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SHAKING-TABLE TESTING OF SINGLE-STORY CLAY MASONRY VENEER WOOD-FRAME BUILDING

H. O. Okail[1], S. Jo[2], P. B. Shing[3], R. E. Klingner[4], W. M. McGinley[5], and D.I. McLean[6]

ABSTRACT

Many residential and commercial buildings throughout the US and Canada have an exterior façade consisting of a wythe of clay masonry veneer backed by a wood-stud wall. The brick veneer is connected to the backing by metal anchors. This paper presents the results of shaking-table tests conducted on a full-scale, one-story building with brick veneer on a wood-stud backup. The experiments were part of a coordinated research project funded by the US National Science Foundation under the Network for Earthquake Engineering Simulation Program. The structure had different types of veneer anchors with different anchor spacings, and wall segments of different aspect ratios, all conforming to the requirements of the 2008 Masonry Standards Joint Committee Specification for Seismic Design Category D or above. This paper presents a summary of the experimental program, the shaking-table test results, and the implications for current design provisions.

Introduction

Clay masonry veneer is commonly used in low-rise residential and commercial buildings in many parts of the United States and Canada. Many of these systems consist of an exterior clay brick wythe connected to a structural wood-stud wall by metal anchors, also referred to as veneer ties (Drysdale 1999). The anchors span across an air space (typically at least 25 mm) that acts as a drainage cavity. Moisture exits the cavity through weep holes located at the bottom course of the veneer wythe (BIA 2002). Waterproof flashing at the base of the veneer and at openings collects moisture and directs it to the exterior of the wall system. Advantages of masonry veneer wall systems include excellent thermal insulation, fire resistance, durability, minimal long-term maintenance cost along with an attractive architectural appearance. Traditional design for brick

[1] Graduate Research Assistant, Dept. of Structural Engineering, University of California – San Diego, La Jolla, CA 92093, USA
[2] Graduate Research Assistant, Dept. of Civil and Environmental Engineering, University of Texas – Austin, Austin, TX 78712, USA
[3] Professor, Dept. of Structural Engineering, University of California – San Diego, La Jolla, CA 92093, USA
[4] Professor, Dept. of Civil and Environmental Engineering, University of Texas – Austin, Austin, TX 78712, USA
[5] Professor, Dept. of Civil and Environmental Engineering, University of Louisville, KY 40292, USA
[6] Professor, Dept. of Civil and Environmental Engineering, Washington State University, Pullman, WA 99164, USA

veneer treats the masonry as a non-structural component, which is designed to support only its own weight and to transfer out-of-plane loads such as wind or seismic forces to the structural backing system. Under earthquake excitation, a veneer wall system can be subjected to both in-plane and out-of-plane loading. Veneer ties can be subjected to high axial and shear forces depending on the direction of the earthquake excitation. The design of a veneer system is based on prescriptive code provisions that limit the tributary area and spacing of veneer anchors. However, little information is available on the dynamic performance of such a system, including the capacities of various anchor types and the force demand on the anchors (Reneckis et al. 2004).

Under the auspices of the George E. Brown, Jr. Network for Earthquake Engineering Simulation Program (NEES) of the US National Science Foundation (NSF), a collaborative research project has been carried out to study the seismic performance of masonry veneer walls and veneer anchors in wood-stud and reinforced masonry buildings. This paper presents the findings of the shaking table experiments that were conducted as part of the aforementioned project to study the seismic performance of a full-scale single-story building with masonry veneer on a wood-stud backing frame. The building was designed and constructed in accordance with current US code provisions (MSJC 2008, IRC 2006) and industry recommendations (BIA 2002).

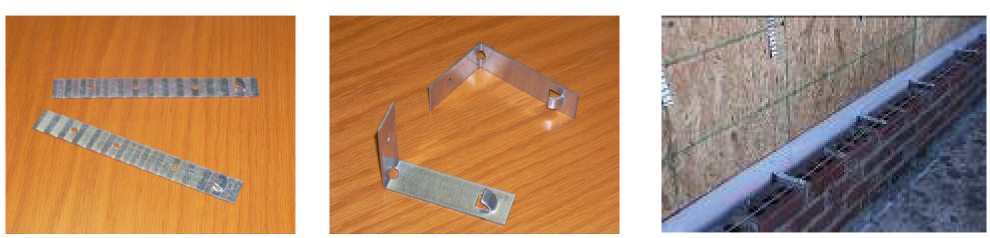
Experimental Program

Design of Test Structure

The wood-stud test structure is shown in Figure 1. It had a square footprint, with a veneer wall measuring 6.30 m end-to-end on each side. In the shaking-table tests, it was subjected to table motions along the east-west direction. The structure was designed and constructed in accordance with current prescriptive requirements for SDC D2 (IRC 2006). The top and bottom plates and studs in the walls were No. 2 Douglas fir with nominal dimensions of 38 mm x 89 mm. Single bottom plates and double top plates were used, and studs were spaced at 406 mm on center. The studs were covered on the outside by 11-mm, exterior-grade oriented strand board (OSB) fastened by 8d nails 152 mm on center on the edge studs, and 304 mm on center on the intermediate studs. The exterior and intermediate studs were covered on the inside by 12-mm gypsum wallboard fastened by drywall screws at 101 mm and 203 mm on centers, respectively. Seismic hold-downs were used at the ends of each panel. The clay masonry veneer consisted of standard modular clay units, conforming to ASTM C216 with greater than 75% solid (ASTM 2006 and a nominal width of 101 mm, laid with Type N masonry cement mortar. The base of the veneer had 30-mil EPDM flashing. The structure had a 25-mm specified air space between the OSB and the veneer, because that best represents current construction. On the west side, the veneer was attached to the wood-stud frame using 22-gage corrugated ties (Figure 2a) spaced at 406 mm horizontally and vertically, and fixed to the wood with 8d electro-galvanized box nails (10-1/4 gauge, 2.87-mm diameter, 64-mm long). The veneer on the west side was separated from the veneer on the north and south sides by open vertical movement joints. On the east side, the veneer was attached using rigid ties (Figure 2b) spaced at 406 mm horizontally and 609 mm vertically, and fixed to the wood with #10 screws (nominal shank diameter of 4.76 mm, 64 mm long). The ties were provided with seismic clips and joint reinforcement (Figure 2c). On the north and south sides, to achieve symmetry with respect to the plane of shaking, the veneer was attached with the same corrugated ties as on the west side. While the veneer on the north side had ties anchored with seismic clips and joint reinforcement, the veneer on the south side did not have joint reinforcement.

The roof system was composed of nominal 38-mm x 292-mm, No. 2 Douglas fir, joists spaced at 406 mm on center, sheathed by 11-mm OSB structural sheathing and fastened as required by the 2006 IRC with 6d nails spaced at 152 mm on center. The interior joists were oriented perpendicular to the direction of shaking. The rim joists were connected to the top stud wall plate with metal shear plates at 609 mm on center. Each joist was fastened to the top plate with metal clip angles. These clip angles and plates were designed to transfer the diaphragm shear to the top of the shear walls. The bottoms of the roof joists were sheathed by 12-mm gypsum wallboard fastened with screws spaced at 152 mm on center. Because the horizontal seismic reaction from the out-of-plane walls was vertically eccentric to the roof sheathing, torsion would be imposed on the rim joists oriented perpendicular to the direction of shaking. To guard against the rotation of the rim joists due to such torsion ("joist rolling"), the end joist space was blocked, using nominal 38 mm x 89 mm members spaced at 406 mm. In addition, the bottoms of the ring joists were attached to the tops of the first interior joists by metal straps. These straps are recommended for use as bridging for joists, and were incorporated as tension braces for the end joist to provide additional torsional resistance.

Figure 1. Prototypical wood-stud frame with wood sheathing, clay masonry veneer and connectors, and interior gypsum wallboard. (1 inch = 25.4 mm)



(a) (b) (c)

Figure 2. (a) Corrugated Ties, (b) Rigid Ties, (c) Corrugated Ties with Joint Reinforcement

Instrumentation and Testing Protocol

The building was tested on the NEES Large High Performance Outdoor Shaking Table (LHPOST) at the University of California at San Diego. Displacement transducers and accelerometers were used to monitor the relative displacements and total accelerations of the veneer and the wood-frame at the positions of the veneer ties. The accelerometers were mounted on the specimen while the displacement transducers were mounted on reference frames located close to the tested walls (Figure 3). Most of the instruments on the veneer side of the specimen were removed prior to the anticipated failure run of the shaking-table to avoid their damage during collapse. The roof of the wood structure was instrumented with accelerometers along the entire perimeter and the diaphragm center. All the sensors were oriented to record displacements and accelerations in the same direction as the table motion. Shaking-table tests were conducted using the Sylmar – 6-story County Hospital Parking Lot record (360-degree component) from the 1994 Northridge (California) Earthquake. The record was obtained from the Center for Engineering Strong Motion Data (*www.strongmotioncenter.org*). The original record is shown in Figure 4. The house was subjected to a sequence of Sylmar ground motion histories with the acceleration scaled to different levels, namely 25%, 50%, 80%, 120%, 150% and 200% (repeated two times) of the original record. The scaling used design basis response spectra that are representative of SDC D and E as a reference. As shown in Figure 5, the original Sylmar with an 80% acceleration scale factor matches the spectral coordinate of the design spectrum for SDC D and is slightly more severe than that for SDC E at the initial fundamental period of the structure of 0.10 second. White-noise excitation was used in between the earthquake runs to assess the dynamic properties of the wall system and to track the progression of damage. The white noise had a root-mean-squared acceleration of 0.03g and swept a frequency range of 1 – 33 Hz.

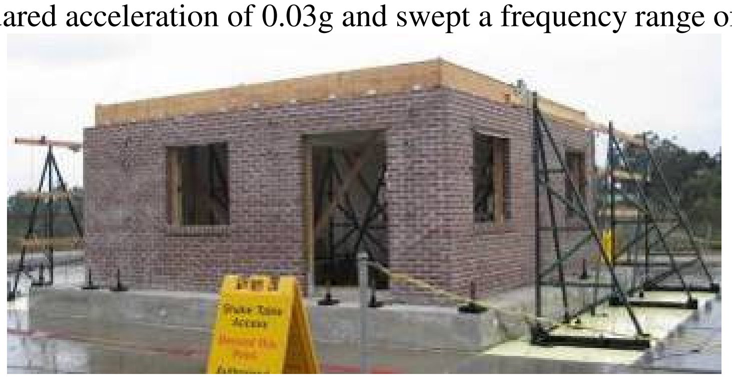


Figure 3. Test Setup and Reference Frames for Displacement Transducers

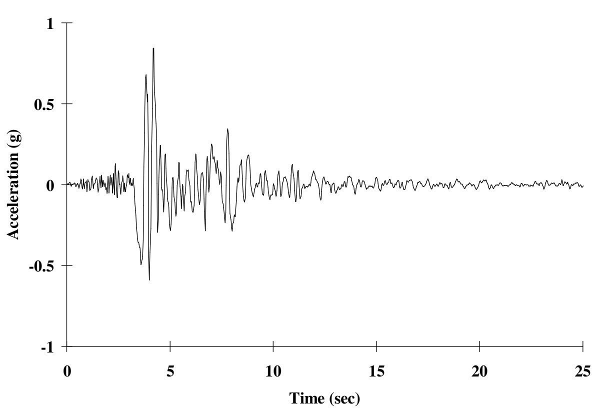


Figure 4. Sylmar Record

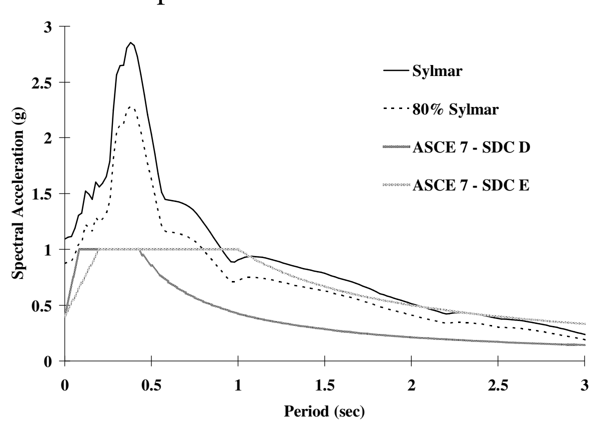


Figure 5. Response Spectra (5% Damping)

Experimental Observations

Behavior under Low Level Ground Motions

The building sustained ground shaking levels of Sylmar 25% and Sylmar 50% with no visible signs of damage. There was no evidence of anchor distress on the east wall or of veneer sliding in the in-plane walls (north and south walls). This behavior is consistent with the performance objectives at such levels of ground shaking. Very small amounts of nail extraction were observed at the top row of ties on the west wall.

Behavior under Design Basis and Maximum Considered Earthquakes

During ground shaking of Sylmar 80%, the veneer on the west wall peeled off the structure by the formation of a horizontal bed-joint crack above the 18^{th} brick course (Figure 6). The collapse was characterized by complete nail extraction from the wood studs (Figure 7). Based on the observations of the tests conducted earlier on individual wall segments and the analytical studies conducted prior to the building tests, this mode of failure was not expected until higher levels of ground shaking. The rest of the structure, in contrast, showed no signs of distress under this level of shaking. During shaking at Sylmar 120 %, the rest of the veneer on the west wall detached from the backing wall by nail extraction from the wooden studs. The rest of the structure showed no signs of distress under this level of shaking.

Figure 6. Collapse of Veneer on West Wall

Figure 7. Typical Nail Extraction

Behavior under Severe Ground Shaking

No major additional damage was observed during the ground motion level of Sylmar 150%. The east wall responded in the out-of-plane direction with noticeable two-way bending, and no visible cracking. Relative motion between the rim joist and the double top plate was observed on the east wall, coupled with minor deformation of the metal plates connecting the outside of the rim joist and the top plate. During the first shaking at Sylmar 200%, the top portion of the veneer on the east wall collapsed (Figure 8). Failure was characterized by bed-joint cracking and pullout of most of the anchors from the mortar joints (Figure 9). A few anchors detached when screw heads pulled through deformed holes in the rigid ties. Diagonal cracking was also observed at the top and the bottom of the smaller veneer segments in the north and south walls, due to rocking of those segments. During the second application of Sylmar 200% shaking, more of the veneer on the east wall peeled off. Joint reinforcement in the top row of ties pulled out of the mortar joints and was not able to hold the dislodged pieces of veneer together.

The entire south wall pulled away from the backing frame and collapsed as shown in Figure 10. The collapse was initiated by the falling of the door lintel which was already diagonally cracked on both sides of the door during the previous test. After that, all the veneer anchors pulled out of the backing after experiencing severe cyclic shearing deformations (Figure 11). The north wall veneer was on the verge of collapse. It suffered excessive sliding and most of the anchors were pulled out of the backing wood-stud frame as shown in Figure 12. This pullout of veneer anchors was due to in-plane shearing deformations leading to the loss of any out-of-plane restraint as shown in Figure 13. The lintel beam over the door opening also suffered excessive diagonal cracking similar to that observed on the south wall. The joint reinforcement did not seem to help in arresting the propagated cracks (Figure 14). The roof of the structure, which had already suffered some minor differential movements between the rim joist and the double top plate, generally behaved well. During this level of excitation, the end nails at the blockings next to the east wall pulled out as shown in Figure 15. The magnitude of the pullout along the length of the east side suggests the bowing of the rim joist due to the two-way bending action of the entire east wall (veneer and backing). After the test, the gypsum board of the ceiling in this region was removed, and visual inspection showed a significant separation between the blockings and the rim joist as well as pullout of the braces connecting the rim joist to the first interior joist (Figure 16).

Figure 8. East Wall


Figure 9. Anchor Pullout


Figure 10. South Wall

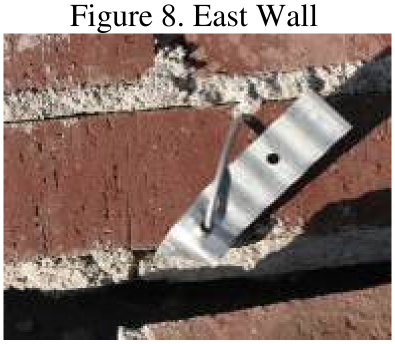


Figure 11. Deformed Anchor


Figure 12. Anchor Extraction

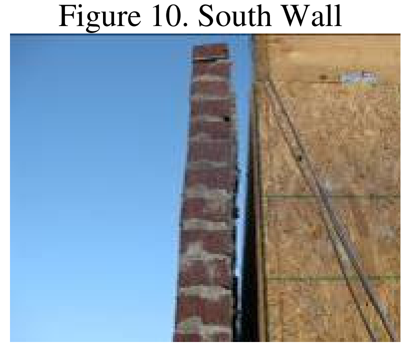


Figure 13. North Wall

Figure 14. Cracked Lintel


Figure 15. Rim Joist

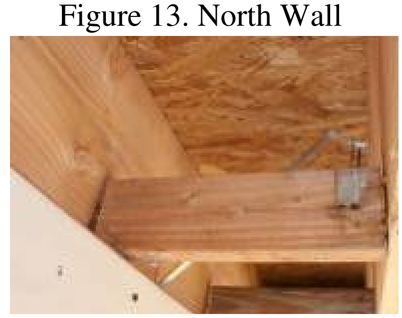


Figure 16. Brace Failure

Evaluation and Discussion of the Seismic Performance

Acceleration time histories from the veneer and the backing subjected to the white noise excitation applied at the beginning of the tests and after each earthquake ground motion test were analyzed. The results show that the initial fundamental frequency of the west wall (7.85 Hz) was significantly different from that of the east wall (9.90 Hz). This striking difference may be attributed to the discontinuity in the veneer around the northwest and southwest corners of the building. Another reason is the initial slack in the corrugated ties which straightened under low levels of shaking. This straightening was possible because of the free edges of the veneer. The results also show that the west wall experienced a decrease in fundamental frequency to below 6.00 Hz right before failure. The east wall of the structure showed no signs of distress until after MCE (Sylmar 120%), with failure again occurring after a decrease in fundamental frequency to about 6.00 Hz. The north and south walls and the roof showed no significant changes in fundamental frequency during the entire test sequence.

Figure 17 presents a plot of the sliding displacement at the base of the veneer in the north and south walls versus peak ground acceleration (PGA). The figure shows no sliding until a PGA corresponding to the ground motion level of Sylmar 120%. The figure also shows that the north and south veneer walls behaved similarly under in-plane shaking. Both walls showed less sliding at the smaller segments due to the flange effect provided by their connection to the east wall, and also due to their tendency to rock.

Figure 18 presents the plots of the average peak roof acceleration and the average dynamic amplification (ratio of the peak average roof acceleration to the peak base acceleration) versus the PGA. The latter plot shows that the structure experienced an overall average dynamic amplification of 1.75 to 2.00 with the exception of the ground motion level that corresponds to Sylmar 80%, during which a dynamic amplification of 2.19 was recorded. This was the level at which the west wall veneer collapsed out-of-plane. Figures 19 and 20 present the peak out-of-plane accelerations at the top of the veneer and the backing versus the PGA for the west and east walls, respectively. The figures show that the west wall veneer experienced a peak acceleration of 2.07g, about twice that experienced by the east wall having the rigid ties under Sylmar 50%. This high acceleration corresponds to a local dynamic amplification of about 4.22 on the west wall veneer. This can be attributed to the straightening of the corrugated ties, eliminating the initial slack, on the west wall, which induced a shock to the veneer. The discontinuous corners at the edges of the west wall provided no restraint for the wall motion. Prior to the next shaking (Sylmar 80%), visual inspection of the top row of ties in the west wall revealed partial extraction of the nails from the wood-stud backing. The figures show that the east wall experienced more than twice the peak acceleration recorded on the west wall before failure. This can be attributed to 1) the rigid ties used on this wall, which had no initial slack; 2) the use of screws, which had significantly higher extraction capacities than nails; and 3) the bonded corners, which restrained the wall segments adjacent to the window from out-of-plane motion.

Figure 21 shows the net in-plane veneer displacement, defined as the difference between the top displacement and the base sliding, plotted against the PGA. This net displacement reflects the rocking of the veneer. The plots show no major motion till the MCE level The small wall segment on the south side exhibited much more significant rocking than the other segments.

Figure 22 shows the relative top displacement between the veneer and the backing. Once again, the plots show no major relative displacement till the MCE level with the small wall segment on the south side exhibited the most severe relative displacement.

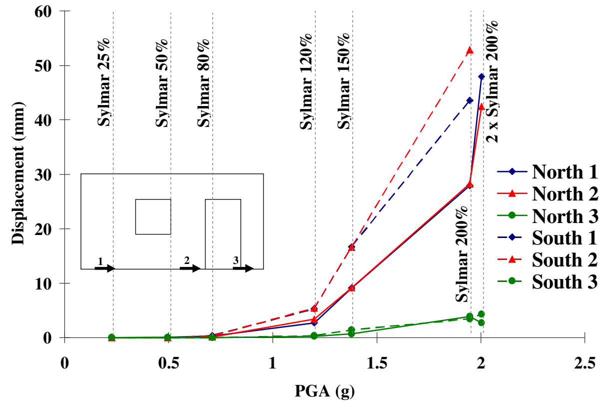


Figure 17. In-Plane Veneer Sliding vs. PGA

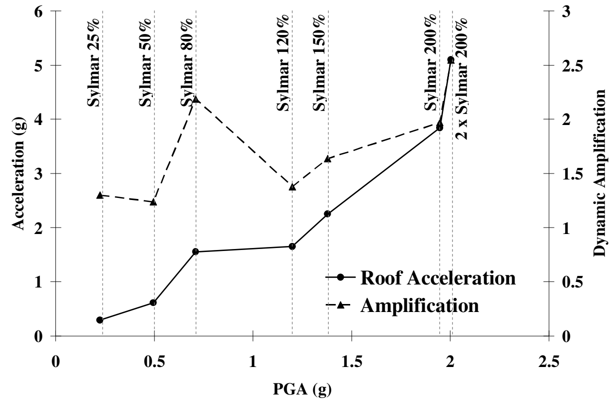


Figure 18. Roof Acceleration and Dynamic Amplification vs. PGA

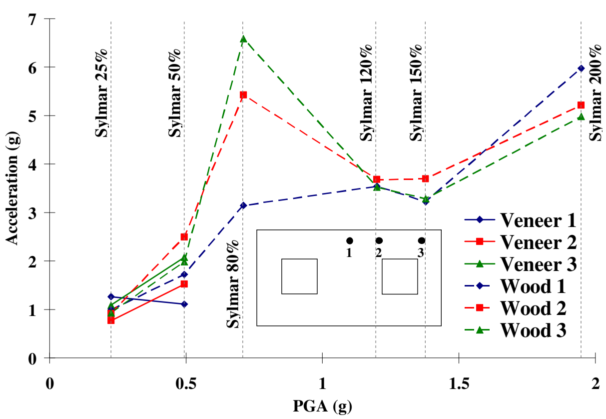


Figure 19. Out-of-Plane Acceleration vs. PGA (West Wall)

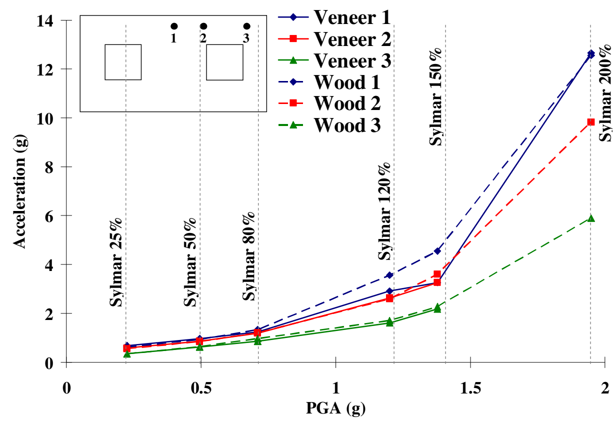


Figure 20. Out-of-Plane Acceleration vs. PGA (East Wall)

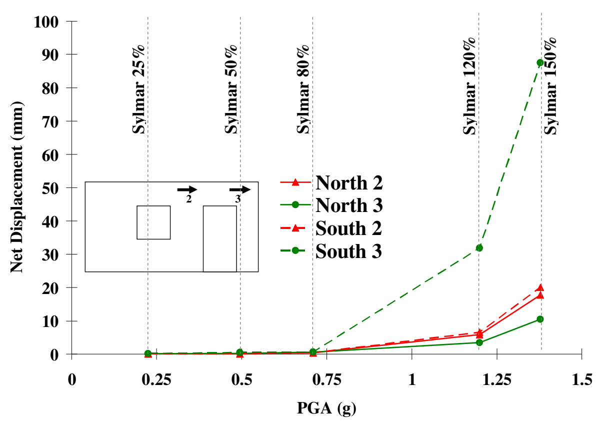


Figure 21. Net In-Plane Veneer Displacement vs. PGA

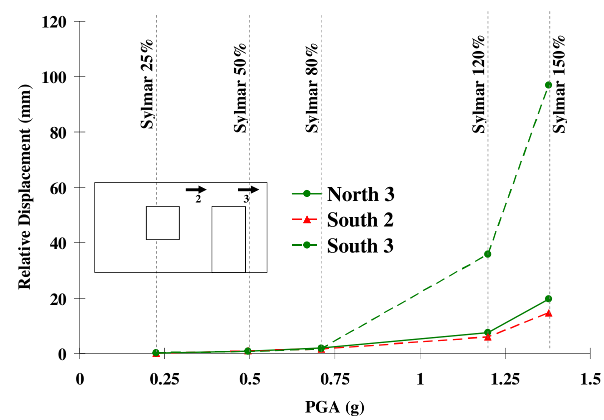


Figure 22. Relative In-Plane Top Displacement between the Veneer and the Wood Backing

Summary and Conclusions

A full-scale, wood-frame, masonry-veneer structure was tested on the NEES shaking-table at the University of California at San Diego. Except for one veneer wall, the wood-frame structure and the masonry veneer did not collapse under levels of shaking far in excess of a representative maximum considered earthquake (MCE) for SDC D (high seismicity). One veneer wall collapsed under an out-of-plane shaking that corresponds to a representative design-basis earthquake (DBE), while other parts showed distress only at levels of shaking in excess of MCE. The premature veneer failure was due to nail extraction of the corrugated veneer ties from the wood studs. Comparison of the test results with those of an earlier experimental study (Okail et al. 2008), part of the same research effort, revealed that the resistance of nails to extraction is highly affected by the moisture content of the wood. Nevertheless, given the fact that these nails certainly meet current requirements for anchored veneer, their performance casts doubt on the adequacy of these provisions. The results showed a striking contrast between the performance of veneer on the west side (where practically every connector failed by nail extraction), and the performance of veneer on the east side, which collapsed at more than twice the ground acceleration of the west side with bed-joint cracking and pullout of the veneer anchors from the fractured joints. Except for the nail extraction problem, clay masonry veneer, designed and constructed according to the requirements of the MSJC Specification, experienced only minor cracking and stayed fully connected to the wood-stud walls above MCE.

The interior gypsum wallboard showed some local distress, including local crushing at panel edges, and local in-plane cracking around door and window openings. Even though the small veneer segment in the south wall, which did not have joint reinforcement, exhibited more severe rocking and the whole south wall collapsed during the second run of Sylmar 200%, the north wall did not perform better and was on the verge of collapse at the same level of excitation. Both walls had their behavior governed by nail extraction. This study and the previous individual wall segment tests (Okail et al. 2008) have not shown any beneficial influence of joint reinforcement on the seismic performance of veneer walls. This calls into question the justification for current MSJC requirement for seismic clips and joint reinforcement for veneer in higher seismic design categories.

One aspect of observed performance that merits further studies is the behavior of wood diaphragms. The wood-stud elements of the test structure were designed and constructed precisely in accordance with current prescriptive requirements for SDC D2. Nevertheless, considerable extraction of the end nails was observed from the transverse blocking at the perimeter of the diaphragm on the east side. This is consistent with distress to the wood elements that were intended to transfer horizontal inertial forces from the diaphragm to the top plate of the wood-stud walls.

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