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OUT-OF-PLANE SEISMIC PERFORMANCE OF UNREINFORCED CLAY BRICK MASONRY WALLS

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ABSTRACT

Given sufficient anchorage to the diaphragms, out-of-plane walls in unreinforced masonry buildings have been shown to crack above mid-height and rock as two rigid bodies. This study investigates the sensitivity of the rocking response to the type of ground motion and the quality of the wall construction. A parametric study using a nonlinear-elastic single-degree-of-freedom model suggests that buildings located on firm ground sites are less likely to experience out-of-plane wall failures compared with buildings located on soft soil sites. Results are presented for shake table tests on two full-scale three-wythe unreinforced masonry walls; one with good quality collar joints, the other with poor collar joints. Stable rocking behaviour was observed for both walls when the input ground motion was scaled beyond the uniform hazard spectrum from the proposed 2005 National Building Code of Canada (NBCC).

KEYWORDS: earthquakes, shake table tests, out-of-plane, unreinforced masonry walls, clay brick

INTRODUCTION

Clay brick multi-wythe unreinforced masonry walls form the primary structural system of approximately 20 school buildings in Vancouver and Victoria, British Columbia. Similar structures have suffered considerable damage in past earthquakes (e.g. Long Beach, 1933; Loma Prieta, 1989; Northridge, 1994). In the case of Loma Prieta, an increase in damage was observed for URM buildings located on soft soil sites [1]. Typical damage observed for unreinforced masonry (URM) buildings includes: collapse of parapets or gables, diagonal shear failure or sliding shears failure of in-plane walls, and out-of-plane wall failures. The potential collapse of parapets and gables poses a significant hazard to people next to the building at the time of the earthquake. Bracing is frequently provided during a seismic retrofit of a URM building to avoid this failure mode. In-plane wall failures result in a reduction in the lateral load capacity, however, without out-of-plane movement, such failure modes do not necessarily result in collapse of the wall due to continued support of gravity loads across the failure plane. In contrast, out-of-plane wall failures can result in collapse of the load-bearing wall and partial or total collapse of the building. In an effort to provide an improved assessment of the collapse potential of URM school buildings during earthquakes, this study focuses on the out-of-plane response of multi-wythe URM walls.

Out-of-plane wall failures most commonly occur due to inadequate anchorage of the wall to the floor diaphragms. In such cases, the wall behaves as a cantilever and collapses if the inertia forces on the wall push it beyond the point of instability, or half of the wall width (see Figure 1a). Given sufficient anchorage to the diaphragms, out-of-plane walls will respond as vertical "beams" in bending as the inertia forces on the walls are distributed to the attached diaphragms. Due to limited tensile strength of the mortar, anchored URM walls will frequently crack just above mid-height. This results in rocking of the top and bottom wall segments in the out-of-plane direction. If the displacements induced by the ground motion are large enough (i.e. exceeding the wall width at the crack location, see Figure 1b), the wall can become unstable and collapse. Considering the improvement in behaviour for the relatively modest cost of anchoring the walls to the diaphragms, it is assumed in this study that the walls are sufficiently anchored to the floor diaphragm to develop the beam-bending mode of failure.

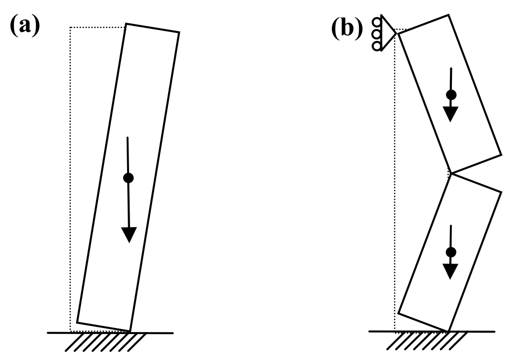


Figure 1 - Out-of-Plane Failure Modes
(a) Cantilever Mode and (b) Beam Bending Mode

For out-of-plane walls with sufficient anchorage to the diaphragms, current seismic rehabilitation guidelines, FEMA 356 [2], specify acceptance criteria based on the height to thickness ratio (*h/t*). Figure 2 provides the FEMA 356 *h/t* limits for walls at the top of a multi-story building expressed as a function of the spectral acceleration at a structural period of 1.0 seconds. These are the most stringent *h/t* limits provided since the walls at the top story are the most vulnerable to failure due to the low axial loads.

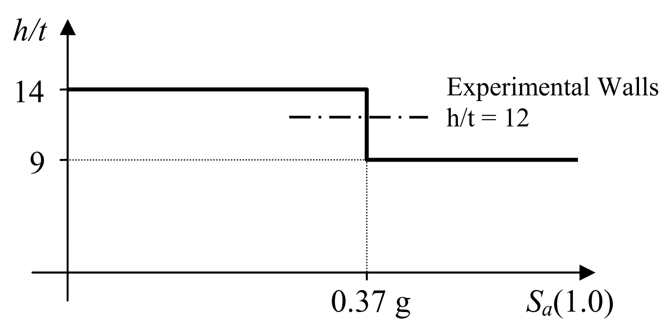


Figure 2 - Height to Thickness Limits for Wall in Top Story (FEMA, 2000)

The *h/t* limits in FEMA 356 are based on tests conducted by ABK [3]. During this pioneering study of out-of-plane performance of URM walls, 22 wall specimens of varying *h/t* ratios were subjected to dynamic loading at the top and bottom of the walls. Of the 22 walls, three were clay-brick multi-wythe walls, and only one of the three had a low overburden typical of top story walls. This wall had an *h/t* ratio of 14. The wall collapsed when subjected to a ground motion with a peak ground acceleration of 0.68 g and a peak ground displacement of 185 mm.

Several shake table tests on single-wythe walls have demonstrated that, given sufficient anchorage to the diaphragms, out-of-plane URM walls can maintain stability when subjected to severe ground motions [4,5,6]. Gulkan et al. [4] tested single-story masonry houses and noted large displacements as the out-of-plane walls rocked at the mid-height crack without collapse. Griffith et al. [5] observed that the out-of-plane rocking response of the wall was sensitive to the displacement demand of the selected ground motion. Ground motions with low peak ground displacements (PGD) would not collapse the wall specimens, while ground motions with high PGD resulted in rocking beyond the stability limit. Simsir et al. [6] included the effect of a flexible diaphragm and noted an increase in the out-of-plane wall displacements. Cracking at mid-height was not observed by Simsir et al. due to high overburden pressure applied to the wall. Despite the experimental evidence indicating that out-of-plane walls can remain stable given sufficient anchorage to the diaphragms, engineers have frequently chosen to not rely on the rocking response of the wall after cracking and have opted for expensive retrofit measures including carbon fibre sheeting of all out-of-plane walls.

As noted, this study focuses on the out-of-plane response of clay-brick multi-wythe URM walls typically used in turn-of-the-century school buildings in southwest British Columbia. These buildings are located on very dense or stiff soil sites (sites C and D based on NBCC 2005). The quality of construction, including the ability of the collar joints between the wythes to maintain composite action during out-of-plane response, is very difficult to assess for the existing structures. Given the limited number of tests on clay-brick multi-wythe walls discussed in the literature, it is not possible to determine the sensitivity of the out-of-plane response to soil conditions, local seismicity, and wall construction quality. This paper reports on shake table tests designed to address these issues and assess the need for retrofit measures for walls adequately anchored to the diaphragms. The following sections describe the analysis used to select the ground motions, the design of the test setup, and results from the first series of shake table tests.

OUT-OF-PLANE RESPONSE ANALYSIS

A nonlinear-elastic SDOF model, developed at the University of Adelaide [7], was used to estimate the post-cracking rocking behaviour of the unreinforced masonry walls. The program, ROWMANRY, performs non-linear dynamic analysis on a SDOF system with the relevant degree of freedom being the displacement at mid-height of the wall and thus the cracking of the wall is assumed to occur at that height. As the wall is subjected to a specified ground motion, the program calculates the displacement, velocity and acceleration time-histories at the mid-height of the wall. The stiffness utilized is based on the nonlinear-elastic force-displacement relationship shown in Figure 3, where Δ_1 and Δ_2 are selected based on the level of damage at the crack. The unreinforced clay brick masonry walls were modelled with no overburden and the reaction at the top and bottom of the wall was assumed to be at the leeward face. The point of instability,

$\Delta_{Instability}$, was taken as the width of the wall as this is the point when the resultant of the weight of the upper portion of the wall is outside the wall width and the system becomes unstable (Figure 1).

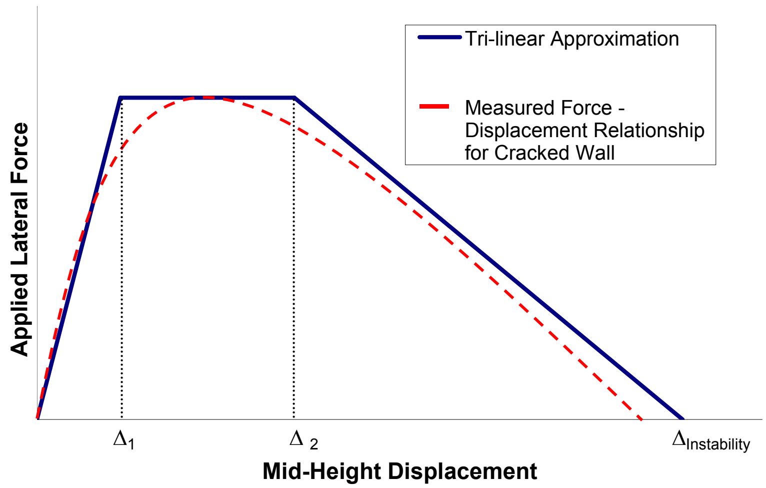


Figure 3 - Tri-linear Stiffness Model (adapted from Doherty, 2000)

Rayleigh damping was incorporated into the model with 5.9% of critical damping at a period 0.5s and 8.9% at 1.0s. The analysis was conducted assuming a moderate level of damage in the wall, which defines $\Delta_1/\ \Delta_{Instability}$ as 13% and $\Delta_2/\ \Delta_{Instability}$ as 40% [7]. This model was calibrated based on the results of dynamic shake table tests on single-wythe walls. Results from the current study will enable verification of the model for three-wythe clay-brick walls.

A parametric study was undertaken for the purpose of evaluating the sensitivity of the out-of-plane response to the site conditions and for selecting the ground motions for the dynamic testing of the unreinforced clay-brick masonry walls. The suite of ground motions used in the study consisted of 80 records from various soil conditions; 20 ground motions from each of site class B (760 m/s < Vs $\leq$ 1500 m/s), site class C (360 m/s < Vs < 760 m/s), site class D (180 m/s < Vs < 360 m/s) and site class E (Vs < 180 m/s). A response spectrum was generated for each of the ground motions and compared to the appropriate Uniform Hazard Spectrum (UHS) for Vancouver from the proposed 2005 National Building Code of Canada (Adams and Atkinson, 2003). The ground motion spectra were then scaled to match the UHS in the period range of 0.5 seconds to 1.0 seconds, the anticipated period range of the cracked walls [7].

Analyses were conducted with the amplitude of each ground motion scaled from 0.10 to 2.50 of the NBCC level at 0.01 increments. The maximum mid-height displacement from each analysis was recorded. Figure 4 shows a plot of the maximum mid-height displacement vs. scaling factor for a ground motion from site classes C, D and E. An instability factor was defined as the scaling factor required to achieve instability.

The instability factor was determined for each of the 80 ground motions used in the study. After removing records which needed to be scaled by a factor greater than 7 to match the UHS, the distribution of scaling factors was plotted and can be seen in Figure 5. These values show that the soil conditions play a significant role in determining the stability of the wall. With the increased level of displacements seen in softer soil conditions, the average instability factor decreased from 2.09 for site class B to 0.98 for site class E. The ground motion used for the shake table tests described below was recorded on firm ground (site class C) in Gilroy, California during the 1989 Loma Prieta Earthquake and has an instability factor of 1.52. When scaled to the UHS, this record has a PGA of 0.63g (Figure 6) and a PGD of 8.9cm and Sa(1.0) = 0.4g. Given this seismic demand, Figure 2 indicates that the FEMA 356 h/t limit is 9.0. For future tests, a ground motion with longer duration and recorded on a softer soil site will be considered.

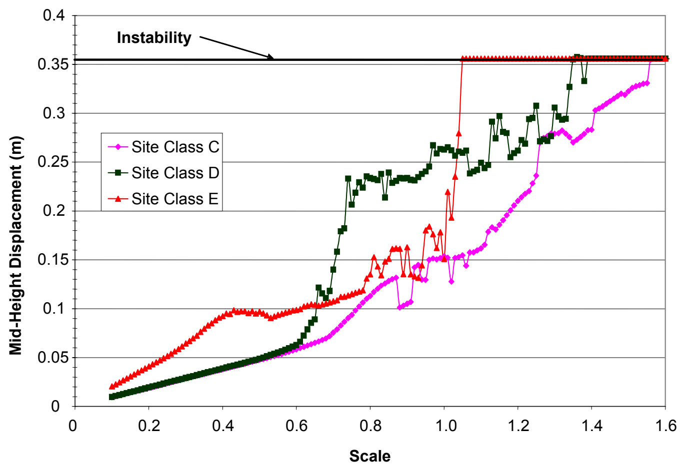


Figure 4 - Peak Mid-Height Displacement vs. Ground Motion Scaling

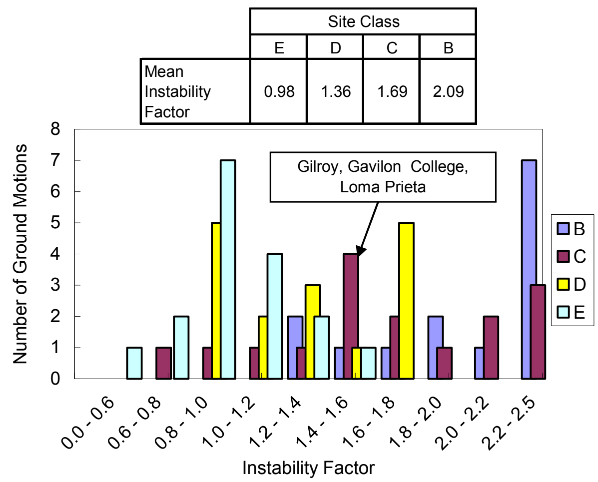
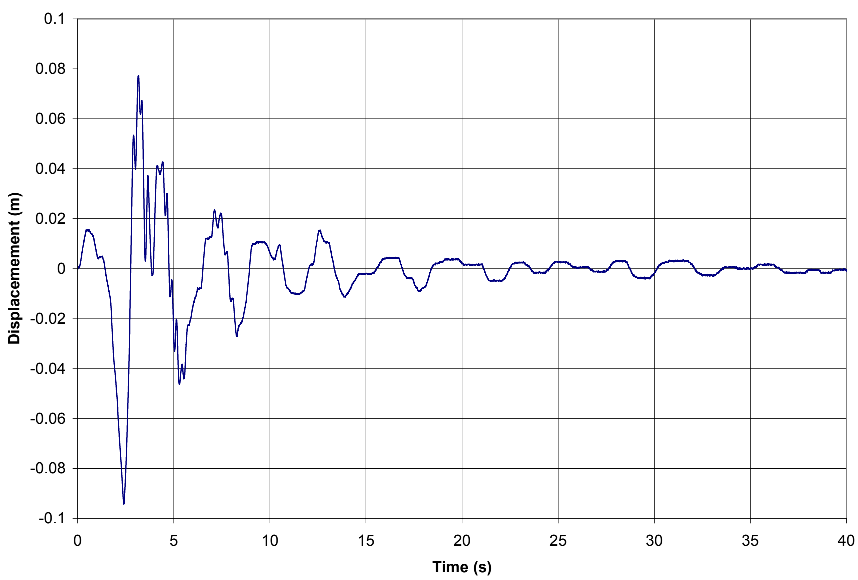


Figure 5 - Distribution of Instability Factors



PGA = 0.63 g
PGV = 45.83 cm/s
PGD = 8.93 cm

Figure 6 - Gilroy Motion Scaled to UHS

SHAKE TABLE TEST SETUP

Specimens were intended to represent a portion of the top story of an early 1900's URM school building in British Columbia, including mortar quality and construction methods. Due to its high lime content and relatively low compressive strength, Type O mortar was considered an appropriate representation of mortar quality for existing URM buildings. To further represent the deterioration of mortar in existing buildings, the brick units were placed dry creating a weak bond between the brick and the mortar. Brick units were solid and measured 220x110x60mm.

Two types of collar joints were constructed to provide bounds on construction variability:

1. Poor Collar Joint - Light mortar bed, collar joints not slushed
2. Good Collar Joint - Heavy mortar bed, collar joints slushed

Four three-wythe wall specimens were constructed; two with poor collar joints and two with good collar joints. American bond, with a header coarse at every sixth course, was used for all specimens (see Figure 7).

Walls were held in place on the shake table by angles on either side of the wall with rubber spacers to allow for rotation. The top support was designed to restrain movement in the out-of-plane direction, but allow for rotation and vertical movement of the top of the wall.

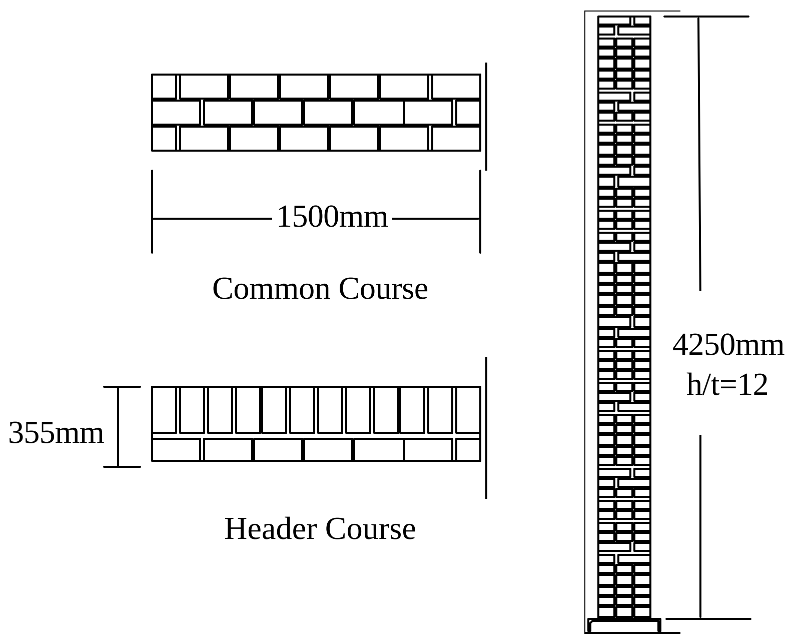


Figure 7 - Wall Dimensions

TEST RESULTS

At the time of publication, two walls had been tested; one with good collar joints, the other with poor collar joints. Both were subjected to the site class C ground motion discussed above. The ground motions were applied with increasing magnitude until collapse. Hammer tests were performed between applied ground motions to determine the dynamic wall properties. The testing sequence and general observations are summarized in Table 1.

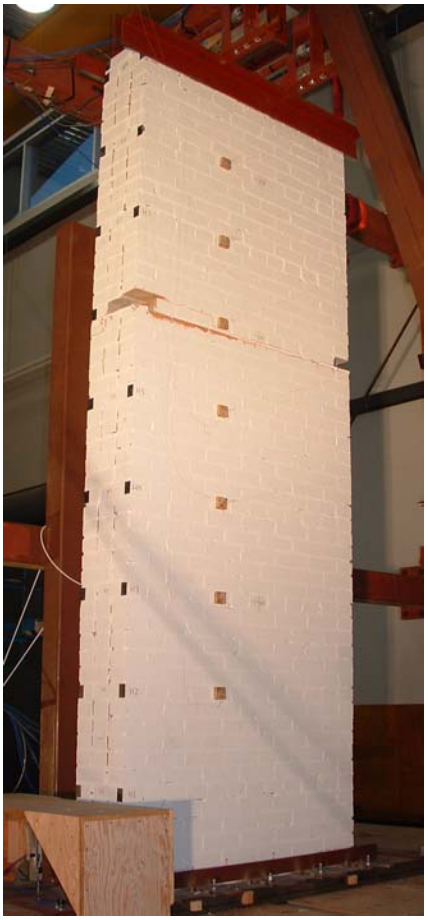
Table 1 - Site Class C Testing Sequence

Test	Scaling Factor to UHS	Return Period (years)	Scaled To	PGA (g)	SDOF Analysis	Good Quality Collar Joint Wall	Poor Quality Collar Joint Wall
1	0.21	72	PGA	0.13	Lower Bound Cracking Limit	No Observed Damage	No Observed Damage
2	0.42	475	PGA	0.26		Crack Formed at Wall Base and Header 6	Crack Formed at Wall Base and Header 6
3	0.53	475	0.5-1.0s	0.33		Rigid Body Rocking	Rigid Body Rocking
4	1.00	2475	0.5-1.0s	0.63	Upper Bound Cracking Limit	Rocking, Lose of Bricks at Header 6	Rocking
5	1.25		0.5-1.0s	0.79		Rocking, Interference at Top Restraint	Rocking, Crack Formed at Header 1
6	1.52		0.5–1.0s	0.96	Instability Limit	Rocking, Interference at Top Restraint	Rocking
7	1.75		0.5–1.0s	1.10		Rocking, Interference at Top Restraint	Rocking, Loss of one wythe of bricks at Header 1

Before wall cracking was observed, the walls underwent an approximately constant acceleration and displacement profile along the height. During Test 2, both walls experienced cracking at the base and at header 6 (i.e. approximately 2750 mm from the base of the wall). Rigid body rocking; where the wall acted as two rigid blocks pivoting about the top restraint, header 6, and the base; was observed for the remainder of the tests.

At the 2005 NBCC level ground motion (Test 4, 1.0 scaling), the good quality wall began to lose bricks at header 6, and crushing of the outside bricks was also evident. Observed damage can be seen in Figure 8. For the good quality wall, at scaling levels of 1.25 and greater (tests 5-8) slight bearing between the wall and the top restraint was noted. This interference may have resulted in limiting mid-height displacements for these tests.

For the poor quality wall, cracks began to form at header 1 after Test 5. Further damage also occurred at header 6 with crushing of the outside brick corners and daylight was seen through the crack. During Test 7, two levels of outer wythe bricks at header 1 were lost, thereby changing the walls rocking behaviour and apparently decreasing the mid-height displacements. Due to the significant damage, the wall was considered to be unstable and all instruments were removed. Figure 9 shows the final collapsed profile of the wall.



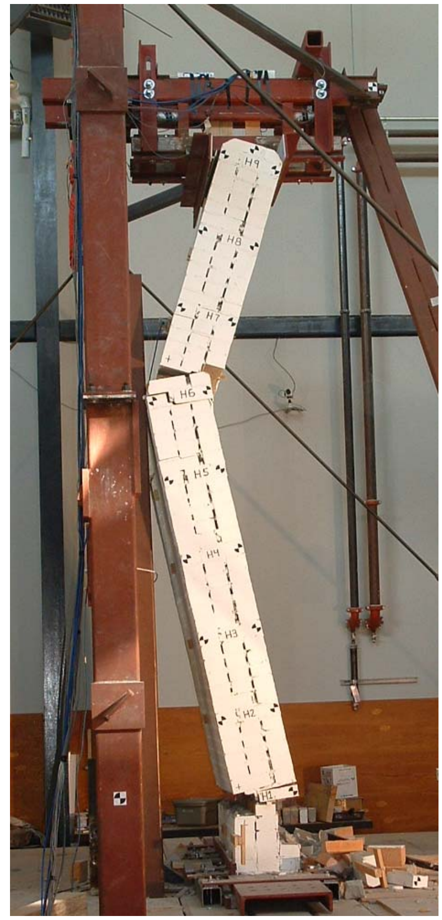


Figure 8 - Good Quality Wall with Crack Formed at Header 6 (Test 4)

Figure 9 - Poor Quality Wall at End of Test

Sample results from the 2005 NBCC level ground motion (Test 4) for the relative deflection at header 6 for the poor and good quality walls are shown in Figures 10 and 11, respectively. The hysteretic response of the walls for the same ground motion are shown in Figures 12 and 13.

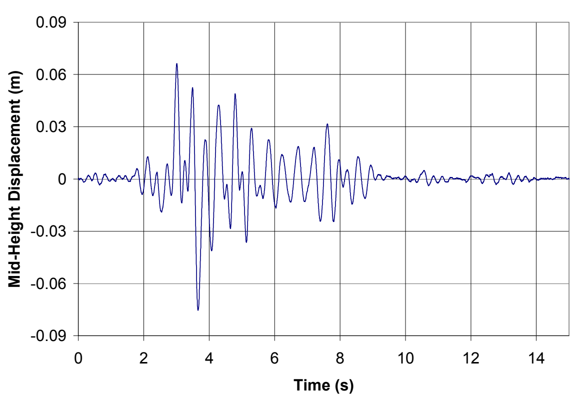


Figure 10 - Header 6 Displacement –Good Quality Wall (Test 4)

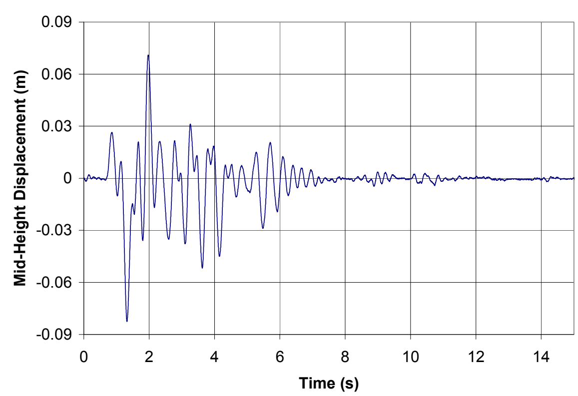


Figure 11 - Header 6 Displacement –Poor Quality Wall (Test 4)

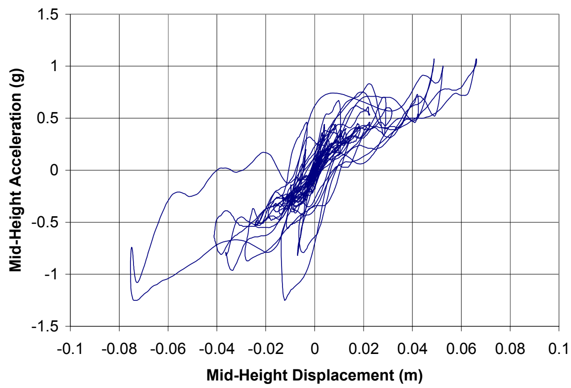


Figure 12 - Hysteretic Response – Good Quality Wall (Test 4)

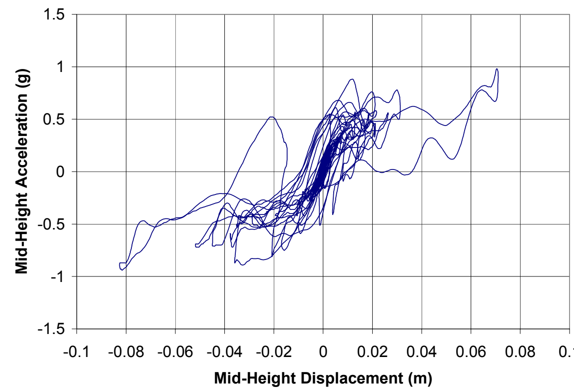


Figure 13 - Hysteretic Response – Poor Quality Wall (Test 4)

The maximum header 6 displacements vs. scale factor for the two walls are compared against the analytical SDOF results in Figure 14. The observed peak displacements of the poor quality wall compare very well to the analytical results until significant damage occurs at header course 1 during Test 7. The good quality wall compares reasonably well with the analytical results up to the code level (Test 4). Applied ground motions above code level resulted in limited moment resistance at the top of the wall due to interference of the top restraint. As shown by the dashed line in Figure 14, this interference appears to decrease the expected peak displacement. Further investigation of this applied moment and its effect on wall stability is required.

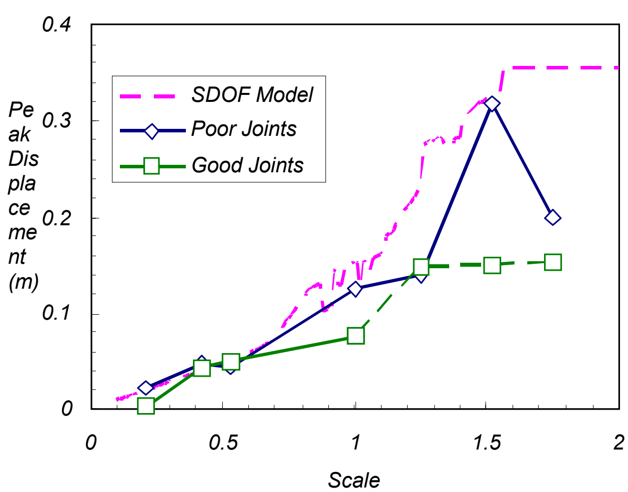


Figure 14 - Peak Mid-Height Displacement vs. Ground Motion Scaling

Since the walls remained stable well beyond the code level ground motion, the h/t ratios from FEMA 356 appear conservative for the evaluation of existing construction. Although the walls cracked at 2/3 of their height, they still remain stable, dissipating energy through rigid body rocking.

CONCLUSIONS AND FUTURE WORK

The study presented here has investigated the sensitivity of the out-of-plane response of multi-wythe URM walls to the type of ground motion and the quality of the wall construction. Analyses based on a SDOF model by Doherty [7] indicate that, given sufficient anchorage of the walls to the diaphragms, URM buildings located on soft soil sites are more likely to experience out-of-plane wall failures than buildings located on firm ground. On average, the intensity of the site class C ground motions had to be scaled 1.7 times higher than the level of the 2005 NBCC to observe instability; while the site class E ground motions caused instability of the wall just below the level of the 2005 NBCC. The shake table tests on full-scale URM walls indicate that the SDOF model adequately captures the peak response of multi-wythe URM walls. The quality of the collar joints was varied for the two specimens, but did not appear to have a significant impact on the response of the walls up to the 2005 NBCC level ground motion. Given the observed stable rocking response beyond the code level ground motion, it appears the h/t limits in FEMA 356 may be somewhat conservative. Further tests will be conducted to investigate the influence of duration and soft soil ground motions on the out-of-plane response of URM walls.

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REFERENCES

1. SEAOC 1991. Reflection on the Loma Prieta Earthquake, Structural Engineers Association of California. October 1989.
2. ASCE. Prestandard and Commentary for the Seismic Rehabilitation of Buildings, FEMA 356. Federal Emergency Management Agency, Washington, D.C., November 2000.
3. ABK. Methodology for Mitigation of Seismic Hazards in Existing Unreinforced Masonry Buildings: Wall Testing, Out-Of-Plane. ABK Topical Report 04. California, 1981.
4. Gülkan, P. Clough, R. Mayes, R., and Manos, G. Seismic Testing of Single-Story Masonry Houses: Part 1 and 2. Journal of Structural Engineering, Vol. 116, No. 1: 235-274. Jan 1990.
5. Griffith, M., Lam, N., Wilson, J., and Doherty, K. Experimental Investigation of Unreinforced Brick Masonry Walls in Flexure. Journal of Structural Engineering, Vol. 130, No. 3: 423-432. March 2004.
6. Simsir, C., Aschheim, M. and Abrams, D., Out-Of-Plane Dynamic Response of Unreinforced Masonry Bearing Walls Attached to Flexible Diaphragms. 13th World Conference on Earthquake Engineering. Vancouver, BC, Canada. 2004.
7. Doherty, K.T. An Investigation of the Weak Links in the Seismic Load Path of Unreinforced Masonry Buildings, Ph.D. Thesis, Department of Civil and Environmental Engineering, University of Adelaide, Australia. 2000.
8. Adams, J., and Atkinson, G. Development of Seismic Hazard Maps for the Proposed 2005 Edition of the National Building Code of Canada. Canadian Journal of Civil Engineering, Vol. 30: 255-271. 2003.