Seismic Design of Anchored Brick Veneer

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SUMMARY:
Recent earthquakes in the U.S. and New Zealand have caused damage to residential brick veneer walls that is disproportionately more severe than damage to the supporting wood frame homes. Collapsed walls typically revealed poor anchorage construction practices. This paper investigates current design and construction requirements in order to understand whether the brick veneer wall damage is due to inadequate construction practices or deficient code requirements, or both. Prescriptive design and construction requirements and an alternative strength design method for anchored brick veneer, as set forth in the Masonry Standards Joint Committee (MSJC) Building Code, are analysed via a performance based design approach employing seismic fragility curves with performance limits and safety objectives in accordance with the ASCE/SEI 41-06 Standard. Ultimately, prescriptive requirements for anchored brick veneer should be followed as a minimum; however, seismic performance of brick veneer walls will only be improved by proper installation of tie connections, or by retrofitting existing walls with post-installed anchors.

Keywords: Brick masonry veneer, Wood framing, Metal tie connections

1. INTRODUCTION AND BACKGROUND

Wood frame structures with anchored brick masonry veneer are a common type of residential construction throughout North America, Australasia, and other regions of the world. This type of construction typically comprises an interior wood frame backup structure and an exterior masonry wall (separated by an air cavity), with regularly spaced corrugated sheet metal ties used to connect the brick masonry to the backup. During design and construction of residential brick veneer walls, a number of performance requirements must be considered. The masonry veneer should be able to carry its own weight and to transfer out-of-plane loads (due to earthquakes and wind) through the tie connections across the wall cavity to the wood frame backup and then eventually into the foundation. Therefore, the wood frame backup walls need to be designed to resist all of the exterior lateral loading, as well as any gravity loads from the home structure floor or roof framing (Drysdale et al. 1999). In reality, however, such brick veneer walls often carry some of the lateral loads, due to the relatively higher stiffness of the masonry than that of typical wood frame backup (BIA 2002). For adequate performance of brick veneer wall systems, the design and construction details should also account for possible differential movement between the masonry and backup walls, as well as for water penetration of the exterior masonry wall (Drysdale et al. 1999). Prescriptive requirements for strength and serviceability design of brick veneer built over wood frame home structures are provided by the Masonry Standards Joint Committee (MSJC) Building Code (MSJC 2011), the International Residential Code (IRC) for One- and Two-Family Dwellings (ICC 2009), and the Brick Industry Association (BIA) Technical Note 28 (BIA 2002). In addition to the prescriptive requirements, MSJC (2011) allows for an alternative strength design method involving load and deflection analysis of brick veneer walls.

Earthquake damage of brick veneer walls has mainly been attributed to their vulnerability to out-of-plane loading, as the brick veneer moves away from the wood backup, placing a high demand on the tensile and displacement capacity of the tie connections. In such cases, tie connections typically exhibit one of three types of failure: tie fracture, tie pullout from the mortar joint, or tie fastener (nail) pullout from the wood backup. As discussed in more detail in the next section of this paper, veneer wall damage has often been explained by improper material use and/or poor workmanship during construction, particularly as relates to installation of the tie connections, as has been observed following recent seismic events around the U.S. and in Christchurch, New Zealand. Observed damage from these recent events is meant to motivate further study of this topic, as well as to outline deficiencies in existing brick veneer construction.

Experimental and analytical studies have been conducted at the University of Illinois on the out-of-plane seismic performance of anchored brick veneer with wood-frame backup wall systems, to evaluate prescriptive design requirements and common construction practices in the U.S. (Reneckis and LaFave 2009). As part of the final phase of the project, finite element (FE) models have been utilized to evaluate the seismic fragility of this form of construction (Reneckis and LaFave 2012). Expanding on those experimental and analytical studies, this paper provides a detailed summary of U.S. standard prescriptive design and construction requirements for anchored brick veneer, followed by an evaluation of an alternative strength design methodology (as permitted by MSJC), with emphasis on structural behaviour of the tie connections. Seismic design forces were computed for four U.S. locations (where brick veneer wall damage has been observed in recent earthquakes), employing the procedures for *Seismic Demands on Nonstructural Components* set forth in the ASCE/SEI 7-10 Standard. Tie connection capacities were then computed for various fastener types, as limited by the fastener pullout strength from the wood backup, in accordance with National Design Specification (NDS) for Wood Construction (NDS 2005). This design methodology is then compared to a performance based design approach, employing seismic fragility curves with performance limits and safety objectives defined in the ASCE/SEI 41-06 Standard. This paper shows that prescriptive design and construction requirements for anchored brick veneer should be followed as a minimum; however, ultimately seismic performance of brick veneer walls will only be improved by proper installation of tie connections in new wall construction, and/or by retrofitting existing walls with post-installed anchors.

2. RECENT EARTHQUAKES AND BRICK VENEER DAMAGE

Within the past five years, brick veneer wall damage (including cracking, relative movement, and collapse) has been observed following four seismic events around the United States, as well as the earthquake and aftershocks in Christchurch, New Zealand. Table 1 presents a summary of these earthquake locations, dates, and peak ground accelerations (PGAs) made available by USGS for select site locations.

Table 1. Summary of recent earthquakes and PGAs from USGS (http://earthquake.usgs.gov/earthquakes/shakemap/).

Earthquake Location		Date	PGA (g)
Nearest City or Town	**Latitude / Longitude**		
Sparks, Oklahoma, USA	35.54°N / 96.75°W	2011 Nov 6	0.16
Mineral, Virginia, USA	37.94°N / 77.93°W	2011 Aug 23	0.25
Christchurch, New Zealand	43.58°S / 172.68°E	2011 Feb 21	1.63
West Salem, Illinois, USA	38.45°N / 87.89°W	2008 Apr 18	0.21
Wells, Nevada, USA	41.15°N / 114.87°W	2008 Feb 21	0.13

After the weak to moderate earthquakes in the U.S., collapsed brick veneer walls generally exposed inadequate or lack of anchorage for the brick masonry to the wood frame backup. For example, after the event in Virginia, several residential homes with brick veneer revealed that tie connections were completely ignored, and therefore the brick veneer was built as a free-standing masonry wall around the wood frame home exterior, as shown in Figure 1(a,b). In older residential construction present in Virginia, as shown in Figure 1(c,d), collapsed brick masonry revealed that anchorage was only provided by driving 16d nails into the wood backup, and then setting the head of the nail into the mortar joints. This type of connection typically provides very little pullout resistance from the wood frame backup because the nail is generally driven a short distance into the wood frame backup. On the other hand, following the strong shaking in Christchurch, New Zealand, homes with collapsed brick veneer generally revealed an adequate number of tie connections, as shown in Figure (e,f); however, as a result of the strong shaking, the masonry itself was the weak component, and anchorage to the backup was lost due to tie connections failing at the mortar joints. The desktop study presented in Section 3.3 on the alternative strength design methodology was carried out for the four U.S. locations where recent seismic events resulted in brick veneer wall damage and/or collapse.

Figure 1. Brick veneer wall damage and collapse following earthquakes in (a-d) August 2011, Mineral, Virginia *(photos by Matthew Eatherton, Virginia Tech)*, and (e,f) February 2011, Christchurch, New Zealand *(photos by Kevin Jackson, Thornton Tomasetti)*.

3. DESIGN AND CONSTRUCTION OF ANCHORED BRICK VENEER

This section begins with an overview of general requirements for design and construction of a residential home with exterior brick veneer, followed by a detailed description of prescriptive requirements. An alternative strength design methodology is then presented and its feasibility is studied with comparison to a performance based design approach.

3.1. General

Residential wood frame home structures with exterior brick masonry veneer are typically built in one or two story configurations, as seen earlier in Figure 1. The wood frame structure and the brick veneer must be supported by a noncombustible foundation, usually made of concrete or masonry. The wood backup structure typically comprises floor framing, walls built of 2x4 (1-1/2 in. x 3-1/2 in. [38 mm x 89 mm]) studs spaced at 16 in. (406 mm) on center (with exterior sheathing and interior gypsum wallboard), and roof/ceiling framing. The IRC (ICC 2009) provides design and construction requirements for all structural components of a home. MSJC (2011) simply recommends that such designs comply with ACI 318 for the concrete foundation, and with NDS (2005) for the wood frame structure; MSJC (2011) would be referenced for the case of designing a masonry foundation.

The MSJC (2011) code and BIA Technical Note 28 (BIA 2002) present requirements for design and construction of the brick masonry veneer itself. The out-of-plane stability of a brick masonry veneer wall is controlled by the masonry wall materials, its height and thickness, and also by the layout and properties of the corrugated sheet metal tie connections that anchor it to the wood frame backup. The brick masonry units in brick veneer should be at least 2-5/8 in. (66.7 mm) thick; however, the mortar mix as well as strength of brick masonry materials are generally not specified in brick veneer walls because, under service loading, there is no consideration for stresses in the veneer, and cracking of the veneer can be tolerated. For seismic design category C or below, brick masonry with Type N mortar is usually used, which is adequate for carrying the self-weight, transferring loads to the tie connections, and limiting flexural cracking of the brick veneer. Type S or M mortars are recommended if a higher masonry flexural strength is needed, as with seismic design categories D and above (and/or in areas of high wind). Additionally, to ensure stability of exterior brick veneer and to control cracking in the masonry, MSJC (2011) requires limiting the out-of-plane service load deflections of the backup wall; however, deflection limits are not specified for wood frame backup walls. Prescriptive requirements for the installation of tie connections, as well as for dimensioning residential brick veneer walls, are summarized in the next sub-section, followed by an overview of an alternative strength design approach.

3.2. Prescriptive Requirements and Common Construction Practice

Prescriptive requirements for brick veneer over wood frame backup wall design and construction are specified in the MSJC (2011), the IRC (ICC 2009), and the BIA Technical Notes (2002, 2003). The tie connections should satisfy a set of performance requirements such as: a) sufficient strength and stiffness (in tension and compression) to transfer lateral loads to the backup, b) adequate transverse flexibility to accommodate differential vertical movements between exterior and interior walls, and c) resistance to corrosion and moisture transfer across the air cavity (BIA 2003). For anchoring brick veneer to a wood frame backup, the minimum tie thickness is specified as 22 ga., installed with a maximum bend eccentricity of 1/2 in. (12.7 mm) (with the exception of IRC, which does not specify tie bend eccentricity limits), and attached to the wood backup studs with at least 8d nails, as shown in Figure 2(a). Furthermore, the maximum wall area to be supported by the ties is limited to 2.67 ft^2 (0.25 m^2) for construction in seismic design categories C and below (and for typical wind exposure conditions), and it should be reduced to 2 ft^2 (0.19 m^2) in seismic design category D and above; respectively, these wall areas correspond to tie grid spacings of 24 in. x 16 in. (610 mm x 406 mm), and 16 in. x 16 in. (406 mm x 406 mm), in actual construction. Also, for seismic design category D and above, the ties should be attached to the wood backup with a No. 10 corrosion resistant screw, or a fastener having equivalent or greater pullout strength (MSJC 2011).

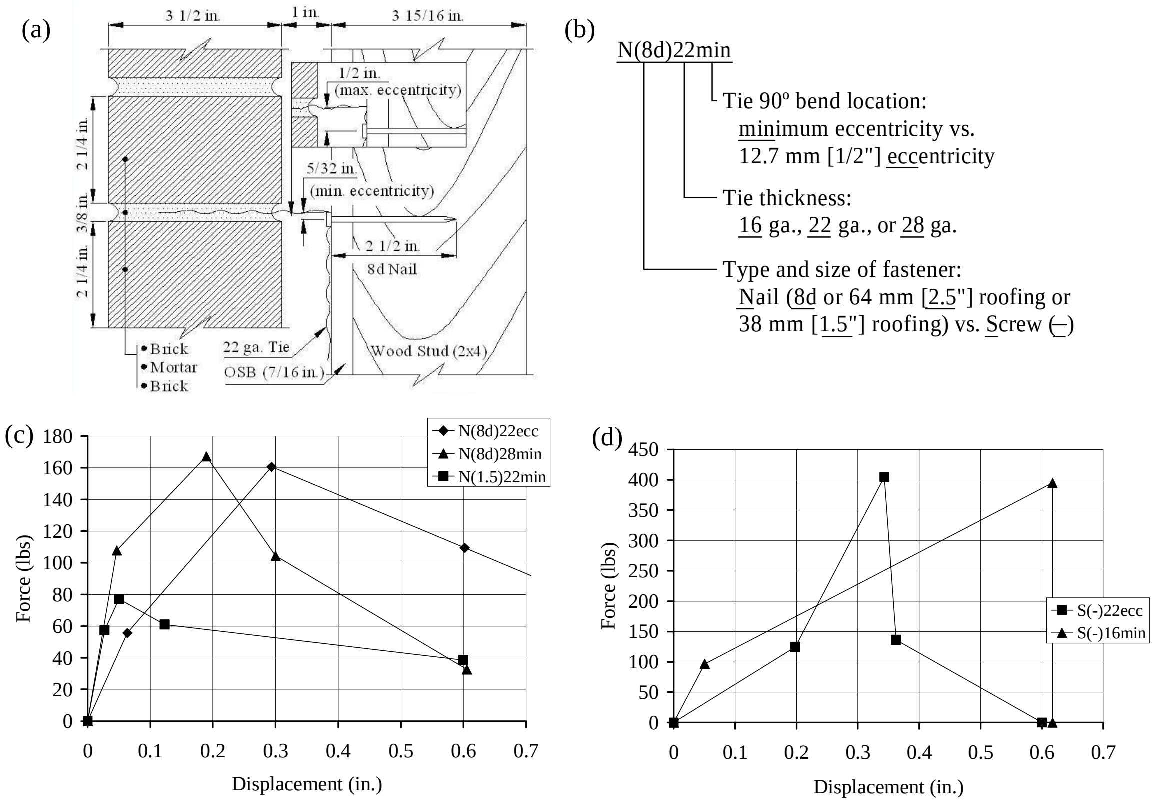

Figure 2. Tie connection details: (a) section view of installation, (b) parameters and nomenclature, (c,d) idealized force-displacement behaviour in tension. (1 in. = 25.4 mm; 1 lb = 4.45 N)

Furthermore, MSJC (2011) and IRC (ICC 2003) require that ties be provided within 12 in. (305 mm) of wall edges near openings for all seismic design categories. This dimension is reduced to 8 in. (203 mm) in BIA (2003), where the maximum edge distance is recommended for tie placement near openings and at other discontinuities in brick veneer walls (such as at wall edges, expansion joints, or shelf angles). The height of a brick veneer wall is typically limited to 30 ft (9.14 m) above its support, with an additional 8 ft (2.44 m) permitted at gable ends of a home structure. For more severe seismic conditions, the IRC also specifies reducing the brick veneer wall height limit by 10 ft (3.05 m), as well as isolating brick veneer walls from one another. Furthermore, the MSJC code requires the use of horizontal joint reinforcement spaced at 18 in. (457 mm) vertically, with ties mechanically attached to the reinforcement, in all brick masonry for buildings with seismic design categories E and higher, as well as supporting the brick veneer independently at each level of the building.

In actual construction practice, however, tie installation in brick veneer walls frequently deviates from these requirements; 28 ga. ties and/or shorter roofing nails are commonly used as substitutes, with a variety of tie layouts. Deviation from code requirements has also been noted earlier in the post-earthquake examples of collapsed brick veneer walls. A number of tie connection properties have been investigated earlier by Reneckis and LaFave (2009), representing typical brick veneer walls built in accordance with prescriptive requirements, as well as per methods employed in actual construction practice. Figure 2(b-d) shows the nomenclature and tensile force-displacement results from that earlier study for a number of tie connections subjected to monotonic tensile loading. For ties fastened to the backup with nails, the predominant failure mode observed in these monotonic tension tests of tie connections was nail pullout from the wood stud. For ties fastened to the backup with wood screws, the typical failure mode was tie pullout from the mortar joint. Among other things, these results clearly depict the effect on tie connection strength as a function of fastener type. As seen in Figure 2(c), the tensile strength of tie connections is reduced by over 50% when short roofing nails are used instead of 8d nails. Then, Figure 2(d) shows the significant increase in tensile strength when wood screws are used instead of nails. These tie connection parameters are studied further in the alternative strength design procedures described below.

3.3. Alternative Strength Design Requirements and Example

An alternative strength design approach is provided by MSJC (2011) for anchored brick veneer wall systems. Seismic and wind design forces must be computed, followed by a structural analysis and design of the brick veneer wall and its connection to the wood backup. The masonry veneer should be able to carry its own weight and to transfer out-of-plane face and inertial loads, through the tie connections, back to the wood frame home structure. (On the other hand, subjected to in-plane seismic or wind loading, brick veneer walls will generally have adequate shear strength and overturning resistance.) The out-of-plane deflection of the backup should also be limited to maintain brick veneer stability. The brick veneer itself is not required to resist flexural tensile stresses, and therefore a strength review of the masonry is generally not required for walls subjected to out-of-plane loading.

Seismic design forces were computed for four U.S. locations (where brick veneer wall damage has been observed in recent earthquakes, Table 1), employing the procedures for *Seismic Demands on Nonstructural Components* set forth in the ASCE/SEI 7-10 Standard, as listed in Table 2. Two sets of strength design loads are presented, including: (A) exterior nonstructural wall elements and connections / fasteners of the connecting system, and (B) veneer / low deformability elements and attachments. This was done because both sets of loads appear to be applicable to design of anchored brick veneer; the resulting forces vary because different component response modification (R_p) and amplification (a_p) factors apply for each component. Computed forces for "fasteners of the connecting system" appear to be applicable for tie connection fastener design.

Table 2. Seismic design forces for brick veneer components per ASCE 7-10 (1 ft^2 = 0.0929 m^2; 1 lb = 4.45 N).
(A) Exterior nonstructural wall elements and connections / Fasteners of the connecting system
(B) Veneer / Low deformability elements and attachments

Location / City or Town	**Seismic Design Data**[a]		**Brick Veneer Wall Properties**[b]		**Seismic Design Force, F_p (lbs)**[c]	
	Seismic Design Category	Short Period Spectral Acceleration, S_{DS} (g)	Supported Wall Area, A (ft^2)	Supported Wall Weight, W_p (lbs)	**(A)** R_p = 1.0; a_p = 1.25	**(B)** R_p = 1.5; a_p = 1.0
Sparks, OK	B	0.194	2.67	107	31.1	16.6
Mineral, VA	B	0.232	2.67	107	37.2	19.9
West Salem, IL	D	0.519	2.00	80	62.3	33.2
Wells, NV	C	0.451	2.67	107	72.4	38.6

(a) Risk Category I; Low hazard; Site Class D

(b) Weight of brick masonry veneer, 40 lbs/ft^2 (1.92 kPa)

(c) Seismic design force per ASCE 7-10, Section 13.3.1

The design seismic loads were then compared with the nominal resistance of brick veneer tie connections, which were assumed to be equal to the pullout strength of various fasteners, with estimated strengths computed per NDS (2005). Table 3 shows typical standard fastener pullout values in ASD and LRFD, for two backup conditions. The first set of results presents fastener pullout from a 2x4 stud alone, and the second is pullout from a stud with a 7/16 in. (11 mm) layer of oriented strand board (OSB) sheathing. (The layer of sheathing results in slightly higher pullout strength because of OSB's higher density.) Fastener pullout strengths computed per LRFD, and where OSB is present, are approximately 20% lower than the experimentally measured tensile failure loads of brick veneer tie connections (Figure 2(c-d)) from earlier studies by Reneckis and LaFave (2009). Furthermore, it should be noted that wood moisture content can lead to a fair amount of variability in pullout capacity, as has been shown by Okail et al (2010).

Ignoring the positive effect of OSB sheathing, it can be seen that 8d nails (as prescribed for tie connections up through seismic design category C) can be employed for resisting design loads of up to 112 lbs (0.498 kN), and #10 screws (as prescribed for tie connections in seismic design category D and above) can resist loads of up to 342 lbs (1.52 kN). Compared to the computed design forces in Table 2, an 8d nail appears to meet the load demands for brick veneer tie connection design in all example cities (including that in seismic design category D). Furthermore, these results imply that fasteners as short as 1.25 in. (32 mm) roofing nails are adequate for connection design in Sparks, Oklahoma, and Mineral, Virginia (seismic design category B), and 6d nails appear to be adequate for tie connections in West Salem, Illinois (seismic design category D). The feasibility of using shorter fasteners in tie connections for these locations is explored further in the performance based design example presented below.

Table 3. Fastener pullout capacities per NDS (2005). (1 in. = 25.4 mm; 1 lb = 4.45 N)

Type of Fastener	Fastener Properties		Fastener Pullout Capacity from SPF 2x4[(a)]			Fastener Pullout Capacity from SPF 2x4 with 7/16" OSB[(a,b)]		
	Length, L (in.)	Diameter, D (in.)	W (lbs)	W'_{ASD} (lbs)[(c)]	W'_{LRFD} (lbs)[(d)]	W (lbs)	W'_{ASD} (lbs)[(c)]	W'_{LRFD} (lbs)[(d)]
1.25" Roofing Nail	1.25	0.113	22	36	48	27	42	57
1.5" Roofing Nail	1.5	0.113	27	43	58	31	50	67
2.5" Roofing Nail	2.5	0.113	45	71	96	49	78	105
6d Nail	2.0	0.099	31	50	67	35	56	76
8d Nail	**2.5**	**0.131**	**52**	**83**	**112**	**57**	**91**	**122**
#8 Screw	2.5	0.164	137	220	297	147	236	318
#10 Screw	**2.5**	**0.189**	**158**	**253**	**342**	**170**	**272**	**367**

(a) 2x4 wood frame backup: Standard Grade Spruce-Pine-Fir (SPF); Specific gravity, G = 0.42

(b) OSB sheathing: APA Rated 24/16; Thickness, t = 0.4375 in.; Specific gravity, G = 0.50

(c) ASD adjustment factors: $C_D = 1.6$; $C_M = C_t = C_{eg} = C_{tn} = 1.0$

(d) LRFD adjustment factors: $K_F = 3.32$; $\phi_z = 0.65$; $\lambda = 1.0$

3.4. Performance Based Design

This section begins with a description of performance limit states for brick veneer walls. Then, seismic fragility curves developed earlier by Reneckis and LaFave (2012) are described and utilized for performance based design of brick veneer for the example U.S. cities, in accordance with ASCE 41-06. Limitations of the alternative strength design methodology are discussed.

3.4.1 Brick Veneer Wall Performance Limit States

According to the ASCE 41-06 Standard for Seismic Rehabilitation of Existing Buildings (ASCE 2006), the seismic performance objectives for buildings can be described qualitatively in terms of: the safety afforded to building occupants during and after the event; the cost and feasibility of restoring the building to its pre-earthquake condition; the length of time the building is removed from service to effect repairs; and economic, architectural, or historic impacts on the larger community. These performance characteristics are directly related to the extent of damage that would be sustained by the building. It appears that the primary objectives for the seismic performance of residential anchored brick veneer will be related to maintaining occupant safety, along with cost and feasibility of repairs.

Table 4. Performance levels for architectural cladding components per ASCE 41-06.

Immediate Occupancy (IO)	Life Safety (LS)	Hazards Reduced (HR)
Connections yield; minor cracks (< 1/16 in. width) or bending in cladding.	Severe distortion in connections. Distributed cracking, bending, crushing, and spalling of cladding components. Some fracturing of cladding, but panels do not fall.	Severe distortion in connections. Distributed cracking, bending, crushing, and spalling of cladding components. Some fracturing of cladding, but panels do not fall in areas of public assembly.

In terms of safety objectives, ASCE 41-06 requires that anchored brick veneer wall components satisfy three performance levels, including: Immediate Occupancy (IO), Life Safety (LS), and

Hazards Reduced (HR). Qualitative descriptions of these performance levels for architectural cladding components (most closely applicable to anchored brick veneer) are summarized in Table 4. Brick veneer wall damage can also be evaluated in terms of cost and feasibility of repairs. Repairable damage will typically involve re-anchoring, as well as some tuckpointing or crack repair of the brick veneer; at the ultimate limit state, collapse will require partial or full reconstruction of the brick veneer. Overall, it can be expected that "repairable damage" will result in repair costs of approximately several hundred dollars (perhaps up to a few thousand dollars). Reconstruction of collapsed walls, on the other hand, might result in a few thousand and maybe up to tens of thousands of dollars' worth of repairs (a significant portion of the total cost of a single-family home). This type of information can be utilized by building owners, as well as insurance companies, to estimate probable financial losses of residential brick veneer construction during earthquakes.

3.4.2 Seismic Fragility Curves

During experimental studies of brick veneer wall panels, it was noted that the overall veneer wall response depended primarily on the tensile performance of the tie connections (Reneckis and LaFave 2009). At the onset of tie damage in brick veneer walls, peak measured tie elongations were found to be closely related to elongations determined for ultimate loading during tie subassembly tests. Different ranges of brick veneer wall behaviour (including *elastic*, *intermediate*, and *ultimate*) and related damage limit states were then identified and evaluated analytically with 3-D FE models by focusing on the tensile performance of key tie connections, without explicitly evaluating for cracking of the brick veneer. Based on the observed performance, a simplified 2-D brick veneer wall strip model has been developed for fragility assessment of this form of construction (Reneckis and LaFave 2012). With this simplified model, two damage limit states are evaluated: *(i-ii)* onset/accumulation of tie failure at the top of the wall (a combination of the first two damage limit states evaluated earlier experimentally and analytically with 3-D models), and *(iii)* tie failure at the lower rows from the top (representing brick veneer wall instability/collapse). In general, both IO and LS performance levels can therefore be related to limit state (*i-ii*), and the HR performance level can be related to limit state (*iii*). In terms of repair costs, these two damage limit states can generally be described as: (*i-ii*) repairable damage, and (*iii*) collapse (possibly requiring major reconstruction).

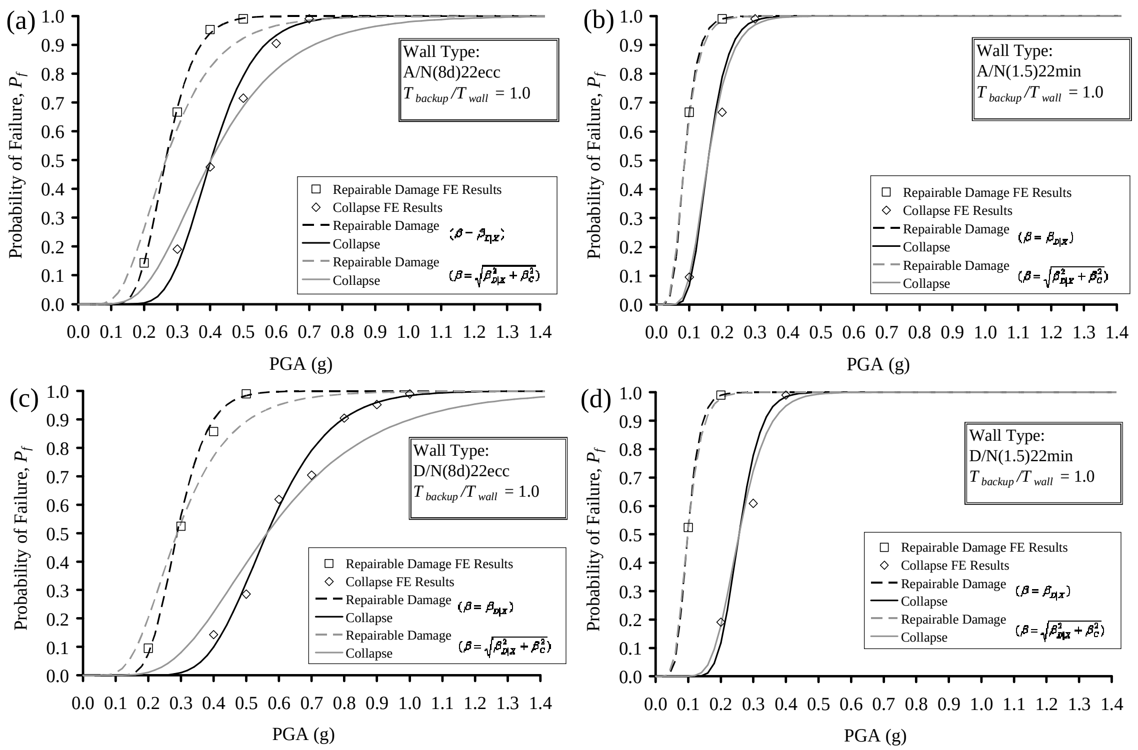

Figure 3. Seismic fragility curves for brick veneer walls with (a) N(8d)22ecc and (b) N(1.5)22min types of tie connections spaced at 16 in. horizontally and 24 in. vertically. Seismic fragility curves for brick veneer walls with (c) N(8d)22ecc and (d) N(1.5)22min types of tie connections spaced at 16 in. horizontally and vertically.

The seismic performance and damage of brick veneer walls have been attributed to the performance of the corrugated sheet metal tie connections; therefore, brick veneer wall fragilities were evaluated earlier by Reneckis and LaFave (2012) as a function of three representative types of tie connection properties: (1) code compliant 22 ga. ties with 1/2 in. maximum bend eccentricity, attached to the wood stud by an 8d nail (N(8d)22ecc); (2) thinner 28 ga. ties without a bend eccentricity, also attached by an 8d nail (N(8d)28min); and (3) 22 ga. ties without a bend eccentricity, attached by a 1.5 in. (38 mm) roofing nail (N(1.5)22min), representing poor workmanship during tie installation. (The idealized tensile force-displacement relationships for these tie connections was shown earlier in Figure 2(c), from experimental tie subassembly test results by Reneckis and LaFave (2009).) For development of these fragility curves, a horizontal tie spacing of 16 in. (406 mm) with a vertical tie spacing of 24 in. (610 mm) (labelled as wall type A), were assigned to represent the maximum supported brick veneer wall area (per tie) requirement in seismic design category C or lower per MSJC (2011). Then, the vertical spacing was reduced to 16 in. (406 mm) (labelled as wall type D), representing a maximum supported wall area requirement for seismic design category D or higher. A set of fragility curves from the earlier study are shown in Figure 3, representing N(8d)22ecc and N(1.5)22min types of tie connections, including wall types A and D.

3.4.3 Seismic Hazard of Brick Veneer Located in Example Cities

Damage limit states for anchored residential brick veneer can be compared with the acceptable seismic performance levels established in ASCE 41-06, as described above and shown in Table 4. The Basic Safety Objective (BSO) for the seismic performance of buildings requires that the Life Safety (LS) performance objective be met for a 10% in 50 year earthquake hazard level, and that the Collapse Prevention (CP) objective be met for a 2% in 50 year hazard. Therefore, the BSO for residential anchored brick veneer can be achieved when damage limit state (*i*-*ii*) onset/accumulation of wall tie damage (similar to the IO and LS performance objectives) is met for the 10% in 50 year earthquake hazard, and damage limit state (*iii*) wall instability/collapse (similar to the Hazards Reduced (HR) performance objective) is met for the 2% in 50 year hazard. (The performance objectives and associated damage limit states identified for brick veneer walls are based on the authors' engineering judgment with respect to interpreting ASCE 41-06. Collapse of brick masonry veneer during an earthquake can pose a substantial hazard to the public (due to the significant mass of brick masonry), and therefore meeting the HR performance objective for the 2% in 50 year hazard appears reasonable, although it could be viewed as somewhat conservative for a nonstructural building component.)

Seismic fragility functions for brick veneer walls were then implemented to assess the seismic hazard of this form of construction for the example U.S. cities. Peak ground accelerations for 10% and 2% in 50 year earthquake hazards were obtained from the USGS website (http://gldims.cr.usgs.gov/) for each city, and the probabilities of brick veneer damage were then evaluated for each and listed in Table 5. The results shown are for damage limit state (*i*-*ii*) onset/accumulation of wall tie damage at the 10% in 50 year earthquake hazard, and damage limit state (*iii*) wall instability/collapse at the 2% in 50 year hazard. The Table 5 seismic hazard results do have some limitations due to the fragility assessment methodology, particularly as relates to synthetic earthquake selection and scaling methodology, described in greater detail elsewhere (Reneckis and LaFave 2012).

From these results, it can be seen that all brick veneer walls studied herein are expected to perform well in Sparks, Oklahoma. In Mineral, Virginia, a veneer wall with a standard tie layout (wall type A, with 16 in. horizontal and 24 in. vertical) and tie connections containing roofing nails (N(1.5)22min) has a probability of 23% for collapse; this is in contrast with the alternative strength design results which indicate that short roofing nails are adequate for resisting seismic design loads for this location. Brick veneer construction in West Salem, Illinois, results in very high probabilities of failure for standard tie layouts (wall type A) and when roofing nails are used; there is a slight improvement in expected performance of these walls when the tie spacing is reduced. Tie connections with 8d nails show a great improvement in performance. Fragility analysis results for West Salem, Illinois, also disagree with the results obtained following the alternative strength design approach; as can be seen from Tables 2 and 3, the computed seismic design load of 62 lbs (0.276 kN) is less than the pullout capacity of 67 lbs (0.298 kN) for 1.5 in. (38 mm) roofing nail from a 2x4 with a layer of OSB. (This comparison is valid here because fragility analysis models have been calibrated to strength test results from tie connection specimens containing a layer of OSB; see Reneckis and LaFave 2009.) Results for Wells, Nevada, also indicate that walls built with ties containing short roofing nails will not perform well; however, using 8d nails appears to be acceptable.

Table 5. Example U.S. cities, seismic hazard PGAs, and brick veneer wall failure probabilities (fragilities).

Location / City or Town	Seismic Hazard PGA (g)		Wall Type / Tie Connection Property: A/N(8d)22ecc		A/N(1.5)22min		D/N(8d)22ecc		D/N(1.5)22min	
	10%/50	2%/50	10%/50 *(i-ii)*	2%/50 *(iii)*	10%/50 *(i-ii)*	2%/50 *(iii)*	10%/50 *(i-ii)*	2%/50 *(iii)*	10%/50 *(i-ii)*	2%/50 *(iii)*
Sparks, OK	0.031	0.094	0.00	0.00	0.00	0.04	0.00	0.00	0.00	0.00
Mineral, VA	0.029	0.126	0.00	0.00	0.00	0.23	0.00	0.00	0.00	0.00
West Salem, IL	0.116	0.332	0.00	0.24	0.80	0.99	0.00	0.02	0.71	0.90
Wells, NV	0.078	0.198	0.00	0.00	0.41	0.78	0.00	0.00	0.23	0.10

4. SUMMARY AND CONCLUSIONS

This paper provides a detailed summary of U.S. standard prescriptive design and construction requirements for anchored brick veneer, followed by an evaluation of an alternative strength design methodology (as permitted by MSJC), with emphasis on structural behaviour of the tie connections. Seismic design forces were computed for four U.S. locations (where brick veneer wall damage has been observed in recent earthquakes), employing the procedures for *Seismic Demands on Nonstructural Components* set forth in ASCE/SEI 7-10. Tie connection tensile capacities have been computed, as limited by the fastener pullout strength from the wood backup, in accordance with National Design Specification (NDS) for Wood Construction (NDS 2005). This design methodology has been compared to a performance based design approach, employing seismic fragility curves with performance limits and safety objectives defined in the ASCE/SEI 41-06 Standard. Computed loads per ASCE 7-10 appeared to be too low, and did not effectively capture the response and amplification characteristics of actual brick veneer wall construction. Results showed that prescriptive design and construction requirements for anchored brick veneer should be followed as a minimum; however, ultimately seismic performance of brick veneer walls will only be improved by proper installation of tie connections in new wall construction, or by retrofitting existing walls with post-installed anchors.

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