9TH CANADIAN MASONRY SYMPOSIUM

INFLUENCE OF DIAPHRAGM FLEXIBILITY ON THE OUT-OF-PLANE RESPONSE OF UNREINFORCED MASONRY WALLS

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ABSTRACT

The paper concerns the seismic response of masonry buildings—in particular, the effects of diaphragm flexibility on the dynamic response of unreinforced masonry walls responding out of plane. Previous static and dynamic studies of out-of-plane response resulted in midheight or rocking collapse of the walls. These modes of failure were enforced by the test setups. No such failures resulted in the present tests, which were conducted on the earthquake simulator (shake-table) at the University of Illinois. In these tests, inertial loads were applied to the out-of-plane walls through the diaphragm element. Different diaphragm stiffnesses and various earthquake ground motions were used. Results from this ongoing experimental study as well as an analytical method to determine the dynamic stability of the out-of-plane walls are reported.

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Keywords: Unreinforced masonry, dynamic testing, out-of-plane wall, diaphragm flexibility.

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INTRODUCTION

There is a concern for out-of-plane seismic response of structural systems with flexible diaphragms in the low-rise masonry building stock. This concern is greater in the Central and Eastern United States, where the majority of the masonry building stock is unreinforced, and was designed and constructed with little or no consideration for earthquake loads. Intra-continental seismic hazard exists in this region that has produced infrequent, large magnitude earthquakes in the past. The low rate of attenuation of seismic waves increases the risk of damage to structures in the Central and Eastern United States.

Out-of-plane failure of unreinforced masonry (URM) walls around the world is a common phenomenon during earthquakes, even during those of moderate size. Figure 1 shows the failure of the second story 200 mm (8 inch) thick unreinforced solid brick masonry walls of a commercial building in Coalinga, California in the 1983 Coalinga Earthquake. Figure 2 illustrates the partial collapse of a wall at the top story of a URM building in Seattle during the 2001 Nisqually Earthquake.

Figure 1. Out-of-plane wall failure in the 1983 Coalinga Earthquake.
Source: www.eerc.berkeley.edu/bertero/html/

Figure 2. Out-of-plane wall failure in the 2001 Nisqually Earthquake.
Source: AP Photo/Elaine Thompson

Laboratory tests of URM walls responding out of plane have been performed in past static and dynamic studies. In the static out-of-plane tests by Yokel and Dickers (1971), the main objective was to study the effects of wind load; the lateral load was applied as a distributed pressure by means of inflated air bags. Clay brick as well as concrete block specimens were tested. Base and Baker (1973), West et al (1973), Yokel and Fattal (1976), and West et al (1977) carried out similar tests on wall panels that were simply supported at the top and bottom, and loaded axially at the top. These researchers all reported a flexural mode of failure at the mid-height of the walls. Anderson (1984) reported on tests with various support conditions; a stabilizing moment at the base of the wall, attributed to the eccentricity of the vertical load resulting from rotation of the wall relative to its support, was found to increase the lateral resistance of the wall.

The ABK Joint Venture (ABK 1981 and 1984), a team of Californian engineers, performed dynamic tests on reinforced and unreinforced masonry walls responding out of plane. The unreinforced clay brick and concrete block masonry walls had varying height-to-thickness ratios, and were axially loaded. Controlled displacement histories were applied dynamically by separate actuators at the top and bottom of each wall. Floor diaphragm flexibility was accounted for by amplifying the displacement histories applied at the top of the wall. Most of the URM walls displayed horizontal cracks approximately at their mid-heights and near the base well before failure, with collapse occurring as the mid-height cracks opened substantially. Several specimens also collapsed due to rocking about their bases. It was found that the URM wall panels could withstand accelerations well exceeding their elastic capabilities. Based on these tests, Kariotis et al (1985) and Adham (1985) identified allowable wall height-to-thickness ratios as a function of the overburden ratio (superimposed weight over wall weight) and peak input velocities at the top and base of the wall.

Bariola et al (1990) reported a series of dynamic tests on clay brick parapet URM walls. These walls cantilevered from their bases and had no axial load applied other than self-weight. A series of shake-table tests was performed on these walls. Wall height-to-thickness ratio was not found to have a clear influence on peak acceleration required to cause instability. For walls of the same height-to-thickness ratio but with different thicknesses, the thicker wall appeared to be more stable. Lam et al (1995) performed similar tests that complement Bariola's. Doherty (2000) carried out shake-table tests on clay brick URM walls supported at the top and bottom by a rigid frame. Specimens were axially loaded, initially concentrically. The axial load was forced to shift its position during the tests producing eccentric load on the walls, which failed at mid-height. Floor diaphragm flexibility was not represented in these tests.

Current seismic provisions for rehabilitation of existing buildings (FEMA 273 and 274, 1997) tabulate permissible height-to-thickness ratios for URM walls based on the potential for out-of-plane failure. The tabulated values are based on the work done by ABK but are tabulated as a function of design spectral accelerations. The design spectra are established based on maps developed by the USGS (1996) for the United States and are included in the International Building Code (2000). For design of new structures, IBC 2000 requires the height-to-thickness ratio not to exceed 18 for masonry bearing walls.

This paper reports on work in progress at the University of Illinois earthquake simulator (shake-table) sponsored by the Mid-America Earthquake Center. The research focuses on the out-of-plane response of URM walls to which inertial loads are applied through a diaphragm element. Described are the test specimens, test results, and comparisons of the results with simulations of the response. The mid-height cracking and mid-height failures observed in previous out-of-plane dynamic tests of URM walls did not develop in these tests; explanations for this difference are offered.

SPECIMEN DESCRIPTION

An idealized model masonry building was constructed on the earthquake simulator (shake-table) at the University of Illinois (Figure 3). The masonry walls were built from half-scale concrete hollow blocks with dimensions 203x102x102 mm (8x4x4 inch). Measured strengths of the unit blocks and other materials are given in Table 1.

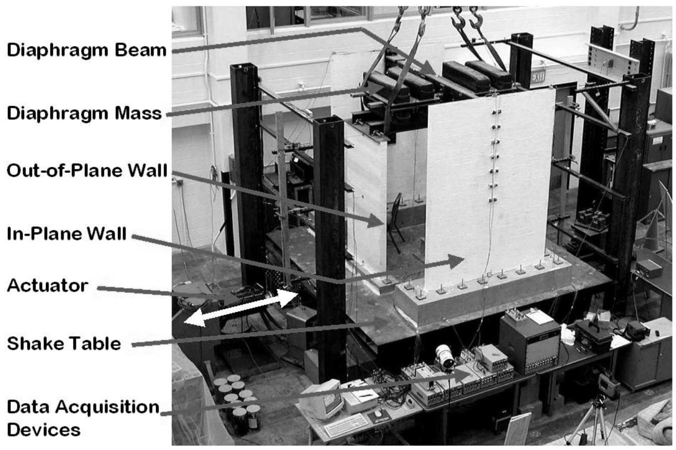

Figure 3. Specimen on shake-table.

The two out-of-plane wall panels are 1016 mm (40 inch) long, 2032 mm (80 inch) tall, and 102 mm (4 inch) thick (single wythe). These dimensions result in a height-to-thickness ratio of 20, the largest permissible value in FEMA 273 (1997). Type O mortar was used for the ungrouted out-of-plane walls, to reflect the weak materials in many existing unreinforced masonry (URM) buildings in the Central and Eastern United States.

Table 1. Material Properties

Material	Compressive Strength, MPa (psi)	Tensile Strength, MPa (psi)
Unit concrete block	12.76 (1850)	
Type O mortar	1.59 (230)	
Masonry prism (with Type O mortar)	10.55 (1530)	0.083 (12)
Masonry prism (with Type S mortar)	11.31 (1640)	
Grout	31.16 (4520)	

The two in-plane walls are 1829 mm (72 inch) long, 2540 mm (100 inch) tall, and 102 mm (4 inch) thick (single wythe). Based on masonry strength design, they are adequately reinforced with vertical and horizontal steel reinforcing bars to withstand an acceleration of 5 g of the diaphragm mass (3175 kg or 7 kips). The in-plane walls are grouted and are firmly anchored to the shake-table through reinforced concrete footings. Type S mortar was used. The in-plane walls simply provide a load path for inertial shear forces and are not the main object of the current investigation.

The diaphragm mass is supported vertically by the out-of-plane walls. The mass of 3175 kg (7 kips) was selected to develop an axial stress representative of a 3-story building. Ball bearings are mounted on to a steel plate that is anchored by steel studs to the grouted top course of the out-of-plane wall (Figure 4). Only the top course of the out-of-plane wall is grouted. The pin connection allows rotation at the top of the wall with respect to the diaphragm, while keeping the gravity load applied concentrically on the wall.

Figure 4. Diaphragm mass to out-of-plane wall connection detail.

The floor diaphragm is represented by an A36 steel beam with a rectangular hollow cross-section spanning 2540 mm (100-inch) between the two in-plane walls. Both stiff and flexible diaphragms were represented by installing different tube cross sections. The stiff diaphragm used a 305x102x6.4 mm (12x4x0.25 inch) tube in weak-axis bending; its stiffness was 5914 N/mm (33.77 kips/inch). The flexible diaphragm used a 203x51x4.76 mm (8x2x0.19 inch) tube in weak-axis bending, and had a stiffness of 651 N/mm (3.72 kips/inch). From free vibration tests, the structure was determined to have a natural period of 0.16 seconds with the stiff diaphragm, and 0.37 seconds with the flexible diaphragm. While the stiff beam would correspond to a concrete floor slab, the flexible beam characterized a single straight sheathed wood diaphragm whose stiffness is estimated from an equation offered in FEMA 273 (1997) for a real building 6.1 m x 21.3 m (20 ft x 70 ft) in plan:

$$\Delta = vL^4 / (G_d b^3) \tag{1}$$

In Equation (1), Δ is diaphragm deflection, v is maximum shear per unit length in the direction under consideration and equals to 1750 N/m (120 lbs/ft) at yield, L is diaphragm span between shear walls (21.3 m or 70 ft), b is diaphragm width (6.1 m or 20 ft), and diaphragm shear stiffness G_d = 35000 N/mm (200 kips/inch).

The diaphragm beam was connected to the in-plane walls by ball bearings mounted on a steel plate; the steel plate was anchored to the wall (Figure 5a). This “universal” steel plate was used so that the elevation of the diaphragm beam could be changed, to accommodate out-of-plane walls with different possible heights. The hinged connection allowed for rotation and vertical displacement but prevented sliding of the diaphragm beam with respect to the in-plane walls. The diaphragm beam to diaphragm mass connection (Figure 5b) transmitted inertial forces between the two. Slotted holes on the beam allowed for vertical movement of the mass with respect to the beam that could develop as the out-of-plane walls rock. In short, the test set-up was built to investigate the response of out-of-plane wall component as an integral part of the building system.

Figure 5. (a) Diaphragm beam to in-plane wall connection detail. (b) Diaphragm beam-to-diaphragm mass connection detail.

TESTS

A total of 20 runs on the shake-table were done. Table 2 presents for all runs the name of the ground motion record, the peak table acceleration, the type of diaphragm on the specimen, and the maximum measured displacement at the top of the out-of-plane wall relative to the table.

In between the runs tabulated in Table 2, frequency sweep tests were performed to determine the natural period of the structure, which often increased due to damage to the specimen. The sweep tests consisted of 2-second long table excitations by sinusoidal acceleration data at varying frequencies. The input sine waves had amplitude of 0.02 g, and the frequency producing the largest response was identified as the current natural frequency of the specimen. The range of fundamental periods observed over different runs is shown in Figures 6 and 7, together with the pseudo-acceleration response spectra of Nahanni and Big Bear Earthquakes. The spectra shown were computed for viscous damping equal to 1.4% of critical damping, which is representative of the damping values determined from the decay of the low amplitude responses induced in the frequency sweep tests.

Table 2. Shake-table runs.

Run Number	Record Name	Peak Table Acceleration, g	Diaphragm Type	Peak Displacement at the Top of the Out-of-Plane Wall, mm (inches)
1	Nahanni	0.057	Stiff	0.97 (0.038)
2	Nahanni	0.109	Stiff	1.70 (0.067)
3	Nahanni	0.149	Stiff	4.06 (0.16)
4	Nahanni	0.186	Stiff	5.33 (0.21)
5	Nahanni	0.267	Stiff	5.84 (0.23)
6	Nahanni	0.283	Stiff	5.84 (0.23)
7	Nahanni	0.339	Stiff	7.11 (0.28)
8	Nahanni	0.501	Stiff	7.87 (0.31)
9	Nahanni	0.674	Stiff	11.94 (0.47)
10	Nahanni	0.909	Stiff	15.24 (0.60)
11	Nahanni	0.248	Stiff	3.56 (0.14)
12	Nahanni	1.177	Stiff	14.22 (0.56)
13	Big Bear	0.374	Stiff	5.33 (0.21)
14	Big Bear	0.618	Stiff	9.40 (0.37)
15	Big Bear	recording error	Stiff	19.81 (0.78)
16	Big Bear	1.197	Stiff	18.54 (0.73)
17	Big Bear	0.134	Flexible	11.94 (0.47)
18	Big Bear	0.372	Flexible	33.02 (1.30)
19	Big Bear	0.616	Flexible	45.97 (1.81)
20	Big Bear	1.085	Flexible	65.53 (2.58)

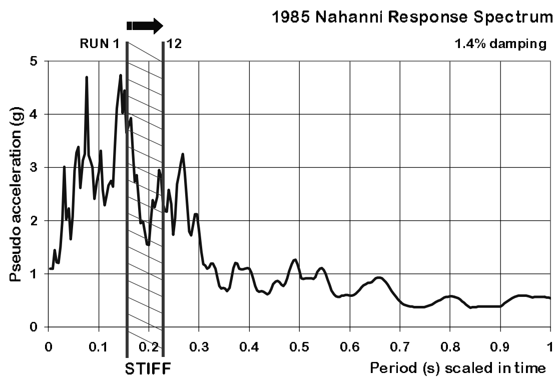

Figure 6. Shift in natural period for stiff specimen subjected to 1985 Nahanni Earthquake.

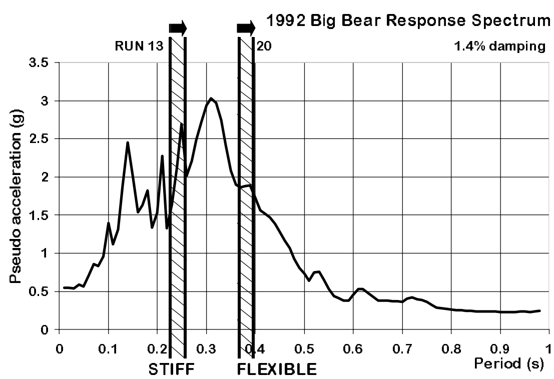

Figure 7. Shift in natural periods for stiff and flexible specimens subjected to 1992 Big Bear Earthquake.

Ground acceleration histories from 1985 Nahanni Earthquake (Northwest Territories, Canada) and 1992 Big Bear Earthquake (California) were utilized as input functions to the uniaxial shake-table. Nahanni possesses the characteristics of an intra-continental earthquake and may be representative of a future earthquake in Central and Eastern United States. On the other hand, the selection of Big Bear as input ground motion is based on its effectiveness to amplify the response. As seen in Figure 7, as the effective period of vibration may shift to higher values as nonlinearities develop, increasing spectral accelerations result for the stiff specimen with the Big Bear record, and reduced acceleration demands are computed for the flexible specimen. The reduced accelerations correspond to increased spectral displacements.

TEST RESULTS

The specimen was visually examined after each test. Horizontal cracks at the base of the out-of-plane walls were observed for the first time after 7th test run. These cracks, located between the bottom course of block and the concrete footings, became more evident in the subsequent runs. No cracks or failures occurred above the base of the out-of-plane walls at any time. The in-plane walls sustained diagonal shear cracks. A few of the steel reinforcing bars inside the in-plane walls exceeded their yield strain during the 15th run, and the flexible diaphragm beam yielded at mid-span during the 20th run.

Displacements at the top and mid-height of the out-of-plane wall, relative to the table, are plotted in Figure 8 for the 20th run. In this and the other runs, the mid-height displacements were in phase with and approximately one-half of the displacements measured at the top of the out-of-plane wall, indicating nearly rigid-body rocking of the wall about its base. The largest wall displacement response was obtained in the 20th run in which the flexible diaphragm forced the out-of-plane walls to displace as much as 65.5 mm (2.58 inch), corresponding to a 3.2% story drift. A residual drift of 2.5 mm (0.1 inch) was present on the wall after run number 20. FEMA 306 (1999) relates damage due to out-of-plane flexural response of URM walls to FEMA 273 (1997) performance levels. According to this relation, the slight damage observed in the test specimen would correspond to an Immediate Occupancy performance level, even though such large story drifts imply Life Safety or Collapse Prevention demand levels.

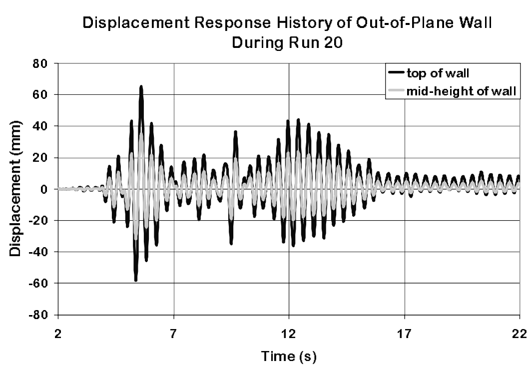

Figure 8. Displacement response history from Run 20.

The effect of diaphragm flexibility on the out-of-plane displacement response is apparent in Figure 9, which plots the peak displacements at the top of the out-of-plane wall for each run. As an example, Runs 14 and 19 indicate that the peak displacement of the flexible diaphragm specimen (46.0 mm) is approximately five times larger than that of the stiff diaphragm specimen (9.4 mm), for the same ground motion record (1992 Big Bear, with PGA=0.62g). This results from the difference in spectral displacements associated with the periods of vibration of the flexible- and stiff-diaphragm structures.

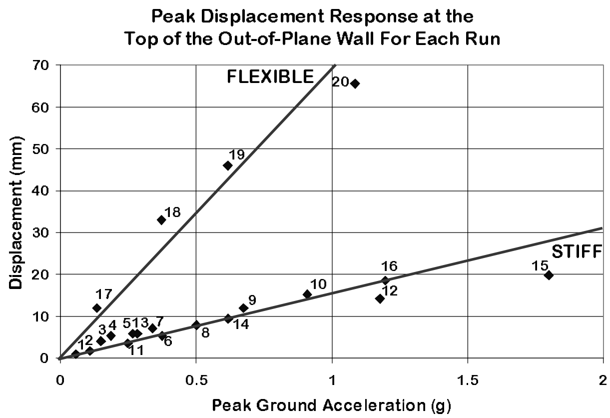

Figure 9. Peak displacement response from all runs.

ESTIMATING THE PEAK DISPLACEMENT

The response of the structure may be estimated using an “equivalent” SDOF model. The spring in Figure 10 incorporates the stiffnesses of the diaphragm and the in-plane wall, which may be modeled as two separate springs connected in series. The diaphragm mass and tributary wall mass is lumped at the end of the spring. The wall is idealized as a rigid body, rocking about its base, even at relatively small excitation amplitudes. Second-order (P-Delta) effects may be considered in this model, but were not included in the present calculations. Response was computed using the program USEE (Inel et al, 2001). A linear elastic model was adequate for the stiff specimen, while a bilinear model was used for the flexible specimen, since the diaphragm beam yielded. Mass, stiffness, strength, and damping characteristic of measured values were used in the ESDOF model.

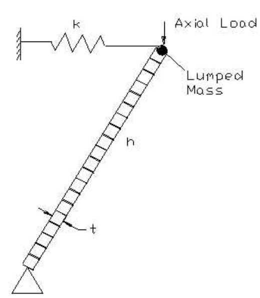

Figure 10. Equivalent SDOF model.

Estimated peak displacements from the equivalent SDOF analysis are compared with the measured values from tests in Figure 11. Standard error between the computed and the measured response is calculated as 1.42 mm (0.056 inch). The correlation coefficient is 0.9970 and the equivalent SDOF model estimates the peak displacements accurately.

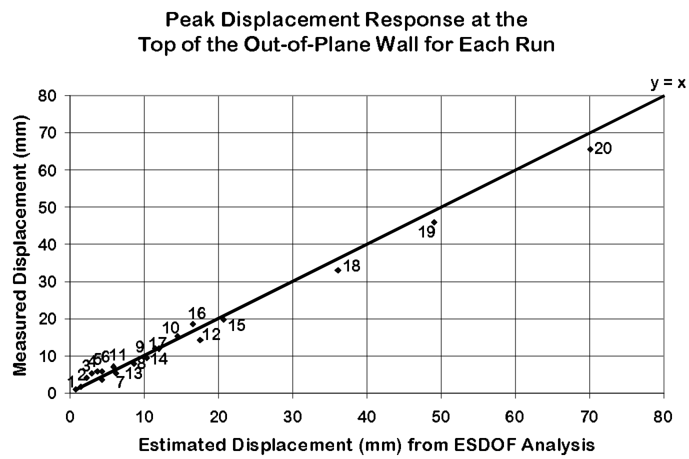

Figure 11. Correlation between measured and computed response.

CONCLUSIONS

Work is in progress at the University of Illinois where experiments have been carried out on the shake-table to investigate effects of diaphragm flexibility on unreinforced masonry (URM) walls responding out of plane. In these experiments, capable connections prevented sliding or pullout of the diaphragm relative to the masonry walls. The unreinforced out-of-plane walls were discrete elements, not integrally connected to the reinforced masonry in-plane walls. Uniaxial excitations applied to develop out-of-plane response did not simultaneously develop in-plane shear. Under these conditions, the following results have been obtained:

1. The present test series utilized rigid and flexible diaphragms to load the out-of-plane walls with inertial forces on a shake-table. Unlike previous tests that used different setups for loading the walls, no mid-height collapses resulted with the present setup.

2. At peak drifts of 3.2%, the only damage apparent were minor cracks at the base of the wall resulting from rocking of the walls at their bases. Residual drifts were negligible (0.13%). The observed damage corresponds to Immediate Occupancy performance. To reach Life Safety and Collapse Prevention demand levels, more substantial damage, such as mortar spalling and out-of-plane offsets at the cracks, would have to occur. Such damage was not observed in the present tests even though the peak drift exceeded 2.5%, a value generally associated with the Collapse Prevention performance level.

3. Diaphragm flexibility significantly increases the out-of-plane displacement response.

4. Peak displacements can be estimated with reasonable accuracy using a simple equivalent SDOF analysis, if the floor mass, viscous damping ratio, and stiffnesses of the diaphragm and in-plane walls are known.

5. Based on vulnerability in past earthquakes, adequate anchorage of URM walls to the diaphragm is key to a reliable load path and ensuring good performance of the walls. The out-of-plane performance of walls subject simultaneously to significant in-plane shear cannot be assessed with the present tests.

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