PSEUDO-DYNAMIC TESTING OF UNREINFORCED MASONRY BUILDING WITH FLEXIBLE DIAPHRAGM

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ABSTRACT

A full-scale one-story unreinforced brick masonry specimen having a wood diaphragm was subjected to earthquake excitations using pseudo-dynamic testing. The specimen was designed to better understand the flexible-floor/rigid-wall interaction, the impact of wall continuity at the building corners and the effect of a relatively weak diaphragm on the expected seismic behavior. After a first series of pseudo-dynamic tests, the unreinforced masonry walls of this building were repaired with fiberglass materials and re-tested. The overall building was found to be relatively resilient to earthquake excitation, even though cracking was extensive. The repair procedure was demonstrated to enhance this behavior. The results were compared with predictions from existing seismic evaluation methodologies. It was found that even though the diaphragm did not experience significant inelastic deformation, some (but not all) of the existing seismic evaluation methodologies accurately capture the rocking/sliding behavior that developed in the shear walls under large displacement.

Introduction

The Uniform Code for Building Conservation (UCBC) (ICBO 1997) *Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings* presents 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 by the ABK joint venture (ABK 1984, FEMA 1992), used extensively in the Los Angeles area, 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 URM 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 URM building having wood diaphragms has not been conducted. Test described here was design and conducted to complement the computer simulations and small-scale shake table tests by other researchers (Costley and Abrams 1995).

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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 had 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 UCBC to be a favorable stable failure mechanism. The specimen had 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 UCBC. Material properties were obtained from simple component tests, such as a three-point flexural bending test 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, it also permits to observe the impact of in-plane rotation of the diaphragm's ends on wall corners.

Experimental Procedure

The unreinforced brick masonry specimen was subjected to a first series of tests under an earthquake of progressively increasing intensity. Non-linear inelastic analyses were conducted to determine an appropriate seismic input motion that would initiate significant pier rocking from the diaphragm response. The selected input motion was a synthetic ground motion for La Malbaie, Canada with a peak ground acceleration of 0.453g.

Figs. 3 to 6 illustrate the behavior observed during the tests. A stable combined rocking and sliding mechanisms formed and large deformations developed without significant strength degradation. The hysteretic response of the west and east shear walls is shown in Figs. 3(a) and 3(b), respectively. Special clip gages were installed at expected crack locations around all the piers to record crack opening and closing during the pier's rocking cycle. This rocking motion is clearly shown in Fig. 4 where the crack opens when the force acts in one direction and remains closed in the reverse direction. Rocking response is shown for the central pier. A different stiffness for the east and west walls was observed at the beginning, during low intensity seismic motion. However, the hysteretic curves during a higher intensity seismic motion, La Malbaie x 2.0, are very similar, as shown in Figs. 3(a) and 3(b). This suggests that the effect of continuous/discontinuous corners becomes somehow negligible during high intensity seismic motion.

Analysis of Results

These results are compared with predictions from existing seismic evaluation methodologies for URM such as the NEHRP Handbook for Seismic Evaluation of Existing Buildings (FEMA 178) (FEMA 1992), Appendix 1 of the Uniform Code for Building Conservation (UCBC) (ICBO 1997) (Appendix 1 of the UCBC is similar to the FEMA 178 document but is based on allowable stress values, i.e. working stress design), Appendix A of the Canadian Guidelines for Seismic Evaluation of Existing Buildings (CGSEEB) (NRC 1992) (Appendix A of the CGSEEB is also similar to FEMA 178 but is adjusted for Canadian codes and practice), the NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273) (FEMA 1997), and FEMA 306 (FEMA 1999a) entitled Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings.

The evaluation of URM walls subjected to lateral forces applied in-plane is performed by calculating the capacities corresponding to each possible individual modes of behavior, the lowest value being the governing failure mode. All behavior modes mentioned below, are summarized in Table 1, showing in which documents they are addressed. The possible modes of behavior include: pier rocking (V_r), sliding shear resistance (V_a) (termed "bed joint sliding with bond plus friction" (V_{bjs1}) in FEMA 273 and FEMA 306), bed joint sliding with friction only (V_{bjs2}) (found only in FEMA 306), diagonal tension (V_{dt}), and toe crushing (V_{tc}). Note that both V_{dt} and V_{tc} are found only in FEMA 273 and FEMA 306. Both rocking and bed joint sliding are considered to be deformation-controlled behaviors able to sustain large lateral deformations while strength remains almost constant, while diagonal tension and toe crushing are considered as force-controlled behaviors.

Following the procedure outlined in FEMA 273, the governing failure mode for each pier is rocking (V_r), as shown in Table 2. Thus, the lateral capacity for each shear wall is the summation of each individual pier rocking capacity, and is equal to 46.7 kN. Likewise, FEMA 306 gives a procedure to evaluate lateral capacity based on observed damage caused by an earthquake. As such, it requires to use the effective height (h_{eff}) of pier reflecting the observed crack pattern. Therefore, the capacities for the individual modes of behavior for each pier shown in Table 2, were re-calculated using the crack pattern observed after pseudo-dynamic tests. The effective height used and resulting capacities are presented in Table 3.

The FEMA 273 nonlinear static procedure was used to establish the idealized nonlinear force-deflection relation for the wall. In this procedure, permissible deformations are established as drift percentages for primary elements (walls considered to be part of the lateral-force system) and secondary elements (walls not considered as part of the lateral-force-resisting system but supporting gravity loads) for the different performance levels of immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The expected capacities for FEMA 273 (46.7 kN), and FEMA 306 (23.0 kN and 22.2 kN for the west and east wall, respectively) were used and the walls are treated as primary elements. The idealized nonlinear force-deflection is plotted against the hysteretic response of the west wall and east wall in Figs. 5(a) and 5(b), respectively. As shown here and noted in FEMA 307 (FEMA 1999b), the experimentally obtained displacements that occurred under a stable rocking mechanism exceed the proposed "*d*" drift value for collapse of $0.4h_{eff}/L$, and equal to 0.52% for primary element, as specified in FEMA 273. Furthermore, as also noted in FEMA 307 and observed here, the rocking capacity does not drop to a "*c*" value of 60% of the initial capacity as proposed by FEMA 273. Finally, FEMA 307 comments that a sequence of different behaviors is common in experiments. The rocking shifting to bed-joint sliding for the central pier, observed when pushing the building in the south direction, is consistent with this expectation.

After this first series of tests, it was observed that the diaphragm remained elastic throughout the tests, as shown in Fig. 6. Therefore, to force the diaphragm into the inelastic range and to investigate the effectiveness of a repair procedure, it was decided to reinforced the shear walls with fiberglass materials.

Repair

The shear walls were repaired using Tyfo fiberglass strips as shown in Fig. 7. Note that these strips are frequently used to enhance the out-of-plane performance of unreinforced masonry walls. They typically enhance the flexural elastic resistance of walls undergoing out-of-plane displacements (Tyfo Systems 1997). The in-plane rocking behavior of unreinforced masonry walls is generally perceived as a stable desirable behavior, but there may be instances where the available rocking strength of such walls may still be inadequate. In that perspective, Tyfo strips were applied to the shear walls to increase their in-plane capacity. They were designed to increase the rocking force capacity of each pier, but to keep that rocking capacity below the pier shear capacity. Hence, the objective of this repair strategy is to use the Tyfo strips to preserve the desirable pier rocking mode, increase capacity and enhance the displacement ductility of the repaired shear walls. The corners of the continuous and discontinuous walls were wrapped with Tyfo WEB to increase their shear resistance. This fabric not only provides additional shear strength, but also maintains the wall’s integrity by preventing spalled portions of the wall from breaking off and becoming safety hazards.

The specimen was re-tested with the same input motion as before. For comparison, the time history of the diaphragm center-span displacement is shown in Fig. 8. This repair solution increased the stiffness of the specimen as shown by the reduced rocking motion (Fig. 4). The repaired unreinforced masonry specimen was able to resist up to large peak ground amplifications (up to nearly 2.0g). At this level of excitation, some strips started to de-bond but still provided enough capacity to allow rocking as shown in Fig. 9 where a crack opening of 22 mm is easily visible. However, for the pier having a bed-joint sliding behavior, the Tyfo strips provided limited resistance, as shown in Fig. 10, and failed in shear. Some tears were observed in the Tyfo Web wrapping the corners due to out-of-plane tensile cracks (Fig. 11). Finally the specimen was subjected to more conventional cyclic-testing, by increasing center-span displacement until a large proportion of the Tyfo material (strips and web) was almost completely de-bonded from the shear wall surface. Evidence suggests that repointing prior to the repair would not have improved the observed behavior. However, a different behavior could have been observed in a retrofit perspective because the original structure would not have been pre-cracked prior to application of the fiberglass material.

Strengthening the shear walls with Tyfo materials did increase the force on the diaphragm, as shown in Fig. 12, comparing diaphragm response with shear walls repaired with Tyfo for La Malbaie x 2.0, and x 4.0. At La Malbaie x 4.0 for the repaired specimen, some nonlinear diaphragm behavior initiated, as seen in Fig. 12. However, because most of the Tyfo material had de-bonded and became ineffective in strengthening the shear walls, the diaphragm did not experience any additional nonlinear inelastic behavior, and thus simply slid like a rigid body on the top of the shear walls. After the test, examination showed that, contrary to pre-test calculations that predicted otherwise, the diaphragm remained relatively intact. Damage was limited to some popped out nails at each ends of the diaphragm, as shown in Fig. 13.

Conclusions

A full-scale one-story unreinforced brick masonry specimen having a flexible wood diaphragm was tested pseudo-dynamically. Tests results have shown that stable combined rocking and sliding mechanisms formed and large deformations developed without significant strength degradation. The diaphragm remained, however, essentially elastic throughout. The difference in wall response due to the presence of continuous or discontinuous corners was somehow negligible during high intensity seismic excitation producing inelastic wall response. The specimen was repaired using Tyfo fiberglass strips, which increased the lateral strength of the shear wall while significantly reducing the displacements. While subjected to higher force, the diaphragm showed some nonlinear inelastic behavior. The theoretical seismic response was calculated using different codified evaluation methodologies. It was found that the FEMA 273 procedure predicted the same behavior for the shear walls as the CGSEEB, i.e. a rocking mode for all piers but strengths in excess of experimentally obtained results. The FEMA 306 procedure, used to evaluate the lateral capacity of concrete and masonry buildings after an earthquake, gave results that closely matched the observed behavior. None of the codified procedure account for the presence of continuous corners, but this continuity was observed to have a negligible impact on the lateral strength of the shear wall during high intensity input motion.

Acknowledgments

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Table 1. Possible lateral behavior modes as per different codes and methodologies

Modes of behavior (1)	FEMA 178 (2)	CGSEEB (3)	UCBC 1997 (4)	FEMA 273 (5)	FEMA 306 (6)
Rocking	X	X	X	X	X
Shear/Bed joint sliding w/bond + friction	X	X	X	X	X
Bed joint sliding w/friction only					X
Diagonal tension				X	X
Toe crushing				X	X

Table 2. Calculation of pier possible behavior mode based on FEMA 273

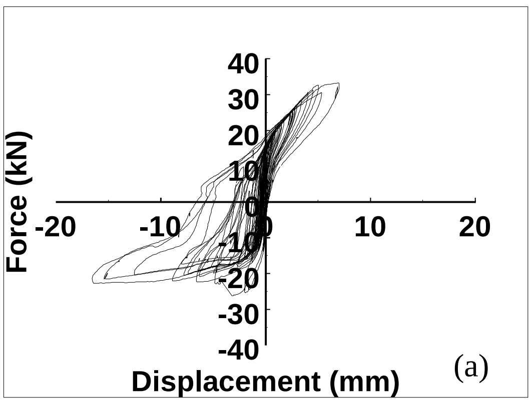
Pier (1)	Pier's Height h (mm) (2)	Rocking V_r (kN) (3)	Bed-joint sliding V_{bjs1} (kN) (4)	Diagonal tension V_{dt} (kN) (5)	Toe crushing V_{tc} (kN) (6)
Door	1842	6.08	39.8	24.5	6.70
Central	953	34.5	65.2	59.8	37.9
Window	953	6.11	27.0	16.6	6.72

Table 3. Calculation of pier possible behavior mode based on FEMA 306

Wall (1)	Pier (2)	h_{eff} (mm) (3)	V_r (kN) (4)	V_{bjs1} (kN) (5)	V_{bjs2} (kN) (6)	V_{dt} (kN) (7)	V_{tc} (kN) (8)
West	Door	1842	6.08	39.8	7.05	24.5	6.73
	Central	1335	24.6	65.2	12.95	59.8	27.3
	Window	1469	3.97	27.0	5.6	16.6	4.34
East	Door	2043	5.48	39.8	7.05	24.5	6.03
	Central	1278	25.7	65.2	12.95	59.8	28.3
	Window	1546	3.77	27.0	5.6	16.6	4.12

Figure 1. URM specimen.

Figure 2. Wood diaphragm.



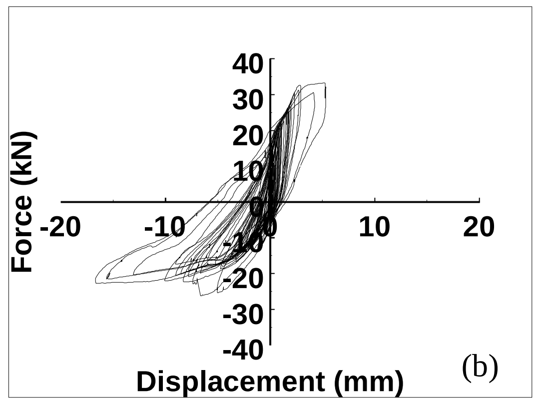


Figure 3. Hysteretic response during La Malbaie x 2.0 of: (a) West wall; (b) East wall.

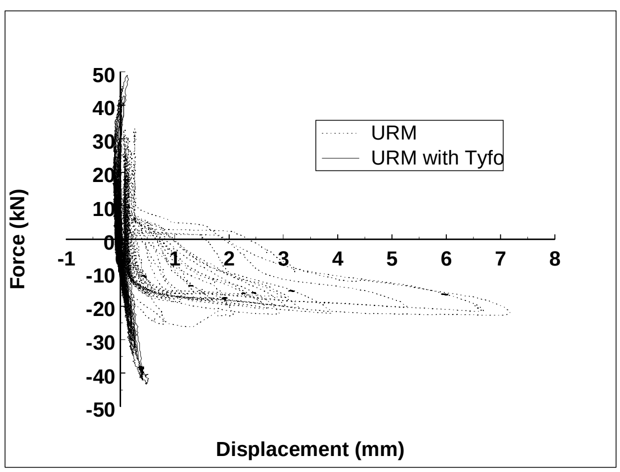
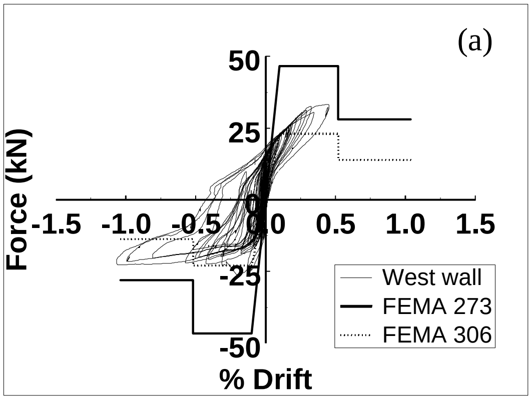


Figure 4. Door pier rocking response at the base before and after Tyfo repair for La Malbaie x 2.0.



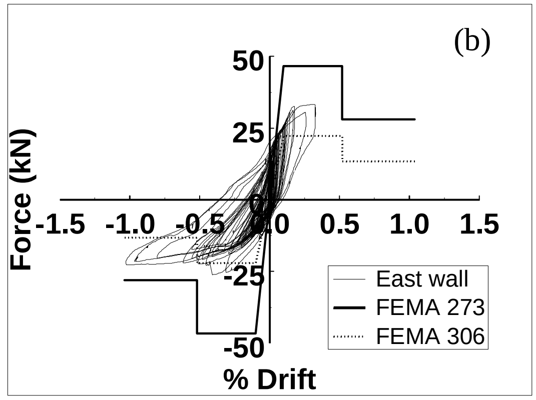


Figure 5. Comparison with idealized force-deflection model using expected capacities from FEMA 273 and FEMA 306 during La Malbaie x 2.0, for: (a) West wall; (b) East wall.

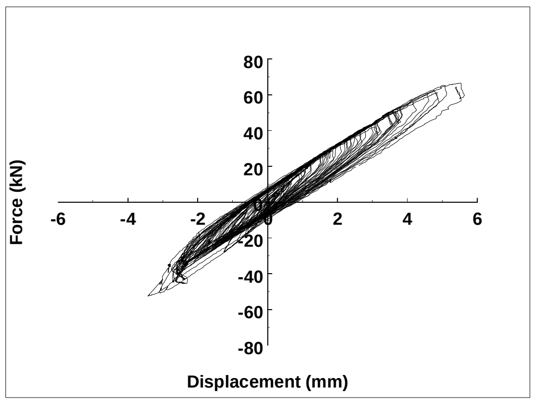


Figure 6. Hysteretic response of wood diaphragm at center-span during La Malbaie x 2.0.

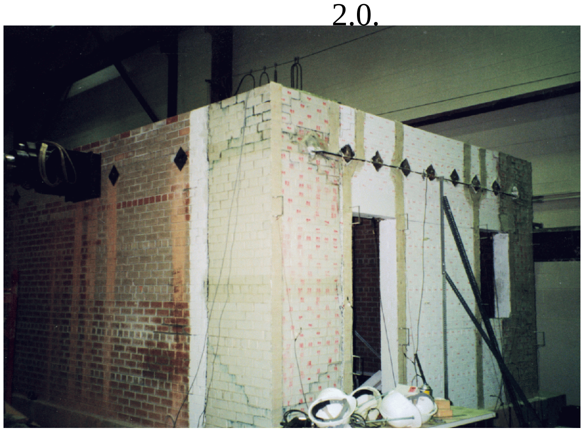


Figure 7. URM repaired with Tyfo material (strips and web).

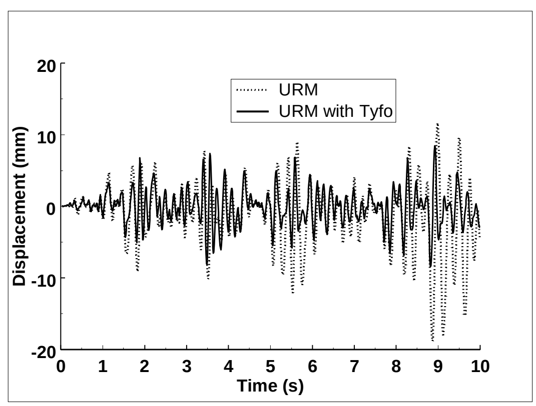


Figure 8. Comparison of diaphragm center-span response before and after Tyfo repair for La Malbaie x 2.0.

Figure 9. Pier rocking at base of central pier with Tyfo repair during La Malbaie x 4.0.

Figure 10. Tyfo strip failed in shear.

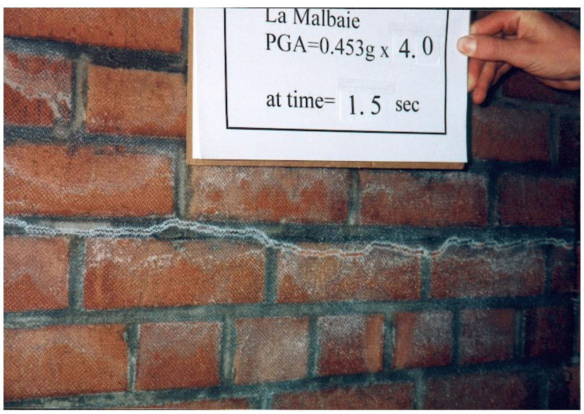


Figure 11. Tears in Tyfo WEB due to out-of-plane tensile cracks.

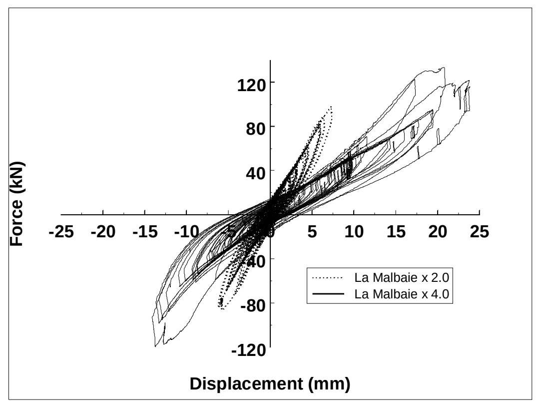


Figure 12. Comparison of diaphragm center-span hysteretic response with shear wall repaired with Tyfo material during La Malbaie x 2.0 and x 4.0.

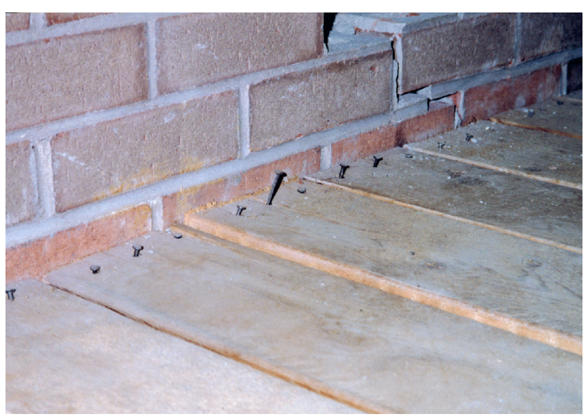


Figure 13. Popped out nails at end of diaphragm.

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