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SEISMIC PERFORMANCE OF CLAY MASONRY VENEER

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ABSTRACT:

This paper presents an experimental investigation of the out-of-plane seismic behavior of clay brick veneer over wood-stud walls. A series of shaking-table tests was performed on wall panels consisting of a clay brick wythe attached to a wood-stud frame with metal anchors. Different anchor types and spacings conforming to the current design standards in the US were used. Some walls had a window opening and some did not. The walls were tested to failure through shaking in the out-of-plane direction with increasing earthquake ground motion levels. This paper presents a summary of the experimental program and results, design implications, and an appraisal of the current code provisions. The test results indicate that the veneer walls could sustain ground motion levels considerably beyond the maximum considered earthquake.

KEYWORDS: Seismic Performance, Shake Table, Brick Veneer, Veneer Anchors, Wood Frames.

1. INTRODUCTION

Brick veneer is often found in low-rise residential and commercial constructions in many parts of the United States [1]. The system consists of a clay brick wythe backed by a wood-stud frame. The brick wall is connected to the backing through metal veneer anchors (also referred to as veneer ties) [2]. The ties span over an air gap (typically 25 or 50 mm), acting as a drainage cavity to allow the passage of the moisture out of the structure through weep holes located at the bottom brick course. A waterproof flashing is placed below the veneer wythe to prevent water permeation at the base [3]. As a non-structural component, a veneer is designed to support only its own weight and to transfer face loads, like wind- or earthquake-induced forces, to the backing system, which is normally part of the load-carrying system of the building [4]. Under earthquake loads, however, the veneer and the backing system will develop a composite action when the two are securely tied. The interaction of the two is influenced by their respective stiffness and inertial properties, and may result in significant tie forces. A veneer wall system can be subjected to both in-plane and out-of-plane loading. Veneer ties can be subjected to high demands of axial and shear forces depending on the direction of the earthquake excitation. Under a severe excitation, such systems can fail by fracture of the veneer at the mortar joints or by failure of the ties due to various causes. Under the auspices of the George E. Brown, Jr. Network for Earthquake Engineering Simulation Program (NEES) of the US National Science Foundation, a collaborative research project has been carried out to study the seismic performance of brick veneer and brick veneer anchors over wood-stud and masonry backing systems. This paper presents results from a study of the seismic performance of clay brick veneer backed by wood-stud walls. In this study, a set of experiments was conducted on single wall panels representative of typical sections of a one-story building. The walls were tested in their out-of-plane direction under increasing ground motion levels.

2. EXPERIMENTAL PROGRAM

2.1. Details of Tested Walls

Seven single wall panels were tested to investigate the out-of-plane behavior of brick veneer over wood-stud backing. Table 1 and Figure 1 present a summary of the design and the details of the tested walls. The specimens were designed in accordance with the structural design provisions of the Masonry Standards Joint Committee (MSJC) [5,6] and the International Residential Code (IRC) [7], and also followed the serviceability recommendations of the Brick Industry Association (BIA) Technical Notes [2,3]. The veneer walls were constructed by professional masons according to common practice. They were constructed over a concrete slab with a 30-mil (0.76-mm) Ethylene Propylene Diene Monomer (EPDM) flashing. Clay masonry units were laid in running bond using nominal 101-mm x 68-mm x 203-mm standard modular brick units conforming to ASTM C216 [8]. Type N masonry cement mortar was used conforming with the single-bag proportion specification of ASTM C270 [8] (masonry cement-to-sand ratio of 1:3 by volume). The veneer wythe was separated from the wood-stud backing with a 25-mm air gap. The wood-stud backing was assembled using 38-mm x 89-mm Standard Grade Douglas Fir studs according to IRC requirements [7]. The vertical studs in each wall were spaced at 406 mm on center with a double top plate and a sole plate nailed to the vertical studs using two 16d end nails. An 11-mm thick oriented strand board (OSB) and a 12-mm gypsum wall board were attached on the exterior (veneer) and interior sides of the wood-stud frame respectively, according to IRC requirements [7]. The sole plate was connected to the reinforced concrete foundation using 12-mm threaded rods cast into the concrete. In addition, the exterior vertical studs were connected to the foundation using Simpson strong ties (HDU4-SDS2.5). Two types of metal ties were used to anchor the brick wythe to the wood stud backing. The first was 22-ga. corrugated sheet metal ties anchored to the backing with standard 8d nails, while the second was 16-ga. rigid ties (steel brackets bent at a 90-degree angle) connected to the backing with 8d screws. Joint reinforcement, wherever used, was 9-ga. wires mechanically attached to the ties through a built-in hook.

Table 2.1 Details of Tested Panels

Specimen ID	Nominal Dimensions	Window Opening	Anchor Type	Joint Reinforcement	Horizontal Tie Spacing	Vertical Tie Spacing	SDC
Wood 5	1.22 m x 2.44 m		Corrugated		406 mm	406 mm	D
Wood 6	1.22 m x 2.44 m		Corrugated	X	406 mm	406 mm	E
Wood 7	2.44 m x 2.44 m	X	Corrugated	X	406 mm	406 mm	E
Wood 7X*	2.44 m x 2.44 m	X	Corrugated	X	406 mm	406 mm	E
Wood 8	1.22 m x 2.44 m		Corrugated		406 mm	203 mm	D+**
Wood 9	1.22 m x 2.44 m		Rigid	X	406 mm	609 mm	E
Wood 10	2.44 m x 2.44 m	X	Rigid	X	406 mm	609 mm	E

* Same as specimen Wood 7 but without the top row of ties ** Upgraded east-coast solution

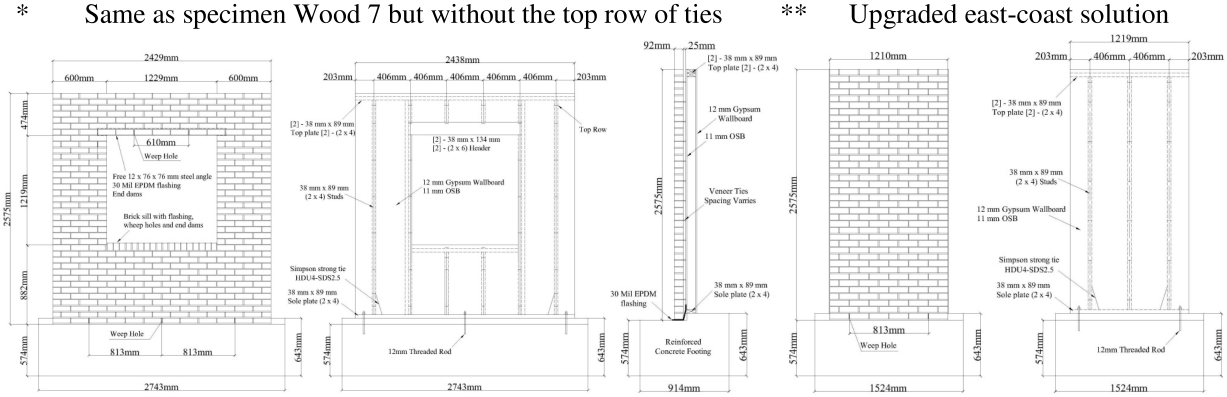

Figure 1 – Typical Details of Tested Specimens [Wood 7 (Left) and Wood 5 (Right)]

2.2. Shake Table Experiments

The specimens were tested on the NEES Large High Performance Outdoor Shake Table (LHPOST) at the University of California at San Diego. The walls were secured onto the table by 45-mm steel post-tensioning rods. Walls were oriented parallel or perpendicular to the direction of the table motion to impose the desired in-plane and out-of-plane shaking, respectively (Figure 2). This paper addresses only the out-of-plane tests. Steel support frames were used to restrain the double top plate of the wood-stud backing. Displacement transducers and accelerometers were used to monitor the relative displacements and total accelerations of the veneer and the wood-frames at the positions of the veneer ties. The accelerometers were mounted on the specimens while the displacement transducers were mounted on wooden reference frames located close to the tested walls. Most of the instruments on the veneer side of a specimen were removed prior to the anticipated failure run of the shake table to avoid their damage during collapse.

Two ground motion records from the 1994 Northridge (California) Earthquake were used for the tests. They are the Sylmar – 6 story County Hospital Parking Lot record (360 degree direction) and the Tarzana – Cedar Hill Nursery A record (90 degree direction). Figures 3a and 3b show the acceleration time histories for both records. Each wall was first subjected to a sequence of Sylmar ground motions scaled up to 150% of the original level, and then to a sequence of scaled Tarzana ground motions. The scaling was based on a design response spectrum for Seismic Design Category (SDC E) according to ASCE 7 [9] considering structural periods less than 0.3 seconds. The fundamental periods of the specimens were expected to be 0.15 seconds or lower. The design spectrum is shown in Figure 4 along with the original and scaled Sylmar and Tarzana records. As shown in that figure, for the expected fundamental frequencies of the specimens, the design basis earthquake (DBE) corresponds to 80% of the original Sylmar record and 36% of Tarzana, while the maximum considered earthquake (MCE) corresponds to 120% Sylmar and 54% Tarzana. After applying the highest level of the Sylmar record (150%), the Tarzana record was then applied at 70% of its original level to give the same spectral ordinate as 150% Sylmar. After each earthquake record, white-noise excitation was used to assess the dynamic properties of each specimen and to track the progression of damage. The white noise had peak ground acceleration (PGA) of 0.03g and swept a frequency range of 1 – 33 Hz.

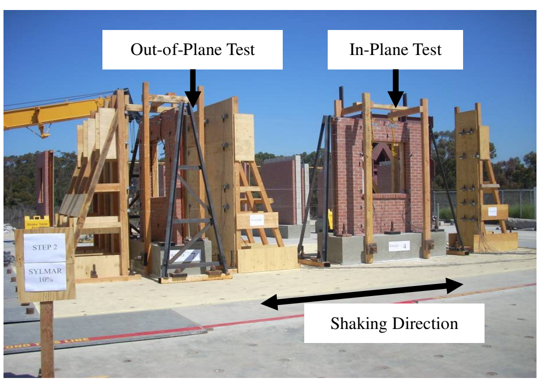

Figure 2 – Test Setup

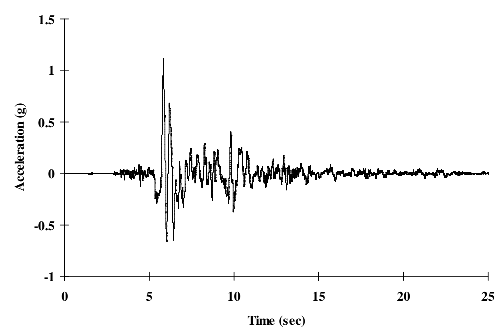

Figure 3a – Sylmar Record

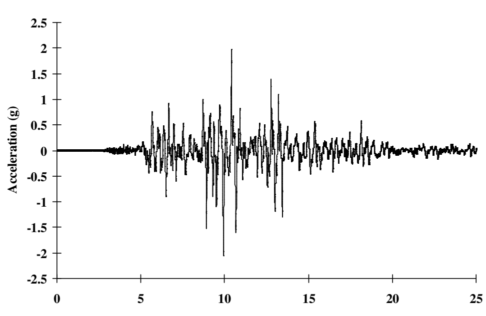

Figure 3b – Tarzana Record

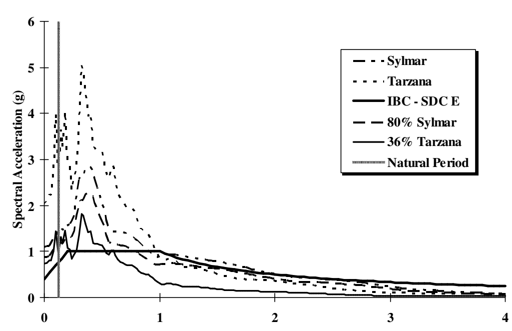

Figure 4 – Response Spectra for 5% Damping

3. DISCUSSION OF EXPERIMENTAL RESULTS

3.1. Summary of Experimental Observations

A summary of the test observations is presented in Table 3. For Wood 5 and Wood 6, which had corrugated ties, damage was initiated by minor nail extraction from the wood studs and bed-joint cracking close to the mid-height of the wall. Under higher ground motion levels, the lateral deflection of the veneer at the cracked bed joint increased, leading to more nail extraction from the studs (Figure 5). Eventually an additional bed-joint crack formed above the first crack and the lower portion of the wall rotated about the base, leading to the formation of a collapse mechanism. Wood 8, which had closer vertical tie spacing than Wood 5 and Wood 6, failed by the formation of a bed-joint crack five courses from the top of the wall with a combination of nail extraction from the studs and tie extraction from the mortar joints at more or less the same ground motion level as Wood 5. It is worth mentioning that the top row of ties in this wall had higher tributary area than the other rows along the wall. Wood 7, which had a window opening, failed in the piers with cracks at the top, bottom, and mid-height. Wood 7X failed under a significantly lower ground motion level than the other walls due to the omission of the top row of ties. Failure occurred by the formation of a bed joint crack at the base of the lintel accompanied by the complete extraction of the ties in the first row above the window. For the walls with rigid ties (Wood 9 & Wood 10), failure was sudden and marked by pullout of ties from fractured mortar joints (Figure 6). Pullout of the screws from the studs and slippage of screw heads through deformed tie holes were also observed at some locations. With the exception of Wood 7X, which failed under 125% of Sylmar, all the walls had veneer collapse occurring at levels much higher than MCE, which corresponds to 54% of Tarzana and a PGA of 1.11g.

Table 3 – Behavior of Tested Specimens

Specimen ID	Veneer Dims. (m)	Window	Anchor Type	Joint Reinf.	Horizon. Tie Spacing (mm)	Vert. Tie Spacing (mm)	PGA	Failure Mode
Wood 5	1.22 × 2.44		Corr.		406	406	2.59 g	Nail Pullout-Bed Joint Cracking
Wood 6	1.22 × 2.44		Corr.	x	406	406	3.08 g	Nail Pullout-Bed Joint Cracking
Wood 7	2.44 × 2.44	x	Corr.	x	406	406	2.55 g	Nail Pullout-Bed Joint Cracking at window pier
Wood 7X*	2.44 × 2.44	x	Corr.	x	406	406	1.35 g	Nail Pullout-Bed Joint Cracking at lintel beam
Wood 8	1.22 × 2.44		Corr.		406	203	2.56 g	Nail Pullout-Bed Joint Cracking at top row
Wood 9	1.22 × 2.44		Rigid	x	406	609	2.03 g	Screw Pullout from tie-Bed Joint Cracking-Tie Pullout form mortar joint
Wood 10	2.44 × 2.44	x	Rigid	x	406	609	2.53 g	Screw Pullout from tie-Bed Joint Cracking at window piers-Tie Pullout form mortar joint


Figure 5 – Nail Extraction from Studs (Corrugated)


Figure 6 – Tie Extraction from Mortar Joints (Rigid)

3.2. Detailed Evaluation of Wall Behavior

Acceleration time histories from the veneer and the backing under the white noise excitation were analyzed for all the walls to determine the change of the fundamental frequency of each specimen at different levels of earthquake excitation. Apart from Wood 7X, which failed early under 125% of Sylmar, all the walls had a reduction of the fundamental frequency between 2% - 15% of its initial (intact wall) value up to the MCE. This reduction although small indicates the progressive damage of the walls. This could be attributed to the micro-cracking of the bed joints, the straightening of the corrugated ties, and minor nail extraction from the backing as observed in the tests.
Figures 7a and 7b compare the time histories of relative displacements between a veneer and the backing wall in Wood 5 and Wood 9 under 70% of Tarzana at elevations where the relative displacements are most severe. These reflect the extent of tie deformation. The figures show that the ties generally experienced more tensile (positive) deformation than compressive deformation. For the corrugated ties in Wood 5, this can be attributed to the presence of mortar droppings between the veneer wythe and the wood-stud backing (Figure 5), which prevented the buckling of the ties under compression. Figure 7b shows that the compressive deformation of the rigid ties in Wood 9 is extremely small. This can be attributed to the higher stiffness of the rigid ties.

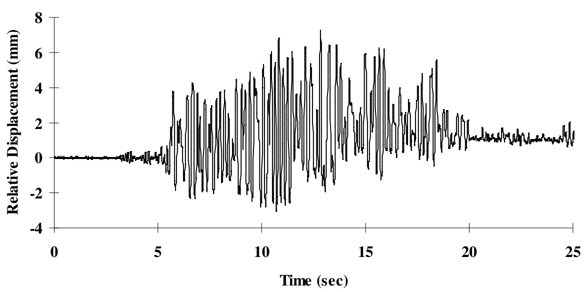

Figure 7a – Tie Deformation for Wood 5

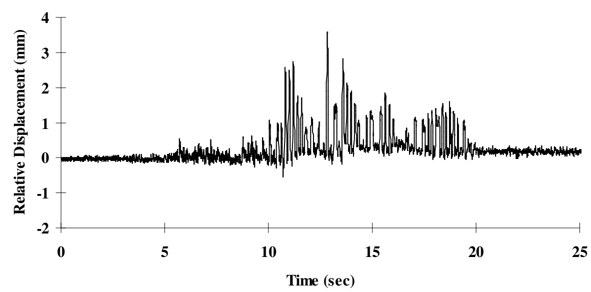

Figure 7b – Tie Deformation for Wood 9

Figure 8 presents the plots of the absolute maximum accelerations of the backing walls against the peak ground accelerations for all the tested walls. The plots show that the walls experienced a dynamic amplification of about two for the Tarzana ground motion and less than two for the Sylmar ground motion. This is due to the fact that the Tarzana record has more severe high-frequency components close to the fundamental frequencies of the walls as shown in Figure 4. Furthermore, with the exception of Wood 7X, the curves are more or less linear. The failure of Wood 7X occurred much earlier than that of Wood 7 due to the omission of the top row of ties, which resulted in a much larger tributary area for the remaining row of ties above the window opening. As a result, in Wood 7X, the entire veneer above the window collapsed at 125% of Sylmar as shown in Figure 9, while Wood 7 had pier failures under a much higher ground acceleration as shown in Figure 10.

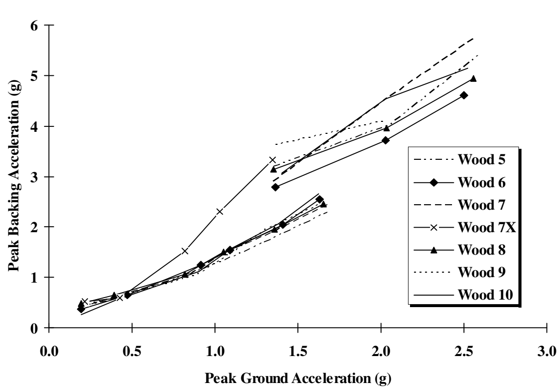

Figure 8 – PGA vs. Peak Response Acceleration

Figure 9 – Failure Mode for Wood 7X

Figure 10 – Failure Mode for Wood 7

3.3. Estimation of Tie Capacity

Since the mass of a wood-stud wall is negligible compared to that of the veneer, the veneer acts as the sole driving inertial mass with the wood stud wall following its motion. This mode of deformation induces tensile forces in the veneer ties when the veneer wall moves away from the backing. However, the tie forces are influenced by the bending resistance of the veneer as well as the inertia forces. Indeed, analysis has shown that the variation of the tie forces along the wall height is influenced by the cracking of the veneer. For an uncracked veneer, the tie forces are expected to be much higher at the top and the bottom of the wall than at mid-height. However, for a veneer that is cracked close to the mid-height, the distribution of tie forces changes, with high tie forces near mid-height and lower forces at top and bottom. This confirms the observations that most of the walls first experienced tie pullout near mid-height. When a veneer cracks, the vertical distribution of tie forces is governed by the inertia force. Hence, one can access the tie force at each elevation based on the measured acceleration in the veneer at that level. In this study, the tie capacity has been conservatively estimated as the tie force beyond which the tensile stiffness of the tie changes significantly, rather than the ultimate pullout force. Figures 11a and 11b show the displacement profiles of the veneer and wood-stud backing, and the acceleration profile of the backing along the height of Wood 5 at the instant this tie force was reached at about mid-height. This occurred when the specimen was subjected to 100% of Tarzana, which is about twice MCE. The position of the bed-joint crack in the veneer is depicted by the horizontal line in Figure 11a. Since data on veneer acceleration after 70% of Tarzana are not available for most of the walls, it is assumed that the veneer and the backing wall had more or less the same acceleration because of the small tie deformation. This acceleration is used to assess the tie capacity at the most critical location. The tie capacities calculated in this manner for the tested walls are presented in Table 4. These values are calculated based on a unit weight of masonry of 1650 N/m^2, which was measured from masonry prisms.

Table 4 –Tie Capacities

Specimen ID	Tributary Area	Backing Acceleration	Tie Capacity
Wood 5	0.165 m^2	2.98 g	811 N
Wood 6	0.165 m^2	3.19 g	868 N
Wood 7	0.165 m^2	3.01 g	819 N
Wood 8	0.092 m^2	4.46 g*	888 N
Wood 9	0.248 m^2	2.85 g	1166 N
Wood 10	0.248 m^2	3.44 g	1407 N

* Veneer acceleration

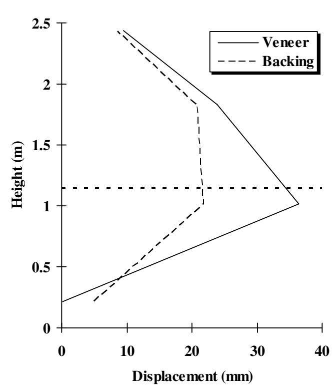

Figure 11a – Displacement Profile (Wood 5)

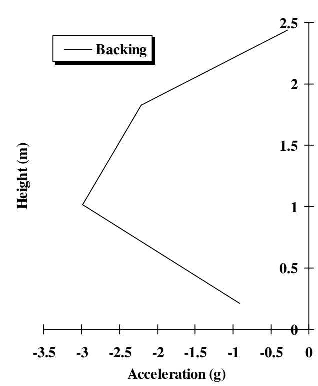

Figure 11b – Acceleration Profile (Wood 5)

4. DESIGN IMPLICATIONS

4.1. Influence of Joint Reinforcement

The argument in favor of joint reinforcement is that it will hold pieces of cracked veneer together and improve the anchorage of the ties in the veneer. Results of this study do not support this contention. Test results show that cracks occurred consistently in bed joints without any stepping into head joints. Hence, the joint reinforcement could not have improved the out-of-plane behavior of a cracked veneer. Furthermore, as Table 4 shows, the tie capacity for Wood 5, which had no joint reinforcement, is only slightly lower than that for Wood 6, which had joint reinforcement. In fact, the failure of the ties in both walls was induced by nail pullout.

4.2. Tie Performance

Analysis of test data shows that the corrugated and rigid ties had an average tie pullout capacity of 847 N and 1287 N respectively. This capacity should also be affected by the type and grade of the wood studs. It is worth mentioning that the use of screws with a larger head diameter for the rigid ties may improve their performance by preventing the pullout of a screw through the deformed hole of a rigid tie.

4.3. Tie Spacing

The tests showed satisfactory performance of all the veneer walls under the design basis and maximum considered earthquakes for SDC E. The key parameters affecting the performance of such wall systems are the tributary area of a tie and the tie capacity. These two parameters determine the maximum acceleration that can be experienced by a wall without failure. The tie capacities deduced from this study can be used in analytical models to evaluate the adequacy of current code provisions for other structural configurations using the same type of metal anchors.

5. CONCLUSIONS

Results of the shaking-table testing indicate that clay masonry veneer walls over wood-stud backing, designed according to the current MSJC requirements for Seismic Design Categories D and E, had satisfactory performance. The tensile capacity of the ties deteriorated at levels of shaking about twice the maximum considered earthquake, and the veneer walls did not collapse until even higher levels of shaking.

The out-of-plane behavior of clay masonry veneer over wood-stud walls was governed by nail pullout from the backing for the corrugated ties, and by pullout from mortar joints for the rigid ties. Walls with corrugated ties and with rigid ties showed comparable overall strength. The rigid ties had a higher capacity but a larger vertical spacing. However, the failure of the veneer with the rigid ties appeared to be more sudden. No obvious behavior enhancement was observed for the specimens with joint reinforcement, which may be explained by the ineffectiveness of joint reinforcement when the behavior is dominated by bed-joint cracking.

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