SHAKE TABLE STUDY ON OUT-OF-PLANE DYNAMIC STABILITY OF UNREINFORCED MASONRY WALLS

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ABSTRACT
The vulnerability of unreinforced masonry (URM) buildings to out-of-plane damage and collapse has been clearly demonstrated in past earthquakes, most recently in the 2010 and 2011 earthquakes near Christchurch, New Zealand. A cost-effective, widely-used approach for reducing the out-of-plane vulnerability of URM walls is to connect the walls to the diaphragms. Given sufficient anchorage to the diaphragms, a URM wall subjected to out-of-plane inertial forces will likely develop a horizontal crack above mid-height. This crack will cause the wall to behave as two semi-rigid bodies, which rock in the out-of-plane direction. Past studies have demonstrated that the out-of-plane stability of a URM wall connected to the diaphragms can be related to the height to thickness ratio and the spectral acceleration at 1.0 second. However, treatment of the effect of diaphragm flexibility on out-of-plane wall stability in studies to date has been limited.

This paper describes an experimental study examining the out-of-plane stability under seismic loading of URM walls connected to flexible diaphragms. Five full-scale unreinforced solid clay brick wall specimens spanning one storey were subjected to earthquake ground motions on a shake table. The top and bottom of the walls were connected to the shake table through coil springs, simulating the flexibility of the diaphragms. The apparatus allowed the wall supports to undergo large absolute displacements, as well as out-of-phase top and bottom displacements, consistent with the expected performance of URM buildings with unretrofitted timber diaphragms. Variables examined included diaphragm stiffness and wall height. Experimental results are compared with output from an analytical rigid-body rocking model.

KEYWORDS: unreinforced masonry, wall, shake table, out-of-plane, diaphragm, stability, earthquakes, seismic retrofit

INTRODUCTION
Buildings with unreinforced masonry (URM) walls have experienced considerable damage in past earthquakes. The typical damages that URM buildings suffer include: collapse of parapets or gables, diagonal shear failure or sliding shear failure of in-plane walls, and out-of-plane failure. Of these failure modes, out-of-plane wall failures pose the greatest risk to the safety of the people inside and outside of the building, since this mode will result in collapse of the load bearing wall and partial or complete collapse of the building.

A fundamental assumption in this study is that out-of-plane walls are securely anchored to the floor or roof diaphragms at each level, since this is a simple, low-cost retrofit that greatly reduces the risk of out-of-plane wall collapse. Unanchored walls act as cantilevers about their base and are therefore highly vulnerable to collapse at low levels of seismic excitation. This study focuses on whether the installation of diaphragm-to-wall anchorage alone is sufficient to ensure adequate out-of-plane wall stability, or whether additional wall retrofit is required.

Floor diaphragms in URM buildings commonly consist of timber sheathing supported on timber framing. In smaller buildings, joists typically span directly between load-bearing URM walls, and are either supported on the ledge created by a change in the number of wythes between adjacent stories, or are embedded in cavities created in the walls for this purpose. In larger buildings, joists may be supported by heavier timber or steel members, and by steel columns in large open plan areas. Sheathing arrangements vary, and include either straight sheathing (perpendicular to the joists) or diagonal sheathing (typically at 45° to the joists), applied in either one or two layers. While the in-plane stiffness of such diaphragms varies depending on the configuration, in general the stiffness is very low, and the diaphragm response is dominated by shear deformation.

Under seismic loading in a simple URM building with walls connected to the diaphragms, the inertial forces from the out-of-plane walls are transferred through the floor diaphragms to the in-plane walls, which carry the forces to the foundation. Clearly, the response of the floor diaphragm in such a load resisting system will have a significant influence both on the displacement demands imposed on the out-of-plane walls as well as the loads induced on the in-plane walls. Should stiff, uncracked out-of-plane walls be spanning vertically between floor diaphragms, the response of such a system could be readily modeled using traditional methods. However, the 2-way interaction between cracked out-of-plane walls and flexible floor diaphragms is neither intuitively understood nor easily modeled using traditional methods.

A study currently under way at the University of British Columbia (UBC) intends to address this issue in greater detail through experimental and analytical means. Full-scale shake table tests were carried out on URM wall specimens using a testing apparatus which allows for the simulation of flexible diaphragm boundary conditions. Five wall specimens were successfully tested. This paper describes the setup and results of the experimental portion of the study, as well as preliminary analytical modeling.

TEST APPARATUS

The test apparatus (Figure 1) consists of steel frame components fastened to a uni-axial displacement-controlled shake table. When loaded into the apparatus, the base of each wall specimen is supported on a rolling steel carriage which travels on rails on top of the shake table, in the direction of motion of the shake table. This carriage is connected to the shake table using coil springs, which have been designed to simulate the 1st-mode in-plane response of a flexible floor diaphragm.

A stiff steel braced frame – representing the in-plane walls – of the same height as the wall specimen is fastened to the shake table. The table motion is transferred to the top of this frame with minimal amplification. The study thus assumes that the flexibility of the in-plane walls is negligible compared to that of the diaphragms. A second rolling steel carriage travels on top of this steel frame, and is connected to the frame with coil springs identical to those at the base. The top of the wall is connected to this carriage, thereby also simulating the response of a flexible diaphragm at the top of the wall. Both the top and bottom carriages can be ‘locked out’ by fastening the carriages rigidly to the steel frame. In this case, the ground motion is applied directly to the wall – simulating the scenario of rigid diaphragms with stiff in-plane walls. The base and top of the wall are each constrained to match the horizontal displacement of the respective carriage, but are free to undergo rotation and uplift. Further details regarding the test apparatus are given in Penner and Elwood (2012).

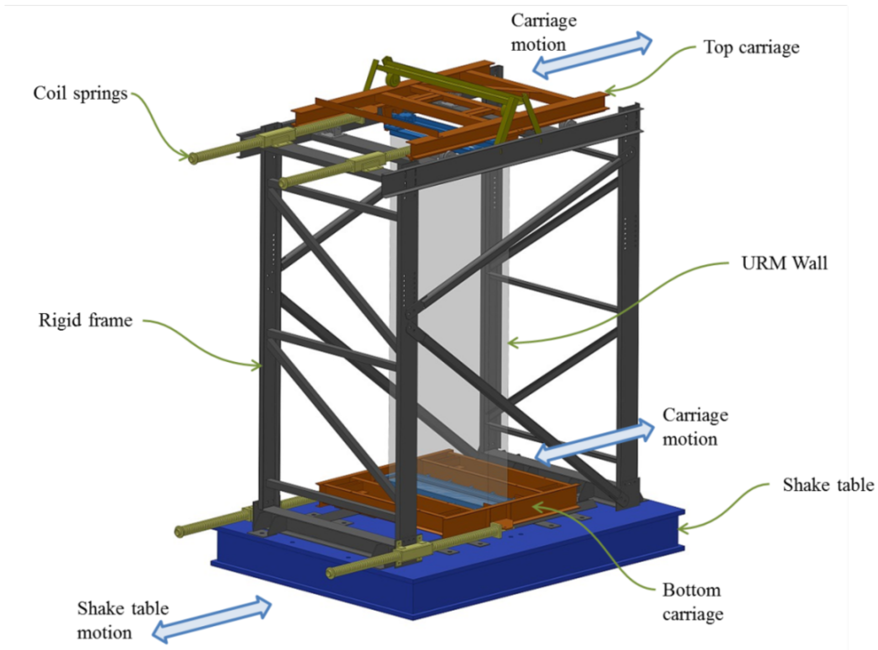

Figure 1: Graphical representation of test apparatus

TEST SPECIMENS

Five wall specimens were constructed by professional masons in the Earthquake Engineering Research Facility (EERF) at the University of British Columbia (UBC). Brick units were solid and measured 64x89x191 mm. The wall specimens were intended to represent a portion of a top-storey wall in an early 1900s load-bearing URM building in British Columbia. Type O mortar (1:2:9 cement:lime:sand) was considered an appropriate representation of existing URM building mortar quality because of its low compressive strength. Brick units were placed dry to further minimize the bond strength. Mortar compressive strength at the time of testing was approximately 4 MPa.

Four 3-wythe walls and one 2-wythe wall were constructed. American bond was used in all walls with a single header course at every sixth course (Figure 2). Wall dimensions and diaphragm test configurations are listed in Table 1. Each period was calculated using the mass of half the wall plus the respective carriage assembly.

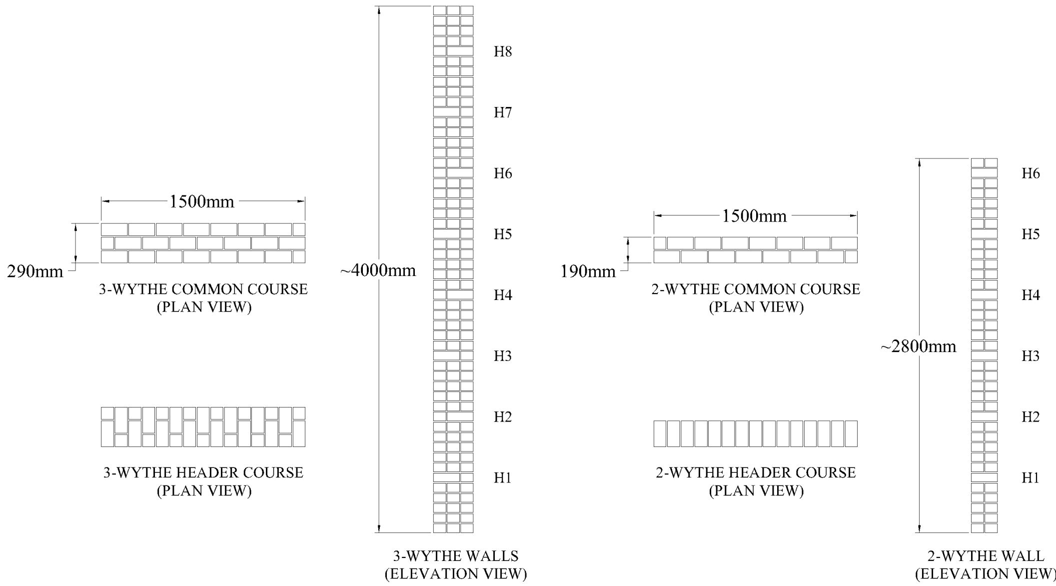

Figure 2: Test wall geometry

Table 1: Specimen geometry and test configuration

		FF-3*	FR-3*	FF-2*	SS-3*	RR-3*
Thickness	*[mm]*	291	291	191	300	296
Length	*[mm]*	1509	1500	1504	1518	1513
Height	*[mm]*	3947	3984	2790	3985	3973
Height/thickness	*[---]*	13.6	13.7	14.6	13.3	13.4
Mass	*[kg]*	3627	3614	1739	3833	3768
Mean density	*[kg/m^3]*	2095	2081	2176	2113	2118
Top stiffness	*[kN/m]*	37.0	37.0	37.0	146.9	∞
Bottom stiffness	*[kN/m]*	39.4	∞	39.4	142.4	∞
Top period	*[sec]*	1.63	1.62	1.28	0.83	0
Bottom period	*[sec]*	1.55	0	1.21	0.83	0

***** Specimens are named by diaphragm condition (F=flexible, S=stiff, R=rigid) at top and bottom, and the number of wythes in the wall specimen.

GROUND MOTIONS

Two ground motions were used as input to the shake table, with one motion selected for significant long-period spectral response and the other for a dominant short-period spectral response. The long-period motion selected (CHHC1) was recorded during the 22 February 2011 earthquake in Christchurch, New Zealand at the Christchurch Hospital. The short-period motion selected (NGA0763) was recorded during the 18 October 1989 Loma Prieta earthquake at the Gavilan College in Gilroy, California. Response spectra and displacement time histories of the two motions as recorded on the shake table are shown in Figures 3 and 4, respectively. Scale factors are shown relative to the original motion as recorded during the earthquake, and reference the amplitude of the displacement time history. It can be observed that the displacement control of the shake table results in significant response amplification at the natural frequency of the hydraulic system, producing a large response peak at a period of about 0.1 to 0.15 seconds. The effect of this amplification may be notable for runs in which the carriages were 'locked out'; however, for runs in which the carriages were driven through the springs, this amplification was filtered out due to the much longer natural period of the spring-carriage-wall system.

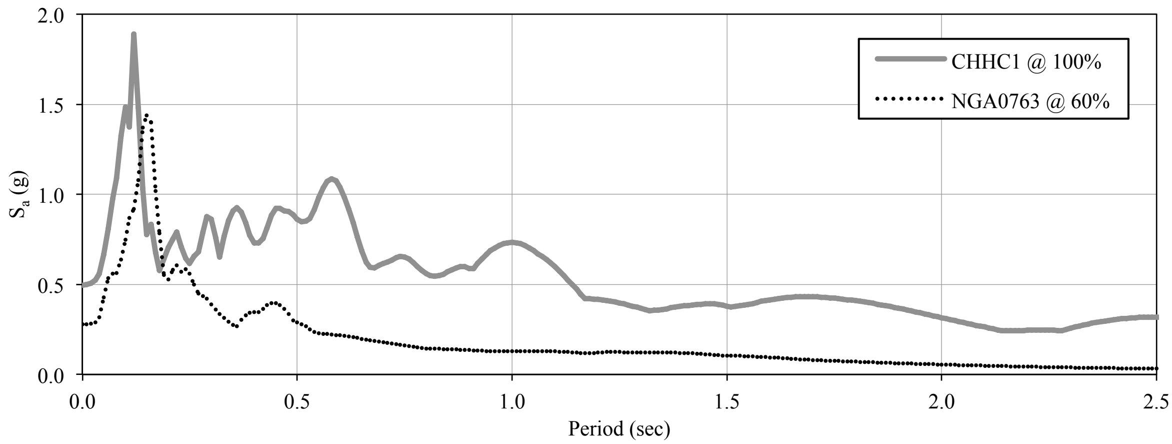

Figure 3: Response spectra of shake table motions

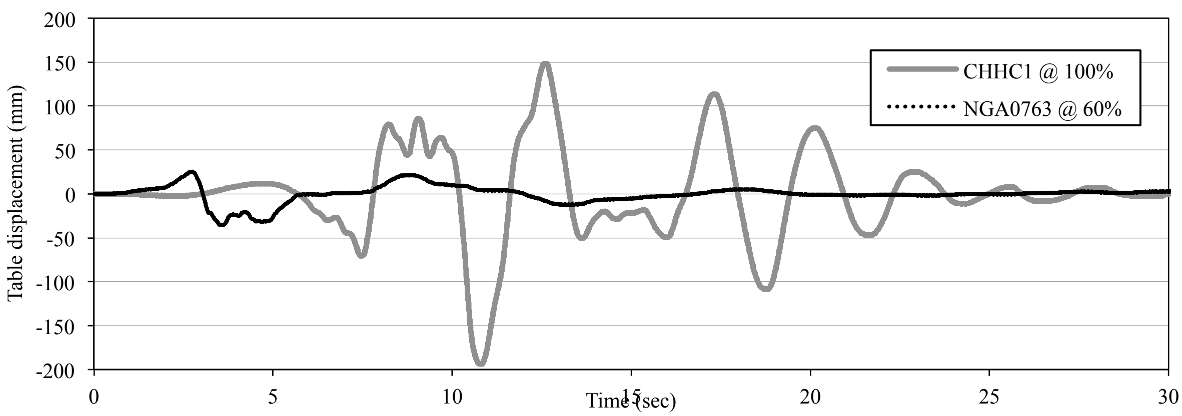

Figure 4: Displacement time histories of shake table motions

TEST PROTOCOL

The mortar used in the construction of the test walls (Type O) is of significantly lower strength than that used in modern structural masonry. However, in particular the flexural bond strength of walls found in early 1900s buildings may be weaker still than that of the test walls. It was therefore decided not to rely on the cracking resistance of the test walls in assessing their dynamic stability on the shake table, but rather to assume that the walls would experience cracking at very low levels of excitation. To ensure that walls would remain stable after crack initiation, allowing further tests to be carried out, cracking was initiated by running the NGA0763 motion with both top and bottom carriages locked out (rigid diaphragm conditions). After cracking was achieved, the carriage connections were adjusted for the desired diaphragm conditions, and subsequent runs were made using the CHHC1 motion at increasing amplitude of input motion until collapse was observed.

RESULTS OF DYNAMIC TESTING

Prior to undergoing significant rigid-body rocking, cracks were visually nearly imperceptible. In each wall, a single horizontal crack occurred. In every case, the crack occurred at the brick-mortar interface. In Walls FF-3, FR-3, and SS-3, the crack was located in a single horizontal plane across the entire wall section. In Walls FF-2 and RR-3, the crack stepped down by 1 course. Cracks occurred both at header courses and at common courses. Even after sustained rocking in later runs, all cracks consistently closed up without any horizontal offset and with minimal spalling of mortar or brick.

Rocking displacement is defined as the difference between the measured horizontal displacement of the wall at the crack height and the straight-line interpolation between the top and bottom of the wall at the same height. In Figure 5, the peak rocking displacement from each run is shown relative to the intensity of the ground motion in that run, with the rocking displacement normalized to the wall thickness. The static instability limit can be defined as the point when the normalized rocking displacement is equal to 1.

Wall FR-3, with rigid bottom diaphragm condition, underwent limited rocking in all runs prior to the collapse run despite large displacements (up to 70% of the wall thickness) of the top flexible diaphragm springs. This is in contrast to the other specimens, which in general underwent significant rocking excursions in runs prior to the collapse run. Note the larger than typical increase in intensity between the final two runs of Wall FF-3 – indicating that the collapse intensity for FF-3 may be slightly lower than that shown in Figure 5. Pertinent summary data for each specimen are shown in Table 2.

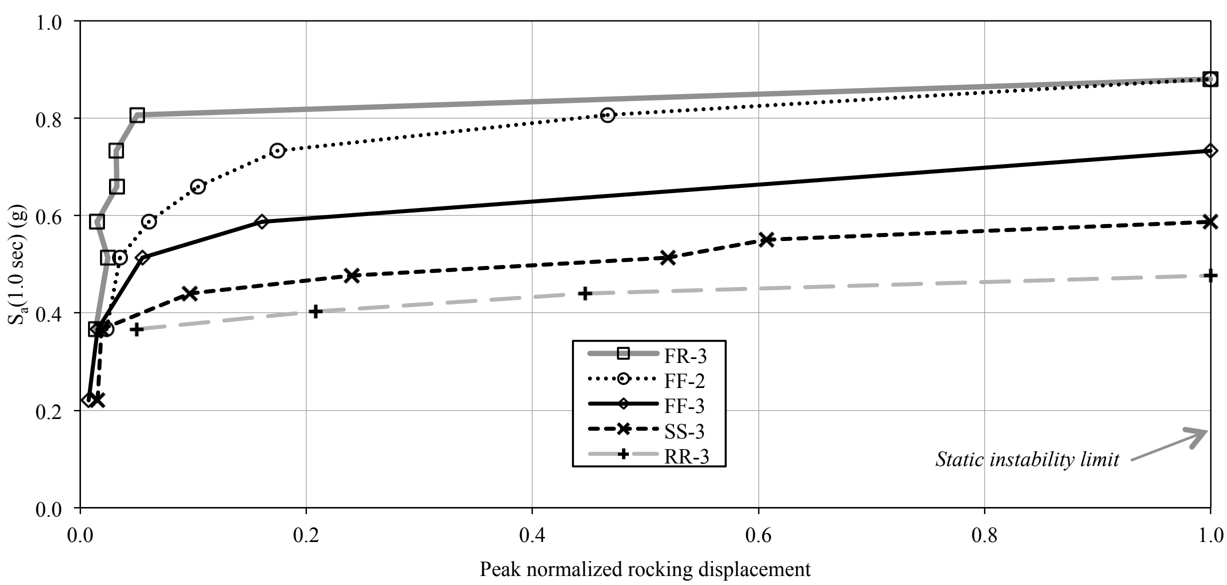

Figure 5: Rocking displacement vs. intensity of table excitation

Wall RR-3 (rigid diaphragms) exhibited lower stability than any wall with a flexible diaphragm condition. While some portion of this effect may likely be attributed to the rigid diaphragm condition, it is important to note that this wall cracked at a height of 0.74 times the wall height, while all other walls cracked between 0.47 and 0.55 times the wall height. This discrepancy was not intentional – the procedure used to obtain the crack was identical between all walls – but it may have had a notable effect on the wall's stability. This result points to the randomness associated with the dynamic response of brittle systems.

Table 2: Summary of Dynamic Test Results

				FF-3	**FR-3**	**FF-2**	**SS-3**	**RR-3**
Cracking run	Normalized crack height		*[---]*	0.47	0.55	0.49	0.51	0.74
	Peak total wall inertia force*		*[kN/kN]*	0.46	0.51	0.66	0.49	0.52
Highest stable run	Motion scale		*[---]*	0.80	1.10	1.10	0.75	0.65
	Peak normalized rocking displacement		*[---]*	0.16	0.05	0.47	0.61	0.45
	Peak carriage displacement	Top	*[mm]*	157	206	102	61	0
		Bottom	*[mm]*	180	0	142	91	0
		Differential	*[mm]*	73	206	74	52	0
	Peak wall rotation	Top	*[deg]*	2.3	2.8	4.7	5.3	6.7
		Bottom	*[deg]*	0.4	3.2	2.5	5.0	2.6
	Peak total wall inertia force*		*[kN/kN]*	0.26	0.31	0.41	0.45	0.31
Collapse run	Motion scale		*[---]*	1.00	1.20	1.20	0.80	0.65

* Wall inertia forces are normalized with respect to the total wall weight.

Carriage displacements up to 206 mm (70% of the wall thickness) were recorded without causing wall collapse. In runs where neither carriage was locked in the rigid condition, peak differential displacements between top and bottom carriages varied between 52 and 74 mm. The peak rotation of the wall segments from vertical varied between 0.4 and 6.7 degrees.
Force demands imposed on the wall-to-diaphragm connections are of interest to engineers involved in building assessment and retrofit design. The total connection demand (sum of top and bottom) was determined based on the measured wall accelerations. The acceleration measured on the wall at each header course was multiplied by the wall mass tributary to the elevation of that course. Data from either 8 (for the 4 m high walls) or 6 (for the 2.8 m high wall) accelerometers were used in this calculation.

Connection force demands were consistently higher for cracking runs as opposed to post-cracking runs, as shown in Figure 6. In cracking runs, peak normalized demands varied between 0.46 and 0.66 kN/kN. In post-cracking runs, peak demands varied between 0.26 and 0.45 kN/kN, with walls FF-2 and SS-3 producing the highest demands.

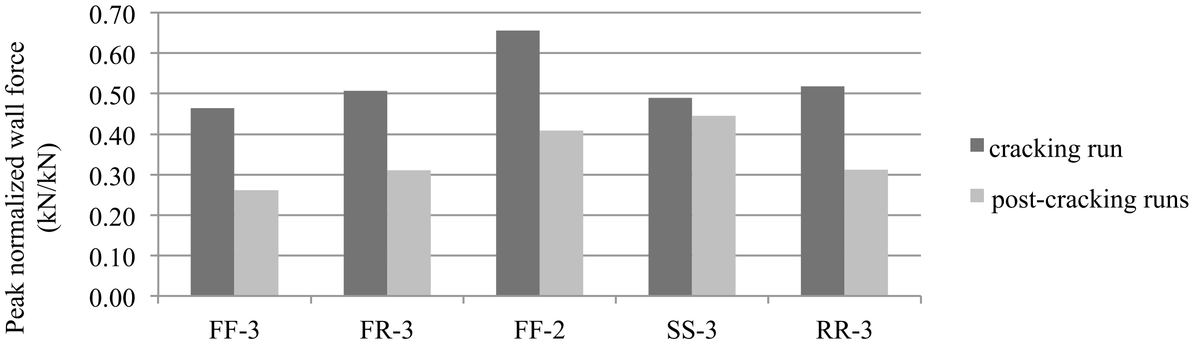

Figure 6: Peak normalized total wall inertia force

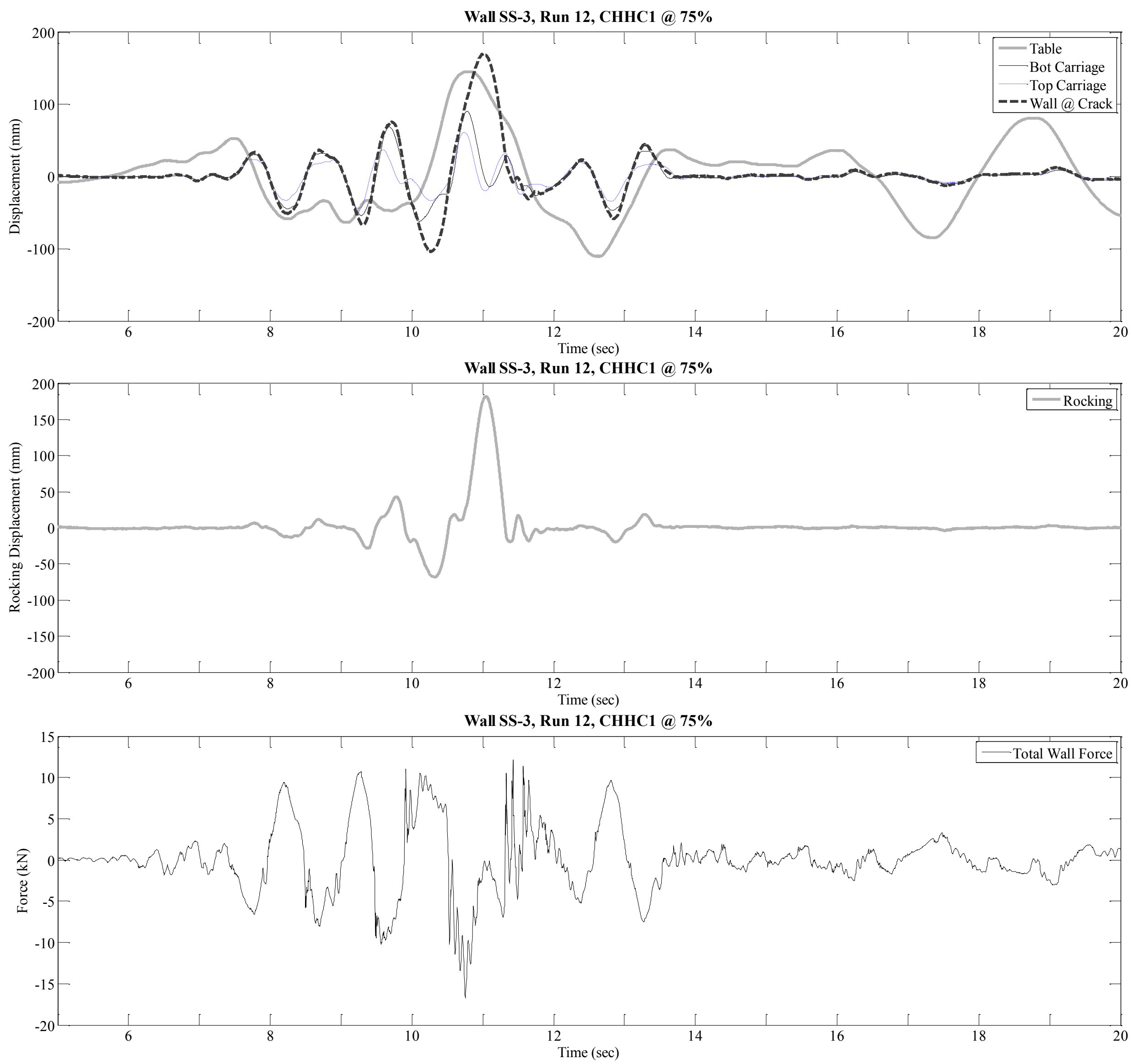

Figure 7: Time series for Wall SS-3, CHHC1 @ 75%

Figure 7(a) shows time series of the shake table displacement (absolute reference) and of the carriages and the wall at the crack height (relative to the shake table), for the last stable run of wall SS-3. Rocking displacement is shown in Figure 7(b), and total connection demand is shown in Figure 7(c).

ANALYTICAL MODEL

Two basic categories of analysis models are available to assess the dynamic out-of-plane behaviour of URM walls: stiffness-based models and rigid-body rocking models. Single degree of freedom (Doherty et al. 2002) and multi-degree of freedom (Simsir et al. 2004) stiffness-based models have been developed to predict the out-of-plane response of URM walls, where a nonlinear-elastic hysteretic model is used to represent the rocking behavior after cracking. Sharif et al. (2007) proposed a rigid body rocking model calibrated to shake table tests by Meisl et al (2007).

The model proposed by Sharif et al. (2007), using the software Working Model 2D, was modified in the current study to account for the influence of flexible diaphragms. The diaphragms were represented by mass-spring-damper units connected between the rigid shake table frame and the wall specimen, as shown in Figure 8. The top diaphragm mass was constrained to travel in a horizontal plane, such that, consistent with the test apparatus, it did not place any overburden load on the wall.

Boundary conditions in the model were detailed to represent as accurately as possible the conditions in the test apparatus. A vertical slot element allowing unrestrained vertical movement is placed in the top diaphragm, connected to a pin element at the top of the wall. At the base of the wall, a rigid link connects each side of the wall to a point further out on the carriage. Only one of these two links is active at any given time step, depending on which way the wall is rocking. A similar constraint is imposed at the crack interface. This configuration of the crack interface was necessary to prevent sliding of the top wall block relative to the bottom wall block (relative sliding between blocks was not observed during testing).

The effect of spalling at the crack and at the base of the wall was included by modeling the wall blocks as rectangles with corners chamfered at 45°. Chamfers of 2 mm at the crack interface and 10 mm at the base of the wall were used in all simulations. The crack chamfer is consistent with visual observations during testing (very limited spalling was observed), and the base chamfer was calibrated empirically since visual observation during testing was not possible. Damping values for the diaphragm springs were calibrated empirically and were set at 8% for the 3-wythe walls and 12% for the 2-wythe wall, relative to the combined wall and diaphragm mass.

The model uses a direct-integration solution method (Design Simulation Technologies, 2010). A time step of 0.005 seconds was used for runs with flexible links to carriages. Where carriages were locked out, stability issues required dropping the time step to 0.0025 seconds. The recorded shake table motion from each run was used as the ground motion input in the model. When using a time step of 0.005 seconds, the model runs roughly in real time.

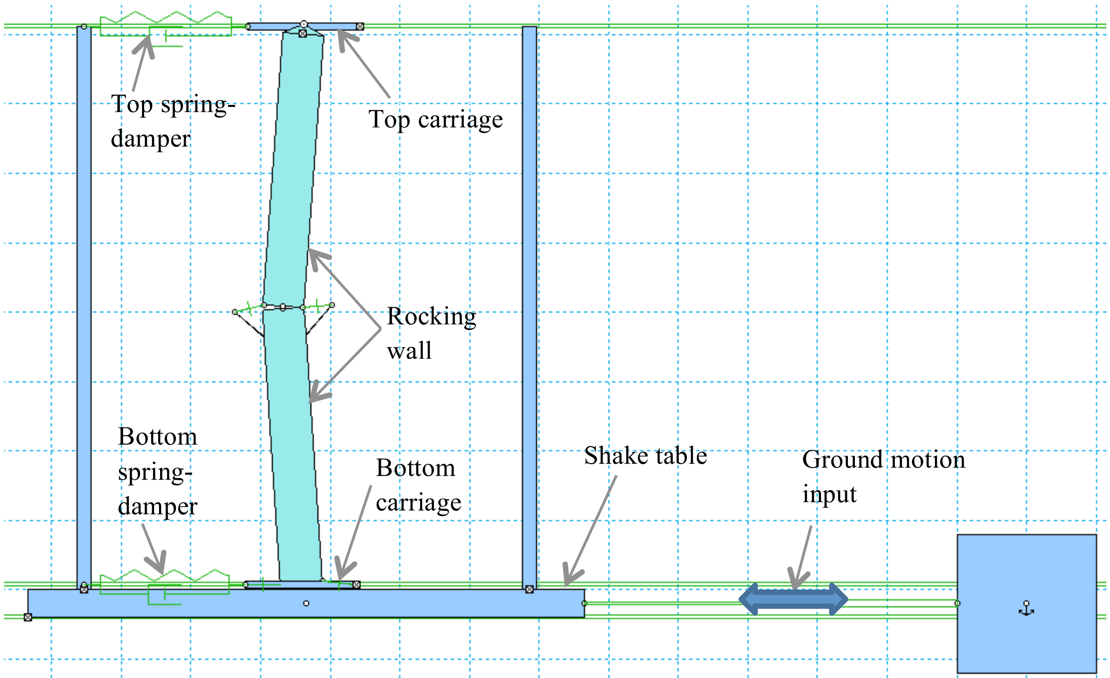

Figure 8: Configuration of analytical model

The simulation result of primary interest is the peak rocking displacement and the ground motion intensity causing collapse. Both of these quantities were reproduced well by the model, as shown in Figure 9. Here, solid lines indicate the test results and dashed lines indicate the simulation results. In addition, the chaotic and non-periodic time-history of the rocking displacement was reproduced well by the model, an aspect in which stiffness-based models typically struggle. Example rocking and top carriage displacement time history traces are shown in Figure 10 for Wall FF-3, and in Figure 11 for Wall RR-3. These runs are identified on Figure 9 by arrows.

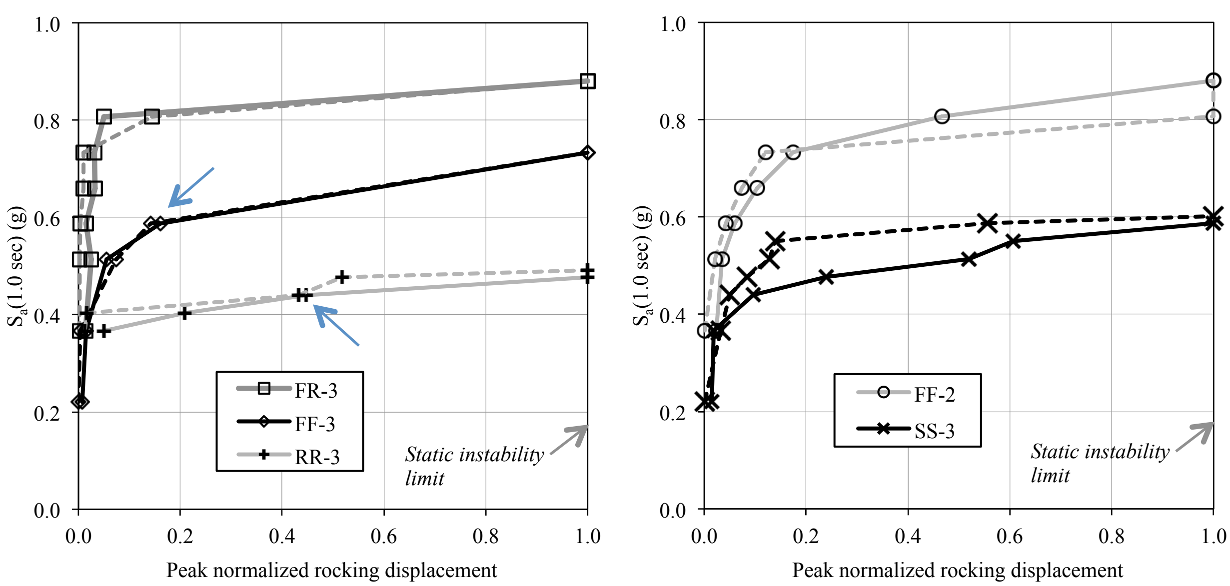

Figure 9: Modeled and recorded peak rocking displacements

(solid lines = test data; dashed lines = simulation data)

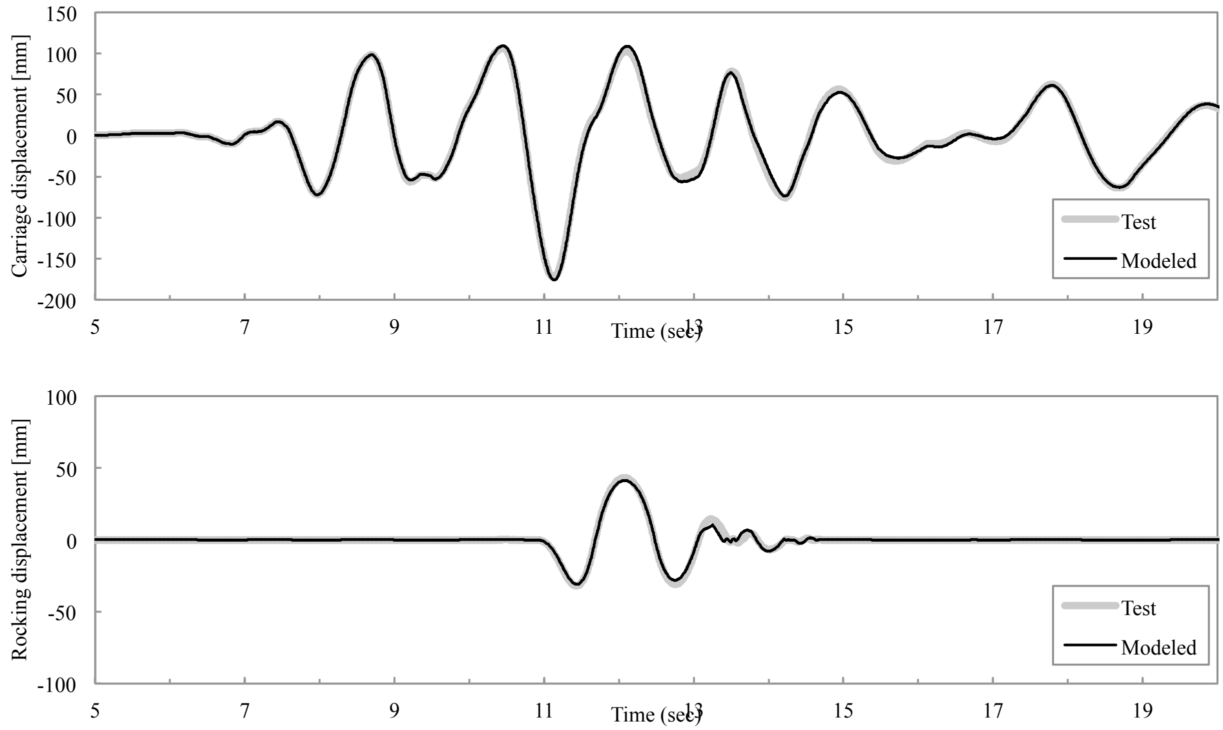

Figure 10: Modeled and recorded top carriage and rocking displacements
Wall FF-3, $S_a(1.0s) = 0.59g$ (CHHC1 @ 80%)

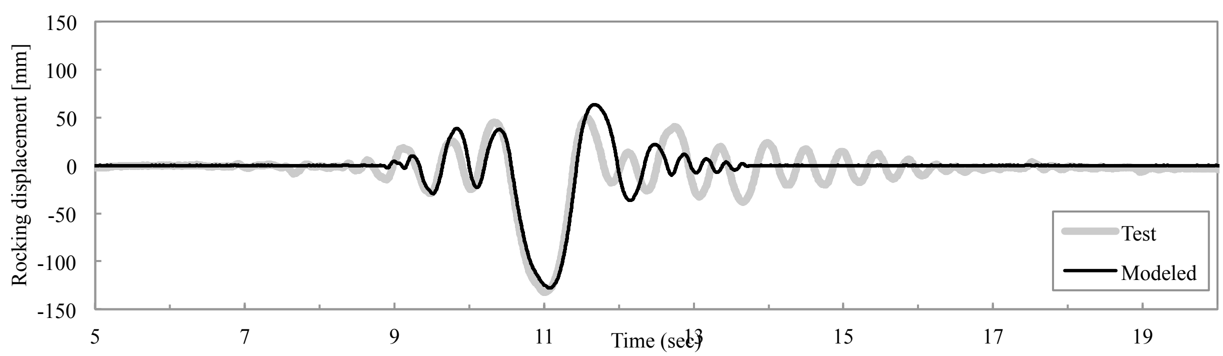

Figure 11: Modeled and recorded rocking displacements
Wall RR-3, $S_a(1.0s) = 0.44g$ (CHHC1 @ 60%)

CONCLUSIONS AND FUTURE WORK

The experimental portion of the study has produced an important dataset of out-of-plane URM wall performance with flexible diaphragm boundary conditions. The test apparatus created a simplified representation of real-world boundary conditions. A rigid-body analytical model was validated using the test results; the model was able to reproduce the peak rocking displacements, intensity of ground motion causing collapse, and the time history of the chaotic rocking motion reasonably well for the full range of tested boundary conditions.

Only preliminary conclusions can be drawn from the portion of the study carried out to date – only one ground motion was used, a limited number of boundary conditions were tested, and some variables, such as crack height, were not consistent between all specimens. Nevertheless, the testing showed that diaphragm displacements up to 200mm, including differential displacements between top and bottom of the wall, were mobilized without causing wall collapse. In addition, for the particular conditions in the test, walls connected to soft diaphragms were more stable than walls connected to stiff or rigid diaphragms. Finally, it was observed that the transition from limited rocking to collapse may occur very suddenly for some boundary conditions; this should be carefully considered in any future revisions to acceptance criteria for out-of-plane URM walls.

The final phase of the research program will consist of a parametric study using the analytical model intended to address supplement the limited test dataset. A large suite of ground motions will be run on a set of models with varying configuration and boundary conditions. Additional boundary conditions not implemented in the tests, for example overburden load, eccentricity of the overburden load, and rotational resistance at the wall-diaphragm connections may be considered. The results of this parametric study will be considered in the evaluation and possible revision of acceptance criteria in seismic rehabilitation standards (e.g. ASCE 2006).

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