8th Canadian Conference on Earthquake Engineering / 8ième Conférence canadienne sur le génie paraséismique
Vancouver – 1999

Seismic Resistance of Full-Scale Single Story Brick Masonry Building Specimen

Jocelyn Paquette[1] and Michel Bruneau[2]

ABSTRACT

Analytical and experimental research is underway to investigate the seismic resistance of unreinforced brick masonry buildings. Analyses reported here demonstrate the dominant role of flexible diaphragms on structural response, and support the chosen pseudo-dynamic experimental set-up adopted for full-scale testing of one such building.

INTRODUCTION

The NRC Guidelines for the Seismic Evaluation of Existing Buildings (GSEEB) (NRC 1992) present a systematic procedure for the evaluation and seismic strengthening of unreinforced masonry (URM) bearing wall buildings having flexible diaphragms. This special procedure, adapted from one developed and used in the United States (ABK 1984, FEMA 1992, ICBO 1997) and described in details by Bruneau (1994a, 1994b), has made it economically possible to significantly reduce the seismic hazard posed by these buildings, as evinced by the considerably lesser damage suffered by seismically retrofitted buildings in recent earthquakes, compared to non-retrofitted ones (Bruneau 1990, 1995, Rutherford and Chekene 1991). However, even though this procedure is founded on extensive component testing, full scale testing of an entire 3-D building having wood diaphragms has not been conducted. Such a test would complement the computer simulations and small-scale shake table tests by other researchers conducted to better understand the flexible-floor/rigid-wall interaction and the impact of wall continuity at the building corners on the expected seismic behavior. This paper reports on the analytical studies conducted to establish how an adequate pseudo-dynamic test can be best conducted to achieve the above objectives, and on the details of a full-scale specimen constructed for that purpose. These analyses also demonstrate how flexible diaphragms can dominate structural response, and the consequences of inelastic diaphragm response on seismic behavior.

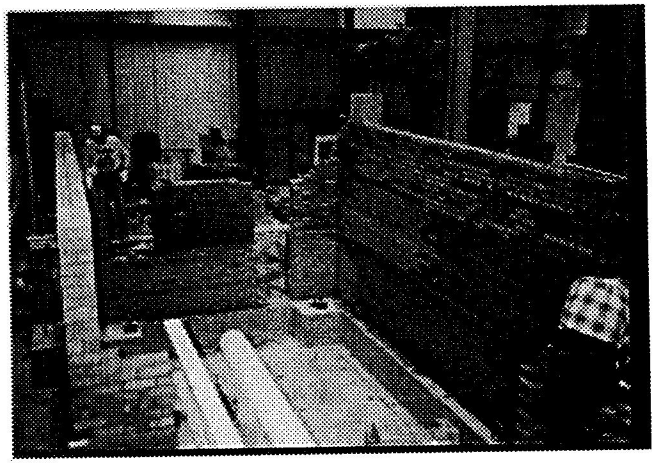
EXPERIMENTAL SPECIMEN

The single-story full-scale unreinforced brick masonry building constructed for this experimental program is shown in Fig. 1. This rectangular shaped building was constructed with two wythes solid brick walls (collar joint filled) and type O mortar was used to replicate old construction methods and materials. The specimen has two load-bearing shear walls, each with two openings (a window and a door). Shear walls were designed such that all piers would successively develop a pier-rocking behavior during seismic response. This rigid-body mechanism is recognized by the GSEEB to be a favorable stable failure mechanism. The specimen has a flexible diaphragm constructed with wood joists and covered with diagonal boards with a straight board overlay (Fig. 2). The diaphragm was anchored to the walls with through-wall bolts in accordance to the special procedure of the GSEEB (Fig. 3). Material properties were obtained from simple component tests, such as a three-point flexural bending test (Fig. 4) of a small beam in order to determine the tensile strength of the mortar used.

At the corners of the building at one of its ends, gaps were left between the shear wall and its perpendicular walls. At the other end, walls were continuous over the building corners. This permits a comparison between the plane models considered by many engineers and the actual behavior at the building corners, and allows to assess the significance of this discrepancy on seismic performance, particularly when piers are expected to be subjected to rocking. To some extent, this also permits to observe the impact of in-plane rotation of the diaphragm's ends on wall corners.

1 Ph.D. Candidate, Ottawa Carleton Earthquake Engineering Research Centre, Department of Civil Engineering, University of Ottawa, Ottawa, Ontario, Canada, K1N 6N5.

2 Deputy Director and Professor, Multidisciplinary Center for Earthquake Engineering Research, State University of New York at Buffalo, 105 Red Jacket Quadrangle, Buffalo, NY 14260



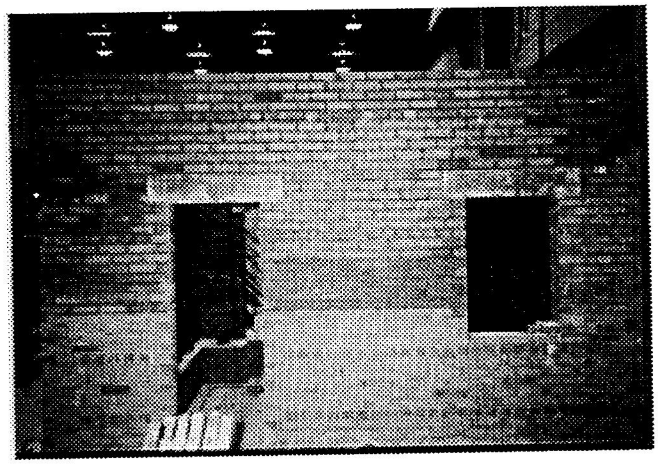


Figure 1: Full-scale URM specimen during (left) and after (right) construction.

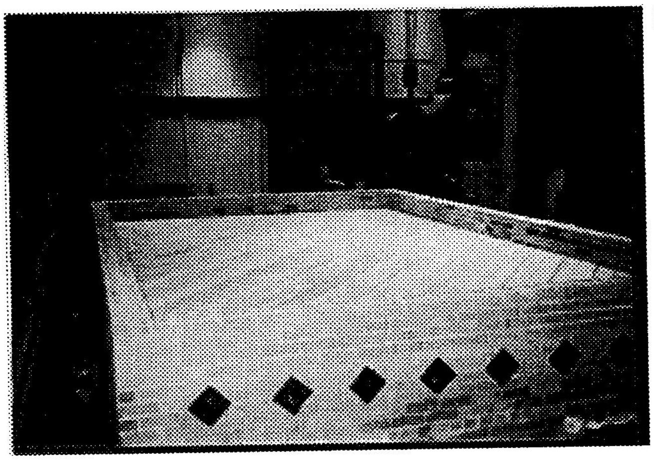


Figure 2: Anchored wood floor diaphragm

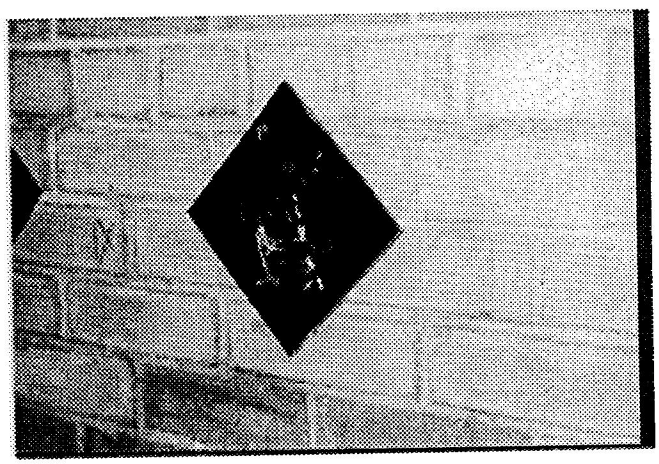


Figure 3: Typical anchor bolt and plate

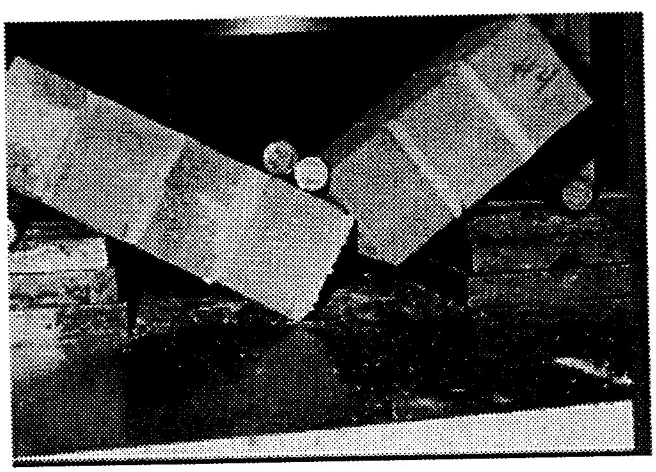


Figure 4: Three-point bending mortar tension-test

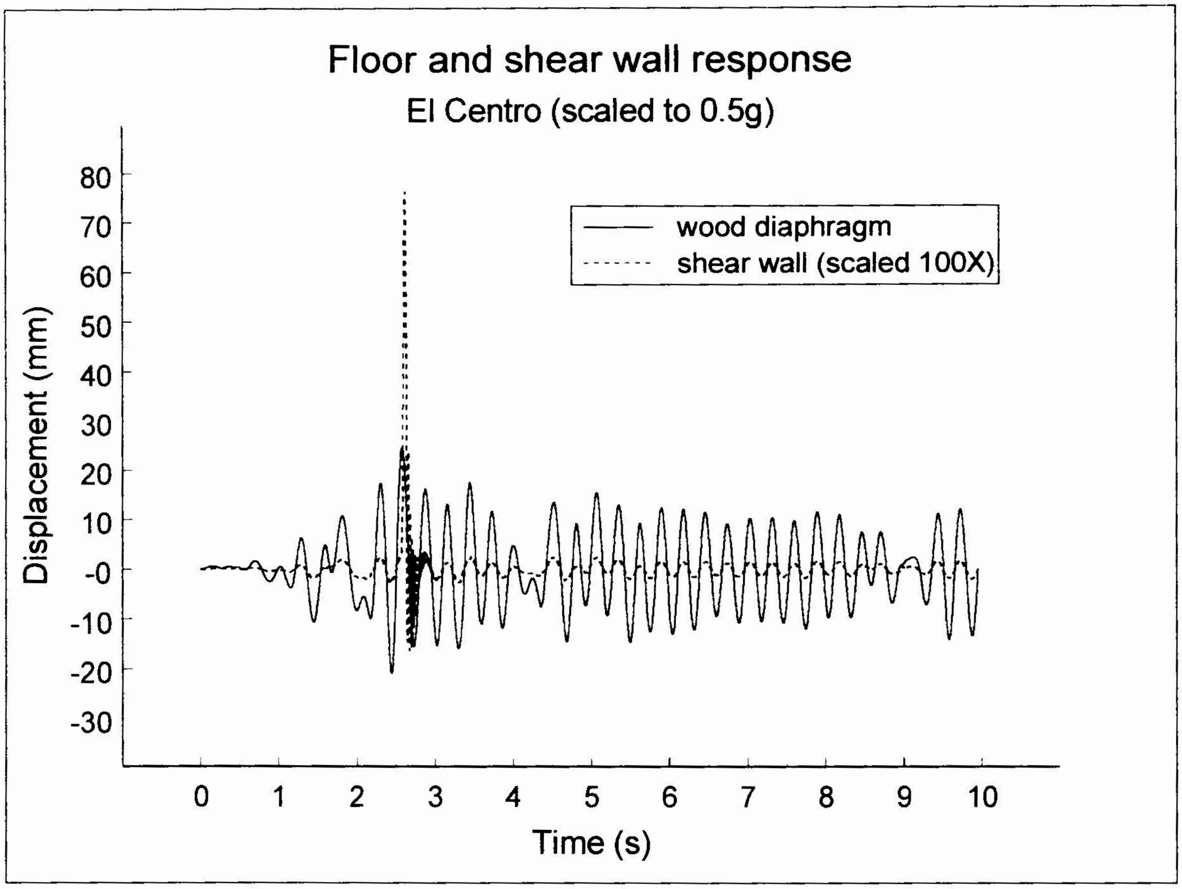


Figure 5: Comparison of wood diaphragm center-span response with magnified (100X) end-wall response

Further to a first series of test under earthquakes of progressively increasing intensity, and after full rocking of the shear walls has developed, the wood diaphragm is strengthen with plywood sheething to investigate the impact of different levels of flexible diaphragm inelastic response on the seismic behavior of the building. After this second series of tests, the shear walls are retrofitted using shotcrete on one shear wall, and fiberglass fibers strips on the other wall, and the building is re-tested. These tests will provide valuable information on both the seismic resistance of existing unreinforced masonry buildings having rocking piers and flexible yielding diaphragms, the adequacy of the existing models of these behavior, and the effectiveness of some seismic retrofitting methods. However, prior to testing, analytical work provided some valuable observations on expected seismic behavior, particularly on diaphragm response respective to wall response, and make possible to considerably simplify the originally planned pseudo-dynamic test set-up. These results are presented below.

ANALYTICAL RESULTS

Non-linear inelastic analyses are conducted to investigate the seismic behavior of the specimen one-story unreinforced masonry building. Note that the building was designed with a window and door opening on each end-walls (i.e. the walls stressed in-plane when subjected to the seismic excitation), and that all resulting piers of those walls behaved in a rocking mode after their flexural cracking (definitions of in-plane, out-of-plane, and other relevant terms, with illustrations of resulting damage, are presented in Bruneau 1994a, 1994b, and Bruneau and Lamontagne 1994). For all analyses reported here, flexural strength of the piers until tensile fracture along the mortar joint, prior to rocking, has been neglected. This

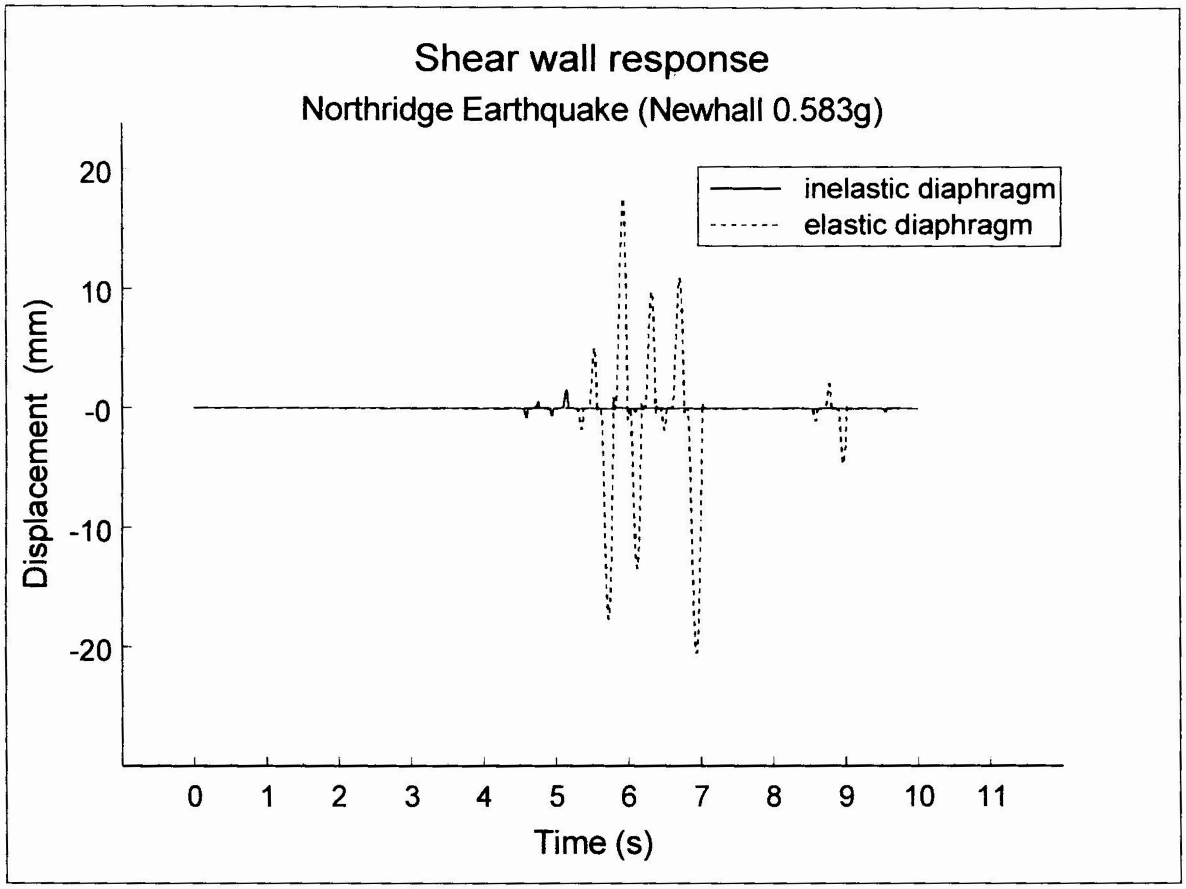


Figure 6: Comparison of end-wall response considering elastic or inelastic wood floor diaphragms

is a reasonable assumption because the mortar used to build the actual specimen has only a 0.18 MPa tensile strength, as revealed by component testing (Fig. 4). Note that rocking could start to occur at a higher threshold of peak-ground-acceleration if a very large mortar tensile strength existed (e.g. Costley and Abrams (1992) reported 2 MPa in their experiment).

For a first series of analyses, elastic floor diaphragm response was considered. For the El Centro earthquake, the entire structure remained elastic until 0.5g, when first occurrence of inelastic rocking was observed. Displacement time histories of the floor diaphragm at mid-span is compared with that at the top of the shear wall in Fig. 5 (note that in Figs. 5 to 7, only approximately 10 seconds from the entire time-history response is plotted to better show the relative behavior of the structural elements of interest). The wood diaphragm, as expected being more flexible, dominate the response, with peak displacement of 24.5 mm versus 0.953 mm for the wall (response of the walls is magnified by 100 to make it visible on the same figure). The walls, with considerably higher frequency, are thus driven by the floor behavior, clearly vibrating in phase with them, as shown in Fig. 5. The only exception occurs at 2.62 seconds when rocking of the wall occurs. Upon return from its rocking excursion, the wall is seen to vibrate at its high natural frequency, but this motion damps itself rapidly as a consequence of that same high natural frequency (logarithmic decrement of peak amplitudes occurs over a relatively short absolute time for a high frequency structural element).

More evidence of wall rocking is visible when earthquakes having more significant velocity pulses occurs. This is consistent with the empirical evidence and somewhat addressed by the existing methodologies for the seismic evaluation of unreinforced masonry buildings. For example, for the Northridge earthquake Newhall fire station record (a near-fault

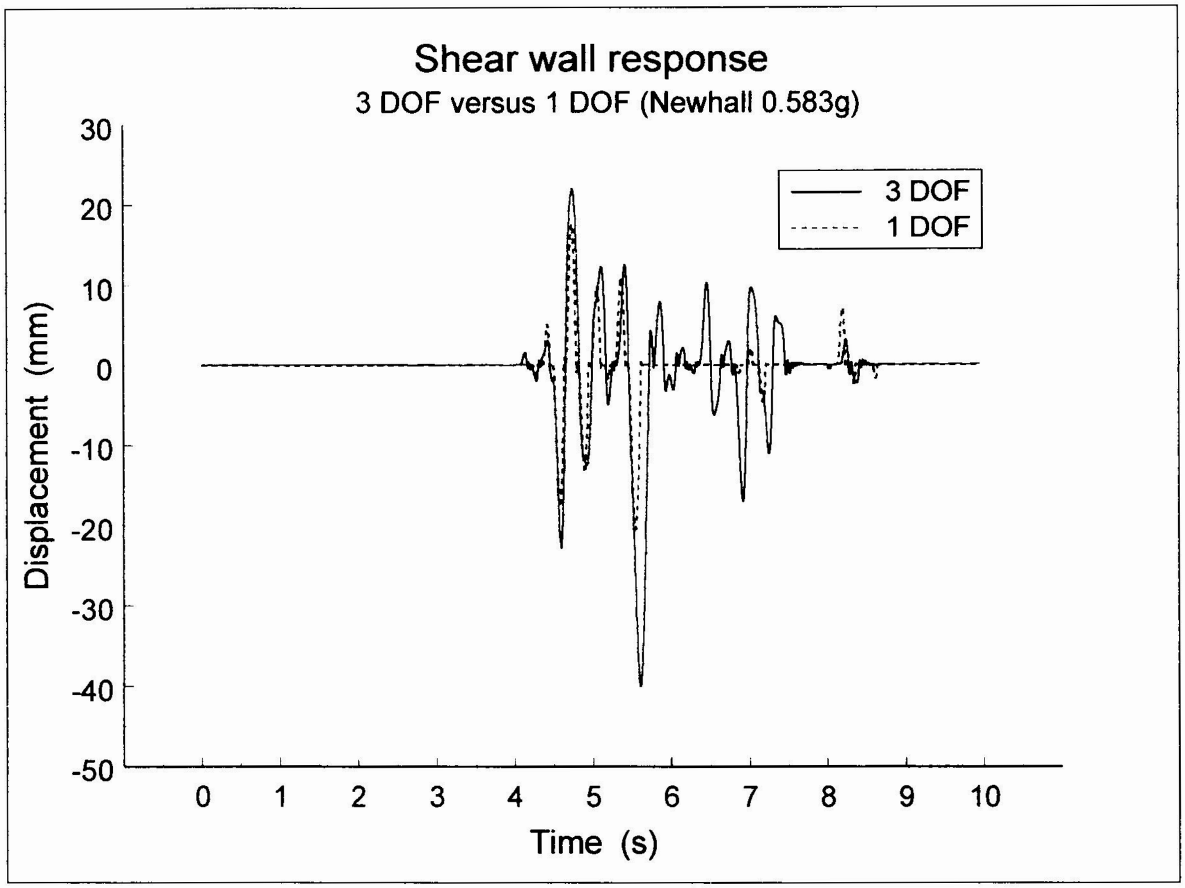


Figure 7: End-wall rocking response obtained considering 3 DOF or 1 DOF models

record with large peak-ground-velocity), at a peak-ground-acceleration of 0.583g, considerably more rocking takes place. Response is shown in Fig. 6, with a peak wall displacement 26 mm (peak diaphragm displacement, not shown here, is 53 mm). Note that the scale of rocking response makes the elastic wall response prior and after rocking barely visible.

Seismic response was then calculated considering inelastic response of the diaphragms. For this purpose, two types of wood floor diaphragms were considered: (a) 1" x 6" diagonal with 1"x6" straight sheething overlay (having a shear strength of 29.8 kN/m); (b) 3/4" plywood with a second 3/4" plywood overlay (with a shear strength of 42.1 kN/m). Results are presented here for the first floor type. As seen in Fig. 6, inelastic response of this rather weak diaphragm limits the force transmitted to the walls, to a magnitude below that needed to initiate significant pier rocking. A maximum wall displacement of 1.73 mm is observed, even though the peak diaphragm displacement (not shown here) remains 53 mm.

The above observations illustrate how in-plane wall response varies from extremely small under elastic response, to relatively large during pier rocking. For certain pseudo-dynamic test set-ups, this could pose some execution difficulties, particularly during the early stages of seismic response when elastic response of a wall is only a fraction of its peak elastic response, and even a magnitude smaller compared to its total inelastic rocking displacement. For example, a first logical actuator configuration for the test of interest here would be to use one actuator to excite the tributary mass at each end-wall location, and another to displace the tributary mass at the diaphragm center-span. This is referred to as the three degree-of-freedom model (3 DOF) here. However, this test set-up could suffer from the aforementioned difficulties. Furthermore, in light of the analytical results that show how wall response is largely driven by the diaphragm response, one could argue that sufficiently accurate seismic response can be captured by using only a single actuator acting at the diaphragm center-span, i.e. using a single-degree-of-freedom model (1 DOF). Further analyses confirm this to be the case. Although all analyses conducted to validate this concept and determine the effective tributary mass to consider are not shown here, results for the Newhall record are presented in Fig. 7. As shown in that figure, most of the instances and magnitudes of pier rocking that were observed using the 3 DOF analytical model are captured using the 1 DOF model. The high frequency wall vibrations that follow rocking, visible when using a 3 DOF model, are obviously missed by the 1 DOF model, but the previous discussion as well as Fig. 7 shows these to be of no significance. Given the supporting evidence from analytical studies, and the fact that using a single actuator results in a simpler test set-up, with considerable savings, the 1 DOF configuration is used in this testing program.

CONCLUSIONS

Non-linear inelastic analyses demonstrate the dominant role of flexible diaphragms on structural response, and support the chosen pseudo-dynamic experimental set-up adopted for full-scale testing of a building for which piers in end-walls behave in rocking during seismic response. In particular, time history analyses results show how the wall response is driven by the diaphragm response. The strength of wood diaphragm is also shown to have a dominant impact on wall response, with yielding of the diaphragms limiting the magnitude of the force that is transmitted to the walls. These analyses set the stage for the full-scale testing of this one-story unreinforced masonry building intended to provide valuable information on both the seismic resistance of existing unreinforced masonry buildings having rocking piers and flexible yielding diaphragms, the adequacy of the existing models of these behavior, and the effectiveness of some seismic retrofitting methods.

REFERENCES

ABK, A Joint Venture, (1984). Methodology for mitigation of seismic hazards in existing unreinforced masonry buildings: The Methodology, ABK-TR-08, Agbabian Associates, El Segundo, Cal., USA

Bruneau, M., (1990). "Preliminary Report of Structural Damage from the Loma Prieta (San Francisco) Earthquake of 1989 and Pertinence to Canadian Structural Engineering Practice", Canadian Journal of Civil Engineering, Vol.17, No.2, pp. 198-208.

Bruneau, M., (1995). "Performance of Masonry Structures during the 1994, Northridge (L.A.) Earthquake", Canadian Journal of Civil Engineering, Vol.22, No.2, pp.378-402, 1995.

Bruneau, M., Lamontagne, M., (1994). "Impact from 20th Century Earthquakes in Eastern Canada and Lessons Learned on the Seismic Vulnerability of Unreinforced Masonry Buildings", Canadian Journal of Civil Engineering, Vol.21, No.4, pp. 643-662.

Bruneau, M., (1994a). "Seismic Evaluation of Unreinforced Masonry Buildings - A State-of-the-Art Report", Canadian J. of Civil Engineering, Vol.21, No.3, pp.512-539.

Bruneau, M., (1994b). "State-of-the-Art Report on the Seismic Performance of Unreinforced Masonry Buildings", ASCE Journal of Structural Engineering, Vol.120, No.1, pp.230-251.

Costley, A.C. and Abrams, D.P., (1995). "Dynamic response of unreinforced masonry buildings with flexible diaphragms", University of Illinois at Urbana-Champaign, Department of Civil Engineering Report UILU-ENG-95-2009, Urbana, Illinois, 281p.

FEMA, (1992). "NEHRP Handbook for the Seismic Evaluation of Existing Buildings", Building Seismic Safety Council, Washington, DC, USA.

ICBO, (1997). Uniform Code for Building Conservation, International Conference of Building Officials, Whittier, California, USA.

NRC (1992). "Guidelines for Seismic Evaluation of Existing Buildings", Institute for Research in Construction, National Research Council, Ottawa, Canada, 151 p.

Rutherford and Chekene, (1991). "Damage to Unreinforced Masonry Buildings in the Loma Prieta Earthquake of October 17, 1989," California Seismic Safety Commission, Sacramento, USA, 38 p.