0.36
3	1.675	1.12	1	1	0.448	4.1	2.05	1.34	0.50	0.30
4	1.675	1	1	1	0.4	8.2	6.15	2.03	0.50	0.41
5	1.675	1.33	1	1	0.532	8.2	6.15	2.03	0.50	0.54
6	1.675	1.12	1	1	0.448	8.2	6.15	2.03	0.50	0.45
7	0.998	1	1	1	0.4	8.2	2.05	1.34	1.50	0.81
8	0.998	1.33	1	1	0.532	8.2	2.05	1.34	1.50	1.07
9	0.998	1.12	1	1	0.448	8.2	2.05	1.34	1.50	0.90

* Part risk factor, R_P, is assumed equal to one, representing category P.1 from Table 8.1 of the standard

The results in Table 2 vary greatly from the suggested values in other available standards. ASCE (2007) suggests 9 and 15 as the maximum slenderness ratio of walls located in the top-storey (similar to Wall 6) and first-storey (similar to Wall 8) of multi-storey buildings located in the areas with the highest seismicity in USA. Although a higher NZ seismicity may partially contribute to this huge difference, the NZSEE procedure will still remain open to discussion due to the relatively good performance of the existing URM walls in New Zealand during past earthquakes. As discussed later, obtained critical slenderness ratios from the NZSEE assessment method may not be correct.

Table 4: Range of S values * of 2-leaf single or two-storey walls for a given hazard class, calculated based on the NZSEE (2006) procedure**

	Z, (Zone factor)	Overburden, kPa: 0 (single-storey) Low	0 (single-storey) Moderate	0 (2nd storey***) Low	0 (2nd storey***) Moderate	75 (1st storey***) Low	75 (1st storey***) Moderate
Soil Type A or B	0.13	N/A	N/A	S<29	N/A	N/A	N/A
	0.20	S<28	N/A	S<21	N/A	N/A	N/A
	0.25	S<24	N/A	S<9	9≤S<30	S<21	N/A
	0.31	S<20	N/A	S<6	6≤S<26	S<16	N/A
	0.36	S<18	18≤S<30	S<5	5≤S<22	S<15	N/A
	0.40	S<9	9≤S<28	S<4	4≤S<21	S<13	N/A
	0.45	S<7	7≤S<26	S<4	4≤S<19	S<12	N/A
Soil Type C	0.13	S<31	N/A	S<23	N/A	N/A	N/A
	0.20	S<22	N/A	S<8	8≤S<28	S<19	N/A
	0.25	S<19	19≤S<32	S<5	5≤S<24	S<15	N/A
	0.31	S<9	9≤S<28	S<4	4≤S<21	S<13	N/A
	0.36	S<6	6≤S<24	S<4	4≤S<18	S<12	12≤S<22
	0.40	S<5	5≤S<22	S<3	3≤S<8	S<11	11≤S<19
	0.45	S<5	5≤S<21	S<3	3≤S<6	S<10	10≤S<17
Soil Type D or E	0.13	N/A	N/A	S<27	N/A	N/A	N/A
	0.20	S<26	N/A	S<19	N/A	N/A	N/A
	0.25	S<22	N/A	S<7	7≤S<28	S<18	N/A
	0.31	S<19	19≤S<32	S<5	5≤S<24	S<15	N/A
	0.36	S<9	9≤S<28	S<4	4≤S<20	S<13	N/A
	0.40	S<7	7≤S<26	S<4	4≤S<19	S<12	N/A
	0.45	S<6	6≤S<23	S<3	3≤S<9	S<11	11≤S<21

* Values rounded to the nearest whole number; N/A refers to an upper limit above 32. **Wall density and other applicable properties to match data in Table 2 & 3 *** First storey height is 4100 mm

In addition to the above two-leaf wall study, the case of single-storey one-leaf walls was also considered. Table 5 summarises the critical slenderness ratios for this case. One can see that values obtained for 1-leaf walls vary from those obtained for single-storey two-leaf walls (Table 4). The NZSEE assessment results vary again highly from the ASCE (2007) recommendations.

Table 5: Range of slenderness ratios for a 1-leaf single-storey non-load-bearing wall for a given hazard class, calculated based on the NZSEE (2006) procedure

Z	Soil Type A/B Low	Soil Type A/B Moderate	Soil Type C Low	Soil Type C Moderate	Soil Type D/E Low	Soil Type D/E Moderate
0.13	N/A	N/A	S<17	N/A	N/A	N/A
0.20	S<12	N/A	S<8	N/A	S<10	N/A
0.25	S<8	N/A	S<5	5≤S<37	S<7	N/A
0.31	S<6	N/A	S<5	5≤S<12	S<5	5≤S<17
0.36	S<5	5≤S<15	S<4	4≤S<9	S<5	5≤S<12
0.40	S<5	5≤S<12	S<4	4≤S<8	S<5	5≤S<10
0.45	S<4	4≤S<10	S<3	3≤S<6	S<4	4≤S<8

In order to explain the low critical slenderness ratios obtained for the "Low Hazard" level in Table 4, the assessment procedure was reviewed in more detail. The calculated results for %NBS and three other parameters were plotted against the wall slenderness ratio as shown in Figure 1. In the derivation of Figure 1, the height of Wall 1 has been increased, while keeping other parameters constant. A shift can be traced in all plots shown in Figure 1 at a wall slenderness ratio of about 14.5. For wall slenderness ratios higher than 14.5, an increase in wall slenderness ratio results in a lower %NBS, whereas for a wide range of wall slenderness ratios less than 14.5 the reverse is correct.

As detailed in Figure 1, the aforementioned shift in %NBS results can be traced back in the effects of the wall slenderness ratio on other parameters listed in Tables 2 and 3. According to the NZSEE (2006) guidelines, the rocking displacement demand of walls, D_{ph}, can be obtained from Equation 1. This equation shows that D_{ph} is a second-order function of T_P, and is also in direct relationship with $C_P(T_P)$, the seismic coefficient. $C_P(T_P)$ in turn is in direct relationship with $C_i(T_p)$, the part spectral shape factor (NZS 2004). For periods ranging from 0.75 to 1.5 seconds, $C_i(T_p)$ is a linear function of T_P. This results in D_{ph} to be a third-order function of T_P within the mentioned range. The highly curved plot, appearing for lower slenderness ratios in Figure 1-c, and consequently in Figure 1-d, is a result of this third-order function. Beyond a point corresponding to T_P=1.5 seconds, D_{ph} become a second-order function of T_P, as $C_i(T_p)$ assumes a constant value of 0.5. This split effect of $C_i(T_p)$ results in a split figure of %NBS over a wide, still practical, range of slenderness ratios. This shift of results makes it impossible to judge a wall for stability based on its slenderness ratio.

$$D_{ph} = \gamma.(T_P/2\pi)^2.C_P(T_P).R_P.g \qquad (1)$$

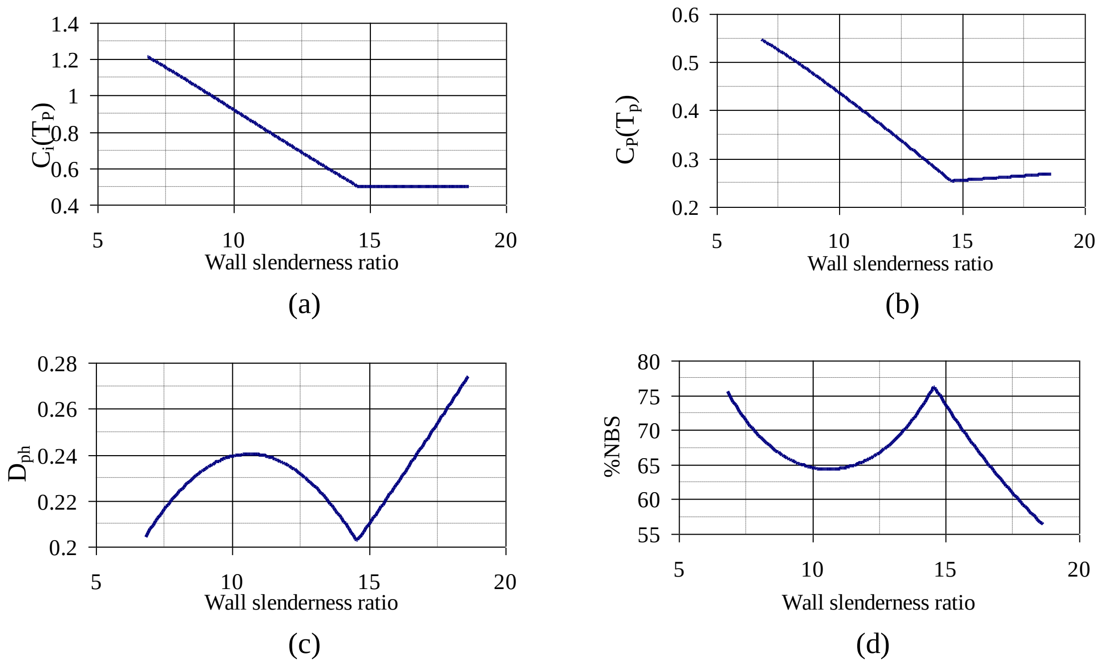

Figure 1: Parametric trends plotted against wall slenderness ratio

4 TIME HISTORY ANALYSIS

Calculation of the seismic response of simply-supported one-way out-of-plane URM walls connected to rigid diaphragms is well formulated in Doherty (2000). Considering dynamic equilibrium of such walls, cracked at mid-height and subjected to earthquake excitation, a SDOF equation of motion can be written as Equation 2, according to Doherty (2000).

$$a_m(t)+\frac{3}{2}\left(\frac{4g}{H}\left[\frac{t}{\Delta(t)}-1\right]\right)\Delta(t)=-\frac{3}{2}a_g(t) \qquad (2)$$

Where $\Delta(t)$ is the time-dependent relative displacement of the wall at mid-height, $a_g(t)$ is the ground excitation, and $a_m(t)$ is the relative acceleration component within the system. The term in parentheses corresponds to the static bilinear force-displacement relationship of the cracked wall. Similarly, the above equation of motion was re-written in Doherty (2000) as a split function to account for a tri-linear force-displacement model. A tri-linear model for an out-of-plane URM wall is shown in Figure 2 along with the bilinear model as well as the real behaviour. The tri-linear model incorporates the bilinear properties, the effects of material crushing, and any modification made by elastic deformations occurred in the URM wall. Doherty (2000) used a customised FE program, ROWMANRY (reported in Doherty et al. (2002)) to solve the split equation of motion. While the computer program proved to be effective for SDOF modelling of out-of-plane walls, it is not straightforward to extend the code to include 2DOF or MDOF model analyses. These analyses are necessary when out-of-plane investigation of URM walls connected to flexible diaphragms is sought.

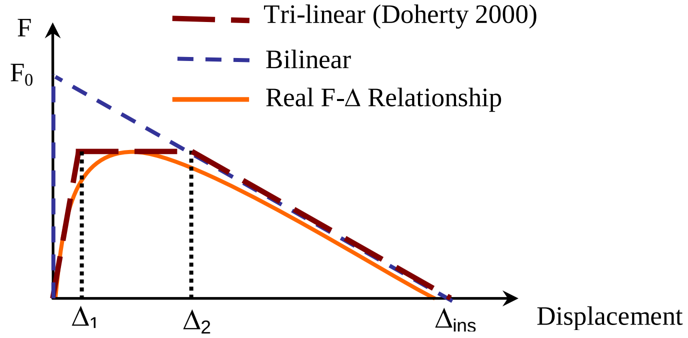

Figure 2: Different representations of the real F-Δ curve for a wall subjected to out-of-plane loading

THA were performed on a SDOF system representative of Wall 2. Regardless of the simplicity of the model, which was only a SDOF system, a commercially available computer program (Sap2000® 2005) was used in place of ROWMANRY to facilitate future extensions. An ideal correlation (Figure 3) was obtained between the results from analysis using SAP2000 and the ROWMANRY computer program, when both were used to predict the behaviour of Wall 2 subjected to a half-sine pulse excitation. Mass and stiffness proportional damping equal to 1.65 and 0.0053 was adopted in both analyses. The tri-linear model properties were adopted based on an earlier study (Derakhshan and Ingham, 2009). As there is no experimental dynamic response history of Wall 2 currently available for calibration purpose, the correlation shown in Figure 3 is regarded as a reasonable basis for continuing the THA using SAP2000, recognising that ROWMANRY has been successfully correlated with experimental results in Doherty (2000).

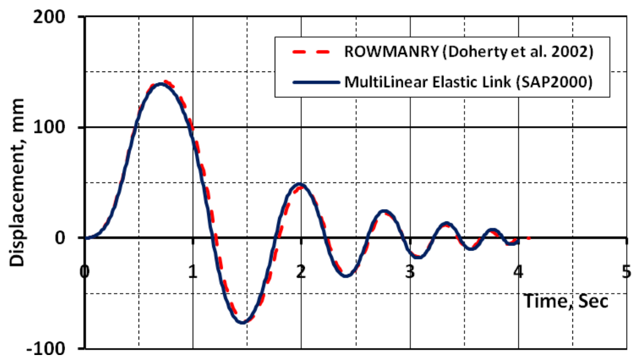

Figure 3: Calculated response of Wall 2 subjected to impulse acceleration ($a(t)= -2.5\sin(2\pi t)$, m/s^2)

Table 6 lists 7 earthquake records used as input and the maximum displacement obtained from 30 analyses performed on Wall 2. The suite of time-histories and the relevant primary scale factors were acquired from Oyarzo-Vera et al. (2008). Each time history was scaled again to an average of 5 different levels, as detailed in Table 6, and the responses of the wall to all of the input accelerations were calculated. While primary scale factors shown in Table 6 were applied to all analyses done in this study (to match the seismological signature of the original records to the seismic characteristics of the study site), the secondary scaling factors were used in series with the primary scaling factors to assess wall stability through scenarios with increasing level of peak ground acceleration (PGA). Each row in Table 6 ends with a hatched area, which marks those secondary scaling factors resulting in an incipient failure in the wall model. For example, when the Hokkaido 2003 ground acceleration was scaled in series to 0.526 and 145% and used, the wall model became unstable, indicating that a displacement more than Δ_{ins} (defined in Section 2) was reached. Again for the Hokkaido 2003 record, one can find that the result for a secondary scale factor of 142.5%, which is only slightly less than the scaling factor that caused instability, is 62 mm. This suggests that a small increase in the input PGA will result in a very sharp increase in wall displacement when subjected to the Hokkaido 2003 acceleration record. However, the results from analyses using other records were less sensitive. Contrary to the mentioned example of the Hokkaido record, results from several other records suggested that the wall displacement increased up to 110% of the instability displacement without failure occurring. The wall then returned to the stable limits as a result of a next “reverse” pulse. Figure 4 shows a maximum response of -240 mm achieved in one case.

Table 6: Calculated maximum displacement of Wall 2 subjected to ground acceleration histories, mm

Record	Primary scale factor	Secondary scale factor (%)									
		33	50	66.7	80	82.5	87.5	100	142.5	145	177.5
(1) Liolleo 1985	0.984	21	32	73				100		165	
(2) El-Centro 1940	1.443	5.8	70	144	220						
(3) Hokkaido 2003	0.526	2.3	3	8.7				20	62		
(4) Landers 1992	0.842	11	30	133							
(5) La Union 1985	2.280	35	100	145							
(6) Tabas 1978	0.540	12	37	164	109	105					
(7) Izmit 1999	1.652	12	29	113	134	145					

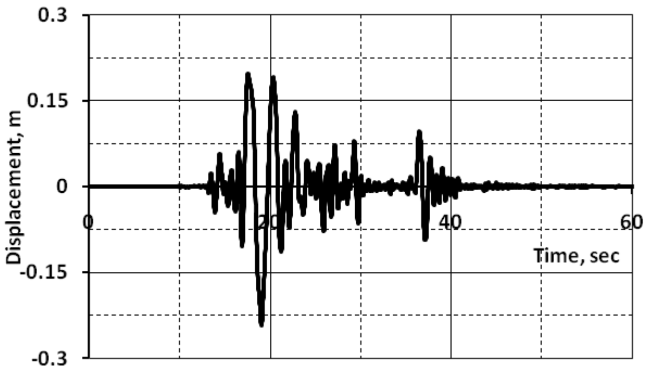

Figure 4: Calculated response of Wall 2 subjected to Record 5 (Secondary scale factor: 100%)

As the response of the out-of-plane wall is highly sensitive to the input ground motion, particularly when the wall approaches the instability displacement, it was necessary to assume a maximum allowable displacement, Δ_m, for the wall. Concluding from Table 6, it was assumed that adopting a maximum allowable displacement equal to two-thirds of the instability displacement (144 mm) was reasonably conservative. Similar assumptions have been made in other studies. The NZSEE (2006), for instance, assumes Δ_m equal to 60% of the instability displacement based on Blaikie (2002).

5 ASSESSMENT OF WALL 2

In this section Wall 2 is assessed using results from THA performed in the previous section and based on the requirements of The Building Act (2004). The intended criteria for assessment of the building components in that legislation are based on the earthquake shakings at the site of the building under study, which should be distinguished from the structural response.

As mentioned earlier, in order to compare the status of an existing out-of-plane URM wall to a similar wall that would be designed according to the current seismic standards, the NZSEE (2006) uses the parameter %NBS, which for an out-of-plane wall is calculated by dividing the maximum allowable displacement to the displacement demand of the wall when subjected to earthquake ground motion. As exemplified in Table 6, the response of an out-of-plane wall does not have a linear relationship with the applied ground acceleration. Therefore, wall displacement can not be used to linearly represent the earthquake "shaking" at the site of the building under study. The NZSEE (2006) approach can be justified only if there is a linear relationship between the response of the structure and the input ground accelerations.

An alternative approach is employed here to calculate %NBS, which utilises the input PGA as a basis for assessment. The secondary scale factor is increased for every record mentioned in Table 6, so that the response of the wall to that record approaches the maximum allowable displacement (144 mm). The maximum scale factor calculated for every record is denoted by S_{mi}, where i stands for individual records from Table 6. As S_{mi} corresponds to the ratio of the maximum PGA, which causes the wall to reach its displacement limit, to the design PGA (obtained by scaling the records to the primary scale factors only), S_{mi} is regarded as %NBS. The overall %NBS of the wall is then calculated as the average %NBS value obtained for individual records. Table 7 shows the calculated values for Wall 2. As an example calculation of the %NBS for Record 1 is explained. From Table 6, one can interpolate the secondary scale factor corresponding to 144 mm to be equal to 130. Correspondingly, 130 is listed in Table 7 as the %NBS for Record 1. S_{m3} and S_{m4} were assumed to be equal to 142 and 67, respectively, as the wall responses beyond these scales approached sharply to Δ_m. As shown in Table 7, the overall %NBS for Wall 2 was calculated as 89, which is about 211% higher than the obtained %NBS from Table 2. This suggests that the NZSEE (2006) guidelines are overly conservative regarding the evaluation of Wall 2 studied here.

Table 7: Calculation of %NBS

Record	(1)	(2)	(3)	(4)	(5)	(6)	(7)	Ave(Overall %NBS)
%NBS	130	67	142	67	67	66	82	89

With the procedure discussed above, calculation of the %NBS for the other walls listed in Table 1 requires at least additional 400 analyses to be performed. In order to extend the results to include all walls within a reasonable range of slenderness ratios and to, eventually, obtain the critical slenderness ratios based on the results an estimated 15000 more analyses would be necessary. The SDOF model used herein will be used in a reasonable time frame to perform the analyses.

6 CONCLUSION

The NZSEE (2006) methodology was investigated for several simply-supported walls located at different sites. It was shown that a certain combination of parameters used in the existing procedure leads to erroneous results over a range of practical wall slenderness ratios. Commercial FE modelling software was used to model negative stiffness existing in the tri-linear force-displacement relationship of URM walls. A method was introduced for assessment of out-of-plane URM walls based on input accelerations. A simply-supported wall connected to a rigid diaphragm was assessed using the results from time-history computations, and the results of the assessment were compared with the results from the NZSEE (2006) procedure. It was concluded that the NZSEE (2006) procedure was conservative in assessment of a simply-supported wall connected to rigid diaphragms by a factor of 2.11.

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