ANALYSIS OF BRICK VENEER ON CONCRETE MASONRY WALL SUBJECTED
TO IN-PLANE LOADS

Thesis

Submitted to

The School of Engineering of the
UNIVERSITY OF DAYTON

In Partial Fulfillment of the Requirements for

The Degree of

Master of Science in Civil Engineering

By

Stephen A. Marziale

Dayton, Ohio

August, 2014

ANALYSIS OF BRICK VENEER ON CONCRETE MASONRY WALL SUBJECTED
TO IN-PLANE LOADS

Name: Marziale, Stephen Andrew

APPROVED BY:

Elias Toubia, Ph.D., P.E.
Advisory Committee Chairman
Assistant Professor
Department of Civil
& Environmental Engineering

Ömer Bilgin, Ph.D., P.E.
Committee Member
Assistant Professor
Department of Civil &
& Environmental Engineering

Thomas Whitney, Ph.D., P.E.
Committee Member
Assistant Professor
Department of Civil &
Environmental Engineering

John G. Weber, Ph.D.
Associate Dean
School of Engineering

Eddy M. Rojas, Ph.D., M.A., P.E.
Dean, School of Engineering

© Copyright by
Stephen Andrew Marziale
All rights reserved
2014

ABSTRACT

ANALYSIS OF BRICK VENEER ON CONCRETE MASONRY WALL SUBJECTED TO IN-PLANE LOADS

Name: Marziale, Stephen Andrew
University of Dayton

Advisor: Dr. Elias Toubia

Brick veneers are commonplace in modern building construction. Current building codes require veneers to be anchored to a structural backing in order to transfer out-of-plane loads. However, for in-plane loads the code assigns brick veneers as nonparticipating elements. This study utilizes a modified analytical method to examine the in-plane coupling between brick veneers and concrete masonry shear walls. The amount of load transferred through wall ties depends on factors such as tie spacing, tie stiffness, reinforcement, etc. Results indicate that some degrees of composite action exist; around 12 % to 37 % of the applied shear load is transferred to the brick veneer. Veneers should be isolated in their own plane from the seismic-force-resisting system. An optimum location of the isolation joint is proposed to minimize rocking behavior and limit design story drift.

Dedicated to my family

ACKNOWLEDGEMENTS

I would like to share my deepest thanks to my advisor, Dr. Elias Toubia, for his excellent assistance and guidance. His expert knowledge and direction were vital to the completion of this work.

I would also like to thank Mr. James Lintz, whose previous research I am continuing. I especially appreciate his work in developing the analytical method and model which has been further extended in this thesis.

TABLE OF CONTENTS

ABSTRACT	iv
DEDICATION	v
ACKNOWLEDGEMENTS	vi
LIST OF FIGURES	viii
LIST OF TABLES	x
LIST OF ABBREVIATIONS AND NOTATIONS	xi
CHAPTER 1: INTRODUCTION	1
CHAPTER 2: BACKGROUND	5
Related Code Requirements	5
Related Literature Survey	6
CHAPTER 3: METHOD	9
CHAPTER 4: RESULTS	14
Effects of Adding Reinforcement to Brick Veneer with a Wood Shear Wall	14
Influence of Tie Spacing	20
Influence of Tie Stiffness	24
Influence of CMU Compressive Strength	26
Influence of Brick and CMU Thickness	27
Width of Isolation	28
CHAPTER 5: CONCLUSION	31
BIBLIOGRAPHY	33

LIST OF FIGURES

Figure 1. Typical cross section of a brick veneer anchored to a CMU wall.....	2
Figure 2. Location of expansion joints at corners as recommended by the BIA	3
Figure 3. Load-Displacement of various wall ties	8
Figure 4. Diagram of model using springs to represent brick and tie stiffness	9
Figure 5. Illustration of moment of inertia including transformed reinforcement.....	12
Figure 6. Load in tie rows as applied load increases – Case A, RM	15
Figure 7. Deflected veneer shape as applied load increases - Case A, RM.....	15
Figure 8. Load in tie rows at 100% of OSB shear capacity – URM.....	16
Figure 9. Load in tie rows at 100% of OSB shear capacity – RM.....	16
Figure 10. Deflected veneer shape at 100% of OSB shear capacity – URM.....	17
Figure 11. Deflected veneer shape at 100% of OSB shear capacity – RM	17
Figure 12. Loads to failure with an OSB shear wall.....	19
Figure 13. Loads to failure with a plywood shear wall.....	19
Figure 14. Load in tie rows at 100% of CMU shear capacity – CMU and brick reinforced	21
Figure 15. Deflected veneer shape at 100% of CMU shear capacity – CMU and brick reinforced	21
Figure 16. Percentage of applied load transferred to veneer – CMU and brick reinforced	22

Figure 17. Applied load to failure with a CMU shear wall	23
Figure 18. Load in tie rows using T1 and T2 ties	24
Figure 19. Deflected wall shape of veneers using T1 and T2 ties	25
Figure 20. Diagram of an isolated section of brick veneer	29
Figure 21. Width of isolation analysis for various H/L_1	29

LIST OF TABLES

Table 1. MSJC requirements for corrugated sheet metal anchors	5
Table 2. MSJC requirements for adjustable wire anchors	5
Table 3. International standards for maximum tie spacing	6
Table 4. Spacing cases for analysis using a wood shear wall	14
Table 5. Applied load at various failure modes for brick anchored to a wood shear wall	18
Table 6. Spacing cases for analysis using a CMU shear wall.....	20
Table 7. Load to failure for various tie spacings	23
Table 8. Adjustable wall tie stiffnesses.....	24
Table 9. Load to failure for veneers with rigid adjustable wall ties	25
Table 10. Load to failure for various CMU compressive strengths.....	26
Table 11. Load at failure for varying wall thicknesses.....	27

LIST OF ABBREVIATIONS AND NOTATIONS

A_g	Gross cross-sectional area of masonry
A_s	Cross-sectional area of steel
A_v	Net shear area of masonry
ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
BIA	Brick Industry Association
CMU	Concrete masonry unit
CSA	Canadian Standards Association
E	Modulus of Elasticity
F_f	Frictional force
f'_m	Compressive strength of the masonry
f_y	Yielding stress of steel reinforcement
G_a	Apparent wood shear wall stiffness
G_m	Shear modulus of masonry
H	Total vertical height of the wall
h	Height from ground to tie row
I	Moment of Inertia
IBC	International Building Code

k	Stiffness
L	Total in-plane length of the wall
L_1	In-plane length of an isolated wall section
LEED	Leadership in Energy and Environmental Design
MSJC	Masonry Standards Joint Committee
NCMA	National Concrete Masonry Association
n	Ratio of elastic modulus of steel and masonry, E_s/E_m
NDS	National Design Specification
OSB	Oriented strand board
P	Applied in-plane load on the CMU wall
RM	Reinforced masonry
S_h	Horizontal spacing between wall anchors
S_v	Vertical spacing between wall anchors
SNZ	Standards New Zealand
T_{allow}	Maximum allowable tensile load of shear wall anchor
TMS	The Masonry Society
URM	Unreinforced masonry
Δ_a	Maximum allowable elongation of shear wall anchor
Δ_F	Deflection of the spring due to the force in the spring
$\Delta_{k \text{ change}}$	Deflection at which ties switch from initial to secondary stiffness
Δ_P	Deflection of the spring due to the applied load P

Δ_T	Total deflection of the spring at a tie row
δ_{sw}	Deflection of a wood shear wall
μ	Coefficient of friction

CHAPTER 1

INTRODUCTION

Historically, masonry has been a reliable material for centuries of builders and is still prevalent in modern construction. Some of the benefits of masonry include ease of construction, durability, and fire resistance. Additionally, masonry is attractive as a sustainable building material that can earn designers Leadership in Energy and Environmental Design (LEED) credits. Masonry units can be created from recycled aggregate and cementitious materials which can be sourced from local sites. Due to its high thermal mass and specific heat, masonry provides very good insulation and thermal properties which reduce the overall heating and cooling loads of buildings (NCMA TEK 6-9C, 2009).

Brick masonry is especially common as a material for veneer walls. A veneer is a wythe of masonry used as an exterior façade which is connected to a backing material such as steel studs, wood, or concrete masonry. The veneer can be anchored to the backing wall with metal ties, or adhered to the backing with a bonding agent. The two walls are separated by an air gap, typically 2” to 4” wide, which allows moisture to drain from the wall assembly without penetrating the backing material. This air gap further enhances a veneer wall’s thermal properties by allowing heat to dissipate more quickly. A veneer wall is a type of cavity wall that exhibits non-composite behavior. The veneer directly transfers out-of-plane loads to the backing material without adding any strength

or stiffness to the wall system. However, the backing material is assumed to carry the entire in-plane load, and any transfer of in-plane loads and stresses from the shear backing to the veneer is considered to be negligible by the MSJC code. Figure 1 visualizes a typical detail of an anchored brick veneer connected to a backing of concrete masonry units (CMUs).

Figure 1. Typical cross section of a brick veneer anchored to a CMU wall

In order to limit cracking and other failures in the veneer, the Masonry Standards Joint Committee (MSJC) requires designers to limit the deflection of the backing wall but does not specify an exact deflection design limit. Instead, in the commentary of Section 6.1.2 of the *Building Code Requirements for Masonry Structures (Building Code)*, the MSJC references limits recommended by other organizations such as the Brick Industry Association (BIA), who suggest that designers choose a backing deflection limit of either $L/720$ or $L/600$ (TMS 402-11, 2011). Section 1604.3 of the *International Building Code (IBC)* recommends a deflection limit of $L/240$ for brittle exterior walls and interior partitions that utilize brick masonry (IBC, 2012).

Brick tends to expand when exposed to changes in temperature or moisture, and if this movement is restrained, stresses will build up and cause the brickwork to crack. Cracks due to distress can be either compression related (including spalling, crushing, or buckling), or shear and tension related (producing tensile cracking). The MSJC (2011) requires designers to plan for brick expansion and movement and provide expansion joints as necessary. Expansion joints allow the brick to move, relieving compressive stresses. Joints should be placed at wall intersections, corners, windows and other openings, and abrupt changes in wall height. However, the TMS-402 (2011) does not specify the distance from corners at which an expansion or isolation joint should be placed. In order to minimize cracking at corners due to expansion movements, the BIA suggests placing expansion joints within 10 feet (3 m) from the edge of the corner. In addition, the total wraparound distance between expansion joints on opposite sides of a corner need to be greater than 25 feet (7.6 m), as shown in Figure 2 below (BIA Tech. Note 18A, 2006).

Figure 2. Location of expansion joints at corners as recommended by the BIA

The BIA's recommendations on expansion joint details are based on past experience (empirical) as there is currently no consensus on a method of precisely calculating a safe and appropriate placement distance. Similarly, TMS-402 (2011) recognizes that nonparticipating elements should be isolated from the seismic force-resisting system of a structure, but fail to specify a specific method for determining an appropriate width of isolation. The TMS-402 (2011) acknowledges the need for further research on design options that allow non-isolated, nonparticipating elements with corresponding checks for strength, stiffness, and compatibility (MSJC Section 1.18.3.1). This thesis presents further evidence that anchored brick veneers exhibit characteristics of composite action and recommends a method for determining width of isolation.

CHAPTER 2

BACKGROUND

Related Code Requirements

In addition to the aforementioned recommended deflection limits, the TMS-402 (2011) sets forth other requirements for brick veneer construction. The requirements pertaining to dimensions of corrugated sheet metal ties and adjustable wire ties are presented in Tables 1 and 2, respectively. It should be noted that MSJC Code Sections 6.2.2.6.2 and 6.2.2.6.3 in Table 1 apply to a veneer anchored to wood framing.

Table 1. MSJC requirements for corrugated sheet metal anchors

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)
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)

Table 2. MSJC requirements for adjustable wire anchors

Code Section	Category	Requirement
6.2.2.5.3.1	Min. Wire Size	W1.7 (MW11)
6.2.2.5.3.1	Min. Extension From Bend	2 in. (50.8 mm)
6.2.2.5.3.2	Min. Embedment in Mortar Joint	1.5 in. (38.1 mm)
6.2.2.5.3.2	Min. Cover to Outside Face	0.625 in. (15.9 mm)
6.2.2.5.5.2	Max. Clearance Between Connecting Parts	1/16 in. (1.6 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)

Table 3 compares the MSJC code requirements for tie spacing with Canadian and New Zealand standard tie spacings.

Table 3. International standards for maximum tie spacing

MSJC Code Section	Category	Requirement	CAN/CSA-A370-04 Code Section	Category	Requirement	SNZ HB 4236:2002 Code Section	Category	Requirement
6.2.2.5.6.3	Max. Vertical Spacing	25 in. (635 mm)	7.1.1 a.	Max. Vertical Spacing	23.6 in. (600 mm)	2.9.7.1	Max. Vertical Spacing	15.7 in. (400 mm)
6.2.2.5.6.3	Max. Horizontal Spacing	32 in. (813 mm)	7.1.1 b.	Max. Horizontal Spacing	31.5 in. (800 mm)	2.9.7.1	Max. Horizontal Spacing	23.6 in. (600 mm)
6.2.2.5.6.1	Max. Area Per Anchor	2.67 ft ² (0.25 m ²)	10.5.1.4	Max. Vertical Spacing of Corrugated Strip Ties	23.6 in. (600 mm) or 15.7 in. (400 mm)			
			10.5.1.4	Max. Horizontal Spacing of Corrugated Strip Ties	15.7 in. (400 mm) or 23.6 in. (600 mm)			

The Canadian Standards Association (CSA) limits maximum vertical spacing to 600 mm (23.6 in.) and horizontal spacing to 800 mm (31.5 in.) in Section 7.1.1 of CAN/CSA-A370-04. Interestingly, the CSA further reduces the limits for corrugated metal strip ties. Corrugated ties can be spaced at either: 600 mm (23.6 in.) vertically and 400 mm (15.7 in.) horizontally, or 400 mm (15.7 in.) vertically and 600 mm (23.6 in.) horizontally (CSA, 2006). In Section 2.9.7.1 of SNZ HB 4236:2002, Standards New Zealand (SNZ) restricts tie spacing to 400 mm (15.7 in.) vertically and 600 mm (23.6 in.) horizontally.

Related Literature Survey

Much of the past research on in-plane loading scenarios has focused on brick veneers coupled with wood shear walls. In his doctoral dissertation, “The Influence of Brick Veneer on Racking Behavior of Light Frame Wood Stud Walls,” Nikola Zisi (2009) found that an anchored brick veneer can increase the overall stiffness of the wall

assembly between 41% and 60%. Although the combined wall stiffness increased significantly, Zisi (2009) calculated only about a 5% increase in wall strength. Zisi (2009) concluded that the most important factors contributing to the performance of the wall assembly were tie type and tie spacing.

In 2004, Choi and LaFave tested small sub-assemblies of brick connected to wood with 22 gauge corrugated metal ties. They applied monotonic and cyclic loading patterns on the subassembly and determined that the ties deflected based on an initial stiffness, but after a certain deflection the ties would begin to twist and switch to a lesser secondary stiffness.

Zisi and Bennett (2011) also tested the strength and stiffness of 22 gauge corrugated metal ties with brick and wood subassemblies. They reported similar twisting tendencies as Choi and LaFave (2004); however, their values for initial and secondary tie stiffness were far less than Choi and LaFave's (2004) stiffness values.

Williams and Hamid (2005) analyzed a variety of adjustable wire ties connecting brick and a CMU backing. Two types of ties included in their study are an eye & pintle tie, which restricts horizontal movement but allows free vertical movement, and a slotted block tie, which allows movement in both the horizontal and vertical planes. These adjustable ties allow the brick and CMU walls to expand and shrink independently while maintaining a reliable connection between the two walls. Williams and Hamid (2005) labeled the eye & pintle tie as T1 and the slotted block tie as T2. The average stiffness values of both ties are compared with the stiffness values reported by Choi and LaFave (2004) and Zisi and Bennett (2011) in Figure 3.

Figure 3. Load-Displacement of various wall ties (Note: 1 in. = 2.54 cm, 1 lb = 4.448 N)

Lintz and Toubia (2013) created a simplified analytical method to estimate the amount of load transferred through ties and predict the resulting reaction of the brick veneer. After analyzing a variety of wood shear wall designs, they found that an unreinforced brick veneer can overturn or slide along the flashing plane at the base of the wall before the wood shear wall reaches its anticipated capacity.

CHAPTER 3

METHOD

The method for calculating load transfer in brick veneer backed by CMU is a slightly modified version of the method proposed by Lintz and Toubia (2013). This model assumes that the ties and brick veneer can be represented by springs. Since each tie is assumed to deflect horizontally in-plane, ties located in the same horizontal row are considered to act in parallel. Therefore, the total stiffness of a horizontal row of ties is equal to the stiffness of an individual tie multiplied by the number of ties in the row. Then, each tie row stiffness is placed in series with a corresponding brick stiffness at that level. Figure 4 shows a simplified version of this method with two rows of ties.

Figure 4. Diagram of model using springs to represent brick and tie stiffness

The total stiffness of the effective spring at each row can then be calculated using Equation 1.

$$\frac{1}{k_{eff}} = \frac{1}{k_{tie\ row}} + \frac{1}{k_{brick}} \quad (1)$$

Using superposition, the total deflection of the spring equals the deflection caused by the applied load minus deflection caused by the resisting force of the ties and brick at each row. This principle is represented in Equation 2.

$$\Delta_T = \Delta_P - \Delta_F \quad (2)$$

Δ_T = Total deflection of the spring

Δ_P = Deflection of the spring due to the applied load P

Δ_F = Deflection of the spring due to the force in the spring

As more rows are added, the model becomes more complex since the resisting force in the spring at one row will cause a deflection at every other row. Equation 2 is applied at each row and modified to create Equations 3 and 4, assuming two rows of springs.

$$\Delta_{T1} = \Delta_{P1} - \Delta_{F11} - \Delta_{F12} \quad (3)$$

$$\Delta_{T2} = \Delta_{P2} - \Delta_{F21} - \Delta_{F22} \quad (4)$$

Δ_{T1} = Total deflection of the spring at row 1

Δ_{P1} = Deflection of the spring at row 1 due to the applied load P

Δ_{F11} = Deflection of the spring at row 1 due to the force in the spring at row 1

Δ_{F12} = Deflection of the spring at row 1 due to the force in the spring at row 2

Δ_{T2} = Total deflection of the spring at row 2

Δ_{P2} = Deflection of the spring at row 2 due to the applied load P

Δ_{F21} = Deflection of the spring at row 2 due to the force in the spring at row 1

Δ_{F22} = Deflection of the spring at row 2 due to the force in the spring at row 2

This process can be extrapolated for any number of rows to form a system of linear equations that describe the total deflection at each row. These deflections are

calculated by combining traditional deflection equations for a cantilever beam and specific deflection equations for masonry. Lintz and Toubia (2013) use a modified version of equation 4.3-1 from the 2008 National Design Specification (NDS) Wind & Seismic code to describe the deflection of a wood shear wall. The modified equation is presented as Equation 5. The first term expresses deflection due to bending, the second term expresses deflection due to shear, and the third term expresses wall deflection due to anchor pullout as the base of the wall.

$$\delta_{sw} = \frac{2PH^3}{3EAL^2} + \frac{PH}{G_aL} + \frac{PH^2\Delta_a}{L^2T_{allow}} \quad (5)$$

Deflection of masonry is defined by Equation 6, where $A_v = (5/6)*A_g$ and $G_m = (2/5)*E_m$ for clay brick masonry.

$$\Delta_m = \frac{Ph^3}{3E_mI_m} + \frac{Ph}{A_vG_m} \quad (6)$$

After developing the equations to represent the deflection of the brick and masonry walls at each row, the equations are substituted into a linear system based on Equations 3 and 4, and the only unknown variables remaining are the forces in each spring. Once the forces in each spring are found, the deflection of the brick veneer at each level can be calculated. A more detailed explanation of this method can be found in Lintz and Toubia (2013).

Their proposed method is based on an unreinforced brick veneer connected to a wood shear wall. In order to account for steel reinforcement in the brick and CMU shear wall, the area of steel was transformed using a ratio of steel's elastic modulus and the elastic modulus of masonry to create an equivalent area of masonry. The area of

transformed steel was then accounted for in the moment of inertia of the wall, I_m , as shown in Equation 7, where n is the ratio of modulus of elasticity of steel to the modulus of elasticity of masonry.

Figure 5. Illustration of moment of inertia including transformed reinforcement

$$I_m = \frac{1}{12}th^3 + thd_1^2 + nA_s d_2^2 \quad (7)$$

Additionally, a term similar the anchor pullout term in Equation 5 was added to Equation 6 to simulate deflection due to anchor pullout. The adjusted equation for masonry deflection is shown in Equation 8. The maximum tension force in the steel anchor, T_{allow} , was controlled by the bond strength between the steel and grout, assuming a bond strength of 160 psi according to section 2.1.7.2 of the TMS-402 (2011). The corresponding maximum allowable deflection, Δ_a , is the deflection of the steel reinforcement at T_{allow} .

$$\Delta_m = \frac{Ph^3}{3E_m I_m} + \frac{Ph}{A_v G_m} + \frac{PH^2 \Delta_a}{L^2 T_{allow}} \quad (8)$$

Another modification incorporates the steel reinforcement anchors into equation for sliding along the flashing. Equation 9, proposed by Ahmadi et al. (2013), accounts for the friction between the flashing and the veneer as well as the additional resistance provided by the steel reinforcement. Ahmadi et al. (2013) determined a coefficient of friction, μ , of 0.63.

$$V_s = \mu(A_s f_y + P_n) \quad (9)$$

First, this method was used to analyze the effects of reinforcing a brick veneer attached to a wood shear wall. Then, the wood shear wall was replaced with a CMU shear wall and variables such as tie spacing, tie type, CMU f'_m , and wythe thickness were adjusted to develop an understanding of how influential each factor is to the overall strength and stiffness of the wall assembly. For each trial, load is applied to the shear wall in incremental steps to account for the change in stiffness of the ties until the failure modes are reached. The load step is relative to the total shear capacity of the backing wall. All calculations of wood shear wall strength use Allowable Stress Design (ASD) and conform to the 2012 NDS Wind & Seismic code. Calculations for brick and CMU also use ASD and refer to Chapter 2 of the TMS-402 (2011).

CHAPTER 4

RESULTS

Effects of Adding Reinforcement to Brick Veneer with a Wood Shear Wall

The strength of a wood shear wall can vary widely based on the construction details of the wall. For this analysis, 15/32" thick sheathing and 8d nails were chosen for an 8' x 8' (2.44 m x 2.44 m) shear wall. In addition to examining reinforced and unreinforced veneers, walls composed of oriented strand board (OSB) and plywood were compared. One no. 3 bar was added at each edge of the wall for the reinforced cases. All cases use the stiffness values for corrugated sheet metal ties reported by Zisi & Bennett. The various tie spacing arrangements are shown in Table 4. Case A satisfies MSJC, CSA, and SNZ code requirements, while case C approximates CSA standard 10.5.1.4. Cases B and D are used to note the effects of increasing horizontal and vertical tie spacing individually.

Table 4. Spacing cases for analysis using a wood shear wall

Spacing Case	S_v in (mm)	S_h in (mm)
A	12 (304.8)	16 (406.4)
B	12 (304.8)	32 (812.8)
C	24 (609.6)	16 (406.4)
D	32 (812.8)	16 (406.4)

Case D violates Section 6.2.2.5.6.1 of the TMS-402, however it is included in the following graphs of load in tie rows and wall deflection for analytical purposes. Case D is excluded in the analysis of failure modes.

Figures 6 and 7 illustrate how the reinforced brick veneer with spacing case A and an OSB shear wall reacts to gradual increases in load. Effects at an applied load of 50%, 100%, and 200% of the wood shear capacity are shown. Figure 6 also displays how the ties at the top of the wall twist first, and subsequent tie rows begin to twist as more load is applied to the shear wall.

Figure 6. Load in tie rows as applied load increases – Case A, RM (Note: 1 in. = 25.4 mm, 1 kip = 4.448 kN)

Figure 7. Deflected veneer shape as applied load increases - Case A, RM (Note: 1 in. = 25.4 mm)

Figure 8. Load in tie rows at 100% of OSB shear capacity – URM (Note: 1 in. = 25.4 mm, 1 kip = 4.448 kN)

Figure 9. Load in tie rows at 100% of OSB shear capacity – RM (Note: 1 in. = 25.4 mm, 1 kip = 4.448 kN)

Figure 10. Deflected veneer shape at 100% of OSB shear capacity – URM (Note: 1 in. = 25.4 mm)

Figure 11. Deflected veneer shape at 100% of OSB shear capacity – RM (Note: 1 in. = 25.4 mm)

As evidenced by Figures 8 through 11, reinforcing the brick veneer has a very minimal effect on the amount load transfer through ties or brick deflection. This is mainly due to the backup wall supporting the diaphragm being the primary lateral load resisting system. In addition, two small no. 3 bars provide little additional stiffness. However, the reinforcement does strengthen the veneer as the calculated loads to failure show in Table 5.

Table 5. Applied load at various failure modes for brick anchored to a wood shear wall

Wall Type	Spacing Case	Wood Shear Wall kip (kN)	Brick Sliding $\mu=0.65$ kip (kN)	Brick Overturning kip (kN)	Load at $\Delta = L/720$ kip (kN)
8'x8', 15/32", 8d @ 3", OSB, RM	A	4.40 (19.57)	76.58 (340.64)	54.98 (244.56)	2.81 (12.49)
8'x8', 15/32", 8d @ 3", OSB, RM	B	4.40 (19.57)	143.68 (639.09)	103.65 (461.05)	2.54 (11.32)
8'x8', 15/32", 8d @ 3", OSB, RM	C	4.40 (19.57)	160.51 (713.93)	117.18 (521.24)	2.47 (10.98)
8'x8', 15/32", 8d @ 3", OSB, URM	A	4.40 (19.57)	7.62 (33.88)	12.59 (55.98)	2.81 (12.49)
8'x8', 15/32", 8d @ 3", OSB, URM	B	4.40 (19.57)	22.97 (102.16)	33.36 (148.36)	2.54 (11.32)
8'x8', 15/32", 8d @ 3", OSB, URM	C	4.40 (19.57)	26.82 (119.28)	38.55 (171.49)	2.47 (10.98)
8'x8', 15/32", 8d @ 3", Plywood, RM	A	4.40 (19.57)	59.29 (263.74)	42.76 (190.20)	2.37 (10.55)
8'x8', 15/32", 8d @ 3", Plywood, RM	B	4.40 (19.57)	111.38 (495.40)	80.53 (358.20)	2.09 (9.30)
8'x8', 15/32", 8d @ 3", Plywood, RM	C	4.40 (19.57)	125.89 (559.95)	89.80 (399.43)	2.03 (9.04)
8'x8', 15/32", 8d @ 3", Plywood, URM	A	4.40 (19.57)	5.80 (25.78)	10.63 (47.43)	2.37 (10.55)
8'x8', 15/32", 8d @ 3", Plywood, URM	B	4.40 (19.57)	17.97 (79.91)	25.91 (115.25)	2.09 (9.30)
8'x8', 15/32", 8d @ 3", Plywood, URM	C	4.40 (19.57)	22.53 (100.21)	30.94 (137.62)	2.03 (9.04)

When the brick veneer is reinforced, the load to failure increases significantly in all cases. Otherwise, the brick veneer is at risk of failure, especially sliding, before the wood shear wall reaches its anticipated capacity. The brick sliding failure mode assumes a coefficient of friction of 0.65, which is a little high. According to Lintz and Toubia (2013), coefficients of friction can range from 0.32 to 0.73, depending on the materials used at the sliding interface. If a low coefficient of friction is used along with close tie

spacing and unreinforced brick, the veneer is prone to sliding. Figures 12 and 13 provide visual representations of Table 5.

Figure 12. Loads to failure with an OSB shear wall (Note: 1 kip = 4.448 kN)

Figure 13. Loads to failure with a plywood shear wall (Note: 1 kip = 4.448 kN)

Influence of Tie Spacing

The tie used in this portion will feature the T1 stiffness values found by Williams and Hamid (2005). The tie spacing needs to be adjusted to reflect the geometry of the CMU which dictates vertical spacing in 8” intervals. Similar to the prior scenario with case D, tie spacing cases B and D violate Section 6.2.2.5.6.1 of the TMS-402 (2011), but are included in this analysis to study any trends of increasing tie spacing.

Table 6. Spacing cases for analysis using a CMU shear wall

Spacing Case	S_v in (mm)	S_h in (mm)
A	16 (406.4)	16 (406.4)
B	16 (406.4)	32 (812.8)
C	24 (609.6)	16 (406.4)
D	32 (812.8)	16 (406.4)

For each tie spacing case, the wall assembly was studied with both walls unreinforced, CMU reinforced and brick unreinforced, and both walls reinforced. The CMU wall was reinforced with no. 4 bars placed at 48” on center. The brick veneer was reinforced with one no. 3 bar at each edge of the wall. Figures 14 and 15 display load transfer and veneer deflection patterns at an applied load equal to the maximum shear capacity of the CMU wall. These figures are taken from the case of both walls reinforced.

Figure 14. Load in tie rows at 100% of CMU shear capacity – CMU and brick reinforced
 (Note: 1 in. = 25.4 mm, 1 kip = 4.448 kN)

Figure 15. Deflected veneer shape at 100% of CMU shear capacity – CMU and brick reinforced
 (Note: 1 in. = 25.4 mm)

Figure 16. Percentage of applied load transferred to veneer – CMU and brick reinforced
(Note: 1 kip = 4.448 kN)

Figure 16 shows how much of the applied load is transferred through the ties to the veneer when both walls are reinforced. The brick resists the most force immediately at low loads before eventually settling to a constant percentage. This is due to the high initial stiffness of the wall ties. After the ties begin to twist (or locally deform), their secondary stiffness will engage and less load is transferred to the brick. Figure 16 also indicates that the closest tie spacing, case A (horizontal and vertical), transfers the most load.

Table 7 presents the applied load to failure for various tie spacing and reinforcement cases. Although CMU shear failure and overturning are possible failures, the model ignores any vertical axial load on the CMU wall. In reality, the CMU wall would support the loads of floors and roofs above its level which increases the CMU

wall's ability to resist overturning and shear. Therefore, these CMU failure modes are unlikely.

In most cases, the brick veneer will slide before it overturns; particularly if the brick is unreinforced.

Table 7. Load to failure for various tie spacings

Wall Type	Spacing Case	Brick Sliding $\mu=0.35$ kip (kN)	Brick Sliding $\mu=0.65$ kip (kN)	Brick Overturning kip (kN)	Load at $\Delta = L/720$ kip (kN)	Load at $\Delta = L/240$ kip (kN)	Max. % of Load Transferred to Brick
8'x8', All Unreinforced	A	5.65 (24.14)	13.23 (58.85)	14.24 (63.34)	2.49 (11.06)	7.18 (31.92)	19.16%
8'x8', All Unreinforced	B	10.98 (48.85)	24.05 (106.97)	25.75 (114.55)	2.37 (10.56)	6.95 (30.91)	12.38%
8'x8', All Unreinforced	C	8.78 (39.06)	20.26 (90.11)	21.90 (97.43)	2.34 (10.66)	7.00 (31.14)	14.03%
8'x8', CMU Reinf., Brick Unreinf.	A	6.60 (29.35)	15.41 (68.86)	16.75 (74.49)	2.93 (13.05)	8.56 (38.05)	17.30%
8'x8', CMU Reinf., Brick Unreinf.	B	13.13 (58.39)	28.57 (127.08)	30.81 (137.04)	2.82 (12.55)	8.32 (37.01)	10.83%
8'x8', CMU Reinf., Brick Unreinf.	C	11.56 (51.54)	25.31 (112.59)	27.43 (121.99)	2.84 (12.61)	8.34 (37.09)	12.28%
8'x8', CMU Reinf., Brick Reinf.	A	36.64 (162.96)	72.87 (324.11)	49.59 (220.56)	2.93 (13.05)	8.56 (38.05)	17.30%
8'x8', CMU Reinf., Brick Reinf.	B	65.05 (289.33)	125.89 (559.96)	86.53 (384.86)	2.82 (12.55)	8.32 (37.01)	10.83%
8'x8', CMU Reinf., Brick Reinf.	C	56.81 (252.71)	109.36 (486.42)	76.32 (339.48)	2.84 (12.61)	8.34 (37.09)	12.28%

Figure 17. Applied load to failure with a CMU shear wall (Note: 1 kip = 4.448 kN)

Influence of Tie Stiffness

Using spacing case A as a baseline, two of the adjustable wall ties researched by Williams and Hamid (2005) can be compared. The initial and secondary stiffness of each tie is displayed in Table 8.

Table 8. Adjustable wall tie stiffnesses (Note: 1 in. = 25.4 mm, 1 lb = 4.448 N)

	T1	T2	
k_{initial}	309.8	974	lb/in.
k_{secondary}	123.9	310	lb/in.
Δ_k change	0.0646	0.3	in.

Since T2 ties are far more rigid than T1 ties, T2 ties transfer more load to the veneer as shown in Figure 18. Figure 19 demonstrates that even though the T2 ties transfer more load, they are so rigid that they deflect slightly less than the T1 ties. Both figures were generated with an applied load equal to the shear capacity of the CMU wall.

Figure 18. Load in tie rows using T1 and T2 ties (Note: 1 in. = 25.4 mm, 1 kip = 4.448 kN)

Figure 19. Deflected wall shape of veneers using T1 and T2 ties (Note: 1 in. = 25.4 mm)

Since a rigid tie will force more load on the brick, the veneer with the more rigid T2 ties fails locally at lower loads than the veneer with T1 ties, as shown in Table 9. Also, the veneer with T2 ties carries a higher percentage of the total applied load.

Table 9. Load to failure for veneers with rigid adjustable wall ties

Wall Type	Spacing Case	Tie Type	Brick Sliding $\mu=0.35$ kip (kN)	Brick Sliding $\mu=0.65$ kip (kN)	Brick Overturning kip (kN)	Load at $\Delta = L/720$ kip (kN)	Load at $\Delta = L/240$ kip (kN)	Max. % of Load Transferred to Brick
8'x8', All Unreinforced	A	T1	5.65 (24.14)	13.23 (58.85)	14.24 (63.34)	2.49 (11.06)	7.18 (31.92)	19.16%
8'x8', All Unreinforced	A	T2	4.56 (20.27)	8.46 (37.65)	8.55 (38.03)	2.49 (11.06)	8.76 (38.98)	37.35%
8'x8', CMU Reinf., Brick Unreinf.	A	T1	6.60 (29.35)	15.41 (68.86)	16.75 (74.49)	2.93 (13.05)	8.56 (38.05)	17.30%
8'x8', CMU Reinf., Brick Unreinf.	A	T2	5.05 (22.45)	9.37 (41.69)	9.43 (41.95)	2.98 (13.23)	10.60 (47.15)	36.43%
8'x8', CMU Reinf., Brick Reinf.	A	T1	36.64 (162.96)	72.87 (324.11)	49.59 (220.56)	2.93 (13.05)	8.56 (38.05)	17.30%
8'x8', CMU Reinf., Brick Reinf.	A	T2	15.43 (68.64)	23.88 (106.20)	17.19 (76.48)	2.98 (13.23)	10.60 (47.16)	36.44%

Influence of CMU Compressive Strength

The elastic modulus of masonry is related to compressive strength, f'_m . Since the elastic modulus is featured in the equation for masonry deflection, the compressive strength of the CMU could affect the behavior of the veneer. CMU compressive strengths of 2000 psi, 1500 psi, and 2500 psi were modeled and the results are presented in Table 10. None of the cases exhibited noticeable differences in failure loads when compressive strength was adjusted.

Table 10. Load to failure for various CMU compressive strengths

Wall Type	Spacing Case	CMU f'_m psi (kPa)	Brick Sliding $\mu=0.35$ kip (kN)	Brick Sliding $\mu=0.65$ kip (kN)	Brick Overturning kip (kN)	Load at $\Delta = L/720$ kip (kN)	Load at $\Delta = L/240$ kip (kN)	Max. % of Load Transferred to Brick
8'x8', All Unreinforced	A	2000 (290)	5.65 (24.14)	13.23 (58.85)	14.24 (63.34)	2.49 (11.06)	7.18 (31.92)	19.16%
8'x8', All Unreinforced	A	1500 (218)	5.69 (25.32)	13.26 (59.00)	14.27 (63.45)	2.48 (11.04)	7.15 (31.82)	19.27%
8'x8', All Unreinforced	A	2500 (363)	5.67 (25.21)	13.26 (59.00)	14.27 (63.48)	2.49 (11.08)	7.19 (31.97)	19.13%
8'x8', CMU Reinf., Brick Unreinf.	A	2000 (290)	6.60 (29.35)	15.41 (68.86)	16.75 (74.49)	2.93 (13.05)	8.56 (38.05)	17.30%
8'x8', CMU Reinf., Brick Unreinf.	A	1500 (218)	6.22 (27.68)	14.89 (66.24)	16.00 (71.17)	2.97 (13.20)	8.57 (38.14)	17.36%
8'x8', CMU Reinf., Brick Unreinf.	A	2500 (363)	6.33 (28.13)	15.03 (66.85)	16.39 (72.90)	2.95 (13.10)	8.59 (38.19)	17.26%
8'x8', CMU Reinf., Brick Reinf.	A	2000 (290)	36.64 (162.96)	72.87 (324.11)	49.59 (220.56)	2.93 (13.05)	8.56 (38.05)	17.30%
8'x8', CMU Reinf., Brick Reinf.	A	1500 (218)	36.00 (160.14)	72.10 (320.68)	48.71 (216.68)	2.97 (13.20)	8.57 (38.14)	17.36%
8'x8', CMU Reinf., Brick Reinf.	A	2500 (363)	36.26 (161.29)	72.57 (322.81)	49.29 (219.22)	2.95 (13.10)	8.59 (38.19)	17.26%

Influence of Brick and CMU Thickness

Altering wall thickness can affect the moment of inertia of the wall which is another factor in the wall's deflection. Nominal thicknesses of four and eight inches for brick and CMU, respectively, were chosen as baseline values. As shown in Table 11, increasing either brick or CMU has little effect on any of the failure modes.

Table 11. Load at failure for varying wall thicknesses

Wall Type	Nominal Brick Thickness in (cm)	Nominal CMU Thickness in (cm)	Brick Sliding $\mu=0.35$ kip (kN)	Brick Sliding $\mu=0.65$ kip (kN)	Brick Overturning kip (kN)	Load at $\Delta = L/720$ kip (kN)	Load at $\Delta = L/240$ kip (kN)	Max. % of Load Transferred to Brick
8'x8', All Unreinforced	4 (10.16)	8 (20.32)	5.65 (24.14)	13.23 (58.85)	14.24 (63.34)	2.49 (11.06)	7.18 (31.92)	19.16%
8'x8', All Unreinforced	4 (10.16)	12 (30.48)	5.83 (25.94)	13.62 (60.60)	14.89 (66.21)	2.46 (10.96)	7.16 (31.86)	18.23%
8'x8', All Unreinforced	6 (15.24)	8 (20.32)	5.65 (25.12)	13.22 (58.82)	14.23 (63.31)	2.49 (11.06)	7.18 (31.92)	19.17%
8'x8', CMU Reinf., Brick Unreinf.	4 (10.16)	8 (20.32)	6.60 (29.35)	15.41 (68.86)	16.75 (74.49)	2.93 (13.05)	8.56 (38.05)	17.30%
8'x8', CMU Reinf., Brick Unreinf.	4 (10.16)	12 (30.48)	5.85 (26.03)	14.47 (64.35)	15.68 (69.76)	2.97 (13.22)	8.64 (38.42)	17.24%
8'x8', CMU Reinf., Brick Unreinf.	6 (15.24)	8 (20.32)	6.59 (29.33)	15.41 (68.53)	16.74 (74.45)	2.93 (13.05)	8.56 (38.06)	17.31%
8'x8', CMU Reinf., Brick Reinf.	4 (10.16)	8 (20.32)	36.64 (162.96)	72.87 (324.11)	49.59 (220.56)	2.93 (13.05)	8.56 (38.05)	17.30%
8'x8', CMU Reinf., Brick Reinf.	4 (10.16)	12 (30.48)	35.69 (158.73)	72.06 (320.54)	48.61 (216.24)	2.97 (13.22)	8.64 (38.42)	17.24%
8'x8', CMU Reinf., Brick Reinf.	6 (15.24)	8 (20.32)	36.62 (162.89)	72.84 (324.00)	49.57 (220.47)	2.93 (13.05)	8.56 (38.06)	17.31%

Width of Isolation

As discussed in the introduction section of this thesis, there is no consensus method for calculating an appropriate width of isolation. The MSJC (2011) states that non-isolated, nonparticipating elements can influence a structure's strength and stiffness. Therefore, placing the isolation joints close to the corners of the brick veneer façade can prevent rocking behavior under cyclic loading. The BIA recommends isolating the brick at a distance less than 10 feet from the corner, but ultimately the placement of expansion joints is left to the discretion of the designer. Based on the results discussed in previous sections of this thesis, significant amounts of load can be transferred through ties. If the width of isolation is too narrow, these transferred forces could deflect the isolated section. This model provides a way to approximate a width of isolation based on the forces transferred to the brick through ties.

Using the model developed in chapter 3, the loads in each tie row were calculated at a certain applied load. For this example, a load equal to 25% of the CMU shear capacity (within the elastic response range) was imposed on an 8'x 8' reinforced shear wall with a case A tie spacing. The loads in each tie row were then reduced by a ratio of L_1/L and applied to a brick wall with length L_1 and height H , where L_1 is the length of the isolated section. A diagram of an isolated wall section including applied tie row forces transferred to the brick veneer is shown in Figure 20.

Figure 20. Diagram of an isolated section of brick veneer

The deflection at each tie row was calculated based on the applied tie row forces. This process was repeated for various widths of isolation. Then, the loads in each tie row and the corresponding deflections at each row were plotted, and the result is shown in Figure 21.

Figure 21. Width of isolation analysis for various H/L_1 (Note: 1 in. = 25.4 mm, 1 kip = 4.448 kN)

As L_1 increases, H/L_1 decreases, and the lines begin to converge toward a point where the isolated wall section does not deflect due to transferred forces. Investigating Figure 21, one can notice that an optimum design target would consider a minimal deflection (story drift) and small load in the top tie row. A large deflection will crack the bed and head thin joints, causing the serviceability of the veneer to degrade. Additionally, a large load transferred can locally deform the tie, or spall off and break the interface bond between the mortar and the tie. This will weaken the out-of-plane stiffness and affect the serviceability and water tightness of the first rain wall barrier (veneer). Using this approach, a designer can choose an approximate width of isolation based on an H/L_1 ratio between 3 and 4. For example, for a 12 foot veneer, an isolation joint should be placed at approximately $12/3$, or 4 feet from the edge of each corner.

CHAPTER 5

CONCLUSION

This research work presents a modified version of the method proposed by Lintz and Toubia (2013) to calculate in-plane load transferred through wall ties as well as the deflection of a brick veneer caused by such forces. These modifications include replacing wood backing with concrete masonry backing, and allowing the masonry to be reinforced or unreinforced.

Using the model of a brick veneer anchored to a CMU shear wall, tie stiffness and spacing were found to significantly influence the failure modes of the brick veneer, whereas CMU compressive strength and wall thickness had minimal influence. Close tie spacing and stiff ties will form a more rigid connection between the CMU and brick, and some composite action will occur. As shown in Table 9, a brick veneer anchored with rigid adjustable wall ties spaced at 16 inches horizontally and vertically can be subjected to approximately 37% of the load applied to the shear wall. Even using the largest tie spacing permissible by the MSJC *Building Code* (defined as case C in this thesis), an unreinforced veneer still resists approximately 12% of the applied load. Since the veneer exhibits composite action, it is recommended that brick veneers be isolated (when unreinforced) or include some form of reinforcement as a precaution.

If a designer chooses to not reinforce the veneer, they should limit deflection of the backing to $L/720$. This limit prevented any of the failure modes to occur in the model.

The IBC's suggested limit of $L/240$ is not recommended because, in certain cases, the brick failed due to sliding before reaching the $L/240$ limit. In order to prevent veneer sliding, a flashing material with a coefficient of friction of 0.65 should be used. If a brick veneer is to be unreinforced, the ends of the walls should be isolated at a length approximately $H/3$ or $H/4$ from the edge.

BIBLIOGRAPHY

Ahmadi, Farhad, Jaime Hernandez, Geoff Scheid, and Richard E. Klinger. "Sliding Shear Resistance of Reinforced Masonry Shear Walls." *The Masonry Society Journal*, December 2013. (2013).

American Forest and Paper Association. "Special Design Provisions for Wind and Seismic." American Forest and Paper Association. Washington, D.C. (2011).

Brick Industry Association. "Technical Note 18A: Accommodating Expansion of Brickwork." *Technical Notes on Brick Construction*. Brick Industry Association. Reston, VA. (2006).

Canadian Standards Association. "CAN/CSA-A370-04: Connectors for Masonry." Canadian Standards Association. Toronto, ON, Canada. (2006).

Choi, Young-Hwan, and James M. LaFave. "Performance of Corrugated Metal Ties for Brick Veneer Wall System." *Journal of Materials in Civil Engineering*, May/June 2004, 202-211. (2004).

Filloramo, Richard, and David Sovinski. "Movement Control: Issues in Masonry." *The Construction Specifier*. Aug. 2006.

International Code Council. *2012 International Building Code*. International Code Council, Inc. Country Club Hills, IL. (2011).

Lintz, James M., and Elias A. Toubia. “In-Plane Loading of Brick Veneer Over Wood Shear Walls.” *The Masonry Society Journal*, December 2013. (2013).

Masonry Standards Joint Committee. *Building Code Requirements for Masonry Structures*. (TMS 402-11/ACI 530-11/ASCE 5-11). The Masonry Society, American Concrete Institute, and American Society of Civil Engineers. Boulder, CO, Farmington Hills, MI, and Reston, VA. (2011).

National Concrete Masonry Association. “TEK 3-6C: Concrete Masonry Veneers.” National Concrete Masonry Association. Herndon, VA. (2012).

National Concrete Masonry Association. “TEK 6-9C: Concrete Masonry & Hardscape Products in LEED 2009.” National Concrete Masonry Association. Herndon, VA. (2009).

National Concrete Masonry Association. “TEK 10-4: Crack Control for Concrete Brick and Other Concrete Masonry Veneers.” National Concrete Masonry Association. Herndon, VA. (2001).

National Concrete Masonry Association. “TEK 12-1B: Anchors and Ties for Masonry.” National Concrete Masonry Association. Herndon, VA. (2011).

Standards New Zealand. “SNZ HB 4236:2002: Masonry Veneer Wall Cladding.” Standards New Zealand. Wellington, New Zealand. (2002).

The Masonry Society. *Building Code Requirements and Specifications for Masonry Structures*. (TMS 402-11/ACI 530-11/ASCE 5-11). The Masonry Society, Longmont, CO. (2011).

- Williams, C. R., and A. A. Hamid. "In-Plane Stiffness and Strength of Adjustable Wall Ties." 10th Canadian Masonry Symposium. (2005).
- Yi, Junyi, David Laird, Bill McEwen, and Nigel G. Shrive. "Analysis of Load in Ties in Masonry Veneer Walls." *Canadian Journal of Civil Engineering*, Vol. 30. (2003).
- Zisi, Nikola V. "The Influence of Brick Veneer on Racking Behavior of Light Frame Wood Stud Walls." Ph.D. thesis, University of Tennessee, Knoxville. Knoxville, TN. (2009).
- Zisi, Nikola V., and Richard M. Bennett. "Shear Behavior of Corrugated Metal Tie Connections in Anchored Brick Veneer-Wood Frame Wall Systems." *Journal of Materials in Civil Engineering*, February 2011, 120-130. (2011).