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SEISMIC PERFORMANCE OF BRICK VENEER ON WOOD FRAME CONSTRUCTION

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SUMMARY

Full-scale brick veneer wall panel specimens, representing typical residential construction practice, were investigated under static and dynamic out-of-plane lateral loading on a shake table. The tests captured the overall performance of the wall system, including interaction and load-sharing between the brick masonry veneer, corrugated sheet metal ties, and wood frame backup. The tests evaluated the effects of two different tie installation methods, as well as a veneer-to-backup connection repair using post-installed mechanical anchors. The progression of system damage was noted up until partial collapse of the veneer walls; tie stiffness and strength were found to significantly affect wall performance at all stages of behavior. Analytical models for veneer wall systems are being developed based on results from the shake table and (earlier) tie subassembly testing programs. Additional research is underway to explore the effects of different architectural features on veneer wall performance.

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INTRODUCTION & BACKGROUND

A common type of residential construction, particularly in the central and southeastern United States (areas of low to moderate seismic activity), is a wood-framed structure with brick masonry veneer. Brick veneer construction is especially valued for its appealing appearance, excellent thermal performance, and prevention of water penetration [1]. Residential brick veneer wall systems typically comprise an exterior masonry wall and interior wood backup framing, separated by an air cavity, as well as regularly spaced metal ties connecting the exterior masonry wall to the interior backup through the cavity [2]. The cavity acts as a thermal barrier, providing for drainage as well as weather resistance [3]. The design concept for a typical veneer wall system is that the wood backup should resist all gravity loads (except brick self-weight) and all lateral loads; however, veneer walls often do carry some of the lateral loads due to their relatively higher stiffness than that of typical wood frame backup construction [4]. Regardless of whether or not brick veneer walls are intended to participate in load resistance, there is no doubt that the veneer is an essential component of the overall building envelope.

In a veneer wall system, it is assumed that lateral loads on the exterior walls will be transferred to the backup through the metal ties, so the properties of these ties can play a key role in the overall behavior and performance of brick veneer walls. Corrugated sheet metal ties are typically used for connecting brick veneer to the wood backup in residential construction. These ties should satisfy performance requirements such as: a) sufficient strength and stiffness (in tension and compression) to transfer lateral loads, b) adequate transverse flexibility to accommodate differential movements, and c) resistance to corrosion and moisture transfer across the cavity [5]. Various design codes and other standards specify prescriptive construction requirements for the use of corrugated sheet metal ties on wood framing, such as minimum tie dimensions and maximum tie spacing [5-7]. However, ties that do not satisfy all of these requirements can often be found used in actual residential construction.

Brick veneer wall damage (including cracking, relative movement, and even collapse of masonry walls under out-of-plane loading) has been observed in recent years resulting from strong wind events and moderate earthquakes [8-9]. In some cases, the damage was explained by wall ties that were too widely spaced, too flexible, or not adequately anchored to the brick or the backup. Out-of-plane wall damage is most likely to occur as the veneer moves away from the backup, placing a high demand on the tensile force and displacement capacities of the ties. In recent years, a few experimental and analytical studies have been conducted on veneer walls subjected to wind and earthquake loads to understand the inter-relationship between the masonry, ties, and backup framing (metal and wood) [10-15]. However, for wood frame construction these studies have mainly focused on veneer systems built using older construction practices, and they did not fully explore strength limits of the ties [14-15].

To address more current and widespread brick veneer residential construction practice, a study has been undertaken at the University of Illinois (UIUC), primarily supported by the Mid-America Earthquake (MAE) Center, to investigate the out-of-plane performance of brick masonry veneer wall systems over wood framing. In the first phase of the study, tests were conducted on brick-tie-wood subassemblies, each consisting of two standard bricks, one 2x4 stud, and one corrugated sheet metal tie. The subassemblies represented a localized portion of a brick veneer wall system, and they therefore captured the interaction between each component of the system (rather than just the behavior of the tie itself). The test specimens represented a variety of possible tie installation conditions that could occur in actual brick veneer construction. Tie subassembly strengths, stiffnesses, and failure modes were determined for different types of loading, including: monotonic tension, monotonic compression, cyclic (low-cycle) tension-compression, monotonic shear, and cyclic shear [16].

The tie study led into the second phase of the project, described herein, which involved laboratory testing of full-scale brick veneer wall systems subjected to static and dynamic out-of-plane lateral loading

on a shake table. The test specimens were constructed to represent typical current construction practices. The experimental study was conducted to assess the overall behavior of brick veneer walls, including effects of different tie installation methods, as well as to test a possible retrofit strategy to enhance the veneer-to-backup connection. Test results from both the tie and wall experimental studies provide important input data for analytical models of residential brick veneer wall systems, which can lead to a better understanding of the vulnerability of this type of construction under both wind and seismic loading.

DESCRIPTION OF THE TEST STRUCTURE

A full-scale brick veneer and wood frame test structure was designed to represent a portion of the wall system in a single-family home. The one-story wall panel and surrounding components were proportioned and constructed based on typical residential construction practices, in general conformance with Brick Industry Association (BIA) Technical Notes [3-5], the Masonry Standards Joint Committee (MSJC) Code [6], and the International Residential Code (IRC) for One- and Two-Family Dwellings [7]. Two veneer wall specimens were constructed and tested; they were anchored to a wood frame using corrugated sheet metal ties, with each wall utilizing different tie installation methods. In addition, mechanical expansion anchors were temporarily installed in the second wall specimen to test a possible repair and strengthening procedure.

Supports and Wood Frame Wall Structure

Elevation views of the complete wall test structure are shown in Fig. 1. The brick veneer and wood frame wall structure was set up (to be excited in the out-of-plane direction) on a uniaxial shake table (with a 3.66 m x 3.66 m surface) in the Newmark Structural Engineering Laboratory (NSEL) at UIUC, as shown in Fig. 2. The brick veneer and wood frame were supported by a reinforced concrete foundation (representing the top portion of a typical foundation wall) and a steel reaction frame (designed to provide support for the wall structure in a manner similar to that found in a typical home). Pinned-end steel cross beams supported the back ends of the floor and roof/ceiling framing stub joists. Although the steel braces at the ends of the wall were quite stiff, the cross beams were flexible enough to permit some relative out-of-plane movements of the floor and roof/ceiling framing.

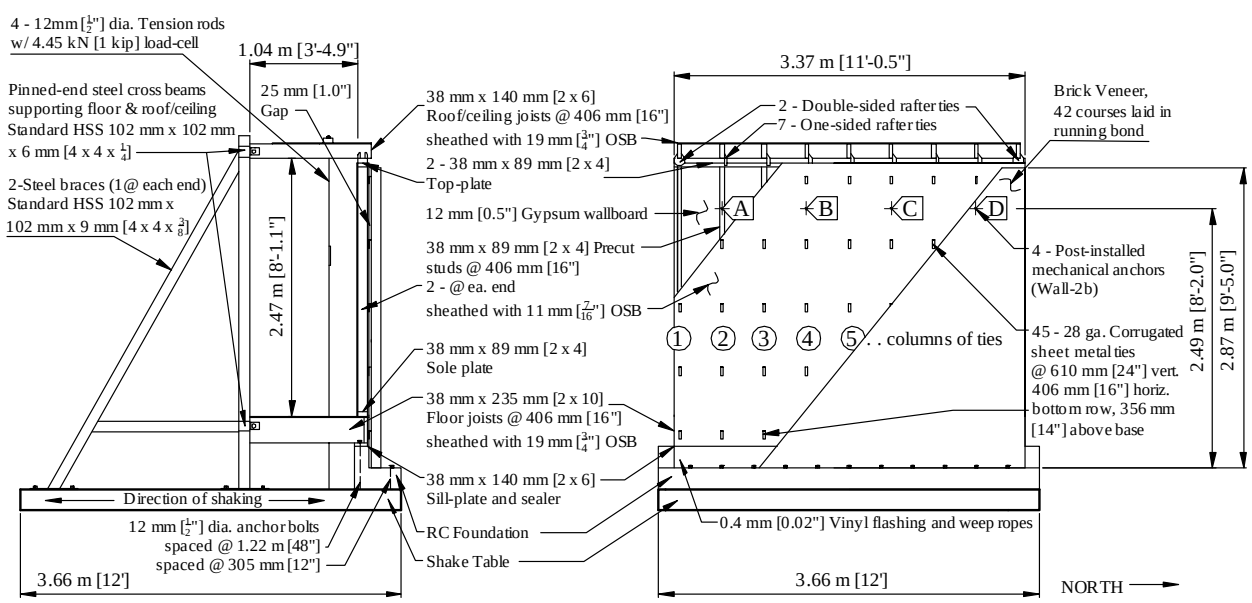

Fig. 1. Elevation views of the wall test structure.

The 3.37 m length of the test structure wall panel was governed by the size of the shake table. Similar length (and longer) exterior walls without openings are often found in residential construction, particularly at garages. The wood framing in the test structure used Standard Grade Spruce-Pine-Fir; the 2x4 (38 mm x 89 mm) stud wall (with exterior oriented strand board (OSB) sheathing) and surrounding wood frame components were constructed in conformance with IRC requirements. The wood wall panel rested on a partial floor frame, which was in turn supported by a sill-plate, attached to the concrete foundation (Fig. 1). The top of the wood wall panel supported the outer edge of the partial roof/ceiling diaphragm, which in turn laterally braced the top of the wall across its length. The roof/ceiling joists were attached to the wall frame top-plate (exterior face) with one-sided rafter ties, using 8d nails. At each of the outermost joists, two one-sided (on the exterior face) and one double-sided (on the interior face) rafter ties were installed to provide additional strength and stiffness at the corner connections of the wood wall panel. The IRC specifies that roof/ceiling joist connections to an exterior wall top-plate should include toe nailing plus rafter ties spaced no more than 1.2 m on-center along the length of the wall (with more rafter ties added as needed for higher design wind uplift resistance). The test structure employed rafter ties at each connection to ensure sufficient durability for carrying repeated loads from the dynamic tests of a couple of veneer walls, while also permitting a connection flexibility comparable to toe nailing.

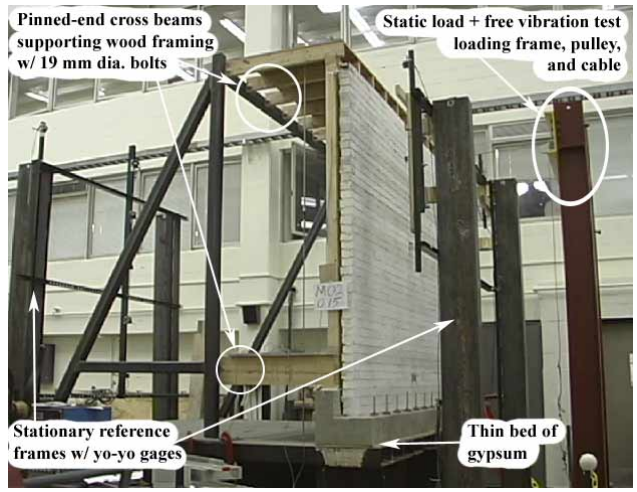

Fig. 2. Wall test structure setup on the shake table.

A simple pre-compression system was installed to include the effect of roof/ceiling dead load on wall behavior; however, the mass of this dead load was not represented. Four 12 mm diameter threaded rods were fastened to the shake table and ran up through small holes in the subflooring, roof/ceiling sheathing, and a 2x4 laid flat across the top of the sheathing (Fig. 1). A nut at the top of each rod was adjusted to provide 2.7 kN of tension, transferring a compressive force to the stud wall of approximately 0.8 kN per stud. The pre-compression load was chosen based on an evaluation of typical design dead loads for residential construction, in conjunction with detailed material quantity take-offs for residential home roof structures.

Brick Masonry Veneer

The 2.87 m tall and 3.37 m long brick veneer walls of the test structure were constructed by professional masons. The test specimen veneer walls had free edges (open ends), similar to those found in residential construction with "front face" veneer walls only (where the masonry is terminated at a corner and some other siding material is used on perpendicular exterior walls). Veneer wall separation at edges is also common in practice at corners with control joints and near large window and door openings, which permit individual sections of veneer to move independently of one another [10]. The bricks used were 89 mm x 194 mm x 57 mm standard modular "Colonial Reds" with three holes, joined by type N mortar (cement : lime : sand = 1 : 1 : 6) in running bond. Mortar for the first course of bricks was placed on flashing material, which had little bond to the top of the foundation surface (Fig. 1). (Flashing, though not always used in residential construction, is mandated by building codes to collect and discharge condensation at foundation wall surfaces and around openings.) Masonry material properties were evaluated by performing standard ASTM tests: the masonry compressive strength (f'_m) was 23.5 MPa; the modulus of elasticity (E_m) was 13.9 GPa; the modulus of rupture (f_r) was 596 kPa; and the mortar compressive strength (f_u) was 6.4 MPa.

During construction, an air space of 25 mm was maintained between the outside face of the wood frame sheathing and the inside face of the brick veneer. While laying the brick, some excess mortar seeped out into the air space, on the interior face of the veneer; at a few locations, mortar even landed on the corrugated sheet metal ties and locally filled in the space between the veneer and the sheathing. (Small amounts of excess mortar cannot be avoided, but recommended practice is to limit such "mortar droppings" into the air space as much as possible, as they may provide a conduit for moisture movement across the cavity, while at the same time impairing drainage of moisture out of the cavity.)

Veneer Ties and Anchors

As part of masonry installation, brick veneer was attached to the wood frame wall panel test structure using corrugated sheet metal ties. The use of at least 22 ga. (0.8 mm thick) ties is required by MSJC and IRC; however, for the wall test specimens, typical residential construction practice was followed by using thinner 28 ga. (0.4 mm thick) ties, as shown in Fig. 3a. Tie spacing was also based on typical construction practice, namely 406 mm horizontally and 610 mm vertically, which is in general conformance with various specifications, as listed in Table 1. A total of 45 ties were installed in the wall specimens with galvanized 8d nails, arranged in nine columns (numbered 1 through 9, starting at the south edge) and five rows (Fig. 1).

For the first series of tests (Wall-1), all of the ties were installed following a typical "best-case" construction practice, where the 90-degree bend was located at the nail, as shown in Fig. 3b, with just a small eccentricity of the bend due to the head of the nail. This wall specimen was subjected to static and dynamic tests up until partial collapse of the veneer (including destruction of the top two rows of ties), as will be described in more detail below. The wood frame backup and the remaining veneer did not sustain visible damage during the first set of tests, so they were kept in place for a second series of tests (Wall-2). For Wall-2, the collapsed portion of the veneer from Wall-1 was rebuilt with new brick masonry, attached using the same type of ties. However, the 90-degree bend in these ties was located 12.7 mm above the nail (also shown in Fig. 3b); this eccentricity of the bend is the maximum permitted by MSJC and IRC. (Analysis of test results from Wall-1 showed that ties in the upper rows dominated the

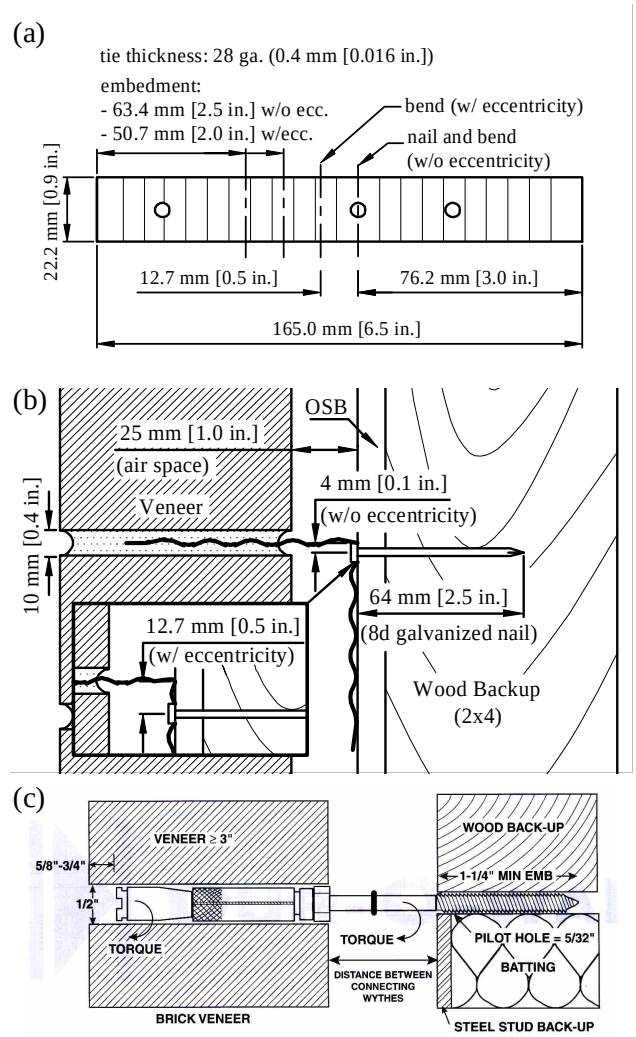

Fig. 3. (a,b) Corrugated sheet metal ties and (c) *Series 5300 Dur-O-Wal* anchors [17].

Table 1. Maximum tie spacing specifications and actual specimen configuration

	Spacing, horizontal & vertical (mm)	Wall area per tie (m^2)
IRC	610 & n/a	0.30
MSJC	813 & 457	0.25
BIA	610 & 610	0.30
Specimen	406 & 610	0.25

wall's behavior, so the rebuilt specimen was intended to fully explore the influence of a different tie installation method, even though the bottom portion of the veneer wall system from the first series of tests was reused.)

Average behavior of a single 28 ga. tie under monotonic tensile and compressive loading is shown in Fig. 4 for both installation techniques, based on previous tests of brick-tie-wood subassemblies [16]. (Test results of 22 ga. ties for both installation methods exhibited somewhat higher initial stiffness and compressive strength; however, the ultimate tensile strength was similar to that of the 28 ga. ties. Cyclic tension-compression tie subassembly test results showed that the average envelope curves for cyclic behavior were similar to the companion monotonic test results.) Specifications for horizontal and vertical spacing at installation of the ties limit the strength demands placed on them; the BIA also recommends a minimum initial stiffness of 0.37 kN/mm (in both tension and compression) [5], which was only satisfied in subassembly tension tests for ties with the bend at the nail.

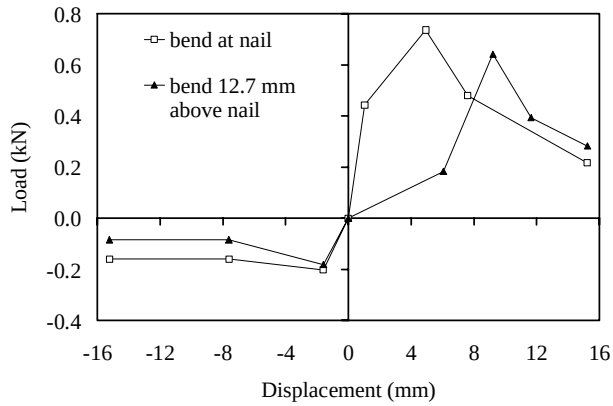

Fig. 4. Behaviors of 28 ga. ties [16].

While testing Wall-2, it was apparent that the veneer-to-backup connection was much more flexible than in Wall-1. To increase the connection stiffness and explore a possible retrofit strategy, four mechanical expansion anchors (Fig. 3c) (with an average ultimate tension capacity of 4.0 kN per anchor [17]) were installed between the top two rows of ties (designated as A through D in Fig. 1) for tests of Wall-2b. In effect, the four mechanical anchors were intended to completely replace the capacity of the top two rows of ties. These anchors were removed after their effects on wall performance were understood, and the wall specimen (Wall-2) was then subjected to additional dynamic tests until collapse.

SHAKE TABLE TESTING PROGRAM

Static Loading

Preliminary static displacement and free vibration tests were performed to evaluate the variation in lateral stiffness of the wall structure from before to after construction of the brick veneer, as well as any variation resulting from different veneer-to-backup connections. The static displacement tests were performed by applying horizontal point (pull) loads directly to the bare wood frame wall panel (before veneer construction) and then later to the brick veneer itself (before any damage had occurred due to subsequent dynamic testing). Free vibration tests were performed by suddenly releasing a point load applied to the brick veneer. For these purposes, a steel frame was constructed on the laboratory testing floor, facing the wall structure (Fig. 2), with an attached pulley aligned with the centerline of the wall (and adjusted for desired vertical location). Weights ranging from 1.1 to 1.8 kN were suspended on a steel cable, running through the pulley and linked to points on the wall, for static testing, and then released as needed for free vibration testing.

Input Motions

Three scaled earthquake records were used during shake table testing (as well as sine-sweep inputs to characterize the dynamic properties). The earthquake records included two synthetic motions and one recorded ground motion, chosen to be representative of intra-plate earthquakes found in the central and eastern U.S, as shown in Fig. 5. The synthetic motions were from a MAE Center project that,

among other things, developed records to represent seismic hazard levels in Memphis, Tennessee, with probabilities of exceedance of 10% and 2% in 50 years [18]. The two synthetic records used (labeled as m10_01s (M10) and m02_03s (M02)) were selected from sets of ten available at each hazard level; the specific records were chosen in part because they had the smoothest spectral accelerations in the low period range (a library of the synthetic ground motions can be found at [19]). The recorded ground motion used was from the Nahanni earthquake of December 23, 1985 (Site-1, component 010) [20]. The sine-sweep input, used to evaluate the dynamic properties, had a frequency range of 1 to 10 Hz and was scaled to very low peak ground accelerations (PGA) of approximately 0.02 to 0.04g. Throughout dynamic testing, the input PGA intensities were progressively increased (starting from very low values) by increments of approximately 0.04 to 0.06g.

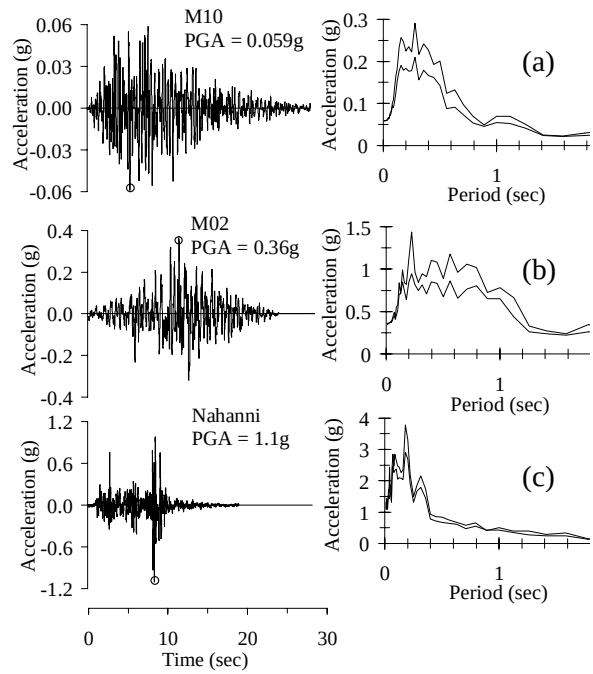

Fig. 5. Acceleration time histories and response spectra at 3% and 6% damping.

Specimen Instrumentation

Displacements and accelerations were measured at various locations on the wall specimens, as shown in Fig. 6. Veneer and backup displacements were measured along the vertical centerline and at the top south corner of the walls, at tie locations (except for Wall-1, where displacements were not measured at the top corner of the wood frame). Shake table "input" displacements were recorded by a transducer located in the actuator piston. Accelerometers were placed at five locations on the wall specimens and one on the shake table.

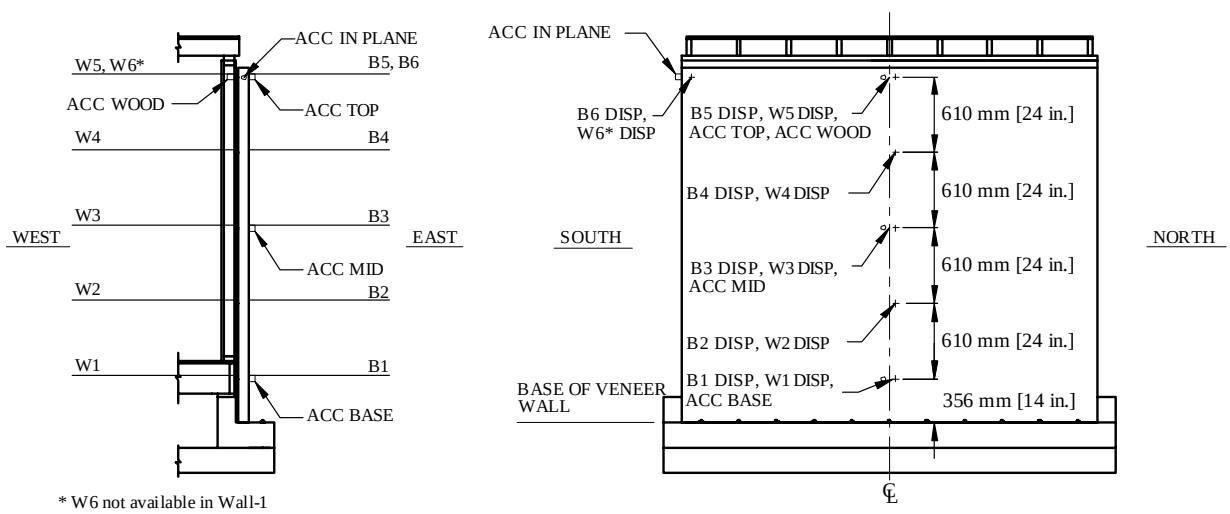

Fig. 6. Specimen instrumentation layout.

EXPERIMENTAL RESULTS

Static and dynamic shake table tests were conducted on the wall specimens following the procedures described above. The most important experimental results are summarized in Table 2 and explained in greater detail below.

Table 2. Summary of experimental results

			Top center of brick veneer:			Peak tie elongations (centerline):			(corner):	
	Ground Motion	PGA (g)	Acc. (g)	Disp. (mm)	Period (sec)	B3-W3 (mm)	B4-W4 (mm)	B5-W5 (mm)	B6-W6[a] (mm)	Damage
Wall-1					**0.10**					
elastic	M02	0.19	-0.38	1.0		0.5	0.5	0.6	1.0	
	M10	0.22	-0.47	1.6		0.4	0.5	0.7	1.5	
	M10	0.37	0.84	3.8	0.12	0.4	0.5	1.4	3.5	Cracks at mortar-to-concrete foundation interface
intermediate	M10	0.51	1.09	7.3		0.6	0.8	2.0	6.6	
	M10	0.58	n/a	n/a	0.15	n/a	n/a	n/a	n/a	Fracture: tie-1, top row
	NA	0.30	1.39	7.7		0.5	1.1	3.1	8.4	Fracture: tie-9, top row
ultimate	M10	0.66	2.19	17.5	0.22	0.8	2.9	8.4	18.5	Fractures: tie-8, top row; tie-1, second row from top Nail pullout (~6mm): tie-3, top row; tie-9, second row from top
	M10	0.64	-5.01	42.9		2.5	15.3	43.0	43.4	Veneer collapse
Wall-2					**0.17**					
elastic	M02	0.19	0.79	7.2		0.8	2.6	4.8	6.8	
	M10	0.23	1.52	9.3	0.23	0.7	2.8	6.0	7.4	
intermediate	M02	0.20	0.68	6.2		0.9	2.1	4.5	6.5	
	M10	0.22	-0.91	7.8		0.8	2.0	5.0	7.5	Fracture: tie-1, top row
	M02	0.24	0.75	7.8		0.9	2.0	4.8	8.3	Fracture and nail pullout: tie-1, third row
ultimate	M10	0.30	1.07	11.4	0.21	1.0	2.3	6.0	10.6	Crack across veneer
	M02	0.30	0.95	9.7		0.9	2.2	5.6	9.3	Partial fracture: tie-9, top row
	M10	0.41	1.63	13.2		0.8	2.9	7.1	11.2	Partial fracture: tie-4, top row Fracture: tie-6, top row
	M02	0.31	1.23	11.9		0.8	3.7	8.2	11.4	Nail pullout (~3mm): tie-5, top row
	M10	0.49	-2.98	46.9		5.0	23.6	44.0	48.6	Veneer collapse
Wall-2b					**0.14**					
	M02	0.18	0.41	2.4		0.8	0.7	1.1	2.2	
	M10	0.22	0.53	3.7	0.14	0.8	0.7	1.2	3.0	
	NA	0.35	1.78	8.2		0.8	0.7	2.0	4.8	
	M10	0.59	1.74	11.2	0.15	1.0	0.8	1.9	6.1	Slipping at anchor-A
	M10	0.67	2.00	12.9	0.16	1.2	0.9	2.2	6.8	
	M10	0.80	-3.00	13.4	0.17	1.0	1.4	2.7	7.5	

[a] For Wall-1 only, top south corner elongations are estimates; results listed are net brick displacements.

Preliminary Tests – Static Displacements and Dynamic Properties

Static load tests revealed some of the differences in the out-of-plane lateral stiffness of the wall structures. Point (pull) loads of 1.3 kN and 1.8 kN were applied to the bare wood wall panel directly at the top-plate and at 0.4 m below it, respectively. Overall, measured out-of-plane displacements (with respect to the shake table) along the centerline and at the top-south corner, shown in Fig. 7a, portray the

variation in stiffness throughout the wood wall panel. Displacements at the top middle of the wall panel indicate that the rafter ties were, in general, flexible enough to allow significant deformations across the wall panel-to-roof/ceiling joist connections.

Point loads were also applied to the veneer of Wall-1 (1.3 kN), and Walls-2 and -2b (1.1 kN) at 1.0 m and 0.4 m below the top of the veneer, respectively. The elastic displacements, shown in Fig. 7 (b through d), portray the effects of the different veneer-to-backup connections on overall wall deformations. As seen in Fig. 7e, the highest relative displacements (tie elongations) between the veneer and the backup along the wall centerline were measured in Wall-2; however, after installing the mechanical anchors (Wall-2b), the tie elongations resembled those measured in Wall-1. Also, the total displacements were significantly lower for the (undamaged) walls with veneer compared to the bare wood frame wall panel, which shows that the presence of the brick veneer enhanced the out-of-plane stiffness of the system. The brick veneer displaced almost as a rigid body, which in effect spread the applied point load to the tie and/or anchor connections, which then more uniformly distributed the load to the wood frame backup, resulting in smaller overall displacements.

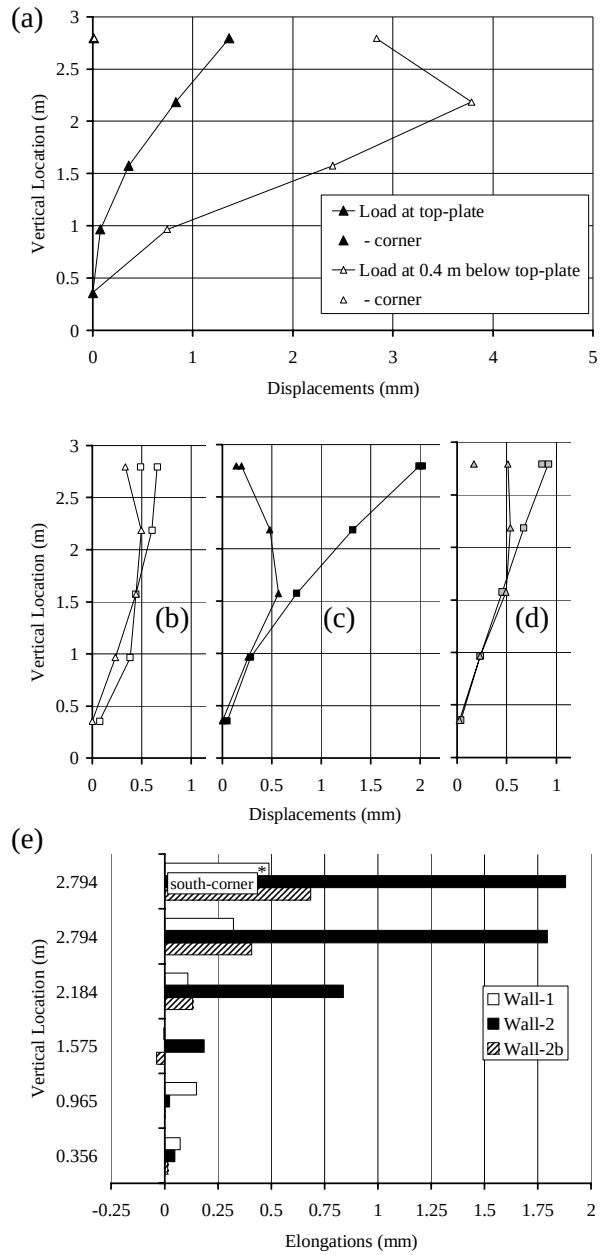

Fig. 7. Displacement results from static load tests. (a) Bare wood frame backup. Brick veneer and wood frame of (b) Wall-1, (c) Wall-2, and (d) Wall-2b. (e) Tie elongations (* – net brick displacement).

The free vibration period was evaluated before testing (upon suddenly releasing a static load applied to the veneer), and then again after certain dynamic tests of the wall specimens; it was an indicator of the variation in specimen stiffness for different veneer-to-backup connections, and also as damage occurred in the specimens. Furthermore, by performing forced vibration tests (using the sine-sweep record described above), the natural period of vibration could also be determined from the excitation frequency that caused a resonant response in the wall specimens [21]. Throughout wall testing, each method produced similar results, which are listed in Table 2. In addition, the viscous damping ratios were estimated for the wall specimens early in the dynamic testing sequence (before any visible damage) as approximately 3% and 5% for Walls-1 and -2, respectively (by applying the logarithmic decrement method to sets of experimental results that clearly exhibited the decay of motion amplitude [21]).

Dynamic Tests

Dynamic shake table tests were conducted to evaluate the overall performance of brick veneer walls under distributed (inertial) loads. For each wall specimen, sets of the most important dynamic test results are listed in Table 2. For Walls-1 and -2, the results correspond to three levels of specimen response and damage, which can be described as: *elastic* (no visible damage), *intermediate* (onset of tie and veneer damage), and *ultimate* (accumulation of tie and veneer damage sufficient to lead to collapse).

In general, the maximum response of the test specimens, corresponding to the peak measured accelerations and displacements, occurred shortly after the PGA of the earthquake input records. The peak acceleration values were collected from the acceleration traces measured on the wall specimens. The maximum positive displacements (veneer deflecting away from the backup) of the brick veneer and of the wood backup, as well as the peak positive tie deformations (elongations) were also of particular interest. The brick and wood displacements were evaluated as the difference between the wall and shake table values (i.e., they were the displacements relative to the shake table). The tie deformations were evaluated as the relative displacements between the brick veneer and the wood frame backup (except for at the top south corner of Wall-1, where wood backup displacements were not measured, so tie elongations were (over)estimated as the total brick displacement relative to the shake table).

Listed in Table 2 are: names of the input earthquake records with the scaled PGA; peak measured accelerations and displacements at the top-center of the brick veneer; period of vibration (evaluated after certain dynamic tests); tie elongations at the upper three rows, along the wall centerline, and also at the top south corner; and descriptions of tie and veneer wall damage (as appropriate).

Wall-1 was subjected to dynamic loading until collapse of the upper portion of the brick veneer, as shown in Fig. 8a, after having suffered a few tie fractures during previous dynamic runs. The collapsed veneer pivoted about a crack at the horizontal mortar joint above the 27th course of bricks, midway between the second and third rows of ties (from the top). All previously unbroken ties in the upper two rows, as well as the entire upper portion of the wall, were completely damaged at this stage.

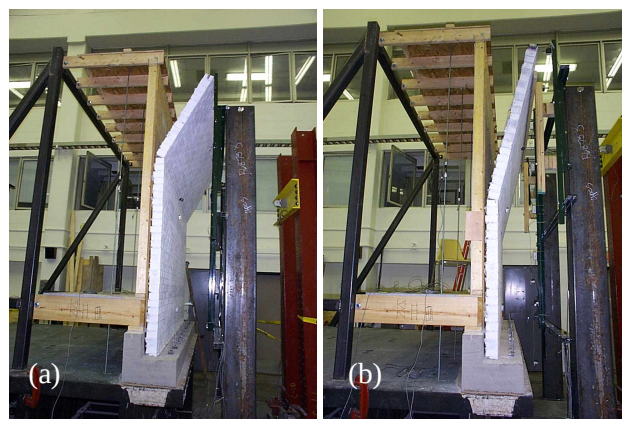

Fig. 8. Collapse of the brick veneer in (a) Wall-1 and (b) Wall-2.

Wall-2 was subjected to dynamic loading until the end of the *elastic* tests (following the M10-0.23g run), when the mechanical expansion anchors were installed for Wall-2b testing (as described below). After the anchors were removed, testing of Wall-2 resumed by approximately repeating the pre-anchor runs of M02-0.19g and M10-0.23g, which exhibited similar response as before the retrofit and Wall-2b testing. (These results indicate that the overall wall structure did not sustain significant damage during the Wall-2b tests.) *Intermediate* and *ultimate* tests (M02-0.24g through M10-0.41g) resulted in further tie and veneer damage, as described in Table 2. Testing of Wall-2 continued until the upper half of the brick veneer collapsed about a horizontal crack (initial horizontal cracking occurred during an earlier run), as shown in Fig. 8b. Any remaining unbroken ties in the top three rows were broken as the veneer collapsed. (Most of the tie failure modes noted in both of the veneer walls matched those observed in the individual tie subassembly study [16].)

Wall-2b was subjected to dynamic loading well-exceeding the collapse loading of Wall-1. There was no damage noted in the Wall-2b veneer or ties, except for modest slipping at anchor-A (Fig. 1),

resulting in a slight increase in the period of vibration. (The part of the anchor that expands and grips the masonry overcame the preload friction, which is expected to occur after about half the ultimate tension capacity is reached [17].)

ANALYSIS OF EXPERIMENTAL RESULTS

Veneer wall performance and damage observed throughout dynamic testing correlated well with the measured displacement data. The *elastic* tests characterized the dynamic response of brick veneer wall systems, and showed the effects of different tie installation methods (and post-installed anchors), without structural damage. Walls-1 and -2 showed similar overall displacements at the onset of tie damage (*intermediate* tests) and during collapse of the upper regions of the veneer (*ultimate* tests), while being subjected to quite different dynamic loads. When subjected to the highest dynamic input, Wall-2b reacted without any damage, exceeding the strength of the walls with corrugated sheet metal tie connections only.

Initial Response of the Veneer Walls

As the walls were subjected to dynamic inputs, the mass of the brick veneer produced inertial forces that were transferred through the ties into the wood frame backup. The different tie installation methods (and the post-installed anchors) affected the peak dynamic response of the wall specimens (generally, the trends during *elastic* response were similar to those from preliminary static load tests (Fig. 7b-e)). The brick veneer essentially displayed rigid body rotation about its base, as recognized from the almost linear veneer displacements measured along the centerline (in conjunction with the cracked mortar beneath the first course of bricks). The displaced shape of the wood frame backup depended on both the backup stiffness and the stiffness of the ties and/or anchors through which the veneer forces were transferred to the backup. As presented in Table 2, the ties in the top two rows were subjected to the highest inertial forces, resulting in the highest tie elongations, while the lower three rows of ties experienced relatively small peak elongations, typically comparable to or less than the third row peak elongations. In the top row, the corner ties experienced higher elongations than at the center, mainly due to variations in backup stiffness. In general, then, ties anchored near (or directly at) stiffer regions of the wood backup frame had to absorb the highest loads generated by movement of the brick veneer, whereas other ties had much less demand placed on them. (Experiments on a full-scale brick veneer home structure subjected to static lateral face loads (representing wind pressure) exhibited similar results for ties at different vertical locations [14].)

Towards the end of the *elastic* tests for Walls-1 and -2, the period of vibration increased due to some stiffness loss in the brick veneer-to-wood backup connections. Excess mortar in the air gap initially constrained some of the ties, protecting them from buckling, and also increasing their initial tension stiffness. The excess mortar also ensured that the 25 mm air gap could not close up very much in compression. (No buckled ties were observed after any of the dynamic tests; however, some results during testing did show relative displacements for the top row of ties in the negative direction that were in excess of the “tie buckling” displacements given in Fig. 4 from the previous subassembly tests.) At this point of specimen testing (end of the *elastic* range), tie elongations across the top row were just reaching the second stiffness slope on the idealized tension strength and stiffness curves (Fig. 4).

Onset of Veneer System Damage

First structural damage in the veneer wall systems (both Walls-1 and -2) occurred at the top south corner tie (tie-1). For Walls-1 and -2, Fig. 9 shows the peak displacements and tie elongations from the dynamic tests right before tie damage (M10-0.51g and M10-0.23g for Walls-1 and -2, respectively). The brick veneer and wood frame displacements were relatively similar in both walls, even though the dynamic loading intensities were quite different. The relative displacements (tie elongations), however, were different, with the veneer more closely coupled to the backup in Wall-1. During tests when the first

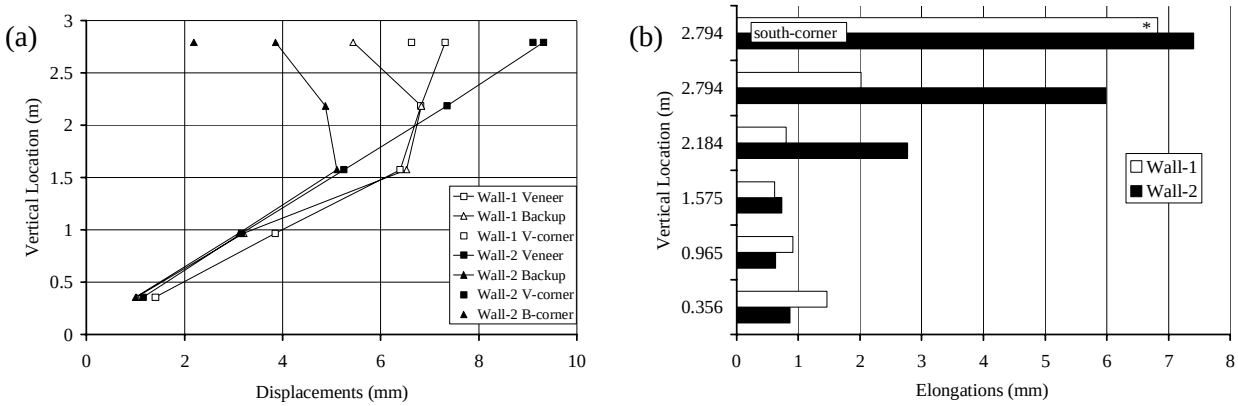

Fig. 9. (a) Peak positive displacements during dynamic runs right before first tie damage; and (b) peak tie elongations for those same tests (* – net brick displacement).

tie failures occurred, the peak tie elongation at the top corner in Wall-1 was estimated to be approximately 5 mm (because of missing data for the M10-0.58g run, the peak veneer displacements at the top corner were considered from similar dynamic tests, and a peak wood backup corner displacement was estimated from similar Wall-2b tests); the peak tie elongation at the top corner in Wall-2 was measured as 7.5 mm, during the M10-0.22g run. The top corner tie elongation at first tie fracture in Wall-1 was directly at, and in Wall-2 was slightly less than, the opening displacements (elongations) at ultimate tensile loading determined from the tie subassembly (monotonic tension) tests [16]. The failure modes of the top corner ties (which were comparable to failures seen in the tie subassembly cyclic load tests) suggest that low-cycle fatigue may have contributed to their fracture.

In Wall-2, a crack formed in the mortar bed joints across the mid-height of the brick veneer because of the more flexible veneer-to-backup connection. Walls-1 and -2b did not exhibit any veneer cracking after being subjected to more than twice the scaled dynamic input that caused the veneer to crack in Wall-2 (mainly because the veneer was more closely coupled to the backup in those walls). In Wall-2, the large brick mass had more freedom to move independently from the backup, developing high enough inertial forces to bend and crack the veneer, without complete tie failure and collapse, at the upper region of the veneer during the M10-0.30g input run. (For anchored veneer crack control, BIA and MSJC only specify *backup* deflection limits, based primarily on studies with metal stud framing where a relationship between backup deflection and veneer flexural cracking has been established [12-13].)

Collapse of the Brick Veneer

As tie damage progressed in the upper region of Walls-1 and -2, the distribution of inertial forces along the height of the wall placed a greater demand on the remaining ties. Peak measured displacements from the most severe dynamic tests before veneer collapse (M10-0.66g and M10-0.41g for Walls-1 and -2, respectively) are shown in Fig. 10 (which also includes the final test of Wall-2b for comparison). Veneer and backup displacements were similar in shape and magnitude for Walls-1 and -2 (Fig. 10a); the relative displacements were also quite similar for both walls at this higher stage of damage (Fig. 10b). After these tests on Walls-1 and -2, all of the ties along the centerline were still intact; even though during testing, the top center ties in both walls were exceeding or approaching the elongations corresponding to ultimate tensile loading (as found in subassembly tests). Finally in Fig. 10, the measured displacements in Wall-2b at much higher dynamic loading (M10-0.80g) demonstrate that the mechanical anchors were quite effective at transferring a major part of the inertial forces. (Similarly, another study showed that the performance of older (turn of the 20th century) veneer construction could be improved by post-installing anchors [15].) Finally, collapse of the brick veneer walls (Walls-1 and -2) resembled a “zipper” effect –

initial tie damage occurred across the upper rows, immediately placing more demand on the next row of ties below, which also experienced damage, and so on.

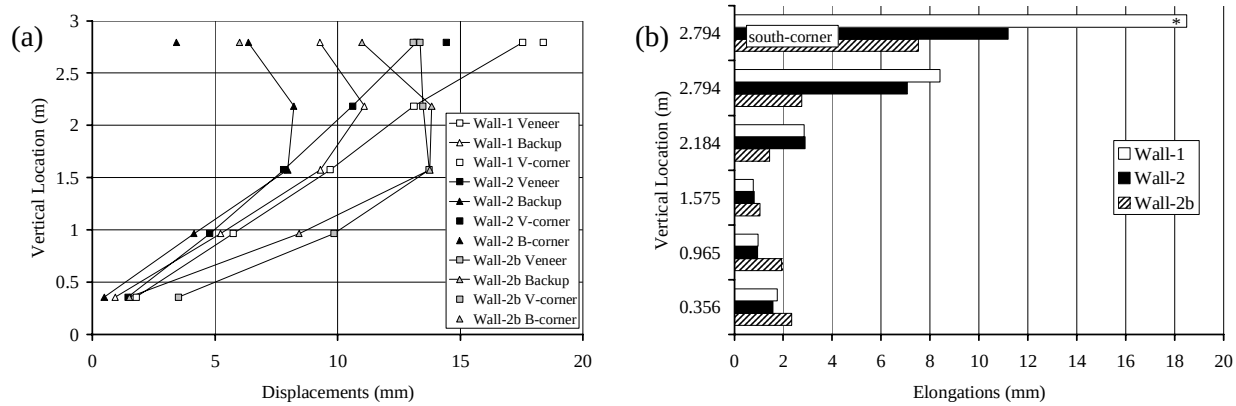

Fig. 10. (a) Peak positive displacements during the highest dynamic runs (prior to collapse of Walls-1 and -2); and (b) peak relative displacements for those same tests (* – net brick displacement).

SUMMARY & CONCLUSIONS

Performance of brick veneer walls on a wood frame backup (typical of residential construction) was investigated under static and dynamic out-of-plane lateral loading on a shake table. The test specimens, representing common construction practice, comprised full-scale brick veneer walls attached to a wood frame backup with 28 ga. corrugated sheet metal ties (utilizing two different installation methods), as well as with post-installed mechanical "retrofit" anchors. With respect to overall wall behavior and the effect of different veneer-to-backup connections, the most important results and conclusions may be summarized as follows:

- Preliminary static tests showed that brick veneer enhanced the out-of-plane stiffness of the wall system, compared to the stiffness of the bare wood frame wall panel. The free vibration period of the veneer walls varied in relation to the initial stiffness of the veneer-to-backup connections; furthermore, changes in period of vibration were a good measure of the progression of damage in the wall system.
- The brick veneer rotated as a rigid body about its base when subjected to dynamic input, producing inertial forces transferred through the ties into the wood frame backup. As a result, the ties in the upper rows controlled veneer wall system performance because they were subjected to the highest displacements (elongations). Ties anchored at or near stiff regions of the wood frame backup (floor or roof/ceiling framing) were more highly loaded than ties anchored near more flexible regions (half-way up the wall panel), where the wood frame backup could deflect together with the veneer; these results were similar to those from other studies for ties at different vertical locations in veneer walls subjected to wind loads.
- "Mortar droppings" in the air space between the veneer and the backup increased the initial stiffness of some ties (by providing constraint) and also reduced the demand on ties in compression by locally filling the air space.
- Overall wall deformations depended on the stiffness of the ties, particularly in tension. With stiffer ties, the veneer was more closely coupled to the backup than in walls with more flexible ties, which allowed the veneer to move somewhat independently from the backup. This made the brick veneer more susceptible to cracking in the case of flexible tie connections.
- Post-installed mechanical anchors were able to improve the performance of the brick veneer wall system, compared to using corrugated sheet metal ties only. The anchors effectively transferred a large portion of the inertial loads, while securing the veneer closely to the backup.

- Initial tie damage always occurred near the top corners of the veneer walls. As tie damage spread, gradually reducing the stiffness and strength of the veneer-to-backup connections, a portion of the veneer became unstable and eventually collapsed. The majority of ties near the top of the walls experienced brittle fracture at collapse, whereas ties near the lower rows of the collapsed veneer region often underwent more ductile tearing damage. Tie failure modes noted in the veneer wall collapses matched well with those observed in a previous study on individual tie behavior.
- For veneer walls subjected to dynamic input, lower inertial loads were produced by the veneer when stiffer ties were used. The veneer wall with a "best-case" installation method of 28 ga. ties (bent at the nail) was able to sustain dynamic input up to the M10-0.58g record without any tie damage, and up through the M10-0.66g record before collapse of the brick veneer. On the other hand, the veneer wall with a worse installation method of 28 ga. ties (bent at an eccentricity from the nail) could only sustain dynamic input up to the M10-0.22g record without tie damage, and up through the M10-0.41g record before collapse of the brick veneer. Finally, the veneer wall with mechanical expansion anchors was able to sustain dynamic input up to and including the M10-0.80g record without any veneer system damage. These results emphasize the importance of both stiffness and strength in the veneer-to-backup connections.

The above findings indicate the following recommendations for brick veneer construction: a) more ties (at closer spacings) should be used at the top of walls and near stiffer regions of the backup, b) ties should be bent as close as possible to their point of attachment to obtain the highest tie and wall stiffness, c) thicker (and therefore stiffer) ties should be used (at least the minimum specified thickness) to ensure adequate strength (especially with respect to low-cycle fatigue) and stiffness (especially in compression for cases where the air space is free of mortar droppings), and d) for older and/or damaged veneer walls (where ties may be corroded or otherwise inadequate), using post-installed anchors can be an inexpensive and effective repair technique.

As has already been described, the wall specimens utilized 28 ga. ties, even though 22 ga. ties are the minimum specified. Subassembly tests have shown that the ultimate tensile strength is similar for both types of ties, while the initial stiffness is slightly higher for the 22 ga. ties (for each installation method); therefore, a veneer wall with 22 ga. ties may perform only slightly better than the walls tested with 28 ga. ties. Finite element (FE) models for brick veneer walls have been developed, based on the experimental observations, and they are being used to investigate specific effects on overall wall behavior, of different: types (thicknesses) of ties, tie installation methods (with or without eccentricity; nail vs. screw attached), and tie spacings.

Many of the experimental results from the shake table tests (and the earlier tie subassembly tests) are relevant to brick veneer construction in general, when subjected to any type of out-of-plane loading. However, additional experimental and analytical research is underway at UIUC to explore the effects of different architectural features on veneer wall performance and to better understand the response of brick veneer to wind loads. Wall specimens including a medium-sized window opening and a gabled region (similar to those found at the "ends" of many wood-frame homes with brick veneer) will be tested, and additional FE studies will be conducted.

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