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In-Plane Loading of Brick Veneer over Wood Shear Walls

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James M. Lintz[1] and Elias A. Toubia[2]

INTRODUCTION

In the design of wood stud walls with brick veneer, current design building codes specify that the wood stud wall should resist all in-plane and out-of-plane loads (IBC 2009). For out-of-plane loads, this assumption is entirely justified as the brittle brick veneer will crack and lose its capacity to resist bending. For in-plane loads, the brick veneer is significantly stiffer than the wood shear wall, and the veneer is unlikely to crack before the wood shear wall reaches its allowable capacity. The assumption that the wood shear wall resists the entire load is based on the further assumption that the ties which connect the stud wall to the veneer will be sufficiently flexible to not transfer significant loads. Research has shown that this is not the case for typical US residential construction practices. The brick veneer can, in fact, resist significant in-plane loads.

Typical wood stud wall construction consists of 2x wood studs with an exterior plywood or OSB sheathing with a waterproofing membrane attached to the exterior of the sheathing. For walls with brick veneer, an air gap, typically 1 inch (25.4mm), is provided between the veneer and the sheathing to allow water that penetrates the veneer to drain. At the bottom of the wall, flashing channels the water out of the wall through weep holes in the veneer (BIA 2002). Attaching the stud wall to the veneer are ties that transfer out-of-plane load from the veneer to the stud wall. Many different types of ties are used in construction; this paper will focus on the corrugated steel type typically used in US residential construction, as seen in Figure 1.

A typical wall section showing standard construction practice is shown in Figure 2.The Masonry Standards Joint Committee (MSJC) *Building Code Requirements and Specification for Masonry Structures* (MSJC 2008) has many prescriptive requirements for anchoring veneer

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in place to help ensure proper construction practices, while limiting the amount of design required. These requirements provide the basis for typical construction practice, and all assumptions and calculations in this paper conform to the listed requirements as shown in Table 1.

Using typical US construction practices on small 2ft x 3ft wall specimens, Johnson and McGinley (2003) showed that the corrugated ties had the potential to transfer a significant amount of in-plane lateral load from a wood shear wall to brick veneer. Subsequent small scale testing on wood-tie-brick sub-assemblies has furthered this finding and tested many factors for determining how much load will be transferred to the veneer. Testing performed by Choi and LaFave (2004) produced load-displacement curves for corrugated ties on these sub-assemblies for both monotonic and cyclic loading. The 22 gauge corrugated ties were shown to have an initial stiffness, which after a small amount of deflection, changed to a much smaller secondary stiffness. This testing showed that nail pullout was the most common failure mode under monotonic loading and that tie fracture was the most common failure mode under cyclic loading. The maximum load for the 22 gauge corrugated ties under cyclic loading was found to be approximately 80% of the maximum under monotonic loading.

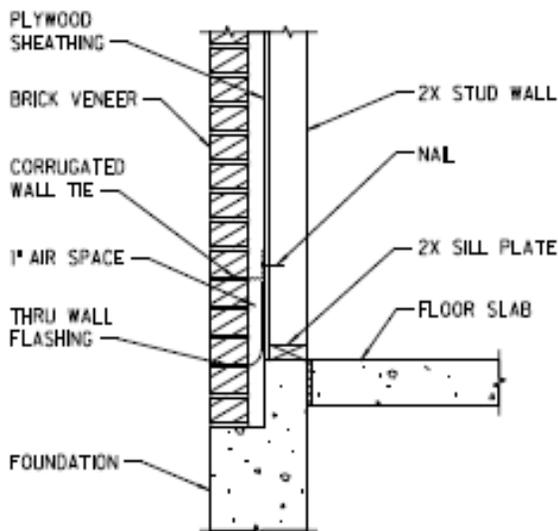**Figure 1 - Typical Corrugated Sheet-Metal Ties**

Figure 2 - Wood Stud Wall with Brick Veneer Detail

Table 1. Corrugated Sheet-Metal Brick Anchor Requirements per 2008 MSJC

Code Section	Category	Requirement
6.2.2.5.1.1	Min. Width	0.875 in. (22.2 mm)
6.2.2.5.1.1	Min. Thickness	0.03 in. (0.76 mm) 22 gauge
6.2.2.5.1.1	Corrugation Wavelength	0.3 - 0.5 in. (7.6 - 12.7 mm)
6.2.2.5.1.1	Corrugation Amplitude	0.06 - 0.10 in. (1.5 - 2.5 mm)
6.2.2.5.1.2	Min. Embedment in Mortar Joint	1.5 in. (38.1 mm)
6.2.2.5.1.2	Min. Cover to Outside Face	0.625 in. (15.9 mm)
6.2.2.5.6.1	Max. Area Per Anchor (SDC A,B,C) (Wind ≤110 mph)	2.67 ft^2 (0.25 m^2)
6.2.2.10.2.2	Max. Area Per Anchor (SDC D,E,F) (Wind ≤110 mph)	2.00 ft^2 (0.19 m^2)
6.2.2.11	Max. Area Per Anchor (Wind >110 mph,≤130 mph)	1.87 ft^2 (0.17 m^2)
6.2.2.5.6.3	Max. Vertical Spacing	25 in. (635 mm)
6.2.2.5.6.3	Max. Horizontal Spacing (Wind ≤110 mph)	32 in. (813 mm)
6.2.2.11	Max. Horizontal & Vertical Spacing (Wind >110 mph, ≤130 mph)	18 in. (457 mm)
6.2.2.6.2	Min. Nail Size	8d Common
6.2.2.6.2	Max. Distance Fastener to Bend	0.5 in. (12.7 mm)
6.2.2.6.3	Max. Distance Sheathing to Veneer	1.0 in. (25.4 mm)

Additional testing performed by Zisi and Bennett (2011) on wood-tie-brick subassemblies, showed 22 gauge corrugated ties to have only about 50% of the initial stiffness and strength values as found by Choi and LaFave (2004). This is likely due to the lack of out-of-plane restraint of the brick in the testing conducted by Zisi and Bennett (2011). Their testing found that nail pullout was the dominant failure mode under cyclic loading. They also produced hysteresis curves which showed pinching due to the damage of the wood fibers around the fastener. Their findings also showed that a change in the distance from the bend to the anchor produced large effects on the stiffness of the ties. Decreasing this distance led to a larger initial stiffness, more dissipated energy, and higher cyclic envelopes. Ideal load-displacement curves for 22 gauge corrugated ties under cyclic loading, approximated from the sub-assembly testing done by Choi and LaFave (2004) and Zisi and Bennett (2011), are shown in Figure 3.

Large scale experimental testing performed by JFA Moore (1978) comparing wood shear walls with and without brick veneer, showed that for walls with brick veneer, the capacity is approximately 70% of the in-plane load when the wood was loaded, and approximately 90% of the in-plane load when the brick was loaded. His tests also showed a significant increase in the strength and stiffness of the walls with veneer, which can be over 4 times the stiffness compared to the wood shear wall without brick veneer.

More recent testing done by Thurston and Beattie (2008), using construction techniques and standards conforming to the New Zealand code of practice, showed that for an isolated wall panel with masonry veneer, the veneer wall would continue to resist load until it would slide along the joint between the brick mortar and the concrete foundation. They found that using a coefficient of friction of 0.63, provided good agreement with their testing for when the veneer would slide. Current US practice dictates using flashing at the bottom of a veneer wall. The load at which the veneer will slide will therefore depend on the coefficient of friction between the flashing used (metal, PCV, EPDM, etc.) and the veneer/mortar. Their testing also showed that for walls with closed corners (no joint), the movement of the veneer wall was caused entirely by the rocking of the wall and not sliding, presumably due to the extra weight of the veneer from around the corner. In all of their testing presented, no sliding occurred along the horizontal cracks between brick rows. This was due to the mortar droppings in the holes in the brick forming dowels which greatly increased the shear strength of the veneer wall.

Full scale shake table testing done by Okail et al. (2011) with a building constructed according to US building codes showed similar results to the testing done by Thurston and Beattie (2008). The movement of wall segments with closed corners and a large height to length ratio was caused almost entirely by rocking instead of

sliding, while for other segments of the wall, deflection was mainly due to sliding. Also shown was that the rocking motion of the veneer created additional seismic load on the wood structure at high excitation levels. Their testing also showed only a 1% maximum drift at a peak ground acceleration of 2g which is partly attributed to the restraint provided by the veneer. This led them to the conclusion that "in-plane masonry veneer should not simply be treated as added mass," but that, "its contribution to in-plane resistance and energy dissipation should be recognized" (Okail et al. 2011).

To date, no simple analytical method to predict the amount of load transferred through the ties to the brick veneer has been presented. This study provides an analytical approach and design equations to quantify the contribution of rows of ties on the in-plane load performance of a wood shear wall attached to a brick wythe.

TECHNICAL APPROACH

In order to determine the load that will be transferred from the wood shear wall through the brick ties to the brick veneer, the assembly is modeled as a shear wall supported by linear springs. To better understand the concept, consider first a simplified example of a cantilevered beam with a spring as shown in Figure 4. The beam will be assumed to deflect under bending only (no shear deflection). If the spring is removed, the load at the top of the cantilevered beam will cause a deflection related to the stiffness of the beam. If the spring is

included, the actual deflection will be less due to the resisting force of the spring on the beam. By superposition, the total deflection of the beam/spring will equal the deflection of the beam caused by the load P minus the deflection of the beam caused by the force F in the spring as shown in Equation 1. Using established equations for calculating the deflection of a cantilevered beam and the deflection of a spring, Equation 1 becomes Equation 2. The resulting equation can then be solved for F giving the force in the spring, and once the force is known, the deflection can be easily calculated.

$$\Delta_P - \Delta_F = \Delta_T \qquad (1)$$

Where:
Δ_P is the deflection of the beam at x due to the load P
Δ_F is the deflection of the beam at x due to the force
    F in the spring
Δ_T is the deflection of the spring

$$\frac{P}{6EI}\left[2H^3 - 3H^2(H-x) + (H-x)^3\right] - \frac{Fx^3}{3EI} = \frac{F}{k} \qquad (2)$$

Where:
H is the length of the beam
E is the modulus of elasticity of the beam
I is the moment of inertia of the beam
x is the distance to the spring
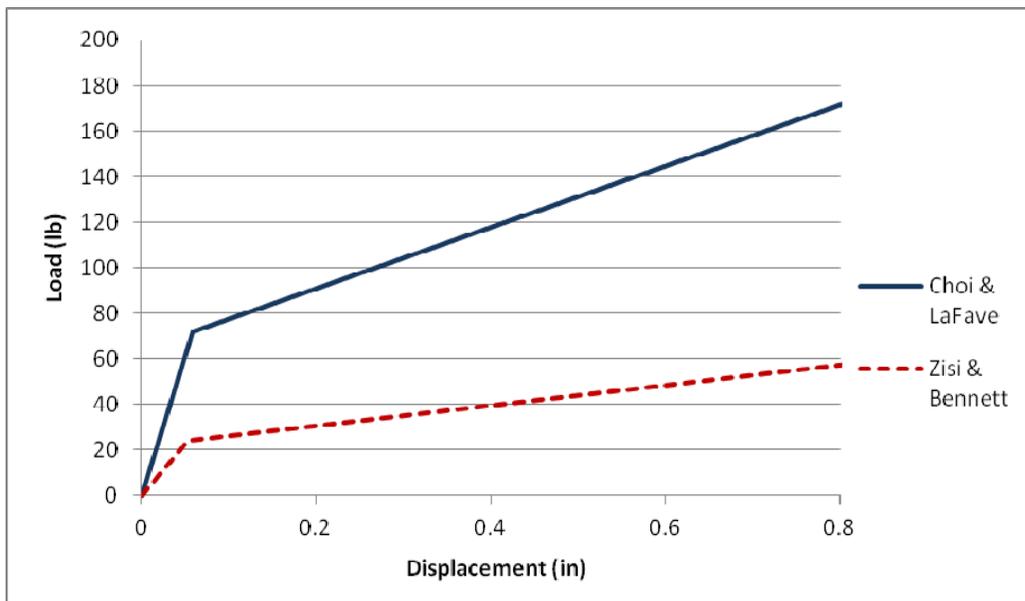k is the stiffness of the spring

Figure 3 - Approximate Load-Displacement Curves for Single Straight 22 Gauge Brick Ties
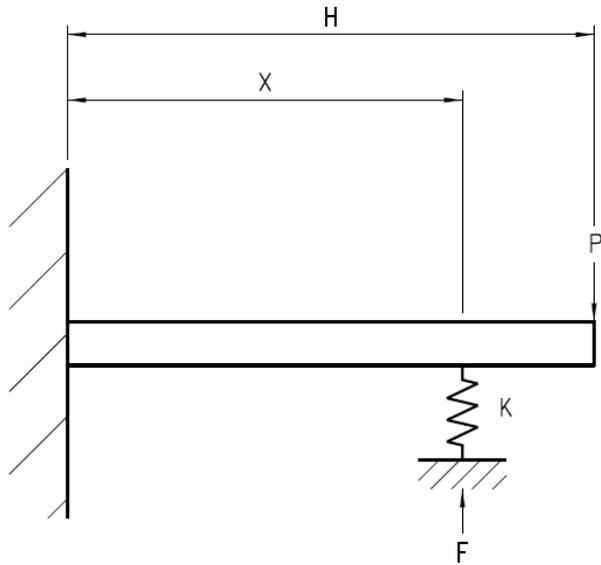
Figure 4 - Cantilever Beam with Spring

This method, if extrapolated, can model the in-plane deflection of a wood shear wall connected with n-number of ties per row to the brick veneer; however, several factors should be included for an accurate model to be produced.

The 2008 NDS Wind and Seismic code (AF&PA 2009) provides an equation for the deflection of a wood shear wall as shown below in Equation 3. The total shear wall deflection is made up of three terms, the first being the deflection due to bending, the second term the deflection due to shear and nail slip, and the last term accounts for deflection due to wall anchorage slip.

$$\delta_{sw} = \frac{8vH^3}{EAl} + \frac{vH}{1000G_a} + \frac{H\Delta_a}{L} \tag{3}$$

A few minor modifications to Equation 3 allow Equation 4 to be easily derived. For Equation 4, the total load P is used in place of the unit shear v. The units of the apparent shear wall stiffness, G_a, are changed to lb/in from kips/in. The ratio of the elongation of the anchor to its maximum allowable elongation is assumed directly proportional to the ratio of the tensile load in the anchor to the maximum allowable tensile load on the anchor ($T/T_{allow} = \Delta_a/\Delta_{allow}$), and zero compressive force is assumed on the wall. The shear wall will always have some compressive force due to the self weight of the wall; however, this is ignored for simplicity. Relatively, the anchorage slip will typically have little effect on the overall deflection of the wall.

$$\delta_{sw} = \frac{2PH^3}{3EAl^2} + \frac{PH}{G_aL} + \frac{PH^2\Delta_a}{L^2 T_{allow}} \tag{4}$$

Most wood shear walls with brick veneer will have multiple rows of ties. This creates complexity in which the force in multiple springs needs to be determined as shown in Figure 5. To solve this, a set of simultaneous linear equations will be required. The load in each spring can be determined in a similar way to the first model. The total deflection of the wall at h_1 will equal the deflection caused by the load P at h_1 minus the deflection caused by the force in spring 1 at h_1 minus the deflection caused by the force in spring 2 at h_1. Similarly, this analogy is applied at each level of ties. This can be seen in Equation 5 and Equation 6.

$$\Delta_{Ph_1} - \Delta_{F_1h_1} - \Delta_{F_2h_1} = \Delta_{Th_1} \tag{5}$$

$$\Delta_{Ph_2} - \Delta_{F_1h_2} - \Delta_{F_2h_2} = \Delta_{Th_2} \tag{6}$$

Where Δ_{Ph1} is the deflection of the shear wall at h_1 due to load P, Δ_{F1h1} is the deflection of the shear wall at h_1 due to the force in the spring at h_1, Δ_{F2h1} is the deflection of the shear wall at h_1 due to the force in the spring at h_2, Δ_{Th1} is the deflection of the spring at h_1, Δ_{Ph2} is the deflection of the shear wall at h_2 due to load P, Δ_{F1h2} is the deflection of the shear wall at h_2 due to the force in the spring at h_1, Δ_{F2h2} is the deflection of the shear wall at h_2 due to the force in the spring at h_2, and Δ_{Th2} is the deflection of the spring at h_2.
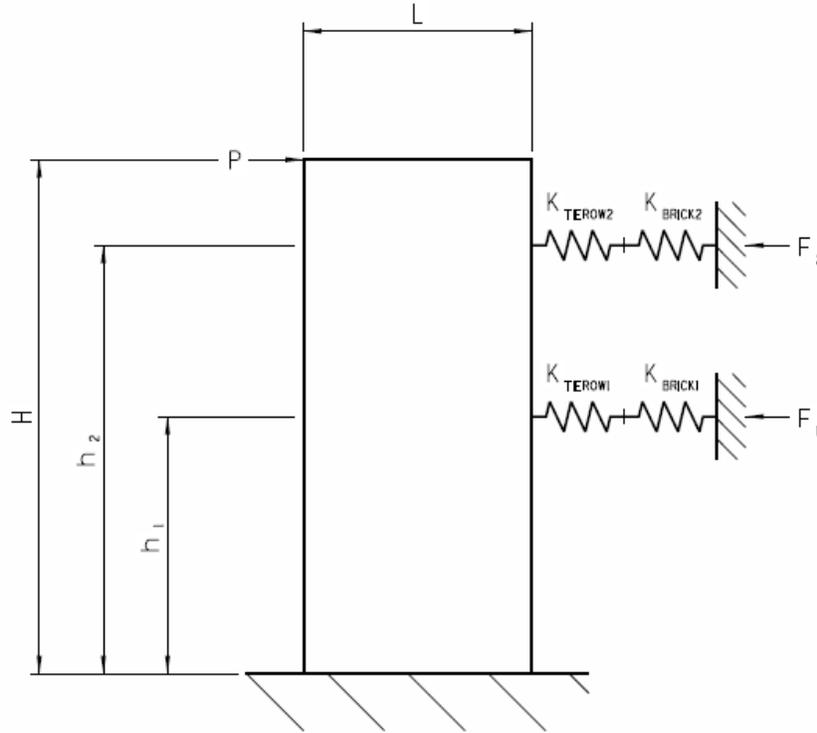
Figure 5 - Shear Wall with Two Rows of Two Springs in Series

While the brick veneer is significantly stiffer than the wood shear wall under in-plane loading, the brick veneer will also deflect. Under in-plane load, the wood shear wall deflection will cause the brick ties to deflect relative to their stiffness, causing a force on the brick veneer which will also deflect. To account for this factor, the brick veneer is modeled as a spring in series with each tie row as shown in Figure 5.

The total stiffness at each tie row is given by the following:

$$\frac{1}{k_{eff}} = \frac{1}{k_{tierow}} + \frac{1}{k_{brick}} \tag{7}$$

The stiffness of a single tie can be determined from brick tie testing as was done by Choi and LaFave (2004) and Zisi and Bennett (2011). The total stiffness per row of ties (assuming n-number of ties) can then be calculated as springs in parallel as

$$k_{tierow} = \sum_{i=1}^{n} k_{tie} = nk_{tie} \tag{8}$$

The deflection of a row of ties can then be found simply by using the equation

$$\Delta_{tierow} = \frac{F_{tierow}}{k_{tierow}} \tag{9}$$

The in-plane deflection of a plain cantilever masonry shear wall can be calculated as shown in Equation 10. The first term is the deflection due to bending while the second term is the deflection due to shear.

$$\Delta_{masonry} = \frac{Ph^3}{3E_m I_m} + \frac{Ph}{A_v G_m} \tag{10}$$

For clay brick $A_v=(5/6)A_g$ where A_g is the gross cross sectional area of the masonry and $G_m=(2/5)E_m$.

For a relatively simple problem with only two rows of ties as shown in Figure 5, plugging in equations 4, 9, and 10 into equations 5 and 6, yields the following results:

$$(F_1)\left[\left(\frac{h_1^3}{3EI}\right)+\left(\frac{h_1}{G_aL}\right)+\left(\frac{h_1\Delta_{anchor}}{LT_{allow}}\right)\left(\frac{h_1}{L}\right)+\left(\frac{1}{k_{tierow}}\right)+\left(\frac{h_1^3}{3E_mI_m}\right)+\left(\frac{h_1}{(5/6)tLG_m}\right)\right]+$$

$$(F_2)\left[\left(\frac{h_1^2}{6EI}\right)[3h_2-H-(H-h_1)]+\left(\frac{h_2}{G_aL}\right)\left(\frac{h_1}{h_2}\right)+\left(\frac{h_2\Delta_{anchor}}{LT_{allow}}\right)\left(\frac{h_2}{L}\right)\left(\frac{h_1}{h_2}\right)\right]$$

$$=\left(\frac{P}{6EI}\right)\left[2H^3-3H^2(H-h_1)+(H-h_1)^3\right]+\left(\frac{PH}{G_aL}\right)\left(\frac{h_1}{H}\right)+\left(\frac{Ph_1\Delta_{anchor}}{LT_{allow}}\right)\left(\frac{H}{L}\right)\left(\frac{h_1}{H}\right)$$

$$(F_2)\left[\left(\frac{h_2^3}{3EI}\right)+\left(\frac{h_2}{G_aL}\right)+\left(\frac{h_2\Delta_{anchor}}{LT_{allow}}\right)\left(\frac{h_2}{L}\right)+\left(\frac{1}{k_{tierow}}\right)+\left(\frac{h_2^3}{3E_mI_m}\right)+\left(\frac{h_2}{(5/6)tLG_m}\right)\right]+$$

$$(F_1)\left[\left(\frac{(H-h_1)^2}{6EI}\right)[3H-3(H-h_2)-h_1]+\left(\frac{h_1}{G_aL}\right)+\left(\frac{h_1\Delta_{anchor}}{LT_{allow}}\right)\left(\frac{h_1}{L}\right)\left(\frac{h_2}{h_1}\right)\right]$$

$$=\left(\frac{P}{6EI}\right)\left[2H^3-3H^2(H-h_2)+(H-h_2)^3\right]+\left(\frac{PH}{G_aL}\right)\left(\frac{h_2}{H}\right)+\left(\frac{Ph_2\Delta_{anchor}}{LT_{allow}}\right)\left(\frac{H}{L}\right)\left(\frac{h_2}{H}\right)$$

In order to solve for the force in each tie row of a wall with multiple rows of ties, an Excel spreadsheet was set up.

The above procedure gives the force in each row of springs, but it assumes a completely linear load-deflection relationship for each tie; however, testing of wood-tie-brick assemblies has shown a deviation from linear behavior. Testing by Choi and LaFave (2004) showed that the assemblies have a two stage load-deflection curve as shown in Figure 3. The initial stage shows a linear load-deflection relationship with much greater stiffness than the linear second stage. In order to account for this load-deflection relationship in the proposed equations, the total load on the shear wall can be increased incrementally. This allows the stiffness of each tie row to be changed from the initial stiffness to the secondary stiffness once the deflection of the tie row reaches a specified level. The deflection and load in each row of ties is calculated after each load step and added to the deflection and load of the tie row from the previous steps. An" if-then" statement in the equations of the $1/k$ matrix can be used to change from the initial to the secondary stiffness. The general equations to solve for the deflection and force at a tie row for a wall with multiple rows of ties are given by

$$[1/k][F]=[\Delta] \tag{11}$$

$$[F]=[1/k]^{-1}[\Delta] \tag{12}$$

Where:
$[1/k]$ is the compliance matrix
$[F]$ is the force matrix
$[\Delta]$ is the deflection matrix

Coefficient of Static Friction Testing

The approach described above allows for the calculation of the load transferred from the wood shear wall to the brick veneer at each tie row. When the total load transferred to the brick veneer is sufficient to overcome the force of friction between the wall and the thru -wall flashing, the veneer will slide. Simple testing was performed to determine the approximate value of the coefficient of friction between different flashing materials and the brick and mortar of the wall.

Table 2 shows the results for the testing done on three types of flashing. The results show hardened mortar and brick to have static coefficient of friction values similar to one another for each flashing type, but a distinct range can be seen for the static coefficient of friction values between each flashing type.

Table 2. Experimental Values of Coefficient of Friction Values

	Brick	Mortar
Galvanized Steel	.38 - .40	.32 - .39
Plastic	.46 - .53	.46 - .51
Rubber	.69 - .71	.61 - .73

RESULTS

Consider for example an 8 ft. x 8 ft. (2.438 m x 2.438 m) wood shear wall connected to the brick veneer with 22 gauge corrugated ties using typical US construction practices. Using the proposed analytical approach and solving for the load in each tie row based on the ideal load-deflection curves from either Choi and LaFave (2004) or Zisi and Bennett (2011) shown in Figure 3, one can notice that the higher up the tie row, the greater the load transfer from the wood shear wall to the brick veneer (Figure 6). This is due to the increasing difference in stiffness between the wood shear wall and the brick veneer at increasing heights. As the load is increased on the shear wall, the highest row of ties related to the story level considered will deflect to the point that its initial stiffness transfers over to its lesser secondary stiffness caused by the twisting of the ties. When the load is increased further, the second highest row of ties will transfer to its secondary stiffness and so on down the wall. As can be seen in Figure 6, the transition of the tie rows to their secondary stiffness as the load on the wall increases causes the load in successive tie rows to increase more linearly as compared to the exponential increase at relatively small loads.
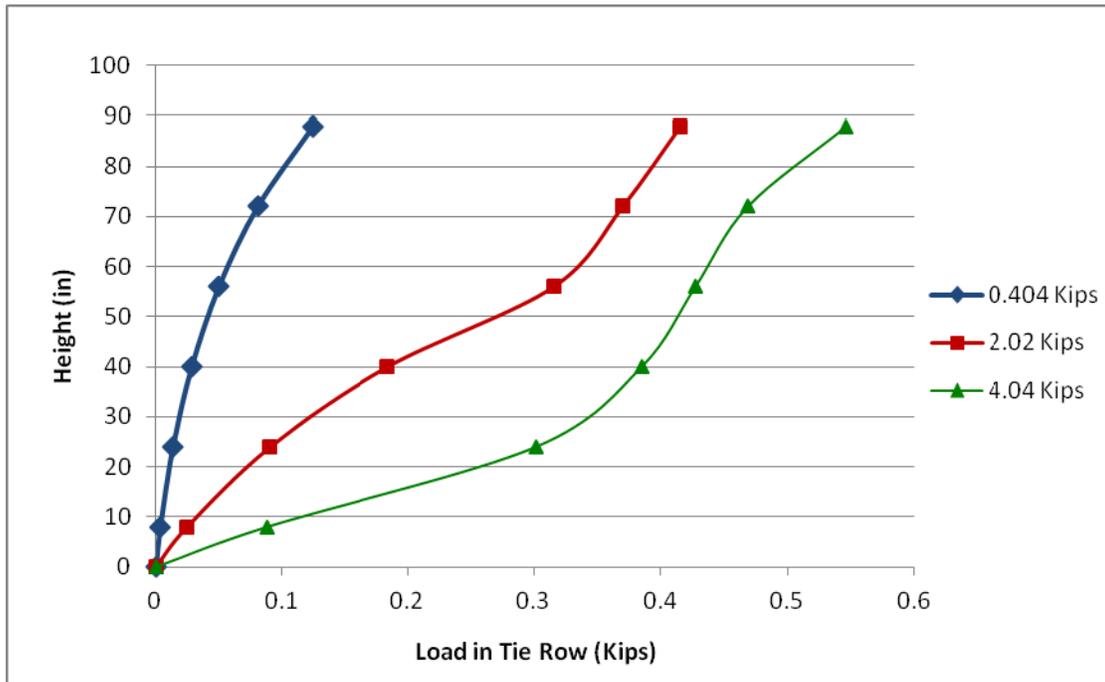

Figure 6 - **Predicted Load Transferred to Tie Rows at 20%, 100%, and 200% Allowable Shear Wall Capacity(ASD) (Note: 1 Kip = 4.448 KN, 1 in = 25.4 mm)**
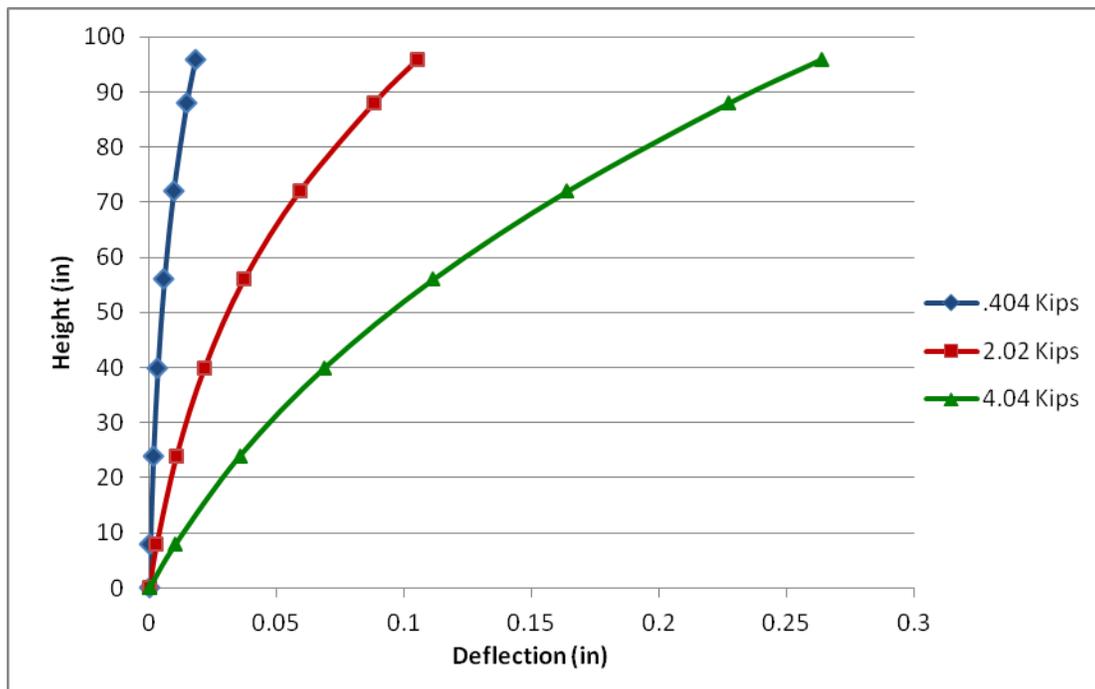
Figure 7 - **Predicted Deflected Shape of Wall at 20%, 100%, and 200% Allowable Shear Wall Capacity (ASD) (Note: 1 Kip = 4.448 KN, 1 in = 25.4 mm)**

Figure 7 shows the deflected shape of the wood wall calculated at the same loading stages as were used in Figure 6 for an 8 ft. x 8 ft. (2.438 m x 2.438 m) wall. The loading stages of .404 kips (1.80 kN), 2.02 kips (8.99 kN), and 4.04 kips (17.97 kN) correspond to 20 percent, 100 percent, and 200 percent respectively of the allowable shear load which could be applied to the wood shear wall alone based on the 2008 NDS wind and seismic code provisions.

When the calculated maximum in-plane deflection in a wood shear wall with brick veneer is compared to the calculated maximum in-plane deflection of the wood shear wall alone, one can notice that the veneer adds stiffness to the wall (See Figure 8). This agrees well with the results of sub-scale testing as described in the introduction section. Previous research has shown large variation in the stiffness of straight corrugated ties. Much of this difference is believed to be due to the different testing methods used by different researchers. Choi and LaFave (2004) had out of plane restraint whereas Zisi and Bennett (2011) did not. Zisi and Bennett (2011) concluded that their results are likely more indicative of actual wall behavior due to the lack of out of plane restraint for the veneer at the top of the wall. A significant difference can be seen in Figure 8 between the predicted results based on the ideal load-deflection curve found by Choi and LaFave (2004) compared to that found by Zisi

and Bennett (2011). The greater stiffness of the ties as shown by Choi and LaFave (2004) translates into a significant increase in predicted overall wall stiffness. Load-deflection curves for full scale wood shear walls with brick veneer constructed with typical U.S. construction practices could not be found in the literature. Full scale testing of this sort would allow a comparison between the accuracy at full scale of the Choi and LaFave (2004) data and the Zisi and Bennett (2011) data. It would also provide a validation to the accuracy of the analytical method used in this study. The authors believe that the behavior of actual walls would fall somewhere between the two testing results. While there is no out of plane restraint for the veneer at the top of an actual wall the tie rows themselves as well as the stiffness of the brick veneer will provide some out of plane restraint, although likely not as much as was provided in the Choi and LaFave (2004) testing. Further research could provide more clarity on this subject.

By determining the load in each tie row as described in the previous section, the load required at the top of the wood shear wall for the brick veneer to reach various failure modes can be calculated. The results of these calculations for six different wall types can be seen in Table 3 below. Figures 9 and 10 provide a simple visual representation of the results shown in Table 3.
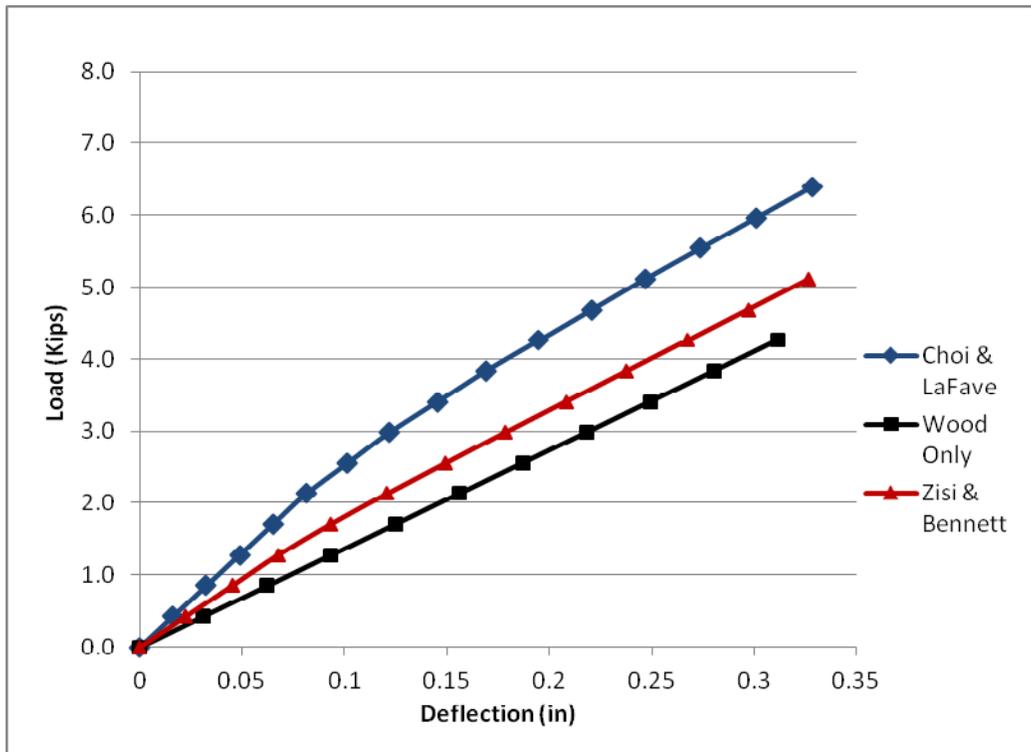
Figure 8 - Predicted Load-Displacement Curves for Top of 8 ft. x 8 ft. (2.438 m x 2.438 m) Ga=19 Wood Shear Wall (Note: 1 Kip = 4.448 KN, 1 in = 25.4 mm)

Table 3. Load to Failure Values for Different Wall Types and Failure Modes (1 Kip = 4.448KN)

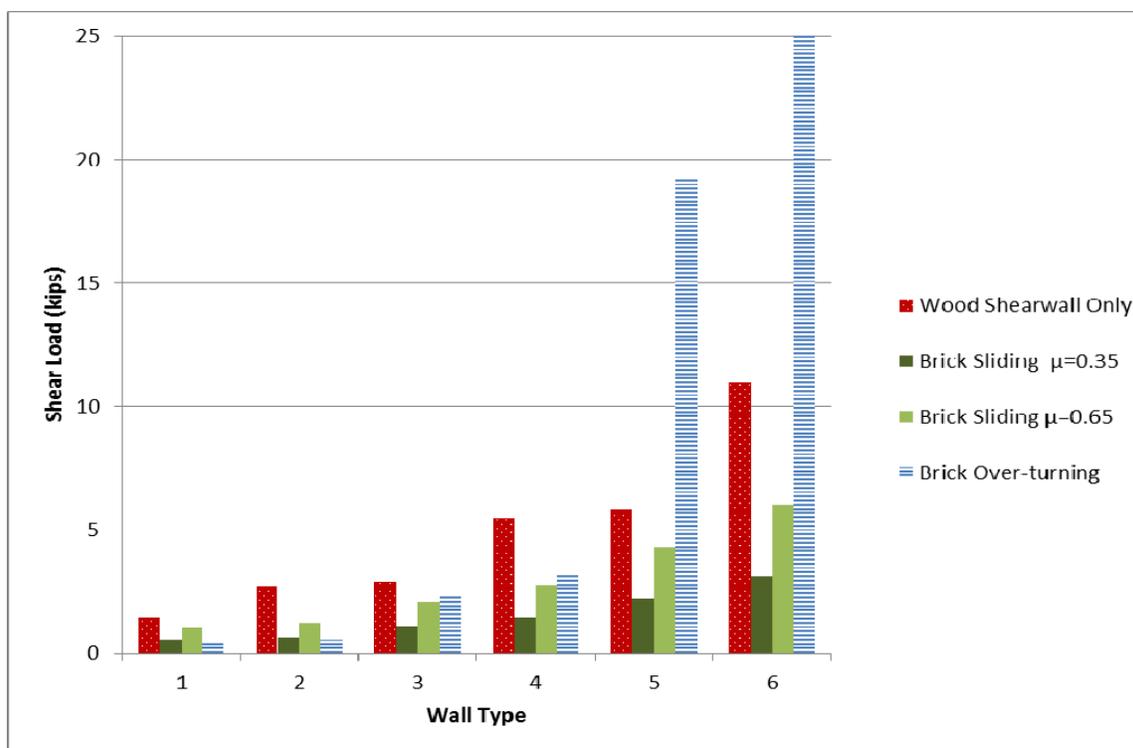|		Failure Mode							
	Wall Type	Wood Shearwall Only (Kips)	Brick Sliding μ=0.35 (Kips)	Brick Sliding μ=0.5 (Kips)	Brick Sliding μ=0.65 (Kips)	Brick Over-turning (Kips)	Brick Shear (Kips)	Ties (Kips)	Deflection (Kips)
Choi & LaFave	1. 4x8,8d@6",15/32" PLY	1.460	0.573	0.769	1.014	0.495	17.64	4.971	2.522
	2. 4x8,8d@3",15/32" OSB	2.740	0.651	0.930	1.246	0.604	28.97	7.571	3.850
	3. 8x8,8d@6",15/32" PLY	2.920	1.098	1.576	2.088	2.384	43.72	10.79	4.769
	4. 8x8,8d@3",15/32" OSB	5.480	1.440	2.058	2.773	3.242	77.78	18.06	7.290
	5. 16x8,8d@6",15/32" PLY	5.840	2.247	3.227	4.315	19.27	97.72	21.50	9.157
	6. 16x8,8d@3",15/32" OSB	10.96	3.126	4.465	6.04	35.05	192.8	42.67	14.21
Zisi & Bennett	1. 4x8,8d@6",15/32" PLY	1.460	0.844	1.652	2.888	1.064	49.00	4.637	1.842
	2. 4x8,8d@3",15/32" OSB	2.740	1.315	2.748	4.961	1.663	87.51	7.689	3.124
	3. 8x8,8d@6",15/32" PLY	2.920	2.069	4.674	7.787	12.17	124.4	10.38	3.573
	4. 8x8,8d@3",15/32" OSB	5.480	3.625	8.719	15.31	23.10	252.0	19.69	6.108
	5. 16x8,8d@6",15/32" PLY	5.840	4.868	11.14	18.57	70.05	286.5	22.21	7.022
	6. 16x8,8d@3",15/32" OSB	10.96	9.39	23.14	38.80	151.6	633.8	46.40	12.08

Figure 9 - **Load to Failure Mode Chart Based on Choi & LaFave (2004) Tie Stiffness (Note: 1 Kip = 4.448 KN)**
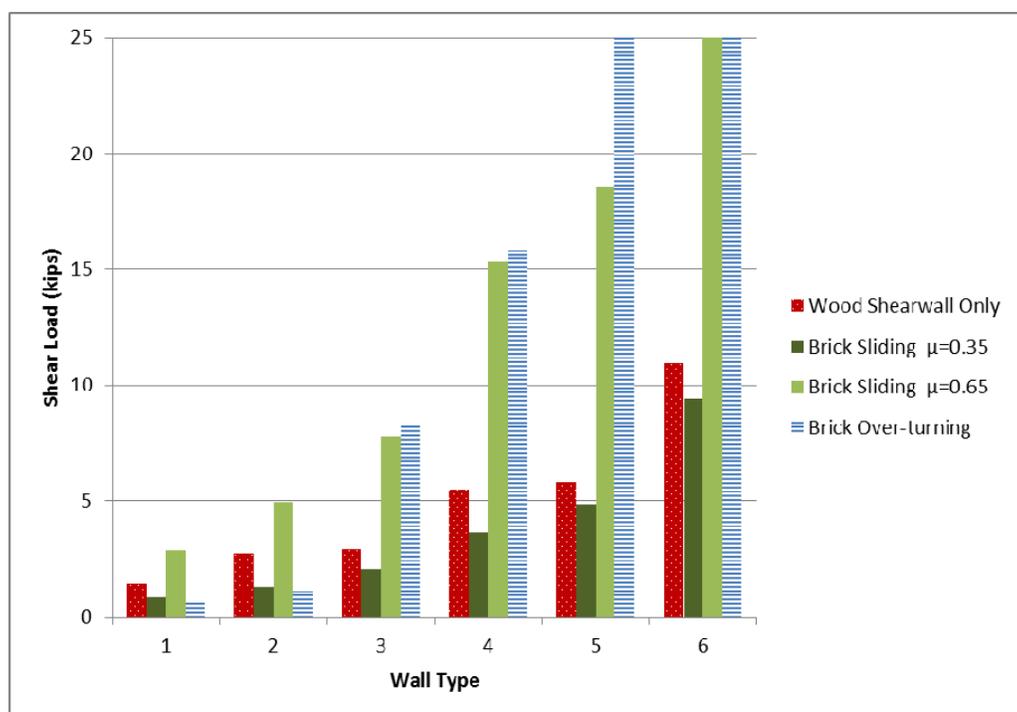

Figure 10 - **Load to Failure Mode Chart Based on Zisi and Bennett (2011) Tie Stiffness (Note: 1 Kip = 4.448 KN)**

Wood shear wall only failure was calculated to occur based on the 2008 SDPWS table for wind loads on a shear wall (ASD). Brick shear failure values are calculated based on unreinforced masonry shear design (ASD) and are greater than any other predicted failure mode. Tie failure is assumed to occur for the Choi and LaFave (2004) tie stiffness based on their reported average maximum cyclic failure load for an individual tie of 179 lb (796 N). Furthermore, Zisi and Bennett (2011) did not report a maximum cyclic failure load for an individual tie, but instead considered failure to occur at 1 in. (25.4 mm) tie deflection Zisi (2009), which was also considered as failure in this report. Interestingly, tie failure is predicted to occur at roughly the same loading based on either tie stiffness value used even though other failure modes differ greatly. The deflection failure mode in Table 3 is not necessarily a failure mode. The load shown in the table (Deflection column) is the load required to reach the same level of deflection at the top of the wall as the wood shear wall alone will have at its calculated failure load. This would only be considered failure if the design of the wall was being controlled by deflection.

The failure modes are also compared based on the tie stiffness values determined by Choi and LaFave (2004) and Zisi and Bennett (2011). The greater tie stiffness found by Choi and LaFave (2004) causes a significant amount of load transferred to the veneer. As such, the veneer failure modes are typically reached at loads that are less than half the load required to failure based on the stiffness calculated by Zisi and Bennett (2011) (Table 3-Brick Shear column). For either tie stiffness at low coefficient of friction values between the wall and the flashing, the prediction is that veneer sliding will occur before the wood shear wall alone will reach its maximum allowable shear capacity. Based on the Choi and LaFave (2004) tie stiffness, when higher coefficient of friction values are used, the predicted sliding failure mode will still occur before the wood shear wall alone reaches its shear capacity; however, based on the Zisi and Bennett (2011) tie stiffness, the wood shear wall alone would fail first. This result gives credence to the tie stiffness results of Zisi and Bennett (2011) since full scale testing has shown an increase in strength and stiffness of wood shear walls with brick veneer over wood shear walls without brick veneer.

RECOMMENDATIONS

As shown in the Table 3 and previous full scale experimental testing discussed in the introduction section, veneer sliding or overturning are expected failure modes for most wood shear walls with brick veneer that use standard corrugated ties. This can occur at levels much closer to or below the plain wood shear wall failure values than masonry or tie failure. A possible construction method to prevent both failure modes would be to embed vertical reinforcing bars across the flashing joint. Placing vertical reinforcement at the ends of veneer segments as shown in Figure 11 would provide both shear and uplift resistance and increase the load to failure on the wall. The holes in the clay brick vary in size and pattern with the openings in some bricks being too small to allow for the reinforcing and mortar needed for bonding. This could possibly be solved by knocking out holes in the center of the brick or using brick with a standard hole size large enough for the reinforcing bars to develop. The additional labor and material cost required to add reinforcing bars could be offset by a reduction in the thickness of the sheathing and the number of nails required in the shear wall.
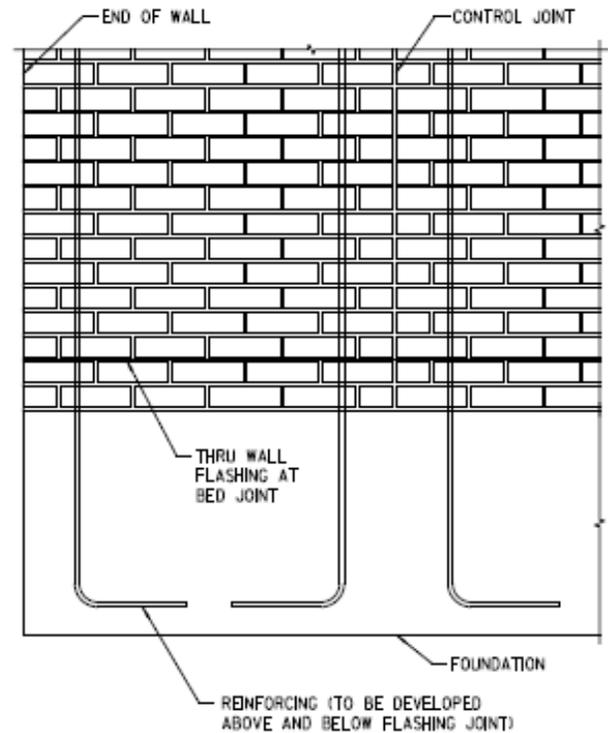

Figure 11 - Veneer Reinforcing Developed on Each Side of the Flashing

CONCLUSIONS

Calculating the in-plane load transferred from a wood shear wall to the brick veneer attached with wall ties can be performed by modeling each row of brick ties and the veneer as springs resisting the lateral load on the wood wall. This method allows for calculations by hand or spreadsheet as opposed to finite element analysis. Whereas, finite element modeling of such structural systems would likely be time and cost prohibitive in typical engineering practice, the method presented in this study can readily produce an answer with a few simple inputs in a spreadsheet. Each input required to solve the equations shown previously can be found in standard codes or chosen by the engineer except for the stiffness of the wall ties. The stiffness of the corrugated wall ties used in standard construction practice has been researched by two groups whose results showed some difference in the stiffness of the ties. Further research is needed to determine the apparent stiffness of a tie in a full scale wall for this method of calculation to better predict the results of the in-plane loading of a wood shear wall with brick veneer.

Solving for typical wood shear walls with brick veneer using the method specified in this paper shows that the load transferred to the brick veneer will stiffen the wood shear wall but has the potential to cause sliding and rocking of the veneer. This is of particular concern for shear walls with large height to width ratios where the rocking of the veneer can be a driving force on the wall behind in strong seismic events. The development of vertical steel reinforcement on each side of the thru-wall flashing could prevent sliding and rocking of the veneer and allow for an increase of the design strength and stiffness of the wall.

ACKNOWLEDGMENTS

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NOTATION

A = Area of end post cross section

A_v = Cross sectional area of masonry veneer available for shear

A_g = Gross cross sectional area of masonry veneer

E = Modulus of elasticity of end post

E_m = Modulus of elasticity of masonry veneer

F = Force in spring

F_n = Force in tie row n

F_{tierow} = Force in one row of ties

G_a = Apparent shear wall stiffness from nail slip and panel shear deformation

G_m = Modulus of rigidity (shear modulus) of masonry veneer

H = Total height of wall

h_n = Height to tie row n from base

I_m = Moment of inertia of masonry veneer

k = Stiffness

k_{brick} = Stiffness of brick veneer

k_{eff} = Net effective stiffness

k_{tie} = Stiffness of individual brick tie

k_{tierow} = Total stiffness of all brick ties in a single row

L = Length of wall

n = Number of ties in a single row

P = Total load on shear wall

T = Tension in wall anchorage system

T_{allow} = Allowable tensile load in wall anchorage system

t = Thickness of masonry veneer

x = Distance along beam to spring

Δ = Deflection

Δ_a = Total vertical elongation of wall anchorage system

Δ_{allow} = Total vertical elongation of wall anchorage system at its maximum allowable tensile load

Δ_F = Deflection of beam at x due to the force F in the spring

Δ_{F1h1} = Deflection of shear wall at h_1 due to the force in the spring at h_1

Δ_{F1h2} = Deflection of shear wall at h_2 due to the force in the spring at h_1

Δ_{F2h1} = Deflection of shear wall at h_1 due to the force in the spring at h_2

Δ_{F2h2} = Deflection of shear wall at h_2 due to the force in the spring at h_2

$\Delta_{masonry}$ = Deflection of masonry veneer

Δ_P = Deflection of beam at x due to the load P

Δ_{Ph1} = Deflection of shear wall at h_1 due to load P

Δ_{Ph2} = Deflection of shear wall at h_2 due to load P

Δ_T = Deflection of spring

Δ_{Th1} = Deflection of spring at h_1

Δ_{Th2} = Deflection of spring at h_2

Δ_{tierow} = Deflection of a single row of ties

δ_{sw} = Deflection of shear wall

μ = Coefficient of static friction

v = Induced unit shear