SHAKE-TABLE TESTING OF A 3-STORY, FULL-SCALE, REINFORCED MASONRY WALL SYSTEM

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Between January 12 and February 8, 2011, a 3-story, full-scale, reinforced masonry shear-wall specimen was tested on the large outdoor shake-table at the University of California at San Diego. In this summary report, the characteristics of the specimen are reviewed; its development is described; and its performance in the shake-table testing is summarized. The specimen was very strong and stiff, and suffered little damage when subjected to ground motions with intensities exceeding the maximum considered earthquake (MCE) level. Its performance validates the requirements of the 2008 Masonry Standards Joint Committee (MSJC) Code (used throughout the USA) for design and detailing of special reinforced masonry shear walls.

**Keywords:** codes and standards, design, earthquakes

DESCRIPTION OF 3-STORY, FULL-SCALE MASONRY SPECIMEN

The 3-story, full-scale masonry specimen is shown schematically in Figure 1, and as constructed on the shake-table, in Figure 2.

The specimen was designed according to requirements of ASCE7-05 and the 2008 MSJC Code for Seismic Design Category D, and was thus considered a special load-bearing wall system. It was detailed in accordance with 2008 MSJC Code requirements.
Figure 1 Schematic views of 3-story, full-scale reinforced masonry specimen

Figure 2 Three-story, full-scale masonry specimen as constructed on UCSD shake-table

DESIGN OF SPECIMEN

Plan and Elevation of Prototype Building

The plan and elevation of the prototype building are shown in Figure 3 and Figure 4. The building is intended to be a typical apartment or office. Primary dimensions are given in inches because those units were used for bidding and construction. SI units are also given for
completeness. For simplicity and symmetry, the vertical shafts that would have been required for stairs and mechanical ducts were not provided in this specimen.

**Figure 3** Plan view of typical floor of three-story prototype building

**Figure 4** Elevation of three-story prototype building

**Plan and Elevation of Specimen**

The dashed rectangle in the plan view on the left-hand side of Figure 3 shows the plan area occupied by the specimen. In the specimen, the walls parallel to the direction of shaking are two symmetrical T-walls and one lineal wall (a wall without flanges). The walls perpendicular to the direction of shaking are two lineal half-walls. For consistency in describing the specimen, the walls parallel to the direction of shaking are described as “longitudinal” walls, and the walls perpendicular to the direction of shaking are described as “transverse” walls.
Force-based Seismic Design of Specimen

Design earthquake loads are calculated according to Section 1613 of the 2009 International Building Code. That section essentially references ASCE 7-05 (Supplement). Seismic design criteria are given in Chapter 11 of ASCE 7-05. The seismic design provisions of ASCE 7-05 (Supplement) begin in Chapter 12, which prescribes basic requirements (including the requirement for continuous load paths) (Section 12.1); selection of structural systems (Section 12.2); diaphragm characteristics and other possible irregularities (Section 12.3); seismic load effects and combinations (Section 12.4); direction of loading (Section 12.5); analysis procedures (Section 12.6); modeling procedures (Section 12.7); and specific design approaches. Four procedures are prescribed: an equivalent lateral force procedure (Section 12.8); a modal response-spectrum analysis procedure (Section 12.9); a simplified alternative procedure (Section 12.14); and a seismic response history procedure (Chapter 16 of ASCE7-05). The equivalent lateral-force procedure was used here, because it is relatively simple, and is permitted in most situations. The simplified alternative procedure is permitted in only a few situations. The other procedures are permitted in all situations, and are required in only a few situations.

Because the equivalent lateral force procedure is being considered, the response spectrum curve is not required. Nevertheless, for completeness, it is shown in Figure 5 for a typical site in the San Diego area, which is considered for the design of the test specimen.

![Figure 5 Design response spectrum for San Diego, CA](image)

The specimen was designed neglecting the effects of coupling effect of the floor and roof slabs. This greatly simplifies the design, and is generally conservative. Wall segments were designed and detailed to meet the requirements of the 2008 MSJC Code for special wall systems, including capacity design for shear. For T-walls W-1 and W-3, the web required flexural reinforcement consisting of No. 4 (12 mm.) bars at 8 in. (203 mm.), plus one additional No. 4 (12 mm.) bar in the end cell of the web. In addition, the flanges had 5 additional No. 4 longitudinal bars. Transverse reinforcement consisting of No. 4 (12-mm) horizontal bars at 16 in. (406 mm) was used in the webs and flanges. Lineal wall W-2 had longitudinal reinforcement consisting of No. 4 (12-mm) bars at 8 in. (203 mm), and transverse reinforcement consisting of No. 4 (12-mm) horizontal bars at 16 in. (406 mm).
Wall 1 (Figure 2) had vertical reinforcement spliced at the mid-height of the ground story, while Wall 3 had vertical reinforcement spliced at the base, in a potential plastic hinge zone. The former complies with the requirement of ASCE 7-05, while the latter is permitted by the MSJC Code. Walls were not provided with shear keys. Horizontal reinforcement in walls was placed starting in the lowest course. Control joints were introduced on each side of the lintel beams above door openings, and the flexural reinforcing bars in the lintels were de-bonded in regions beyond the control joints to reduce the coupling moments transmitted to the wall elements.

TESTING OF SPECIMEN

Between January 12 and February 8, 2011, the specimen was subjected to an extended series of different ground motions. In this section, the test history and specimen response are first summarized, and the significance of that response is then discussed.

These test series used ground-motion records consistent with the seismic intensity of the assumed site. In Figure 6, the response spectra for those ground motions are compared with design response spectra at the DBE and MCE levels. In ASCE7-05, “DBE” (Design Basis Earthquake) corresponds to an event with a return period of 476 years, and “MCE” (Maximum Considered Earthquake) corresponds to an event with a return period of about 2500 years. The initial fundamental period of the structure was 0.09 sec.

![Figure 6 Ground motion spectra defined for test series before scaling](image)

**Significance of Specimen from a Code Viewpoint**

The test history and specimen response are summarized in Table 1. The specimen was stiffer and stronger than anticipated. It successfully resisted repeated ground motions well in excess of MCE (maximum considered earthquake). Its response was a validation of 2008 MSJC Code requirements for the design and detailing of special reinforced masonry shear walls. The 2008 MSJC permits splices of vertical reinforcement in potential plastic hinge zones, and does not require shear keys at wall bases. ASCE7-05, in contrast, prohibits splices in plastic hinge zones, and requires shear keys. In this specimen, Wall 1 (the west wall) had its vertical reinforcement spliced at mid-height, while Wall 3 (the east wall) had its vertical reinforcement spliced at the base. No difference in performance was observed, even though both splices were subjected to severe histories of reversed cyclic shear. The response of this
specimen may argue for the validity of the MSJC requirements over those of ASCE7-05. However, the coupling shear of the roof and floor slabs reduced the axial compressive force in a T-wall when its web was subjected to flexural compression, and thereby alleviated toe crushing. The performance of lap splices in plastic hinge zones with severe flexural compression requires further investigation. This is being studied in quasi-static tests of wall segments at Washington State University.

Table 1 Summary of test history and specimen response

<table>
<thead>
<tr>
<th>Date</th>
<th>Ground Motion</th>
<th>Level of Excitation</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/12/2011</td>
<td>20% El Centro</td>
<td>Expected Design Level Earthquake based on</td>
<td>Structural period $T$ before and after test = 0.09 sec</td>
</tr>
<tr>
<td></td>
<td>979</td>
<td>original ground motion record</td>
<td></td>
</tr>
<tr>
<td>1/13/2011</td>
<td>45% El Centro</td>
<td>Expected Design Level Earthquake based on</td>
<td></td>
</tr>
<tr>
<td></td>
<td>979</td>
<td>original ground motion record</td>
<td></td>
</tr>
<tr>
<td>1/18/2011</td>
<td>120% El Centro</td>
<td>Realized Design Level Earthquake based on</td>
<td>Flexural cracks developed at ends of lintel beams near control joints</td>
</tr>
<tr>
<td></td>
<td>979</td>
<td>table motion</td>
<td></td>
</tr>
<tr>
<td>1/15/2011</td>
<td>150% El Centro</td>
<td>Slightly below MCE (Maximum Considered</td>
<td>Flexural cracks developed at wall base; vertical reinforcement in flange of</td>
</tr>
<tr>
<td></td>
<td>979</td>
<td>Earthquake)</td>
<td>Wall 1 (west T-wall) reached tensile strain of 0.01; similar strain level for</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>bars in Wall 2 (middle wall)</td>
</tr>
<tr>
<td>1/19/2011</td>
<td>180% El Centro</td>
<td>Slightly above MCE</td>
<td></td>
</tr>
<tr>
<td>2/20/2011</td>
<td>250% El Centro</td>
<td>Slightly above MCE</td>
<td>Wall 2 (middle wall) had very minor base sliding; fine diagonal shear cracks</td>
</tr>
<tr>
<td></td>
<td>1979</td>
<td></td>
<td>developed on Wall 1 (west T-wall); max. 1st story drift = 0.25%; after test</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$T = 0.2$ sec</td>
</tr>
<tr>
<td>1/18/2011</td>
<td>300% El Centro</td>
<td>Slightly above MCE</td>
<td>Max. 1st story drift = 0.14%; after test $T = 0.2$ sec</td>
</tr>
<tr>
<td>1/19/2011</td>
<td>125% Sylmar</td>
<td>Slightly above MCE</td>
<td>Diagonal shear cracks extended in Wall 1 (west T-wall); horizontal flexural</td>
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<tr>
<td></td>
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<td></td>
<td>cracks develop near top of 1st and 2nd story walls; max. 1st story drift =</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.23%</td>
</tr>
<tr>
<td>1/26/2011</td>
<td>160% Sylmar</td>
<td>Above MCE</td>
<td>Diagonal shear cracks extended in Wall 1 (west T-wall); flexural cracks observed on top of 2nd story slab close to edges of door openings; max. 1st story drift = 0.38%</td>
</tr>
<tr>
<td>1/26/2011</td>
<td>140% Rinaldi</td>
<td>Slightly above MCE</td>
<td>After test $T = 0.22$ sec</td>
</tr>
<tr>
<td>2/2/2011</td>
<td>100% Chi Chi</td>
<td>Above MCE</td>
<td>Flexural–shear cracks developed near bottom of T-walls; cracks on top of 2nd story slab extended throughout the entire width (10 ft); max. 1st story drift = 0.35%</td>
</tr>
<tr>
<td>1/26/2011</td>
<td>1st 150% Chi Chi</td>
<td>About 2 X MCE</td>
<td>Severe diagonal shear cracks developed in both 1st story T-walls; residual shear crack width of 0.06 in. (1.54 mm.); Wall 2 (middle wall) slid on the base; max. 1st story drift = 0.73%; after test $T = 0.25$ sec</td>
</tr>
<tr>
<td>2/2/2011</td>
<td>2nd 150% Chi Chi</td>
<td></td>
<td>Structure was severely damaged; residual shear crack width of 0.38 in. (9.6 mm.); signs of toe crushing in webs of T walls; max. 1st story drift = 1.52 %</td>
</tr>
</tbody>
</table>

Response of the specimen validates 2008 MSJC Code requirements that transverse (horizontal) reinforcement be hooked around extreme-fiber vertical reinforcement. It also
suggests that horizontal reinforcement should be placed in the lowest course, to protect lap splices there from degradation. Response of the specimen validates the 2008 MSJC Code requirements for capacity design for shear of special reinforced masonry shear walls. Because of the unexpected strength of the coupling slabs, the walls in the direction of shaking were subjected to greater shears than anticipated in design. Nevertheless, they continued to be effective in resisting shear, with no visible signs of fracture of transverse reinforcement.

**Significance of Specimen from a Design Viewpoint**

From a design viewpoint, the specimen’s response was quite interesting. In contrast to the design assumption of zero coupling, the floor planks actually were very stiff, strong coupling elements. The coupling moments developed by the slabs contributed to the high lateral stiffness and strength of the structure. The coupling helped to prevent toe crushing in the web of the T-walls by alleviating the compressive stress when the web was subjected to flexural compression. In the later stage of the test series, the middle wall began to slide, and did not contribute much to the lateral load resistance. Most of the lateral resistance came from the T-wall on the leeward side (the T-wall acting in compression). The rotational restraints introduced at the top of the bottom-story walls by the floor slab reduced the effective shear-span ratio of the walls and eventually led to the shear failure of the T-walls.

**Details of Specimen Response**

Initial response of the specimen was marked by the appearance of flexural cracks at the bases of Wall 1, Wall 2, and Wall 3, and by the appearance of lintel cracks at the control joints on both ends of the lintel connecting Wall 1 and Wall 2. Although the longitudinal reinforcement passing through the lintels had been debonded on one side of each control joint, the strong connection between the precast planks and Walls 1, 2, and 3 caused the lintels to move with the planks rather than the walls, and caused some lintel cracking. The strut action of the lintels also tended to reduce the clear height of the walls.

As shown in Figure 7 and Figure 8, more severe shaking caused the start of shear cracks at the bases of Wall 1 and Wall 3, and produced some minor base sliding of Wall 2.

![Figure 7](Figure 7 Shear cracking of Wall 1 after 100% Chi Chi)

![Figure 8](Figure 8 Shear cracking of Wall 3 after 100% Chi Chi)

As shown in Figure 9 and Figure 10, still stronger shaking increased the observed flexural and shear cracking.
As shown in Figure 11 through Figure 16, still stronger shaking produced additional shear cracking at the base of Walls 1 and 3, and out-of-plane flexural cracking at the bases of the out-of-plane walls.
As shown in Figure 17 through Figure 20, continued shaking caused crushing at the corner of a door opening due to the rocking of the lintel beam, the widening of shear cracks at the bases of Wall 1 and Wall 3, and the crushing of the toe of Wall 3. Of particular significance is the splice region at the compression toe of base of Wall 3, shown in Figure 20. Note how the lowest transverse reinforcing bar, hooked around the splice, keeps the spliced bars from coming apart.
Finally, at the end of the test, Figure 21 shows how wide shear cracks had opened in Wall 1 and Wall 3 at the ground level. As shown in Figure 22, flexural cracking near the top of Wall 2 indicated that the floor planks were still acting as stiff, strong coupling elements.

![Figure 21 Shear crack on ground story of Wall 1 after second run of 150% Chi Chi](image)

![Figure 22 Wall 2 at second story after second run of 150% Chi Chi](image)

**SUMMARY AND CONCLUSIONS**

In early 2011, a 3-story, full-scale, reinforced masonry shear-wall specimen was tested on the large outdoor shake-table at the University of California at San Diego. The specimen was designed and constructed using then-current United States requirements for the seismic design and construction of reinforced masonry. The specimen was very strong and stiff, and suffered little damage when subjected to ground motions with intensities exceeding the maximum considered earthquake (MCE) level. Its performance validates the United States requirements for the seismic design and construction of reinforced masonry.

**ACKNOWLEDGEMENTS**

The work described in this paper was supported by the United States National Institute for Standards and Technology (NIST). The authors gratefully acknowledge the use of facilities provided by the United States National Science Foundation Network for Earthquake Engineering Simulation (NSF NEES).

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