

In-Plane Shear Investigation on Masonry Walls

A Dissertation
Presented for the
Doctor of Philosophy Degree
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Dedication

To my parents, Bill and Brendia Morrison:
Thank you for all of your help, encouragement, love, and insight

To my wife, April:
Thank you for all the encouragement and reassurance

To my grandparents, Curtis and Frances Morrison:
Even though you didn't see me complete this degree,
thoughts of you both were with me always

Acknowledgements

I would like to thank several people for being a part of my research over the last few years, and who have contributed greatly “to the cause” over that time.

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I would like to extend a sincere thanks to my Mom and Dad and my grandparents, Mae and Roddy Reece—all of whom raised me to work hard in whatever I choose to do, and for providing me with the essential tools and advice in life that can never be learned in any institution.

I thank my wife, April, for all of her support and encouragement through all of the stress-filled times that were a product of this research.

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Lastly, and most importantly, I thank God for His Son's saving grace and the wonderful opportunity He has given me to continue my education, to meet great people along the way, and to give me a chance to share a portion of what I have learned over the last ten years of my life with those that will come after.

Abstract

Analysis has been performed on approximately 200 shear walls of various masonry types, reinforcement, and grouting scenarios to determine the trends of various wall parameters to strength. The database is assembled from that belonging to researchers over the last 30 years.

Strut-and-tie methods from current concrete codes are employed and new methods proposed to calculate the in-plane shear strength of masonry walls. In addition, a simplified method based upon strut-and-tie methodology is presented to calculate the masonry contribution to the in-plane strength of these walls, with contributions from axial load and horizontal reinforcement based upon past data and efficiency of reinforcement, respectively.

The proposed strut-and-tie model was validated by the use of structural analysis and finite element software. Out of 35 fully grouted walls from the database and 13 finite element walls constructed with a macro-model approach, strength calculations produced an average $\frac{V_{exp}}{V_{pred}}$ [experimental shear strength/predicted shear strength] of 1.63 with a coefficient of variation [COV] of 0.26, while 23 partially grouted walls from the database along with 12 finite element analysis walls produced an average of 1.44 with COV of 0.25. Average $\frac{V_{exp}}{V_{pred}}$ for 108 clay and concrete walls utilizing the simplified strut-and-tie method is 1.25 with a COV of 0.24 for fully grouted walls and 1.17 with a COV of 0.31 for 87 partially grouted walls.

A shear friction equation is proposed in which the contribution to friction from applied normal force and vertical reinforcement are separated. The coefficient from applied normal force is obtained from previous unreinforced masonry tests and from inclined plane tests that

examined various degrees of surface roughness. A coefficient of friction of 0.80 is proposed for this contribution. The contribution of vertical reinforcement perpendicular to the sliding surface was obtained based upon previously reported dowel tests. A value of 0.45 is proposed. The proposed model is compared to full-scale wall tests and was found to decrease the variability in prediction by almost a factor of two for concrete masonry over models that combine both the axial load and steel contribution terms and utilize only one coefficient.

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Chapter 1—Introduction to Masonry

1.1 Introduction

Masonry, as a viable choice for building material, has existed for thousands of years in different societies. From hand-hewn mud bricks to precision cut concrete masonry units, the art and feasibility of “stacking” units together has been a choice for builders for as long as the idea of building structures has been around. These structures contained parts that protected the individual from the elements, and as knowledge progressed, different additives were joined with the mud, bricks, and rocks to make stronger, more weather-resistant walls. After successive failures of these structures, precision wall positioning and reinforcing undoubtedly became part of the learning curve. Walls were “designed” out of necessity to take bending forces and shear forces from all angles. Many of the terms used in construction today to describe the function of a particular system in a building were surely a result of a wall being designed to resist a particular kind of load. One of the main functions of a masonry wall is to resist lateral loads. The term “shear wall” comes from the walls resisting in-plane shear loads provided by wind, earthquake excitation, or any other lateral load that may act on a building.

Some factors that affect the performance of masonry shear walls are size, mortar type, bed joint thickness, base fixity, aspect ratio, axial stress, grouting scenario, and vertical and horizontal reinforcement type and quantity. The three predominant failure modes of masonry shear walls are failure in flexure, characterized by yielding of the vertical reinforcement and horizontal cracking of the masonry, failure in shear, which can be classified as a type of brittle tension failure with characteristic diagonal cracks along the wall, and sliding failure, in which a course of masonry moves horizontally in relation to another course or base. (Drysedale et al., 1999)

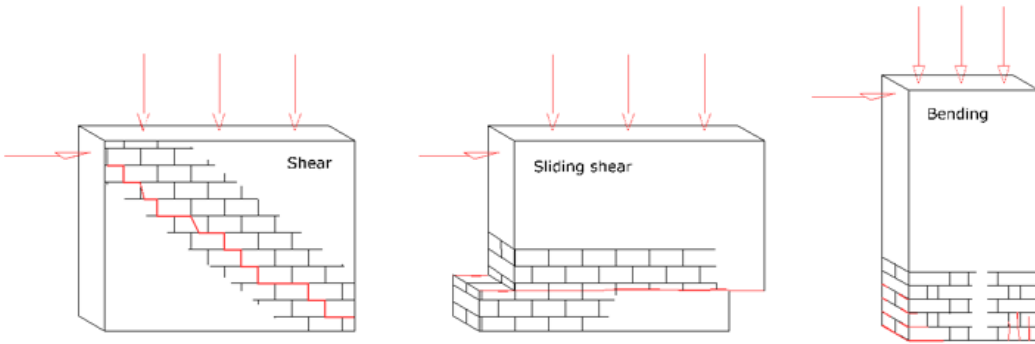


Figure 1.1 (City University, London) Failure Modes of Masonry Walls

Masonry walls generally fall into three categories—unreinforced, reinforced fully grouted, and reinforced partially grouted. It is the partially grouted reinforced walls that have garnered some criticism as of late regarding their capacity in the shear failure mode according to TMS-402 (2011) Masonry Code equations. It appears that the currently accepted design methods have the potential to overestimate the shear strength of partially grouted walls. Several factors, including quantity of shear reinforcement, interpretation of net area for a partially grouted wall, and contributions of usually unacknowledged contributors such as shear friction and dowel action have caused much research to try and determine the true strength of a masonry wall failing in shear.

1.2 Objectives

Currently, no desirable method exists that can closely and consistently predict the lateral load capacity of reinforced partially grouted shear walls, although many improvements and suggested amendments to existing equations have been proposed. One objective of this research

was to evaluate available data on masonry shear wall tests and to utilize strut-based modeling techniques along with other structural theories to accurately account for contributions by the masonry and shear reinforcement in a masonry wall, with an end goal of attaining a sound method for a designer to readily use to accurately and conservatively predict the lateral load capacity of both fully and partially grouted masonry shear walls. It was a goal that the proposed method be “usable” enough that anyone with a basic understanding of how loads and stresses affect a wall in-plane can utilize the theory-based techniques to make sound decisions in regards to designing a masonry wall to resist shear forces.

Additionally, it was a goal to report on and to add to the current database of the true coefficient of friction of concrete masonry units and to develop an expression to account for the ultimate strength of a wall failing in sliding shear. It is believed that simplified, but accurate, inclined plane tests of concrete CMU with various sliding surfaces should lend insight into a correct expression of Mohr-Coulomb failure for sliding of masonry bed joints.

1.3 Masonry Materials

1.3-A Block

The primary component of masonry is the block itself. Often, the term “block” means no block at all, but rather natural stone or even shapes constructed out of glass. The focus of this paper is on the shear strength of walls comprised of traditional concrete, clay, and AAC (Autoclaved Aerated Concrete) block, with an emphasis being placed on concrete masonry units,

as they are generally the most common choice in practice. Therefore, only current manufacturing and common block configurations as applicable to this research is discussed.

Concrete masonry units come in a variety of sizes to suit both architectural and design needs. Some blocks are faced with various cut designs to make them more architecturally pleasing, while others are quite plain and are often covered in stucco, brick, or other forms of siding. Many of the available blocks are shown in Figure 1.2.

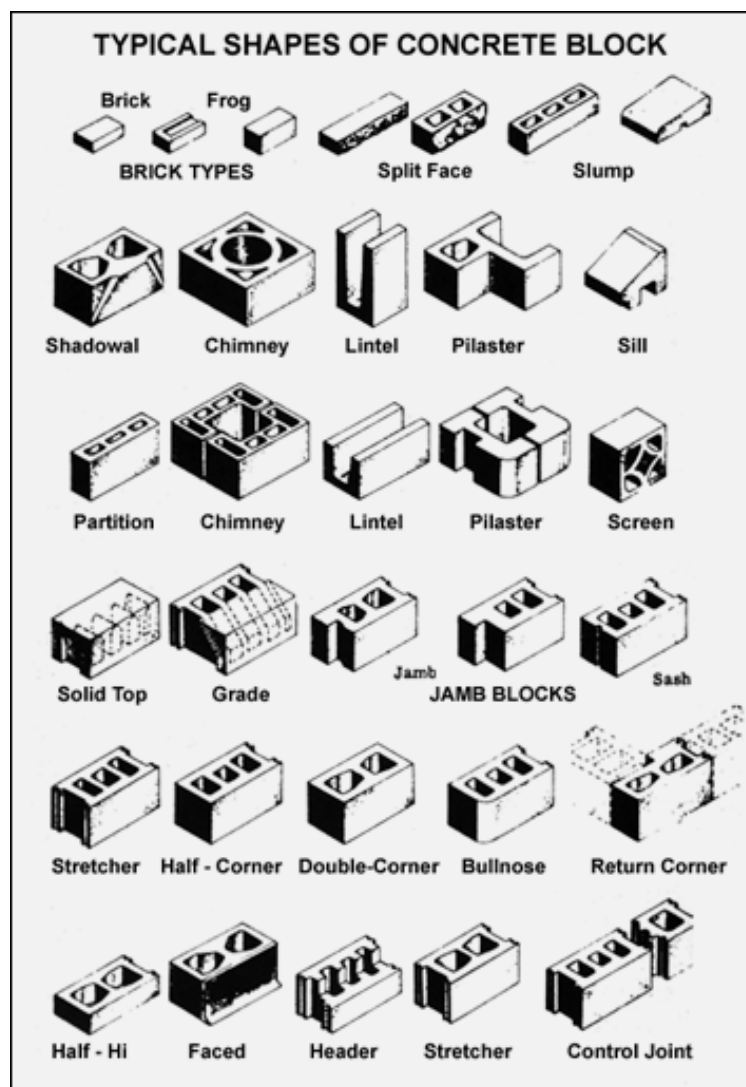


Figure 1.2 (<http://faculty.delhi.edu/hultendc/A220-Week2-Lecture-Web.html>)
Typical Block Configurations

The most common block used in the data sets in this research are the 2-core stretcher block and similar blocks with knockout webs for placement of reinforcement. Blocks that are more than 75% solid are characterized as solid block, while those that are less are known simply as “hollow” block. Face shells of concrete blocks are generally mortared, and occasionally the web to give a full-bedding mortar scenario. For net area purposes, the area that is mortared and/or grouted is considered part of the net area. Minimum face shell thicknesses follow ASTM C90, with values of 1 in. and 1.25 in. for 6 in. and 8 in. nominal CMU, respectively.

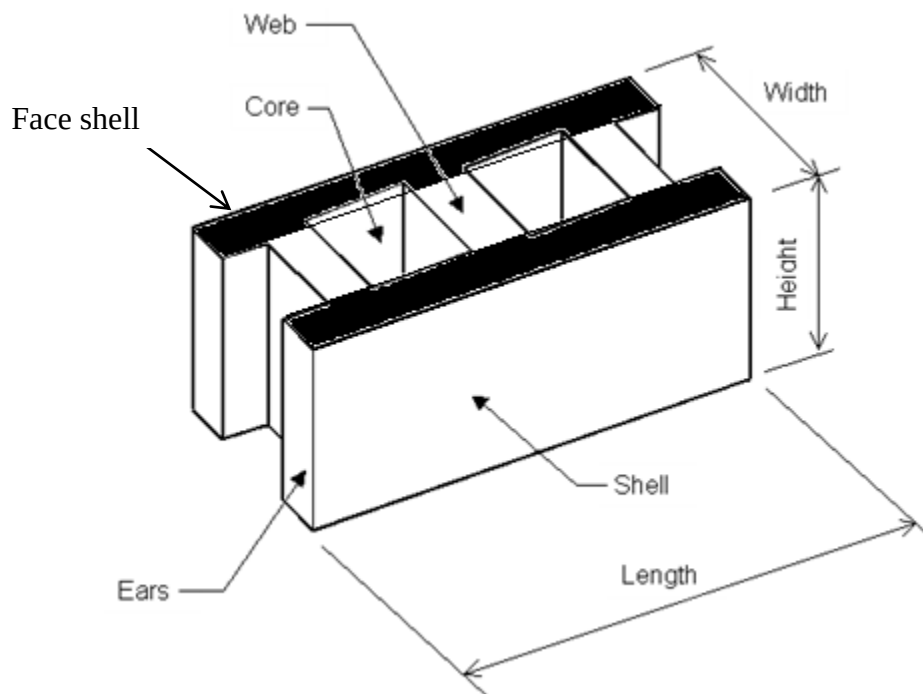


Figure 1.3 (<http://faculty.delhi.edu/hultendc/A220-Week2-Lecture-Web.html>) Typical Concrete Masonry Unit (modified by the author to show face shell area in the shaded regions)

Concrete block show some of the same behavior as traditional concrete, as the materials of manufacture are similar and the concrete block receives its strength from the concrete

hydration process—therefore, concrete “technology” is applicable. A typical stress-strain curve for hollow concrete block is shown in Figure 1.4.

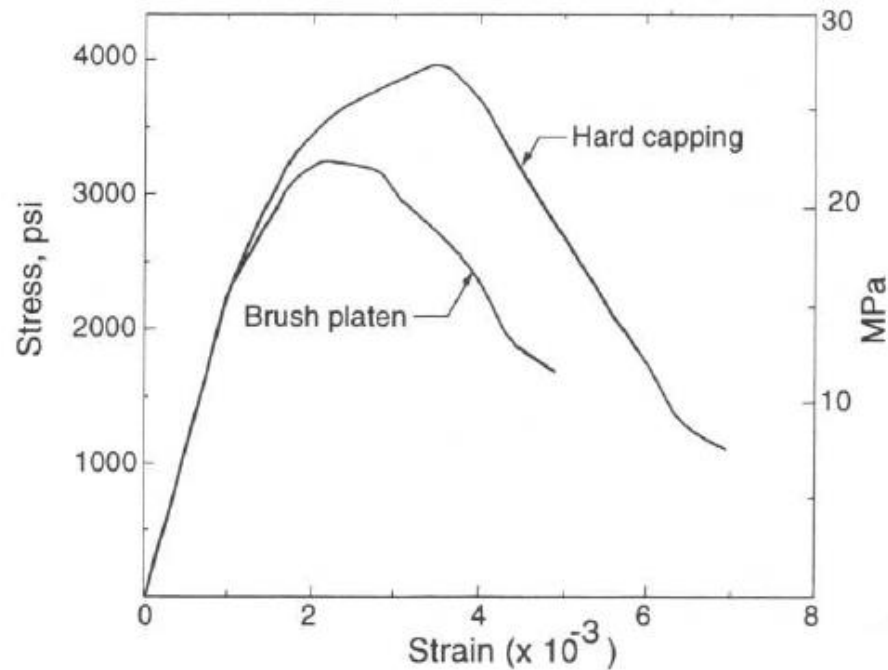


Figure 1.4 (Drysdale et al, 1999) Typical Stress-Strain Curve for Concrete Blocks

Average compressive strengths of concrete blocks are between 1500 and 4000 psi, with a typical average prism strength of slightly less than this (a prism is defined as a small assemblage of the constituents in a member tested to determine the masonry compressive strength, f'_m , as determined by ASTM C1314). (Drysdale et al., 1999)

A representation of block strength as compared with masonry prism strength is shown in Figure 1.5.

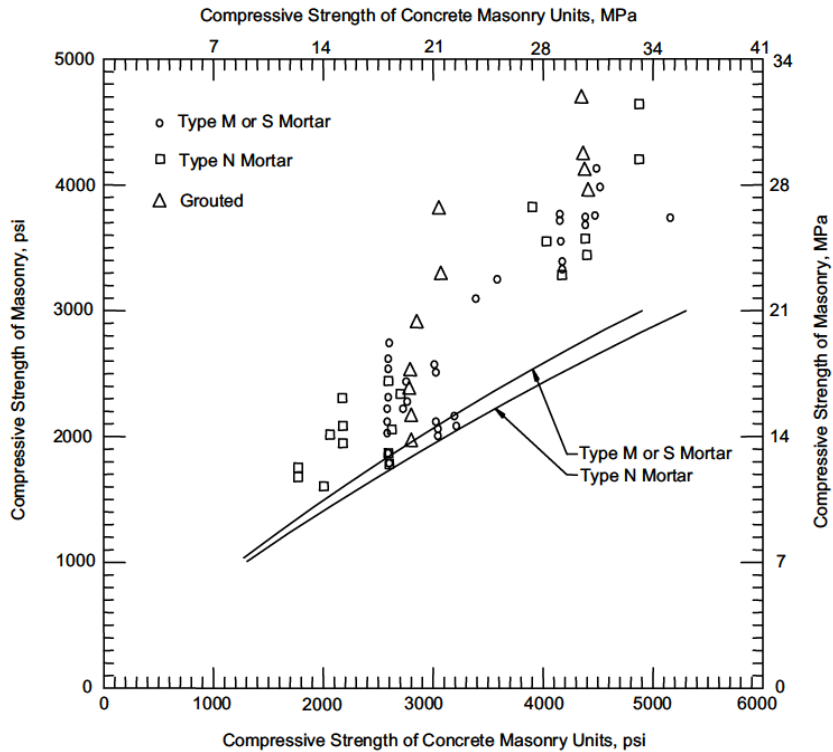


Figure 1.5 (TMS 402-08) Concrete Masonry Prism Strength vs. Block Unit Strength

Typical clay block similar to that used in the analyzed data sets are shown in Figure 1.7. Typical compressive strengths of clay brick/block are in the range of 6000-10000 psi, with prism strengths less than this, as shown in Figure 1.6.

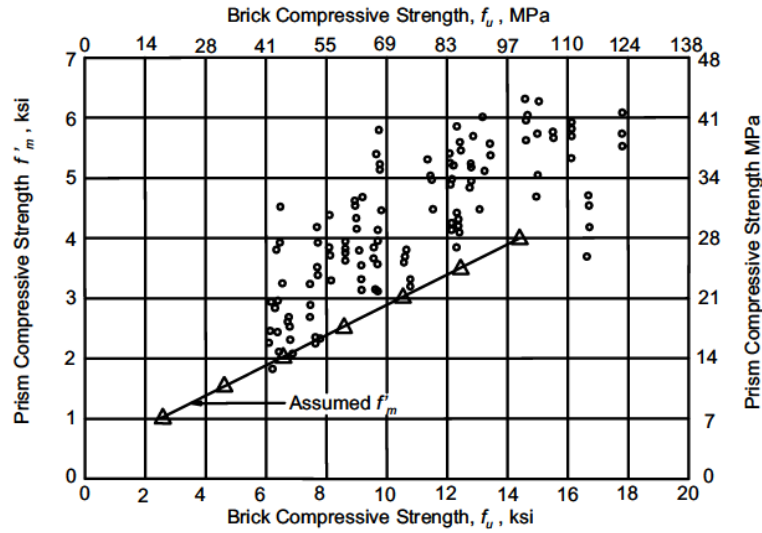


Figure 1.6 (TMS 402-08) Clay Masonry Prism Strength vs. Brick Unit Strength

ASTM 652 covers the minimum face shell width for hollow brick, which is assumed as 1 in. and 1.25 in. for nominal 6 in. and 8 in. bricks, respectively.

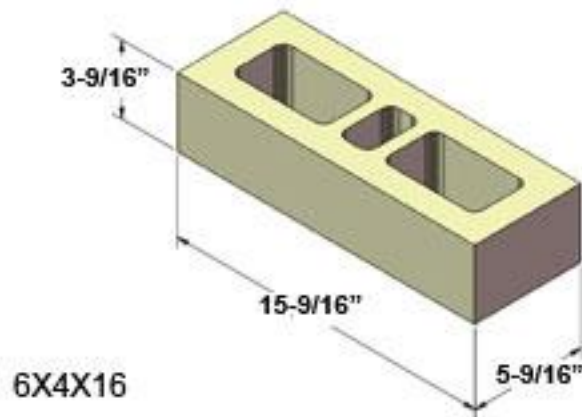


Figure 1.7 (Interstate Brick) Typical 6 in. Clay Block

AAC masonry has been used in this study as a means of verification of the proposed equations for calculation of in-plane strength of shear walls. A description of AAC is given in Tanner (2003), and the typical size of an AAC masonry block for this research is a solid 8 in. x 8

in. x 24 in. unit. The average compressive strengths for the AAC masonry included in this analysis ranges from 400 to 1400 psi.

1.3-B Mortar

Mortar can be classified as the “glue” that binds the masonry units together. The three main types of mortar are lime mortar (a blend of Portland cement, lime, sand, and water), masonry cement, and mortar cement (both of which are generally prepackaged and are able to be readily mixed at a jobsite). The major differences in the masonry and mortar cement are requirements for air content and bond strength. Bond is a measure of the degree to which the masonry units are adhered to each other by way of the mortar, and is affected by the initial rate of absorption of the units, surface roughness, and water retentivity of the mortar. ASTM C270 requires that mortar have strengths ranging from 350 psi to 2500 psi, based upon the type of mortar used. Compressive strength of mortar is typically a less important property than its bond and workability, as a 100% increase in mortar strength may only lead to a 10% increase in wall strength. A typical joint is 3/8” thick, and a wall assemblage is comprised of both head and bed joints, as shown in Figure 1.8. (Drysdale et al., 1999)

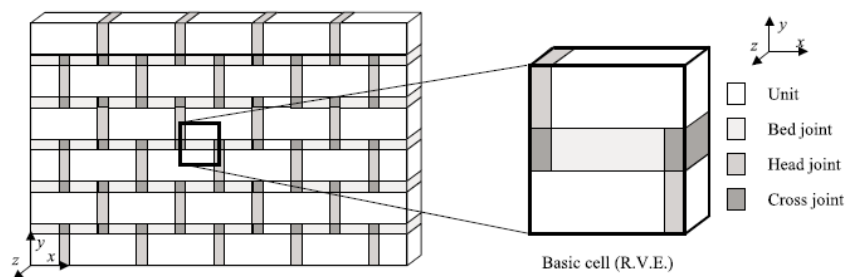


Figure 1.8 (Zucchini and Lourenco, 2002) Joints in Masonry

1.3-C Grout

Grout is typically used to fill cells in a masonry structure that either are required to be fully grouted, or that contain reinforcement. Grout is essentially concrete with small aggregate (usually 3/8" or less) and a high water-cement ratio to achieve a slump of around 8-11 in. The TMS-402 (2011) requires that grout strength be equal to or exceed the compressive strength of the masonry, but not less than 2000 psi. Even though the water-cement ratio is high, generally giving a low material strength, higher strengths are accomplished through extensive absorption through the masonry unit itself.

1.3-D Reinforcement

Typical reinforcement in masonry walls includes both traditional deformed reinforcing bars and ladder-type bed-joint reinforcement that is comprised of small wires of generally high tensile strength material (usually around 70-80 ksi yield strength).

As applied to this research, the TMS-402 (2011) seismic requirements for minimum reinforcement are at least the equivalent of a #4 bar at a maximum spacing of 120 in. on center, and for horizontal reinforcement, at least two longitudinal wires of W1.7 joint reinforcement spaced 16 in. or less on center or at least the equivalent of a #4 bar in a bond beam, not spaced more than 120 in. on center, and within 16 in. of the top of walls.

Shown in Figure 1.9 is a representation of ladder-type bed joint reinforcement and in Figure 1.10, the typical placement of deformed bar reinforcement.

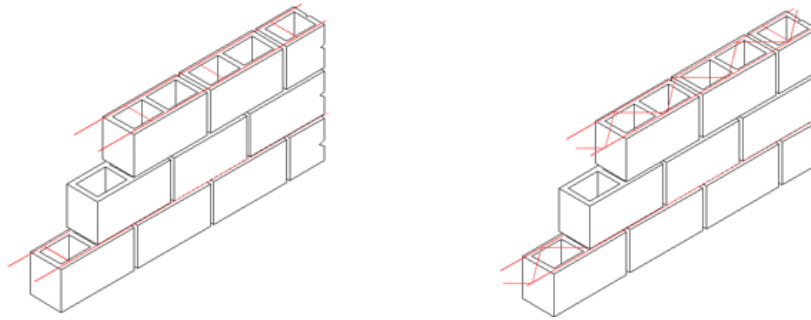


Figure 1.9 (City University, London) Ladder-Type Bedjoint Reinforcement Layout

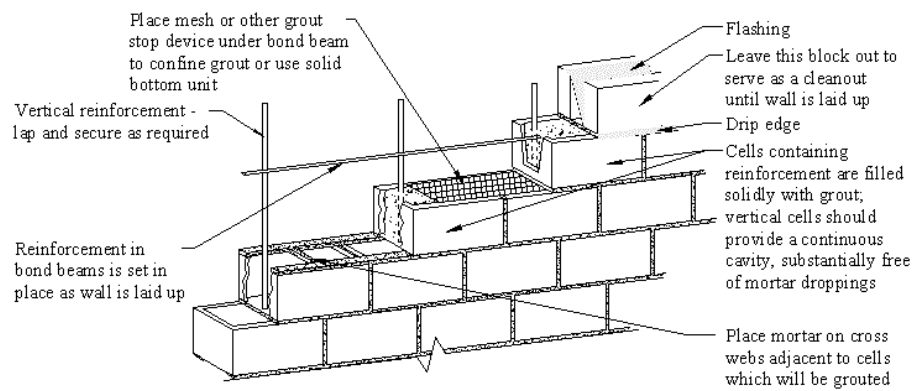


Figure 1.10 (Masonryinstitute.com) Typical Configuration of Vertical and Horizontal Deformed Reinforcement

Typical deformed vertical bars are embedded into a foundation to the required development length, while horizontal bars are often tied to the vertical bars with 180 degree hooks around the end, with specifications conforming to the TMS 402 code.

1.4 Masonry Wall Types

Masonry walls may consist of any type of man-made or natural conglomeration of units and cohesive elements such as concrete masonry units (CMU), clay brick, autoclaved-aerated

concrete (AAC), rock, and paste, mortar, mud, grout, and lime/water combinations, respectively. Of interest to this research are the primary choices for predictability and strength in design, which includes the CMU, clay brick, and AAC masonry elements, joined with various types of mortar, and either fully or partially filled with grout.

Masonry walls included in this research are primarily laid in running bond, as shown in Figure 1.11.

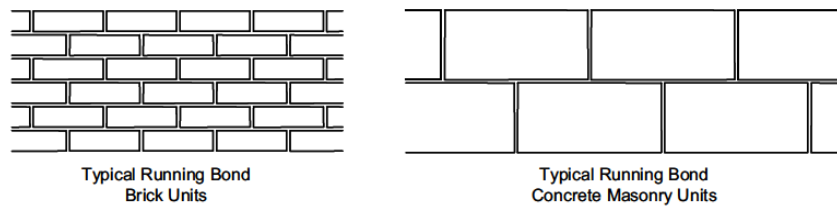


Figure 1.11 (TMS-402, 2011) Running Bond Masonry Wall

1.5 Wall Classification

Masonry walls are generally placed into three categories—slender, square, and squat. In this research, slender walls are those designated with aspect ratios (height/length) greater than 1.0, and squat walls are those with aspect ratios less than 1.0. The TMS-402 code suggests Figure 1.12 in which for substantially squat walls, shear behavior predominates, for walls approaching square, both shear and bending behavior are important, and for substantially slender walls, bending behavior predominates.

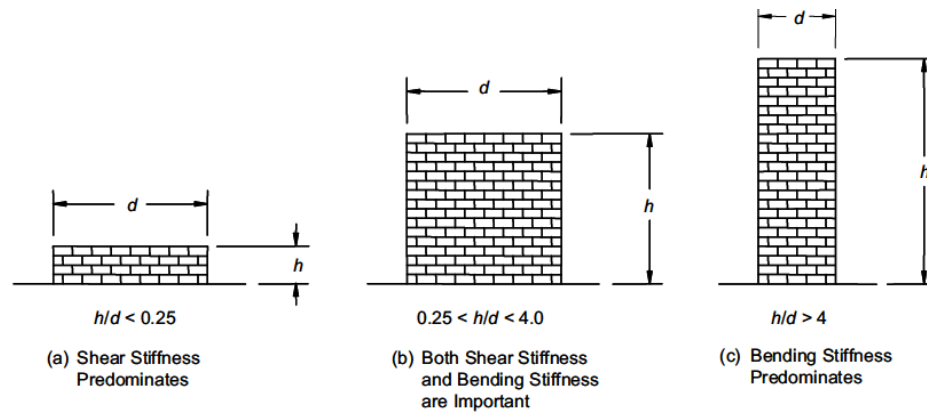


Figure 1.12 (TMS-402, 2011) Wall Classification

Chapter 2—Literature Review

2.1 Literature Review

Various researchers have performed tests on both fully and partially grouted masonry shear walls, highlighting the parameters that appear to influence wall behavior. Some have proposed equations to supplement or alter the currently accepted equations in the Building Code Requirements and Specification for Masonry Structures (TMS-402, 2011)—the predominantly accepted code for masonry design in the United States. Some of those studies are summarized with key points that allude to the behavior and response of masonry shear walls.

In some of the earliest well-documented research on masonry shear walls, the Earthquake Engineering Research Center conducted an extensive study on the behavior of masonry piers (those portions of a shear wall that are between perforations—essentially a small wall) subjected to earthquake-type loading. The test data from this study spans several years and is covered in four volumes of work. Each volume describes the results of testing the masonry piers with varying reinforcement type and placement, axial stress, block type and configuration, and wall type. All walls in the tests were cyclically loaded and fixed against top rotation. Pertinent data for each volume of work is included in Appendix A, Table A-1. Testing configuration and typical wall configuration (with varying reinforcement) is shown in Figures 2.1 and 2.2.

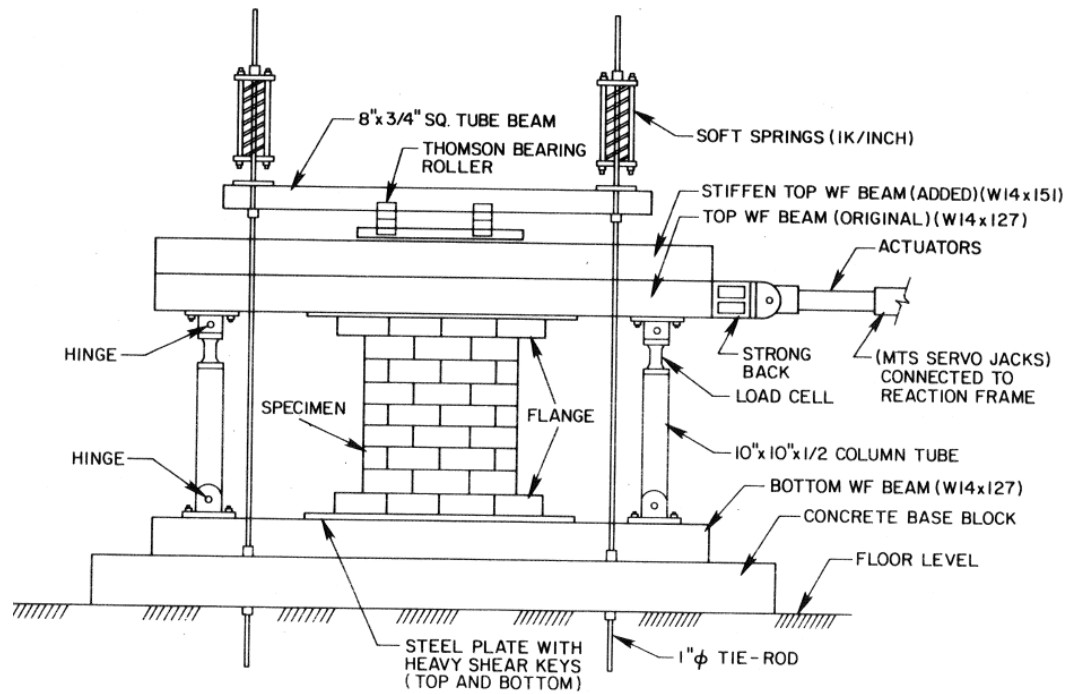


Figure 2.1 (Hidalgo et al, 1978) Test Configuration for EERC Tests, Volumes I-III

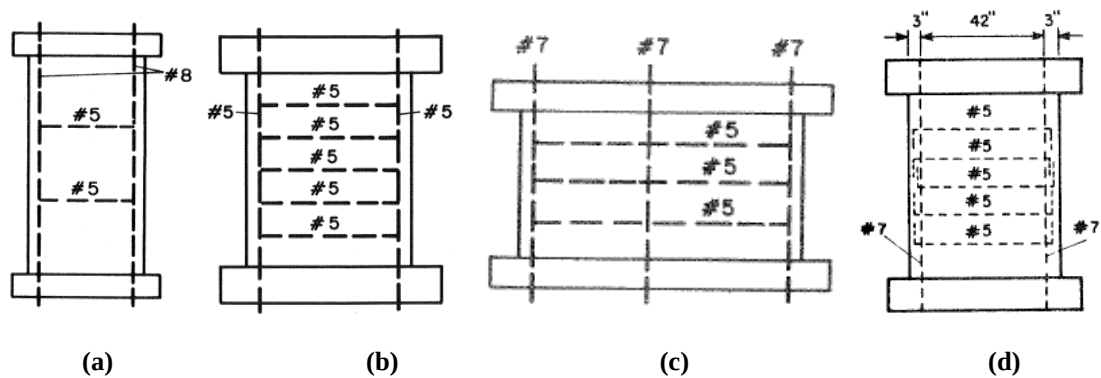


Figure 2.2 EERC Typical Wall Configurations (a) Vol. I (b) Vol. II (c) Vol. III (d) Vol. IV

Volume I, by Hidalgo, et al. (1978) is comprised of the results of 14 clay wall tests in which 10 fully grouted and 3 partially grouted walls failed in shear. Wall $\frac{\text{height}}{\text{length}}$ ratios were around 2.0 and were constructed out of 8 in. hollow brick with a few double-wythe specimens being constructed out of 4 in. solid brick. The walls utilized gr. 40 reinforcement.

Walls in Volumes I-III had applied axial loads, but due to the nature of the testing configuration, had extra axial load introduced during testing. Fully grouted walls in this series generally had ultimate strengths that coincided with a diagonal tension failure, with partially grouted walls exhibiting similar behavior but with lower overall strengths. Additionally, it was stated that the vertical reinforcement in the partially grouted walls either barely reached yield, or did not yield at all. Ultimate strengths were noted to not have increased when a horizontal reinforcement ratio above 0.30% was utilized.

Volume II, by Chen et al. (1978) is comprised of 31 masonry pier tests, in which all walls failed in shear. All walls had $\frac{height}{length}$ ratios of 1.0, with 20 being constructed out of either 8 in. hollow clay blocks or 4 in. brick (double-wythe), with the remaining 11 walls being constructed out of 8 in. concrete masonry units. Out of the 20 clay walls, 4 were partially grouted, and out of the 11 CMU walls, 4 were partially grouted as well. A combination of gr. 40 and 60 steel was used for vertical and horizontal reinforcement.

Fully grouted walls in this series generally had ultimate strengths that coincided with a diagonal tension failure, with partially grouted walls exhibiting similar behavior but with lower overall strengths. It was stated that the vertical reinforcement in the partially grouted walls did not yield at all. It was also noted that ultimate strengths were not proportional to the increase in horizontal reinforcement, but showed rather sporadic values of ultimate strength.

Volume III, by Hidalgo et al. (1979), is comprised of 18 clay and concrete masonry pier tests, in which 4 clay walls and 4 CMU walls failed in shear. All walls had a $\frac{height}{length}$ ratio of approximately 0.5, and were constructed with 8 in. CMU, 8 in. hollow clay block, and 4 in. solid brick (double-wythe). All walls were fully grouted and utilized gr. 60 reinforcement.

Of those walls that failed in shear, the CMU walls had characteristic diagonal cracking with no singular crack crossing the entire width of the pier, while the clay walls included cracks that generally crossed the width of the pier, but with a horizontal segment above midheight of the wall. It was stated that the lateral load strength of the piers was carried by struts that formed between these cracks.

Volume IV, by Sveinsson et al. (1985) is comprised of 32 masonry pier tests in which 10 clay walls and 10 CMU walls failed in shear. All walls had a $\frac{\text{height}}{\text{length}}$ ratio of 1.0 and were constructed with both 6 in. and 8 in. hollow clay and CMU, and 4 in. solid clay bricks (double-wythe). All walls were fully grouted and utilized gr. 40 and gr. 60 deformed bar reinforcement, as well as wire-type bed joint reinforcement.

Unlike the first three volumes of this research, the axial load was maintained at a constant value using the test configuration in Figure 2.3.

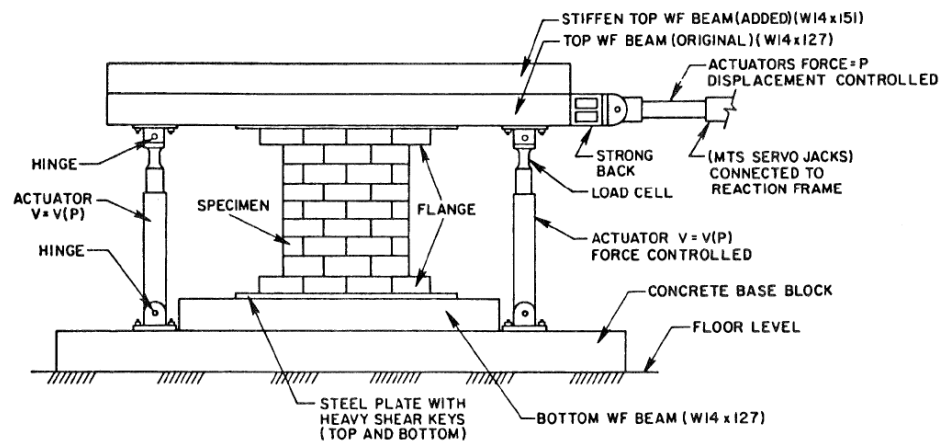


Figure 2.3 (Sveinsson et al, 1985)

Test Configuration for EERC Tests, Volume IV

The specimens failing in shear exhibited both ductile and brittle shear failures, with the ductile failure being characterized by the spreading of diagonal cracks in the wall with minimal crack opening, whereas brittle failures contained very few diagonal cracks achieving wider openings at failure. It was stated that the walls with wire bed joint reinforcement achieved a more desirable ductility than those that only contained deformed bar reinforcement.

Matsumura (1986) tested roughly 80 shear walls of various forms comprised of both 6 in. and 8 in. nominal concrete block, in which most failed in shear. The majority of the specimens were tested in wall-type loading with the top restrained from rotation, so as to yield the inflection point at the middle of the wall (other tests were beam-type loading, and are not included in this analysis). Information about some parameters such as reinforcement, spacing, and other details are lacking and must be inferred based upon similar examples from other researchers in Appendix A. Testing configuration of specimens is shown in Figure 2.4.

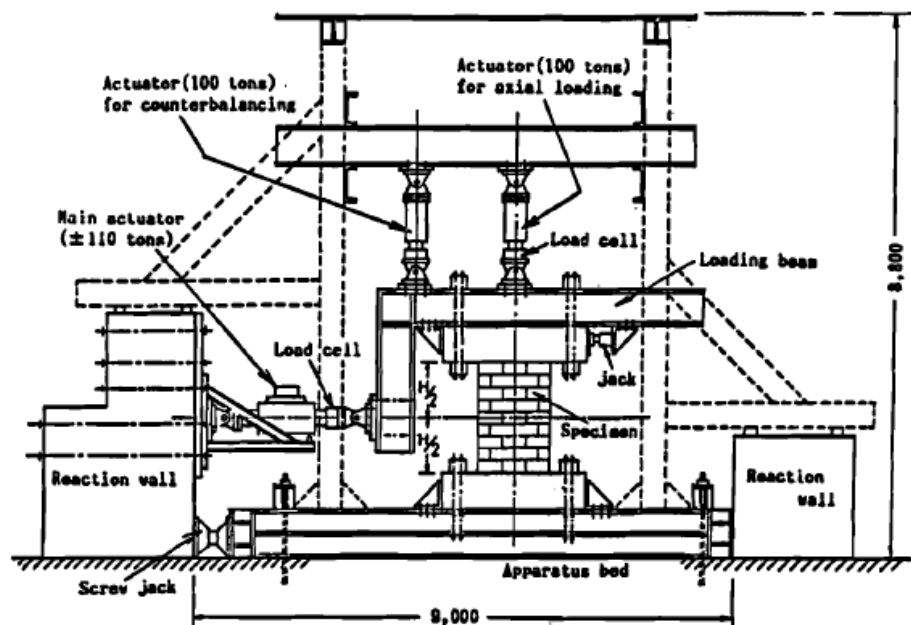


Figure 2.4 (Matsumura, 1986) Testing Configuration

Matsumura stated that shear strength increases with an increase in the masonry compressive strength, roughly proportional to $\sqrt{f'_m}$ as shown in Figure 2.5, however it seems as if there are not enough data points, nor a tight enough correlation to indicate this to a definite degree. It appears that from the data in Figure 2.5, a roughly linear relationship results.

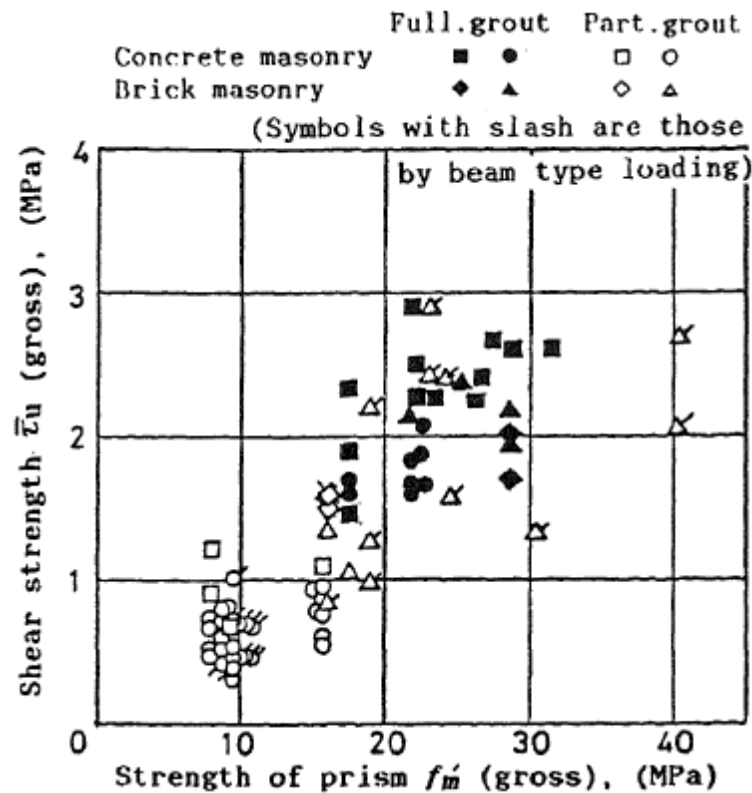


Figure 2.5 (Matsumura, 1986) Shear Strength vs. Prism Strength

It was shown that shear strength increases with increasing horizontal reinforcement ratio, and that shear strength decreases with increasing aspect ratio. Matsumura also correlated axial stress with shear strength by showing that increase of shear strength with respect to an increase in axial load is roughly linear, and equal to approximately 20% of the axial load in both types of walls, as shown in Figure 2.6 and Equation 2.1.

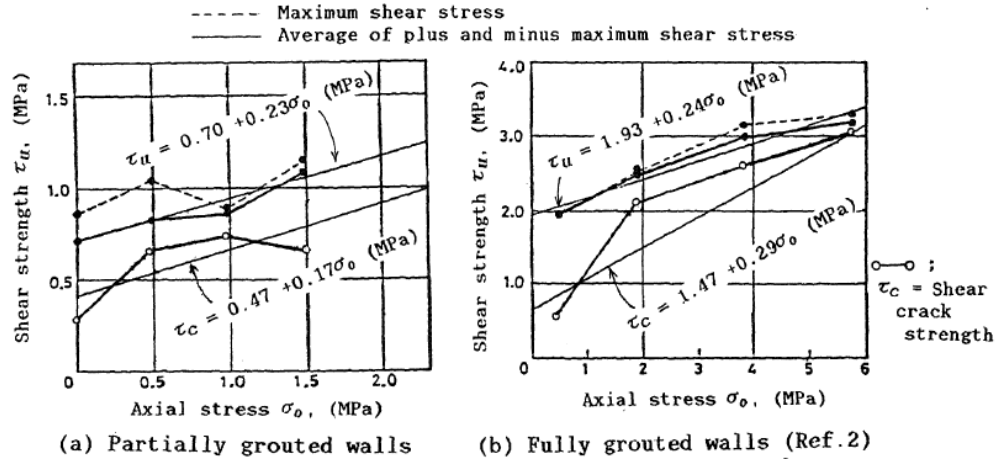


Figure 2.6 (Matsumura, 1988) Shear Strength vs. Axial Stress

$$\tau_u = \tau_{ua} + 0.2\sigma_0 \quad (2.1)$$

Where,

τ_u = shear strength

τ_{ua} = shear strength without axial stress

σ_0 = axial stress (gross area)

Other findings include the lessened effect of horizontal reinforcement in a partially grouted wall, potentially due to less of a “grout-confining” effect in partially grouted walls versus fully grouted ones. In his resulting equation for the overall shear strength of masonry walls, he makes no distinction between brick and concrete masonry except for partially grouted specimens, however, factors are introduced that reduce the strength contributions of masonry and horizontal reinforcement in a partially grouted wall to roughly 60% each of their equivalent fully grouted CMU counterparts. Equations are shown below in accordance with the gross area of

members (stress in MPa and length in meters), with portions addressing the contributions from the masonry, shear reinforcement, and axial load.

$$V_u = \left\{ \begin{array}{l} k_u k_p \left(\frac{0.76}{\frac{h}{d} + 0.7} + 0.012 \right) \sqrt{f'_{m(g)}} + \\ 0.18 \gamma \delta \sqrt{p_h h^{\sigma_y} f'_{m(g)} + 0.2 \sigma_{0(g)}} \end{array} \right\} 10^3 j \quad (2.2)$$

Where,

$k_u = 1.0$ for fully grouted, 0.8 for partially grouted brick, 0.64 for partially grouted concrete
 $k_p = 1.16 P_t^{0.3}$ where P_t is the flexural reinforcement ratio ($\frac{\alpha_t}{t d}$ where α_t is the cross-sectional area of a flexural reinforcing bar) (P_t is listed as % in Matsumura (1986), but it is believed to be a decimal value instead)

h = Height of masonry portion of a wall (mm)

d = effective width of a wall (distance from extreme compression fiber to centroid of flexural tension reinforcement) (mm)

γ = factor addressing grout confinement—1.0 for brick and fully grouted concrete, 0.6 for partially grouted concrete

δ = factor addressing loading method—1.0 for loading which yields inflection point at mid-height, 0.6 for cantilever type

p_h = horizontal reinforcement ratio

h^{σ_y} = yield stress of shear reinforcement (MPa)

σ_0 = axial stress (gross) (MPa)

t = thickness (mm)

$j = \frac{7}{8} d$ (mm)

Fattal (1993) worked to refine Matsumura's equation to better fit available data on shear walls—72 partially grouted specimens. Modifications were made such that the equation proposed by Fattal predicted strengths more consistently than Matsumura's, however, the range of scatter of the strength ratios was shown to be 41-146%, with 68% (50 out of 72 tests) falling within $\pm 20\%$ of measured strength, with the deviation and coefficient of variation being 0.17 and

0.21, respectively. This is in comparison to Matsumura's predicted strengths varying from 23% to 180% of measured strengths while the predicted strength of 39 specimens (54% of the total) exceeded the range of $\pm 20\%$ of test strength. This shows a deviation of 0.25 and a variation of 0.31. By inspection of Equation 2.3, the strength contributions of masonry and reinforcement of partially grouted concrete masonry walls was reduced to approximately 50% of their fully grouted CMU counterparts. Equation 2.3 is in SI units.

$$V_p = k_o k_u \left[\left(\frac{0.5}{r + 0.8} \right) + 0.18 \right] f_m^{0.5} f_{yv}^{0.5} p_v^{0.7} + k_o (0.011) \gamma \delta f_{yh} p_h^{0.31} + k_o (0.012) f_m + 0.2 q \quad (2.3)$$

Where,

$k_o = 0.8$ for partially grouted, 1.0 for fully grouted

$r =$ aspect ratio, $\frac{h}{l}$

$f_m =$ compressive strength from prism tests

$f_{yv} =$ average yield strength of vertical reinforcement

$p_v =$ vertical reinforcement ratio

$f_{yh} =$ yield strength of horizontal reinforcement

$q =$ nominal axial stress on the wall

Yancey and Scribner (1989) tested 13 partially grouted 8 in. nominal CMU shear walls (10 of which are suitable for this study) to determine the effect of various amounts and distribution of horizontal reinforcement on the in-plane shear resistance of concrete masonry walls. It was concluded that small amounts of horizontal reinforcement help to increase post-cracking strength, but that in-plane shear strength does not increase proportionately with increasing amounts of horizontal reinforcement. Additionally, it was shown that bed joint reinforcement placed in every other course is as effective in increasing in-plane shear strength as

when placed in every course. The report on these walls contained no information about the masonry prism compressive strength (f'_m), but this information was found in Fattal's report (1993). Interestingly, these compressive strengths were very low—on the order of 800-1200 psi and the walls did not contain any vertical reinforcement. The walls seem to be tested under restrained top rotation, and their configuration is shown in Figure 2.7.

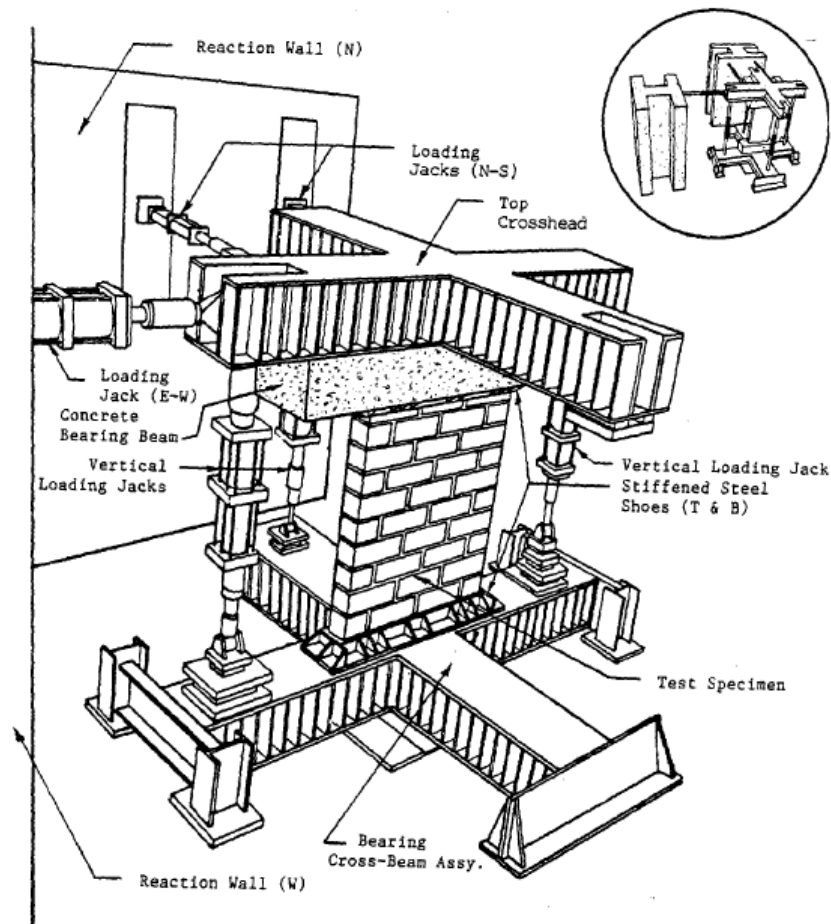


Figure 2.7 (Yancey and Scribner, 1989) Test Configuration

Shing et al. (1990), tested 22 square masonry wall panels and reported the findings relative to then-current design provisions set forth in the 1988 Uniform Building Code. These

tests confirm the predominantly brittle behavior of shear failure in masonry walls, and it is thereby stated that at the printing of the article, there is no “effective theoretical formulation” that can accurately predict the shear strength that is dominated by diagonal cracking in the wall panel. The main variables focused on in the experiment were the amount of reinforcing (both vertical and horizontal), applied axial stress, and the type of masonry units that make up the wall (either hollow-core concrete block or clay bricks). All walls were fully grouted and utilized nominal 6 in. block and brick and grade 60 or grade 40 deformed reinforcing bar, with traditional methods of anchoring and tying reinforcement in a masonry wall—180 degree hooks around the end for horizontal reinforcement, and horizontal reinforcement placed in bond beams at predetermined spacings, with varying levels of axial stress applied at the top of the wall. It appears that these walls were tested as cantilevers and their typical configuration is shown in Figure 2.9.

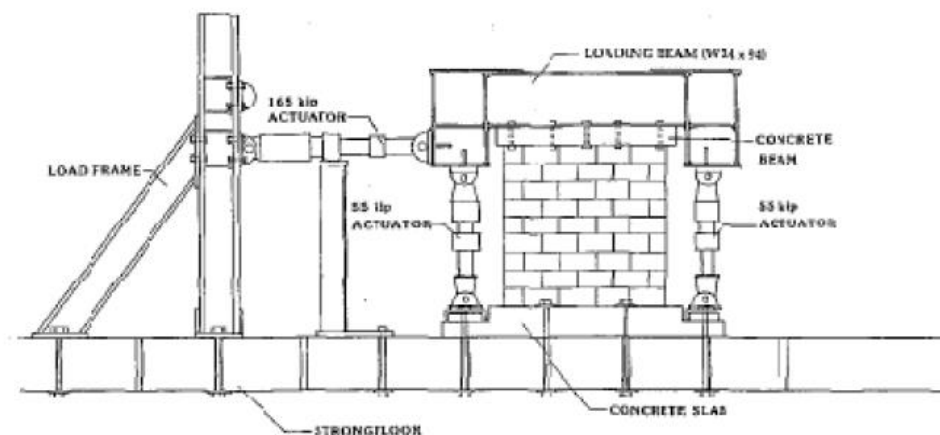


Figure 2.8 (Shing et al, 1990) Test Configuration

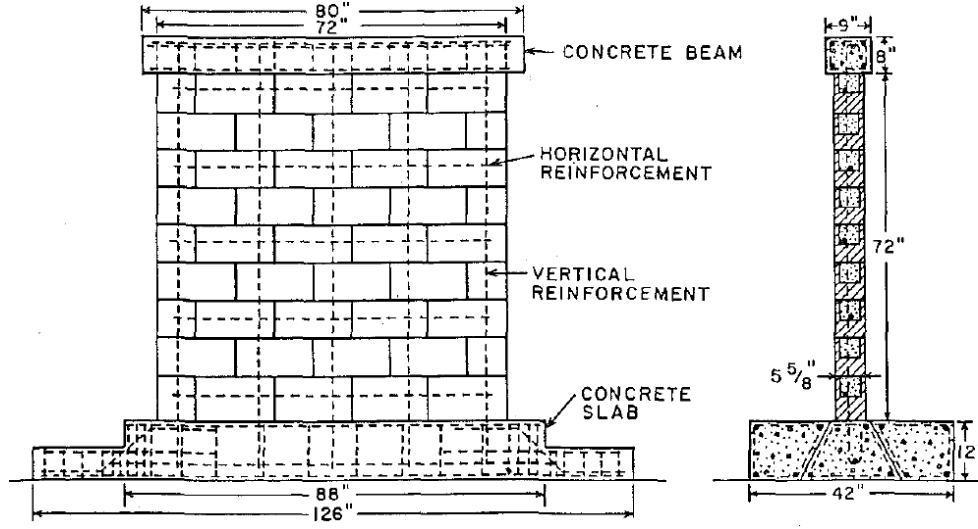


Figure 2.9 (Shing et al, 1990) Wall Configuration

Approximately 12 of the 22 specimens failed in shear (2 specimens being constructed with clay masonry), and are therefore suitable for analysis in the current research. One important finding of Shing et al., and similar to that of Yancey and Scribner (1989) was that increasing the amount the horizontal reinforcement is not a linear relationship when compared to strength—doubling the shear reinforcement does not necessarily lead to a substantial increase in lateral load capacity. It is also pointed out by Shing that specimens with increased vertical reinforcement percentages exhibited increased lateral load capacity after diagonal cracking. Shing goes on to state that the shear strength of masonry walls is composed of dowel action of the vertical steel, aggregate interlock forces, and truss action due to the layout of steel in the wall—but that because of the complexity of these mechanisms, the then-current code models are over-simplistic.

Shing et al. plotted Figure 2.10 showing the relationship of $\frac{\tau_{cr}}{\sqrt{f'_m}}$ where τ_{cr} is the shear stress at the first major diagonal crack, to the applied vertical stress. He proposed that there is no

correlation between the cracking strength of the masonry and the amount of reinforcement present.

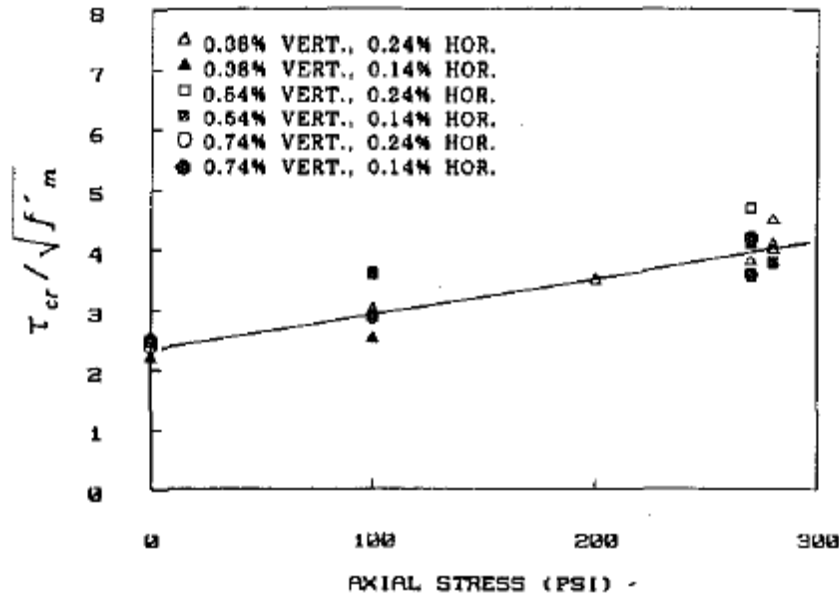


Figure 2.10 (Shing et al, 1990) Shear Stress at First Diagonal Crack vs. Axial Stress for Various Reinforcement Ratios

Equation 2.4 was proposed to account for the contribution of the shear reinforcement, and to consider that only the interior reinforcement is developed enough to participate in transferring tensile stresses (based on a 45 degree crack in the wall panel):

$$V_s = \left(\frac{l-2d'}{s} - 1 \right) A_h f_y \quad (2.4)$$

Where,

l = length of wall

d' = distance from outer edge of wall to nearest vertical bar (jamb steel)

s = vertical spacing of horizontal reinforcement

A_h = area of horizontal reinforcement

f_y = yield stress of horizontal reinforcement

Shultz et al. (1998) tested six partially grouted masonry walls with 8 in. CMU with various $\frac{\text{height}}{\text{length}}$ ratios ranging from 0.5 to 1.0 that included various reinforcement ratios consisting of welded wire ladder-type bed joint reinforcement. It appears that the walls were restrained against rotation at the top. Testing configuration is shown in Figure 2.11.

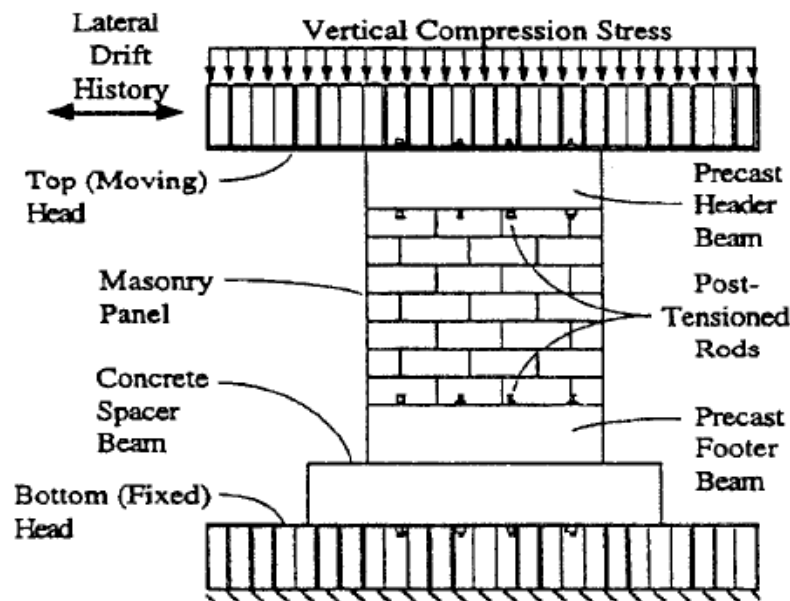


Figure 2.11 (Schultz, et al, 1998) Test Configuration

The object of the experiment was to test how applicable the use of bed joint reinforcement would be in partially grouted masonry walls, which was confirmed by his tests. The specimens were provided with enough flexural vertical steel in the outer cells to cause them to fail in shear. It was found that the lateral load resistance of the walls decreased after the “first major event” (first signs of diagonal cracking, in this case) with increasing aspect ratio (slender walls), and that a near 45 degree diagonal crack occurred for walls that approached a $\frac{\text{height}}{\text{length}}$ ratio of 1.0. It was stated that the increasing horizontal reinforcement ratios were more crucial to

walls that had an increased aspect ratio. Of important note, only the outer vertical cells contained flexural reinforcement (2 #6 Gr. 60 reinforcing bars in each cell) and there was no bond beam at the top of the wall (a pre-cast header beam was attached instead).

In a later study, Elmapruk and ElGawady (2009) compiled and compared 90 partially grouted wall specimens from various researchers and studied the effects of horizontal reinforcement ratio, axial stress, nominal shear strength vs. experimental strength, and accuracy of current shear design equations when applied to partially grouted masonry walls. The researchers state that current shear design equations in the TMS-402 code were based on research involving fully grouted walls—not partially grouted ones. Both net area and face shell area of a masonry unit were utilized in analyzing the TMS-402 shear equations, and it was found that 60 out the 90 specimens had strengths that were overestimated by as much as 100% using the net area of a block, with only 30% of the data falling within 20% of the experimental strength. The current TMS-402 shear equations are shown below, and are discussed in further detail in Chapter 3:

$$V_n = V_{nm} + V_{ns}$$

$$V_{nm} = \left[4.0 - 1.75 \left(\frac{M_u}{V_u d_v} \right) \right] A_n \sqrt{f'_m} + 0.25 P_u$$

$$V_{ns} = 0.5 \left(\frac{A_v}{s} \right) f_y d_v$$

$$V_n \leq 6 A_n \sqrt{f'_m} \quad \text{for } \frac{M_u}{V_u d_v} \leq 0.25$$

$$V_n \leq 4 A_n \sqrt{f'_m} \quad \text{for } \frac{M_u}{V_u d_v} \geq 1.00$$

Where,

V_n = the nominal shear strength provided by the shear element

V_{ns} = Nominal shear strength provided by shear reinforcement

V_{nm} = Nominal shear strength provided by the masonry

f'_m = Compressive strength of the masonry

V_u = Factored shear force

M_u = Factored moment

d_v = shear depth

A_n = Net area in shear

P_u = Factored Axial Load

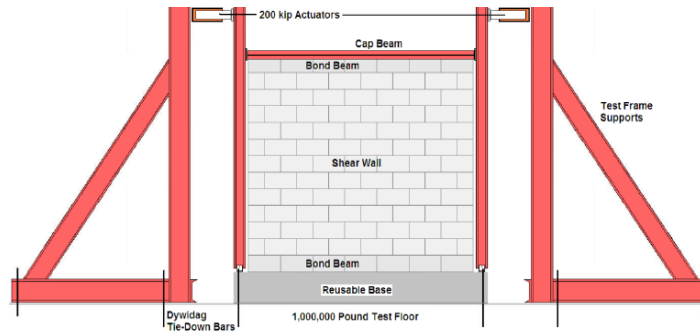
and, $\frac{M_u}{V_u d_v}$ is generally taken as the height over the shear depth $\left(\frac{h}{d_v}\right)$ for a critical load location at the top course of masonry. Using the face shell area as the net area under shear, there were 32 overpredictions. It was also stated that with increasing horizontal reinforcement ratio, the ratio of experimental to predicted strength changed randomly, suggesting revisions need to be made to this part of the equation, and that it overestimates the contribution of shear reinforcement. Sarhat and Sherwood (2011) studied the variations in four different masonry codes on 112 beam shear tests—CSA S304.1-04 (Canada), ACI 530-08 (US), AS 3700-2001 (Australia), and BS 5628-2:2005 (UK). Average values of $\frac{V_{experimental}}{V_{predicted}}$ ranged from 1.05 to 1.54. Sarhat and Sherwood state that most of the codes analyzed use empirical data to derive an expression for shear strength.

Baenzinger and Porter (2010) analyzed the validity and advantages/disadvantages in using bed joint reinforcement vs. traditional deformed bars as a primary means of horizontal shear reinforcement. Their study consisted of eight partially grouted walls and 2 fully grouted walls constructed out of nominal 8 in. CMU, using varying amounts of deformed bar or bed joint reinforcement with a height of 8 ft.-8in. and aspect ratios of 0.93 and 0.62, without any application of vertical stress. In the conclusion of the tests, the experimental shear strength

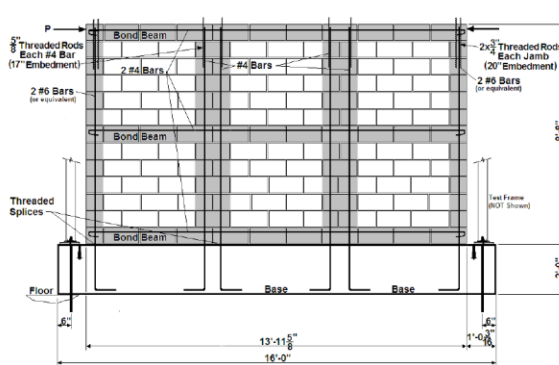
values were compared with the shear strength as calculated by the TMS-402 equations, and it was found that the experimental shear strength exceeded the TMS-402 shear strength in all cases. This leads one to question the repeatability of the TMS-402 equations in regards to partially grouted walls, as in previous work the opposite was found. In addition, the authors state the following:

“The presence of larger quantities of horizontal reinforcement...may not result in a stronger wall if the masonry is not proportionately stronger.....Increasing reinforcement beyond the area of reinforcement required to transfer loads across the cracks may serve only to decrease the reinforcement strains and the apparent factor of safety, because the additional reinforcement is expected to add to capacity but does not. Masonry strength will be the more significant factor in the strength of the overall wall capacity where the aspect ratio is lower...In the higher aspect ratio, shorter walls, the reinforcement plays a more significant role, but the limiting contribution of the reinforcement must still be based on the remaining wall integrity.”

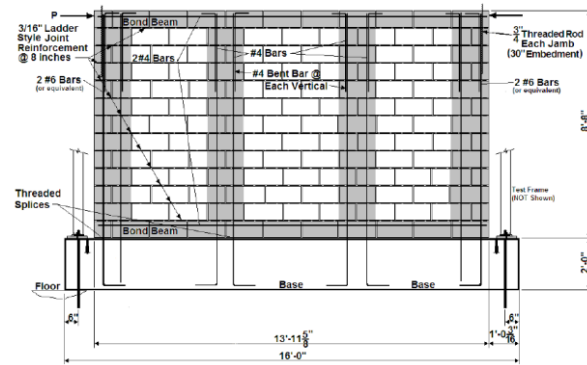
This statement reinforces the notion that increases in shear reinforcement do not constitute a proportional increase in wall strength. Their cantilever wall testing configuration and representations of general wall and reinforcement configuration are given in Figure 2.12.



a)



(b)



(c)

Figure 2.12 (Baenzinger and Porter, 2010) a) Loading Frame, b) Bond Beams and Primary Shear Reinforcement, c) Ladder-type Bed Joint Reinforcement (in every course)

Voon (2007) and Voon and Ingham (2006) conducted research to investigate the components of concrete masonry shear strength in a two part study—one on fully grouted walls and the other on partially grouted perforated walls. It was verified through tests that horizontal shear reinforcement and axial load added shear resistance to masonry shear walls, and that as wall aspect ratio increased, shear strength decreased. The non-perforated walls were used for analysis in this research. The test set included ten cantilever masonry shear walls with varying levels of reinforcement ratio, axial stress, and wall aspect ratios with a focus on walls with relatively low axial stress (approximately 70 psi or less) and low shear reinforcement ratios

(around 0.06% or less). Two out of the ten walls were only partially grout-filled and included no horizontal reinforcement. All units were constructed out of 6 in. CMU and had $\frac{height}{length}$ ratios of 1.0, 2.0, or 0.5. All walls failed in shear except for two, which failed in flexure. It was stated that axial compression increases the amount of lateral load that can be applied to a wall by delaying the initial cracking, although this becomes a more brittle type of failure. General wall testing configuration is shown in Figure 2.13.

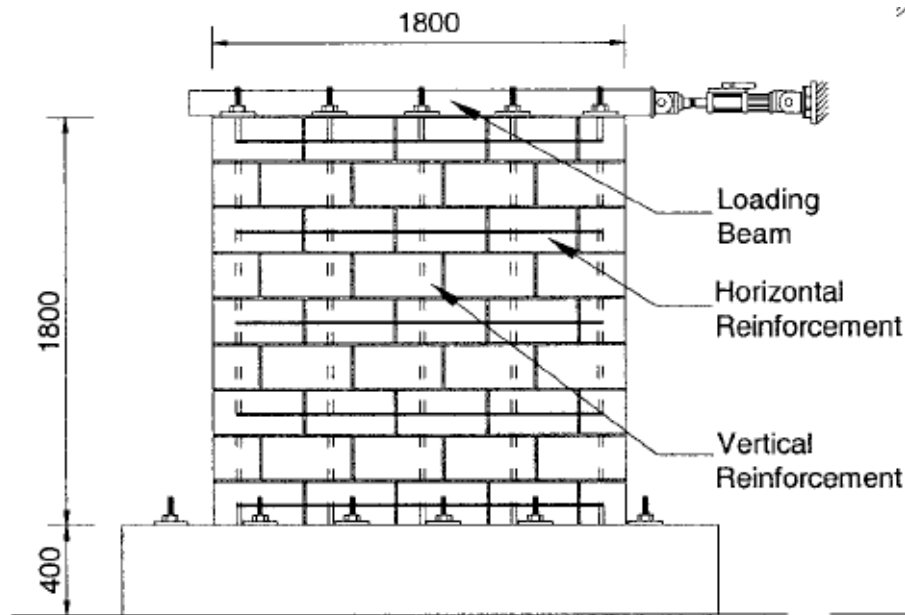


Figure 2.13 (Voon and Ingham, 2006) Test Configuration

Davis (2008) conducted a comprehensive study of fully grouted masonry shear walls from available data and compared the accuracy of various code provisions and proposed equations to experimental values of the in-plane shear strength. In addition, she isolated the effects of various wall parameters such as masonry compressive strength, amount of shear reinforcement, level of axial compressive stress, amount of vertical reinforcement, displacement ductility, and wall aspect ratio as predicted by the various codes and equations. Code equations

analyzed include the TMS-402 (2008) provisions for both allowable stress and strength design, Uniform Building Code (1997) provisions for both working stress and strength design, the New Zealand Standard 4230 (2004), and the Canadian Standards Association S304.1(2004).

Statistics were calculated and included in the table in Figure 2.14, where $\frac{V_{test}}{V_n}$ is consistent with the ratio used later in this dissertation designated as $\frac{V_{experimental}}{V_{predicted}}$.

Shear Equation	V_{test}/V_n					
	Mean	Standard Deviation	Coeff. Of Variation	Minimum Value	Maximum Value	5th Percentile
MSJC SD/NEHRP	1.16	0.17	0.15	0.77	1.55	0.88
MSJC ASD V_m	8.51	2.09	0.25	3.83	13.65	5.08
MSJC ASD V_s	9.62	4.59	0.48	3.99	24.71	2.07
UBC	1.51	0.37	0.25	0.86	3.11	0.90
CSA	1.50	0.22	0.15	0.96	1.95	1.16
Shing et al.	1.12	0.24	0.21	0.54	1.66	0.72
NZS	1.38	0.26	0.19	0.85	2.05	1.01
Anderson and Priestley	1.35	0.40	0.30	0.82	2.28	0.69

Figure 2.14 (Davis, 2008) Comparison Statistics

Effectiveness of various parameters as predicted by each equation were evaluated, but presented in Figure 2.15 are only the comparisons for the TMS-402 strength design provisions as applied to the various researchers Davis investigated.

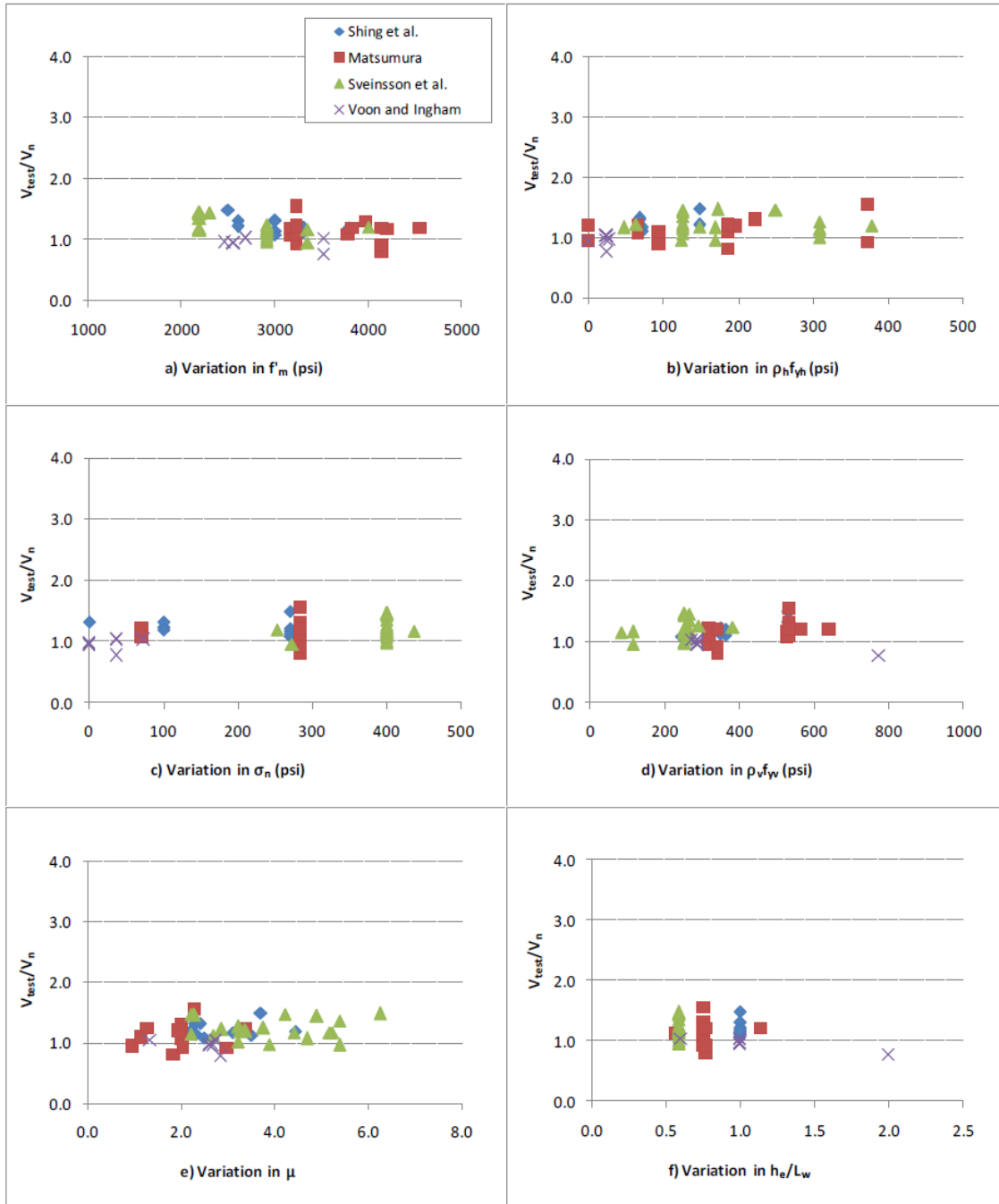


Figure 2.15 (Davis, 2008) Parameter Comparisons for the TMS-402 Equations

It was determined by Davis that the TMS-402 SD provisions were the best overall predictor of shear strength of fully grouted masonry wall specimens evaluated in the given data sets, as it performed the best in four out the six categories presented in Figure 2.15 with the least

scatter and bias among the analyzed equations. It was determined that the TMS-402 SD provisions reasonably account for the varying amounts of shear reinforcement, masonry prism compressive strength, axial loading, and vertical reinforcement (even though there is no term in the equations for this type of reinforcement) for the specimens analyzed. Additionally, the effect of aspect ratio was predicted well, although with a slight negative bias. Davis goes on to state that similar analysis for partially grouted walls needs to be conducted, as this study only encompassed fully grouted walls.

Minaie et al. (2010) tested four partially grouted 8 in. CMU masonry shear walls and along with analysis of available partially grouted wall data from other researchers, came to the conclusion that partially grouted wall strengths as predicted by the TMS-402 were generally unconservative, and become more so as the shear wall area increases, the aspect ratio ($\frac{\text{height}}{\text{length}}$) of the shear wall decreases below 1.0, and the spacing of vertical and horizontal reinforcement increases. The four walls tested by this group were essentially identical except for the boundary conditions—two walls were tested as cantilevers, and the other two were restrained against top rotation. This testing setup is shown in Figure 2.16.

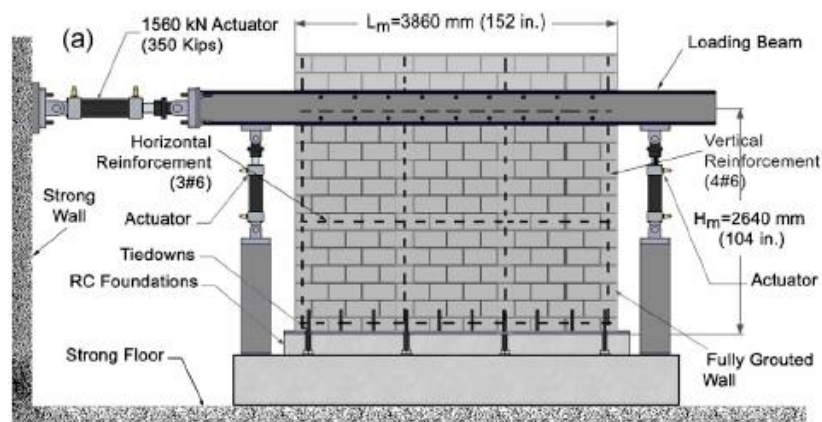


Figure 2.16 (Miniaie et al, 2010) Test Configuration

Vaughan (2010) conducted research to evaluate the performance of masonry walls compliant to the TMS-402 seismic provisions. He gathered data from various researchers, one of which—Eikanas—is used in this study. According to Vaughan (Eikanas' data could not be located elsewhere), Eikanas studied the effects of varying wall aspect ratio and vertical reinforcement on wall behavior. Walls in this study were fully grouted, nominal 8 in. CMU and were tested as cantilevers with constant axial stress. Two of these walls failed in shear, and are therefore used in this research.

Nolph (2010) and Nolph and ElGawady (2012) researched five 8 in. CMU partially grouted masonry shear walls which were essentially 8 feet square with varying vertical reinforcement spacing and a consistent horizontal reinforcement spacing of approximately 48 inches. Axial load was kept at a constant 11.1 kips for all walls. Various amounts of horizontal reinforcement were tested from 0.085% to 0.169%, and vertical reinforcement configurations for each wall type are shown in Figure 2.17.

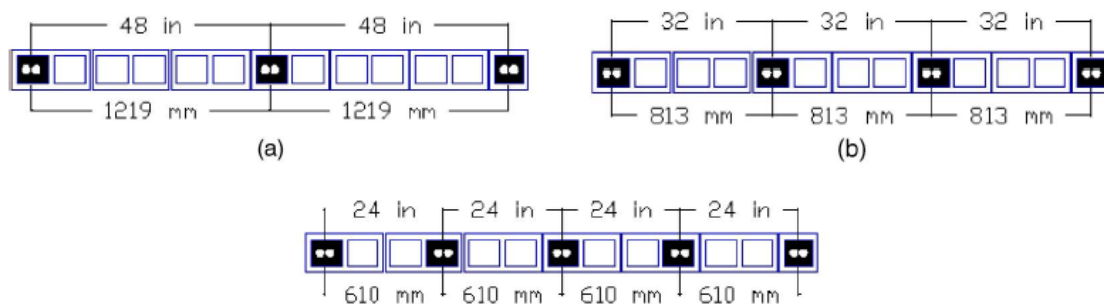


Figure 2.17 (Nolph and ElGawady, 2012) Vertical Reinforcement Spacing (Plan View)

It was concluded that there appeared to be a maximum value of horizontal shear reinforcement after which no additional benefit is gained in horizontal shear capacity—approximately 0.11% for the horizontal reinforcement ratio in this study.

Figure 2.18 shows the comparison between TMS-402 equations and actual failure loads for the three specimens that had all parameters constant except for horizontal reinforcement ratio. The TMS-402 over-predicted the strength of all three specimens.

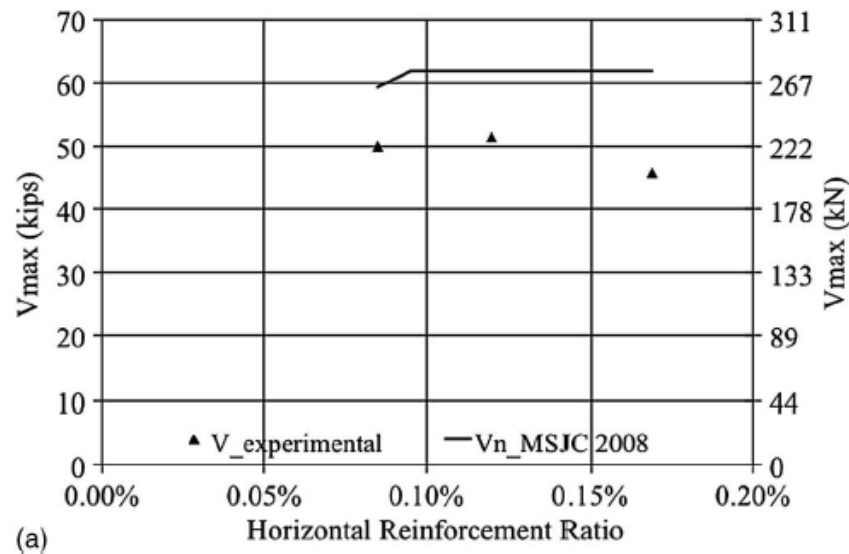


Figure 2.18 (Nolph and ElGawady, 2012) Failure Load vs. Horizontal Reinforcement Ratio

Like Elmapruk et al. (2009) and Sarhat et al (2011), the TMS-402 equations for shear were compared to the experimental values, and it was concluded that the shear strength of specimens with greater than a 32 in. vertical reinforcement spacing and specimens with increased horizontal reinforcement ratios were overestimated. Strut and tie models were said to have accurately predicted the failure within 16% of all walls in this study, with examples given in Nolph (2010). Comparison of $\frac{V_{predicted}}{V_{experimental}}$ (usually the inverse of this ratio is presented) between available codes and equations, including strut and tie, are shown in Figure 2.19 for the five walls that were tested.

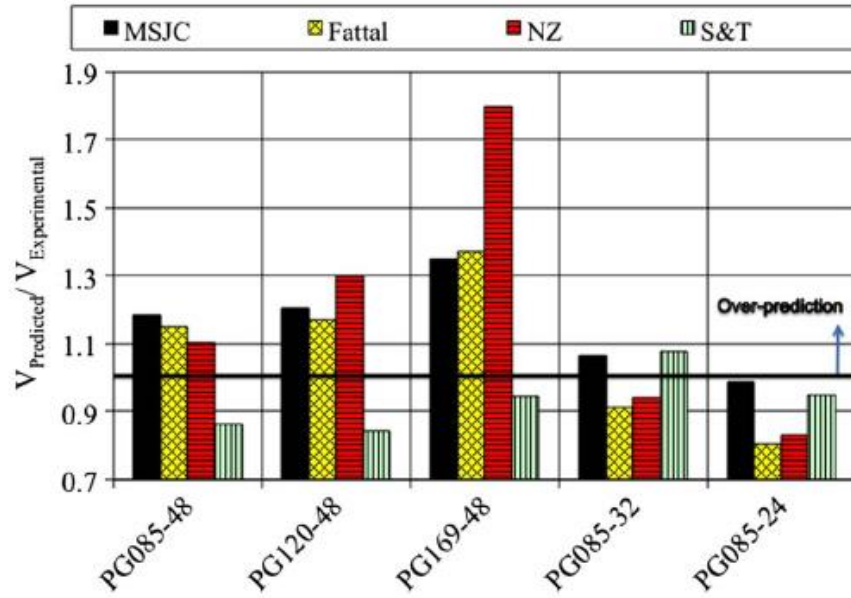


Figure 2.19 (Nolph and ElGawady, 2012) Accuracy of Equations

The cantilever wall testing setup and a representation of a typical wall is shown in Figure 2.20.

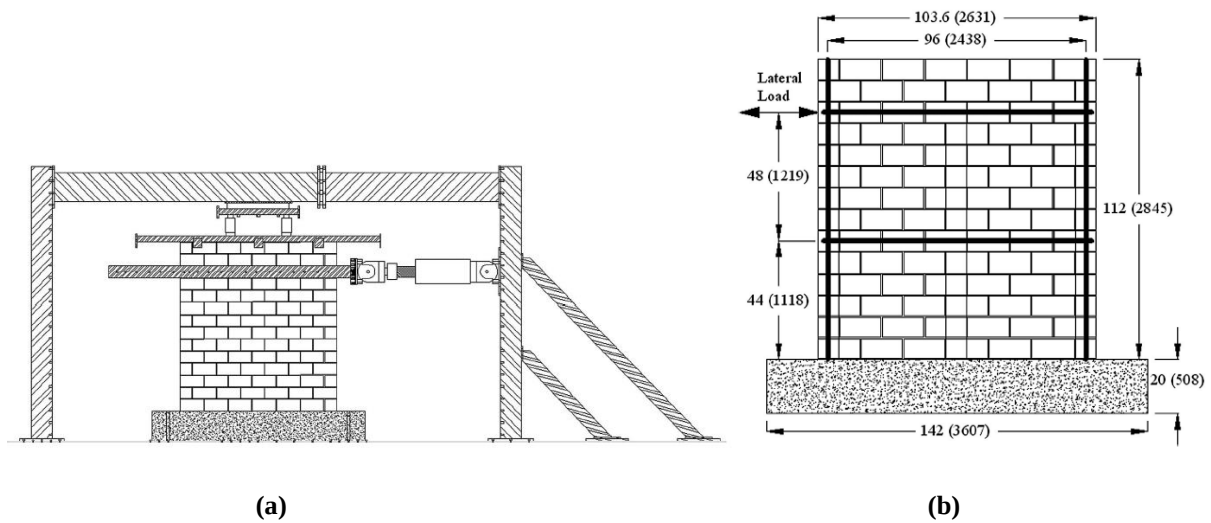


Figure 2.20 (Nolph and ElGawady, 2012) (a) Test Configuration and (b) Typical Wall

In addition to the researchers presented previously which dealt primarily with concrete and clay masonry units, there exists a small amount of data concerning Autoclaved-Aerated Concrete masonry failing in shear. This data, along with the respective researchers, will be discussed in a later chapter of this dissertation. All walls parameters from the data sets presented in this section are given in Appendix A.

Chapter 3—TMS-402 Equations and Issues

3.1 TMS-402 Shear Equations and Provisions

The leading masonry requirements for the United States are specified by the TMS-402 Building Code Requirements for Masonry Structures. This code covers design expressions for shear for various forms of commonly used masonry such as clay, concrete, and AAC in both strength design and allowable stress design (ASD). The ASD provisions are similar to the strength design provisions, but divided by a factor of safety of 2.0. The strength design equations and methods were of importance in this research, therefore, only those equations relating to this method of design are discussed.

Prescriptive provisions for seismic design are given in section 1.17 of the TMS-402 code, for both traditional masonry and AAC masonry. For a more detailed description of reinforcement requirements, see TMS-402 (2011).

Common reinforcement sizes and spacing differ according to the region of interest. For example, shear walls built in the eastern United States may have vertical reinforcement of #4 bars every 48 in. and horizontal reinforcement (shear reinforcement) of #4 bars or bed joint reinforcement spaced only to meet the minimum required code value of 10 feet, whereas in the western United States, common reinforcement may consist of #5 vertical bars spaced 16-32 in. and #5 horizontal bars spaced at 16-32 in.

All equations presented in this section are in English units unless otherwise stated. The nominal shear strength, V_n , of unreinforced masonry laid in running bond is determined based upon the smallest of the following:

$$3.8 A_n \sqrt{f'_m} \quad (3.1)$$

$$300 A_n \quad (3.2)$$

$$56 A_n + 0.45 N_u \text{ (not solidly grouted)} \quad (3.3)$$

$$90 A_n + 0.45 N_u \text{ (solidly grouted)} \quad (3.4)$$

Where,

A_n = net cross-sectional area of a member (in²)

N_u = factored compressive force acting normal to the shear surface (lb)

f'_m = masonry compressive strength (psi)

The final two equations follow a Mohr-Coulomb failure criterion for interface sliding, where an initial bond is considered along with a friction term.

Reinforced masonry strength design equations are as follows:

$$V_n = V_{nm} + V_{ns} \quad (3.5)$$

$$V_{nm} = \left[4.0 - 1.75 \left(\frac{M_u}{V_u d_v} \right) \right] A_{nv} \sqrt{f'_m} + 0.25 P_u \quad (3.6)$$

$$V_{ns} = 0.5 \left(\frac{A_v}{s} \right) f_y d_v \quad (3.7)$$

$$V_n \leq 6 A_{nv} \sqrt{f'_m} \quad \text{for} \quad \frac{M_u}{V_u d_v} \leq 0.25 \quad (3.8)$$

$$V_n \leq 4 A_{nv} \sqrt{f'_m} \quad \text{for} \quad \frac{M_u}{V_u d_v} \geq 1.00 \quad (3.9)$$

Where,

V_n = the nominal shear strength provided by the shear element (lb)

V_{ns} = Nominal shear strength provided by shear reinforcement (lb)

V_{nm} = Nominal shear strength provided by the masonry (lb)

f'_m = Compressive strength of the masonry (psi)

$\sqrt{f'_m}$ = units of psi

V_u = Factored shear force (lb)

M_u = Factored moment (lb-in)

d_v = shear depth (in.)

A_v = Area of shear reinforcement (in²)

A_{nv} = Net area in shear (in²)

P_u = Factored Axial Load (lb)

Equation 3.6 follows a Mohr-Coulomb failure criterion with an empirical diagonal tension strength. These equations apply for both clay and concrete masonry for any shear element in a masonry structure, including both beams and lintels over openings and for shear walls.

Nominal shear strength of AAC masonry walls is computed as in Equation 3.10-3.17, with limits based upon equations in the TMS-402 code.

$$V_n = V_{nAAC} + V_{ns} \quad (3.10)$$

$$V_n \leq 6 A_n \sqrt{f'_{AAC}} \quad \text{for } \frac{M_u}{V_u d_v} \leq 0.25 \quad (3.11)$$

$$V_n \leq 4 A_n \sqrt{f'_{AAC}} \quad \text{for } \frac{M_u}{V_u d_v} \geq 1.00 \quad (3.12)$$

Where,

f'_{AAC} = specified compressive strength of AAC masonry

The nominal strength contribution of the masonry for running bond, V_{nAAC} , is computed as (a) through (c).

(a) Governed by web-shear cracking

$$V_{nAAC} = 0.95 l_w t \sqrt{f'_{AAC}} \sqrt{1 + \frac{P_u}{2.4 \sqrt{f'_{AAC}} l_w t}} \text{ (for mortared head joints)} \quad (3.13)$$

$$V_{nAAC} = 0.66 l_w t \sqrt{f'_{AAC}} \sqrt{1 + \frac{P_u}{2.4 \sqrt{f'_{AAC}} l_w t}} \text{ (for unmortared head joints)} \quad (3.14)$$

(b) Governed by crushing of a diagonal compressive strut (for walls with $\frac{M_u}{V_u d_v} < 1.5$; this mode need not be calculated for values exceeding 1.5)

$$V_{nAAC} = 0.17 f'_{AAC} t \frac{h^* l_w^2}{h^2 + (\frac{3}{4} l_w)^2} \quad (3.15)$$

(c) Governed by sliding shear

$$V_{nAAC} = \mu_{AAC} P_u \quad (3.16)$$

Shear reinforcement contribution:

$$V_{ns} = 0.5 \left(\frac{A_v}{s} \right) f_y d_v \quad (3.17)$$

Where,

l_w = entire length of wall or segment considered in direction of shear force (in.)

t = nominal thickness of member (in.)

h = effective height of wall (in.)

μ_{AAC} = coefficient of friction for AAC (0.75 for AAC to AAC and 1.0 for AAC and thin-bed or leveling mortar)

These are the only shear equations in the TMS-402 that directly address web-shear cracking and crushing of a diagonal strut.

3.2 TMS-402 Shear Equation Performance

Figure 3.1 provides comparisons for 53 fully grouted CMU shear walls to the performance of the TMS-402 equations as a function of various parameters. Figures 3.2-3.3 provide similar comparisons for fully grouted clay and AAC walls, respectively. Full details of each wall are given in Appendix A. Fixity was considered, $\frac{M_u}{V_u d_v}$ was not taken greater 1.0 per the TMS-402 provisions, and the net area under shear was calculated utilizing the minimum face shell thickness and any vertically grouted cell.

Fully Grouted CMU Walls:

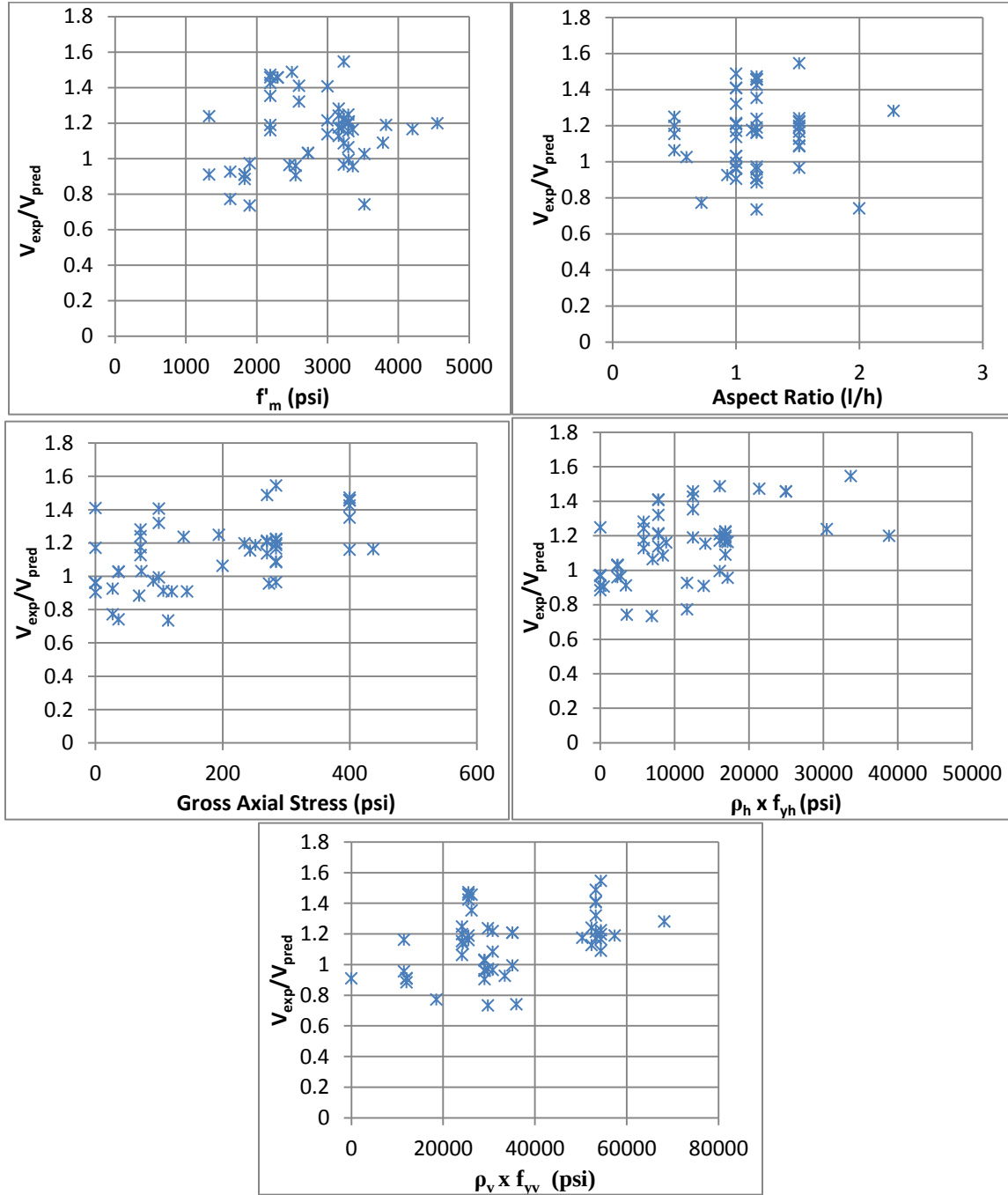


Figure 3.1 Fully Grouted CMU Walls Parameter Comparison

Fully Grouted Clay Walls:

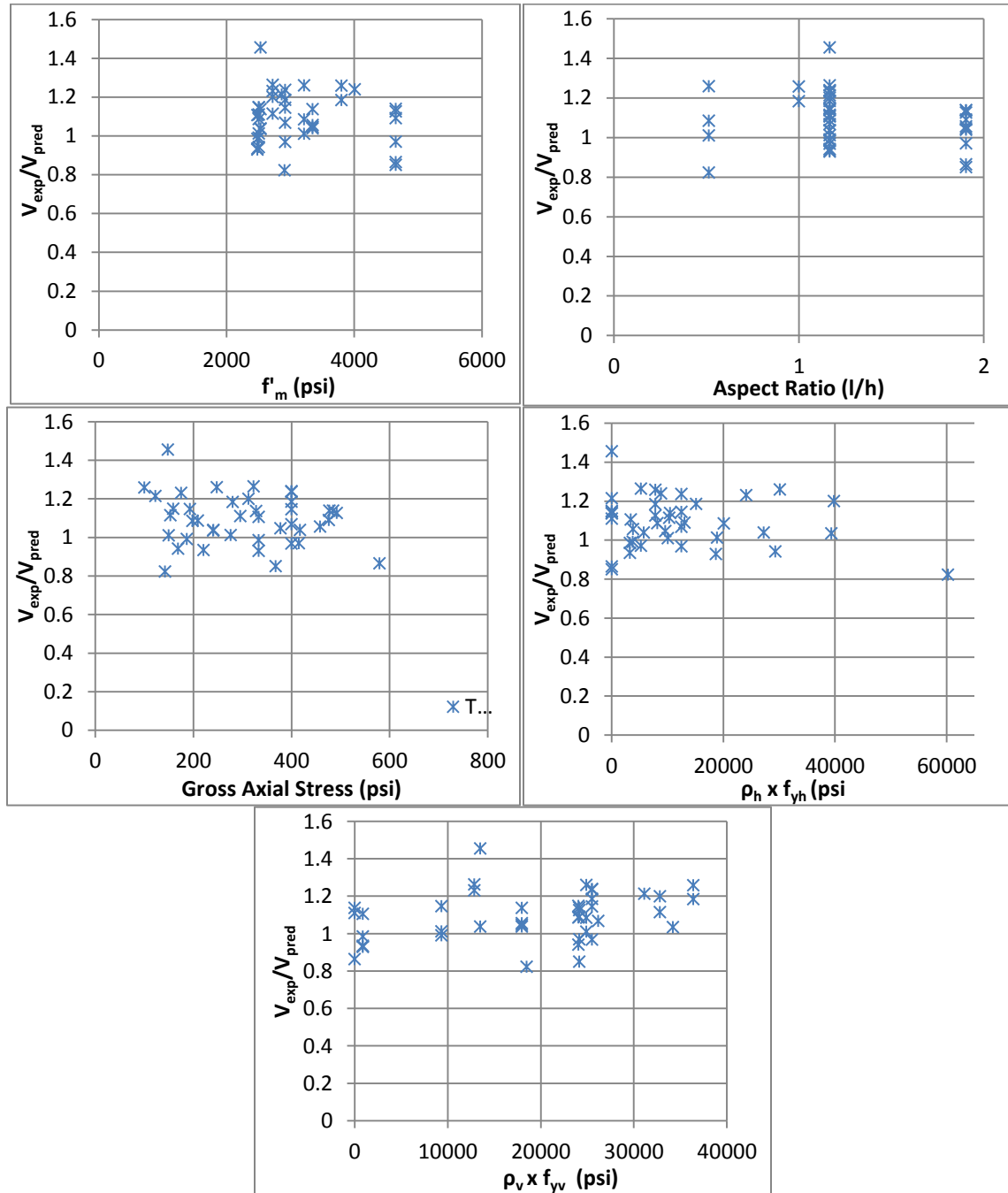


Figure 3.2 Fully Grouted Clay Walls Parameter Comparison

AAC Walls:

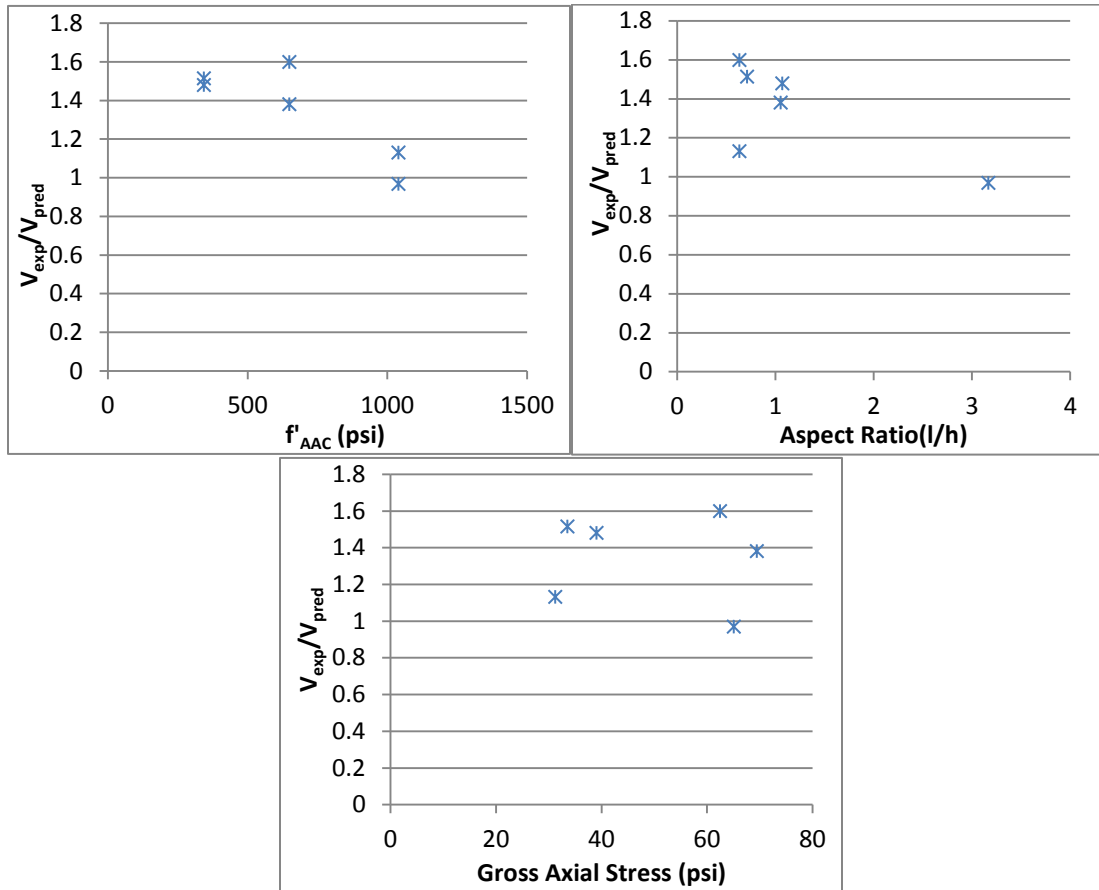


Figure 3.3 AAC Walls Parameter Comparison

Table 3.1—Fully Grouted Wall Statistics

Fully Grouted Walls											
	Fixed Walls			Cantilever Walls			Overall				
	#	Avg. $\frac{V_{exp}}{V_{pred}}$	COV	#	Avg. $\frac{V_{exp}}{V_{pred}}$	COV	Total	> 10% Overpredicted	> 25% Underpredicted	Avg. $\frac{V_{exp}}{V_{pred}}$	COV
CMU	34	1.17	0.17	19	1.10	0.19	53	4	12	1.15	0.18
Clay	40	1.06	0.20	2	1.22	0.04	42	3	4	1.09	0.12
AAC				6	1.35	0.18	6	0	4	1.35	0.18

Table 3.1 provides a summary for various unit types. Fully grouted walls are predicted well by the TMS-402 shear equations. For CMU, gross axial stress, $\rho_v \times f_{yv}$, and $\rho_h \times f_{yh}$ have a slight positive bias, indicating an under-prediction, although data is lacking in the upper ranges for these parameters. In general, the lower ranges of all parameters gave un-conservative values for CMU. Overall, fixed walls were predicted with less scatter than cantilever walls for CMU, but were more conservative. Clay and AAC walls showed no significant trends to suggest an over- or under-prediction. Data for AAC walls with units comparable to typical clay and CMU is lacking, however, 6 specimens have close to typical masonry unit sizes.

Figure 3.4 provides comparisons for 68 partially grouted CMU shear walls to the performance of the TMS-402 equations as a function of various parameters. A small set of comparisons for partially grouted clay walls are presented in Figure 3.5. Details for all walls are given in Appendix A.

Partially Grouted CMU Walls:

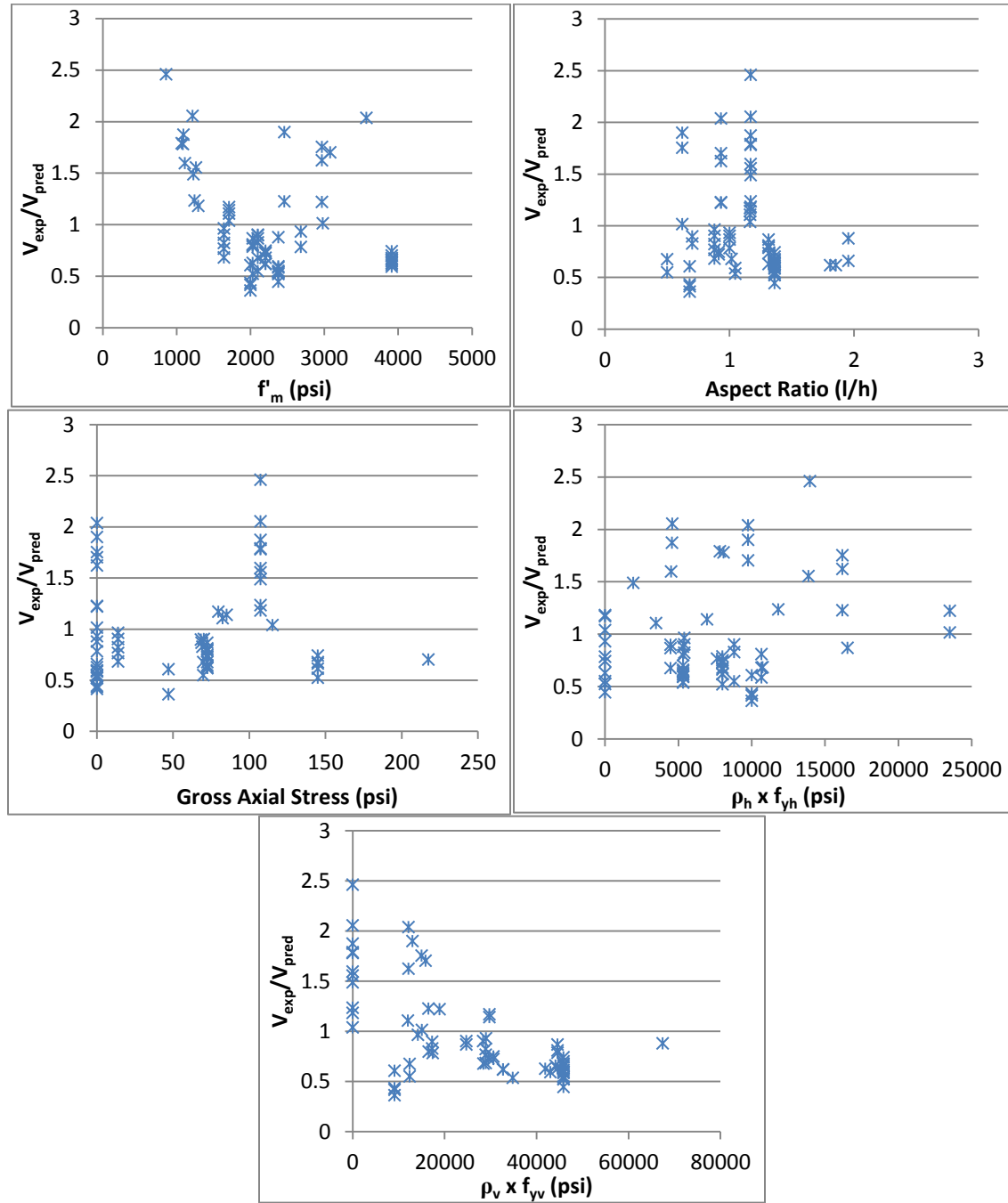


Figure 3.4 Partially Grouted CMU Walls Parameter Comparison

Partially Grouted Clay Walls:

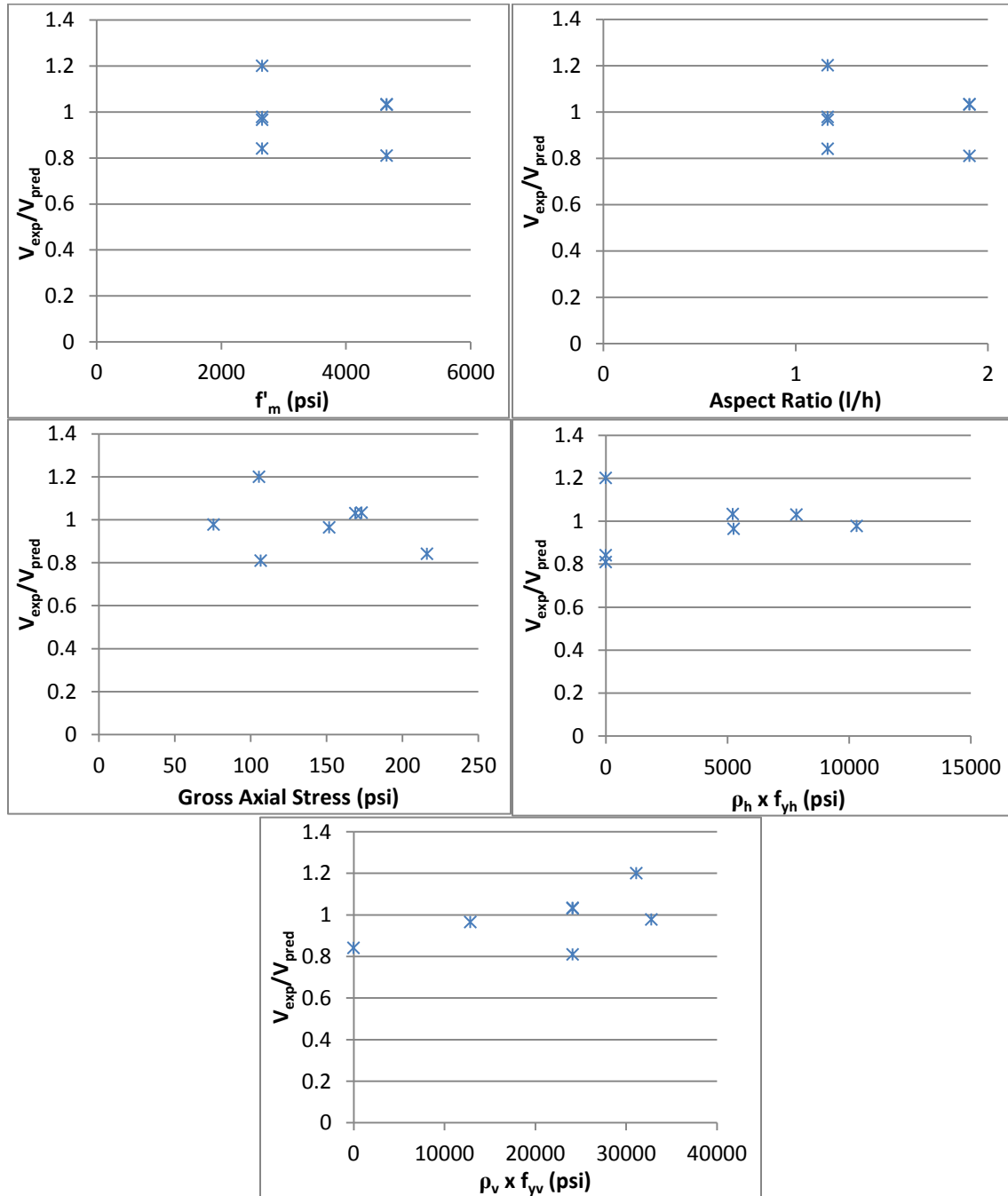


Figure 3.5 Partially Grouted Clay Walls Parameter Comparison

Table 3.2—Partially Grouted Wall Statistics

Partially Grouted Walls											
	Fixed Walls			Cantilever Walls			Overall				
	#	Avg. $\frac{V_{exp}}{V_{pred}}$	COV	#	Avg. $\frac{V_{exp}}{V_{pred}}$	COV	Total	> 20% Overpredicted	> 25% Underpredicted	Avg. $\frac{V_{exp}}{V_{pred}}$	COV
CMU	51	0.90	0.51	17	1.14	0.44	68	34	13	0.96	0.49
Clay	7	0.98	0.13				7	0	0	0.98	0.13

Some issues have arisen of late concerning the applicability and accuracy of the TMS-402 strength design capacity equations for partially grouted shear walls. Various researchers (Nolph and ElGawady, 2012; Minaie, et al., 2010; Elmapruk and ElGawady, 2009; Sarhat and Sherwood, 2009) have shown that the TMS-402 equations have the potential to over-predict the capacity of a partially grouted shear wall. Table 3.2 provides a summary for concrete and clay units. Partially grouted walls are not predicted well by the TMS-402 shear equations. From Figure 3.4, CMU wall parameter performance was generally un-conservative for all parameters, showing a negative bias for increasing masonry compressive strength and $\rho_v \times f_{yv}$. The TMS-402 predicted the small set of partially grouted clay walls quite well, but with a slightly un-conservative average $\frac{V_{exp}}{V_{pred}}$. One potential cause of the un-conservatism for a partially grouted wall is that the above equations were generated from tests on fully grouted walls, with the assumption that partially grouted walls have similar behavior (Elmapruk and ElGawady, 2012).

The V_{ns} term for shear reinforcement strength contribution considers half of the shear reinforcement in the wall to fully contribute to the strength of the wall, as evidenced by the rearrangement of the equation into the following form:

$$V_{ns} = 0.5 \left(\frac{d_v}{s} \right) f_y A_v \quad (3.18)$$

The $\frac{d_v}{s}$ term relates the shear depth of the wall (the length of the wall) with the vertical spacing of horizontal bars. One issue with this is, also shown by Nolph (2010), is that for a long wall, more bars could be considered than are actually present to resist shear. Often, when larger bars are used in each bond beam, the yield strength may never be reached. Equation 3.18 assumes the bars reach yield, and then reduces the contribution by half to account for those bars that are effective in providing shear resistance. However, for a small wall with only a top and bottom bond beam, neither location potentially should be considered to provide full yield strength as the reinforcement in the bond beams will most likely not be developed if diagonal cracking occurs. This leads to an over-prediction of strength provided by the steel.

This is also evidenced in the research conducted by Baenzinger and Porter (2010) and Nolph and ElGawady (2012).

Table 3.3—Maximum Strain/Average Yield Strain

Researcher	Wall I.D	Max. Shear Reinforcement Strain	Avg. Shear Reinforcement Yield Strain	ρ_h	<u>Max. Strain</u> Avg. Yield Strain
Baenzinger	1	0.00220	0.00242	0.151	0.91
Baenzinger	2	0.00334	0.00288	0.184	1.16
Baenzinger	3	0.00177	0.00242	0.151	0.73
Baenzinger	4	0.02320	0.00288	0.184	8.06
Baenzinger	5	0.00635	0.00288	0.184	2.20
Baenzinger	6	0.00387	0.00288	0.267	1.34
Baenzinger	7	0.00247	0.00242	0.151	1.02
Baenzinger	8	0.00255	0.00288	0.267	0.89
Baenzinger	9	0.00231	0.00242	0.151	0.95
Baenzinger	10	0.00177	0.00288	0.267	0.61
Nolph	PG085-48	0.00260	0.00219	0.085	1.19
Nolph	PG120-48	0.00180	0.00219	0.12	0.82
Nolph	PG169-48	0.00110	0.00219	0.169	0.50

Table 3.3 shows that for all ten shear walls analyzed for Baenzinger and Porter (2010) (1-8 failed in shear), the maximum recorded strain in the horizontal bars was less than the yield strain in at least half of the cases. Nolph and ElGawady (2012) had similar results as shown by 3 of their specimens in Table 3.3. Figure 3.6 shows the decreasing efficiency of increasing the amount of horizontal reinforcement past a certain value.

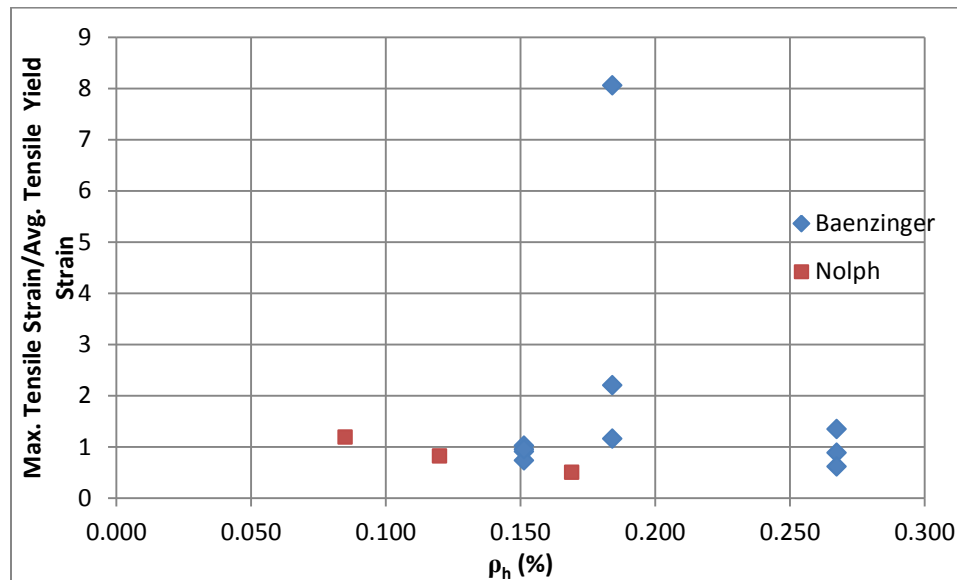


Figure 3.6 Strain Ratio vs. Shear Reinforcement Ratio

It seems that the current shear equations in the TMS-402 (2011) for traditional masonry materials are over-simplistic when it comes to the steel contribution, and perhaps too “all-encompassing” when it comes to the masonry contribution to shear wall strength for partially grouted walls.

Additionally, there are no current provisions in the TMS-402 to account for sliding shear of a wall constructed with traditional masonry, such as is the case with AAC masonry. It is believed that this should be included as a separate design equation to account for the potential of

a masonry course to be subjected to sliding shear. A proposed expression for sliding shear for concrete and clay masonry is presented in Chapter 11.

3.3 Code Comparison

Several masonry design codes have been compared with the TMS-402 with a particular focus on partially grouted walls. At the end of the section, performance of each code is presented in comparison to the TMS-402 for partially grouted clay and concrete masonry walls

3.3-A CSA S304.1-04 Design of Masonry Structures

The code provides requirements and guidelines for the structural design of masonry structures and components in accordance with the limit states design method of the *National Building Code of Canada*.

$$V_r = \phi_m (v_m b_w d_v + 0.25 P_d) \gamma_g + (0.6 \phi_s A_v f_y \frac{d_v}{s}) \quad (3.19)$$

$$V_r \leq 0.4 \phi_m \sqrt{f'_m} b_w d_v \gamma_g \quad (3.20)$$

$$v_m = 0.16 (2 - \frac{M_f}{V_f d_v}) \sqrt{f'_m} \quad (3.21)$$

Where,

V_r = factored shear resistance

v_m = shear strength attributed to the masonry (MPa)

d_v =effective depth of wall (need not be less than 0.8 x length of wall if flexural reinf. is distributed along the length) (mm)
 b_w = overall web width (mm)
 P_d =axial compressive load on the section (0.9 x dead load) (N)
 γ_g =factor to account for partially and ungrouted units (1.0 for solid masonry, otherwise $\frac{A_e}{A_g}$ but not greater than 0.5)
 s =spacing of shear reinforcement, mm
 A_v =cross sectional area of shear reinforcement, mm^2
 A_g =gross cross sectional area, mm^2
 $\frac{M_f}{V_f d_v}$ =a value that shall not be more than 1 or less than 0.25 for the concurrent factored moment, M_f , and factored shear, V_f , at the section under consideration
 ϕ_m =resistance factor for masonry, 0.6
 ϕ_s =resistance factor for steel, 0.85 for reinforcing bars and wire and 0.9 for prestressing steel

Additionally, the limit on V_r can be increased by a factor of $[2 - \frac{h_w}{l_w}]$, where h_w is the wall height and $0.5 \leq \frac{h_w}{l_w} \leq 1.0$ for squat walls with aspect ratio less than 1.0.

The masonry contribution equation has similar form to the TMS-402 masonry contribution, with the same factor on the factored axial load. The steel contribution again uses the shear depth for calculation of participating bars, and is higher than the TMS-402 with a factor of 0.6.

3.3-B Australian Standard AS 3700--1998

The Australian Standard AS-3700 code in-plane shear provisions for reinforced walls externally supported against overturning are given in Equation 3.22.

$$V_d \leq \phi f_{vr} A_d + 0.8 f_{sy} A_s \quad (\text{for walls with } \frac{H}{L} \leq 2.3) \quad (3.22)$$

Where,

V_d = design shear force acting on the cross section of the masonry wall (N)

$\phi = 0.75$ for reinforced masonry

f_{vr} = effective shear strength = $1.5 - 0.5 \frac{H}{L}$ (MPa)

A_d = design cross sectional area of member: (mm²)

= For a fully grouted member, the product of the overall width less the depth of raking (if any), and the length

= For a partially grouted member, the product of the overall width less the depth of raking (if any), and the portion of the length represented by the grouted cores, plus the bedded area of the portion of the length represented by the ungrouted cores

H = height of the member perpendicular to the direction of the shear load (mm)

L = length of the member in the direction of the shear load (mm)

f_{sy} = design yield strength of reinforcement (MPa)

A_s = cross sectional area of reinforcement: (mm²)

= if $\frac{H}{L} > 1.0$, A_s is the total area of anchored horizontal reinforcement

= if $\frac{H}{L} \leq 1.0$, A_s is the total cross sectional area of horizontal reinforcement or total cross sectional area of vertical reinforcement, whichever is less

Walls that have a $\frac{\text{height}}{\text{length}}$ ratio greater than 2.3 must be designed using the “out-of-plane”

design provision, and in addition, walls that are not supported against overturning have alternate provisions. The Australian equations for in-plane shear depend upon the aspect ratio, with no contribution from the axial load and no dependence upon $\sqrt{f'_m}$, as is the case with most other code equations. Also, the steel portion considers all of the reinforcement (lesser of the vertical and horizontal) to contribute, with the exception of a slender wall with aspect ratios greater than 1.0. The factor of 0.8 applied to the steel contribution is in contrast to the 0.5 applied by the TMS-402.

3.3-D New Zealand NZS 4230-2004 Masonry Code

The equations for in-plane shear strength of masonry are given in SI units.

$$V_n = v_n b_w d = (v_m + v_p + v_s) b_w d \quad (3.23)$$

$$v_n \leq 0.45 \sqrt{f'_m} \quad (3.24)$$

$$v_m = (C_1 + C_2) v_{bm} \quad (3.25)$$

$$v_p = 0.9 \frac{N^*}{b_w d} \tan \alpha \quad (3.26)$$

$$v_s = C_3 \frac{A_v f_y}{b_w s} \quad (3.27)$$

Where,

V_n = nominal shear strength (N)

b_w = effective web width (mm)

d = 0.8 x length of wall (mm)

v_n = total nominal shear stress (MPa), $\leq 0.45 \sqrt{f'_m}$ for general conditions

v_m = shear strength provided by the masonry (MPa)

v_p = shear strength provided by the axial load and $\leq 0.1 f'_m$

v_s = shear strength provided by the shear reinforcement

v_{bm} = inspection type-dependent shear strength of masonry (for type "A" $= 0.2 \sqrt{f'_m}$)

$C_1 = 33 \rho_w \frac{f_y}{300}$ (for longitudinal reinforcement ratios $\geq 0.07\%$)

$C_2 = 0.42 [4 - 1.75 \frac{h_g}{l_w}]$ (for $\frac{h_g}{l_w} < 0.25$, $C_2 = 1.5$ and for $\frac{h_g}{l_w} > 1.0$, $C_2 = 1.0$)

$C_3 = 0.8$ (for walls)

$\rho_w = (\frac{A_s + A_{ps}}{b_w d})$

N^* = design axial load in compression and $\leq 0.1 f'_m A_g$

α = angle formed between line of action of axial load and resulting reaction on a component
(considering half the wall length for single bending, and the entire wall length for double bending)

A_v = area of shear reinforcement within a distance, s

f_y =lower characteristic yield strength of non-prestressed reinforcement

s =spacing of shear reinforcement in direction parallel to longitudinal reinforcement

The masonry strength contribution is dependent upon the inspection level of construction, with Equations 3.23-27 belonging to inspection level A, which requires continual inspection and observation of construction of the masonry entity. The factor C_2 is largely similar to the TMS-402 portion of the masonry contribution, while the steel portion takes 80% of the available force. This is similar to the TMS-402 in regards to using the effective shear depth of the wall to calculate a number for horizontal bar contribution. The angle term, α , represents the effective location of the axial load for walls in single and double bending.

All of the presented masonry codes have been used to analyze the available data for both concrete and clay partially grouted walls, along with the equation of Matsumura (1986).

Summary statistics are given in Table 3.4.

Table 3.4—Code Performance for Partially Grouted Walls

Partially Grouted Walls														
Equation	Unit Type	Fixed Walls				Cantilever Walls				Overall				
		#	Avg. $\frac{V_{exp}}{V_{pred}}$	COV		#	Avg. $\frac{V_{exp}}{V_{pred}}$	COV		Total	Avg. $\frac{V_{exp}}{V_{pred}}$	COV	Min.	Max.
TMS-402 2011	CMU	51	0.90	0.51		17	1.14	0.44		68	0.96	0.49	0.36	2.46
	Clay	7	0.98	0.13						7	0.98	0.13	0.81	1.20
	Total									75	0.96	0.47	0.36	2.46
CSA S304.1 2004	CMU	51	1.43	0.50		17	0.92	0.40		68	1.31	0.53	0.45	3.36
	Clay	7	2.65	0.51						7	2.65	0.51	1.17	3.93
	Total									75	1.43	0.60	0.45	3.93
AS 3700 1998	CMU	51	0.77	0.43		17	0.46	0.30		68	0.70	0.47	0.28	1.61
	Clay	7	1.45	0.30						7	1.45	0.30	0.86	2.09
	Total									75	0.77	0.52	0.28	2.09
NZS 4230 2004	CMU	51	1.65	0.30		17	1.04	0.43		68	1.50	0.36	0.55	2.90
	Clay	7	1.43	0.23						7	1.43	0.23	0.93	1.72
	Total									75	1.49	0.35	0.55	2.90
Matsumura 1986	CMU	51	1.90	0.36		17	1.44	0.48		68	1.79	0.40	0.62	4.07
	Clay	7	1.67	0.35						7	1.67	0.35	1.16	2.63
	Total									75	1.77	0.39	0.62	4.07

From the available codes and provisions, it seems that in general the majority of shear strength expressions contain separate contributions from the masonry, steel, and axial load. The masonry contribution is generally presented as a Mohr-Coulomb type failure equation with strength dependent upon $\sqrt{f'_m}$ (with the exception of the Australian code), with empirical factors applied. It seems that no singular shear failure mode is directly addressed by the equations. Horizontal reinforcement contributions seem to trend towards taking a percentage of the total horizontal reinforcement that would be present considering their spacing and shear depth of the wall.

From Table 3.4, it appears that no code can consistently predict the strength of the partially grouted walls in available data. Coefficients of variation are above 0.36 in every case for CMU, with some slightly smaller values for clay units. The two best performers, the New Zealand code and Matsumura (1986) have reasonable variation, but high average $\frac{V_{exp}}{V_{pred}}$.

Chapter 4—Strut and Tie Methods

4.1 Strut and Tie Background

Currently, none of the shear expressions from Chapter 3 are based on strut-and-tie concepts, with the exception of the AAC provisions of the TMS-402 (2011). In this chapter, a brief introduction to strut-and-tie theory is presented followed by a description of the ACI-318 (2011) and AASHTO LRFD (2012) provisions for design using strut-and-tie methods. The equations from these codes serve as a good basis for development of similar equations for masonry.

One of the leading publications on strut-and-tie theory is Schlaich et al. (1987) and serves as the major basis for codes' use of strut-and-tie theory. The authors sought to remedy the issue of designing those parts of a structure where strains deviated from linear behavior, and where stress discontinuities were introduced. The authors enforce the notion that for proper evaluation of structures, both the geometry and loads need to be considered.

Strut and tie theory is commonly used for design of deep concrete beams and in steel design of deep plate girders by means of tension-field action. In general, strut-and-tie methods are used where there are discontinuities in a structure, such as corners, concentrated loadings, and concentrated reaction forces. Typically, in a beam-type member, the disturbed regions in the vicinity of these entities are usually ignored, and are assumed to dissipate within the portions that have normal behavior within a member depth away from either side of the discontinuity (St. Venant's Principle). However, in a wall-type structure that is analogous to a very deep beam, this disturbed region based upon St. Venant's Principle would most likely include the entire wall. Figure 4.1 is an example of stress trajectories in a deep beam with a concentrated vertical load at the top. The trajectory lines become more "uniform" as the depth increases away from the

concentrated load and reactions. It is in these regions, generally called “D-regions,” that a strut-and-tie analysis is preferred.

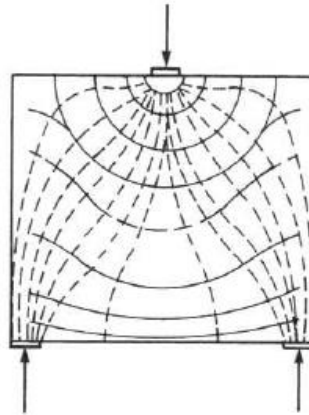


Figure 4.1 (Macgregor and Wight, 2005) Stress Trajectories in a Deep Beam

Strut-and-tie models consist of various elements and/or regions that are assumed to form in a deep section, such as a wall, when loading occurs. These elements primarily include struts, ties, and nodes. A strut and tie model for a deep beam with a vertical load is presented in Figure 4.2 to clarify the terms.

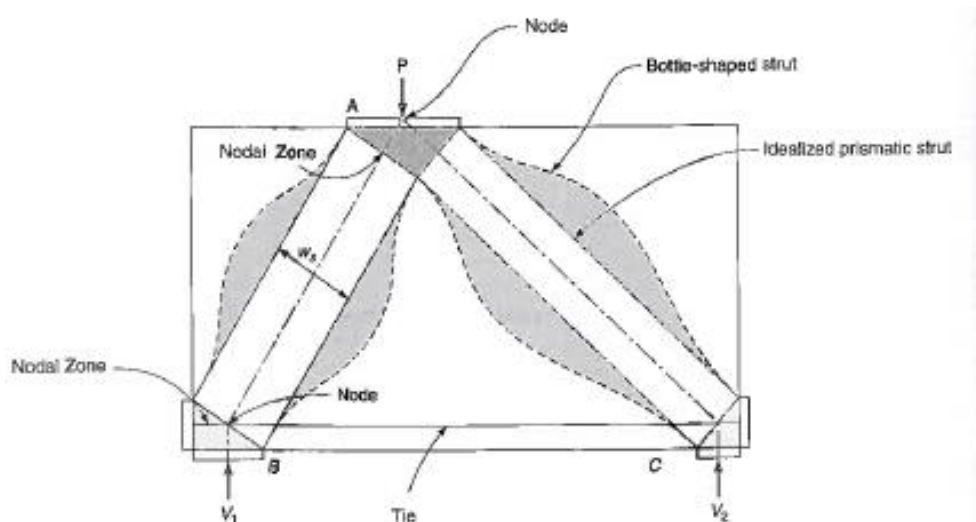


Figure 4.2 (Macgregor and Wight, 2005) Strut-and-Tie Model for a Deep Beam

The “struts” represent the compressive stress fields in the member, with compression stresses acting perpendicular to the principle tensile stresses. If the correct model is chosen, the actual compressive stresses will act parallel with the inclination of the strut. The typical failure modes of a strut are crushing and severe cracking, in which the principal tensile stress pulls apart the member and may cause it to crack longitudinally, or, if enough transverse reinforcement is provided, the member may crush in the case of a concrete-like material. (Macgregor and Wight, 2005)

The strength of the concrete strut depends largely upon the assumption of the area of the strut, which is controlled by assumed cracking angle and tie width. Also, as stated in MacGregor and Wight (2005), strut compressive strength is a function of:

- A. Concrete strength
- B. Load duration effects
- C. Transverse tensile strains in the strut
- D. Cracked struts

Collins et al. (1996) provides the graph in Figure 4.3, acknowledging the degradation of the concrete compressive strength due to transverse tensile strains and cracking. Struts can be various shapes—from bottle-necked to prismatic to tapered. The primary focus of struts in this research is on a shape that can be readily generated and analyzed, therefore, only prismatic struts have been utilized.

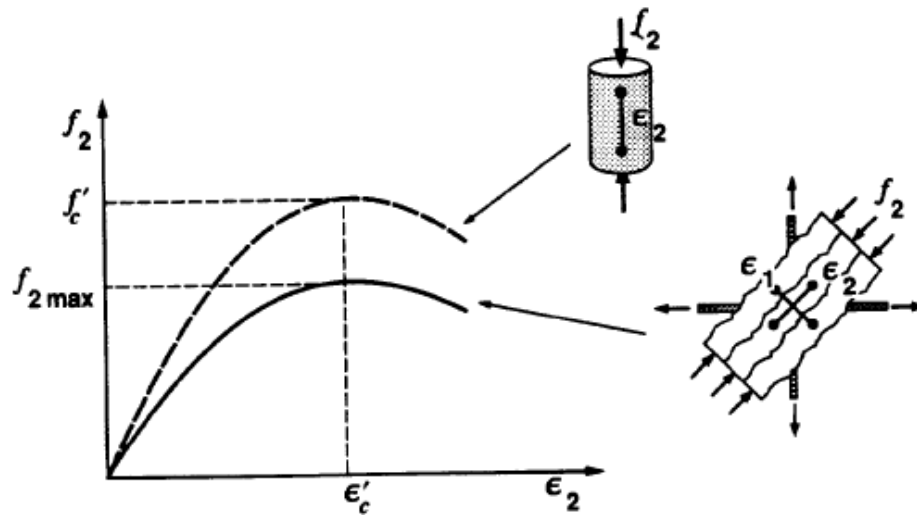


Figure 4.3 (Collins et al, 1996)—Qualitative Graph Showing Degradation of Principal Compressive Strength of Concrete Due to Transverse Tensile Strain

The “ties” are reinforcement within the member that are assumed to provide the tensile resistance to the applied loads. The ties in a brittle material like concrete are assumed to have a casing of concrete around them, to generate an effective width for use in determining the width of the struts. Factors affecting the strength of ties as applied to reinforced concrete members are as follows:

- A. Strength of reinforcement
- B. Size
- C. Anchorage
- D. Bond to surrounding concrete

Nodal zones are the regions that bind all the elements together, and can be related to a pinned joint in a truss (MacGregor and Wight, 2005). In the case of a structural member, the regions deemed to be struts and nodes are fictitious until loading is applied. The nodal regions are essentially the intersections of the struts and ties, creating a zone that can have both compressive and tensile forces. The incompatibility of this arrangement of forces is considered in the ACI-318 and AASHTO LRFD design manuals. The nodes are generally calculated

according to the width of struts and ties in a simplified analysis, usually fail in crushing, and their strength is dependent largely upon the kinds of forces acting in their region and the strength of the material.

A good summary of strut-and-tie methodology is stated by Macgregor and Wight (2005) and is as follows:

“A strut-and-tie model is a model of a portion of the structure that satisfies the following:

- (a) It embodies a system of forces that is in equilibrium with a given set of loads
- (b) The factored-member forces at every section in the struts, ties, and nodal zones do not exceed the corresponding factored-member strengths for the same sections.

The lower-bound theorem of plasticity states that the capacity of a system of members, supports, and applied forces that satisfies both (a) and (b) is a lower bound on the strength of the structure. For the lower-bound theorem to apply, the structure must have:

- (c) The structure has sufficient ductility to make the transition from elastic behavior to enough plastic behavior to redistribute the factored internal forces into a set of forces that satisfy (a) and (b).

The combination of factored loads acting on a structure and distribution of factored internal forces is a lower bound on the strength of the structure, provided that no element is loaded beyond its capacity. For this reason, strut-and-tie models are chosen so that the internal forces in the struts, ties, and nodal zones are somewhere between the elastic distribution and the fully plastic set of internal forces.”

In addition, Schlaich et al (1987) states that “it is helpful to realize that loads try to use the path with the least forces and deformations” and because of this, the model with the least ties is best. However, in a masonry wall, the ties are well defined based upon code requirements for seismic provisions, which could cause many ties to be considered.

4.2 Strut-and-Tie Provisions of ACI-318

Appendix A of the ACI-318 code permits the design of regions of discontinuities by modeling the members as an idealized truss. The general design procedure outlined in ACI-318 is as follows:

1. Define and isolate each D-region (region of discontinuity)
2. Compute resultant forces on each D-region boundary
3. Select a truss model to transfer the resultant forces across the D-region. The axes of the struts and ties, respectively, are chosen to approximately coincide with the axes of compression and tension fields. The forces in the struts and ties are computed.
4. The effective widths of the struts and nodal zones are determined considering the forces from Step 3 and the effective concrete strengths. Reinforcement is provided for the ties considering the steel strength. The required reinforcement for the ties is determined and is anchored in the nodal zones.

The dimensions of struts, ties, and nodal zones should be chosen to fit in the given dimensions of the structure. The angle between struts and ties has a lower bound of 25 degrees to limit cracking and to avoid incompatibilities produced by the arrangement of struts and ties at low angles. (ACI-318, 2011)

The nominal compressive strength of a non-reinforced prismatic strut, F_{ns} , is the smaller of the values calculated at both ends of a strut, given as

$$F_{ns} = f_{ce}A_{cs} \quad (4.1)$$

where A_{cs} is the cross-sectional area at one end of the strut and f_{ce} is the smaller of the effective compressive strength of the concrete and the effective compressive strength of the nodal zone.

The effective compressive strength of concrete in a strut is given as

$$f_{ce} = 0.85\beta_s f'_c \quad (4.2)$$

where $\beta_s = 1.0$ for a prismatic strut. The 15% reduction in the equation is to account for load duration effects of sustained compression on concrete.

The effective strength of concrete in a nodal zone is given as

$$f_{ce} = 0.85\beta_n f'_c \quad (4.3)$$

where $\beta_n = 1.0$ for nodal zones bounded by struts or bearing areas, 0.8 in zones anchoring one tie, and 0.6 in zones anchoring two or more ties, and represents the “increasing degree of disruption of the nodal zones due to the incompatibility of tension strains in the ties and compression strains in the struts.” (ACI-318, 2011)

The strength of nodal zones is given as

$$F_{nn} = f_{ce} A_{nz} \quad (4.4)$$

where A_{nz} is the area on the face of the nodal zone perpendicular to the line of action of the force. In general, when assuming a prismatic strut with a nodal face perpendicular to the line of action of the force in the strut, F_{nn} and F_{ns} will be of the same value due to the criteria developed for the strength of the strut in a solid wall.

The strength of ties is equal to the yield force provided by the reinforcement, as shown in Equation 4.5:

$$F_{nt} = A_{ts} f_y \quad (4.5)$$

where A_{ts} is the area of reinforcement in a tie.

As stated by MacGregor and Wight (2005), stresses can be different on all faces at a node, as long as the following criteria apply:

1. The resultants of the three forces at a node coincide
2. The stresses are within the limits stated earlier
3. The stress is constant on any one face

Diagram illustrating the geometry of the nodal zone and extended nodal zone for a beam-column joint under seismic loading. The diagram shows a 3D view of a joint where a beam of width w_t and length l_b is connected to a column. The beam is inclined at an angle θ to the horizontal. The nodal zone is the region of concrete that is crushed during a seismic event. The extended nodal zone is the region of concrete that is crushed during a seismic event, extending beyond the nodal zone. The diagram shows the geometry of the nodal zone and extended nodal zone, including the beam width w_t , beam length l_b , beam inclination angle θ , and the nodal zone length l_{nc} . The diagram also shows the beam width w_t , beam length l_b , beam inclination angle θ , and the nodal zone length l_{nc} . The diagram also shows the beam width w_t , beam length l_b , beam inclination angle θ , and the nodal zone length l_{nc} .

Figure 4.4 is applicable to a deep beam , where l_b is the length of bearing, l_{anc} is the length of tie anchorage and this value should be sufficient to develop the tie before leaving the extended

nodal zone, w_t is the effective width of the tie, and w_s is the effective width of the strut based upon the assumed tie width and bearing area. The resulting cross sectional area of the strut can then be taken as the effective strut width, w_s , and the thickness through the member. The light gray area in Figure 4.4 is the extended nodal zone that is comprised of the node and all concrete that falls within the bounds of the struts, ties, and bearing areas and can be useful for developing reinforcement in the tie.

4.3 AASHTO LRFD Bridge Design Specifications

In section 5.6 of the AASHTO LRFD Bridge Design Specifications (2012), strut and tie methods are presented as a means to calculate force effects in a structure where conventional methods are not applicable due to nonlinear strain distribution throughout the member. As stated in the AASHTO manual, traditional section-by-section design does not consider the possibility of non-uniform shear distributions and the interaction of shear and moment on the global strength of the structure, but rather considers section-only values to evaluate force effects. The AASHTO manual also emphasizes that instead of a section-based design for deep members, the flow of compressive stresses throughout the member should be analyzed, and sufficient reinforcement provided to resist the applied loads.

Similar to the ACI-318 version of strut and tie, the AASHTO specifications allows the approximation of struts and ties as straight members, and regions of their intersection (nodes) as entities of finite dimensions, based upon the assumed dimensions of those members framing into the nodes. The method to establish geometry of the model involves trial and error in which

member sizes are assumed, geometry is established, member forces determined, and assumed sizes verified. (AASHTO LRFD, 2012)

The AASHTO LRFD Strut-and-Tie Model Design Examples (Portland Cement Association, 2004) gives the following steps for design of a D-region region:

1. Visualize the Flow of Stresses

--Visualize the flow of stresses and define a load path with straight-line struts. Determine the location of tension ties for equilibrium, and write them into the model leaving space for placement and development of the reinforcement

2. Sketch an Idealized Strut-and-Tie Model

--Sketch the model in the form of a truss idealization, and solve for member forces

3. Select Area of Ties

--Choose the required area of reinforcement in the tension ties to ensure that the factored resistance in the ties equals or exceeds the required factored force in the ties.

4. Check Nodal Zone Stresses

--Compare the stresses with the nodal zone limits. The compressive strength of the nodal zones depends on the tensile straining from the intersecting tension ties and confinement due to the presence of transverse reinforcement—the limits on strength depend upon the number of ties intersecting the nodal zone.

5. Check Strength of Struts

--Compare the factored resistance of struts to calculated factored loads in the strut members. The limiting stress depends upon the angle of inclination of the strut and the tensile strain in the tie where it crosses the strut.

6. Provide Adequate Anchorage for the Ties

--Provide adequate anchorage for the ties so that they can develop the required tie force by hooks or development lengths such that the tension in the reinforcement can be resisted at the location where the centroid of the reinforcement crosses the edge of the adjoining strut.

The nominal resistance of an unreinforced compressive strut, P_n , is taken as:

$$P_n = f_{cu} A_{cs} \quad (4.6)$$

Where,

$$f_{cu} = \text{limiting compressive stress, taken as } f_{cu} = \frac{f'_c}{0.8+170\varepsilon_1} \leq 0.85f'_c \text{ (ksi)}$$

ε_1 = principle tensile strain in the concrete = $\varepsilon_s + (\varepsilon_s + 0.002)\cot^2\alpha_s$
 α_s = smallest angle between compressive strut and adjoining tension tie
 ε_s = the tensile strain in the concrete in the direction of the tension tie
 A_{cs} = effective cross-sectional area of a strut (in²)

In Equation 4.6 for principle tensile strain in the concrete, a strain value of $\varepsilon_s = 0.002$ (gr. 60 steel) would correspond to a tie yielding in tension at the location where it crosses the centerline of the strut. If the tension force in the tie varies across the strut, then ε_s can be taken as the value where the tie meets the centerline of the strut. AASHTO states that the concrete can resist a compressive stress of $0.85f'_c$ if not subjected to principal strains greater than 0.002 (the limit for struts not joined to tension ties). If a strut is bonded to a tension tie, such as in a nodal zone, tensile strains can be imposed on the concrete, thereby reducing its strength. For this reason, as tensile strain increases, the limiting compressive stress decreases. (Portland Cement Association, 2004)

The principal tensile strain in the concrete, ε_1 , is essentially a “damage indicator” that controls the ability of the diagonally cracked concrete to carry the compressive stresses (Collins et al, 1996). Figure 4.5 shows how the limiting compressive stress changes with respect to ε_s and α_s .

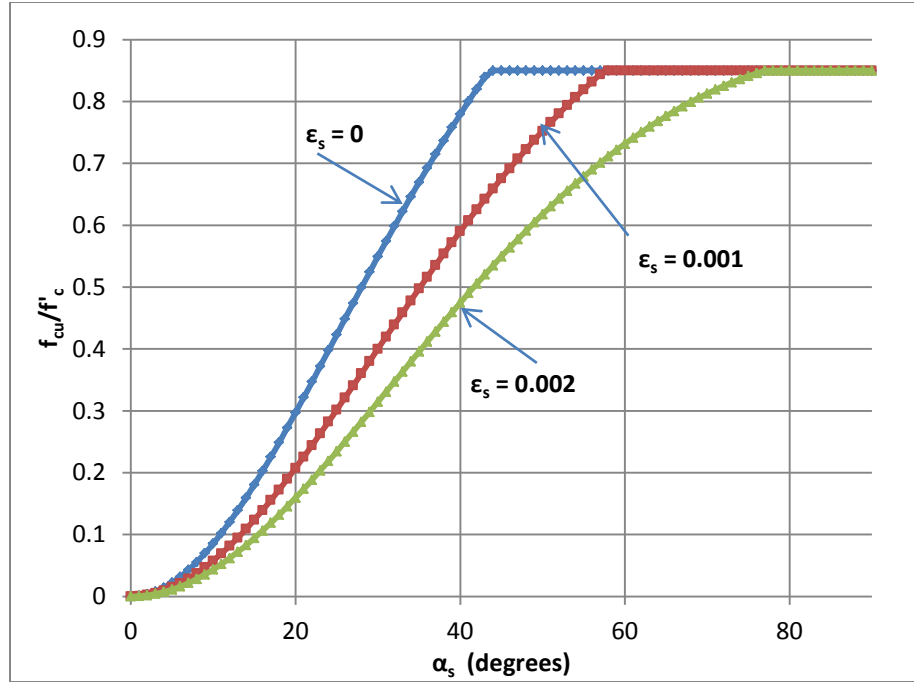


Figure 4.5 (drawn by the author) From AASHTO LRFD Design Examples, 2004—Influence of Parameters on the Limiting Compressive Stress in a Strut

The effective area of a strut is determined by considering the available concrete area and the anchorage conditions at the end of the strut. When a strut is anchored by reinforcement, the effective concrete area is allowed to extend a distance of up to six bar diameters from the anchored bar.

The tension tie force must be developed at the inner face of a nodal zone, and the strength of tension ties without prestressing steel can be taken as:

$$P_n = f_y A_{st} \quad (4.7)$$

Where,

A_{st} = total area of longitudinal mild steel reinforcement in the tie (in²)

The compressive stress in a node should not exceed:

$0.85f'_c$ for nodes bounded by compressive struts and bearing areas

$0.75f'_c$ for nodes anchoring a one-direction tension tie

$0.65f'_c$ for nodes anchoring tension ties in more than one direction

If these limits are reached, stresses in the nodal zones can be reduced by increasing the bearing area, increasing the strut dimensions, or increasing the dimension of the tension ties.

4.4 Strut-and-Tie Calculation of In-Plane Strength of Masonry

Walls

Strut and tie methods have been used by Nolph (2010) and Voon (2007) to analyze masonry shear walls, but the process often requires iteration to determine strut-and-tie configuration and can be lengthy for walls with multiple bond beams and vertical cells containing reinforcement and is generally better suited for a structural analysis program.

Nolph (2010) used Visual Analysis to analyze his data set of six walls according to ACI-318 provisions. Similar methods have been employed in this research to analyze 58 walls (Appendix B) from various data sets referenced earlier to validate the potential use of some of the provisions of the ACI-318 and AASHTO LRFD strut-and-tie provisions for use in masonry shear walls. The 58 walls were chosen with an emphasis on providing a variety of sizes and reinforcement scenarios for both partially and fully grouted walls.

As stated by Schlaich et al. (1987), there are no unique or optimum solutions in strut-and-tie—experience in using strut-and-tie methods will aid the designer in choosing the correct model. To use the ACI-318 and AASHTO LRFD code provisions, certain assumptions must be made for the tie width, angle of struts, and other conditions that may arise. It is the goal of this chapter to lay the foundation for a strut-and-tie based design method for masonry walls that is based loosely upon current code provisions in the ACI-318 and AASHTO LRFD design codes for concrete.

4.5 Procedure for Proposed Strut-and-Tie Use in Masonry

An outline is provided for analysis of masonry walls

Strut-and-Tie Procedure for Masonry:

1. Idealize all horizontal and vertical reinforcement in the wall as straight lines, intersecting at points (nodes). Regions that are bounded by steel will be denoted as “panels.” An initial steel arrangement may be assumed based upon TMS-402 prescriptive requirements and/or experience of similar walls and loadings.
2. Consider a unidirectional loading, having the horizontal load to act at the top-most bond beam, if located sufficiently near the top of the wall (otherwise, the load should act at the top of the wall). Any applied axial load shall be distributed to the nodes along the top of the wall, based on tributary area.
3. Draw struts in every panel of the wall between points of steel intersection to properly coincide with the assumed loading direction.
4. The angle this strut makes with the vertical reinforcement will define the strut angle. To determine the strut width, use the following relation:

$$w_{sd} = \frac{w_t}{\cos \theta} \quad (4.8)$$

Where,

w_{sd} = maximum width of a diagonal strut (in.)

w_t = minimum of reinforcement diameter plus 2 x the clear cover and the wall thickness (in.)

θ = angle between vertical tie and diagonal strut

5. Assume the strut width calculated by Equation 4.8 to be centered about the lines drawn in Step 3 such that any struts and ties acting at the node will be coincidental.
6. The thickness of a diagonal strut is assumed to be the thickness of the wall for a fully grouted wall, and the thickness of the combined minimum face shell thickness for a partially grouted wall.
7. Assume that the vertical ties framing into a node have a resultant coinciding with the line of force of the vertical reinforcement. Any masonry contribution to tensile strength is neglected.
8. To define horizontal and vertical strut areas, the cross-sectional area shall be limited to the dimensions of the cells or courses containing reinforcement. Any reinforcement contribution to compression in a strut is neglected.
9. The nodal zone is defined as that region encompassed by overlapping areas of struts and ties, and is idealized using straight cross sections as in Figure 4.9
10. Tie strengths are assumed to equal the yield strength of the reinforcement.
11. Strut strengths are assigned as below:

$$F_{ns} = f_{me} A_{cs} \quad (4.9)$$

Where,

F_{ns} = strut strength, (lb)

f_{me} = effective masonry strength, taken as $0.85 f'_m$ (psi)

A_{cs} = cross-sectional area of a strut, (in²)

12. The calculation of nodal zone strengths on any one face shall be as follows:

$$F_{nz} = A_{nz} f_{me-n} \quad (4.10)$$

Where,

F_{nz} = strength of a nodal zone

f_{me-n} = effective masonry strength of a node, taken as $0.85 \beta_n f'_m$, psi

β_n = 1.0 for nodes with struts only, 0.8 for nodes anchoring one tie, and 0.6 for nodes anchoring more than one tie; zero-force bars in a layout need not be counted as ties

A_{nz} = area on a face which is perpendicular to the line of action of the force in the strut. Widths of a nodal face are not to be taken greater than any calculated strut

width framing into that face that would correspond to a fully grouted wall. It shall be permitted to use fully-grouted f'_m for calculation of nodal strengths.

Typical areas for consideration of stresses are shown by the distinct cross-sections in the nodal zones in Figure 4.9.

4.6 Discussion of Proposed Method

Masonry walls have distinct tie dimensions, which require slight modification of the methods in the aforementioned design codes. Figure 4.6 is a panel bounded by steel, and shows the process outlined previously.

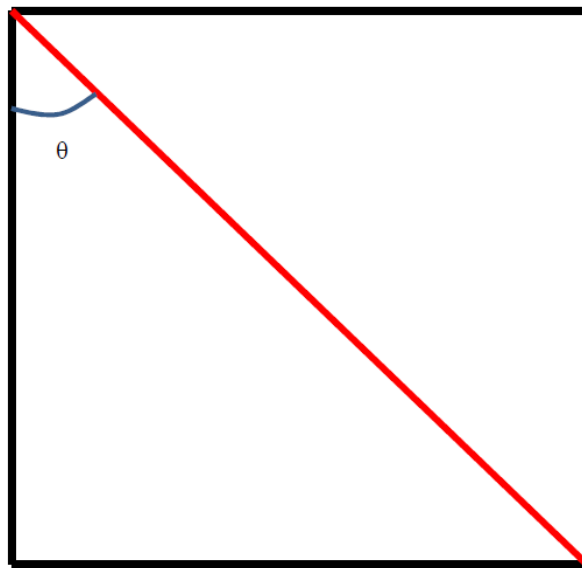


Figure 4.6 Orientation of Strut and Angle Location

To define the strut axis, a straight line is drawn from the resultant line of force from the jamb steel to same on the opposite lower corner of the wall. This holds true for interior panels bounded by nodal regions as well.

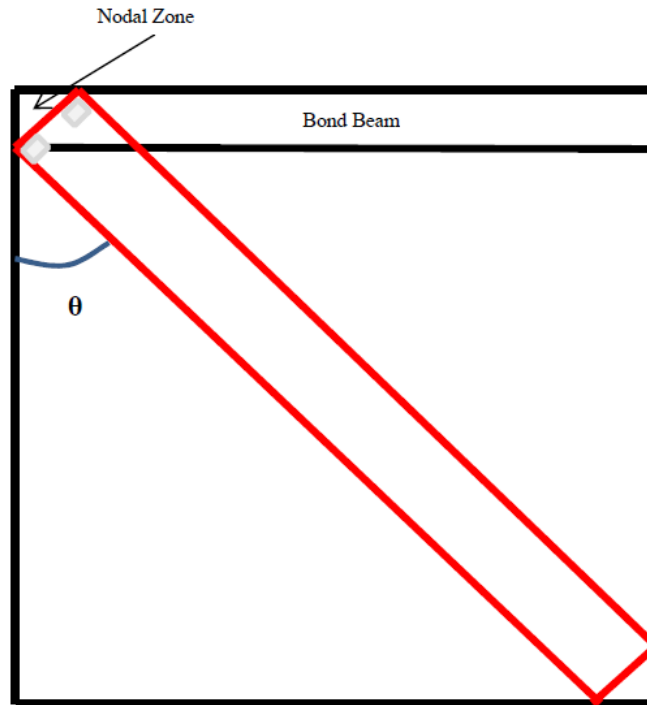


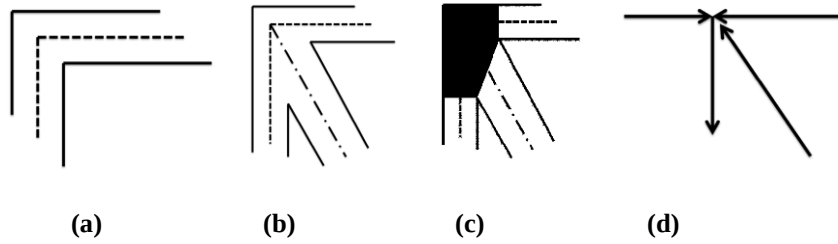
Figure 4.7 Drawn Strut in a Panel/Wall

The tie width, w_t , should be taken as the minimum of the wall thickness, and vertical reinforcement diameter plus twice the clear cover (for units up to 8 in. thick, this will generally coincide with the unit thickness). This seems to accommodate most vertical reinforcement placement scenarios. This gives a value of theta equal to the angle between w_t and the strut width, and thereby a strut width equal to $\frac{w_t}{\cos \theta}$. Lines are then extended to the opposite corner of the wall or panel to complete the prismatic strut. The strut thickness can be assumed as the masonry unit thickness for a fully grouted wall, and thickness of the minimum face shells for a

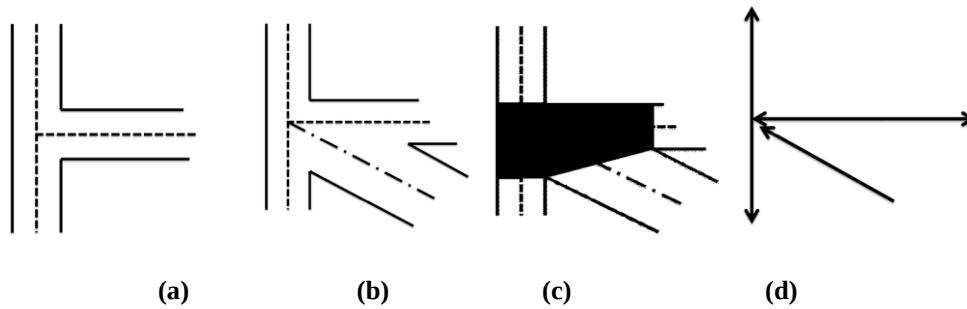
partially grouted wall, as, in a partially grouted wall, the total face shell thickness is the minimum area in the strut.

Although the ACI 318 and AASHTO LRFD design methods for strut and tie are similar, it is believed a combination of the two codes be used for masonry wall, along with the assumptions stated previously that are masonry wall-specific. The fact that resultants must meet at a point will define the area of struts that are other than diagonal. In a masonry wall, the corner of the wall where the load is applied will have a tension tie, a diagonal strut, and a horizontal strut meet at a node. In some cases for walls that have many panels of reinforcement, a horizontal or vertical strut could eventually control over the diagonal strut at certain locations as applied loads are increased and loads are shifted around in the structure as tension ties reach their yield strength (this was observed in Visual Analysis modeling presented in following sections). The size of horizontal struts of this type is limited to height of a bond beam. Therefore, for a wall with a single course bond beam, the strut area is relatively small. In many cases, this horizontal strut will not control, but when it does, its strength is recommended to be equal to twice the strength of an adjoining diagonal strut, otherwise over-conservatism may result. The nodal configurations shown in Figure 4.8 are a few that one could encounter in a masonry wall.

#1.



#2.



#3.

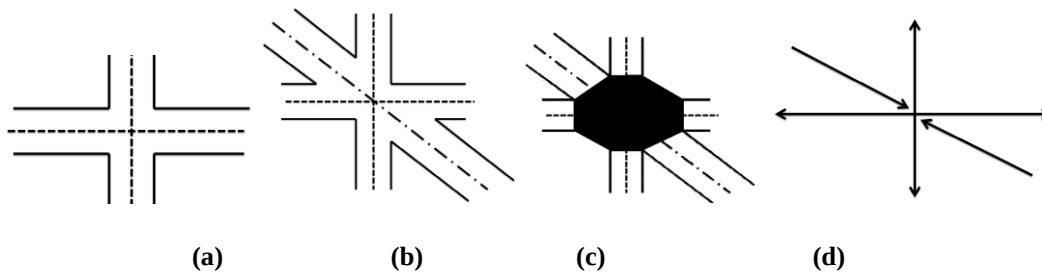


Figure 4.8 Typical Nodal Zones Found in a Masonry Wall, with a) region of interest, b) assumed strut location, c) assumed nodal zone, and d) idealized forces

The criterion for strength is borrowed largely from both the ACI-318 and AASHTO sections on strut-and-tie design, as they are similar in nature. One major difference in the two codes is the calculation of the strut strength. ACI-318 simply uses 85% of the concrete compressive strength for a prismatic strut, whereas AASHTO considers the principle tensile

strain in the concrete. ACI-318 uses the lowest effective compressive strength between a node and a strut to calculate the governing strength of struts. For a prismatic strut in a fully grouted wall, generally the nodal stress criterion will control, but in the case of a partially grouted wall, the controlling area will be the strut. In the proposed strength criteria, the lesser of the strut and node strengths should be used in a partially grouted wall. This is a deviation from taking the lesser of the node and strut effective stress as in the ACI-318. In the AASHTO method, the process is much more complicated in that the stress in the steel at the location of coincidence of the forces at a node must be determined. This process could prove tedious in the case of ultimate strength analysis where loading is unknown, in which a strut-and-tie model is incrementally loaded until failure. In this scenario, stresses in the steel would continually change, and thus principal stresses in the concrete continually change. This way, the strength of struts is continually changing and must be re-evaluated at each load increment. Additionally, using the AASHTO method requires an assumption of how to treat hooked bars such as the standard hooks found in a masonry wall. It seems that values of the strain in the steel are assumptions based upon where the strut crosses the tension tie (shown in Portland Cement Association, 2004) and assuming zero strain at the point of tangency in a hook, even though the steel may or may not have this exact strain at the desired location. It is possible to use a method such as this in masonry walls, but it is believed that this method deviates little from the ACI-318 version when the global strength of the wall is considered. In the proposed procedure for masonry, the strut is assumed to be anchored by the vertical reinforcement. Therefore, the vertical bars at the top of the wall should be developed at the point of intersection of the assumed strut to help ensure a more truss-like behavior. In traditional bending theory, there is little stress in the reinforcement at the top of the wall, and therefore many of the examined walls in the

database have vertical reinforcement that is terminated at the top of wall. The lack of anchorage of the vertical reinforcement did not seem to affect the prediction of wall strengths.

One additional area of concern is the angle that is used in the calculation of the strut size. The ACI-318 prohibits this angle from being less than 25 degrees due to the incompatibility of the shortening of struts and lengthening of ties in almost the same direction, whereas the AASHTO LRFD does not limit this angle, but instead reduces the effective compressive stress allowed in the strut as this angle becomes smaller. In a masonry wall utilizing the proposed method, many strut angles are definitively determined by required and conventional reinforcement spacing. It is possible to allow a strut to cross several ties in the case of very closely spaced horizontal reinforcement, however, if a strut is assumed to act between every steel panel in some cases a very steep or very shallow angle will result for close reinforcement spacing. To keep the inclination of struts between 25 and 65 degrees would be ideal, but it is believed this restriction is not required in a masonry wall, as long as an appropriate strut and tie arrangement is formed and the strut can adequately fit in the panel—that is to say, develop half of its width on either side of the centerline of the strut before encountering the bounding reinforcement in a horizontal or vertical tie or strut. A relaxed angle requirement is used in the following section for analysis, but care is taken to ensure that the strut will fit inside the panel. If this method is not used, over-conservatism may result.

4.7 Visual Analysis Wall Modeling

A procedure similar to Nolph's (2010) was used to verify the assumptions made previously by way of the structural engineering software Visual Analysis. A total of 35 fully grouted and 23 partially grouted walls consisting of concrete, clay, and AAC units were taken from the various available data sets and assigned a strut-and-tie layout according to location of known reinforcement, if possible. All structures were modeled as pinned plane trusses in two dimensions. All members were assigned an arbitrary shape from the database, with the "weightless steel" material feature in Visual Analysis, as only member axial forces were desired. Axial load was distributed evenly at the nodes at the top of the wall based upon tributary area, and all loads were applied at the joints, with the primary in-plane load being applied at the top left corner of the wall. Strength of ties and struts were calculated according to Equations 4.8-10 and each model loaded until either the model became unstable, deflection exceeded 0.05 times the largest dimension of the structure (default warning in Visual Analysis), or the masonry failed in crushing. Reinforcement that yielded before failure of the structure was removed and replaced with an equivalent force equal to its yield strength coming out of its associated nodes and loading continued until one of the failure events above occurred. Since failure loads and steel placement were known for all walls, a general layout was assumed for each wall based on struts crossing each panel of bounding steel. If this arrangement was unstable prior to loading or proved too conservative, another layout was attempted. Appendix B is a list of the investigated walls with various criteria as it pertains to calculations using the proposed methods. Also listed is the first yield (if applicable) and failure loads as found in Visual Analysis. Where odd wall dimensions

or unconventional steel spacing were given in the research data, spacing of reinforcement was assumed as closely as possible, with measurements being assigned as center-to-center of reinforcement. Specific wall dimensions and reinforcement spacing can be found for each individual wall in Appendix A. For some walls, only the ungrouted or grouted masonry compressive strength was given in the data, and therefore, was used to calculate the node and strut strengths (normally, grouted f'_m would be used for node calculation for both fully and partially-grouted walls, and ungrouted f'_m would be used for struts in partially-grouted walls). It is assumed, unless noted otherwise in the research data, that values of compressive strength given for fully grouted walls are considering fully grouted test prisms, and compressive strength values given for partially grouted walls are considering ungrouted test prisms. Only the compressive stress corresponding to the controlling strength is shown and β_n values are shown even if the nodal strength criterion did not control. For some walls, the analysis proved too conservative, and if so, a different strut arrangement was assumed and appropriate β_n assigned according to the assumption. Additionally, some walls have different lengths for assumed struts in the same wall, and these values are given in the table if applicable. In Appendix F are representations of all 58 walls in their initial state with assumed strut locations, and their final state at failure showing all load-replaced bars and deleted struts.

An example wall presented in this section will be one from Nolph (2010). The layout is such that vertical and horizontal members coincided with actual steel locations in the wall, with inclined members being the assumed struts. The in-plane load was incremented and respective forces in each member were recorded until failure occurred. Other walls were analyzed in a similar fashion, but only the initial and final steps will be shown in Appendix F. Actual failure

pictures from other researchers are shown in Appendix H if that data could be found. Table 4.1 gives some of the values of the calculations for PG085-48 from Nolph (2010).

Table 4.1—Strut-and-Tie Values for PG085-48

Researcher/Wall I.D	Type/AR	Vert. Tie Strength (k)	Horiz. Tie Strength (k)	θ	β_n	Fce (psi)	Strut Strength (k)
Nolph-PG085-48	PG/0.9	76.49	19.72	47.5 and 45	0.6	1394	37.57 (top) and 39.35 (bottom)

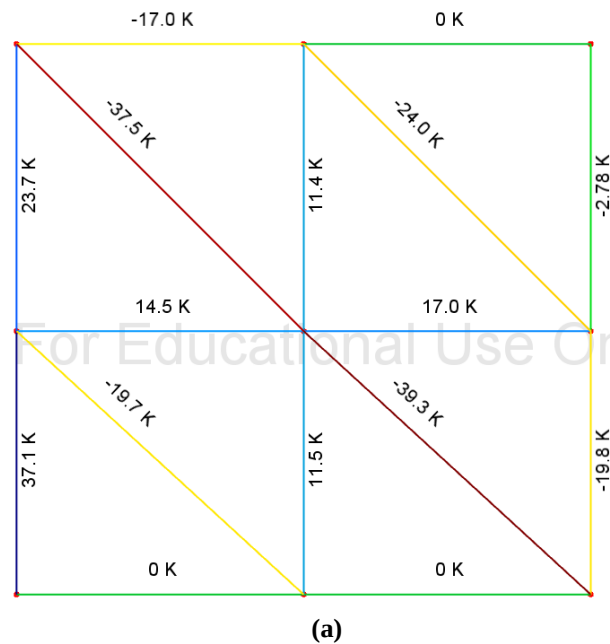


Figure 4.9 PG085-48 Failure at 43.5 k

Wall PG085-48 failed by crushing of the diagonal strut in the top left panel at an in-plane load of 43.5 kips in Visual Analysis. The actual failure load for this wall was 49.8 kips, giving a

$\frac{V_{exp}}{V_{pred}}$ of 1.14. Appendix B shows results from all 58 walls.

In using the proposed provisions along with the assumptions mentioned, it was found that good results could be obtained from the use of strut-and-tie principles for the 58 walls analyzed. The average $\frac{V_{exp}}{V_{pred}}$ for partially grouted walls was 1.49 with a COV of 0.28, while the fully grouted wall average was 1.64 with a COV of 0.25.

It should be noted that for all but 7 of the 23 partially grouted walls analyzed, the failure modes in Visual Analysis was crushing of a diagonal strut, while 15 out of 35 fully grouted walls analyzed produced a failure mode of crushing of a diagonal strut. The larger percentage of partially grouted walls failing due to crushing of a diagonal strut is expected since the effective strut area in a partially grouted wall is much less than the strut area in a fully grouted wall, and could control design.

Twelve fully grouted walls and 4 partially grouted walls necessitated having only one diagonal strut instead of a more conventional multi-panel strut-and-tie layout. The primary reason for doing this was to achieve an adequate failure load for the walls. If a layout was chosen that failed too quickly, generally, using a single diagonal strut would yield a more favorable failure load for that wall. Otherwise, the layout of struts and ties adhered to the actual reinforcement layout as much as possible.

In some cases, no vertical reinforcement was present in the test wall. This is not likely to be encountered in an actual reinforced wall, as there would have to be reinforcement present to prevent overturning and for a flexural design. However, modeling bounding steel for stability in the Visual Analysis program seemed to be an adequate way of analyzing walls such as these.

Some interesting trends were noted in analyzing the strut-and-tie specimens. Figure 4.10 gives various representations of walls that show areas of tie removal that are roughly 45 degrees. This reinforces the notion that struts tend to form at 45 degree angles, regardless of the

dimensions of the wall. Similar trends were noted in the finite element analysis presented in the next chapter.

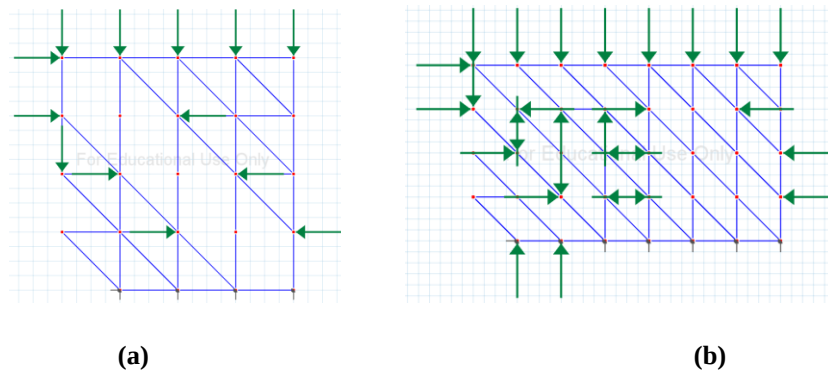


Figure 4.10 Showing 45 Degree Areas of Tie Removal for (a) Shing #16 and (b) Voon A10

A design example is presented in which a partially grouted wall is 10 ft. high, 16 ft. long and utilizes 8 in. concrete masonry units. There is a superimposed dead load of 1 kip/ft. and reinforcement is gr. 60 with f'_m of 1500 psi. Vertical reinforcement is 2 #5 bars at each end cell, with a #5 bar at 48 in. otherwise (one spacing will be 40 in.) Flexural capacity according to the 2011 TMS-402 strength design equations gives a design capacity of 101 kips for maximum in-plane loading. TMS-402 strength design equations for shear require #5 horizontal reinforcement bars at 32 in. This wall is represented in Figure 4.11, part (a). Additionally, there are shear provisions proposed in the 2013 TMS-402 code (Public Comment Review Version, 2012) that utilize a factor of 0.75 on all limits and individual contributors to shear strength. The proposed 2013 strength design equations for shear require #6 horizontal reinforcement bars at 24 in.

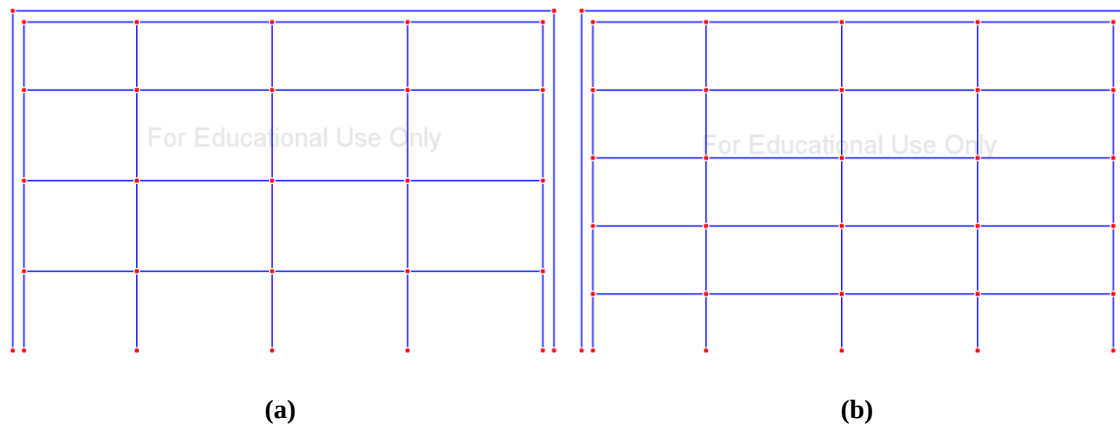


Figure 4.11 Wall designed with (a) 2011 TMS-402 (b) Proposed Strut-and-Tie and Proposed 2013 TMS-402

The wall from Figure 4.11 part (b) was analyzed with the proposed strut-and-tie method initially with only vertical reinforcement and a top course bond beam, while adding an extra horizontal reinforcement bar at a closer spacing for each additional analysis. After several iterations of utilizing #5 bars and various spacing, the reinforcement size was changed to a #6 bar. The final arrangement was #6 bars at 24 in. to give a capacity of approximately 101 kips. The extra reinforcement requirement of the proposed method is due to the unconservatism of the 2011 TMS-402 equations for partially grouted walls and the relatively low masonry compressive strength. Reinforcement requirements match exactly with that of the proposed 2013 TMS-402 equations. Figure 4.12 gives the capacity of the wall as ρ_h increases for the proposed strut-and-tie procedure. Point #5 in Figure 4.12 failed due to instability from removing the yielded ties, and thus created a lower capacity than trial #4, where crushing of the masonry controlled.

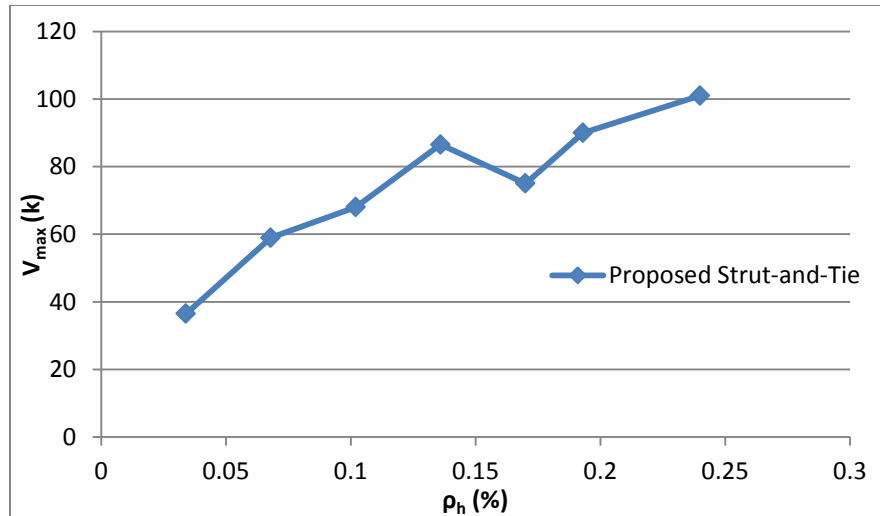


Figure 4.12 Maximum Shear Strength vs. Horizontal Reinforcement Ratio for the Proposed Strut-and-Tie Method

The procedure of using the ACI-318 introduced by Nolph (2010) has been modified as in the aforementioned paragraphs. It is believed that the combined methods of the ACI-318 and AASHTO LRFD proves to be a sound method for calculating the in-plane strength of both partially and fully grouted masonry walls. The proposed strut-and-tie provisions are all-encompassing in the sense that individual contributors to strength are not isolated—no information is given on contributions of masonry, steel, and axial load to the overall shear strength. In all likelihood, a designer would not design the wall to fail in shear, but would instead design the wall to fail in flexure first, as it is a more ductile and desired mode of failure. This would give one the locations of vertical reinforcement and it would be up to the designer to assume an appropriate strut layout through both experience and iteration to achieve a conservative value for the in-plane strength of the wall using this strut-and-tie method.

In Chapter 5, a finite element analysis is presented to extend the scope of the use of the proposed strut-and-tie provisions.

Chapter 5—Use of Finite Element Analysis to Extend Experimental Scope

5.1 Finite Element Analysis Introduction

Due to the lack of data representing aspect ratios of less than 0.5 and greater than 2.0, finite element analysis was used to investigate walls and determine the applicability of strut-and-tie methods for these walls. In typical masonry structures there are rarely just square sections of wall, but rather segments that can be very squat (for example, a retail store), or very slender (such as a slender masonry pier between openings).

Abaqus/CAE Finite Element Software was used to calibrate a square wall, and then to extend the scope of this research by using the calibrated material properties and known behaviors of the model wall to generate reasonably accurate representations of walls with high and low aspect ratios and with different reinforcement layouts than those in the available data.

There are generally two approaches to modeling masonry—micro-modeling and macro-modeling. Micro-modeling involves modeling each constituent and their interaction precisely to provide the means of local and detailed analysis of the masonry structure. This could include individual representations of blocks, mortar, grout, steel, and all of the interfaces and interactions between them. Macro-modeling seeks to quantify the overall behavior and interaction of the constituents, with less focus on each individual material. In this type of modeling, constituents such as grout and mortar are often smeared together with the blocks, in effect creating one material. This is a good example of using masonry prism strength data for the material properties—in the prism, most of the constituents are present, therefore the behavior of the prism can be used in a macro-model of the wall. (Akhavessy et al, 2011)

A more intricate description of modeling types and procedures for each can be found in Lourenco (1996), while referenced examples of both micro- and macro-modeling as applied to

masonry can be found in Minaie (2009). Figure 5.1 gives a representation of traditional micro- and macro-models. A macro-model approach will be used for this study.

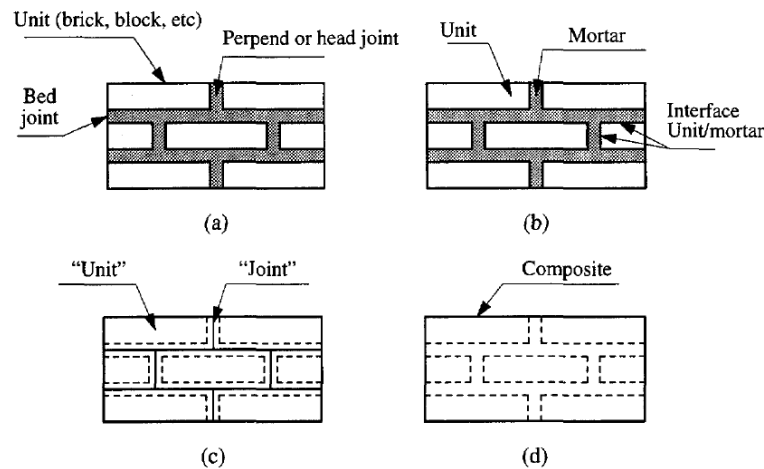


Figure 5.1 (Lourenco—1996) FEA Modeling Types: a) masonry sample, b) detailed micro-modeling, c) simplified micro-modeling, d) macro-modeling

In his research, Minaie (2009) utilized Abaqus/CAE to develop a finite element model to validate his experimental results. He created partially and fully grouted walls without mortar joints that contained individual blocks composed of shell and solid parts shown in Figure 5.2.

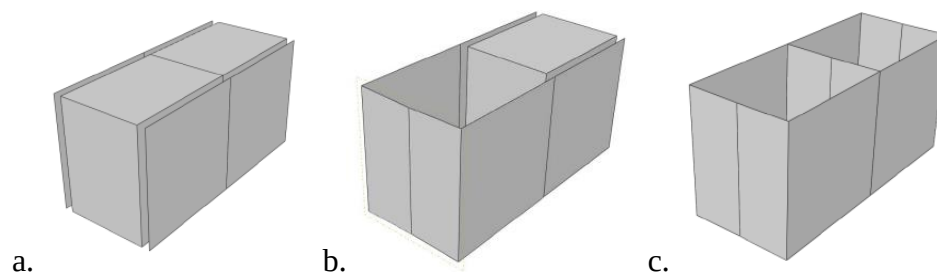
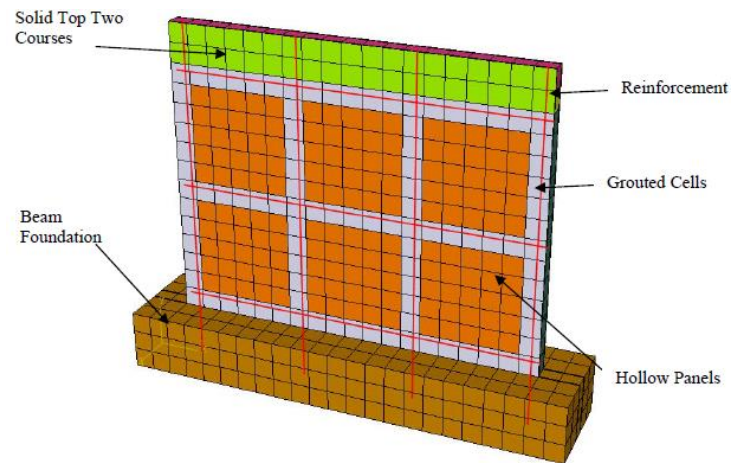
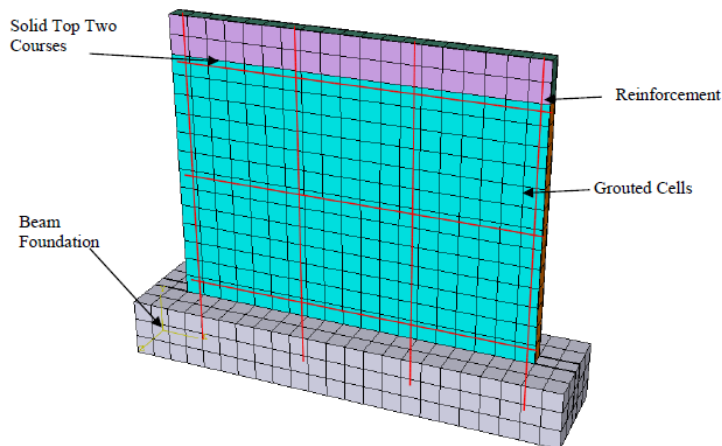


Figure 5.2 (Minaie, 2009) Block Configuration: a) fully, b) partially, and c) hollow concrete blocks

Reinforcement was provided using frame elements, and all the constituents were jointed together in Abaqus using the tie constraint and embedment features to create the walls shown in Figure 5.3.



(a)



(b)

Figure 5.3 (Minaie, 2009) Created Walls: showing a) partially and b) fully grouted walls

Analysis was performed with Abaqus/Explicit using dynamic loading and 8 in. x 8 in. element sizes. It is unclear whether linear or quadratic elements were used, as both are investigated in Minaie (2009), but no clear distinction was made as to the final choice. The Concrete Damaged Plasticity model was used to model the brittle components and material

properties relating to this model and these specimens are presented in Appendix C, as general trends from these properties were used in the current analysis.

5.2 Model Wall

To select a model for calibration, a fully grouted square wall with no axial load was sought in order to isolate the in-plane strength with fewer factors to consider such as grouting scheme and axial load. A wall from Shing et al (1990) with the designation “Shing #4” was selected based upon this criteria and the fact that it failed in shear with documented diagonal cracking. In addition, a good account of key events in the wall in terms of loads and displacements exists so as to aid in calibration of the model.

The model selected from the database is shown in Figure 5.4 along with all given material properties in Table 5.1.

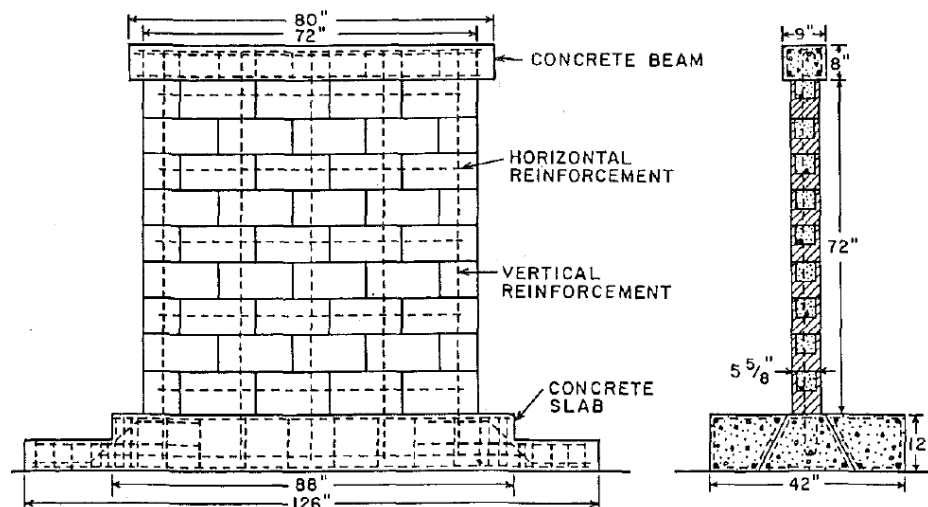


Figure 5.4 (Shing et al, 1990) Model Wall

Table 5.1—Properties of Model Wall

Wall	Vertical Steel/Strength	Horizontal Steel	Axial Stress	Prism Strength	Mortar Strength	Failure Mode
Shing #4	5 #7 Yield: 72000 psi Ultimate: 103000 psi	5 #3 Yield: 56000 psi Ultimate: 81000 psi	0 psi	2600 psi	3000 psi	Shear

While the element size and element type for this type of analysis has been established by Minaie (2009), a small study was undertaken to determine the difference in linear and quadratic elements and seed size for the desired model. As seed size and element type determine computation cost (in time and computing effort), it is desired to use a seed and element type that will capture all of the desired behavior, but will be efficient enough so that multiple analyses will be possible. To achieve this, the wall from Table 5.1 was built in Abaqus/CAE 6.11-1 without reinforcement and using only elastic section properties considering a 6 in. thickness. A slender and squat wall were constructed using the same parameters to determine any variance in accuracy.

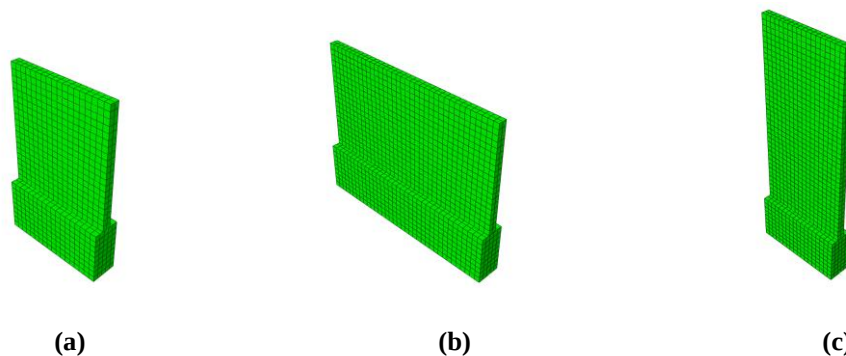


Figure 5.5 Model Wall in Abaqus with (a) square, (b) squat, and (c) slender walls

Displacement controlled loading was used with a 1 in. displacement at the top of the wall to determine the load that would cause this deflection for both linear reduced integration (C3D8R) and quadratic reduced integration (C3D20R) elements and for seed sizes ranging from 2 in. to 16 in. These values were compared to an engineering mechanics approach of deflection calculation for a cantilever beam (Equation 5.1) and an optimum seed and element type were chosen.

$$\Delta_{max} = \frac{PH^3}{3E_m I} + \frac{PH}{GA_v} \quad (5.1)$$

Where,

Δ_{max} = horizontal deflection at the top of the wall

P = load causing the deflection

H = height of wall

E_m = elastic modulus for masonry, taken as $900f'_m$

I = gross moment of inertia in the plane of load application

G = shear modulus, taken as $\frac{E_m}{2(1+\nu)}$ where $\nu = 0.2$

A_v = area considered for shear, taken as length of wall multiplied by the thickness

Figure 5.6 gives the values of applied load calculated by Abaqus/CAE as compared with the 1720000 lb. estimate computed with hand calculations for the square wall.

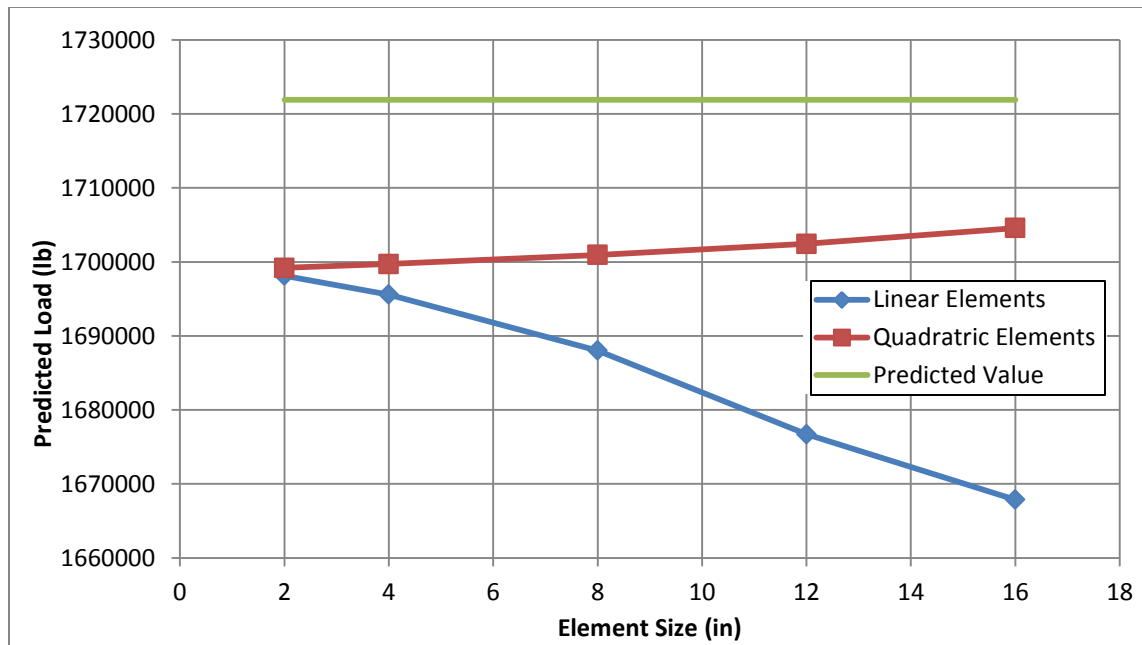


Figure 5.6—Element Type vs. Element Size

As the element size decreased (generally providing for a more accurate analysis), both the quadratic and linear elements converged on a particular solution. The percent difference of the value using linear elements with a 4 in. seed and the hand-calculated value is only 1.5%. For the slender and squat walls, the predicted vs. Abaqus/CAE values differed by 1.5% and 0.25%, respectively. It was deemed that linear elements with a seed size of 4 in. would provide adequate efficiency in later analyses. This is in comparison to an 8 in seed size used by Minaie (2009). It is believed that with a 4 in. seed, visualization of results are much improved.

Having established an appropriate element type and seed size, the wall was built using the GUI in Abaqus/CAE 6.11-1 with a smeared approach—essentially creating a solid wall with properties specific to the model masonry wall. This was deemed appropriate, as many studies of this kind use a smeared approach, and the key parameter, the masonry compressive strength, is itself a smeared value, representing the compressive strength of an assemblage of blocks, mortar, and grout. It is possible to represent each constituent of a masonry wall and “build” the wall

virtually, however, over-constraint issues tend to be a problem, and for the global response of the structure (and this study) is not needed. Representations of all parts created in Abaqus are shown in Figure 5.7.

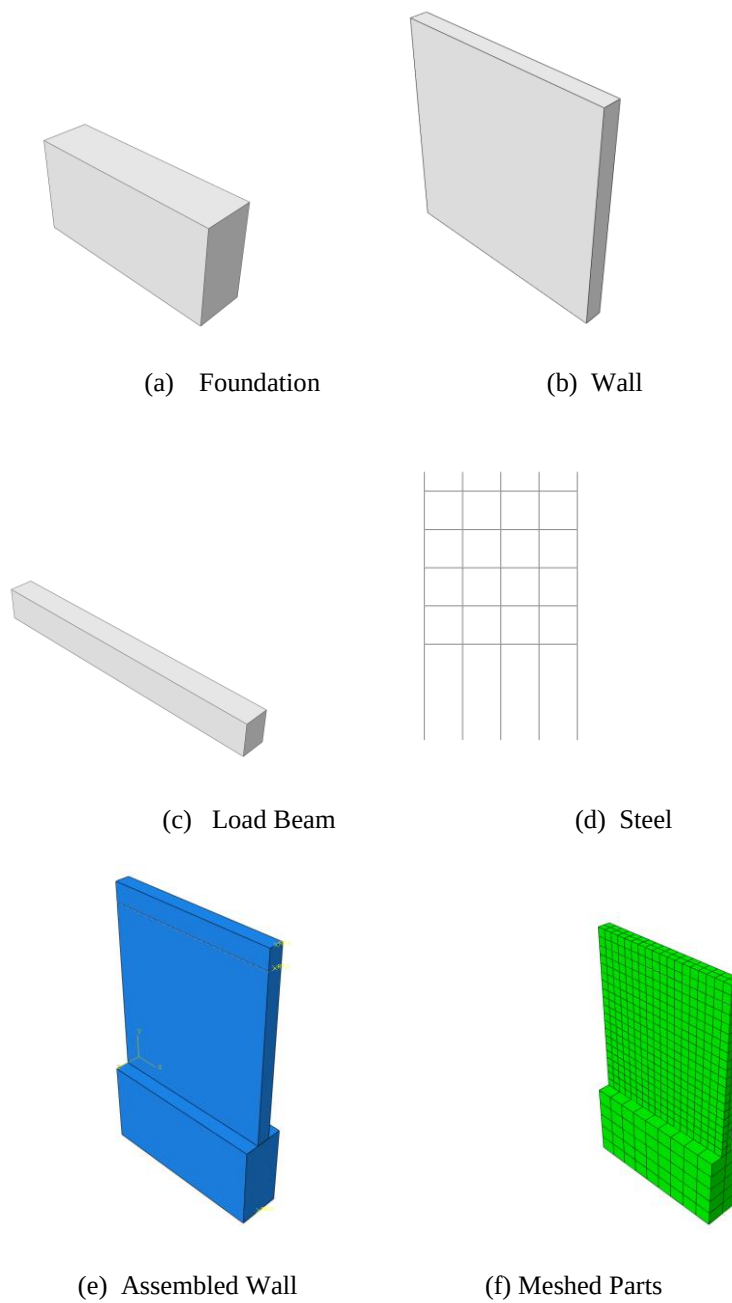


Figure 5.7—Parts Created in Abaqus

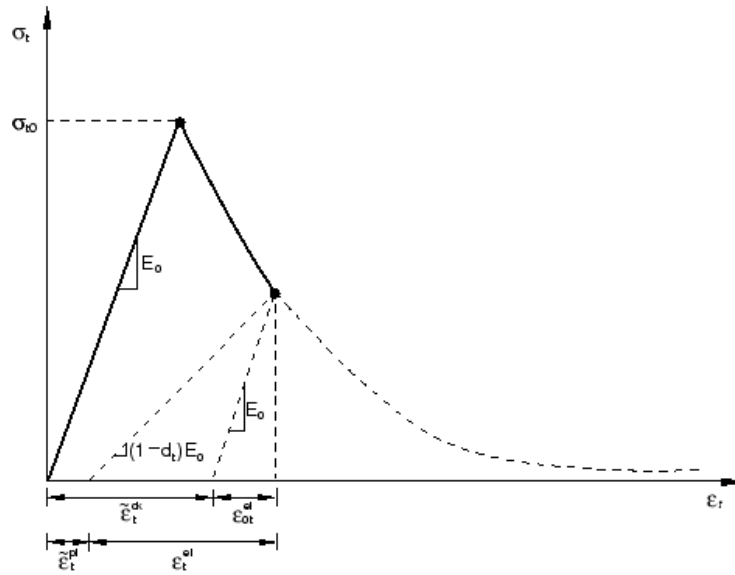
All solid elements were constrained together using the “tie constraints” function, whereas the rebar were placed using the “embedded region” constraint. A linear 8-node element with reduced integration (C3D8R) was used for the wall, foundation, and load beam, while a first-order three-dimensional beam element (B31) was used for the reinforcement. The loading beam and foundation were constrained as rigid bodies. Reinforcement was tied to the masonry through the embedded region constraint—this assumes perfect bond between the reinforcement and masonry material.

A prescribed displacement was applied to a reference node on the loading beam to simulate an actuator in a testing lab. The loading procedure used was an implicit, quasi-static loading that used a ramp feature to increase the displacement from a predetermined small initial value up to the full displacement in one specified direction. This method neglects inertial effects and was chosen over a dynamic cyclic loading, as for the purposes of this research only the global response and in-plane capacity were of interest. One-directional loading was chosen so that static analysis software Visual Analysis could be used to model the strut-and-tie procedure of Chapter 4.

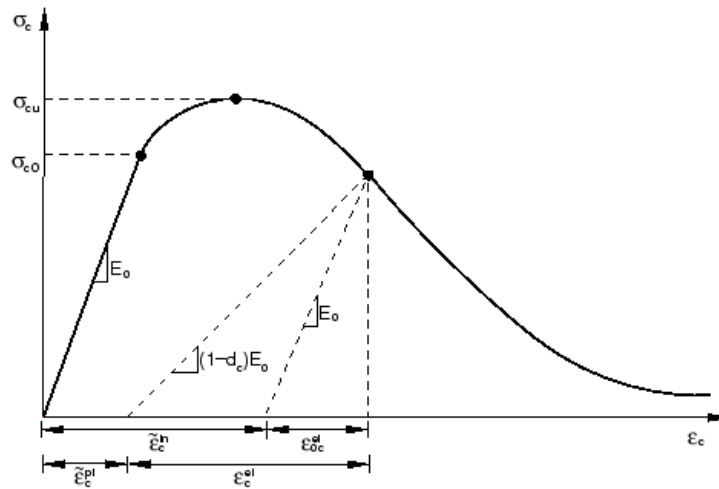
5.3 Material Model and Calibration

The Concrete Damaged Plasticity (CDP) model was used in Abaqus to model the masonry materials. The CDP model is a continuum-based plasticity model with failure modes of tensile cracking and compression crushing. The material model uses multi-hardening plasticity and isotropic damaged elasticity to represent the damage that occurs during cracking. (Abaqus Analysis User’s Manual)

The model assumes uniaxial tension and compression behavior to follow the general curves in Figure 5.8.



(a)



(b)

Figure 5.8 (Abaqus Analysis User's Manual) Material Curves: a) tension b) compression

Where,

E_0 = Initial undamaged elastic stiffness

σ_{t0} = tensile failure stress

σ_{c0} = initial yield stress

σ_{cu} = ultimate stress

$\epsilon_{c,t}^{pl}$ = equivalent plastic strains for compression (c) and tension (t)

$\epsilon_{c,t}^{el}$ = elastic strains for compression (c) and tension (t)

$d_{c,t}$ = damage variables for compression (c) and tension (t)

ϵ_t^{ck} = cracking strain, and is defined by $\epsilon_t^{ck} = \epsilon_t - \epsilon_{0t}^{el}$

ϵ_{0t}^{el} = elastic strain corresponding to the undamaged material, and is defined by $\epsilon_{0t}^{el} = \frac{\sigma_t}{E_0}$

The tensile and compressive stresses, respectively, are calculated as follows:

$$\sigma_{t,c} = (1 - d_{t,c})E_0(\epsilon_{t,c} - \epsilon_{t,c}^{pl}) \quad (5.2)$$

Modeling of reinforcement in Abaqus consisted of using 3D deformable wires along with elasticity models. These can be embedded or defined singly in Abaqus, and bond and slip can be modeled by introducing tension stiffening into the tension curve for the masonry to simulate transfer of loads across cracks. The post-failure strain softening (and thus, tension stiffening) can be defined according to Figure 5.8. Tension stiffening data is required in Abaqus and is input in terms of the cracking strain. These values are converted by Abaqus to plastic strain and used in the analysis.

$$\epsilon_t^{pl} = \epsilon_t^{ck} - \frac{d_t}{(1-d_t)} \frac{\sigma_t}{E_0} \quad (5.3)$$

Abaqus recommends a starting point for tension stiffening data for heavily reinforced members of reducing the stress linearly to zero at a constant rate of about 10 times the strain at failure.

Defining compression data is similar to defining tension data, in regards to the input being inelastic strain which is converted to plastic strain by Abaqus according to Equation 5.4 and Figure 5.8.

$$\epsilon_c^{pl} = \epsilon_c^{in} - \frac{d_c}{(1-d_c)} \frac{\sigma_c}{E_0} \quad (5.4)$$

As with the tension input, if calculated plastic strain values are negative or decreasing, an error message will ensue, generally meaning that the damage curves are incorrect. Stiffness recovery factors are present in each data table for compression (w_c) and tension (w_t) damage, and are taken as a default to be 1.0 and 0.0, respectively. These assumptions correspond to compression stiffness recovery upon crack closure and loss of tensile stiffness with the occurrence of micro-cracks.

In addition to the stated requirements, the CDP model also requires the use of the parameters in Table 5.2.

Table 5.2—Parameter Definitions (Abaqus Analysis User's Manual)

Dilation Angle-- Ψ	Measured in the p-q plane. In degrees.
Eccentricity-- ϵ	Defines the rate at which the hyperbolic flow potential approaches its asymptote. Default = 0.1
fb0/fc0	$\frac{\sigma_{b0}}{\sigma_{c0}}$ = the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress. Default = 1.16
K	The ratio of the second stress invariant on the tensile meridian to that on the compressive meridian at initial yield for any given value of the pressure invariant such that the maximum principal stress is negative. Must be between 0.5 and 1.0. Default is 0.667
Viscosity Parameter-- μ	Used for the visco-plastic regularization of the concrete constitutive equations in Abaqus/Standard analyses. Default is 0.0

The only material properties available were those listed in Table 5.1. From these, material behavior curves were generated as closely as possible to expected behavior, and then adjusted if necessary to match the desired response from Abaqus. The default values from Table 5.2 were also used, with the exception of the dilation angle which required trial and error to find a suitable value. A value of 5 degrees was found to give reasonable results and is comparable to that found in Lourenco et al (2004). As a comparison, typical concrete can have a dilation angle of 30-40 degrees.

Figure 5.9 is a curve that was used as a guideline for the creation of the compression material curve used for the model wall in Abaqus.

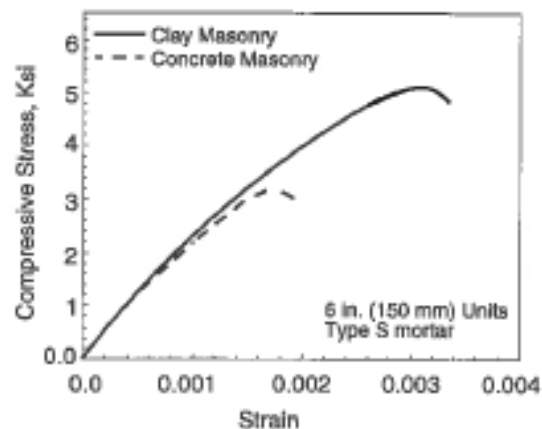


Figure 5.9 (Drysdale, et al 1999) Concrete Masonry Compressive Strength

Key points were noted on this curve and applied to the prism compressive strength of Shing's wall #4, 2600 psi, to generate a rough curve. Modulus of elasticity was found by the TMS-402 equation of $E_m = 900f'_m$ for concrete masonry giving a value of 2340000 psi, while 50% of the maximum compressive strength was used as the practical limit for elastic linearity (Drysdale, et al 1999). Failure was assumed to be at a stress of 20% below the peak with a corresponding strain value of 33% greater than ultimate (0.004). The rough curve was then

smoothed using a third degree polynomial fit in JMP Statistical Analysis software and is shown in Figure 5.10. Abaqus requires that a material curve be fully populated, or a constant stress with increasing strain would ensue. An idealized post-failure behavior of approximately 4 times the strain at failure with linearly decreasing stress was plotted to give the material curve that was input in Abaqus/CAE in Figure 5.10.

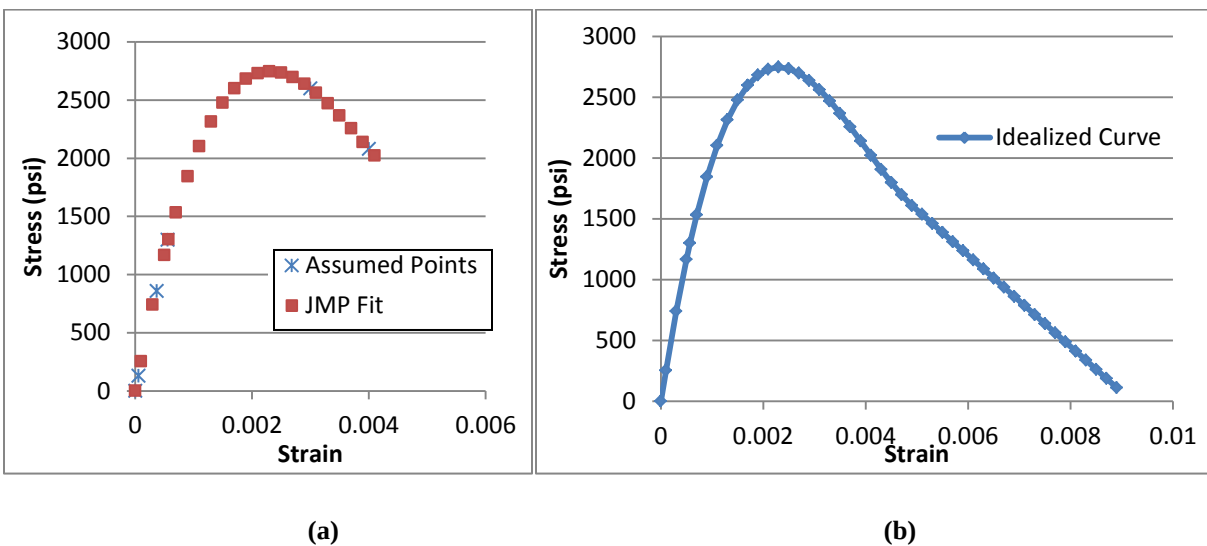


Figure 5.10 (a) Smoothed and (b) Idealized Curves for Masonry Compressive Strength

The idealized curve was utilized with the assumption that it represents true stress and strain in a test prism. The peak stress was 2746 psi as opposed to 2600 psi for Shing #4. The value of 2746 psi was used as the masonry compressive strength in all analyses relating to the extension of scope. The tensile curve was created to follow the curves presented in Abaqus in Figure 5.8. A peak stress was calibrated to have approximately 11% of the prism compressive strength; the value where the model performed the best as compared to Shing #4. Post-peak behavior loosely followed the recommendations of Abaqus documentation with a roughly linear

degradation to 5.75 times the strain at peak stress. This is in contrast to a value of 10 recommended for heavily reinforced concrete members. The resulting tensile curve is presented in Figure 5.11.

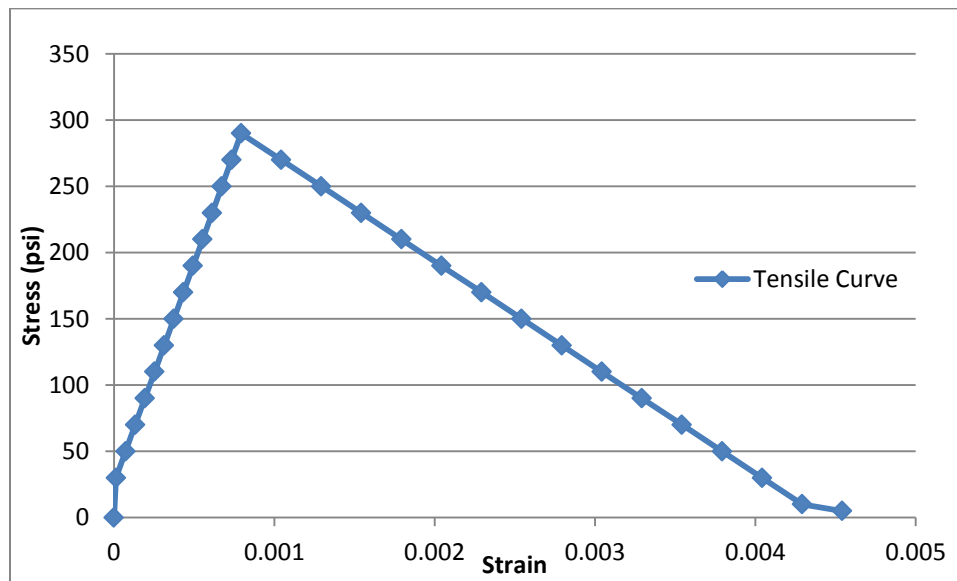


Figure 5.11—Tensile Behavior Curve

One interesting occurrence in this analysis and that was also found by inspecting Miniae’s (2009) material data, is that it was necessary to define the yield stress in tension early in the curve, otherwise behavior of the model did not coincide with that of Shing #4. As can be seen in Figure 5.11, the calculated modulus of elasticity was used up to a stress of about 30 psi, then a portion exhibiting reduced stiffness was used up to the peak load. It is believed that this represents when masonry joints start to crack, leading to reduced stiffness in the wall.

Steel reinforcement behavior was modeled as elastic-perfectly plastic using the reported yield stress for both horizontal and vertical reinforcement. Abaqus requires true stress inputs (force per current area), but reported yield stress was used instead (a difference in “true” yield

stress of less than 0.20%). Abaqus interprets the last input value to be a constant stress at constantly increasing strain. These curves are shown in Figure 5.12.

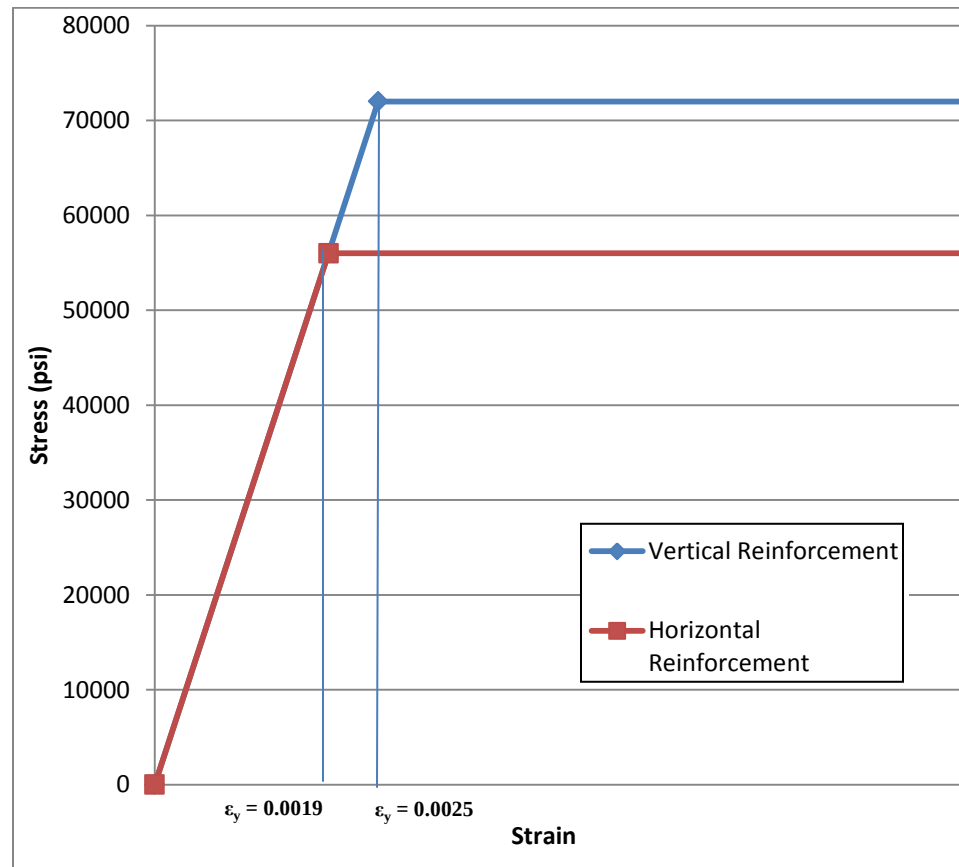


Figure 5.12—Horizontal and Vertical Reinforcement Stress-Strain Curves

In comparison to Shing #4, the virtual wall built in Abaqus performed reasonably well. Shing #4 was tested under cyclic loading, however key points of load and displacement were available. The virtual wall was displaced 1.0 in. and the reaction force measured at the point of load application (in Abaqus, total force reactions can be had through a node that is assigned a boundary condition, such as a displacement). Figure 5.13 shows data plotted for the ascending

portion of a load-displacement curve for key points listed in Shing et al (1990), and those rendered in the finite element analysis.

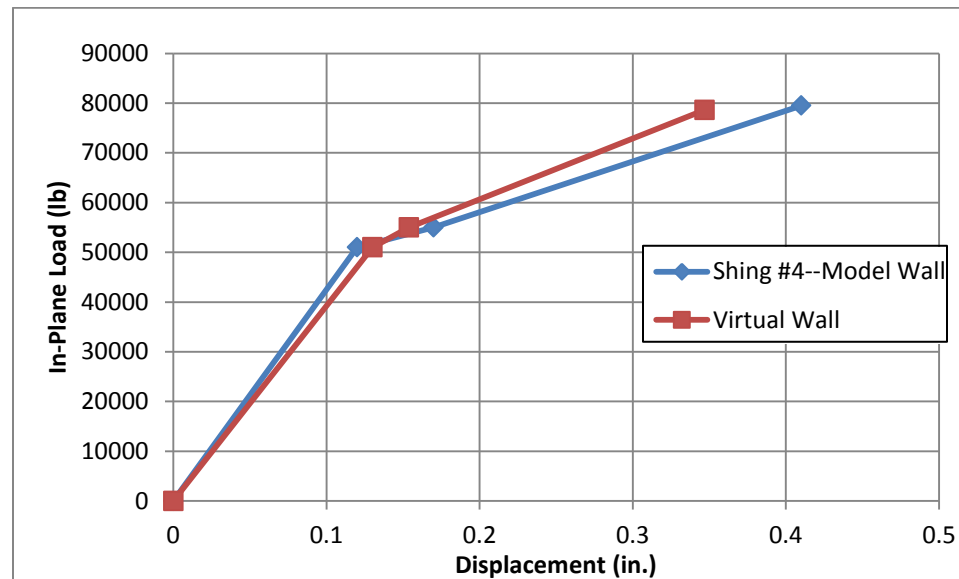
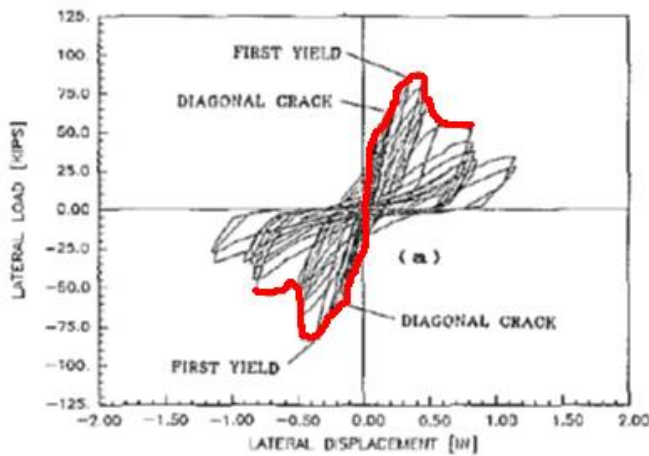
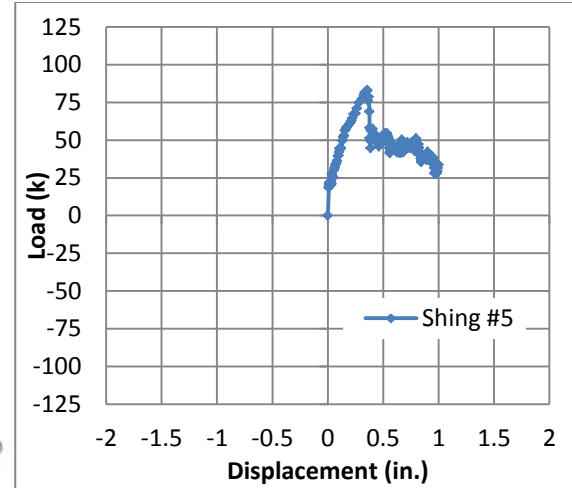


Figure 5.13—Partial Load-Displacement Curve for Model and Virtual Walls

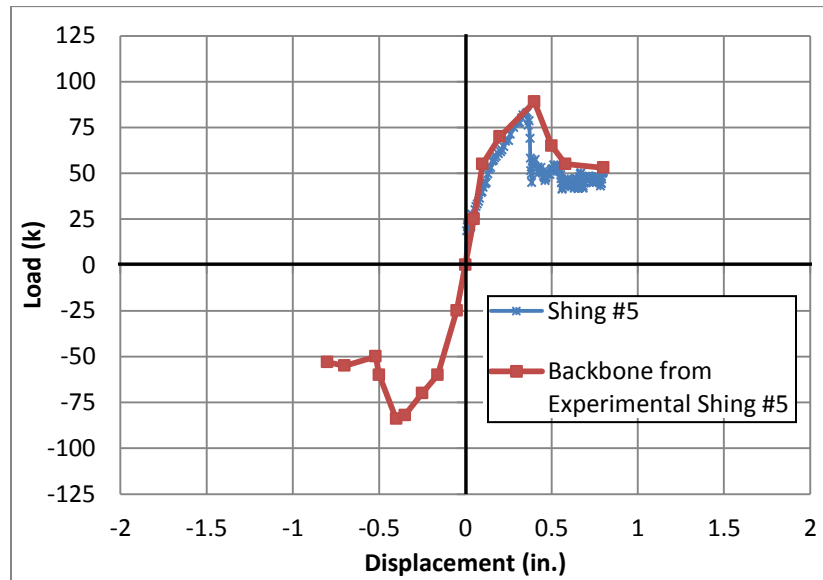
Averages of failure loads in both directions as reported by Shing et al (1990) were used as the final data point in Figure 5.13. A full load-displacement curve could not be found for Shing #4 for comparison, but a similar wall, Shing #5, has a hysteresis curve presented in Figure 5.14. Shing #5 had the same reinforcement as Shing #4, but with the addition of 100 psi axial stress. The virtual wall from Shing #4 was re-analyzed with 100 psi axial stress to generate a load-displacement curve for comparison. It can be seen that the behavior of the fully grouted wall is modeled well by the finite element analysis.



(a)



(b)



(c)

Figure 5.14 Load Displacement Curves for (a) Shing #5 (Shing et al, 1990) (backbone highlighted in red) and (b) Virtual Wall that models Shing #5, and (c) Overlay of Backbone onto Virtual Wall

Loads and deflections matched well for the virtual wall, but damage representation is not as good. In Shing #4, it was reported that diagonal cracks started to occur at 51 kips, with a major diagonal crack occurring at 55 kips. In the Concrete Damaged Plasticity model, crack representation cannot be viewed directly, but can be interpreted through the representation of

principal tensile and compressive strains in the plastic region. In the virtual wall, major diagonal cracking did not occur until just after peak loading. Instead the virtual wall displayed some cracking at the base of the wall with additional smaller plastic tensile strains oriented diagonally in the middle of the wall, suggesting minor cracking had occurred in the middle of the wall, with more damage along the base. This finding suggests that even though the model wall was tested as a cantilever in Shing et al (1990), perhaps there was enough axial load to prolong base sliding and to allow a diagonal tension failure to occur. It was found in future virtual walls that the addition of an axial load allowed the diagonal tension failure mode to occur before extensive base damage. Figure 5.18 provides the representation and orientation of plastic tensile strains in the virtual wall of Shing #4 at peak in-plane load.

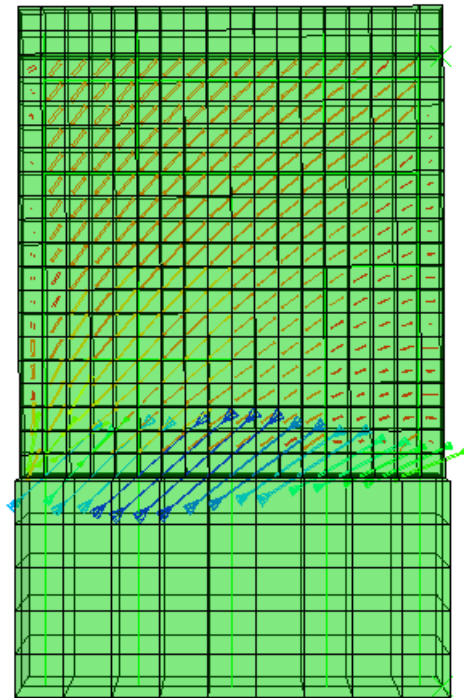


Figure 5.15—Orientation of Plastic Compressive Strains at Failure

The highest concentration of plastic tensile strain is at the bottom of the wall, with smaller strains throughout the middle of the panel. Loading was from right to left, providing for a compression field going from the top right corner to the bottom left corner of the wall.

The analysis of the virtual wall was deemed adequate to extend the scope to other wall sizes and to partially grouted walls. For partially grouted walls, this was achieved using the same material curves and reinforcement, with the assumption that the in-plane strength of a partially grouted wall could be modeled well with the same procedure (future analyses provided loads and deflections that are on par with partially grouted walls of similar size) and that the behavior of a partially grouted wall is reasonably modeled. There were no similar walls to compare the partially grouted analysis to for confirmation of behavior, as the material curves for Shing's walls were calibrated explicitly for those walls; comparison with a wall from another researcher, even with similar wall sizes and properties, would not be a like comparison.

To model the geometry, a reduced cross-sectional area was used amounting to twice the minimum face shell thickness in the regions with no grouting. Regions with grouting were created as solid sections and tied together using the same procedure as for fully grouted walls. This is represented by walls #1-PG and #2-PG in Figure 5.16.

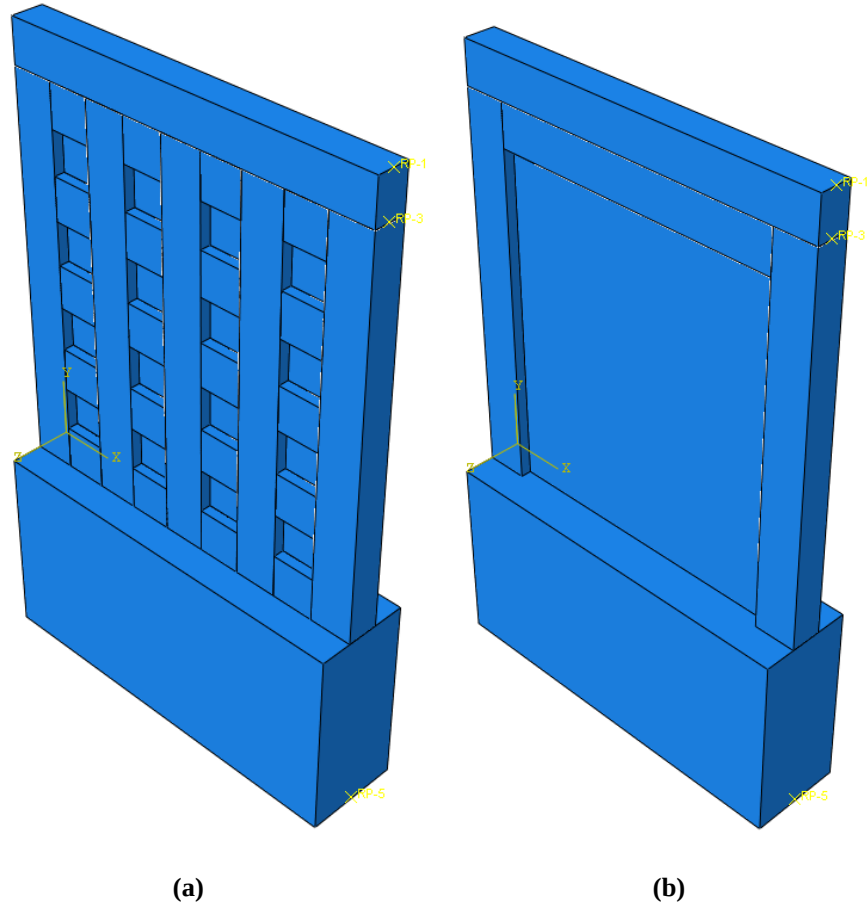


Figure 5.16 Partially Grouted Walls (a) #1-PG (b) #2-PG

5.4 Extension of Scope

The scope was extended by generating both fully and partially grouted walls, forcing them to fail in shear, and then comparing their peak in-plane loads to that calculated using the proposed procedures. Aspect ratios of 0.25 to 3.0, as well large and small square walls were created. Parameters such as size, thickness, axial load, and reinforcement details for all walls are shown in Appendix C.

All walls used the equivalent of #7 reinforcing bars vertically, and #3 reinforcing bars horizontally. Walls were 6 in. or 12 in. thick, fixed at the base to represent a cantilever, and were built using the same procedure as in Section 5.4. Walls #9-FG and #10-FG failed in flexure and are not reported. Appendix D displays all created walls, representations of steel arrangement, view of the plastic strain distribution in the wall at peak load (the only way to view “cracking” or “crushing” in the CDP model in Abaqus), a graph showing the distribution of the strain in a horizontal bar at peak load, and a load-displacement curve. Some typical results of the representations at peak load are given in Figure 5.17, with regions in blue and green representing regions of highest plastic compressive strain. A matrix showing the various parameters tested is given in Table 5.3.

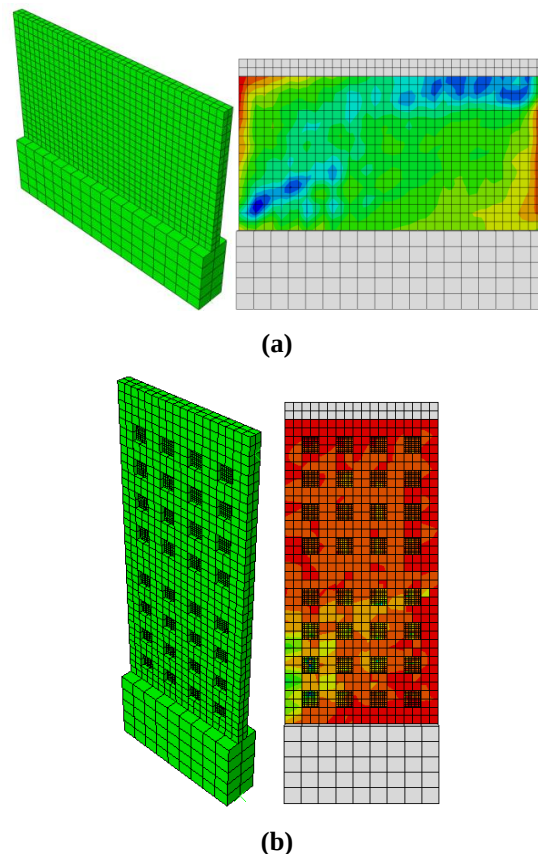


Figure 5.17 Typical Results at Peak Load for (a) #4-FG and (b) #7-PG

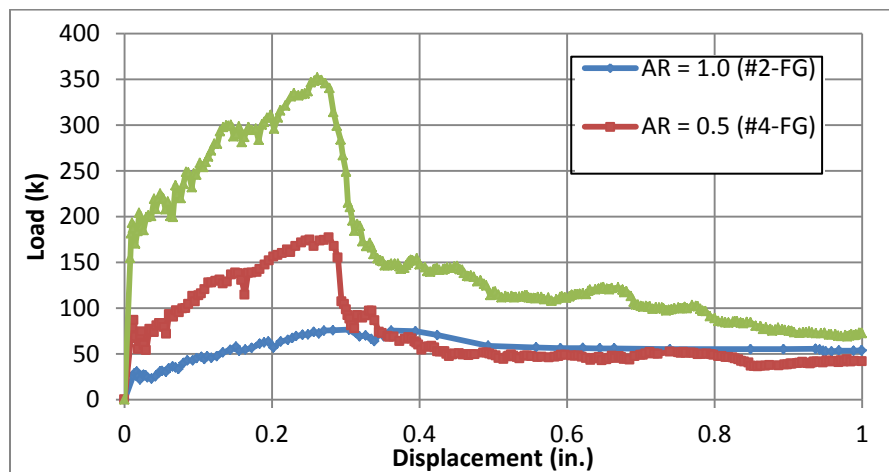
Table 5.3—Parameter Matrix

Matrix of Parameters					
Wall I.D	Aspect Ratio	Gross Axial Stress (psi)	ρ_h (%)	ρ_v (%)	Dimensions h x l x t (in.)
Fully Grouted Walls					
#1-FG	1.0	0	0.13	0.69	72 x 72 x 6
#2-FG	1.0	230	0.03	0.28	72 x 72 x 6
#3-FG	0.5	230	0.13	0.69	72 x 144 x 6
#4-FG	0.5	230	0.03	0.28	72 x 144 x 6
#5-FG	0.25	230	0.13	0.28	72 x 288 x 6
#6-FG	0.25	230	0.03	0.21	72 x 288 x 6
#7-FG	2.0	690	0.13	0.69	144 x 72 x 6
#8-FG	2.0	460	0.05	0.69	144 x 72 x 6
#11-FG	1.0	230	0.12	0.69	144 x 144 x 6
#12-FG	1.0	230	0.12	0.66	288 x 288 x 6
#13-FG	1.0	460	0.15	0.83	24 x 24 x 6
#14-FG	0.5	460	0.15	0.42	24 x 48 x 6
#15-FG	2.0	460	0.08	0.83	48 x 24 x 6
#16-FG	3.0	690	0.05	0.83	72 x 24 x 6
#17-FG	1.0	230	0.06	0.35	72 x 72 x 12
#18-FG	0.25	230	0.013	0.10	288 x 72 x 12
#19-FG	2.0	230	0.03	0.35	144 x 72 x 12
Partially Grouted Walls					
#1-PG	1.0	230	0.13	0.69	72 x 72 x 6
#2-PG	1.0	230	0.03	0.28	72 x 72 x 6
#3-PG	0.5	230	0.13	0.69	72 x 144 x 6
#4-PG	0.5	230	0.03	0.28	72 x 144 x 6
#5-PG	0.25	230	0.13	0.28	72 x 288 x 6
#6-PG	0.25	230	0.03	0.21	72 x 288 x 6
#7-PG	2.0	460	0.13	0.69	144 x 72 x 6
#8-PG	2.0	460	0.05	0.69	144 x 72 x 6
#9-PG	1.0	230	0.12	0.69	144 x 144 x 6
#10-PG	1.0	115	0.06	0.35	72 x 72 x 12
#11-PG	0.25	230	0.013	0.10	72 x 288 x 12
#12-PG	2.0	230	0.03	0.35	144 x 72 x 12

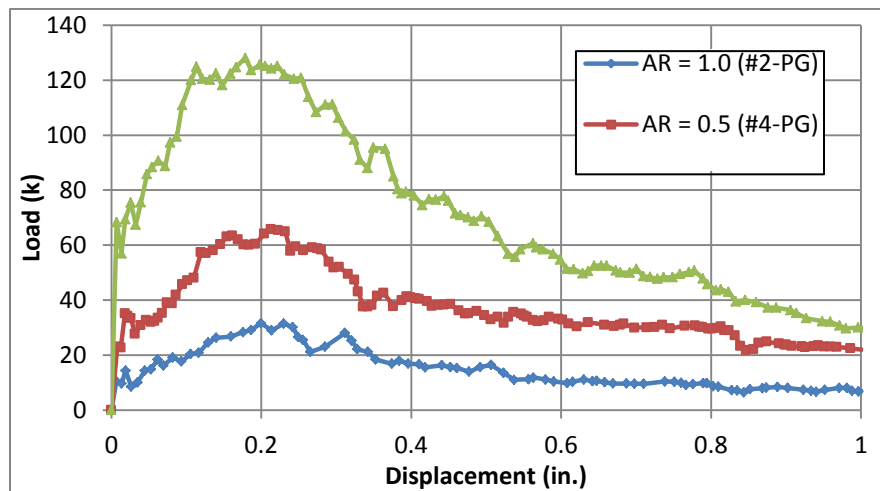
***All values are given based upon gross dimensions**

Out of the 19 fully grouted walls tested, 17 were deemed to have a shear failure. The majority of walls showed a somewhat brittle failure with sudden drops on the load-displacement curve after peak loading. All walls except #1-FG (the only wall with no axial load) exhibited distinct diagonal cracking, with wall #1-FG showing only minimal cracking. Out of the 12

partially grouted wall tests, all were deemed to have a shear failure, with all walls showing distinct diagonal cracking. Comparisons of various parameters are shown in Figures 5.18-22. All comparisons are grouped by walls that had similar vertical and horizontal reinforcement ratios and vertical stress (like comparisons are grouped by boxes in graph legends). A load vs. displacement curve is given for all specimens in Appendix D.



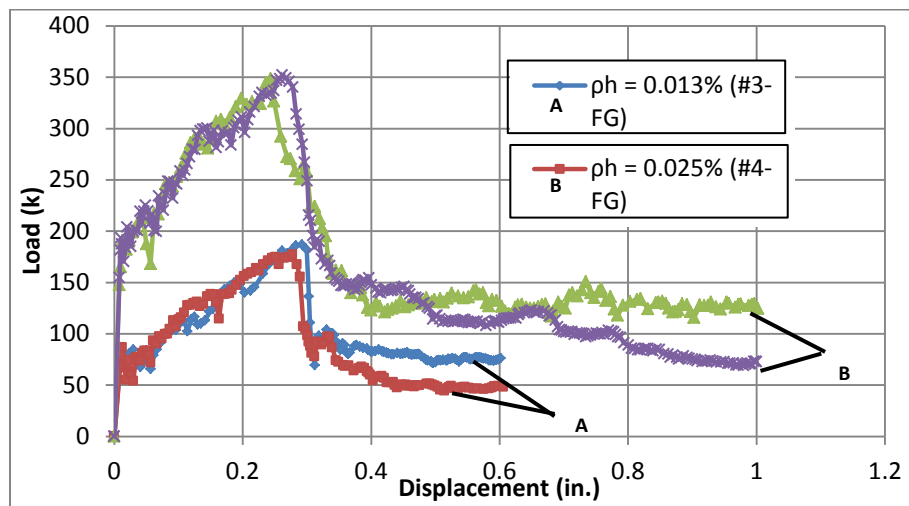
(a)



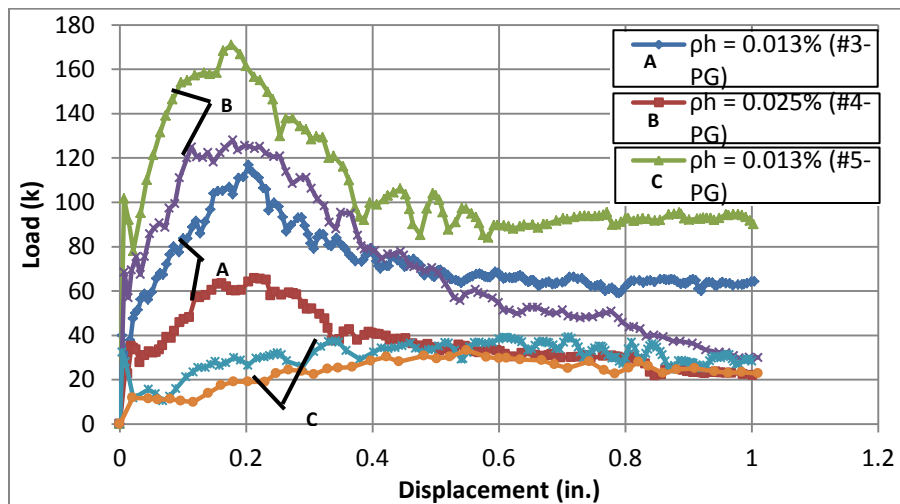
(b)

Figure 5.18 Effect of Aspect Ratio on (a) Fully Grouted and (b) Partially Grouted Wall Behavior

Figure 5.18 gives the effect of aspect ratio ($\frac{height}{length}$) on wall behavior. It can be seen that even though walls with a higher aspect ratio had less strengths, peak load occurred at a slightly larger deflection. The partially grouted walls have a flatter peak and more ductility as compared to the more pronounced peak and sudden drop in the load vs. displacement curve for fully grouted walls.



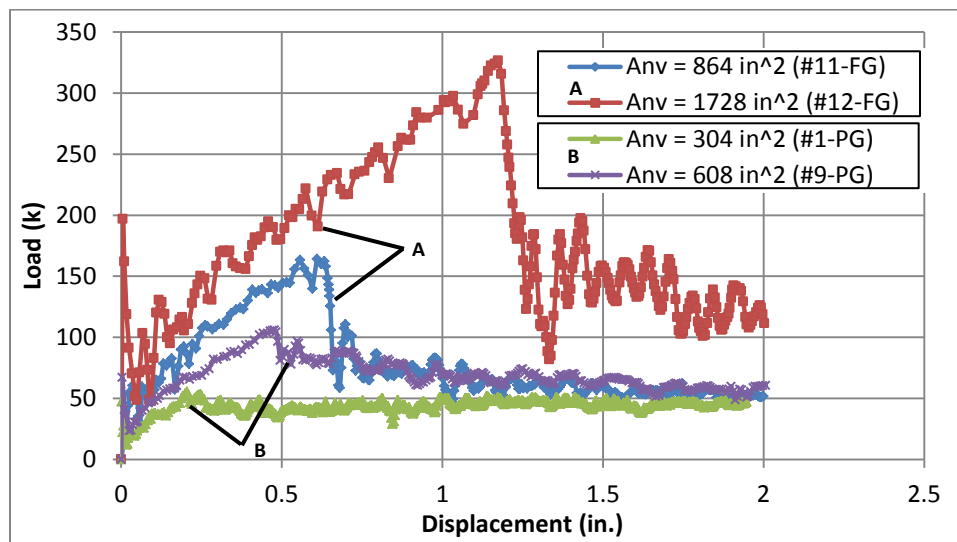
(a)



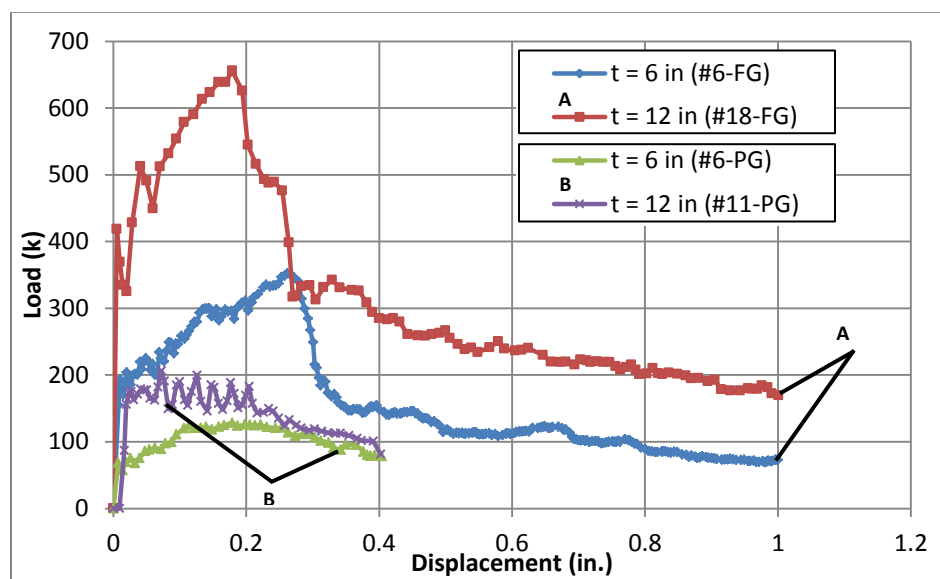
(b)

Figure 5.19 Effect of Horizontal Reinforcement Ratio on (a) Fully Grouted and (b) Partially Grouted Wall Behavior

Figure 5.19 shows the effect of horizontal reinforcement on wall behavior. For identical fully grouted walls with different reinforcement ratios, the difference in peak strength was minimal, but post-peak shear resistance was higher. For partially grouted walls, the difference in reinforcement ratio made a more substantial difference in peak wall strength, with curves that had a lower reinforcement ratio exhibiting a less-pronounced peak.



(a)



(b)

Figure 5.20 Effect of (a) Net Shear Area and (b) Unit Thickness on Wall Behavior

Figure 5.20 gives the effect of the net shear area and unit thickness on wall behavior. For identical walls, as shear area increases, so does the peak load and displacement at which it occurs, however drift ratios at failure for these walls are approximately the same. If the unit thickness is doubled as in (b) of Figure 5.20, the peak load increases, but the displacement at which this load occurs decreases over a smaller block size.

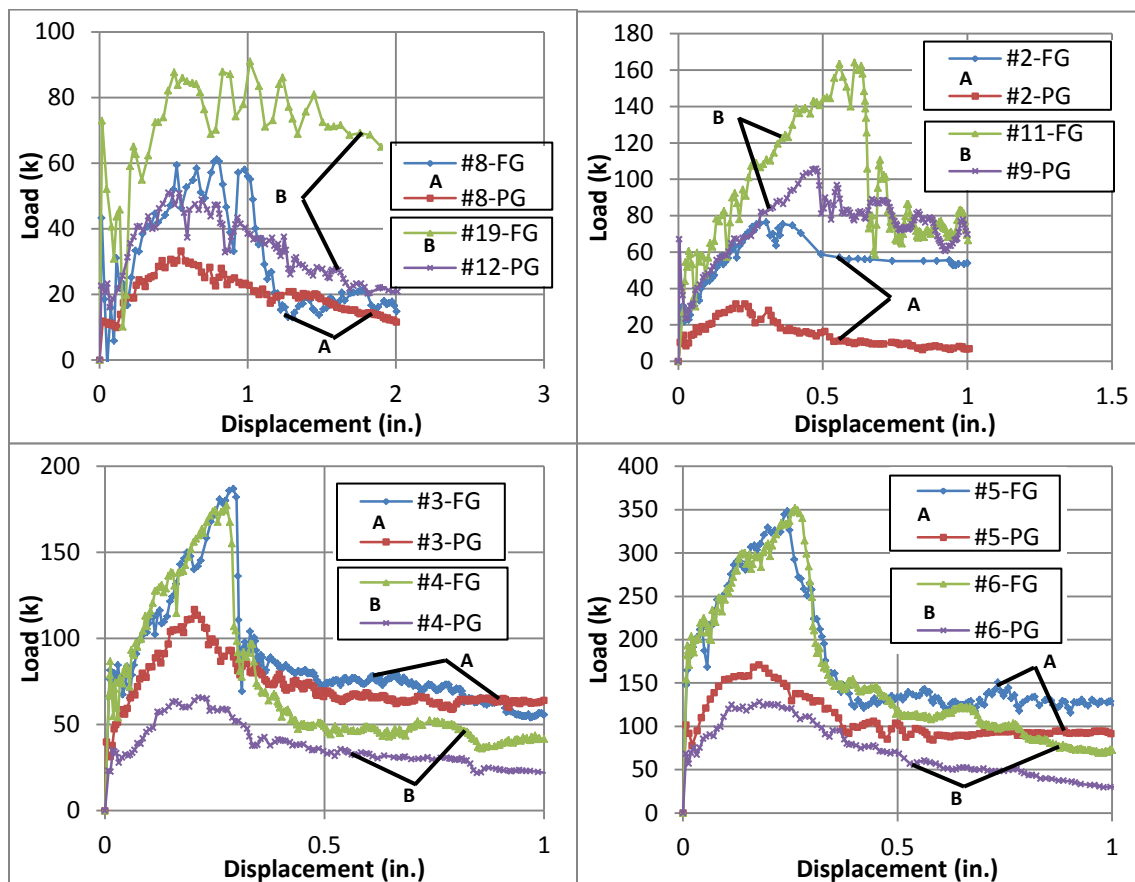
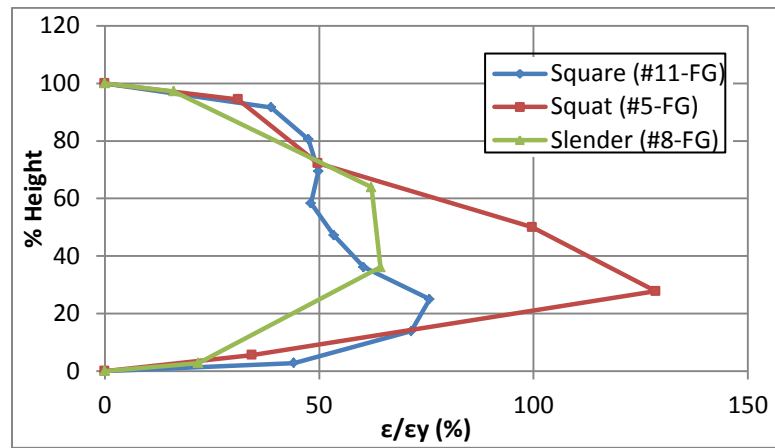


Figure 5.21 Comparisons of Identical Fully and Partially Grouted Walls

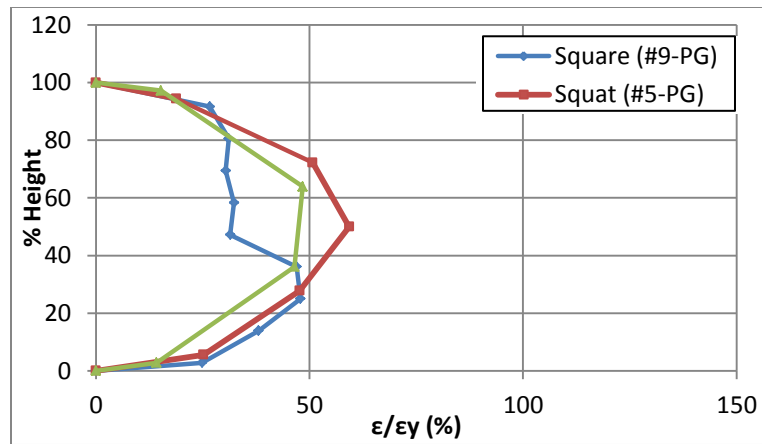
Figure 5.21 gives comparisons of equivalent fully and partially grouted walls. In all comparisons, the partially grouted walls exhibited a less brittle failure than the fully grouted

walls, but peak load occurred at a slightly smaller displacement. In most cases, the partially grouted walls exhibited steadier residual shear capacity after the peak load was obtained.

Figure 5.22 gives a representation of the distribution of maximum reinforcement strain at peak load (ϵ) divided by the yield strain of the horizontal reinforcement (ϵ_y). The data points represent the maximum strain divided by the yield strain for each bar at its respective height within the wall. Similar graphs for each wall are given in Appendix D.



(a)



(b)

Figure 5.22 Distribution of Horizontal Reinforcement Strain in (a) Fully Grouted and (b) Partially Grouted Walls

Fully grouted walls that were square or squat and that contained higher reinforcement ratios generally had higher strains in the middle to lower third of the wall, while partially grouted walls that were square or squat generally had a more even parabolic distribution of bar forces, with the highest forces being in the middle third of the wall. In general, partially grouted reinforcement strain distribution followed the trends of fully grouted walls, but with less strain in the majority of bars. This reinforces the notion that horizontal reinforcement is less efficient in partially grouted walls.

In the entirety of the extension of scope, only two walls contained horizontal reinforcement that yielded (these were square and squat walls), with the slender walls with higher reinforcement ratios coming close to yielding. In no wall did the top or bottom bond beam reinforcement come close to yielding. This matches somewhat with the findings of Baenzinger and Porter (2010) and Nolph (2010), who found that some walls with higher reinforcement ratios did not experience yielding in the shear reinforcement. Of the 14 walls that had reinforcement ratios higher than 0.11%, only 5 had bars with strains within 75% of the yield strain.

5.5 Strut-and-Tie Analysis

All of the created walls were analyzed using the strut-and-tie procedure of Chapter 4. The 17 fully grouted walls were determined to have an average $\frac{V_{exp}}{V_{pred}}$ of 1.38 with COV of 0.44. When the small walls (with heights and/or lengths less than 48 in.) are removed, an average and COV of 1.60 and 0.30 results, respectively, with minimum and maximum values of $\frac{V_{exp}}{V_{pred}}$ of 0.92

and 2.50. Slender walls (#7,8-FG) and large square walls (#11,12-FG) showed increased conservatism over other fully grouted walls.

For the 12 partially grouted walls, which had no walls under 72 in. in height or length, an average $\frac{V_{exp}}{V_{pred}}$ of 1.35 and COV of 0.12 results, with minimum and maximum values of $\frac{V_{exp}}{V_{pred}}$ 1.11 and 1.57. The increased performance of the strut-and-tie method for partially grouted walls is likely due to a partially grouted wall being physically more like a truss than a fully grouted wall.

It seems that strut-and-tie methods do not accurately predict the in-plane strength of very small squat and square walls. In all likelihood, these small walls would not be counted on for strength in design, and currently should not be analyzed with strut-and-tie methods.

Chapter 6—Simplified Strut Method to Calculate In-Plane Strength
of Masonry Walls

6.1 Analysis Introduction

This chapter introduces a piecewise method to calculate the in-plane shear strength of walls, so that the more involved method of strut-and-tie analysis may be neglected if certain wall parameters are met, as discussed in a later section. The principles of strut-and-tie and efficiency of reinforcement are used to arrive at equations that will predict the contributions from axial load, shear reinforcement, and the masonry.

The current TMS-402 (2011) strength design shear equations, along with other masonry codes such as the CSA and New Zealand, and researchers such as Matsumura and Fattal, use combinations of contributions from masonry, reinforcement, and axial load to predict the shear strength of masonry walls. Many of these equations are based on a failure model that resembles Mohr-Coulomb, and are calibrated to experimental tests making it difficult to isolate the contribution from the masonry and reinforcement in a wall. In fitting data by a purely empirical means, the problem arises that unless many wall specimens exist with a wide range of parameters, it is difficult to calibrate equations for every scenario. Several proposed and code equations present factors that try to capture as many scenarios within a wall as possible. Some of these equations work well, but are not based upon any singular failure mode. It is inherent that empirical factors must enter into equations found in this manner, if for none other reason than each wall is unique and no one model can completely capture the behavior.

It is believed that a model that is based on the predominate failure modes of shear walls should be used as a basis for design equations. It is desired that a minimum of empirical factors be used.

6.2 Masonry Contribution to Shear Strength

A true strut-and-tie procedure as in Chapter 4 would involve iteration to determine the failure arrangement of struts, and the ultimate load if it involves yielding of multiple ties, as multiple panels with bounding reinforcement could form struts--thereby making this method only feasible with structural analysis software on strut-and-tie arrangements for anything other than a very minimally reinforced wall. It is proposed that a 45 degree crack assumption be made and a single strut, regardless of reinforcement configuration, be assumed to extend from corner to corner in a square section of wall to follow approximate orientations of principal compressive stress. The cracks at or near failure in masonry shear walls contained in Appendix H show an approximate 45 degree tensile crack, often regardless of their dimensions. The 45 degree crack arises from principle tensile stresses being oriented at approximately 45 degrees from the plane of horizontal load application in a wall. This angle will decrease by a small amount (along with a small reduction in principal tensile stress) as one examines elements farther down in the wall due to increased axial load from the applied axial load and courses above the element, however, these reductions are negligible and will be ignored when assuming a cracking angle.

A strength is assigned to a strut according to an effective compressive strength multiplied by a strut area. The horizontal component of the strength of the strut can then be taken as the horizontal strength contribution from the masonry alone, and the portion of the strut that resists the in-plane load. The concept of taking the horizontal component of a strut as a factor in shear resistance has also been discussed by Hwang, et al., (2001) and Sanchez-Alejandre and Alcocer

(2010). Figure 6.1 is a representation of a simple equilibrium model that shows the generalized strut orientation in a square portion of wall.

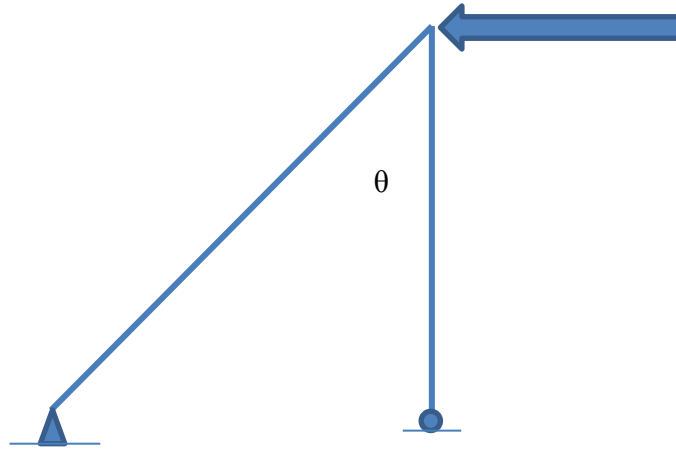


Figure 6.1—Proposed Equilibrium Model

In this simplified model, only the strut strength is of concern—the contribution from the steel in a reinforced wall will be presented in a later section. The strut is assumed to follow the cracking perpendicular to the principal tensile stress, and is theorized to be used for any wall aspect ratio and reinforcement arrangement, as long as a square section is used, with the strut spanning corner to corner of this section of wall (modifications for different walls sizes are shown later). For walls that are slender, their length will determine the height of the section to make it square, and for walls that are squat, their height will determine the length to create a square section.

Strength calculations should be based on the following:

$$F_T = A_{strut} f'_{me} \quad (6.1)$$

$$F_h = F_T \cos \theta \quad (6.2)$$

Where,

F_T = the axial strength of the strut

A_{strut} = area of the strut, as determined by grouting and block size

f'_{me} = effective compressive strength of the masonry

F_h = horizontal component of force in the strut; thereby, the “masonry contribution”

In this simplified method, the tie width is equal to the block thickness, θ is the angle between the diagonal strut and vertical tie (also equivalent to using the angle between the strut and a horizontal tie because of angle assumption) and twice the minimum face shell thickness is used as the thickness of the strut for partially grouted walls. This provides for a nodal face that is perpendicular to the axis of the strut, and an assumed cracking angle of 45 degrees. When the horizontal component is taken as the strength contribution, this causes the strut angle to drop out of the equation, avoiding the problem of trying to discern the actual cracking angle of a wall. This derivation is shown below for a partially grouted wall with an assumed strut thickness equal to twice the minimum face shell thickness.

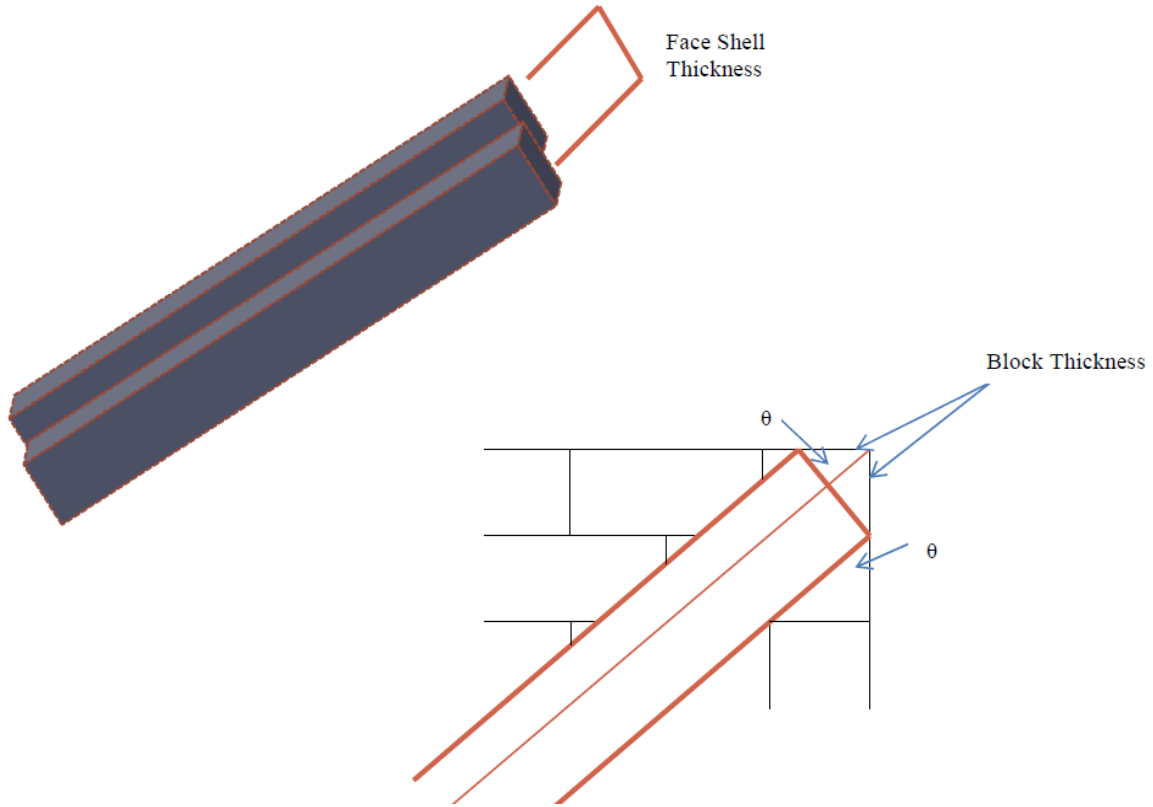


Figure 6.2—Strut and Angle Orientations

In reference to Figure 6.2, the area of the strut for a partially grouted wall is calculated as

$$A_{strut} = (t_f) \frac{b_t}{\cos \theta} \quad (6.3)$$

Therefore, the total strut strength is

$$F_T = \frac{(t_f) b_t f'_{me}}{\cos \theta} \quad (6.4)$$

giving a horizontal component, and thereby the masonry contribution to the strength, of

$$F_h = \frac{(t_f) b_t f'_{me}}{\cos \theta} \cos \theta \quad (6.5)$$

simplifying,

$$F_h = f'_{me}(t_f)b_t \quad (6.6)$$

Where,

t_f = total face shell thickness

b_t = thickness of a masonry unit

f'_{me} = effective masonry strength, equal to $0.7f'_m$

Equation 6.6 is for square walls, however, other geometric possibilities include walls that are squat (aspect ratio less than 1.0) or slender (aspect ratio greater than 1.0). An increase in shear resistance is noted in walls that are more squat than square from the available data, along with a decrease in shear resistance for walls that are more slender than square. For the proposed method it has been found that the increase or decrease in strength due to aspect ratio correlates well with the inverse of the aspect ratio, $\frac{length}{height}$, where the height is the height to the load and is considered to act at the top course of masonry. The method presented uses the philosophy that each masonry wall initially has the same parameters and are assumed the same and square (thereby allowing the use of Figure 6.1 for all walls), and then strength is modified for each wall using its own unique parameters.

The compressive strength can be affected, among other things, by the material strength, load duration effects, tensile splitting within a strut, and cracks that may cross the strut (MacGregor and Wight, 2005). The concept of strut softening deals with the known degradation of strength and stiffness of a member after it has cracked in the presence of transverse tensile strains (Vecchio and Collins 1993). Diagonal cracking is a known occurrence in walls failing in a brittle or diagonal tensile failure mode. The formation of struts, which can be assumed to be aligned with the direction of principal compressive stresses, comes at the expense of cracking, which causes their strength to be reduced (Hwang, et al, 2001). In Equation 6.6, an effective masonry compressive strength is used that reflects the maximum usable stress in compression in the TMS-402 code, $0.8f'_m$, with an additional factor to account for one tension tie within the node of the strut. It is understood that within an interior panel of a wall, two or more ties could be present on a node, but a one-tie assumption has proven to be a good generalization. A reduction of approximately 15% has been assigned to this value to account for the tensile stress and cracking in the compressive region of a node. Together, this constitutes an approximate 30% reduction in strength of the as-tested un-grouted masonry prism strength. Equation 6.6 has been modified to reflect these changes, and gives the masonry contribution to the shear strength of a wall:

$$V_m = 0.7(f'_m)(t_f)(b_t)\left(\frac{l}{h}\right) \quad (6.7)$$

By isolating only those partially grouted walls which contain no horizontal reinforcement and no axial load from within the masonry database shown in Appendix A (4 walls), Equation

6.7 has been shown to effectively predict the masonry contribution to shear strength with the average of the ratio of $\frac{V_{exp}}{V_{pred}}$ to be 1.12 with COV of 0.22.

Fully grouted walls have been shown to be predicted quite well by TMS-402 strength design equations. Nevertheless, the model that is proposed for partially grouted wall masonry contributions will be used to analyze fully grouted walls as well.

It is suggested in Chapter 2 by the equations of Matsumura (1986) and Fattal (1993) that partially grouted masonry has approximately 60% and 50%, respectively, of the masonry contribution to shear strength as that of fully grouted walls. Both of these researchers used appropriate factors to account for different masonry unit types and grouting scenarios in their equations listed in Chapter 2. In adapting the proposed equations for fully grouted walls, a percentage of the thickness of the units was utilized, as this is a well-defined dimension in masonry. The fully grouted walls would ideally have a strut thickness equal to the masonry unit thickness, however, this produced gross overestimations of masonry strength contributions, implying that the full width of a unit is not effective in a strut to resist an in-plane load.

The available data on fully grouted walls with no horizontal reinforcement and no axial load was severely limited, however, what was available was used to calibrate an equation that fit the data well. This resulted in the thickness of a fully-grouted strut to be approximately 65% of the wall thickness for walls up to 8 in. thick, and 40% of the wall thickness for walls over 8 in. thick. This gives a ratio of $\frac{V_{m_{partially\ grouted}}}{V_{m_{fully\ grouted}}}$ to be approximately 0.53 for a wall with similar parameters for both partial and full grouting, representing the masonry contribution from partially grouted walls as being about 53% of that of fully grouted walls. This process of populating a fully grouted strut thickness by taking a percentage of the wall thickness is nearly equivalent to increasing the face shell area by 90%. This empirical finding matches reasonably

well with the factors proposed in Matsumura's and Fattal's research, and correlates well with the overall shear strength as reported in a later section.

The resulting equation for masonry contribution in a fully grouted wall is shown in Equation 6.9.

$$V_m = 0.7(f'_m)(b_t^2)(\frac{l}{h})(\zeta) \quad (6.8)$$

Where,

$\zeta = 0.65$ for walls up to 8 in. thick, 0.40 for walls over 8 in. thick

It is believed that the method presented here, when based upon key assumptions founded on strut-and-tie methodology and information from past research, will yield a good approximation of the in-plane strength contribution of the masonry to the overall shear strength of a masonry wall failing in a diagonal tensile failure mode for the ranges of wall sizes found in the database in Appendix A.

6.3 Steel Contribution to Shear Strength

The steel contribution to masonry shear wall strength is currently represented in the TMS-402 code by Equation 3.7. This assumes that roughly half of the horizontal reinforcement in the wall is effective in resisting shear, however, the number of bars considered is based upon the length of the wall. As shown by Nolph (2010), this could result in more bars being counted than are actually there to resist shear.

After observing the “first major event” in his shear wall tests, Schultz (1998) concluded that “FME (first major event) displacements appear to be independent of horizontal reinforcement ratio,” reinforcing the notion that the steel is ineffective until cracking occurs, and can be considered separately in this simplified analysis.

Therefore, a method is proposed that considers only those bars that are potentially developed within a square section of wall with a 45 degree crack assumption, with appropriate factors based upon the finite element analysis of Chapter 5 to account for those bars not reaching yield. The TMS-402 equation for the required development length of bars is utilized and is shown in Equation 6.9.

$$l_d = \frac{(0.13)d_b^2 f_y \gamma}{K \sqrt{f'_m}} \quad (6.9)$$

Where,

d_b = diameter of reinforcing bar (in²)

f_y = yield strength of reinforcement (psi)

γ = 1.0 for #3-#5 bars, 1.3 for #6-#7 bars, and 1.5 for #8-#9 bars

K = smallest of: min. masonry clear cover, clear spacing between reinforcement splices, $9d_b$ (in.)

f'_m = masonry compressive strength (psi)

If standard hooks are used in horizontal reinforcement, an additional length can be subtracted from Equation 6.9 according to that presented in Equation 6.10 (the embedment length to develop standard hooks in tension).

$$l_e = 13d_b \quad (6.10)$$

Since a 45 degree crack assumption is made in the shear walls, a square section of wall is considered for effective steel. For squat walls, a horizontal portion equal to the height and for slender walls, a vertical portion equal to its length will suffice in dimensioning the walls for the steel analysis, and is the same as that for the masonry contribution. This is represented in Figure 6.3.

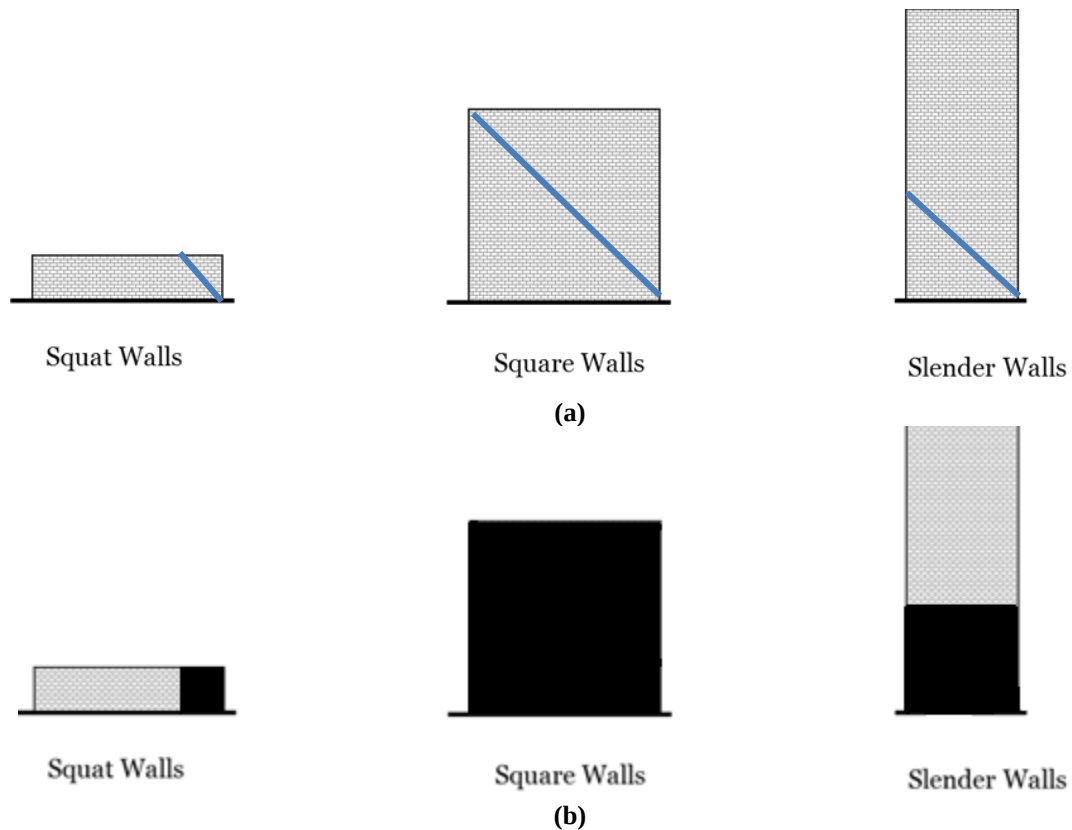


Figure 6.3—Region for Consideration of Effective Shear Reinforcement with (a) assumed 45 degree cracking and (b) region for consideration of effective reinforcement

It is understood that in a squat wall, more than one 45 degree crack may occur, and one should therefore consider this in analysis of bars that are/are not developed. For slender walls, the cracking angle could be steeper than 45 degrees, however, it was found that the 45 degree

generalization worked well in the available data, and corresponds to the regions of highest plastic compressive strain for many of the finite element models in Appendix D.

One can apply the TMS-402 development length equations and trigonometry to establish a vertical distance from the bottom of the wall that is required for development of the horizontal bars. Based upon spacing, one can determine the bars below this height that are undeveloped based upon the crack assumption, and also consider this same number at the top of the wall. This number of undeveloped bars can be subtracted from the total horizontal bars in the square section of wall. This is represented by Equation 6.11.

$$V_s = (N) A_h f_y - \psi(2) A_h f_y \quad (6.11)$$

Where,

N = # of horizontal reinforcing bars from bottom of wall to point of desired load application in a square portion of wall, less the top and bottom bond beams if located at top and bottom of the square section

A_h = area of one horizontal bar

$\psi = \left(\frac{(0.13) d_b^2 f_y \gamma}{(\tan \theta) K s \sqrt{f'_m}} - \frac{13 d_b}{(\tan \theta) s} \right)$ this term is rounded to the nearest whole number

θ = angle of strut measured from the vertical (45 degrees)

s = spacing of horizontal reinforcement

From the finite element analysis of Chapter 5, the total maximum bar forces at peak load were summed and compared to the total yield force of fully developed bars, producing an average of approximately 60% (COV = 0.27). Therefore, it will be assumed that the fully developed bars will contribute roughly 60% of their yield strength to the resistance of in-plane loads.

It is important to note that the bond beams—if located at the point of load application (top of the wall is generally considered the critical point for load application) and if located at the bottom course of a wall—are considered ineffective at resisting shear based upon a 45 degree crack assumption in slender and square walls, as the crack is assumed to emanate from just under or through the bond beam, rendering the steel therein undeveloped. This has also been shown in the finite element analysis of Chapter 5. For squat walls, potentially only a factor of one would be applied to the ψ term in the last half of the equation, as bars at the top (or bottom, depending upon cracking assumption) could be developed and would not need subtracting out (or these could be investigated singly). The term ψ should be rounded to the nearest whole number to account for possible inconsistencies in steel placement. Equation 6.11 works well for square sections with evenly spaced reinforcement, however, for walls with varying horizontal reinforcement spacing, the rounding term (ψ) can be isolated to examine each individual bar to check for development by inputting the spacing from the bottom of the wall.

In the work of Matsumura (1986) and Fattal (1993), there is also additional reduction factors on the steel contribution in partially-grouted walls of 60% and roughly 50%, respectively, of that of fully grouted walls. In comparing equivalent fully and partially grouted walls from the finite element analysis, on the average, the partially grouted walls had approximately 67% (COV of 0.09) of the sum of the maximum bar forces as the fully grouted walls. Matsumura (1986) theorized that part of the reason for a reduction in steel efficiency can be attributed to less grout-confining action of the masonry in partially-grouted walls as compared to fully-grouted walls, which acts to lessen the truss effect within the wall. It is believed that this statement refers to the fact that in an actual arrangement of vertical and horizontal steel in a fully grouted wall, a bond beam, grouted vertical course, and a node in an interior panel would be bounded by grouted

masonry on all sides, whereas in a partially grouted wall, the grouted cells and assumed nodes would be much less confined, as the surrounding cells would be hollow. As a result of these findings, an additional factor of 0.67 is applied to the steel contribution in a partially grouted wall. Equation 6.12 shows the final form of the horizontal reinforcement contribution to shear strength.

$$V_s = \lambda((N) A_h f_y - \psi(2) A_h f_y) \quad (6.12)$$

Where,

$\lambda = 0.6$ for fully grouted walls, and 0.4 for partially grouted walls

Equation 6.12 works well, but is too involved to quickly analyze a shear wall for a required area of steel. Therefore, a simplified version is presented in Equation 6.13 that is nearly equivalent to equation 6.12. Equation 6.13 is used in future analysis.

$$V_s = \lambda((N) A_h f_y - \psi_s A_h f_y) \quad (6.13)$$

Where,

$\lambda = 0.6$ for fully grouted walls, and 0.4 for partially grouted walls

$N = \#$ of horizontal reinforcing bars from bottom of wall to top of section defined by a square portion of wall

A_h = area of one horizontal bar

f_y = yield strength of horizontal reinforcement (psi)

ψ_s = the number of reinforcement bars or bed joint ladders in the top and bottom two courses of masonry in the square section of wall

6.4 Axial Load Contribution to Shear Strength

Current TMS-402 equations use a factor that considers the contribution to shear strength from the axial load to be 25% of that axial load, as shown in Equation 3.6.

Figure 6.4 is a graph of V_{max} vs. Axial Load for five different test sets for walls with the same size, reinforcement (both horizontal and vertical), and grouting, but with different axial load. In using only identical walls, factors such as horizontal and vertical steel and changes in wall size are dismissed, and the true behavior representing the relationship of axial load and strength contribution can be isolated.

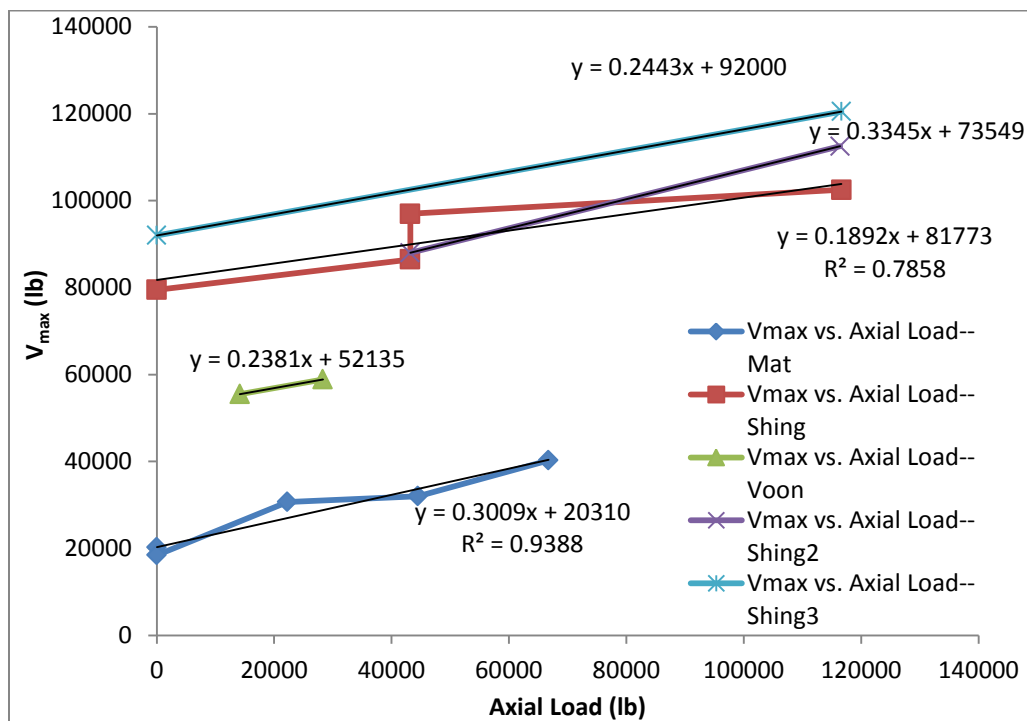


Figure 6.4 V_{max} vs. Axial Load

The curves in Figure 6.4, designated as Shing, Voon, Shing2, and Shing3 are for fully-grouted walls (Shing et al. 1990; Voon and Ingham 2006). The only partially grouted data has the designation Mat and is from Matsumura (1986).

A linear regression has been carried out on each curve, with the slope of the line representing the contribution of axial load. The curves indicate that the increase in shear resistance due to an increase in axial load is nearly a linear relationship. If one averages the slopes of the best fit lines for the fully grouted walls, an average coefficient of 0.26 results (COV = 0.22). The coefficient is approximately 0.30 for partially-grouted walls, but with only one data set this is not enough evidence to state that the axial load contribution in partially-grouted walls is any different than fully-grouted. These coefficients prove that the TMS-402 contribution to shear strength from the axial load correlates well with available data, and is kept as such for purposes of the proposed equations.

$$V_a = 0.25 P_n \quad (6.14)$$

Where,

V_a = axial load contribution to strength

P_n = applied axial load

6.5 Strength Limits and Model Applicability

Current limits on available shear strength calculated according to TMS-402 strength design are shown in Equations 3.8-9. These limits incorporate and recognize the increased strength gained from a squat-type wall and the decrease in strength with the use of a slender-type

wall. This limit is applied to the overall shear strength. Additionally, of the codes presented earlier, the CSA S304.1, and NZ 4230 all apply limits to the overall shear strength. It is believed that a combination of limits be placed on the proposed equations for masonry shear walls to prevent an over-strength calculation.

A proposed limit on the amount of strength the horizontal reinforcement can provide is represented by a limit on bar size. The finite element analysis of Chapter 5 and other data (Baenzinger and Porter, 2010; Nolph and ElGawady, 2012) has shown that an area per placement greater than the equivalent of a #5 bar either does not reach yield or barely reaches yield in test specimens, leading to an over-prediction of strength. In addition, the New Zealand Concrete Masonry Manual (2012) limits the bar size that is allowed in a masonry wall to a #5 and #6 bar for 6 in. and 8 in. block, respectively for 40 ksi steel and a #4 and #5 bar for 6 in. and 8 in. block, respectively, for 72 ksi steel. For this reason, it is recommended that a limit be placed on the horizontal reinforcing size of an equivalent of a #5 bar or smaller. This would ensure a more ductile failure with the same horizontal reinforcement ratio as with larger bars, only more bars would be present at a closer spacing. This is in contrast to the proposed limits on the overall horizontal reinforcement ratio that has been suggested in other research. A limit on bar size seems more applicable, as with a limit simply on the horizontal reinforcement ratio, all of the steel could be located in only a couple of courses and could cause the reinforcement to not provide its assumed strength if sufficient strains aren't developed in the reinforcement. A bar size limit would encourage designers who require a large amount of horizontal reinforcement in a wall to distribute the steel more evenly throughout the height of the wall, effectively creating more crack control and a more ductile wall.

An additional limit that is applied to the overall shear strength of the system is proposed that is based upon the crushing strength of the assumed strut that forms due to 45 degree diagonal cracking. In their testing of pull-off specimens for reinforced concrete, Hwang, et al (2000) adapted a softened strut and tie model to calculate strengths of the specimens and theorized that ultimate failure is caused by the crushing of the concrete in the compression struts that form after cracking. A similar assumption has been applied to the proposed piecewise equations, acknowledging that the reinforcement in the wall would be nearly ineffective due to loss of confinement after severe crushing of the masonry surrounding the steel. This limit is empirically matched to test data and is based on aspect ratio and the assumed masonry contribution to shear strength.

$$V_{n\max} = 1.75V_m \text{ if } \frac{h}{l} \geq 1.0$$

$$V_{n\max} = 1.15V_m \text{ if } \frac{h}{l} \leq 0.25$$

$$V_{n\max} = 3.5V_m \text{ for squat or square walls that are 12 ft. and over in height}$$

Values of $V_{n\max}$ may be linearly interpolated between the limits presented for walls under 12 ft. in height. The masonry contribution of Equations 6.7 and 6.8 does not recognize the increase in strength that a large square wall would have over a small square wall with the same parameters. With the proposed method, technically a wall that is 6 ft. x 6 ft. will have the same masonry contribution as a wall that is 12 ft. x 12 ft., therefore, as square walls get larger (above 12 ft. or so in this data), the method becomes increasingly conservative, and for walls that are considerably small (4 ft. or less in this data) the method becomes increasingly un-conservative.

For large walls, this is due in part to the fact that more than a single strut (or more than what an area of a singular strut would represent) would likely form to aid in carrying the in-plane load. Likewise, for considerably small walls, the strut from the proposed model would encompass the majority of the wall, whereas the actual wall would likely contain an area smaller than that represented by the masonry contribution equations. The simplified model performs well in the data for walls up to 24 ft. high, however, it is believed that actual behavior starts to deviate from the model with large walls over 12 ft. and small walls under 4 ft. A more robust simplified strut-and-tie masonry equation would include a variable masonry contribution with respect to wall size. However, with no real experimental data to correlate the equations with for larger walls, the proposed model should only be used to analyze single-story walls under 12-24 ft. in height, while a true strut-and-tie method as presented in Chapter 4 should be used for larger walls. Analysis of the data sets containing both fully and partially grouted walls using the aforementioned equations and limitations is presented in Chapter 7.

Chapter 7—Analysis of Proposed Equations for the Calculation of
In-Plane Strength of CMU Walls

7.1 Analysis Introduction

Analysis has been performed on 66 fully-grouted and 80 partially-grouted CMU masonry walls that failed in shear to determine the applicability of the proposed piecewise method of strength prediction. The data from these walls (finite element analysis from this study, Nolph and ElGawady, 2012; Baenzinger and Porter, 2010; Vaughan, 2010; Minaie et al., 2010; Voon and Ingham, 2006; Schultz et al., 1998; Shing et al., 1990; Matsumura, 1986, Yancey and Scribner, 1989; Hidalgo et al., 1978; Chen et al., 1978; Hidalgo et al., 1979; Sveinsson et al., 1985) were gathered from shear wall tests from the past 30 years and from the finite element analysis of Chapter 5. Step-by-step examples of calculating shear strength of walls using the piecewise equations are given in Appendix I.

7.2 Partially Grouted Walls

The proposed piecewise equations in Chapter 6 have been applied to 80 partially grouted walls. Data for each contributor to shear strength are presented in Appendix A . The majority of the information necessary for analysis was given in each respective data set, however, where certain steel sizes, spacing, and strengths were not available, an appropriate value was assigned based on similar research and trends.

Out of 80 walls, the proposed method provided a mean $\frac{V_{exp}}{V_{pred}}$ of 1.20 with a coefficient of variation of 0.30. Only 9 walls had over-predicted strengths of more than 20%, with the two extreme deviations from the mean being 0.65 and 2.58. This overall data set contained 16 walls

with aspect ratios between 0.25 and 0.9, 23 walls between 0.9 and 1.1, 34 walls between 1.1 and 1.4, and 7 walls with aspect ratio greater than 1.4. Fifteen walls contained only bed joint reinforcement or a combination of both reinforcing bars and bed joint reinforcement; these walls had an average ratio of 1.20 with a COV of 0.38, with an outlier of ratio 2.58 skewing the results slightly (this wall had unusually low compressive strength of 856 psi, therefore it was predicted low by the proposed equations). Less accuracy with the bed joint reinforced walls could come from the fact that the reinforcement was so closely spaced that the steel contribution term was very sensitive to what bars/joint reinforcement were chosen to have a contribution and which ones were not.

Within the analyzed data sets, there were several walls that were suspect and that had a large deviation in reported final strength from other similar walls. These were not excluded from the analysis, and are taken as statistical variation. One data set that was not included is from Schultz (1996), in which six partially grouted walls with bond beam reinforcement were tested in shear. These walls failed mostly with vertical cracking between the grouted end cells and ungrouted adjacent masonry and were therefore deemed not to fit the diagonal shear failure criterion of the proposed equations.

Figure 7.1 shows the relationships between the experimental shear strength and the shear strength calculated by the proposed method as compared with different wall parameters.

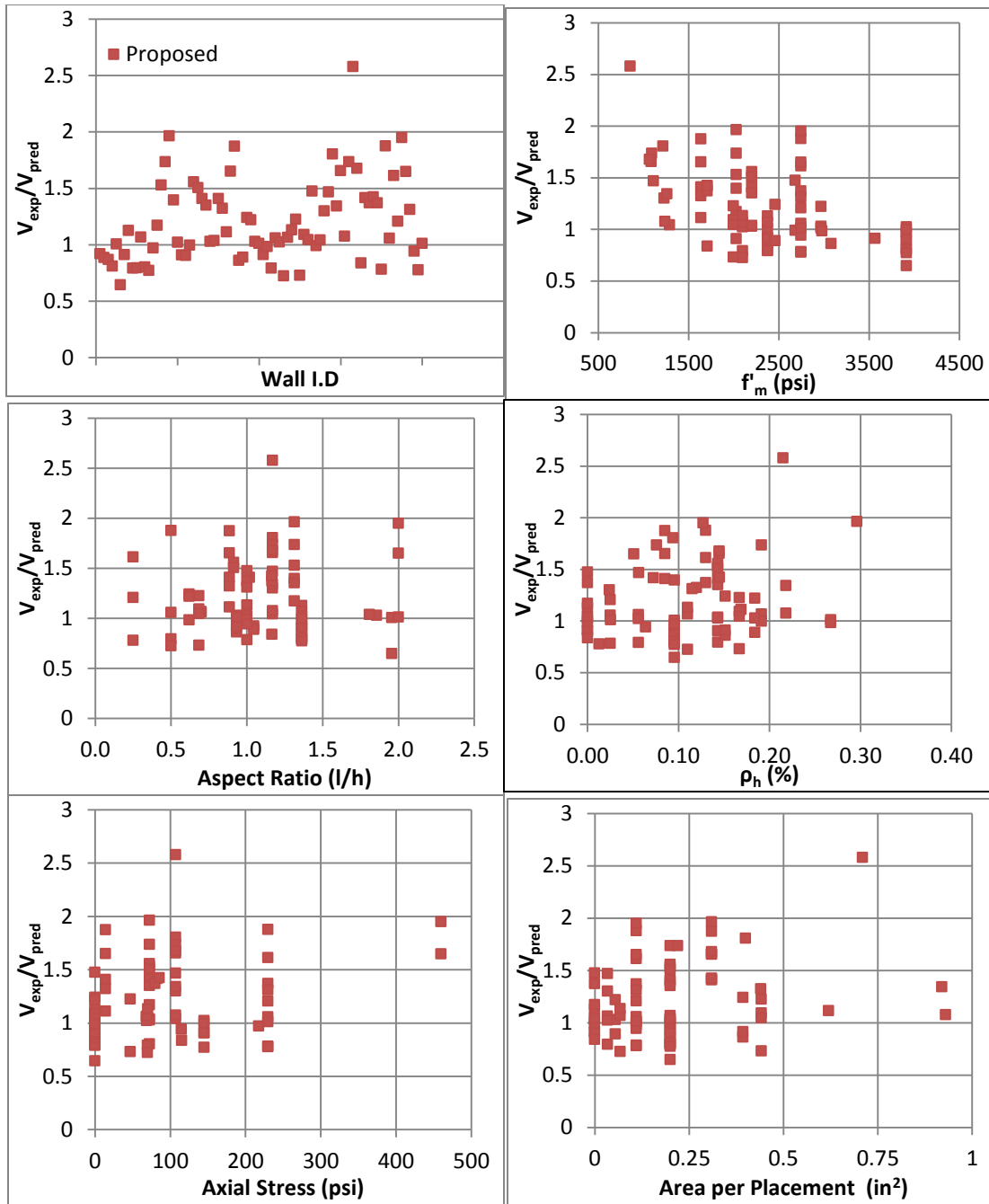


Figure 7.1 Partially Grouted CMU Wall Parameter Performance

From Figure 7.1, the proposed equations match closely with the values for the actual failure load in all the parameters studied, with the exception of walls with a low and high value of f'_m (proposed equations are conservative and unconservative, respectively), low aspect ratio

(proposed equations are a little unconservative), and high horizontal steel ratio and area per placement of horizontal steel (equations are a little conservative).

7.3 Fully Grouted Walls

The piecewise equations were utilized to analyze their effectiveness in predicting the strength of fully grouted walls. Out of 66 walls (the 4 small walls from the finite element analysis of Chapter 5 are not analyzed here) a mean $\frac{V_{exp}}{V_{pred}}$ of 1.32 with a coefficient of variation of 0.26 results. Only 2 walls had strengths that were over-predicted by more than 10%, with the two extreme deviations from the mean being 0.75 and 2.08. The overall fully grouted data set contained 11 walls with aspect ratios between 0.5 and 0.8, 39 walls between 0.9 and 1.2, and 16 walls between 1.5 and 2.3.

Figure 7.2 shows how the proposed equations performed against the actual average failure loads considering various parameters.

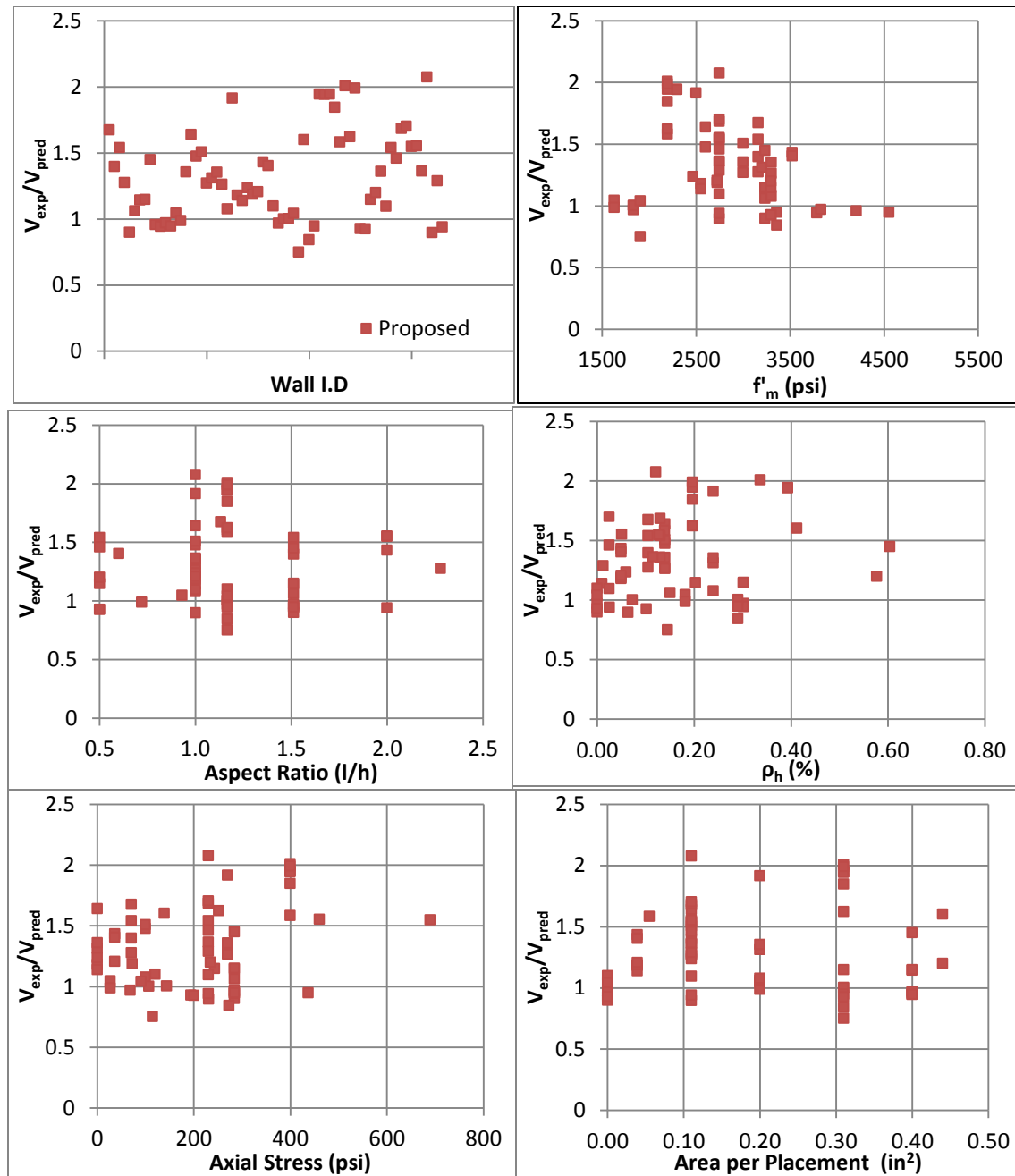


Figure 7.2—Fully Grouted CMU Walls Parameter Performance

Figure 7.2 shows good correlation between the proposed shear strength values and the actual shear strength values as compared with each parameter. The exceptions are walls with a high horizontal reinforcement ratio and a high area of horizontal steel per placement (similar trends noted in partially grouted walls)—the proposed equations are a little conservative.

7.4 Wall Fixity

The database contains a combination of cantilever walls and walls that were fixed against rotation at the top. This wall fixity is discussed in the literature review in Chapter 2, and where the researcher did not report the fixity, it was assumed based upon information given. In the proposed piecewise method, fixity is considered only in the respect that it is assumed the majority of walls will have some restraint against rotation at the top, but in practice, this may be full restraint, or very little restraint. Therefore, no increase in strength due to the wall being fixed against rotation was allowed for in the proposed piecewise equations, and likewise, no reduction in strength was considered for walls that were deemed to be cantilever. It is mildly conservative to assume this for walls that are truly restrained against rotation at the top. The equilibrium model assumes pin-type joints, and was therefore utilized for all walls, regardless of actual fixity in testing. This proved to be adequate in the analysis. Walls that were fixed against rotation at the top had slightly more conservative predicted values. Partially grouted walls tested as a cantilever (29 such walls) had a mean $\frac{V_{exp}}{V_{pred}}$ of 1.22 with a COV of 0.28, while the other 51 fixed walls had a mean ratio of 1.19 and a COV of 0.32. For fully grouted walls, the scatter was less pronounced with 32 cantilever walls having a mean ratio of 1.36 with COV of 0.20 and the other 34 fixed walls having a mean ratio of 1.29 with COV of 0.31.

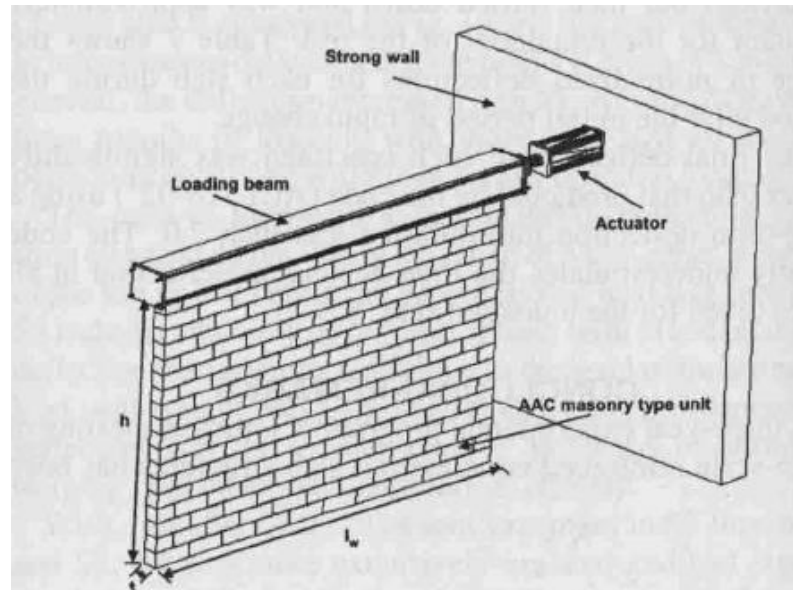
Chapter 8—Use of Proposed Equations/Methods in Other Masonry

8.1 Use of Equations for Other than Concrete Masonry

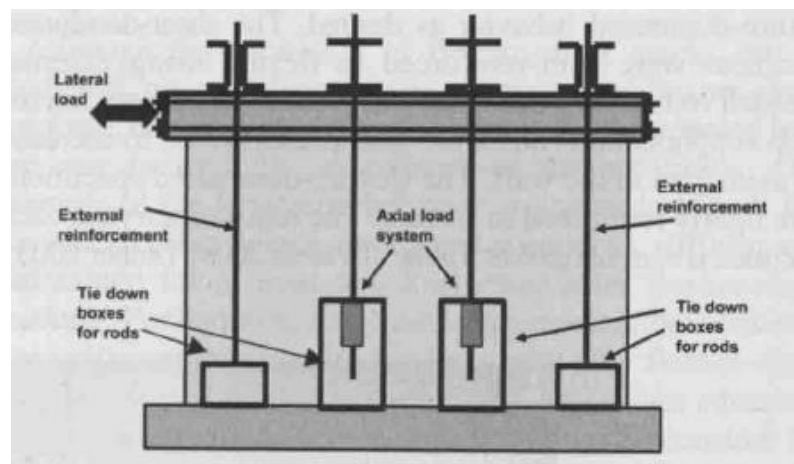
Due to their material-dependent nature, the proposed equations are supposed to be valid for any type of masonry unit, if the same failure mode and equilibrium model is assumed. Therefore, the proposed equations have been analyzed for accuracy in shear walls constructed out of materials other than concrete units.

8.2 AAC Masonry

In her in-depth study on autoclaved-aerated (AAC) concrete masonry, Tanner (2003, 2005) tested various AAC masonry assemblages to determine their behavior characteristics and to establish a means of analyzing various failure modes. A total of 19 shear walls were tested, with 4 walls being constructed out of AAC masonry units of with 8 in. x 8 in. x 24 in. dimensions. These walls have aspect ratios ranging from 0.63 to 3.2, and included only exterior vertical reinforcement. The testing setup and general external reinforcement placement is shown in Figure 8.1



(a)



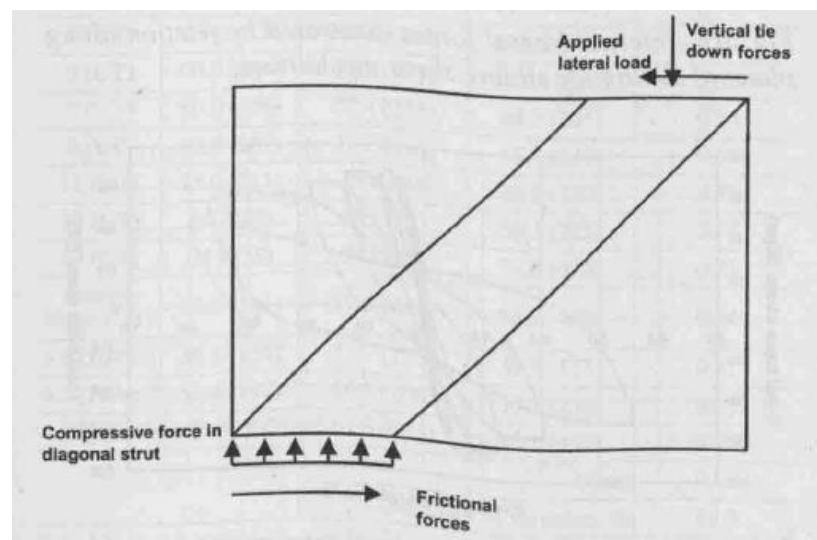
(b)

Figure 8.1—(Tanner, 2005) (a) Test Configuration and (b) Loading Apparatus

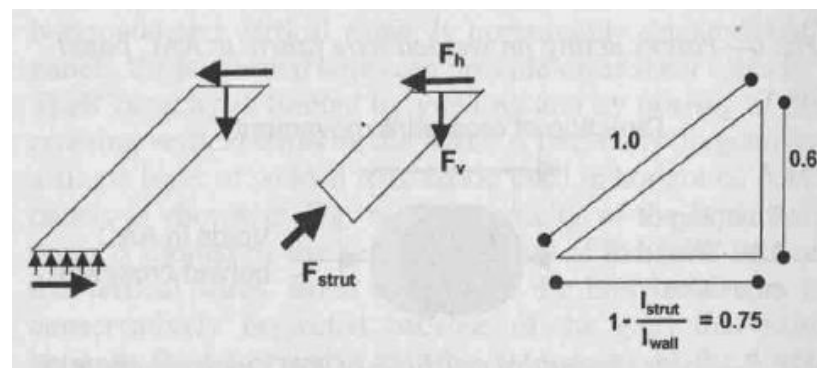
Equations were developed by Tanner to calculate capacity for walls failing in various modes of shear, including web-shear cracking and crushing of a diagonal strut. Her equations are represented in the AAC strength design portion of the TMS-402, as presented in Chapter 3.

Tanner derived the equation for the failure mode of crushing of the diagonal strut using a strut-and-tie model consisting of a diagonal compression strut and a tension tie-down force.

Tanner states that the force in the strut is a result of the vertical tie-down forces and the horizontal force, and because of geometry, the vertical component in the strut is the vertical tie-down force, and the horizontal component is the applied horizontal load (very similar to the basis for the proposed equations in Chapter 6). This is shown in Figure 8.2.



(a)



(b)

Figure 8.2—(Tanner, 2005) Strut in an AAC Wall

In a squat wall test (aspect ratio of 0.6), Tanner observed that crushing extended only a quarter of the length of the wall, and thereby formulated the following equation based upon this one wall that failed in a mode consistent with crushing of the diagonal strut. This is presented in Equation 8.1.

$$V_{ds} = 0.9 f_{AAC} t w_{strut} \left[\frac{h \left(\frac{3}{4} l_w \right)}{h^2 + \left(\frac{3}{4} l_w \right)^2} \right] \quad (8.1)$$

Where,

w_{strut} = horizontal projection of width of diagonal strut

h = height of shearwall

***TMS-402 provisions based upon this equation omit w_{strut} and the 0.9 factor, and instead use 0.17. Additionally, f_{AAC} is replaced with f'_{AAC} and the numerator in brackets is replaced with $h(l_w^2)$

Cancino (2003) had similar objectives in his research as Tanner, with the main goal being to validate the use of Tanner's equations on low-strength AAC in seismic areas. Within this data set, there were two walls that were constructed out of 8 in. x 8 in. x 24 in. AAC masonry units that failed in shear. Aspect ratios were 1.07 and 0.71, and contained reinforcement similar to those in Tanner (2005). A typical wall is shown in Figure 8.3.

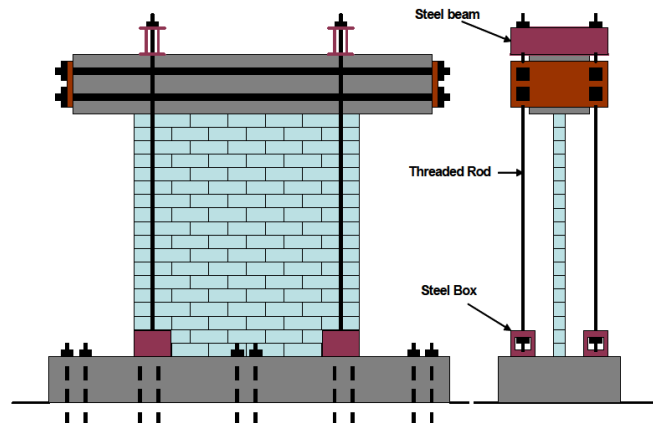


Figure 8.3 (Cancino, 2003) Loading Apparatus and Typical Wall Configuration

Six walls from both Tanner and Cancino were deemed adequate for analysis as they resembled traditional CMU and clay specimen construction. The six walls had aspect ratios ranging from 0.63 to 3.2, axial loads from 25 to 120 kips, and no horizontal reinforcement. The same principals were applied to the analysis of AAC as to the CMU previously discussed, with the exception of the strut area—for AAC, it was deemed more accurate to consider a strut thickness equal to the AAC wall thickness, rather than a percentage of the wall thickness, as with the CMU and clay units. This full strut area was used because the AAC units are cast as a solid, rather than being grout-filled on-site. The six walls had an average $\frac{V_{exp}}{V_{pred}}$ of 1.34 with a coefficient of variation of 0.32, with the two greatest deviations from the mean being 0.77 and 1.76.

8.3 Clay Masonry

The majority of the clay walls failing in shear comes from EERC research conducted from 1978-1985, as presented in Chapter 2, with additional specimens by Shing et al. (1990). Approximately 42 fully grouted walls and 7 partially grouted walls that were deemed to fail in shear were suitable for analysis.

The average $\frac{V_{exp}}{V_{pred}}$ for all 42 fully grouted clay walls using the proposed equations was calculated to be 1.14 with a coefficient of variation of 0.15, with the greatest deviations from the mean being 0.72 and 1.46. Only 4 walls were over-predicted by more than 10%, with the worst performers being walls with a dimension under 48 inches. The average $\frac{V_{exp}}{V_{pred}}$ for the 7 partially grouted walls was calculated to be 0.83 with a COV of 0.21. All of these walls were 48 in. or

less in length, and showed the same trend of unconservatism with small walls as with the strut-and-tie procedure of Chapter 4. Figure 8.4 contains graphs that compare the how the proposed equations performed against the actual average failure loads considering various parameters. Only the fully grouted wall data is presented in the graphs, as the partially grouted data is not a representative sample.

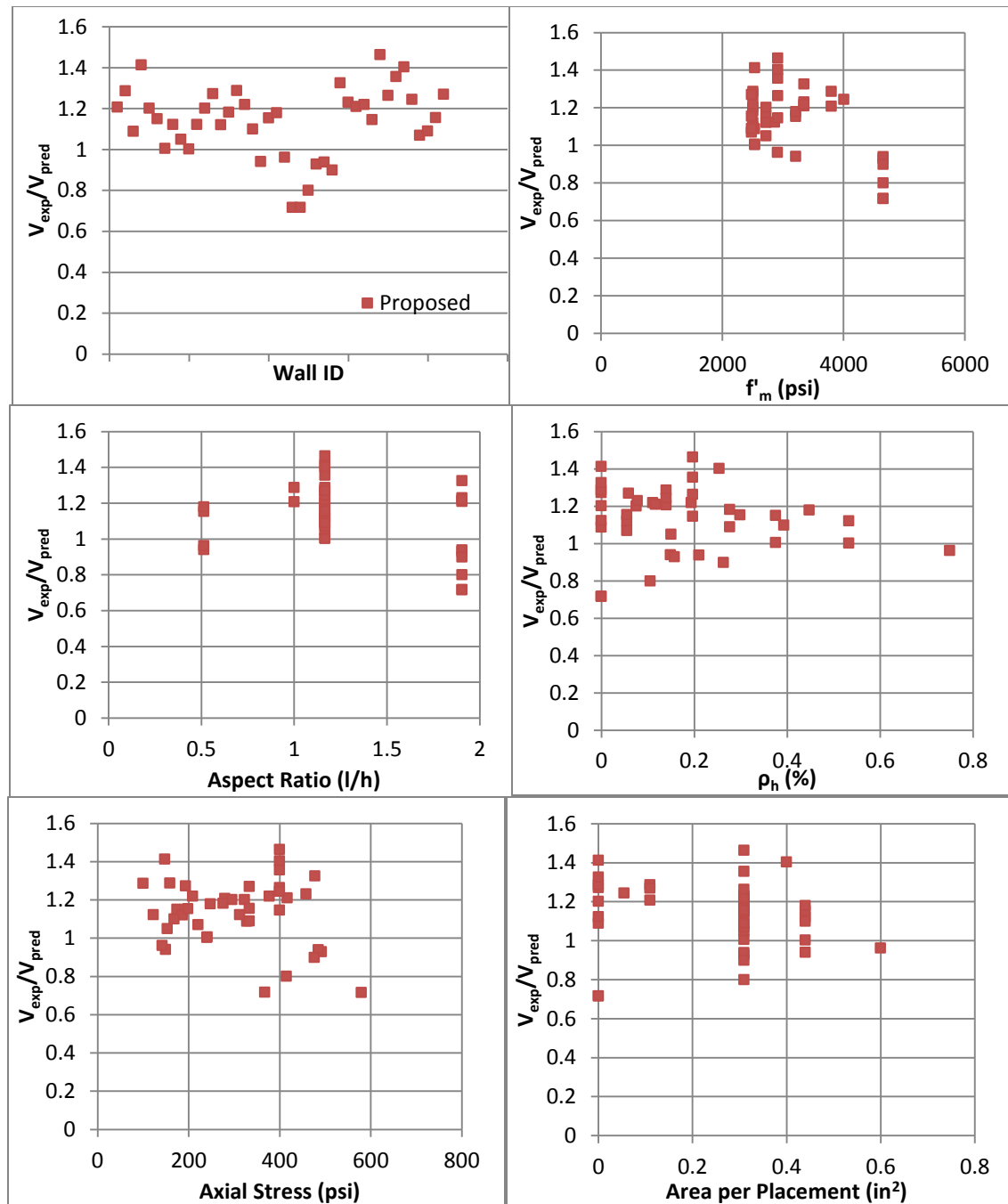


Figure 8.4—Fully Grouted Clay Walls Parameter Comparison

From Figure 8.4, it appears that for clay walls, the proposed equations are a little unconservative for higher values of f'_m , higher aspect ratios, and higher values of axial stress. The

proposed equations predict the in-plane shear capacity of clay walls in the overall data set fairly well.

Chapter 9—Comparison of Proposed Equations with the TMS-402 Shear Provisions

9.1 Comparison with Concrete Masonry Units

To investigate any improvement of the proposed equations over the current TMS-402 code equations, a direct comparison was made between both fully and partially grouted CMU walls that included the majority of the parameters that are of interest in design equations. Parameters such as masonry compressive strength, aspect ratio, gross axial stress, horizontal reinforcement ratio and yield stress, vertical reinforcement ratio and yield stress (if available), and Wall I.D (as a means of overall comparison) were compared against the ratio of $\frac{V_{exp}}{V_{pred}}$ to determine the performance of each method.

9.1-A Fully Grouted Walls

A comparison of values for TMS-402 and the proposed equations are presented in Table 9.1.

Table 9.1—Fully Grouted CMU TMS-402 and Proposed Equations Statistics

Wall Type	Number	$\frac{V_{exp}}{V_{pred}}$	Standard Deviation	Coefficient of Variation	Minimum Value	Maximum Value
TMS-402:						
Fixed	34	1.17	0.19	0.17	0.73	1.55
Cantilever	32	1.01	0.22	0.22	0.61	1.49
Overall	66	1.09	0.22	0.20	0.61	1.55
Proposed:						
Fixed	34	1.29	0.39	0.31	0.75	2.01
Cantilever	32	1.36	0.27	0.20	0.90	2.08
Overall	66	1.32	0.34	0.26	0.75	2.08

In general, it seems that the TMS-402 equations are good predictors of shear strength for fully grouted walls, with a little more accuracy for walls that are fixed, than for those that are cantilever. The proposed equations predicted cantilever walls with more accuracy than fixed walls. The overall ratio of experimental shear strength to predicted strength and variability is slightly higher for the proposed equations. Figure 9.1 gives graphical representations of the performance of various parameters.

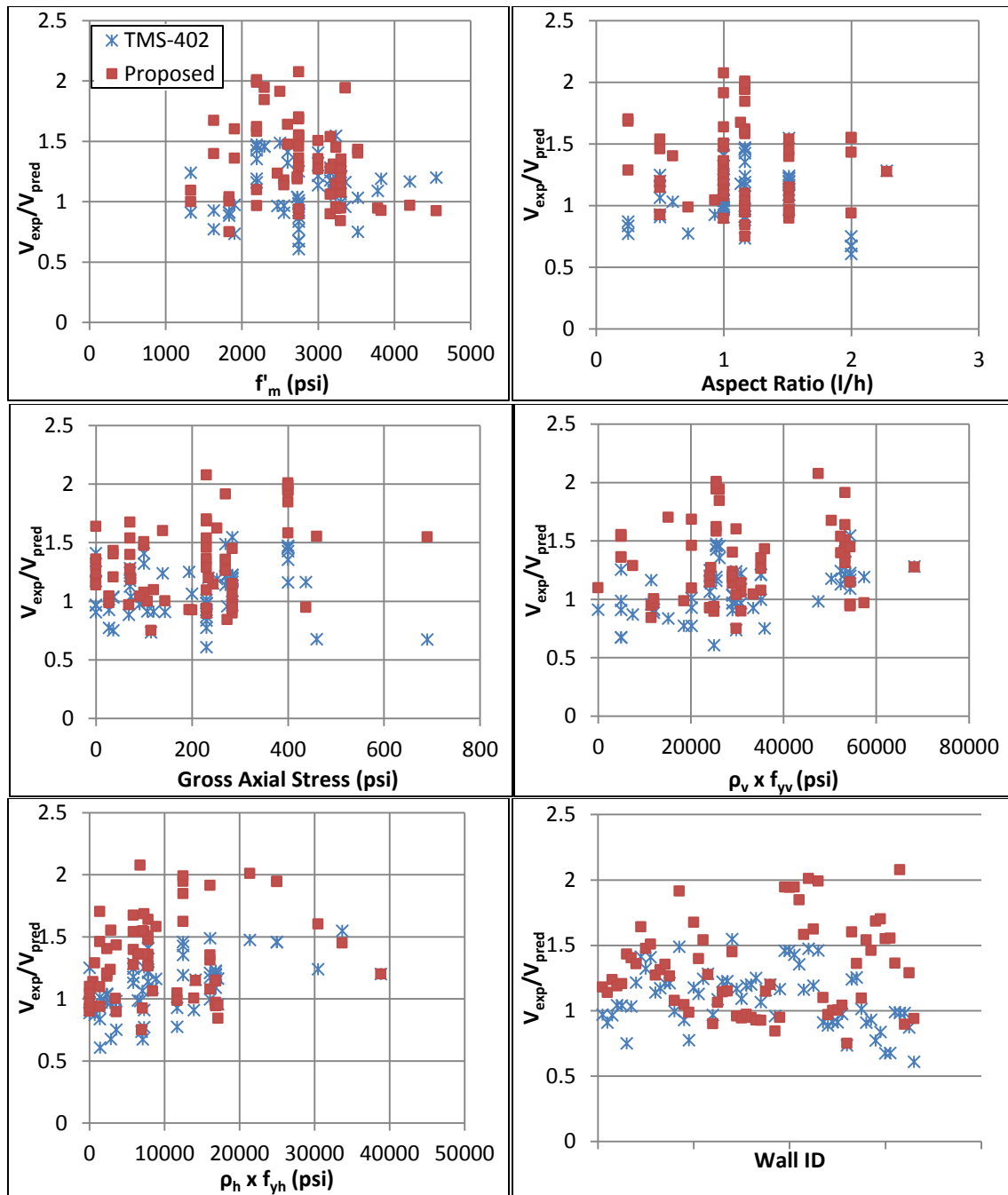


Figure 9.1—Fully Grouted CMU TMS-402 and Proposed Equations Parameter Performance

The TMS-402 equations are slightly more conservative with higher values of f'_m , where the proposed equations are more conservative with lower values of f'_m . The proposed equations seem to be better predictors with lower values of gross axial stress than the TMS-402 equations.

Neither set of equations have provisions for including vertical reinforcing bars into the equations for shear strength, however, when comparing the strength ratio to values of vertical reinforcement ratio multiplied by actual vertical bar yield stress, both the TMS-402 and proposed equations perform well, with the proposed equations having more conservatism. The graph of horizontal reinforcement ratio multiplied by horizontal yield stress uses actual values of yield stress. The proposed equations tend to be more conservative predictors of strength in the higher range of the data, while the TMS-402 was more un-conservative in the lower range. Comparison of the wall identification seeks to graphically represent the averages calculated in Table 9.1. In general, both equations perform well in calculating the strength of fully grouted walls in the available data.

9.1-B Partially Grouted Walls

Similar analysis was conducted on the partially grouted wall database as the fully grouted one and is presented hereafter.

Table 9.2—Partially Grouted CMU TMS-402 and Proposed Equations Statistics

Wall Type	Number	$\frac{V_{exp}}{V_{pred}}$	Standard Deviation	Coefficient of Variation	Minimum Value	Maximum Value
TMS-402:						
Fixed	51	0.90	0.46	0.51	0.42	2.46
Cantilever	29	0.94	0.45	0.48	0.36	2.04
Overall	80	0.92	0.45	0.50	0.36	2.46
Proposed:						
Fixed	51	1.19	0.38	0.32	0.65	2.58
Cantilever	29	1.22	0.35	0.28	0.73	1.95
Overall	80	1.20	0.36	0.30	0.65	2.58

Table 9.2 shows that both sets of equations have little trend in predicting capacity of fixed and cantilever walls—scatter was about the same between the wall fixity types within each equation. The table also shows that the TMS-402 is a poor predictor of walls that were partially grouted. Even though the overall average $\frac{V_{exp}}{V_{pred}}$ is close to 1.0, the coefficient of variation is 0.50, suggesting inconsistency and scatter. The proposed equations are a much better predictor of in-plane shear strength of partially grouted walls with an average $\frac{V_{exp}}{V_{pred}}$ of 1.20 and a coefficient of variation of 0.30, which is about 40% less than the TMS-402 value. Figure 9.2 gives graphical representations of the performance of various parameters.

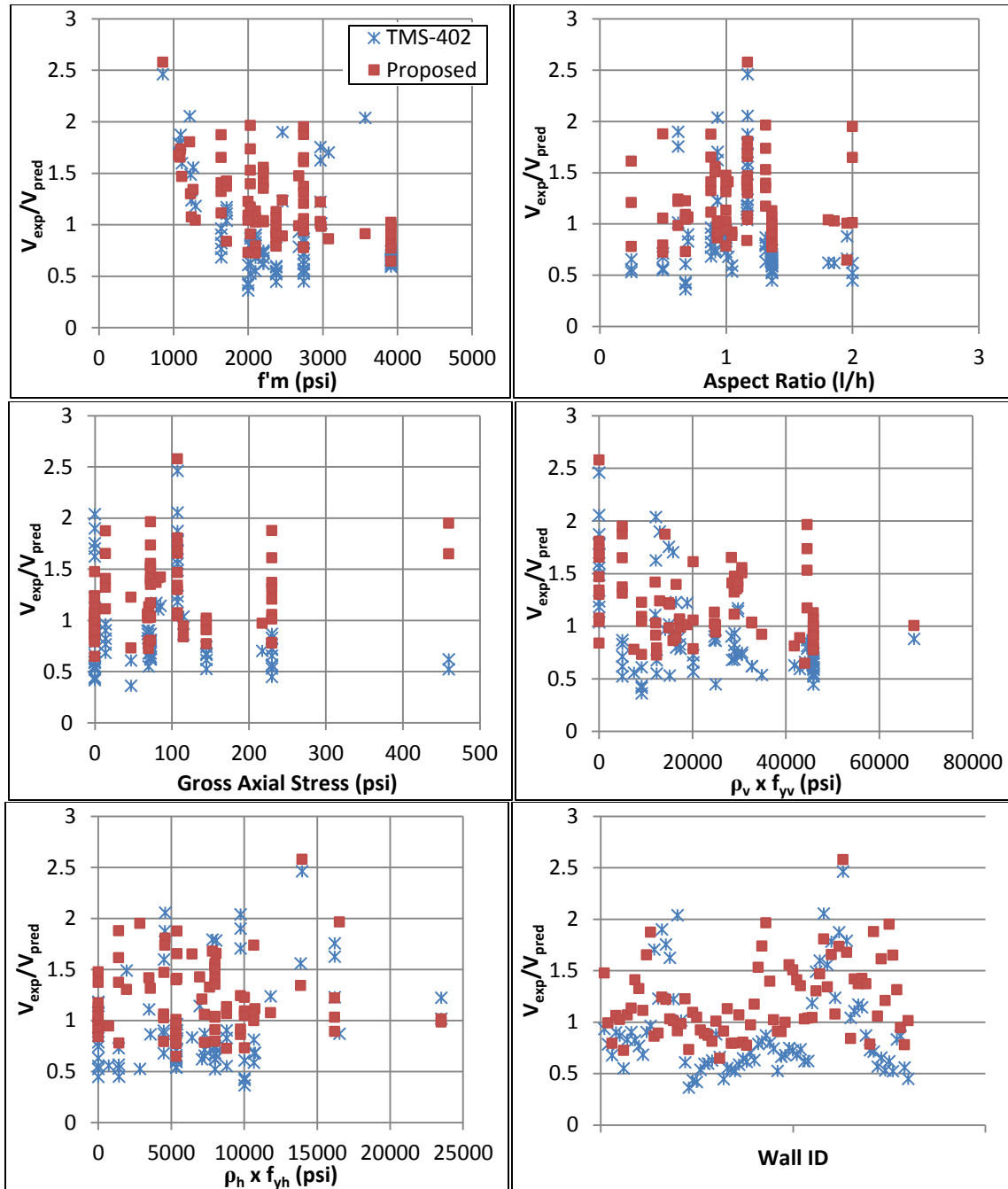


Figure 9.2—Partially Grouted CMU TMS-402 and Proposed Equations Parameter Performance

The proposed equations are better at predicting shear strength at most ranges of f'_m , but are more conservative for values in a lower range. Lower ranges of aspect ratio are a little unconservative with the proposed equations. The remaining parameters show that the proposed

equations contain less scatter than the TMS-402 equations. Even though the proposed equations have no provisions for incorporating the vertical reinforcement, it has shown to be a good predictor of strength for all ranges of vertical steel present. The majority of the TMS-402 values are un-conservative for this parameter. The graph of $\frac{V_{exp}}{V_{pred}}$ vs. Wall ID serves to visually represent the overall average and scatter presented in Table 9.2 for both sets of equations.

In general, it has been shown that the TMS-402 equations for in-plane shear are poor predictors for the shear strength of partially grouted walls, while the proposed equations provide a more consistent approach across various parameters.

9.2 Comparison with Clay Masonry

9.2-A Fully Grouted Walls

Table 9.3 summarizes performance of the TMS-402 and proposed equations for fully grouted clay walls.

Table 9.3—Fully Grouted Clay TMS-402 and Proposed Equations Statistics

Wall Type	Number	$\frac{V_{exp}}{V_{pred}}$	Standard Deviation	Coefficient of Variation	Minimum Value	Maximum Value
TMS-402:						
Fixed	40	1.06	0.21	0.20	0.82	1.46
Cantilever	2	1.22	0.05	0.04	1.18	1.26
Overall	42	1.09	0.13	0.12	0.82	1.46
Proposed:						
Fixed	40	1.13	0.17	0.15	0.72	1.46
Cantilever	2	1.25	0.06	0.04	1.21	1.29
Overall	42	1.14	0.17	0.15	0.72	1.46

It can be seen in Table 9.3 that both the sets of equations predicted the available test data for fully grouted clay specimens well with slight conservative on overall averages, but with the proposed equations having 25% more variability as the TMS-402 equations for fully grouted walls. Parameter comparisons are shown in Figure 9.3.

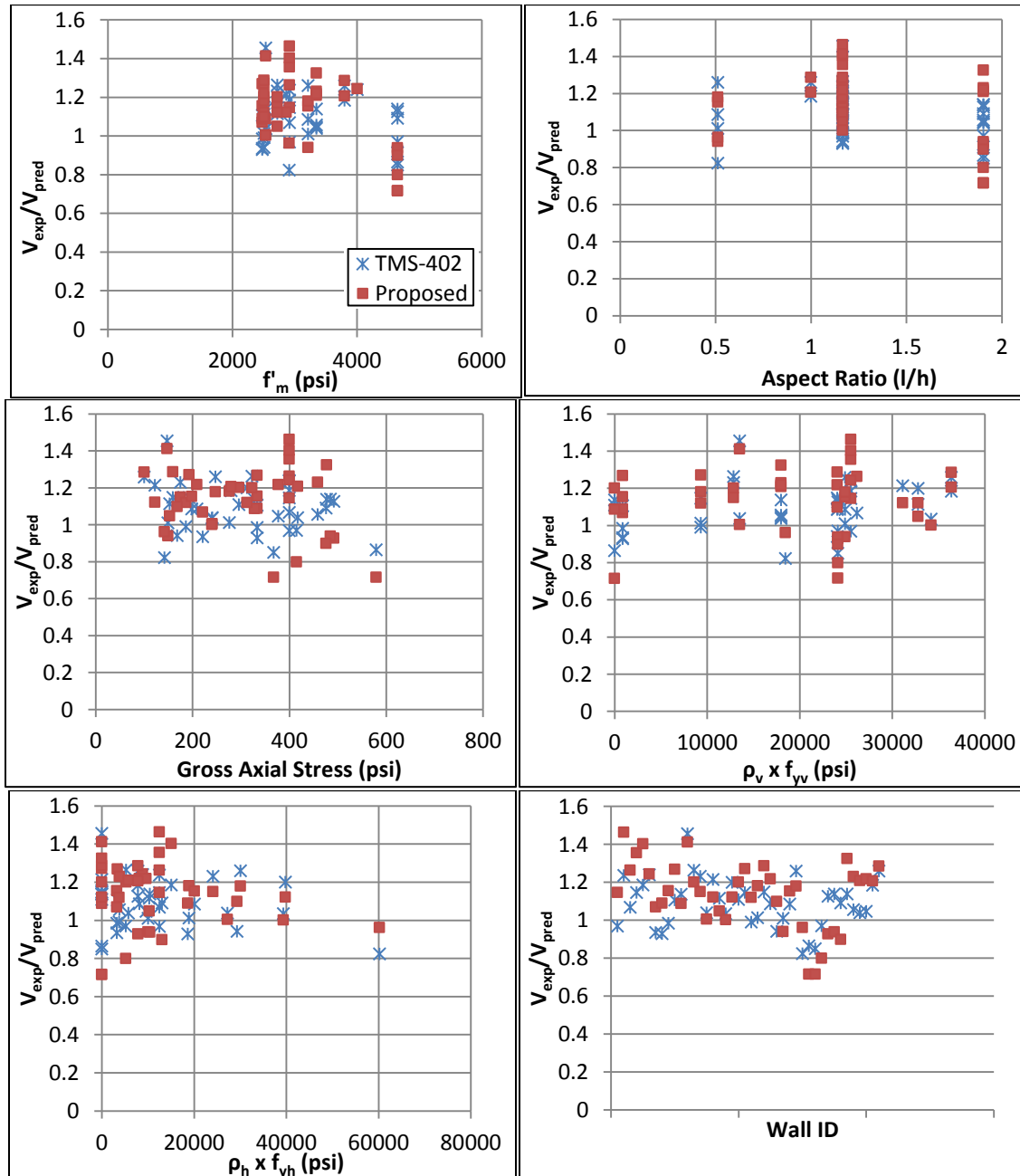


Figure 9.3—Fully Grouted Clay TMS-402 and Proposed Equations Parameter Performance

Figure 9.3 shows that both sets of equations predicted strength across various parameters well, with the proposed equations having a little more variability. The proposed equations were more unconservative with higher values of gross axial stress.

9.2-B Partially Grouted Walls

A small data set exists of partially grouted clay masonry walls failing in shear. Results of analysis using the proposed and TMS-402 equations are shown in Table 9.4.

Table 9.4—Partially Grouted Clay TMS-402 and Proposed Equations Statistics

Wall Type	Number	$\frac{V_{exp}}{V_{pred}}$	Standard Deviation	Coefficient of Variation	Minimum Value	Maximum Value
TMS-402:						
Fixed/Overall	7	0.98	0.13	0.13	0.81	1.20
Proposed:						
Fixed/Overall	7	0.84	0.18	0.21	0.55	1.11

Table 9.4 shows that both the TMS-402 and proposed equations were unconservative in predicting the strength of partially grouted clay walls, with the proposed equations being more so. While the TMS-402 equations are not very good predictors of the shear strength of partially grouted CMU masonry, partially grouted clay masonry strengths were predicted quite well. Parameter comparisons are shown in Figure 9.4.

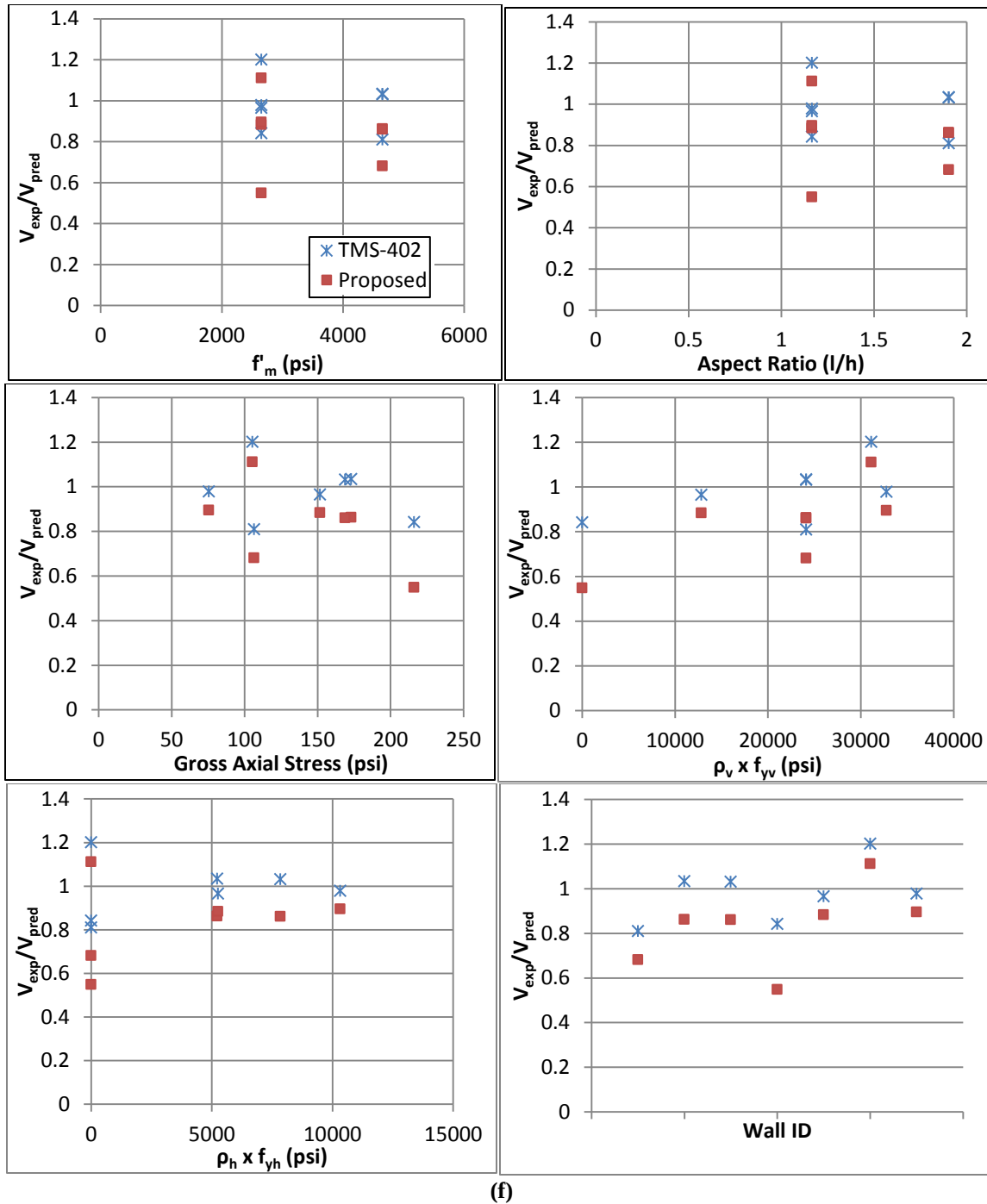


Figure 9.4—Partially Grouted Clay TMS-402 and Proposed Equations Parameter Performance

It can be seen that the proposed equations were more un-conservative than the TMS-402 equations in most ranges of data.

9.3 Comparison with AAC Masonry

A limited set of AAC masonry assemblages were available to analyze. The TMS-402 has entirely different equations for AAC masonry than for clay or concrete masonry. AAC shear provisions in the TMS-402 provide equations that cover several different modes of shear failure—web-shear cracking, crushing of diagonal strut, and sliding shear. All three equations were used to determine the controlling strength (in all cases, the strength as calculated due to web-shear cracking was the controlling value). Additionally, there were no horizontal reinforcement, however, vertical bars were present externally. All walls appeared to be tested as cantilevers. Summarized values are in Table 9.5

Table 9.5—AAC TMS-402 and Proposed Equations Statistics

Wall Type	Number	$\frac{V_{exp}}{V_{pred}}$	Standard Deviation	Coefficient of Variation	Minimum Value	Maximum Value
TMS-402:						
Overall	6	1.35	0.24	0.18	0.97	1.60
Proposed:						
Overall	6	1.37	0.47	0.34	0.77	1.93

Parameter-specific comparisons are shown in Figure 9.5.

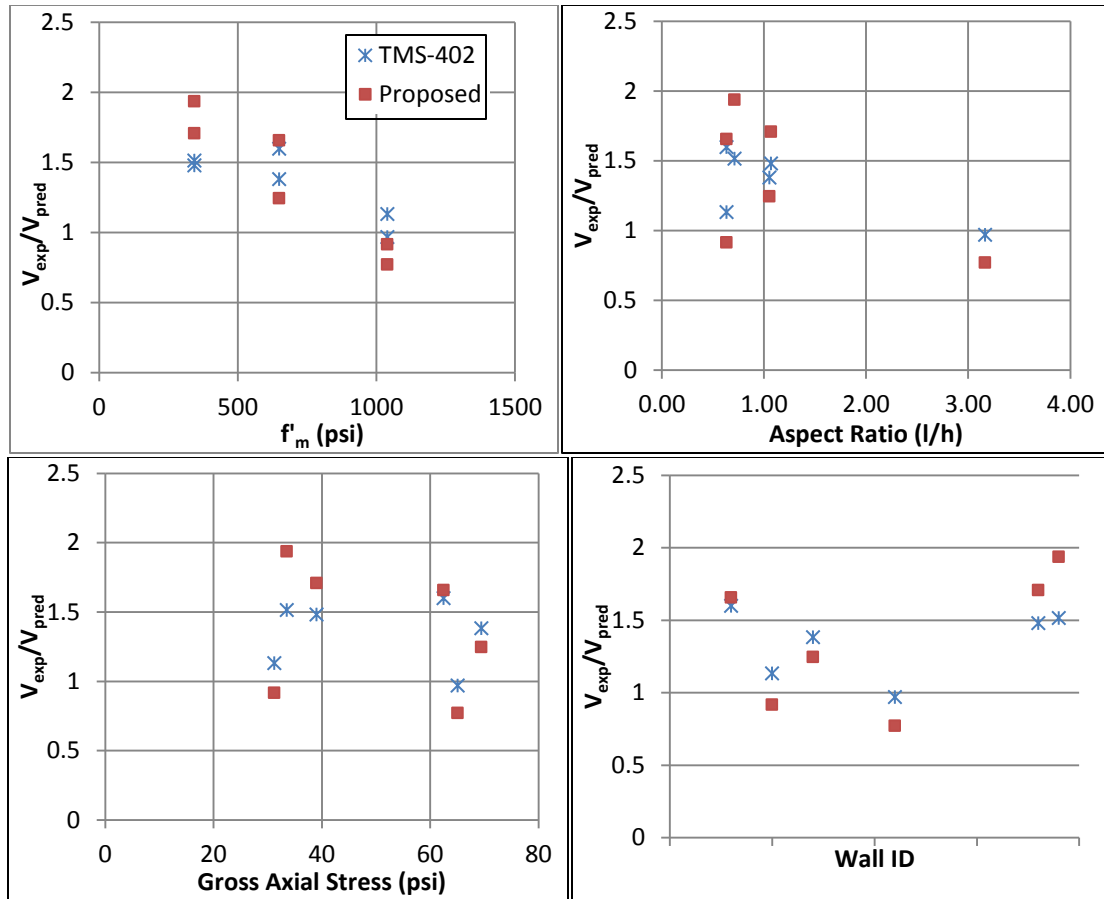


Figure 9.5—AAC TMS-402 and Proposed Equations Parameter Performance

Figure 9.5 shows that the current TMS-402 equations do a good job of predicting the strength of the AAC masonry walls that were built with masonry unit-type blocks. The proposed equations performed less adequately with more scatter and maximum and minimum values further away from the desired ratio of 1.0. In regards to parameter-specific performance, the proposed equations were a little more conservative than the TMS-402 with lower values of f'_m and a little less conservative with higher values of f'_m . Both equations seem to exhibit a nearly linearly decreasing relationship as values of f'_m increase, but AAC masonry generally has low compressive strength. Likewise, for aspect ratio, gross axial stress, and overall ratio (Wall I.D)

the proposed equations followed essentially the same trend as the TMS-402, but with quite a bit more scatter.

It is believed that the current TMS-402 equations adequately predict the shear strength of AAC masonry walls in the data set, even though the data set on walls built with approximate CMU-sized units is severely limited. The proposed equations could be modified further for use with AAC, or could be used as-is for a general estimate of shear strength for an AAC shear wall.

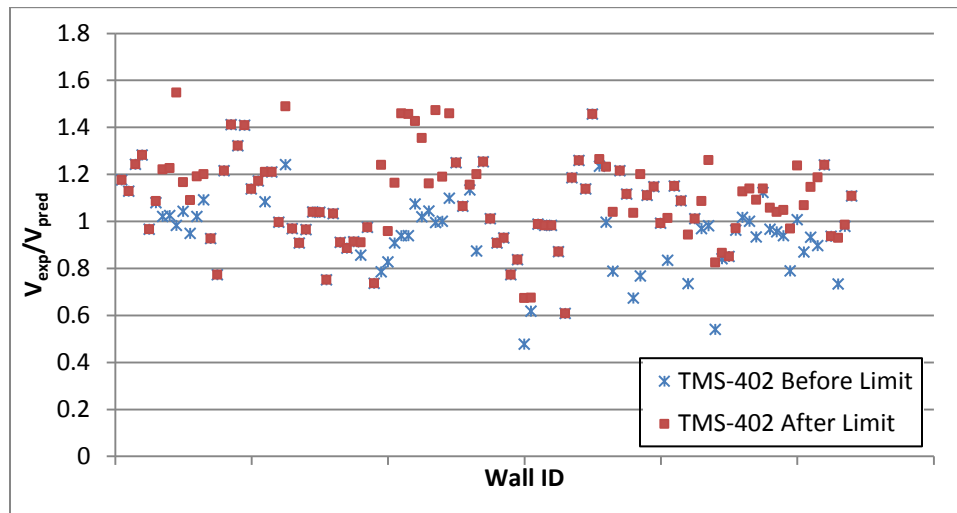
9.4 Overall Performance

The TMS-402 does not distinguish between concrete and clay walls. Table 9.6 is the collective performance of the TMS-402 and the proposed equations for all concrete and clay walls.

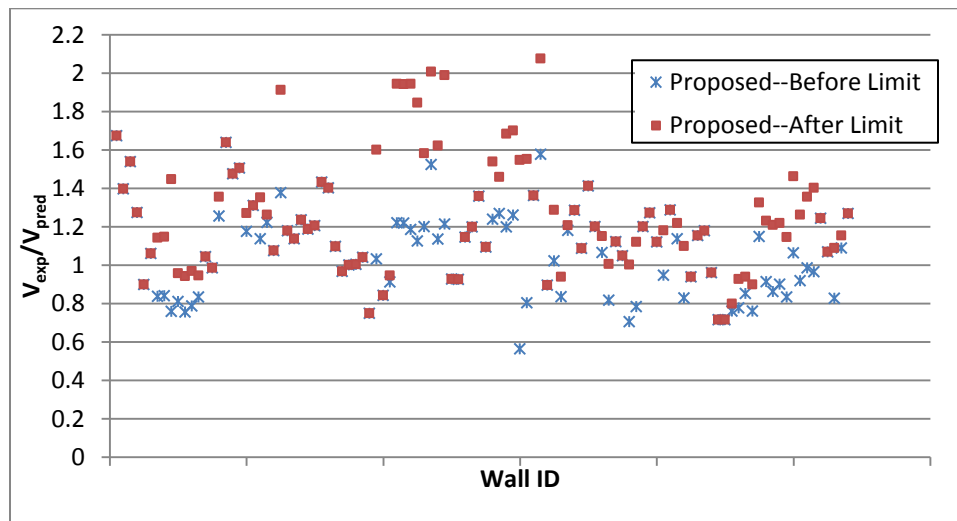
Table 9.6—Collective Performance for Concrete and Clay Masonry

Wall Type	Number	$\frac{V_{exp}}{V_{pred}}$	Standard Deviation	Coefficient of Variation	Minimum Value	Maximum Value
Fully Grouted Walls						
TMS-402:						
Overall	108	1.09	0.19	0.17	0.61	1.55
Proposed:						
Overall	108	1.25	0.30	0.24	0.72	2.08
Partially Grouted Walls						
TMS-402:						
Overall	87	0.92	0.44	0.47	0.36	2.46
Proposed:						
Overall	87	1.17	0.37	0.31	0.55	2.58

The limits imposed on the TMS-402 strength design shear equations are based upon the net area in a shear plane and an empirical tensile strength of the masonry with respect to various $\frac{M_u}{V_u d_v}$ ratios, while the proposed simplified equations are based upon the masonry contribution to strength and aspect ratio. Figures 9.6-7 provide the performance of the limits with respect to fully and partially grouted walls for each set of equations for both clay and concrete masonry combined.

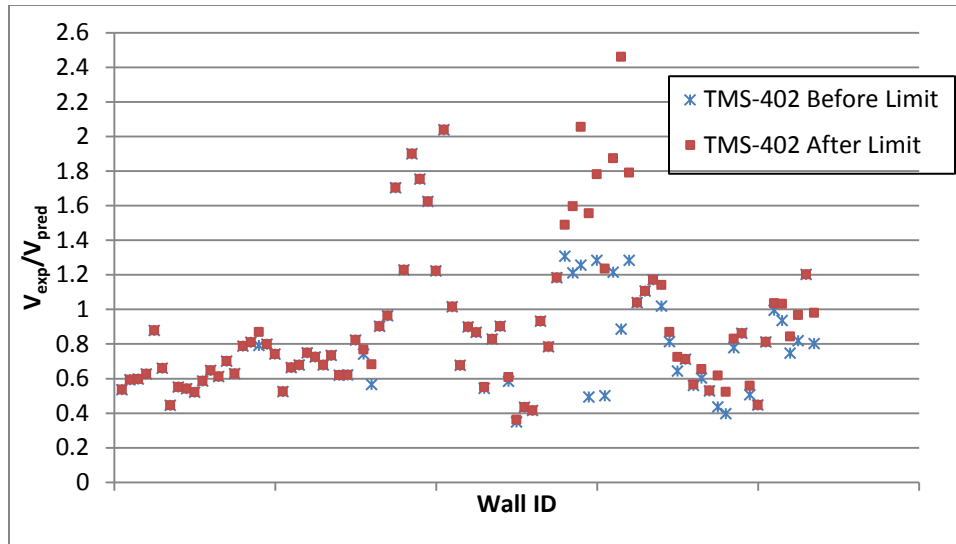


(a)

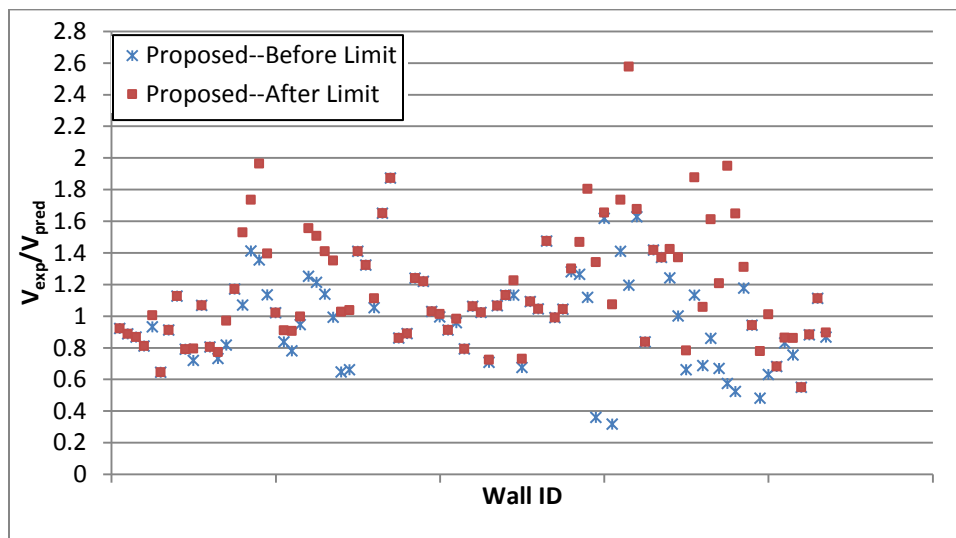


(b)

Figure 9.6 Fully Grouted Limit Performance of (a) TMS-402 and (b) Proposed Simplified Equations



(a)



(b)

Figure 9.7 Partially Grouted Limit Performance of (a) TMS-402 and (b) Proposed Simplified Equations

Table 9.7—Limit Performance

Before Limit		After Limit	
Fully Grouted Walls			
TMS-402:			
Average	0.99	Average	1.09
COV	0.18	COV	0.17
Proposed:			
Average	1.08	Average	1.25
COV	0.21	COV	0.24
Partially Grouted Walls			
TMS-402:			
Average	0.83	Average	0.92
COV	0.41	COV	0.47
Proposed:			
Average	0.99	Average	1.20
COV	0.30	COV	0.31

Figure 9.6 gives the limit performance on fully grouted walls and shows that both the TMS-402 and proposed limits perform well, as the majority of the averages that fall below 1.0 are captured and moved up with the application of the limit. Figure 9.7 gives the limit performance on partially grouted walls. The TMS-402 limit does little to capture the averages below 1.0, while the proposed limit captures more of these values. Table 9.7 gives the average $\frac{V_{exp}}{V_{pred}}$ before and after the limit. In all cases the average improved, but variation increased slightly except for the fully grouted specimens predicted by the TMS-402.

Chapter 10—Shear Friction

10.1 Shear Friction Introduction

Shear friction is a concept in shear design that deals with the available strength or resistance of an interface to an external load where a crack has occurred, or has a great potential to occur. Common cases include brackets and corbels in typical concrete construction and bed joint-to-block interfaces and wall-to-foundation interfaces in traditional masonry construction.

Shear friction quantifies the strength due to the roughness of surfaces at the crack interface, the presence of reinforcing bars perpendicular to the crack, aggregate interlock (mainly in concrete), and initial cohesion of un-cracked surfaces. Currently, the TMS-402 (2011) Building Code Requirements and Specification for Masonry Structures does not contain any provisions for shear friction for concrete and clay masonry structures. Cracking is a typical issue in masonry walls, where settlement and other external loads often cause a crack to form in masonry joints, or where rehabilitation causes new masonry to be joined to old, creating essentially a form of cold joint. Therefore, provisions should be made that acknowledge the reduction in strength of such a location, and that seeks to quantify the available strength in that portion of masonry. Much test data on concrete specimens exist, with fewer masonry test data being available. Several models have been used to represent the aforementioned factors' contributions to the resistance of the interface, and a few of the most accepted ones are presented in this chapter.

10.2 Mohr-Coulomb

Most equations that seek to model shear friction utilize the Mohr-Coulomb failure criterion, in which the allowable shear stress on a section is a factor of some initial bond strength, or cohesion, and the friction due to roughness of a surface and axial load. This is represented in Equation 10.1.

$$\tau = \tau_0 + \mu\sigma_n \quad (10.1)$$

Where,

τ_0 = Initial bond or cohesion

μ = coefficient of friction

σ_n = normal stress

Most design codes use some form of this equation, either by giving values of the coefficient of friction, or by assigning values of both cohesion and coefficient of friction to fit the data. Equations that utilize this type of criterion are labeled “cohesion plus friction” models (MacGregor and Wight, 2005). Cohesion is difficult to quantify because in the case of masonry construction, it relates to the initial bond between the mortar and block. This bond is affected by the type of mortar used, water content in the mortar, cleanliness of the block, and other factors. For this reason, some tend to lump all the factors that are hard to quantify or that are relatively insignificant into this cohesion term. This includes factors such as presence of aggregate in the slip plane, surface asperities in the crack, and dowel action of the reinforcement; this makes it hard to isolate a true “cohesion-only” value from available information on shear friction.

Other models exist that only utilize the friction portion of the Mohr-Coulomb criterion, and are thereby called simply “shear friction” or “friction” models. Equations of this type sometimes utilize high values of the coefficient of friction to match test data. Cohesion is ignored in these models, but only implicitly in the sense that the loss of the cohesion term is somewhat recovered in the high coefficient of friction. One advantage of this method, is that it is easy to use, although it does not accurately capture the behavior of the constituents involved.

10.3 Code Values/Equations

In Table 10.1 is summarized various code equations and values for the coefficient of friction.

Table 10.1—Equations for Shear Friction

Code/Design Aid	Shear Friction Equation and Limits	Coeff. of Friction (μ)	Notes
ACI 318-11	$V_n = A_{vf}f_y\mu$ $V_n \leq \min :$ $\begin{aligned} &0.2 f'_c A_c \\ &(480 + .08f'_c)A_c \\ &1600A_c \end{aligned}$	1.4λ 1.0λ 0.6λ	A_{vf} = area of shear friction reinf. f_y = yield stress of reinf. A_c = Area of concrete resisting shear transfer $\lambda = 1.0$ for normalweight concrete, 0.75 for lightweight concrete f'_c = compressive strength of concrete
PCI Design Handbook (2004)**	$A_{vf} = \frac{V_u}{\phi f_y \mu_e}$ $\mu_e = \frac{1000 A_{cr} \mu}{V_u}$ $\therefore V_n = \sqrt{\frac{A_{vf} f_y 1000 \lambda A_{cr} \mu}{\phi}}$	1.4λ 1.0λ 0.6λ	For $\mu = 1.4\lambda$: $0.3\lambda^2 f'_c A_{cr} \leq 1000\lambda^2 A_{cr}$ For $\mu = 1.0\lambda$: $0.25\lambda^2 f'_c A_{cr} \leq 1000\lambda^2 A_{cr}$ For $\mu = 0.6\lambda$: $0.2\lambda^2 f'_c A_{cr} \leq 800\lambda^2 A_{cr}$ $\lambda = \frac{f_{ct}}{6.7\sqrt{f'_c}} \leq 1.0$ A_{cr} = area of crack interface $\mu_{e\max} = 3.4, 2.9, 2.2$ for $\mu = 1.4, 1.0$, and $.6$ respectively f_{ct} = splitting tensile strength $\phi = 0.75$
CSA S304.1-04 Unreinforced	For shear between bed joints: $V_r = \phi_m \cdot 16\sqrt{f'_m} A_{uc} + \phi_m \mu P_1$ Between first course and support $V_r = \phi_m \mu C$ Limit: $V_r \leq 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g$	1.0 1.0 0.7	V_r = factored shear resistance ϕ_m = resistance factor (0.6) f'_m = compressive strength of masonry normal to the bed joint (MPa) A_{uc} = uncracked area of the cross-section (mm ²) b_w = overall web width d_v = effective depth for shear calculations, which need not be taken as less than 0.8 lw for walls, (mm) γ_g, P_1, C, P_2 = see notes 1-4
CSA S304.1-04 Reinforced	$V_r = \phi_m \mu P_2$ $V_r \leq 0.4\phi_m \sqrt{f'_m} b_w d_v \gamma_g$	1.0 1.0 0.7	
TMS 402-11	For AAC Masonry: $V_{nAAC} = \mu_{AAC} P_u$	1.0 0.75	P_u = factored axial load $\mu_{AAC} = 1.0$ (between AAC and thin-bed mortar or leveling-bed mortar) $= 0.75$ (between AAC masonry)
AASHTO LRFD-2012	$V_n = cA_{cv} + \mu(A_{vf}f_y + P_c)$ $V_n \leq \min :$ $\begin{aligned} &0.2f'_c A_{cv} \\ &0.8A_{cv} \end{aligned}$	$c: 0.4$ 0.24 0.075 $\mu: 1.4$ 1.0 0.6	c = cohesion factor (ksi) A_{cv} = area of concrete engaged in shear transfer A_{vf} = area of shear reinforcement crossing the shear plane P_c = permanent net compressive force perp. to the shear plane (kip)

Where,

*--All units are (lb) and (in.) unless otherwise noted

**--Unless otherwise noted, values for coefficient of friction correspond respectively to concrete placed monolithically, concrete placed against hardened and rough concrete, and concrete placed against hardened concrete. In the case of the CSA code, the values refer respectively to contact of masonry-to-masonry, masonry-to-roughened concrete, and masonry-to-smooth concrete

**--The limits on shear correspond to the values of the coefficient of friction

1-- $\gamma_g = 1.0$ for fully grouted masonry, solid block masonry, solid brick masonry

$= \frac{A_e}{A_g}$, but ≤ 0.5 where A_g = gross cross-sectional area (mm^2), and A_e = effective cross-sectional area (mm^2)

2-- P_1 = compressive force in the unreinforced masonry acting normal to the sliding plane, N

3-- C = compressive force in the masonry acting normal to the sliding plane, usually taken as P_d plus the factored tensile resistance at yield of the vertical reinforcing, N

4-- P_2 = compressive force in the reinforced masonry acting normal to the sliding plane, N

10.3-A Code Comparison

The ACI 318-11 Building Code and Requirements for Concrete gives shear friction provisions that are based on a friction-only model. Provisions in this code also allow for the addition of any permanent compressive stress to the $A_{vf}f_y$ term. The coefficient of friction seeks to quantify the friction of cracked faces, shearing resistance of asperities on the crack interface, and the dowel action of reinforcement.

The PCI Design Handbook (2004) also uses a friction only model, but offers an “effective” coefficient of friction that contains limits, and that decreases with increasing shear load.

The Canadian Masonry Code (CSA 304.1-04) offers shear friction equations for both reinforced and unreinforced masonry. For the unreinforced version, shear strengths are given for both the interface between the base and first course of masonry and between bed joints of any other course. An additional cohesion-type term is provided for shear in bed joints of

unreinforced masonry. The reinforced version takes account of the reinforcement at the interface but offers no cohesion term with coefficients of friction identical to the unreinforced equation.

The TMS-402 (2011) has provisions for shear friction for AAC masonry. This is a friction-only model with no terms for vertical reinforcement or cohesion.

The AASHTO Bridge Design Manual (2012) has a true form of the Mohr-Coulomb criterion, with distinct cohesion and friction terms. The coefficients of friction are identical to the ACI-318 (2011) and the PCI Design Handbook (2004). The cohesion term adds a fair amount of capacity to the wall for larger areas considered.

10.4 Previous Experimental Work

Walraven et al. (1987) tested concrete push-off specimens and found that the shear friction capacity was not affected by repeated or sustained loading. As an aggregate penetrates the concrete matrix, it creates its own “pocket”—thus, it could potentially always generate shear resistance. In masonry however, aggregate of significant size is not present in mortar, which suggests that previous loading or movement of bodies relative to the crack face could affect later performance.

Wong et al. (2007) conducted a study on the shear strength components of concrete by utilizing direct shear tests. It was reported that close to the peak capacity, shear dilation contributes approximately 70% to the total shear capacity. Dilation would be more prevalent in concrete due to larger aggregate size, but dilation in masonry would be minimal. Vinayagam (2004) reported that initial crack widths up to 0.5-0.7 mm did not have a significant effect on capacity. Harries, et al. (2012) tested reinforced concrete push-off specimens to investigate the

mechanisms of shear friction and determined it was necessary that the cracks in the joint area open sufficiently to develop the design stress, $A_{vf}f_y$. In their own specimens, the stress in the interface steel when maximum shear load was reached was much lower than the yield strength of the reinforcement. This suggests that the full yield stress of interface steel should not be used in a shear friction equation.

McCall (1996) tested the strength of masonry joints containing metal flashing. Research was conducted using primarily #4 bars placed in a triplet-test formation with flashing between the joints providing for minimal frictional resistance, as shown in Figure 10.1. Testing in this manner resulted in negligible axial load perpendicular to the joint, thus providing a value for the contribution of reinforcement that is perpendicular to a bed joint in shear.

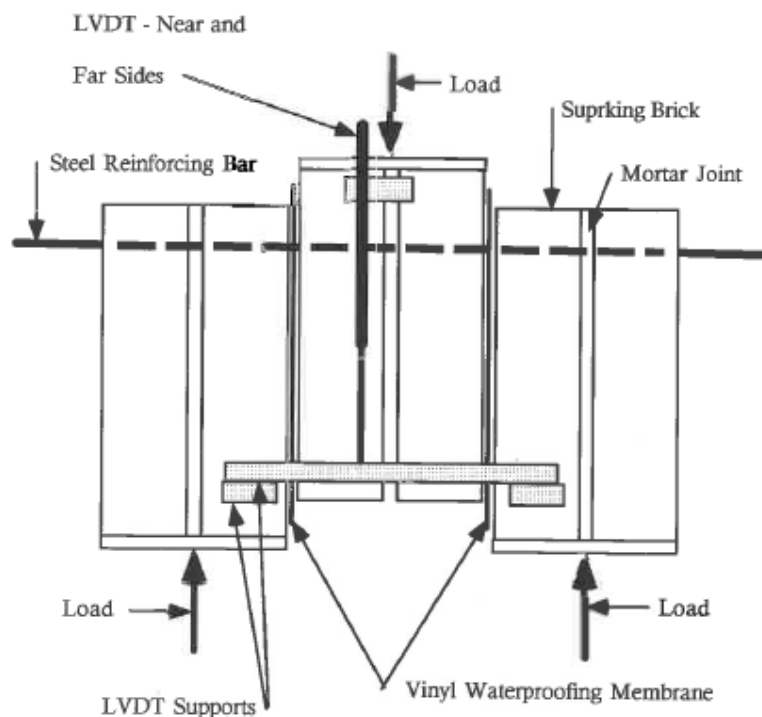


Figure 10.1 (McCall, 1996) Test Specimen Configuration

For test specimens that contained grout in every cell, the coefficient of friction was 0.54, while for specimens with grout in only the reinforced cells, the coefficient of friction was 0.36, resulting in an average coefficient of friction of 0.45. This can be considered an “efficiency factor” for vertical reinforcement, as the loading conditions require the steel to provide the primary shear resistance. A sample graph for dowel capacity versus dowel spacing in a fully grouted specimen is shown in Figure 10.2.

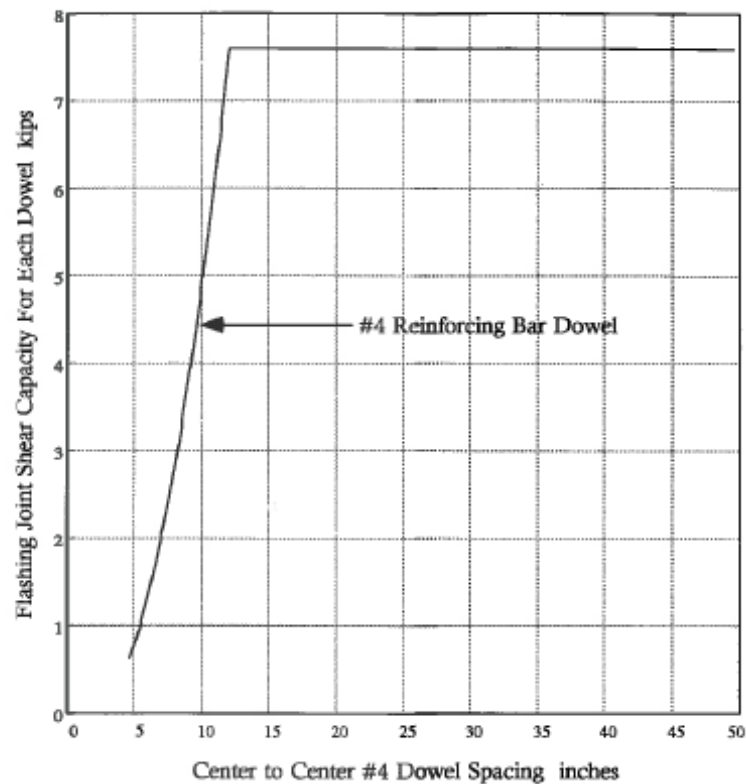


Figure 10.2 (McCall, 1996)—Joint Capacity Based Upon Shear Dowels

Van der Pluijm (1993) conducted an extensive study on the behavior and strength of bed joints in shear for unreinforced brick masonry utilizing couplet-type specimens. Approximately 54 deformation-controlled test were carried out using two single brick specimens with a bed joint, loaded in such a way to provide pure shear at the joint location, as shown in Figure 10.3.

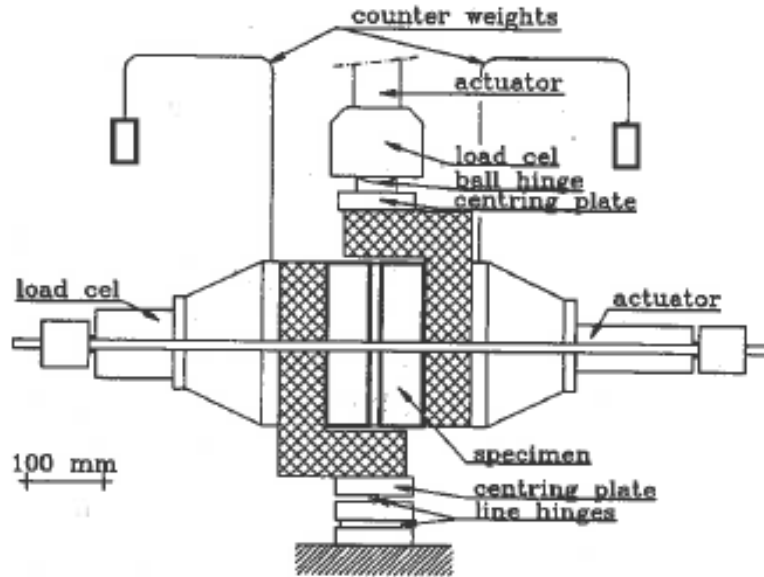


Figure 10.3—Test Configuration (van der Pluijm, 1993)

The mean coefficient of friction for all types of specimens after peak loading had occurred was 0.84 with a COV of 0.12. It was found that the cohesion reduced to zero with increasing shear deformation and was dependent on the type of brick and mortar used. Van der Pluijm also states that the coefficient of friction is independent of the type of materials used in the study, with an internal friction angle of at least 36 degrees ($\mu = 0.72$).

Toumbakari et al. (2007) utilized couplet testing of old bricks and limestone to identify two stages to the strength of masonry walls in shear friction; a chemical stage (cohesion) and a mechanical stage (friction). The chemical bond comes from the reactions of the grout/mortar and the substrate and depends on the type of grout, mortar, amount of fines, porosity, and other parameters. The mechanical stage, which occurs after slip is initiated, is affected by porosity, rugosity, binding material, thickness, applied normal stress, and reinforcement if present. Cohesion is only applicable prior to slip, and it would not be appropriate to include a cohesion term in masonry that is already cracked. From the couplet tests for old bricks with various grout

formulations, the average coefficient of friction was 0.62 with a coefficient of variation (COV) of 0.81.

Atkinson et al. (1989) examined the shear resistance of bed joints. They determined that shear resistance did not depend upon loading rate, and that after the first full cycle of stress reversal the shear resistance of the bed joint became constant, defined by residual shear resistance. Considerable differences were found in the initial peak and post-peak shear resistance due to the degradation of the cohesion. Atkinson et al. (1989) also summarized masonry coefficient of friction data which provided an average coefficient of friction for clay brick of 0.76 with a COV of 0.18 and an average coefficient of friction for concrete block of 0.80 with a COV of 0.17. Mullins et al. (1989) conducted couplet tests on bricks that were laid on a concrete foundation with increasing degrees of surface roughness, various mortar types, and dryness conditions. Specimens were loaded until the joint broke, then loading continued and the shear strength after bond failure was determined. Results of 84 tests produced a mean coefficient of friction after loss of cohesion of 0.96 with a COV of 0.32. Khalaf and Naysmith (1997) presented a summary for the coefficient of friction from various researchers (many of which were not included in the summary by Atkinson et al. (1989)) that included different brick/mortar combinations and testing methods. The average coefficient of friction for clay bricks was 0.70 with a COV of 0.30. It was also stated that brick coring had no real effect on bond strength.

Hamid et al. (1979) evaluated shear friction resistance of concrete blocks. After failing specimens to determine their bond strength, the specimens were tested again with varying levels of pre-compression to determine residual shear capacity. The average coefficient of friction was 1.13 with a COV of 0.22.

Hidalgo et al. (1979) conducted an extensive study on concrete and brick masonry single piers to evaluate their strength in shear. Ten walls from this study failed in a shear sliding mode and are suitable for analysis in this investigation. Important wall parameters are summarized in Appendix E.

Koutalan (2012) tested six reinforced concrete masonry shear walls to determine their sliding shear capacity. Four of these walls failed with significant sliding and are therefore suitable for analysis using shear friction concepts. Additionally, there are specimens that exist that failed in sliding shear from Shing et al. (1990), Voon et al. (2006), and Hernandez (2012). All of these specimens were square, fully-grouted walls that utilized concrete masonry units in their construction. For the most part, all walls other than those tested by Koutalan were “accidental” failures in sliding where the intended failure mode was either flexure or shear, but sliding predominated. The specimens by Hernandez are borderline sliding specimens, as only 17%-42% of their resistance to maximum loads was deemed to have come from sliding shear (Hernandez, 2012), but are used in analysis in future sections as there is a lack of data for wall specimens failing in sliding shear. Appendix E is a summary of these experimental results.

A summary of the average values for the coefficient of friction from previous researchers is presented in Table 10.2. An overall average value of the coefficient of friction is 0.83, with the average of brick, 0.78, being slightly smaller than for block, 0.96.

Table 10.2—Coefficients of Friction from Past Research

Researcher	Specimen Type	Average μ	COV
Toumbakari et al. (2007)	Brick	0.62	0.81
Atkinson et al. (1989)	Brick	0.76	0.18
Van der Pluijm (1993)	Brick	0.84	0.12
Mullins et al. (1989)	Brick	0.96	0.32
Khalaf et al. (2007)	Brick	0.70	0.30
Hamid et al. (1979)	Block	1.13	0.22
Atkinson et al. (1989)	Block	0.80	0.17

Inclined plane tests on concrete masonry units with varying degrees of roughness will be presented in the following chapter, with the results used to analyze the specimens failing in sliding shear from other researchers.

Chapter 11—Inclined Plane Tests of Concrete Masonry Units

11.1 Introduction to Testing

Sliding shear is a type of failure that could potentially govern the failure mode of a shear wall, with a failure being between courses of masonry or between a concrete foundation and bottom course of masonry. Much data exists on shear friction in concrete, but little exists in the world of masonry.

It is known that isolating the contributions of dowel action due to reinforcement, cohesion of bonded surfaces, and surface friction solely between the masonry is very difficult in a large scale test where a wall is forced to slide, which is why some of the equations presented often lump these terms together. A true coefficient of friction is unique to a set of surfaces, and it is believed that one can isolate a true coefficient of friction for a masonry unit by isolating only the units themselves with no cohesion and no reinforcement.

At an attempt to determine the true coefficient of friction for concrete masonry, inclined plane tests (often referred to as tipping tests) of traditional 8 in. lightweight CMU have been performed, the angle at which one surface slides across another determined, and the coefficient of friction for the pair of surfaces calculated. Test setup, measurement, block/mortar configurations, and results are given in the following sections.

11.2 Block Configuration and Construction

It was desired to test several different surface interactions to simulate potential conditions in a masonry wall. The surfaces tested include:

1. Dry block on block
2. Block on a freshly broken mortar joint (aged 7 and 28 days)
3. Block on a mortar joint subjected to cycles of push/pull (aged 7 and 28 days)
4. Block on a trowel-smoothed mortar joint

These interactions simulate sliding conditions that most closely relate to

1. A mortarless sliding surface (can serve as a baseline)
2. A surface that exhibits the condition of a bed joint just after cracking in a masonry wall
3. A cracked surface that has been subjected to cyclic loading (to shear off any surface asperities in the freshly broken joint)
4. A relatively smooth but uneven concrete base (not intentionally roughened)

Specimens consisted of 2-block masonry prisms and were laid using conventional 8 in. lightweight CMU and Quikrete Pro-Finish Type S Mortar Mix (meets or exceeds requirements of ASTM C270 and ASTM C1714 for type S mortar) with a full mortar bed and $\frac{3}{8}$ in. tooled mortar joint. Approximately 0.185 ft³ of water was added to 80 lbs. of mortar mix and pertinent data for mortar is given in Table 3.

Table 11.1—Mortar Properties

Type S Mortar	
Mixing Temperature	55 degrees Fahrenheit
Flow (average of three specimens)	103%
7-Day Compressive Strength (average of three specimens):	2641 psi
28-Day Compressive Strength (average of six specimens):	3496 psi

Ten prisms were created to use with broken bed joint testing and five blocks that contained a mortared bed joint smoothed with a trowel were allowed to cure outdoors at an

approximate ambient temperature of 50 degrees Fahrenheit, uncovered, up to 28 days to simulate actual wall conditions as shown in Figure 11.1. Time of year was late fall, early winter.



Figure 11.1—Storage Conditions for Test Specimens

11.3 Testing

At 7 and 28 days, the prisms were carefully loaded onto the bond wrench shown in Figure 11.2 and the joint was broken by applying pressure at either end of the bar extending from the top block in each prism, with care taken not to disturb the interface between the broken joint after breakage.



Figure 11.2—Bond Wrench

A view of the broken joint is shown in Figure 11.3. This was a typical breakage for both 7 and 28 day specimens as the majority of the specimens had a joint to break with the entirety of the mortar being contained on one block (mortar is on the bottom block).



Figure 11.3—View of Freshly Broken Joint

After breaking the joint, the blocks were restacked into their original configuration and loaded into a hinged apparatus designed to hold the bottom block stationary while allowing the top block to slide when the entire configuration was slowly tipped using a chain hoist. Figure 11.4 shows a rendering of the device which utilizes doubly-thick 2 in. x 12 in. boards, hinges to allow rotation of the top, and angle with slotted holes to allow adjustment on the side and back to accommodate various sizes of brick and block.

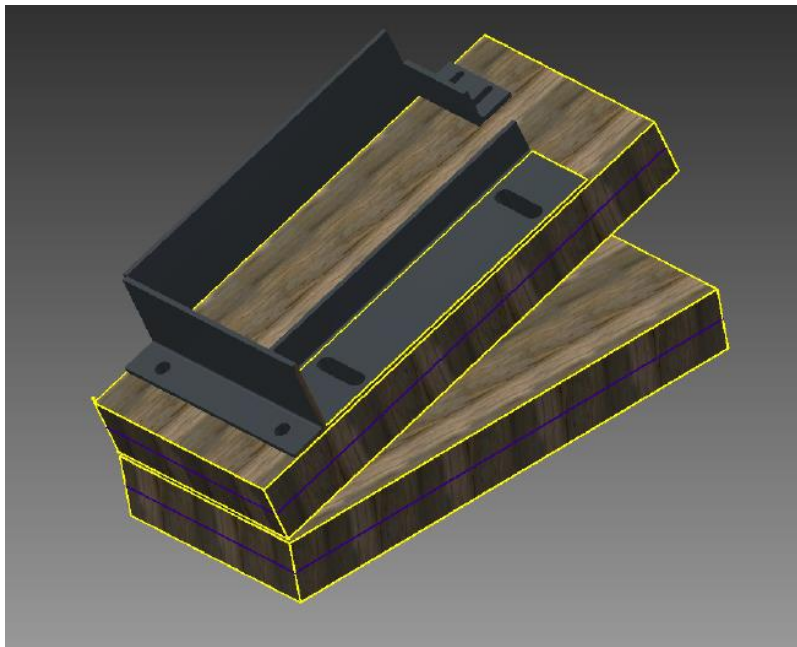
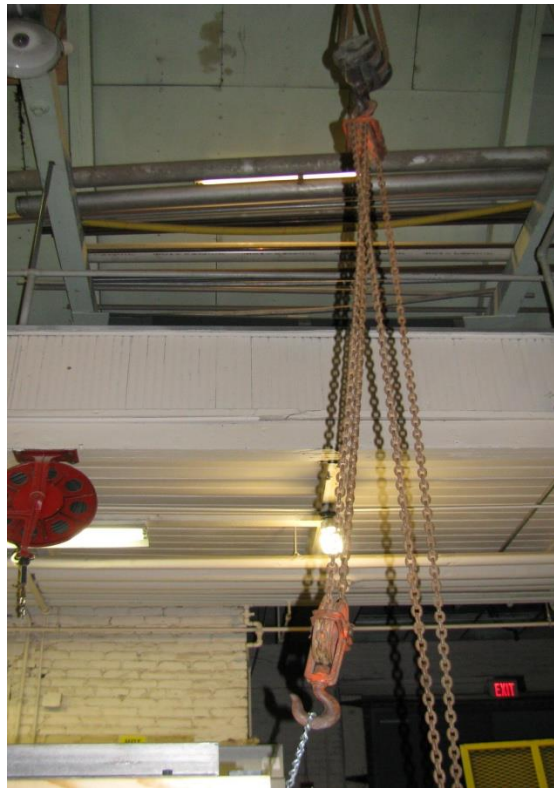


Figure 11.4—Tipping Apparatus

The device was clamped to a table and a chain hoist was attached at the rear and mounted to the ceiling to provide a controlled way to slowly tilt the top portion of the device. This is shown in Figure 11.5.



(a)



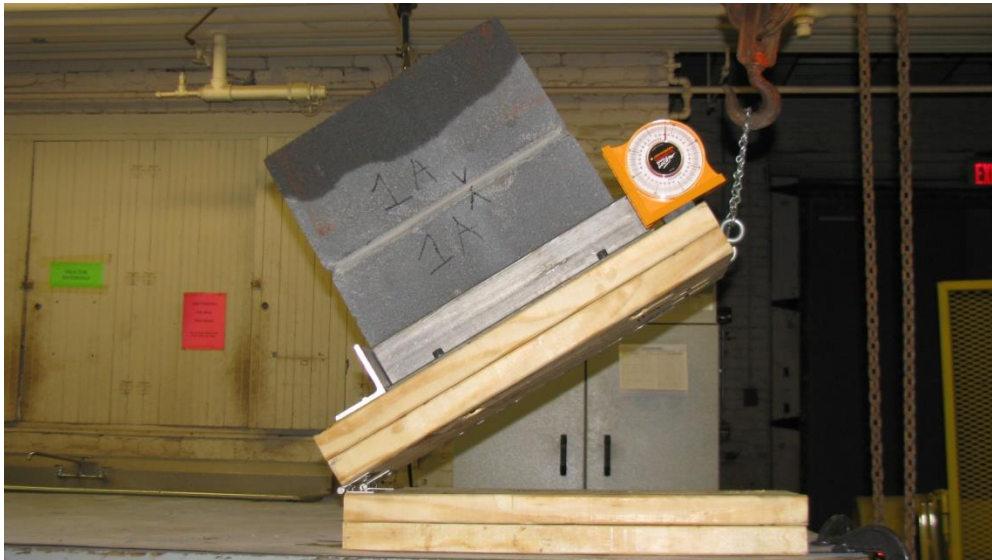
(b)

Figure 11.5—Method of Inclination

Degree of tilt was measured by a dial indicator as shown in Figure 11.6.



(a)



(b)

Figure 11.6—Method of Angle Measure

Each block was tilted until it slid and the degree at which sliding first started to occur was recorded. This was done for both 7 and 28 day tests. Figure 11.7 shows a block at the end of the test.



Figure 11.7—View at End of Test

After the blocks slid, they were restacked, subjected to 20 cycles of push and pull (to shear off asperities), blown with compressed air, and then loaded back into the device and tipped once more. Like the broken joint specimens, “dry” block-on-block sliding was tested as well as block-on-smoothed mortar. These were conducted in the same manner as before, but without the push/pull loading, as there were no substantial asperities for these types of specimens.

11.4 Surface Roughness and Results

In the paragraphs below, each series of surface roughness is discussed, along with the results of each test at 7 and 28 days, if applicable. Coefficients of friction are determined based upon principles of statics in Figure 11.8.

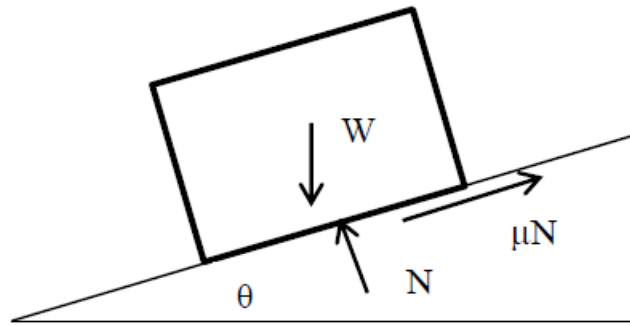


Figure 11.8—Free Body Diagram of Sliding Specimen

Where,

W = weight of the object

N = normal force acting perpendicular to the sliding surface

μ = coefficient of static friction

θ = angle of inclination

At impending motion, this provides a coefficient of friction equal to:

$$\mu = \tan \theta \quad (11.1)$$

11.4-A Blocks Without a Mortar Joint

Blocks that had not been subject to any other tests were utilized to perform dry, block-on-block sliding. This is an unlikely occurrence in the field, but can serve as a baseline to compare other degrees of surface roughness to. Additionally, this test can serve as a guideline for a very even, but fairly rough (containing many small voids) surface-to-surface contact.

11.4-B Trowel-Smoothed Mortar Joint

This simulates a relatively smooth, but uneven concrete base. This test is intended to represent blocks placed against a concrete foundation. A typical specimen is shown in Figure 11.9, while Figure 11.10 shows the “smooth but uneven surface” between blocks as daylight can be seen through the interface. Generally, there was no more than a $\frac{1}{16}$ in. gap.



Figure 11.9—Trowel Smoothed Mortar Joint

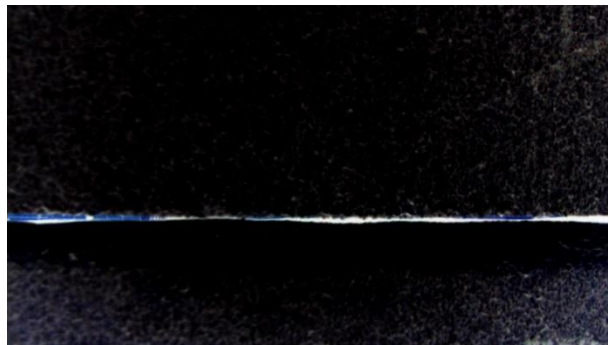


Figure 11.10—View of Daylight Through the Joint

11.4-C Broken Joint Prisms

This simulates the condition of a bed joint just after cracking in a masonry wall. A view of a typical broken joint is shown in Figure 11.11. The majority of the specimens broke with all of the mortar remaining on one block, while the other block contained the fines or “fingers” from absorption of the mortar into the block (mortar is on the right-most block in Figure 11.11). Figure 11.12 gives an indication of surface roughness.

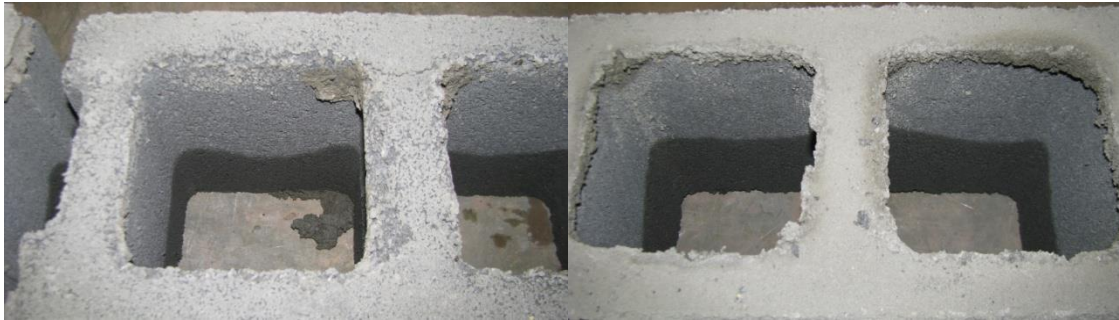


Figure 11.11—View of Broken Joint (mortar on the right)



Figure 11.12—Broken Joint Surface Roughness

As can be seen in Figure 11.12, there were several protrusions on each block around the cell opening because of mortar overflow out of the joint during placement. Some of these protrusions were as high as $\frac{3}{16}$ in. around the inside edge of the block cell. For the 7 day tests, these protrusions were left in place, as they would typically be there during actual wall construction. However, for the 28 day tests, any protrusion around the edge of a cell that exceeded approximately $\frac{1}{8}$ in. was knocked off prior to testing the prism as an attempt to provide for a more consistent test.

11.4-D Blocks Subjected to Cyclic Push/Pull

This simulates a cracked surface that has been subjected to cyclic loading (to shear off any surface asperities). After testing of the broken-joint prisms, the blocks were subjected to 20 cycles of manual push/pull to shear off major asperities between the two surfaces of mortar and block. This was intended to simulate more adequately the conditions in a wall that has had a bed joint crack form and then subjected to cyclic loading of some kind. The push/pull cycles successfully sheared off most asperities making the surfaces considerably smoother, as shown in Figure 11.13.

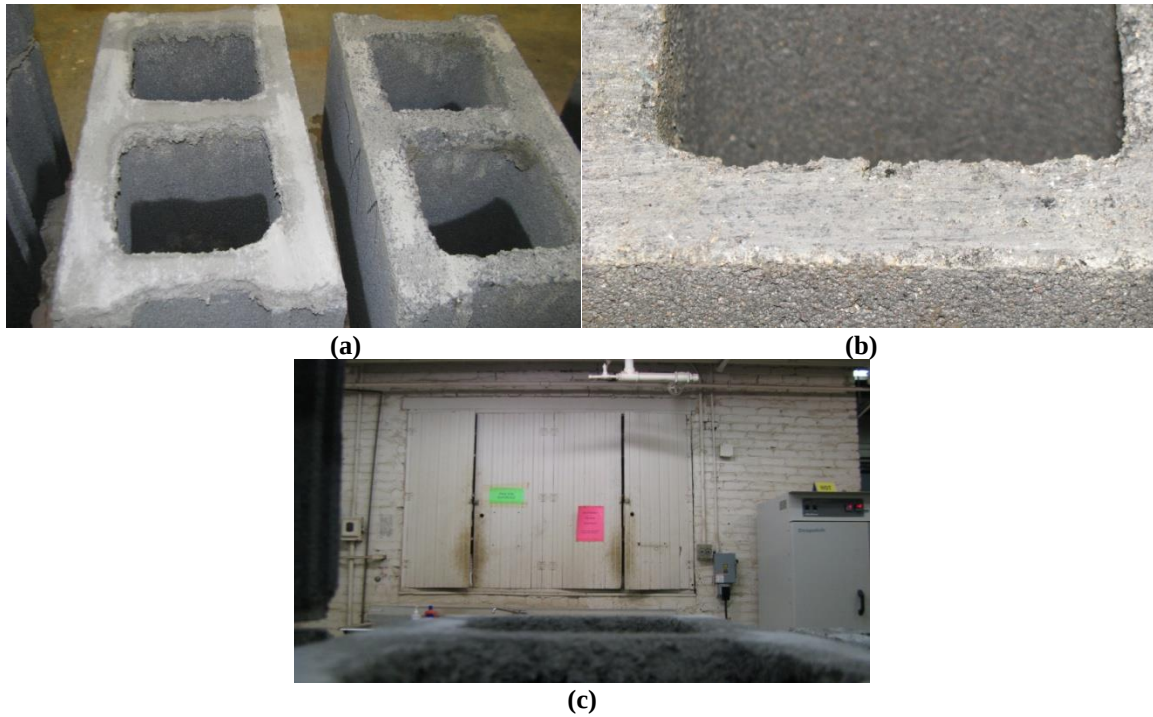


Figure 11.13 a) Prism after Push/Pull Cycles, b) View of Fines Embedded in Block, and c) View of Sheared Asperities

Even under a very low push/pull load, the asperities were easily sheared off after a few cycles—a characteristic not found in traditional concrete due to the coarse aggregate and higher strengths of cement paste. In masonry mortar there are no real coarse aggregate and asperities will shear off readily, leaving a generally smooth surface behind. After the 20 cycles, the surfaces were blown with compressed air to get rid of the debris and dust created during the cyclic loading, although the blocks still retained many of the embedded fines in the voids of the block created during the push/pull cycles.

11.5 Summary of Results

The results of 28-day tests are presented in Table 11.2 with average coefficients of friction for each surface condition.

Table 11.2—Results of Inclined Plane Tests

Specimen	Coefficient of Friction			
	Unmortared Joint	Trowel-Smoothed Joint	Broken Joint	Broken Joint Subjected to Push/Pull
1	1.04	0.78	1.33	0.87
2	1.04	0.70	1.11	0.78
3	0.84	0.81	1.04	0.93
4	0.87	0.84	1.88	0.97
5	0.87	0.84	1.38	0.97
Average	0.93	0.79	1.31	0.90
COV	0.11	0.07	0.25	0.09

Table 11.3 gives a summary of average test results relative to increasing surface roughness.

Table 11.3—Coefficient of Friction for Increasing Surface Roughness

Test	Simulates	Average Coefficient of Friction
Smoothed Mortar Joint	Smooth but uneven surface	0.79
Joint Subjected to Push/Pull	Smooth and regular surface	0.90
Unmortared Bed Joint	Rough but even surface	0.93
Broken Bed Joint	Rough and irregular surface	1.31

The 28 day tests are more representative of a masonry wall since the mortar protrusions in the cells of the blocks were intentionally chipped off prior to testing. In a masonry wall with substantial in-plane loading, the resistance these protrusions have against actual loadings is negligible, but in a small-scale test, it could be substantial. Therefore, the 28 day tests are deemed to be more representative and are the only ones reported.

The Wilcoxon rank-sum test was used to test for statistical significance between surfaces. Results are shown in Table 11.4. For a significance level of 0.05 all conditions except surface comparisons of Unmortared-Push/Pull and Trowel Smoothed-Push/Pull were deemed statistically significant, with the latter barely falling outside of the range for significance. This suggests that surface roughness does play a significant part in the shear resistance of masonry units.

Table 11.4—p-Values

Surface	Unmortared	Broken Joint	Push/Pull	Trowel Smoothed
Unmortared		0.024	0.86	0.024
Broken Joint	0.024		0.008	0.008
Push/Pull	0.86	0.008		0.071
Trowel-Smoothed	0.024	0.008	0.071	

11.6 Discussion/Analysis of Inclined Plane Tests

Tests that simulate the potential surfaces found within the sliding plane of a masonry wall that contains a broken bed joint have been presented. Depending on the degree of roughness, the

calculated coefficients can differ as much as 40%--a significant amount when one is trying to use these values to calculate sliding resistance of a masonry wall. It is believed that the smoothed joint specimens that represent blocks sliding against smooth but uneven concrete and the push/pull specimens are more representative of what could be found in a wall for sliding between a bottom course and foundation and between courses, respectively. Potentially, either value could be used for sliding at the base, as a mortar bed is laid before the first course of blocks, so the same scenario could take place as with the specimens subjected to push/pull with fines being retained in the foundation base just like they were retained in the block specimens. Depending upon initial degree of roughness, the fines could provide increased friction—similar to the addition of sand on a slick road.

The existing data in Appendix E , data acquired during the inclined plane tests and previous data in Table 10.2 have been utilized to analyze the surface friction contribution to shear strength of a broken horizontal joint in a masonry wall. For half of the specimens in Appendix E it was noted in the corresponding references that sliding at the base occurred, and for the other half, failure between courses occurred. Factors of 0.79 and 0.90 as calculated in the inclined plane tests would generally correspond to these sliding conditions. However, for use in a design code, a single coefficient of friction of 0.80 is proposed.

For walls that had sliding at the base, contributions from axial load were subtracted from the reported failure load and divided by $A_s f_y$ for the vertical reinforcement to come up with a theoretical percentage relating to the efficiency of vertical reinforcement. A value of 59% (COV of 0.21) resulted, which matches well with the findings of McCall (1996) for fully grouted specimens (54%).

Equation 11.2 is proposed for calculation of the shear friction strength of clay and concrete masonry walls.

$$V_{n-sl} = \mu(P_n) + \eta(A_s f_y) \quad (11.2)$$

Where,

V_{n-sl} = nominal strength of a masonry wall in sliding shear

μ = coefficient of friction

P_n = axial load

η = efficiency factor for vertical reinforcement

A_s = total area of vertical reinforcement crossing the shear plane

f_y = yield stress of vertical reinforcement

As presented in Table 10.1, concrete codes combine vertical reinforcement yield strength and axial load, and provide a variable coefficient that is applied to these values. This follows with the theory that coarse aggregate will cause increased vertical reinforcement contribution as the portion of wall above a crack is forced to move up over the asperities. This can generally be considered an additional clamping force provided by the vertical reinforcement. The proposed vertical steel contribution equation for masonry is lower than that for concrete and is a constant. In a masonry wall, there is no coarse aggregate and dilatancy would theoretically decrease to zero as increased shear displacement contributes to surface smoothing (Lourenco, et al 2003). Due to a lack of significant vertical dilation, it is believed that the vertical reinforcement contribution in masonry can be mostly attributed to dowel action. Therefore, the vertical reinforcement contribution is taken as less than the yield strength and as a constant for masonry. As shown by the dowel tests of McCall (1996), a steel bar crossing the shear plane can contribute

approximately 45% of its yield strength to resist shear (average of partially and fully grouted specimens). A value of 0.45 will be used for the steel efficiency factor.

Appendix E gives results from using Equation 11.2 with the proposed coefficients. Also shown are coefficients of friction calculated using Equation 10.1 considering the vertical reinforcement yield strength and applied axial load to act together (no cohesion is considered). The average coefficient of friction for use in Equation 10.1 is 0.61 for concrete and base sliding. This is considerably lower than most values in Table 10.2 and values used in most codes in Table 10.1. The proposed model results in reduced variability over a model using Equation 10.1, with an average $\frac{V_{exp}}{V_{pred}}$ of 1.14 and a COV of 0.10. The proposed method provides approximately the same level of conservatism and a lower variation than the TMS 402 masonry code (2011) shear strength provisions for solidly grouted shear walls, which was shown in Chapter 9 to have a $\frac{V_{exp}}{V_{pred}}$ of 1.09 and COV of 0.17.

Appendix E also contains 10 specimens by Hidalgo et al (1979) which were constructed with flanges on top and bottom of the wall and with shear keys between the flanges and supports. This forces a sliding failure between a course above the bottom flange, instead of at the base. Based on the inclined plane tests, a higher coefficient of friction should be used for sliding between courses. Using a coefficient of friction of 1.31, as determined from the inclined plane tests for broken joint prisms, the average $\frac{V_{exp}}{V_{pred}}$ for Hidalgo et al (1979) becomes 1.01 with a COV of 0.11. A coefficient of 1.10 provides an average $\frac{V_{exp}}{V_{pred}}$ of 1.13 with a COV of 0.10, which is on the same order of conservatism as the concrete masonry results presented in Chapter 3.

11.7 Recommendations

A new shear friction model is proposed for concrete and clay masonry shear walls in which the contribution from friction due to applied normal loads and the contribution from reinforcement crossing the shear plane is separated. A more robust model would include a cohesion term (for serviceability concerns) and varying coefficients for initial surface roughness, but due to the variability in bond strength of mortar and surface conditions, and to simplify design, a single coefficient of friction is proposed. In addition, a cohesion term would diminish after cracking.

A single value of the coefficient of friction due to applied normal forces, 0.80, is proposed for base sliding, and a value of 1.10 for sliding between courses. The efficiency factor for reinforcement across a shear plane of 0.45 is proposed. This model is shown to reduce the variation in predicted capacities by a factor of two for base sliding over the more traditional shear friction model utilizing a single coefficient of friction, while adhering to trends in conservatism in fully grouted shear walls in the TMS 402 Building Code Requirements for Masonry Structures (2011).

A potential entry for the MSJC TMS-402 Strength Design section on shear is as follows:

3.3.4.1.2.3 Nominal shear strength provided by sliding resistance for masonry shear walls—Nominal shear strength provided by shear friction in masonry walls, V_{n-sl} , shall be computed as follows:

$$V_{n-sl} = \mu(P_n) + \eta(A_s f_y)$$

Where,

V_{n-sl} = nominal strength of a masonry wall in sliding shear

μ = coefficient of friction; values of 0.80 for base sliding and 1.10 for sliding between courses

P_n = axial load

η = efficiency factor for vertical reinforcement; 0.45

A_s = total area of vertical reinforcement crossing the shear plane

f_y = yield stress of vertical reinforcement

COMMENTARY:

3.3.4.1.2.3 Nominal shear strength provided by sliding resistance for masonry shear walls—The equation for sliding shear resistance for masonry walls was developed based upon Mohr-Coulomb theory and inclined plane tests of concrete units with varying degrees of bed joint roughness, and matches well with prior research. Varying forms of this equation appear in other codes with coefficients of friction that lump together contributions from vertical steel, cohesion, and aggregate interlock, however, this equation seeks to quantify the frictional resistance provided by the masonry and vertical reinforcement only, with an efficiency factor applied to the vertical reinforcement to account for some bars not reaching yield. Use of cohesion in this equation is neglected because resistance due to cohesion is lost after a crack forms. The average $\frac{V_{exp}}{V_{pred}}$ of test specimens of masonry walls is 1.14 with COV of 0.10 and 1.13 with COV of 0.10 for base sliding and sliding between courses, respectively.

Chapter 12—Summary and Conclusions

In Chapter 3, the performance of the TMS-402 strength design equations for shear was assessed, and it was concluded that they are inadequate to consistently calculate the in-plane shear strength of partially grouted masonry walls, but performed well for fully grouted walls. Including the extension of scope of Chapter 5, the average $\frac{V_{exp}}{V_{pred}}$ for 108 concrete and clay fully grouted walls was 1.09 with a COV of 0.17, while 87 partially grouted walls had an average of 0.92 with a COV of 0.47. The New Zealand Code equations and the equations of Matsumura (1986) performed the best with the least variation.

A strut-and-tie method based on current concrete codes and adjusted specifically for masonry walls was presented in Chapter 4. This method is all-encompassing and considers all vertical and horizontal reinforcement, applied axial load, and wall geometry to predict the in-plane strength of masonry walls. Out of 35 fully grouted walls from the database and 13 finite element walls constructed with a macro-model approach in Chapter 5, strength calculations produced an average $\frac{V_{exp}}{V_{pred}}$ of 1.63 with a COV of 0.26, while 23 partially grouted walls from the database along with 12 finite element analysis walls produced an average of 1.44 with COV of 0.25. The guidelines for this method are as follows:

Strut-and-Tie Procedure for Masonry:

1. Idealize all horizontal and vertical reinforcement in the wall as straight lines, intersecting at points (nodes). Regions that are bounded by steel will be denoted as “panels.” An initial steel arrangement may be assumed based upon TMS-402 prescriptive requirements and/or experience of similar walls and loadings.
2. Consider a unidirectional loading, having the horizontal load to act at the top-most bond beam, if located sufficiently near the top of the wall (otherwise, the load should act at the top of the wall). Any applied axial load shall be distributed to the nodes along the top of the wall, based on tributary area.
3. Draw struts in every panel of the wall between points of steel intersection to properly coincide with the assumed loading direction.

4. The angle this strut makes with the vertical reinforcement will define the strut angle. To determine the strut width, use the following relation:

$$w_{sd} = \frac{w_t}{\cos \theta}$$

Where,

w_{sd} = maximum width of a diagonal strut (in.)

w_t = minimum of reinforcement diameter plus 2 x the clear cover and the wall thickness (in.)

θ = angle between vertical tie and diagonal strut

5. Assume the strut width calculated by Equation 4.8 to be centered about the lines drawn in Step 3 such that any struts and ties acting at the node will be coincidental.
6. The thickness of a diagonal strut is assumed to be the thickness of the wall for a fully grouted wall, and the thickness of the combined minimum face shell thickness for a partially grouted wall.
7. Assume that the vertical ties framing into a node have a resultant coinciding with the line of force of the vertical reinforcement. Any masonry contribution to tensile strength is neglected.
8. To define horizontal and vertical strut areas, the cross-sectional area shall be limited to the dimensions of the cells or courses containing reinforcement. Any reinforcement contribution to compression in a strut is neglected.
9. The nodal zone is defined as that region encompassed by overlapping areas of struts and ties, and is idealized using straight cross sections as in Figure 4.9
10. Tie strengths are assumed to equal the yield strength of the reinforcement.
11. Strut strengths are assigned as below:

$$F_{ns} = f_{me} A_{cs}$$

Where,

F_{ns} = strut strength, (lb)

f_{me} = effective masonry strength, taken as $0.85f'_m$ (psi)

A_{cs} = cross-sectional area of a strut, (in²)

12. The calculation of nodal zone strengths on any one face shall be as follows:

$$F_{nz} = A_{nz} f_{me-n}$$

Where,

F_{nz} = strength of a nodal zone

f_{me-n} = effective masonry strength of a node, taken as $0.85\beta_n f'_m$, psi

β_n = 1.0 for nodes with struts only, 0.8 for nodes anchoring one tie, and 0.6 for nodes anchoring more than one tie; zero-force bars in a layout need not be counted as ties

A_{nz} = area on a face which is perpendicular to the line of action of the force in the strut. Widths of a nodal face are not to be taken greater than any calculated strut width framing into that face that would correspond to a fully grouted wall. It shall be permitted to use fully-grouted f'_m for calculation of nodal strengths.

Chapters 6-9 presented a simplified piecewise strut-and-tie method that utilizes separate contributions from the masonry, reinforcement, and applied axial load. The masonry contribution is based on assumed 45 degree orientation of principle compressive stresses and the formation of a single strut in a square section of wall. Horizontal reinforcement contribution is based on development length of reinforcement and assumed efficiency of that reinforcement. Utilizing this method for 108 fully grouted clay and concrete walls produced an average $\frac{V_{exp}}{V_{pred}}$ of 1.25 and a COV of 0.24, and an average of 1.17 and a COV of 0.31 for 87 partially grouted walls. Comparisons of the simplified method were made against current TMS-402 strength design equations for shear, and it was found that the simplified strut-and-tie method reduced variability by approximately 35% for partially grouted walls over the TMS-402. The simplified piecewise equations are as follows:

Overall Strength:

$$V_n = V_m + V_a + V_s$$

Partially Grouted Masonry Contribution:

$$V_m = 0.7(f'_m)(t_f)(b_t)(\frac{l}{h})$$

Fully Grouted Masonry Contribution:

$$V_m = 0.7(f'_m)(b_t^2)(\frac{l}{h})(\zeta)$$

Axial Load Contribution:

$$V_a = 0.25 P_n$$

Horizontal Reinforcement Contribution:

$$V_s = \lambda((N) A_h f_y - \psi_s A_h f_y)$$

Limits:

$$V_{n\max} = 1.75V_m \text{ if } \frac{h}{l} \geq 1.0$$

$$V_{n\max} = 1.15V_m \text{ if } \frac{h}{l} \leq 0.25$$

$$V_{n\max} = 3.5V_m \text{ for squat or square walls that are 12 ft. and over in height}$$

Where,

$\zeta = 0.65$ for walls up to 8 in. thick, 0.40 for walls over 8 in. thick

t_f = total face shell thickness

b_t = thickness of a masonry unit

f'_{me} = effective masonry strength, equal to $0.7f'_m$

P_n = applied axial load

$\lambda = 0.6$ for fully grouted walls, and 0.4 for partially grouted walls

N = # of horizontal reinforcing bars from bottom of wall to top of section defined by a square portion of wall

A_h = area of one horizontal bar

f_y = yield strength of horizontal reinforcement

ψ_s = the number of reinforcement bars or bed joint ladders in the top and bottom two courses of masonry in the square section of wall

Inclined plane tests on concrete masonry units utilizing various degrees of surface roughness was performed and summarized and a shear friction model was proposed for clay and

concrete masonry in Chapter 11 that separates the coefficients for contributions of steel and axial load. This model has been shown to reduce variability by a factor of two over a more traditional approach of combining the reinforcement and axial load terms. Equation is as follows:

$$V_{n-sl} = \mu(P_n) + \eta(A_s f_y)$$

Where,

V_{n-sl} = nominal strength of a masonry wall in sliding shear

μ = coefficient of friction, 0.8 for base sliding, 1.10 for between course sliding

P_n = axial load

η = efficiency factor for vertical reinforcement, 0.45

A_s = total area of vertical reinforcement crossing the shear plane

f_y = yield stress of vertical reinforcement

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Appendices

Appendix A

Walls—Properties and Design Values

General Notes:

- for some researchers, specific steel spacing and size data were not given; in these cases, an appropriate value is assumed. Hidalgo, Chen and Sveinsson had horizontal steel spacing evenly distributed throughout the height of their walls; taken literally, this leads to unconventional spacing, and the values presented here are average spacing values based upon written data and visual representations of steel spacing given in the appropriate references
- for both vertical and horizontal reinforcement, an area per placement (in^2/pl) is given
- V_{sp} and H_{sp} refer to the spacing of vertical and horizontal bars, respectively
- “# Disc.” refers to the number of horizontal bars discounted in the proposed equations
- for partially grouted walls, ungrouted f'_m is used when available, otherwise, the compressive strength given in the reference is used
- A mark of “---” signifies that the entity is unknown

Table A-1—Fully Grouted CMU Wall Properties

Wall #	Researcher	Wall ID	H (in)	L (in)	t (in)	AR	Vert. Reinf. (in ² /pl)	Vsp (in)	ρ_v (%)	f_{yv} (psi)	Horiz. Reinf. (in ² /pl)	# of locations	Hsp (in)	ρ_h (%)	f_{yh} (psi)	Axial Load (k)	f'_m (psi)	Avg. Vmax (k)
1	Shing	3	72	72	6	1.0	0.60	16	0.740	72000	0.11	5	16	0.140	56000	109.35	3000	102.50
2	Shing	4	72	72	6	1.0	0.60	16	0.740	72000	0.11	5	16	0.140	56000	0.00	2600	79.50
3	Shing	5	72	72	6	1.0	0.60	16	0.740	72000	0.11	5	16	0.140	56000	40.50	2600	86.50
4	Shing	7	72	72	6	1.0	0.60	16	0.740	72000	0.11	5	16	0.140	56000	40.50	3000	97.00
5	Shing	9	72	72	6	1.0	0.31	16	0.380	64000	0.11	5	16	0.140	56000	109.35	3000	96.00
6	Shing	11	72	72	6	1.0	0.60	16	0.740	72000	0.20	5	16	0.240	67000	0.00	3200	92.00
7	Shing	13	72	72	6	1.0	0.44	16	0.540	65000	0.20	5	16	0.240	67000	109.35	3300	112.50
8	Shing	14	72	72	6	1.0	0.44	16	0.540	65000	0.11	5	16	0.140	56000	109.35	3300	105.00
9	Shing	15	72	72	6	1.0	0.44	16	0.540	65000	0.20	5	16	0.240	67000	40.50	3300	88.00
10	Shing	16	72	72	6	1.0	0.60	16	0.740	72000	0.20	5	16	0.240	67000	109.35	2500	120.50
11	Voon	A1	71	71	6	1.0	0.49	16	0.620	46100	0.04	5	16	0.050	46539	0.00	2552	47.19
12	Voon	A2	71	71	6	1.0	0.49	16	0.620	46100	0.04	1	71	0.010	46539	0.00	2552	41.80
13	Voon	A4	71	71	6	1.0	0.49	16	0.620	46100	0.11	2	32	0.060	46539	0.00	2465	47.64
14	Voon	A7	71	71	6	1.0	0.49	16	0.620	46100	0.04	5	16	0.050	46539	28.32	2726	58.88
15	Voon	A8	71	71	6	1.0	0.49	16	0.620	46100	0.04	5	16	0.050	46539	14.16	2726	55.51
16	Voon	A9	142	71	6	2.0	0.78	16	0.970	46100	0.04	9	16	0.050	71600	14.16	3523	46.18
17	Voon	A10	71	118	6	0.6	0.49	16	0.600	46100	0.04	5	16	0.050	46539	23.60	3523	131.46
18	Eikanas ¹	4	52	56	8	0.9	0.31	8	0.510	66100	0.20	5	16	0.182	64100	11.45	1630	59.20
19	Eikanas ¹	7	52	72	8	0.7	0.31	16	0.280	66100	0.20	5	16	0.182	64100	14.75	1630	69.80
20	Matsumura ²	kw4-1	71	63	6	1.1	---	---	0.451	55817	0.11	4	16	0.105	55817	26.26	3161	90.58
21	Matsumura ²	kw3-1	71	47	6	1.5	---	---	0.469	55817	0.11	4	16	0.105	55817	19.66	3161	59.17
22	Matsumura ²	kw3s-1	71	47	6	1.5	---	---	0.469	55817	0.11	4	16	0.105	55817	19.66	3161	65.18
23	Matsumura ²	kw2-1	71	31	6	2.3	---	---	0.611	55817	0.11	4	16	0.105	55817	13.05	3161	39.01
24	Matsumura ²	ws2	71	47	8	1.5	---	---	0.276	55817	0.00	0	0	0	55817	99.59	3233	73.17

Table A-1 Continued

Wall #	Researcher	Wall ID	H (in)	L (in)	t (in)	AR	Vert. Reinf. (in ² /pl)	Vsp (in)	ρ_v (%)	f_{yv} (psi)	Horiz. Reinf. (in ² /pl)	# of locations	Hsp (in)	ρ_h (%)	f_{yh} (psi)	Axial Load (k)	f'm (psi)	Avg. Vmax (k)
25	Matsumura ²	ws4	71	47	8	1.5	---	---	0.276	55817	0.20	4	16	0.151	55817	99.59	3233	100.60
26	Matsumura ²	ws5	71	47	8	1.5	---	---	0.276	55817	0.40	8	16	0.302	55817	99.59	3233	113.05
27	Matsumura ²	ws9	71	47	8	1.5	---	---	0.487	55817	0.40	8	16	0.302	55817	99.59	3233	113.56
28	Matsumura ²	ws10	71	47	8	1.5	---	---	0.487	55817	0.40	8	8	0.604	55817	99.59	3233	143.28
29	Matsumura ²	ws9-2	71	47	8	1.5	---	---	0.487	55817	0.40	8	16	0.302	55817	99.59	4204	123.21
30	Matsumura ²	wsb21	71	47	8	1.5	---	---	0.487	55817	0.40	8	16	0.302	55817	99.59	3784	109.24
31	Matsumura ²	wsb3	71	47	8	1.5	---	---	0.514	55817	0.40	8	16	0.302	55817	94.34	3827	113.60
32	Matsumura ²	wsb4	71	47	8	1.5	---	---	0.487	55817	0.40	8	16	0.302	55817	99.59	4552	131.85
33	Hidalgo	HCBL-12-1	40	80	8	0.5	0.6	40 and 32	0.30	80300	0	0	0	0	0	118.50	3296	189.10
34	Hidalgo	HCBL-12-2	40	80	8	0.5	0.6	40 and 32	0.30	80300	0.31	1	20	0.102	69600	122.00	3296	201.50
35	Hidalgo	HCBL-12-3	40	80	8	0.5	0.6	40 and 32	0.30	80300	0.31	2	13	0.203	69600	148.50	3296	242.50
36	Hidalgo	HCBL-12-6	40	80	8	0.5	0.6	40 and 32	0.30	80300	0.44	4	8	0.577	67300	143.00	3296	252.00
37	Chen	HCBL-11-1	56	48	8	1.2	0	0	0	0	0	0	0	0.00	0	44.00	1330	45.20
38	Chen	HCBL-11-3	56	48	8	1.2	0.31	42	0.170	70800	0	0	0	0.00	0	25.10	1833	46.30
39	Chen	HCBL-11-4	56	48	8	1.2	0.31	42	0.170	70800	0.31	1	28	0.07	47900	39.10	1833	60.30
40	Chen	HCBL-11-6	56	48	8	1.2	0.31	42	0.170	70800	0.31	4	11	0.29	47900	52.70	1833	72.80
41	Chen	HCBL-11-7	56	48	8	1.2	0.78	42	0.430	69200	0	0	0	0.00	0	33.30	1905	53.60
42	Chen	HCBL-11-9	56	48	8	1.2	0.78	42	0.430	69200	0.31	2	19	0.15	47900	41.90	1905	53.60
43	Chen	HCBL-11-11	56	48	8	1.2	0.78	42	0.430	69200	0.44	4	11	0.41	73900	50.80	1330	84.50
44	Sveinsson	HCBL-11-13	56	48	8	1.2	0.31	42	0.170	67500	0.31	4	11	0.290	59000	100.00	3359	103.70
45	Sveinsson	HCBL-11-15	56	48	8	1.2	0.31	42	0.170	67500	0.31	4	11	0.290	59000	160.00	3359	126.10
46	Sveinsson	HCBL-11-17	56	48	6	1.2	0.6	42	0.450	56700	0.31	4	11	0.394	63500	108.00	2297	96.40
47	Sveinsson	HCBL-11-18	56	48	6	1.2	0.2	8	0.440	59500	0.31	4	11	0.394	63500	108.00	2297	96.30

Table A-1 Continued

Wall #	Researcher	Wall ID	H (in)	L (in)	t (in)	AR	Vert. Reinf. (in ² /pl)	Vsp (in)	ρ_v (%)	f_{yv} (psi)	Horiz. Reinf. (in ² /pl)	# of locations	Hsp (in)	ρ_h (%)	f_{yh} (psi)	Axial Load (k)	f'_m (psi)	Avg. Vmax (k)
48	Sveinsson	HCBL-11-20	56	48	6	1.2	0.6	42	0.450	56700	0.31	2	19	0.197	63500	108.00	2196	92.20
49	Sveinsson	HCBL-11-21	56	48	6	1.2	0.2	6 and 12	0.440	59500	0.31	2	19	0.197	63500	108.00	2196	87.50
50	Sveinsson	HCBL-11-23	56	48	6	1.2	0.6	42	0.450	56700	0.055	8	8	0.140	63500	108.00	2196	75.00
51	Sveinsson	HCBL-11-24	56	48	6	1.2	0.6	42	0.450	56700	0.055 and 0.31	8	8 and 19	0.337	63500	108.00	2196	95.20
52	Sveinsson	HCBL-11-25	56	48	6	1.2	0.6	42	0.450	56700	0.31	2	19	0.197	63500	68.00	2196	76.90
53	Sveinsson	HCBL-11-26	56	48	6	1.2	0.6	42	0.450	56700	0.31	2	19	0.197	63500	108.00	2196	94.30

1—heights given are heights to the load; this generally coincides with the height of the wall

2—Matsumura used various steel strengths--this is an average of the range given in Matsumura (1986)

Matsumura's definition of ρ_v = cross-sectional area of a reinforcing bar (one side)/(thickness x effective width of wall)

Number of locations for horizontal steel is estimated based on information presented in Matsumura (1986)

Matsumura's definition of ρ_h = cross-sectional area of a reinforcing bar/(thickness of wall x spacing of bars)

Table A-2—Fully Grouted CMU Wall Analysis

		MSJC									Proposed							
Wall #	Researcher	An	$\frac{M_u}{V_u d_v}$	Masonry Cont.	Axial Load Cont	Steel Cont	Total	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$		Masonry Cont.	# Disc	Steel Cont.	Total w/ Axial Load	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$
1	Shing	405	1	49.91	24.6	9.9	84.42	88.73	84.42	1.21		43.19	2	11.09	81.61	75.58	75.58	1.36
2	Shing	405	1	46.47	0	9.9	56.37	82.6	56.37	1.41		37.43	2	11.09	48.52	65.50	48.52	1.64
3	Shing	405	1	46.47	9.11	9.9	65.48	82.6	65.48	1.32		37.43	2	11.09	58.64	65.50	58.64	1.48
4	Shing	405	1	49.91	9.11	9.9	68.92	88.73	68.92	1.41		43.19	2	11.09	64.40	75.58	64.40	1.51
5	Shing	405	1	49.91	24.6	9.9	84.42	88.73	84.42	1.14		43.19	2	11.09	81.61	75.58	75.58	1.27
6	Shing	405	1	51.55	0	27	78.55	91.64	78.55	1.17		46.07	2	24.12	70.19	80.62	70.19	1.31
7	Shing	405	1	52.35	24.6	27	103.95	93.06	93.06	1.21		47.51	2	24.12	98.97	83.14	83.14	1.35
8	Shing	405	1	52.35	24.6	9.9	86.85	93.06	86.85	1.21		47.51	2	11.09	85.93	83.14	83.14	1.26
9	Shing	405	1	52.35	9.11	27	88.46	93.06	88.46	1.00		47.51	2	24.12	81.75	83.14	81.75	1.08
10	Shing	405	1	45.56	24.6	27	97.17	81	81	1.49		35.99	2	24.12	87.45	62.98	62.98	1.91
11	Voon	399	1	45.31	0	3.88	49.19	80.54	49.19	0.96		36.73	2	3.27	40.00	64.29	40.00	1.18
12	Voon	399	1	45.31	0	0.88	46.18	80.54	46.18	0.91		36.73	1	0.00	36.73	64.29	36.73	1.14
13	Voon	399	1	44.53	0	4.87	49.4	79.16	49.4	0.96		35.48	1	3.07	38.55	62.09	38.55	1.24
14	Voon	399	1	46.83	6.37	3.88	57.08	83.24	57.08	1.03		39.24	2	3.27	49.58	68.67	49.58	1.19
15	Voon	399	1	46.83	3.19	3.88	53.89	83.24	53.89	1.03		39.24	2	3.27	46.05	68.67	46.05	1.21
16	Voon	399	1	53.24	3.19	5.82	62.24	94.64	62.24	0.74		25.36	7	3.35	32.25	44.38	32.25	1.43
17	Voon	664	0.60	116.33	5.31	6.47	128.11	199.8	128.11	1.03		84.53	2	3.27	93.70	120.88	93.70	1.40
18	Eikanas	424	0.94	40.48	2.58	20.86	63.92	71.47	63.92	0.93		46.13	4	7.69	56.68	78.14	56.68	1.04
19	Eikanas	546	0.73	60.18	3.32	26.86	90.36	104.31	90.36	0.77		59.39	4	7.69	70.77	90.63	70.77	0.99
20	Matsumura	370	0.57	62.54	5.91	8.61	77.06	107.18	77.06	1.18		40.19	2	7.37	54.13	70.34	54.13	1.67
21	Matsumura	277	0.76	41.63	4.42	6.44	52.5	72.33	52.5	1.13		30.08	2	7.37	42.36	52.64	42.36	1.40
22	Matsumura	277	0.76	41.63	4.42	6.44	52.5	72.33	52.5	1.24		30.08	2	7.37	42.36	52.64	42.36	1.54
23	Matsumura	184	1.00	23.23	2.94	4.28	30.45	41.3	30.45	1.28		19.97	2	7.37	30.60	34.95	30.60	1.27

Table A-2 Continued

Wall #	Researcher	An	$\frac{M_u}{V_u d_v}$	Masonry Cont.	Axial Load Cont	Steel Cont	Total	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$		Masonry Cont.	# Disc	Steel Cont.	Total w/ Axial Load	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$
24	Matsumura	350	0.76	53.33	22.41	0	75.74	92.66	75.74	0.97		56.54	0	0.00	81.44	98.95	81.44	0.90
25	Matsumura	350	0.76	53.33	22.41	17.57	93.31	92.66	92.66	1.09		56.54	2	13.40	94.84	98.95	94.84	1.06
26	Matsumura	350	0.76	53.33	22.41	35.14	110.88	92.66	92.66	1.22		56.54	4	53.58	135.02	98.95	98.95	1.14
27	Matsumura	350	0.76	53.33	22.41	35.14	110.88	92.66	92.66	1.23		56.54	4	53.58	135.02	98.95	98.95	1.15
28	Matsumura	350	0.76	53.33	22.41	70.28	146.02	92.66	92.66	1.55		56.54	8	107.17	188.61	98.95	98.95	1.45
29	Matsumura	350	0.76	60.82	22.41	35.14	118.37	105.66	105.66	1.17		73.53	4	53.58	152.01	128.68	128.68	0.96
30	Matsumura	350	0.76	57.7	22.41	35.14	115.24	100.24	100.24	1.09		66.18	4	53.58	144.66	115.81	115.81	0.94
31	Matsumura	332	0.76	54.98	21.23	35.14	111.34	95.51	95.51	1.19		66.94	4	53.58	144.11	117.14	117.14	0.97
32	Matsumura	350	0.76	63.29	22.41	35.14	120.83	109.95	109.95	1.20		79.62	4	53.58	158.10	139.33	139.33	0.95
33	Hidalgo	610	0.25	124.76	26.66	0	151.42	210.12	151.42	1.25		174.38	0	0.00	204.01	235.42	204.01	0.93
34	Hidalgo	610	0.25	124.76	27.45	37.2	189.41	210.12	189.41	1.06		174.38	0	12.95	217.83	235.42	217.83	0.93
35	Hidalgo	610	0.25	124.76	33.41	55.81	213.99	210.12	210.12	1.15		174.38	2	0.00	211.51	235.42	211.51	1.15
36	Hidalgo	610	0.25	124.76	32.18	132	288.94	210.12	210.12	1.20		174.38	4	0.00	210.13	235.42	210.13	1.20
37	Chen	366	0.58	39.77	9.9	0	49.67	68.22	49.67	0.91		30.16	0	0.00	41.16	52.78	41.16	1.10
38	Chen	366	0.58	46.68	5.65	0	52.33	80.09	52.33	0.89		41.56	0	0.00	47.84	72.74	47.84	0.97
39	Chen	366	0.58	46.68	8.8	10.63	66.11	80.09	66.11	0.91		41.56	0	8.91	60.25	72.74	60.25	1.00
40	Chen	366	0.58	46.68	11.86	26.57	85.11	80.09	80.09	0.91		41.56	2	17.82	72.56	72.74	72.56	1.00
41	Chen	366	0.58	47.59	7.49	0	55.08	81.65	55.08	0.97		43.20	0	0.00	51.52	75.59	51.52	1.04
42	Chen	366	0.58	47.59	9.43	15.94	72.96	81.65	72.96	0.74		43.20	0	17.82	71.49	75.59	71.49	0.75
43	Chen	366	0.58	39.77	11.43	56.57	107.77	68.22	68.22	1.24		30.16	2	39.02	81.88	52.78	52.78	1.60
44	Sveinsson	366	0.58	63.19	22.5	39.86	125.55	108.41	108.41	0.96		76.15	2	21.95	123.10	133.27	123.10	0.84
45	Sveinsson	366	0.58	63.19	36	39.86	139.05	108.41	108.41	1.16		76.15	2	21.95	138.10	133.27	133.27	0.95
46	Sveinsson	270	0.58	38.55	24.3	39.86	102.7	66.13	66.13	1.46		28.34	2	23.62	78.96	49.59	49.59	1.94
47	Sveinsson	270	0.58	38.55	24.3	39.86	102.7	66.13	66.13	1.46		28.34	2	23.62	78.96	49.59	49.59	1.94
48	Sveinsson	270	0.58	37.69	24.3	23.91	85.9	64.66	64.66	1.43		27.09	0	23.62	77.71	47.41	47.41	1.94
49	Sveinsson	270	0.58	37.69	24.3	23.91	85.9	64.66	64.66	1.35		27.09	0	23.62	77.71	47.41	47.41	1.85

Table A-2 Continued

Wall #	Researcher	An	$\frac{M_u}{V_u d_v}$	Masonry Cont.	Axial Load Cont	Steel Cont	Total	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$		Masonry Cont.	# Disc	Steel Cont.	Total w/ Axial Load	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$
50	Sveinsson	270	0.58	37.69	24.3	9.9	71.89	64.66	64.66	1.16		27.09	4	8.38	62.47	47.41	47.41	1.58
51	Sveinsson	270	0.58	37.69	24.3	33.81	95.8	64.66	64.66	1.47		27.09	4	8.38	62.47	47.41	47.41	2.01
52	Sveinsson	270	0.58	37.69	15.3	23.91	76.9	64.66	64.66	1.19		27.09	0	23.62	67.71	47.41	47.41	1.62
53	Sveinsson	270	0.58	37.69	24.3	23.91	85.9	64.66	64.66	1.46		27.09	0	23.62	77.71	47.41	47.41	1.99

Table A-3—Partially Grouted CMU Wall Properties

Wall #	Researcher	Wall ID	H (in)	L (in)	t (in)	AR	Vert. Reinf. (in ² /pl)	Vsp (in)	ρ_v (%)	f_{yv} (psi)	Horiz. Reinf. (in ² /pl)	# of locations	Hsp (in)	ρ_h (%)	f_{yh} (psi)	Axial Load (k)	Gr. f'm (psi)	Ung. f'm (psi)	Avg. Vmax (k)
1	Nolph	PG085-48	92	104	8	0.88	1.20	48	0.450	63600	0.31	2	48	0.085	63600	11.10	2860	1640	49.84
2	Nolph	PG120-48	92	104	8	0.88	1.20	48	0.450	63600	0.44	2	48	0.120	63600	11.10	2860	1640	51.17
3	Nolph	PG169-48	92	104	8	0.88	1.20	48	0.450	63600	0.62	2	48	0.169	63600	11.10	2860	1640	45.60
4	Nolph	PG085-32	92	104	8	0.88	0.88	32	0.440	63600	0.31	2	48	0.085	63600	11.10	2860	1640	58.43
5	Nolph	PG085-24	92	104	8	0.88	0.88 in ends, 0.61 other	24	0.450	63600	0.31	2	48	0.085	63600	11.10	2860	1640	66.29
6	Voon	A5	71	71	8	1.00	0.44	16	0.400	46100	0	0	0	0.000	0	0.00	---	2682	31.12
7	Voon	A6	71	71	8	1.00	0.44	32	0.240	46100	0	0	0	0.000	0	0.00	---	2682	20.90
8	Yancey	R1	56	48	8	1.17	0	0	0	0	0	0	0	0.000	0	39.30	---	1293	25.65
9	Yancey	R2	56	48	8	1.17	0	0	0	0	0.03	3	16	0.024	80000	39.30	---	1230	32.00
10	Yancey	R4	56	48	8	1.17	0	0	0	0	0.03	7	8	0.0564	80000	39.30	---	1112	32.65
11	Yancey	R5	56	48	8	1.17	0	0	0	0	0.40	1	28	0.094	48750	39.30	---	1217	43.95
12	Yancey	R6	56	48	8	1.17	0	0	0	0	0.92	1	28	0.218	63630	39.30	---	1263	33.90
13	Yancey	R7	56	48	8	1.17	0	0	0	0	0.31	2	12 and 52 from bot.	0.145	55800	39.30	---	1087	36.00
14	Yancey	R8	56	48	8	1.17	0	0	0	0	0.93	2	28	0.218	54180	39.30	---	1242	26.70
15	Yancey	R9	56	48	8	1.17	0	0	0	0	0.22 for 2-#3 and 0.0344 for BJ	1 bond beam, 3 BJ	28 and 16	0.076	40000 and 80000	39.30	---	1095	38.00
16	Yancey	R10	56	48	8	1.17	0	0	0	0	0.71 at mid. and 0.0344 for BJ	2 bond beams, 6 BJ	28 and 8	0.215	49500 and 80000	39.30	---	856	44.15

Table A-3 Continued

Wall #	Researcher	Wall ID	H (in)	L (in)	t (in)	AR	Vert. Reinf. (in ² /pl)	Vsp (in)	ρ_v (%)	f_{yv} (psi)	Horiz. Reinf. (in ² /pl)	# of locations	Hsp (in)	ρ_h (%)	f_{yh} (psi)	Axial Load (k)	Gr. f'm (psi)	Ung. f'm (psi)	Avg. Vmax (k)
17	Yancey	R11	56	48	8	1.17	0	0	0	0	0.31	2	12 and 52 from bot.	0.145	54050	39.30	---	1070	35.90
18	Baenzinger	1PG	104	112	8	0.93	0.88 in ends, 0.4 in center	48	0.250	63190 and 66908	0.39	3	48	0.151	64480	0	---	3080	46.80
19	Baenzinger	2PG	104	112	8	0.93	0.88 in ends, 0.4 in center	48	0.250	63191 and 66908	0.06	12	8	0.184	87950	0	---	2460	45.20
20	Baenzinger	3PG	104	168	8	0.62	0.88 in ends, 0.4 other	48	0.190	63192 and 66908	0.39	3	48	0.151	64480	0	---	2460	78.30
21	Baenzinger	4PG	104	168	8	0.62	0.88 in ends, 0.2 adjacent to end, 0.4 other	48	0.230	63193 and 66908	0.06	12	8	0.184	87950	0	---	2970	96.90
22	Baenzinger	5PG	104	112	8	0.93	0.4 in ends, 0.2 adjacent to end, 0.4 other	48	0.190	63194 and 66908	0.06	12	8	0.184	87950	0	---	2970	59.80
23	Baenzinger	6PG	104	112	8	0.93	0.88 in ends, 0.2 adjacent to end, 0.4 other	48	0.300	63195 and 66908	0.11	12	8	0.267	87950	0	---	2970	73.20
24	Baenzinger	7PG	104	112	8	0.93	0.4 in ends, 0.2 adjacent to end, 0.4 other	48	0.190	63196 and 66908	0.39	3	48	0.151	64480	0	---	3570	56.00
25	Baenzinger	8PG	104	168	8	0.62	0.88 in ends, 0.2 adjacent to end, 0.4 other	48	0.230	63197 and 66908	0.11	12	8	0.267	87950	0	---	2980	91.20
26	Schultz ¹	2	56	112	8	0.50	0.88	104	0.210	60000	0.03	7	8	0.056	80000	59.60	2390	2100	58.75
27	Schultz ¹	4	56	80	8	0.70	0.88	72	0.290	60000	0.03	7	8	0.056	80000	41.70	2390	2100	56.99
28	Schultz ¹	6	56	56	8	1.00	0.88	48	0.410	60000	0.03	7	8	0.056	80000	29.30	2390	2100	39.54
29	Schultz ¹	8	56	112	8	0.50	0.88	104	0.210	60000	0.07	7	8	0.110	80000	59.70	2390	2100	54.71

Table A-3 Continued

Wall #	Researcher	Wall ID	H (in)	L (in)	t (in)	AR	Vert. Reinf. (in ² /pl)	Vsp (in)	ρ_v (%)	f_{yv} (psi)	Horiz. Reinf. (in ² /pl)	# of locations	Hsp (in)	ρ_h (%)	f_{yh} (psi)	Axial Load (k)	Gr. f'm (psi)	Ung. f'm (psi)	Avg. Vmax (k)
30	Schultz ¹	10	56	80	8	0.70	0.88	72	0.290	60000	0.07	7	8	0.110	80000	42.20	2390	2100	60.78
31	Schultz ¹	12	56	56	8	1.00	0.88	48	0.410	60000	0.07	7	8	0.110	80000	30.00	2390	2100	47.51
32	Matsumura ²	CW4-1-1	71	68	6	1.05	---	---	0.312	55817	0.20	2	48	0.096	55817	0.00	---	2378	25.51
33	Matsumura ²	CW4-1-2	71	68	6	1.05	---	---	0.385	55817	0.20	2	48	0.096	55817	0.00	---	3914	35.37
34	Matsumura ²	CW3-1-1	71	52	6	1.36	---	---	0.411	55817	0.20	2	48	0.096	55817	0.00	---	2378	20.25
35	Matsumura ²	CW3-1-2	71	52	6	1.36	---	---	0.375	55817	0.20	2	48	0.096	55817	0.00	---	3914	26.47
36	Matsumura ²	CW2-1-1	71	36	6	1.96	---	---	0.604	55817	0.20	2	48	0.096	55817	0.00	---	2378	17.68
37	Matsumura ²	CW2-1-2	71	36	6	1.96	---	---	0.395	55817	0.20	2	48	0.096	55817	0.00	---	3914	16.44
38	Matsumura ²	CW3-0-1	71	52	6	1.36	---	---	0.411	55817	0.00	0	0	0.000	0	0.00	---	2378	13.13
39	Matsumura ²	CW3-0-2	71	52	6	1.36	---	---	0.411	55817	0.00	0	0	0.000	0	0.00	---	2378	16.24
40	Matsumura ²	CW3-1'	71	52	6	1.36	---	---	0.411	55817	0.20	2	48	0.096	55817	0.00	---	2378	18.47
41	Matsumura ²	CW3-2	71	52	6	1.36	---	---	0.411	55817	0.20	3	24	0.143	55817	0.00	---	2378	20.02
42	Matsumura ²	CW3-3	71	52	6	1.36	---	---	0.411	55817	0.20	4	16	0.191	55817	0.00	---	2378	24.92
43	Matsumura ²	CW3-1-A2	71	52	6	1.36	---	---	0.411	55817	0.20	2	48	0.096	55817	22.25	---	3914	30.70
44	Matsumura ²	CW3-1-A3	71	52	6	1.36	---	---	0.411	55817	0.20	2	48	0.096	55817	44.49	---	3914	32.04
45	Matsumura ²	CW3-1-A4	71	52	6	1.36	---	---	0.411	55817	0.20	2	48	0.096	55817	66.74	---	3914	40.27
46	Matsumura ²	CW3-0-A2	71	54	6	1.31	---	---	0.399	55817	0.00	0	0	0.000	0	23.09	---	2030	21.70
47	Matsumura ²	CW3-2-A2	71	54	6	1.31	---	---	0.399	55817	0.20	3	24	0.143	55817	23.09	---	2030	34.17
48	Matsumura ²	CW3-3-A2	71	54	6	1.31	---	---	0.399	55817	0.20	4	16	0.191	55817	23.09	---	2030	38.79
49	Matsumura ²	CW3-4-A2	71	54	6	1.31	---	---	0.399	55817	0.31	4	16	0.296	55817	23.09	---	2030	43.87
50	Matsumura ²	CWB3-1'-A2	71	54	6	1.31	---	---	0.148	55817	0.20	2	48	0.096	55817	23.09	---	2030	31.17
51	Matsumura ²	CW3-0-A3	71	52	6	1.36	---	---	0.411	55817	0.00	0	0	0.000	0	44.49	---	3914	35.60
52	Matsumura ²	CW3-0'-A3	71	52	6	1.36	---	---	0.411	55817	0.00	0	0	0.000	0	44.49	---	2030	19.58

Table A-3 Continued

Wall #	Researcher	Wall ID	H (in)	L (in)	t (in)	AR	Vert. Reinf. (in ² /pl)	Vsp (in)	ρ_v (%)	f_{yv} (psi)	Horiz. Reinf. (in ² /pl)	# of locations	Hsp (in)	ρ_h (%)	f_{yh} (psi)	Axial Load (k)	Gr. f'm (psi)	Ung. f'm (psi)	Avg. Vmax (k)
53	Matsumura ²	CW3-2-A3	71	52	6	1.36	---	---	0.411	55817	0.20	3	24	0.143	55817	44.49	---	3914	37.60
54	Matsumura ²	CW3-3-A3	71	52	6	1.36	---	---	0.411	55817	0.20	4	16	0.191	55817	44.49	---	3914	41.38
55	Matsumura ²	CW5-2'-A2-1	71	78	6	0.91	---	---	0.274	55817	0.20	3	24	0.143	55817	33.20	---	2204	52.13
56	Matsumura ²	CW5-2'-A2-2	71	78	6	0.91	---	---	0.274	55817	0.20	3	24	0.143	55817	33.20	---	2204	50.47
57	Matsumura ²	CW4-2'-A2	71	70	6	1.02	---	---	0.255	55817	0.20	3	24	0.143	55817	29.83	---	2204	44.15
58	Matsumura ²	CW3-2'-A2	71	54	6	1.31	---	---	0.264	55817	0.20	3	24	0.143	55817	23.09	---	2204	32.79
59	Matsumura ²	CW2-2'-A2-1	71	38	6	1.86	---	---	0.293	55817	0.20	3	24	0.143	55817	16.35	---	2204	17.66
60	Matsumura ²	CW2-2'-A2-2	71	39	6	1.81	---	---	0.293	55817	0.20	3	24	0.143	55817	16.77	---	2204	18.28
61	Minaie ³	PCL-1	104	152	8	0.68	0.44	48	0.150	60000	0.44	3	48	0.167	60000	54.40	---	2000	71.50
62	Minaie ³	MC-1	104	152	8	0.68	0.44	48	0.150	60000	0.44	3	48	0.167	60000	54.40	---	2000	42.60
63	Minaie ³	PCL-2	104	152	8	0.68	0.44	48	0.150	60000	0.44	3	48	0.167	60000	0.00	---	2000	54.10
64	Minaie ³	MC-2	104	152	8	0.68	0.44	48	0.150	60000	0.44	3	48	0.167	60000	0.00	---	2000	51.80
65	Chen	HCBL-11-2	56	48	8	1.2	0	0	0	0	0	0	0	0	0	42.20	1710	---	25.20
66	Chen	HCBL-11-5	56	48	8	1.2	0.31	42	0.17	70800	0.31	1	28	0.073	47900	30.20	1710	---	46.80
67	Chen	HCBL-11-8	56	48	8	1.2	0.78	42	0.43	69200	0	0	0	0	0	29.20	1710	---	36.80
68	Chen	HCBL-11-10	56	48	8	1.2	0.78	42	0.43	69200	0.31	2	19	0.145	47900	31.20	1710		48.70

1—Yield strength of horizontal reinforcement for Schultz is estimated based upon other similar data

2—Matsumura designates $f'm$ as a “net” value here. Matsumura used various steel strengths--this is an average of the range given in Matsumura (1986)

Definition of ρ_v = cross-sectional area of a reinforcing bar (one side)/(thickness x effective width of wall)

Number of locations for horizontal steel is estimated based on information presented in Matsumura (1986). Definition of ρ_h = cross-sectional area of a reinforcing bar/(thickness of wall x spacing of bars)

3—Vertical steel strength is assumed

Table A-4—Partially Grouted CMU Wall Analysis

		TMS-402									Proposed							
Wall #	Researcher	An	$\frac{M_u}{V_u d_v}$	Masonry Cont.	Axial Load	Steel Cont.	Total	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$		Masonry Cont.	# Disc.	Steel Cont.	Total w/ Axial Load	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$
1	Nolph	383	0.89	38.03	2.5	20.15	60.68	66.81	60.68	0.82		24.74	1	7.89	35.40	41.01	35.40	1.41
2	Nolph	383	0.89	38.03	2.5	28.6	69.13	66.81	66.81	0.77		24.74	1	11.19	38.71	41.01	38.71	1.32
3	Nolph	383	0.89	38.03	2.5	40.3	80.83	66.81	66.81	0.68		24.74	1	15.77	43.29	41.01	41.01	1.11
4	Nolph	424	0.89	42.1	2.5	20.15	64.75	73.97	64.75	0.90		24.74	1	7.89	35.40	41.01	35.40	1.65
5	Nolph	465	0.89	46.17	2.5	20.15	68.82	81.12	68.82	0.96		24.74	1	7.89	35.40	41.01	35.40	1.87
6	Voon	287	1.00	33.41	0	0	33.41	59.4	33.41	0.93		21.12	0	0.00	21.12	36.96	21.12	1.47
7	Voon	229	1.00	26.65	0	0	26.65	47.38	26.65	0.78		21.12	0	0.00	21.12	36.96	21.12	0.99
8	Yancey	120	0.58	12.86	8.84	0	21.7	22.05	21.7	1.18		14.79	0	0.00	24.61	25.88	24.61	1.04
9	Yancey	120	0.58	12.54	8.84	3.1	24.48	21.51	21.51	1.49		14.07	2	1.10	24.99	24.62	24.62	1.30
10	Yancey	120	0.58	11.92	8.84	6.19	26.96	20.45	20.45	1.60		12.72	4	3.30	25.85	22.26	22.26	1.47
11	Yancey	120	0.58	12.47	8.84	13.71	35.03	21.4	21.4	2.05		13.92	0	15.60	39.34	24.36	24.36	1.80
12	Yancey	120	0.58	12.71	8.84	47.31	68.86	21.8	21.8	1.56		14.45	0	70.25	94.52	25.28	25.28	1.34
13	Yancey	120	0.58	11.79	8.84	7.44	28.07	20.22	20.22	1.78		12.43	2	0.00	22.26	21.76	21.76	1.65
14	Yancey	120	0.58	12.6	8.84	31.89	53.33	21.62	21.62	1.24		14.21	0	60.46	84.50	24.86	24.86	1.07
15	Yancey	120	0.58	11.83	8.84	10.64	31.31	20.3	20.3	1.87		12.52	2 BJ	4.62	26.97	21.92	21.92	1.73
16	Yancey	120	0.58	10.46	8.84	30.54	49.84	17.95	17.95	2.46		9.79	4 BJ	17.36	36.98	17.13	17.13	2.58
17	Yancey	120	0.58	11.69	8.84	7.44	27.98	20.06	20.06	1.79		12.24	2	0.00	22.06	21.42	21.42	1.68
18	Baenzinger	444	0.93	58.38	0	27.49	27.49	103.03	27.49	1.70		44.26	2	10.13	54.39	75.05	54.39	0.86
19	Baenzinger	444	0.93	52.17	0	36.85	36.85	92.08	36.85	1.23		35.35	2 BB, 4 BJ	15.48	50.83	59.94	50.83	0.89
20	Baenzinger	666	0.62	96.29	0	41.23	41.23	165.6	41.23	1.90		53.03	2	10.13	63.15	76.68	63.15	1.24
21	Baenzinger	748	0.62	118.83	0	55.27	55.27	204.37	55.27	1.75		64.02	2 BB, 4 BJ	15.48	79.50	92.57	79.50	1.22
22	Baenzinger	526	0.93	67.91	0	36.85	36.85	119.86	36.85	1.62		42.68	2 BB, 4 BJ	15.48	58.16	72.37	58.16	1.03

Table A-4 Continued

Wall #	Researcher	An	$\frac{M_u}{V_u d_v}$	Masonry Cont.	Axial Load	Steel Cont.	Total	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$		Masonry Cont.	# Disc.	Steel Cont.	Total w/ Axial Load	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$
23	Baenzinger	526	0.93	67.91	0	59.95	59.95	119.86	59.95	1.22		42.68	2 BB, 4 BJ	30.96	73.64	72.37	72.37	1.01
24	Baenzinger	526	0.93	74.45	0	27.49	27.49	131.41	27.49	2.04		51.30	2	10.13	61.43	86.99	61.43	0.91
25	Baenzinger	748	0.62	119.03	0	89.92	89.92	204.71	89.92	1.01		64.23	2 BB, 4 BJ	30.96	95.19	92.88	92.88	0.98
26	Schultz	362	0.25	59.1	13.41	14.45	86.96	99.53	86.96	0.68		56.04	4	3.30	74.25	75.66	74.25	0.79
27	Schultz	282	0.35	43.78	9.38	10.32	63.48	74.09	63.48	0.90		40.03	4	3.30	53.76	60.45	53.76	1.06
28	Schultz	222	0.50	31.79	6.59	7.22	45.61	54.26	45.61	0.87		28.02	4	3.30	38.65	49.04	38.65	1.02
29	Schultz	362	0.25	59.1	13.43	28.27	100.8	99.53	99.53	0.55		56.04	4	6.46	77.43	75.66	75.66	0.72
30	Schultz	282	0.35	43.78	9.5	20.19	73.46	74.09	73.46	0.83		40.03	4	6.46	57.04	60.45	57.04	1.07
31	Schultz	222	0.50	31.79	6.75	14.13	52.68	54.26	52.68	0.90		28.02	4	6.46	41.98	49.04	41.98	1.13
32	Matsumura	279	0.52	42.01	0	5.64	47.65	71.79	47.65	0.54		18.78	0	8.93	27.71	32.87	27.71	0.92
33	Matsumura	280	0.52	54.11	0	5.64	59.75	92.47	59.75	0.59		30.93	0	8.93	39.86	54.12	39.86	0.89
34	Matsumura	216	0.68	29.63	0	4.33	33.96	51.18	33.96	0.60		14.42	0	8.93	23.35	25.23	23.35	0.87
35	Matsumura	216	0.68	37.95	0	4.33	42.28	65.55	42.28	0.63		23.73	0	8.93	32.66	41.53	32.66	0.81
36	Matsumura	153	0.98	17.11	0	3.02	20.13	30.35	20.13	0.88		10.05	0	8.93	18.98	17.58	17.58	1.01
37	Matsumura	153	0.98	21.93	0	3.02	24.95	38.9	24.95	0.66		16.54	0	8.93	25.47	28.95	25.47	0.65
38	Matsumura	216	0.68	29.5	0	0	29.5	50.96	29.5	0.45		14.42	0	0.00	14.42	25.23	14.42	0.91
39	Matsumura	215	0.68	29.48	0	0	29.48	50.93	29.48	0.55		14.42	0	0.00	14.42	25.23	14.42	1.13
40	Matsumura	218	0.68	29.8	0	4.33	34.13	51.47	34.13	0.54		14.42	0	8.93	23.35	25.23	23.35	0.79
41	Matsumura	217	0.68	29.77	0	8.66	38.43	51.42	38.43	0.52		14.42	0	13.40	27.81	25.23	25.23	0.79
42	Matsumura	216	0.68	29.59	0	12.99	42.58	51.11	42.58	0.59		14.42	2	8.93	23.35	25.23	23.35	1.07
43	Matsumura	217	0.68	38.14	5.01	4.33	47.48	65.89	47.48	0.65		23.73	0	8.93	38.23	41.53	38.23	0.80
44	Matsumura	217	0.68	38.04	10.01	43.31	52.39	65.72	52.39	0.61		23.73	0	8.93	43.79	41.53	41.53	0.77
45	Matsumura	217	0.68	38.11	15.02	4.33	57.45	65.82	57.45	0.70		23.73	0	8.93	49.35	41.53	41.53	0.97
46	Matsumura	229	0.66	29.35	5.2	0	34.55	50.61	34.55	0.63		12.77	0	0.00	18.54	22.35	18.54	1.17
47	Matsumura	228	0.66	29.25	5.2	8.99	43.43	50.43	43.43	0.79		12.77	0	13.40	31.94	22.35	22.35	1.53
48	Matsumura	228	0.66	29.24	5.2	13.48	47.92	50.42	47.92	0.81		12.77	2	8.93	27.48	22.35	22.35	1.74
49	Matsumura	228	0.66	29.33	5.2	20.9	55.42	50.57	50.57	0.87		12.77	2	13.84	32.39	22.35	22.35	1.96

Table A-4 Continued

Wall #	Researcher	An	$\frac{M_u}{V_u d_v}$	Masonry Cont.	Axial Load	Steel Cont.	Total	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$		Masonry Cont.	# Disc.	Steel Cont.	Total w/ Axial Load	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$
50	Matsumura	229	0.66	29.37	5.2	4.5	39.06	50.65	39.06	0.80		12.77	0	8.93	27.48	22.35	22.35	1.39
51	Matsumura	216	0.68	37.99	10.01	0	48	65.62	48	0.74		23.73	0	0.00	34.86	41.53	34.86	1.02
52	Matsumura	216	0.68	27.32	10.01	0	37.33	47.2	37.33	0.52		12.31	0	0.00	23.43	21.54	21.54	0.91
53	Matsumura	217	0.68	38.11	10.01	8.66	56.78	65.83	56.78	0.66		23.73	0	13.40	48.25	41.53	41.53	0.91
54	Matsumura	217	0.68	38.12	10.01	12.99	61.12	65.84	61.12	0.68		23.73	2	8.93	43.79	41.53	41.53	1.00
55	Matsumura	328	0.46	49.33	7.47	12.93	69.73	83.98	69.73	0.75		19.94	0	13.40	41.64	33.52	33.52	1.56
56	Matsumura	328	0.46	49.34	7.47	12.93	69.74	83.99	69.74	0.72		19.94	0	13.40	41.64	33.52	33.52	1.51
57	Matsumura	321	0.51	46.8	6.71	11.61	65.13	79.92	65.13	0.68		17.92	0	13.40	38.77	31.35	31.35	1.41
58	Matsumura	228	0.66	30.57	5.2	8.99	44.75	52.71	44.75	0.73		13.87	0	13.40	33.04	24.27	24.27	1.35
59	Matsumura	166	0.93	18.48	3.68	6.37	28.53	32.61	28.53	0.62		9.82	0	13.40	27.30	17.18	17.18	1.03
60	Matsumura	169	0.91	19.2	3.77	6.53	29.5	33.8	29.5	0.62		10.07	0	13.40	27.66	17.62	17.62	1.04
61	Minaie	544	0.68	68.18	12.24	41.97	122.4	117.8	117.8	0.61		39.00	2	10.60 32	63.21	58.40	58.404 57	1.22
62	Minaie	544	0.68	68.18	12.24	41.97	122.4	117.8	117.8	0.36		39.00	2	10.60	63.21	58.40	58.40	0.73
63	Minaie	544	0.34	82.75	0	41.97	124.72	140	124.72	0.43		39.00	2	10.60	49.61	58.40	49.61	1.09
64	Minaie	544	0.34	82.75	0	41.97	124.72	140	124.72	0.42		39.00	2	10.60	49.61	58.40	49.61	1.04
65	Chen	120	0.58	14.78	9.5	0	24.28	25.36	24.28	1.04		19.56	0	0.00	30.11	34.23	30.11	0.84
66	Chen	202	0.58	24.89	6.8	10.63	42.31	42.69	42.31	1.11		19.56	0	5.94	33.05	34.23	33.05	1.42
67	Chen	202	0.58	24.89	6.57	0	31.46	42.69	31.46	1.17		19.56	0	0.00	26.86	34.23	26.86	1.37
68	Chen	202	0.58	24.89	7.02	15.94	47.85	42.69	42.69	1.14		19.56	0	11.88	39.24	34.23	34.23	1.42

Table A-5—Fully Grouted Clay Wall Properties

Wall #	Researcher	Wall ID	H (in)	L (in)	t (in)	AR	Vert. Reinf. (in ² /pl)	Vsp (in)	ρ_v (%)	f_{yv} (psi)	Horiz. Reinf. (in ² /pl)	# of locations	Hsp (in)	ρ_h (%)	f_{yh} (psi)	Axial Load (k)	f'm (psi)	Avg. Vmax (k)
1	Shing	21	72	72	6	1	0.44	16	0.560	65000	0.11	5	16	0.140	56000	113.40	3800	105.50
2	Shing	22	72	72	6	1	0.44	16	0.560	65000	0.11	5	16	0.140	56000	40.50	3800	91.50
3	Sveinsson	HCBR-11-19	56	48	6	1.2	0.6	42	0.45	56700	0.31	2	19	0.197	63500	108.00	2918	72.20
4	Sveinsson	HCBR-11-21	56	48	6	1.2	0.31	12 and 18	0.45	56700	0.31	2	19	0.197	63500	108.00	2918	92.20
5	Sveinsson	HCBR-11-23	56	48	6	1.2	0.2	6 and 12	0.44	59500	0.31	2	19	0.197	63500	108.00	2918	79.60
6	Sveinsson	HCBR-11-25	56	48	6	1.2	0.6	42	0.45	56700	0.31	2	19	0.197	63500	108.00	2918	85.40
7	Sveinsson	HCBR-11-27	56	48	6	1.2	0.6	42	0.45	56700	0.4	2	19	0.254	59500	108.00	2918	88.40
8	Sveinsson	HCBR-11-30	56	48	6	1.2	0.6	42	0.45	56700	0.055	8	8	0.140	63500	108.00	4007.5	105.50
9	Sveinsson	CRBC-11-8s	56	48	10	1.2	0.31	42	0.013	67500	0.31	1	28	0.055	59000	106.00	2483	103.80
10	Sveinsson	CRBC-11-9	56	48	10	1.2	0.31	42	0.013	67500	0.31	5	9	0.277	67500	160.00	2483	113.60
11	Sveinsson	CRBC-11-10	56	48	10	1.2	0.31	42	0.013	67500	0.31	1	28	0.055	59000	160.00	2483	120.40
12	Sveinsson	CRBC-11-12	56	48	10	1.2	0.31	42	0.013	67500	0.11	3	14	0.059	57800	160.00	2483	131.20
13	Chen	HCBR-11-1	56	48	8	1.2	0	0	0	0	0	0	0	0.000	0	116.10	2535	90.10
14	Chen	HCBR-11-3	56	48	8	1.2	0.31	42	0.18	75000	0	0	0	0.000	0	52.30	2535	94.40
15	Chen	HCBR-11-4	56	48	8	1.2	0.31	42	0.18	71340	0.31	1	28	0.075	70000	114.30	2722	119.30
16	Chen	HCBR-11-6	56	48	8	1.2	0.31	42	0.18	71340	0.31	5	9	0.375	64200	61.90	2722	116.20
17	Chen	HCBR-11-7	56	48	8	1.2	0.31	42	0.18	75000	0.31	5	9	0.375	72600	85.30	2535	94.60
18	Chen	HCBR-11-8	56	48	8	1.2	0.785	42	0.45	69200	0	0	0	0.000	0	43.40	2866	80.40
19	Chen	HCBR-11-10	56	48	8	1.2	0.785	42	0.45	72870	0.31	2	19	0.150	68700	54.20	2722	101.60
20	Chen	HCBR-11-12	56	48	8	1.2	0.785	42	0.45	76000	0.44	5	9	0.533	73900	85.00	2535	94.30

Table A-5 Continued

Wall #	Researcher	Wall ID	H (in)	L (in)	t (in)	AR	Vert. Reinf. (in ² /pl)	Vsp (in)	ρ_v (%)	f_{yv} (psi)	Horiz. Reinf. (in ² /pl)	# of locations	Hsp (in)	ρ_h (%)	f_{yh} (psi)	Axial Load (k)	f'_m (psi)	Avg. Vmax (k)
21	Chen	HCBR-11-13	56	48	8	1.2	0.785	42	0.45	72870	0.44	5	9	0.533	74700	110.60	2722	113.30
22	Chen	CBRC-11-1	56	48	10	1.2	0	0	0	0	0	0	0	0.000	0	141.90	2507	114.90
23	Chen	CBRC-11-2	56	48	10	1.2	0.31	42	0.13	71720	0	0	0	0.000	0	92.70	2507	106.00
24	Chen	CBRC-11-3	56	48	10	1.2	0.31	42	0.13	71720	0.31	1	28	0.055	68300	89.50	2507	106.70
25	Chen	CBRC-11-4	56	48	10	1.2	0.31	42	0.13	71720	0.31	5	9	0.277	68300	132.50	2507	124.40
26	Chen	CBRC-11-5	56	48	10	1.2	0.785	42	0.33	72870	0	0	0	0.000	0	76.40	2507	102.00
27	Chen	CBRC-11-6	56	48	10	1.2	0.785	42	0.33	72870	0.31	2	19	0.111	73900	100.30	2507	128.30
28	Chen	CBRC-11-7	56	48	10	1.2	0.785	42	0.33	72870	0.44	5	9	0.393	74700	80.90	2507	115.70
29	Hidalgo	HCBR-12-2	40	78	8	0.5	0.6	36	0.31	80300	0.44	1	20	0.149	67300	86.00	3214	182.70
30	Hidalgo	HCBR-12-3	40	78	8	0.5	0.6	36	0.31	80300	0.44	2	13	0.298	67300	114.10	3214	211.80
31	Hidalgo	HCBR-12-4	40	78	8	0.5	0.6	36	0.31	80300	0.44	3	10	0.447	67300	142.40	3214	245.80
32	Hidalgo	CBRC-12-6	40	78	10	0.5	0.6	36	0.23	80300	0.6	5	7	0.750	80300	110.70	2912	207.30
33	Hidalgo	HCBR-21-1	80	42	8	1.90	0	0	0	0	0	0	0	0.000	0	179.50	4654	75.40
34	Hidalgo	HCBR-21-2	80	42	8	1.90	0.785	36	0.51	47300	0	0	0	0.000	0	113.90	4654	63.70
35	Hidalgo	HCBR-21-4	80	42	8	1.90	0.785	36	0.51	47300	0.31	2	27	0.105	49700	128.60	4654	84.60
36	Hidalgo	HCBR-21-6	80	42	8	1.90	0.785	36	0.51	47300	0.31	3	20	0.158	49700	152.40	4654	98.20
37	Hidalgo	HCBR-21-8	80	42	8	1.90	0.785	36	0.51	47300	0.31	4	16	0.210	49700	150.20	4654	99.30
38	Hidalgo	HCBR-21-9	80	42	8	1.90	0.785	36	0.51	47300	0.31	5	13	0.263	49700	147.50	4654	95.10
39	Hidalgo	CBRC-21-2	80	42	10	1.90	0.785	36	0.38	47300	0	0	0	0.000	0	200.50	3350	114.20
40	Hidalgo	CBRC-21-3	80	42	10	1.90	0.785	36	0.38	47300	0.31	2	27	0.078	49700	192.50	3350	106.00
41	Hidalgo	CBRC-21-4	80	42	10	1.90	0.785	36	0.38	47300	0.31	3	20	0.116	49700	175.20	3350	104.20
42	Hidalgo	CBRC-21-5	80	42	10	1.90	0.785	36	0.38	47300	0.31	5	13	0.194	49700	158.50	3350	105.00

Table A-6—Fully Grouted Clay Wall Analysis

		TMS-402									Proposed							
Wall #	Researcher	An	$\frac{M_u}{V_u d_v}$	Masonry Cont.	Axial Load	Steel Cont.	Total	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$		Masonr y Cont.	# Undev.	Steel Cont.	Total w/ Axial Load	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$
1	Shing	387	1.00	53.68	25.52	9.9	89.09	95.43	89.09	1.18		49.95	2	11.09	89.39	87.42	87.42	1.21
2	Shing	387	1.00	53.68	9.11	9.9	72.69	95.43	72.69	1.26		49.95	2	11.09	71.16	87.42	71.16	1.29
3	Sveinsson	270	0.58	43.45	24.3	23.91	91.66	74.55	74.55	0.97		36.01	0	23.62	86.63	63.01	63.01	1.15
4	Sveinsson	270	0.58	43.45	24.3	23.91	91.66	74.55	74.55	1.24		36.01	0	23.62	86.63	63.01	63.01	1.46
5	Sveinsson	270	0.58	43.45	24.3	23.91	91.66	74.55	74.55	1.07		36.01	0	23.62	86.63	63.01	63.01	1.26
6	Sveinsson	270	0.58	43.45	24.3	23.91	91.66	74.55	74.55	1.15		36.01	0	23.62	86.63	63.01	63.01	1.36
7	Sveinsson	270	0.58	43.45	24.3	30.85	98.6	74.55	74.55	1.19		36.01	0	28.56	91.57	63.01	63.01	1.40
8	Sveinsson	270	0.58	50.92	24.3	9.9	85.12	87.36	85.12	1.24		49.45	4	8.38	84.83	86.54	84.83	1.24
9	Sveinsson	480	0.58	71.26	23.85	15.94	111.0 5	122.2 5	111.0 5	0.94		59.59	0	10.97	97.07	104.29	97.07	1.07
10	Sveinsson	480	0.58	71.26	36	47.85	155.1	122.2 5	122.2 5	0.93		59.59	2	37.67	137.26	104.29	104.29	1.09
11	Sveinsson	480	0.58	71.26	36	15.94	123.2	122.2 5	122.2 5	0.99		59.59	0	10.97	110.57	104.29	104.29	1.15
12	Sveinsson	480	0.58	71.26	36	11.31	118.5 7	122.2 5	118.5 7	1.11		59.59	2	3.81	103.41	104.29	103.41	1.27
13	Chen	354	0.58	53.1	26.12	0	79.22	91.1	79.22	1.14		53.77	0	0.00	82.80	94.10	82.80	1.09
14	Chen	354	0.58	53.1	11.77	0	64.87	91.1	64.87	1.46		53.77	0	0.00	66.85	94.10	66.85	1.41
15	Chen	354	0.58	55.02	25.72	15.94	96.68	94.4	94.4	1.26		57.74	0	13.02	99.33	101.04	99.33	1.20
16	Chen	354	0.58	55.02	13.93	47.85	116.8	94.4	94.4	1.23		57.74	2	35.82	109.04	101.04	101.04	1.15
17	Chen	354	0.58	53.1	19.19	47.85	120.1 4	91.1	91.1	1.04		53.77	2	40.51	115.61	94.10	94.10	1.01
18	Chen	354	0.58	56.46	9.77	0	66.22	96.86	66.22	1.21		60.79	0	0.00	71.64	106.39	71.64	1.12
19	Chen	354	0.58	55.02	12.2	23.91	91.13	94.4	91.13	1.12		57.74	0	25.56	96.85	101.04	96.85	1.05
20	Chen	354	0.58	53.1	19.13	67.91	140.1 3	91.1	91.1	1.04		53.77	2	58.53	133.55	94.10	94.10	1.00
21	Chen	354	0.58	55.02	24.89	67.91	147.8 2	94.4	94.4	1.20		57.74	2	59.16	144.55	101.04	101.04	1.12
22	Chen	480	0.58	71.6	31.93	0	103.5 3	122.8 4	103.5 3	1.11		60.17	0	0.00	95.64	105.29	95.64	1.20

Table A-6 Continued

Wall #	Researcher	An	$\frac{M_u}{V_u d_v}$	Masonr y Cont.	Axial Load	Steel Cont.	Total	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$		Masonr y Cont.	# Undev.	Steel Cont.	Total w/ Axial Load	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$
23	Chen	480	0.58	71.6	20.86	0	92.46	122.8 4	92.46	1.15		60.17	0	0.00	83.34	105.29	83.34	1.27
24	Chen	480	0.58	71.6	20.14	15.94	107.6 8	122.8 4	107.6 8	0.99		60.17	0	12.70	95.25	105.29	95.25	1.12
25	Chen	480	0.58	71.6	29.81	47.85	149.2 6	122.8 4	122.8 4	1.01		60.17	2	38.11	131.40	105.29	105.29	1.18
26	Chen	480	0.58	71.6	17.19	0	88.79	122.8 4	88.79	1.15		60.17	0	0.00	79.27	105.29	79.27	1.29
27	Chen	480	0.58	71.6	22.57	23.91	118.0 8	122.8 4	118.0 8	1.09		60.17	0	27.49	112.73	105.29	105.29	1.22
28	Chen	480	0.58	71.6	18.2	67.91	157.7 1	122.8 4	122.8 4	0.94		60.17	2	59.16	139.56	105.29	105.29	1.10
29	Hidalgo	575	0.26	109.96	19.35	51.48	180.7 9	186.2	180.7 9	1.01		155.10	0	17.77	194.37	210.98	194.37	0.94
30	Hidalgo	575	0.26	115.82	25.67	77.24	218.7 3	195.1 2	195.1 2	1.09		155.10	2	0.00	183.63	210.98	183.63	1.15
31	Hidalgo	575	0.26	115.82	32.04	102.9 6	250.8 2	195.1 2	195.1 2	1.26		155.10	2	17.77	208.47	210.98	208.47	1.18
32	Hidalgo	780	0.26	149.48	24.91	210.5	384.8 8	251.8 3	251.8 3	0.82		159.00	4	28.91	215.58	216.27	215.58	0.96
33	Hidalgo	310	0.95	49.31	40.39	0	89.69	87.21	87.21	0.87		60.47	0	0.00	105.34	105.82	105.34	0.72
34	Hidalgo	310	0.95	49.31	25.63	0	74.93	87.21	74.93	0.85		60.47	0	0.00	88.94	105.82	88.94	0.72
35	Hidalgo	310	0.95	49.31	28.94	9.76	88.01	87.21	87.21	0.97		60.47	0	18.49	111.11	105.82	105.82	0.80
36	Hidalgo	310	0.95	49.31	34.29	13.02	96.62	87.21	87.21	1.13		60.47	0	27.73	126.30	105.82	105.82	0.93
37	Hidalgo	310	0.95	49.31	33.8	16.28	99.38	87.21	87.21	1.14		60.47	2	18.49	116.51	105.82	105.82	0.94
38	Hidalgo	310	0.95	49.31	33.19	19.54	102.0 3	87.21	87.21	1.09		60.47	2	27.73	125.07	105.82	105.82	0.90
39	Hidalgo	420	0.95	56.72	45.11	0	101.8 3	100.3 2	100.3 2	1.14		49.25	0	0.00	99.37	86.18	86.18	1.33
40	Hidalgo	420	0.95	56.72	43.31	9.76	109.8	100.3 2	100.3 2	1.06		49.25	0	18.49	115.86	86.18	86.18	1.23
41	Hidalgo	420	0.95	56.72	39.42	13.02	109.1 6	100.3 2	100.3 2	1.04		49.25	0	27.73	120.78	86.18	86.18	1.21
42	Hidalgo	420	0.95	56.72	35.66	19.54	111.9 2	100.3 2	100.3 2	1.05		49.25	2	27.73	116.60	86.18	86.18	1.22

Table A-7—Partially Grouted Clay Wall Properties

Wall #	Researcher	Wall ID	H (in)	L (in)	t (in)	AR	Vert. Reinf. (in ² /pl)	Vsp (in)	ρ_v (%)	f_{yv} (psi)	Horiz. Reinf. (in ² /pl)	# of locations	Hsp (in)	ρ_h (%)	f_{yh} (psi)	Axial Load (k)	Gr. f'm (psi)	Ung. f'm (psi)	Vmax (k)
1	Hidalgo	HCBR-21-3	80	42	8	1.9	0.785	36	0.51	47300	0	0	0	0.000	0	33.00	4654		27.10
2	Hidalgo	HCBR-21-5	80	42	8	1.9	0.785	36	0.51	47300	0.31	2	27	0.105	49700	53.60	4654		47.60
3	Hidalgo	HCBR-21-7	80	42	8	1.9	0.785	36	0.51	47300	0.31	3	20	0.158	49700	52.30	4654		47.50
4	Chen	HCBR-11-2	56	48	8	1.2	0	0	0	0	0	0	0	0.000	0	76.50	2655		26.60
5	Chen	HCBR-11-5	56	48	8	1.2	0.31	42	0.18	71340	0.31	1	28	0.075	70000	53.70	2655		45.40
6	Chen	HCBR-11-9	56	48	8	1.2	0.785	42	0.45	69200	0	0	0	0.000	0	37.30	2655		43.00
7	Chen	HCBR-11-11	56	48	8	1.2	0.785	42	0.45	72870	0.31	2	19	0.150	68700	26.70	2655		46.00

Table A-8—Partially Grouted Clay Wall Analysis

		MSJC										Proposed						
Wall #	Researcher	An	$\frac{M_u}{V_u d_v}$	Masonry Cont.	Axial Load	Steel Cont.	Total	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$		Masonry Cont.	# Undev	Steel Cont.	Total w/ Axial Load	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$
1	Hidalgo	164	0.95	26.03	7.43	0	33.45	46.03	33.45	0.81		31.53	0	0.00	39.78	55.19	39.78	0.68
2	Hidalgo	164	0.95	26.03	12.06	9.76	47.85	46.03	46.03	1.03		31.53	0	12.33	57.26	55.19	55.19	0.86
3	Hidalgo	164	0.95	26.03	11.77	13.02	50.81	46.03	46.03	1.03		31.53	0	18.49	63.10	55.19	55.19	0.86
4	Chen	120	0.58	18.42	17.21	0	35.63	31.6	31.6	0.84		29.37	0	0.00	48.50	51.40	48.50	0.55
5	Chen	179	0.58	27.4	12.08	15.94	55.43	47.01	47.01	0.97		29.37	0	8.68	51.48	51.40	51.40	0.88
6	Chen	179	0.58	27.4	8.39	0	35.79	47.01	35.79	1.20		29.37	0	0.00	38.70	51.40	38.70	1.11
7	Chen	179	0.58	27.4	6.01	23.91	57.32	47.01	47.01	0.98		29.37	0	17.04	53.08	51.40	51.40	0.89

Table A-9—AAC Wall Properties

Wall #	Researcher	Wall ID	H (in)	L (in)	t (in)	AR	Vert. Reinf. (in ² /pl)	Vsp (in)	ρ_v (%)	f_{yv} (psi)	Horiz. Reinf. (in ² /pl)	# of locations	Hsp (in)	ρ_h (%)	f_{yh} (psi)	Axial Load (k)	f'm (psi)	Vmax (k)
1	Tanner	3	152	240	8	0.63	0	0	0	0	0	0	0	0	0	120.00	650	110.90
2	Tanner	5	152	240	8	0.63	0	0	0	0	0	0	0	0	0	60.00	1040	81.05
3	Tanner	7	152	144	8	1.06	0	0	0	0	0	0	0	0	0	80.00	650	59.20
4	Tanner	11	152	48	8	3.17	0	0	0	0	0	0	0	0	0	25.00	1040	16.15
5	Cancino	18	154	144	8	1.07	0	0	0	0	0	0	0	0	0	45.00	344	43.04
6	Cancino	19	154	216	8	0.71	0	0	0	0	0	0	0	0	0	58.00	344	63.61

***Some AAC walls maintained axial load through external reinforcement—in this analysis, these are not considered as vertical reinforcing bars but instead as axial loading as represented in the appropriate reference

Table A-10—AAC Wall Analysis

AAC Wall Analysis																			
	MSJC											Proposed							
Wall #	Researcher	An	$\frac{M_u}{V_u d_v}$	Web-shear Cracking	Crushing of Strut	Sliding	Steel Cont.	Limits	Vn	$\frac{V_{exp}}{V_{pred}}$		Masonry Cont.	# Undev.	Steel Cont.	Total w/ Axial Load	Limit	Vn	$\frac{V_{exp}}{V_{pred}}$	
1	Tanner	1920	0.63	69.38	139.44	144.00	0.00	243.67	69.38	1.60		45.98	0	0	75.98	66.98	66.98	1.66	
2	Tanner	1920	0.63	71.67	223.11	72.00	0.00	308.22	71.67	1.13		73.57	0	0	88.57	107.16	88.57	0.92	
3	Tanner	1152	1	42.88	80.14	96.00	0.00	117.48	42.88	1.38		27.59	0	0	47.59	48.28	47.59	1.24	
4	Tanner	384	1	16.68	20.30	30.00	0.00	49.54	16.68	0.97		14.71	0	0	20.96	25.75	20.96	0.77	
5	Cancino	1152	1	29.08	42.23	54.00	0.00	85.47	29.08	1.48		14.41	0	0	25.66	25.22	25.22	1.71	
6	Cancino	1728	0.71	42.02	67.28	69.60	0.00	152.73	42.02	1.51		21.62	0	0	36.12	32.86	32.86	1.94	

Appendix B

Strut-and-Tie Specimens—CMU, Clay, and AAC

Table B-1—Partially Grouted Walls Analysis

Strut-and-Tie Values										From Visual Analysis				
Researcher	Wall I.D	AR	Vert. Tie Strength (k)	Horiz. Tie Strength (k)	θ	β_n	f_{me}	Controlling Strength (k)	Actual Failure (k)	First Yield (k)	$\frac{V_{exp}}{V_{pred}}$	Failure Load (k)	Failure Mode	$\frac{V_{exp}}{V_{pred}}$
Nolph	PG085-48	0.9	76.5	19.72	47.5, 45	0.6	1394	37.57, 39.35	49.84	N/A	N/A	43.5	crushing	1.15
Nolph	PG120-48	0.9	76.5	27.98	47.5, 45	0.6	1394	37.57, 39.35	51.17	N/A	N/A	43.5	crushing	1.18
Nolph	PG085-24	0.9	55.97, 39.43	19.72	26.6, 28.6	0.6	1394	29.7, 30.25	66.29	N/A	N/A	44	crushing	1.51
Voon	A5	1	20.56	N/A	41.6	0.8	2280	34.29	31.12	18.5	1.68	18.5	unstable	1.68
Voon	A6	1	20.56	N/A	41.6	0.8	2280	34.29	20.9	18.5	1.13	18.5	unstable	1.13
Schultz	2	0.5	52.8	2.75	61.7	0.8	1785	71.78	58.75	N/A	N/A	65	crushing	0.90
Schultz	4	0.7	52.8	2.75	52.1	0.8	1785	55.42	57	N/A	N/A	44	crushing	1.30
Minaie	PCL-1	0.7	26.4	26.4	45	0.6	1700	45.83	71.5	60	1.19	68	crushing	1.05
Minaie	MC-2	0.7	26.4	26.4	45	0.6	1700	45.83	51.8	41	1.26	61	crushing	0.85
Yancey	R2	1.2	N/A	2.75	45	0.8	1045	28.19	32	N/A	N/A	20	crushing	1.60
Yancey	R5	1.2	N/A	19.5	59.7	0.8	1035	39.134	43.95	N/A	N/A	19.5	crushing	2.25
Baenzinger	4	0.6	70.2	4.84	28.4, 30.26	0.8	2525	54.73, 55.8	96.9	44.5	2.18	57	crushing	1.70
Baenzinger	6	0.9	65.05, 26.76	9.68	28.4	0.8	2525	54.73	73.2	30	2.44	40.5	crushing	1.81
Baenzinger	7	0.9	40.15, 26.76	26.76	45	0.6	3035	81.82	56	42	1.33	54	excessive deflection	1.04
Matsumura	CW2-1-1	2	unknown	11.16	58, 33.7	0.6	2021	42.93, 27.32	17.68	11	1.61	11	excessive deflection	1.61
Matsumura	CW2-2'-A2-1	1.9	unknown	11.16	58, 53	0.6	1873	39.77, 35.12	17.66	11	1.60	11	unstable	1.61
Chen	HCBL-11-5	1.17	21.95	14.85	36.9	0.8	1454	34.64	46.8	N/A	N/A	21	unstable	2.23

Table B-1 Continued

Researcher	Wall I.D	AR	Vert. Tie Strength (k)	Horiz. Tie Strength (k)	θ	β_n	f_{me}	Controlling Strength (k)	Actual Failure (k)	First Yield (k)	$\frac{V_{exp}}{V_{pred}}$	Failure Load (k)	Failure Mode	$\frac{V_{exp}}{V_{pred}}$
Chen	HCBL-11-8	1.17	53.98	0	36.9	0.8	1454	34.65	36.8	N/A	N/A	21	crushing	1.75
Chen	HCBL-11-10	1.17	53.98	14.85	66.8	0.8	1454	34.64	48.7	N/A	N/A	20.8	excessive deflection	2.34
Hidalgo	HCBR-21-3	1.9	37.13	0	24.23	0.8	3956	80	27.1	24.2	1.11	24.2	unstable	1.12
Hidalgo	HCBR-21-5	1.9	37.13	15.4	54.16	0.8	3956	80	47.6	29	1.64	29	unstable	1.64
Chen	HCBR-11-5	1.17	22.12	21.7	56.3	0.8	2257	52	45.4	N/A	N/A	31.5	unstable	1.44
Chen	HCBR-11-9	1.17	54.32	0	36.9	0.8	2257	52	43	N/A	N/A	31.5	crushing	1.37
													Average	1.49
													COV	0.28

Table B-2—Fully Grouted Walls Analysis

Fully Grouted Walls--Analysis														
Strut-and-Tie Values										From Visual Analysis				
Researcher	Wall I.D	AR	Vert. Tie Strength (k)	Horiz. Tie Strength (k)	θ	β_n	f_{me}	Controlling Strength (k)	Actual Failure (k)	First Yield (k)	$\frac{V_{exp}}{V_{pred}}$	Failure Load (k)	Failure Mode	$\frac{V_{exp}}{V_{pred}}$
Shing	4	1	43.3	6.19	45	0.6	1330	59.34	79.5	25	3.18	57	unstable	1.39
Shing	9	1	19.64	6.19	45	0.6	1220	54.77	96	20	4.8	47	unstable	2.04
Shing	13	1	28.72	13.4	45	0.6	1680	75.31	112.5	50	2.25	82	crushing	1.37
Shing	16	1	43.3	13.4	45	0.6	1275	57.05	120.5	47	2.56	81	crushing	1.49
Voon	A4	1	20.29	5.1	26.6	0.6	1260	44.46	47.64	24	1.99	36	excessive deflection	1.32
Voon	A9	2	36.19	N/A	26.6	0.8	2400	84.73	46.18	21.6	2.14	21.6	unstable	2.14
Voon	A10	0.6	20.28	2.06	45	0.6	1800	80.45	131.46	17	7.73	97.5	unstable	1.35
Eikanas	4	0.93	20.49	12.82	26.6	0.6	830	54.04	59.2	33	1.79	56	crushing	1.06
Eikanas	7	0.72	20.49	12.82	45	0.6	830	68.35	69.8	38	1.84	70.7	crushing	0.99
Matsumura	KW2-1	2.3	55.82	N/A	16.7	0.8	2150	70.99	39.01	17.7	2.20	17.7	unstable	2.2
Matsumura	KW3-1	1.5	65.86	N/A	24.2	0.8	2150	74.56	59.17	N/A	N/A	30.5	crushing	1.94
Matsumura	KW3s-1	1.5	65.86	N/A	24.2	0.8	2150	74.56	65.18	N/A	N/A	30.5	crushing	2.14
Matsumura	KW4-1	1.13	87.68, 6.14	6.14	41.2	0.6	1610	67.77	90.58	17	5.33	63	crushing	1.44
Hidalgo	HCBL-12-1	0.5	48.18	0	38.7, 45	0.8	2241	166.8, 184.24	189.1	120	1.58	170	unstable	1.11
Hidalgo	HCBL-12-6	0.5	48.18	29.6	63.4, 53, 68.2, 59	0.6	1680	218.53, 162.9, 263.18, 189.77	252	85	2.96	155	excessive deflection	1.63
Chen	HCBL-11-3	1.17	21.95	0	36.9	0.8	1250	90.59	46.3	26	1.78	26	unstable	1.78
Chen	HCBL-11-4	1.17	21.95	N/A	36.9	0.8	1246	90.59	60.3	31	1.95	31	unstable	1.95
Chen	HCBL-11-9	1.17	54350	N/A	36.9	0.8	1295	94.12	53.6	N/A	N/A	56.5	crushing	0.95
Sveinsson	HCBL-11-13	1.17	20.93	N/A	36.9	0.8	2284	166	103.7	53	1.96	53	unstable	1.96
Sveinsson	HCBL-11-18	1.17	11.9	19.69	33.7	0.6	1172	44.5	96.3	33.5	2.87	48.5	crushing	1.99
Sveinsson	HCBL-11-21	1.17	11.9	19.69	33.7	0.6	1120	42.6, 37.35	87.5	29	3.02	43.2	crushing	2.03
Sveinsson	HCBR-11-21	1.17	17.58	19.69	33.7, 45	0.6	1488	56.6, 66.5	92.2	43	2.14	56.75	crushing	1.62
Sveinsson	HCBR-11-23	1.17	11.9	19.69	33.5, 18.4	0.6	1488	56.6, 49.6	79.6	29	2.74	52	crushing	1.53

Table B-2 Continued

Researcher	Wall ID	AR	Vert. Tie Strength (k)	Horiz. Tie Strength (k)	θ	β_n	f_{me}	Controlling Strength (k)	Actual Failure (k)	First Yield (k)	$\frac{V_{exp}}{V_{pred}}$	Failure Load (k)	Failure Mode	$\frac{V_{exp}}{V_{pred}}$
Sveinsson	CBRC-11-9	1.17	20.93	N/A	36.9	0.8	1688	211	113.6	75.5	1.50	75.5	unstable	1.5
Sveinsson	CBRC-11-12	1.17	20.93	N/A	36.9	0.8	1688	211	131.2	75.5	1.74	75.5	unstable	1.74
Chen	HCBR-11-3	1.17	23.25	0	36.9	0.8	1724	125.29	94.4	37	2.55	37	unstable	2.55
Chen	CBRC-11-6	1.17	57.2	N/A	36.9	0.8	1705	213.1	128.3	81	1.58	81	unstable	1.58
Hidalgo	HCBR-12-3	0.51	48.18	N/A	42	0.8	2186	160	211.8	132	1.60	164	unstable	1.29
Hidalgo	CBRC-12-6	0.51	48.18	48.18	63.43	0.6	1485	332	207.3	145	1.43	153	excessive deflection	1.35
Hidalgo	HCBR-21-2	1.9	37.13	0	24.23	0.8	3165	188.6	63.7	42.5	1.5	42.5	unstable	1.5
Hidalgo	HCBR-21-6	1.9	37.13	N/A	24.23	0.8	3164	188.6	98.2	51	1.93	51	unstable	1.93
Hidalgo	CBRC-21-2	1.9	37.13	0	24.23	0.8	2278	249.9	114.2	62	1.84	62	unstable	1.84
Tanner	3	0.63	N/A	N/A	57.7	0.8	440	52.86	110.9	N/A	N/A	45	crushing	2.46
Tanner	5	0.63	N/A	N/A	57.7	0.8	700	84.58	81.05	N/A	N/A	71.5	crushing	1.13
Tanner	11	3.17	N/A	N/A	17.5	0.8	700	47.44	16.15	N/A	N/A	14.3	crushing	1.13
Average														1.64
COV														0.25

Appendix C

Finite Element Analysis Material Properties and Wall Data

Table C-1—Material Properties

Minaie (2009)					
Density ($\frac{\text{lb}}{\text{in}^3}$)		Elastic Modulus (psi)	Poissons Ratio	Dilation Angle (degrees)	
0.002248		600000	0.2	38	
Eccentricity		$\frac{f_{bo}}{f_{co}}$	K	Viscosity Parameter	
0.1		1.16	0.67	0	
Compression Behavior			Tension Behavior		
Tension Recovery		1.0	Compression Recovery		0.0
Yield Stress (psi)	Inelastic Strain	Damage	Yield Stress (psi)	Cracking Strain	Damage
524	0	0	30	0	0
987	0.000222	0	60	5.00E-05	0
1388	0.000445	0	90	0.0001	0
1730	0.000667	0	120	0.00015	0
2006	0.000889	0.1	150	0.0002	0
2222	0.001111	0.15	180	0.00025	0
2376	0.001334	0.2	210	0.0003	0

This Study					
Density ($\frac{\text{slug}}{\text{in}^3}$)		Elastic Modulus (psi)	Poissons Ratio	Dilation Angle (degrees)	
0.00212		2340000	0.2	5	
Eccentricity		$\frac{f_{bo}}{f_{co}}$	K	Viscosity Parameter	
0.1		1.16	0.67	0	
Compression Behavior			Tension Behavior		
Tension Recovery		0.0	Compression Recovery		1.0
Yield Stress (psi)	Inelastic Strain	Damage	Yield Stress (psi)	Cracking Strain	Damage
1302	0.00000	0	30	0	0
1533	0.00014	0.05	50	0.00006	0.05
1844	0.00034	0.075	70	0.00012	0.07
2103	0.00054	0.1	90	0.00018	0.09
2312	0.00074	0.125	110	0.00024	0.11
2476	0.00094	0.15	130	0.0003	0.13
2598	0.00114	0.175	150	0.00036	0.15

Table C-1 Continued

Yield Stress (psi)	Inelastic Strain	Damage	Yield Stress (psi)	Cracking Strain	Damage	Yield Stress (psi)	Inelastic Strain	Damage	Yield Stress (psi)	Cracking Strain	Damage
2469	0.001556	0.25	240	0.00035	0	2682	0.00134	0.2	170	0.00042	0.17
2500	0.001778	0.3	225	0.00055	0.1	2730	0.00154	0.225	190	0.00048	0.19
2000	0.002778	0.4	210	0.00075	0.1	2746	0.00174	0.25	210	0.00054	0.21
			200	0.00085	0.15	2734	0.00194	0.275	230	0.0006	0.23
			170	0.00105	0.15	2697	0.00214	0.3	250	0.00066	0.25
			130	0.00125	0.2	2638	0.00234	0.325	270	0.00072	0.27
			90	0.00145	0.2	2561	0.00254	0.35	290	0.00078	0.29
			50	0.0017	0.3	2469	0.00274	0.375	270	0.00103	0.3
			10	0.00212	0.45	2366	0.00294	0.4	250	0.00128	0.35
			0.02	0.0029	0.55	2255	0.00314	0.425	230	0.00153	0.4
			0.02	0.0059	0.65	2139	0.00334	0.45	210	0.00178	0.45
			0.02	0.0079	0.95	2022	0.00354	0.5	190	0.00203	0.5
			0.02	0.0099	0.95	1907	0.00374	0.52	170	0.00228	0.55
			0.02	0.0149	0.95	1797	0.00394	0.54	150	0.00253	0.6
			0.02	0.0199	0.95	1697	0.00414	0.56	130	0.00278	0.65
			0.02	0.0299	0.95	1609	0.00434	0.58	110	0.00303	0.7
			0.02	0.05	0.95	1537	0.00454	0.6	90	0.00328	0.75
			0.02	0.1	0.95	1462	0.00474	0.62	70	0.00353	0.8
						1387	0.00494	0.64	50	0.00378	0.85
						1312	0.00514	0.66	30	0.00403	0.9
						1237	0.00534	0.68	10	0.00428	0.95
						1162	0.00554	0.7	5	0.00453	0.95
						1087	0.00574	0.72			
						1012	0.00594	0.74			
						937	0.00614	0.76			
						862	0.00634	0.78			
						787	0.00654	0.8			
						712	0.00674	0.82			
						637	0.00694	0.83			
						562	0.00714	0.84			

Table C-1 Continued

Yield Stress (psi)	Inelastic Strain	Damage	Yield Stress (psi)	Cracking Strain	Damage		Yield Stress (psi)	Inelastic Strain	Damage	Yield Stress (psi)	Cracking Strain	Damage
							487	0.00734	0.85			
							412	0.00754	0.86			
							337	0.00774	0.87			
							262	0.00794	0.88			
							187	0.00814	0.89			
							112	0.00834	0.9			

***Common to all Walls:

$$f'_m = 2746 \text{ psi}$$

Vertical Reinforcement = #7, $f_y=72000$ psi

Horizontal Reinforcement = #3, $f_y=56000$ psi

Table C-2—Wall Properties

Wall ID	H (in)	L (in)	t (in)	AR	Vsp (in)	ρ_v (%)	Hsp (in)	ρ_h (%)	# of Horizontal Locations	Axial Load (lbs)	Avg. Vmax (lbs)
Fully Grouted											
1--FG	72	72	6	1	16	0.69	16	0.13	5	0	76.20
2--FG	72	72	6	1	64	0.28	68	0.025	1	99.40	76.45
3--FG	72	144	6	0.5	16, 8	0.69	16	0.13	5	198.72	186.89
4--FG	72	144	6	0.5	48, 40	0.28	68	0.025	1	198.72	177.2
5--FG	72	288	6	0.25	40	0.28	16	0.13	5	397.44	348.38
6--FG	72	288	6	0.25	56	0.21	68	0.025	1	397.44	351.99
7--FG	144	72	6	2	16	0.69	16, 8	0.127	10	298	60.91
8--FG	144	72	6	2	16	0.69	48, 40	0.051	4	198.72	61.07
11--FG	144	144	6	1	16, 8	0.69	16	0.115	9	198.72	164.2
12--FG	288	288	6	1	16, 8	0.66	16,8	0.121	19	397.44	326.7
13--FG	24	24	6	1	16	0.83	16	0.153	2	66.24	27.8
14--FG	24	48	6	0.5	40	0.42	16	0.153	2	132.48	52.17
15--FG	48	24	6	2	16	0.83	40	0.076	2	66.24	20.17
16--FG	72	24	6	3	16	0.83	64	0.051	2	99.36	19.3
17--FG	72	72	12	1	16	0.347	16	0.064	5	199.58	153.7
18--FG	72	288	12	0.25	56	0.104	68	0.0127	1	794.88	656.04
19--FG	144	72	12	2	16	0.347	48, 40	0.0255	4	198.72	90.93

Table C-2 Continued
Partially Grouted

Wall ID	H (in)	L (in)	t (in)	AR	Vsp (in)	ρ_v (%)	Hsp (in)	ρ_h (%)	# of Horizontal Locations	Axial Load (lbs)	Avg. Vmax (lbs)
1--PG	72	72	6	1	16	0.69	16	0.13	5	99.36	55.3
2--PG	72	72	6	1	64	0.28	68	0.025	1	99.36	31.6
3--PG	72	144	6	0.5	16, 8	0.69	16	0.13	5	198.72	116.82
4--PG	72	144	6	0.5	48, 40	0.28	68	0.025	1	198.72	65.77
5--PG	72	288	6	0.25	40	0.28	16	0.13	5	397.44	171
6--PG	72	288	6	0.25	56	0.21	68	0.025	1	397.44	128
7--PG	144	72	6	2	16	0.69	16, 8	0.127	10	198.72	39.32
8--PG	144	72	6	2	16	0.69	48, 40	0.051	4	198.72	33.26
9--PG	144	144	6	1	16, 8	0.69	16	0.115	9	198.72	105.82
10--PG	72	72	12	1	16	0.347	16	0.064	5	99.36	84.71
11--PG	72	288	12	0.25	56	0.104	68	0.0127	1	794.88	206.12
12--PG	144	72	12	2	16	0.347	48, 40	0.0255	4	198.72	50.99

Table C-3—Wall Analysis

	MSJC										Proposed Simplified Equations						
Wall ID	An (in ²)	$\frac{M_u}{V_u d_v}$	Masonry Cont. (k)	Axial Load Cont. (k)	Steel Cont. (k)	Total (k)	Limit (k)	Vn (k)	$\frac{V_{exp}}{V_{pred}}$		Masonry Cont. (k)	# Undev.	Steel Cont. (k)	Total w/ Axial Load (k)	Limit (k)	Vn (k)	$\frac{V_{exp}}{V_{pred}}$
Fully Grouted																	
1--FG	432	1	50.94	0.00	9.90	60.84	90.55	60.84	1.25		44.98	2	11.09	56.07	78.71	56.07	1.36
2--FG	432	1	50.94	22.37	2.33	75.63	90.55	75.63	1.01		44.98	1	0.00	69.83	78.71	69.83	1.09
3--FG	864	0.5	141.49	44.71	19.80	206.00	241.47	206.00	0.91		89.96	2	11.09	150.73	121.44	121.44	1.54
4--FG	864	0.5	141.49	44.71	4.66	190.86	241.47	190.86	0.93		89.96	1	0.00	139.64	121.44	121.44	1.46
5--FG	1728	0.25	322.59	89.42	39.60	451.61	543.31	451.61	0.77		179.92	2	11.09	290.37	206.91	206.91	1.68
6--FG	1728	0.25	322.59	89.42	9.32	421.33	543.31	421.33	0.84		179.92	1	0.00	279.28	206.91	206.91	1.70
7--FG	432	1	50.94	67.05	9.90	127.89	90.55	90.55	0.67		22.49	7	11.09	108.08	39.36	39.36	1.55
8--FG	432	1	50.94	44.71	3.30	98.95	90.55	90.55	0.67		22.49	3	3.70	75.87	39.36	39.36	1.55
11--FG	864	1	101.87	44.71	19.80	166.38	181.10	166.38	0.99		44.98	2	25.87	120.53	157.43	120.53	1.36
12--FG	1728	1	203.74	89.42	39.60	332.76	362.20	332.76	0.98		44.98	2	62.83	207.17	157.43	157.43	2.08
13--FG	144	1	16.98	14.90	3.30	35.18	30.18	30.18	0.92		44.98	2	0.00	61.54	78.71	61.54	0.45
14--FG	288	0.5	47.16	29.81	6.60	83.57	80.49	80.49	0.65		89.96	2	0.00	123.08	121.44	121.44	0.43
15--FG	144	1	16.98	14.90	1.32	33.20	30.18	30.18	0.67		22.49	2	0.00	39.05	39.36	39.05	0.52
16--FG	144	1	16.98	22.36	0.83	40.16	30.18	30.18	0.64		14.99	2	0.00	39.83	26.24	26.24	0.74
17--FG	864	1	101.87	44.91	9.90	156.68	181.10	156.68	0.98		110.72	2	11.09	171.70	193.76	171.70	0.90
18--FG	3456	0.5	565.94	178.85	9.32	754.11	965.88	754.11	0.87		442.87	1	0.00	641.59	509.31	509.31	1.29
19--FG	864	1	101.87	44.71	3.30	149.88	181.10	149.88	0.61		55.36	3	3.70	108.74	96.88	96.88	0.94
Partially Grouted																	
1--PG	304	1	35.84	22.36	9.90	68.10	63.72	63.72	0.87		23.07	2	7.39	55.30	40.37	40.37	1.37
2--PG	208	1	24.52	22.36	2.33	49.21	43.60	43.60	0.72		23.07	1	0.00	47.91	40.37	40.37	0.78
3--PG	608	0.5	99.56	44.71	19.80	164.08	169.92	164.08	0.71		46.13	2	7.39	103.20	62.28	62.28	1.88

Table C-3 Continued

Wall ID	An (in ²)	$\frac{M_u}{V_u d_v}$	Masonry Cont. (k)	Axial Load Cont. (k)	Steel Cont. (k)	Total (k)	Limit (k)	Vn (k)	$\frac{V_{exp}}{V_{pred}}$		Masonry Cont. (k)	# Undev.	Steel Cont. (k)	Total w/ Axial Load (k)	Limit (k)	Vn (k)	$\frac{V_{exp}}{V_{pred}}$
4--PG	416	0.5	68.12	44.71	4.66	117.49	116.26	116.26	0.57		46.13	1	0.00	95.81	62.28	62.28	1.06
5--PG	832	0.25	155.32	89.42	39.60	284.34	261.59	261.59	0.65		92.27	2	7.39	199.02	106.11	106.11	1.61
6--PG	768	0.25	143.37	89.42	9.32	242.11	241.47	241.47	0.53		92.27	1	0.00	191.63	106.11	106.11	1.21
7--PG	304	1	35.84	44.71	9.90	90.46	63.72	63.72	0.62		11.53	7	7.39	68.61	20.18	20.18	1.95
8--PG	304	1	35.84	44.71	3.30	83.86	63.72	63.72	0.52		11.53	3	2.46	63.68	20.18	20.18	1.65
9--PG	608	1	71.69	44.71	19.80	136.20	127.44	127.44	0.83		23.07	2	17.25	89.99	80.73	80.73	1.31
10--PG	560	1	66.03	22.36	9.90	98.28	117.38	98.28	0.86		57.67	2	7.39	89.90	100.92	89.90	0.94
11--PG	1176	0.25	219.54	178.85	9.32	407.71	369.75	369.75	0.56		230.66	1	0.00	429.38	265.26	265.26	0.78
12--PG	560	1	66.03	44.71	3.30	114.04	117.38	114.04	0.45		28.83	3	2.46	80.98	50.46	50.46	1.01

Table C-4—Wall Analysis, Proposed Strut-and-Tie Method

Wall I.D	θ	β_n	f_{me} (psi)	Controlling Strength (k)	Actual Failure (k)	First Yield (k)	$\frac{V_{exp}}{V_{pred}}$	Failure Load (k)	Failure Mode	$\frac{V_{exp}}{V_{pred}}$
Fully Grouted										
1--FG	45	0.6	1400	71.23	76.20	25	3.05	57	unstable	1.34
2--FG	43.3	0.8	1867	92.3	76.45	N/A	N/A	64	crushing	1.19
3--FG	45, 26.6	0.6	1400	71.23, 56.36	186.89	70	2.67	203	crushing	0.92
4--FG	35.2, 30.5	0.8	1867	82.15, 77.98	177.2	N/A	N/A	126	crushing	1.41
5--FG	68.2	0.6	1400	135.72	348.38	60	5.81	270	crushing	1.29
6--FG	39.5	0.8	1867	87.06	351.99	N/A	N/A	176	crushing	2.00
7--FG	45, 63.4	0.6	1400	71.28, 112.68	60.91	0		25	unstable	2.44
8--FG	18.4, 21.8	0.6	1400	53.13, 54.23	61.07	23	2.66	37	crushing	1.65
11--FG	45, 26.6	0.6	1400	71.23, 56.35	164.2	45	3.65	105	crushing	1.56
12--FG	45, 26.6, 63.4	0.6	1400	71.23, 56.35, 112.68	326.7	72	4.54	130.5	crushing	2.50
13--FG	45	0.8	1867	95	27.8	N/A	N/A	67	crushing	0.41
14--FG	68.2	0.8	1867	181	52.17	N/A	N/A	168	crushing	0.31
15--FG	21.8	0.8	1867	72.4	20.17	N/A	N/A	27	crushing	0.75
16--FG	14	0.8	1867	69.1	19.3	N/A	N/A	16.8	crushing	1.15
17--FG	45	0.6	1400	190.07	153.7	13	11.82	82	unstable	1.87
18--FG	39.5	0.8	1867	232.2	656.04	325	2.02	498	crushing	1.32
19--FG	18.4, 21.8	0.6	1400	141.7, 144.75	90.93	24	3.79	66.5	excess. defl.	1.37
Partially Grouted										
1--PG	45	0.6	2334	39.6	55.3	20	2.77	46.5	crushing	1.19
2--PG	43.3	0.8	2334	38.46	31.6	N/A	N/A	26.5	crushing	1.19
3--PG	45, 26.6	0.6	2334	39.4, 31.3	116.82	68	1.72	105.5	crushing	1.11

Table C-4 Continued

Wall I.D	θ	β_n	f_{me} (psi)	Controlling Strength (k)	Actual Failure (k)	First Yield (k)	$\frac{V_{exp}}{V_{pred}}$	Failure Load (k)	Failure Mode	$\frac{V_{exp}}{V_{pred}}$
4--PG	35.2, 30.5	0.8	2334	34.26, 32.49	65.77	N/A	N/A	48	crushing	1.37
5--PG	68.2	0.6	2334	75.4	171	60	2.85	154	crushing	1.11
6--PG	39.5	0.8	2334	36.27	128	N/A	N/A	84	crushing	1.52
7--PG	45, 63.4	0.6	2334	39.6, 62.6	39.32	13	3.02	25	unstable	1.57
8--PG	18.4, 21.8	0.6	2334	29.5, 30.16	33.26	N/A	N/A	22	crushing	1.51
9--PG	45, 26.56	0.6	2334	39.6, 31.3	105.82	45	2.35	77	crushing	1.37
10--PG	45	0.6	2334	66	84.71	20	4.24	65.5	crushing	1.29
11--PG	39.5	0.8	2334	60.5	206.12	N/A	N/A	140	crushing	1.47
12--PG	18.4, 21.8	0.6	2334	49.2, 50.3	50.99	24	2.12	34.5	crushing	1.48

***All walls had vertical tie strengths of 43.30 k and horizontal tie strengths of 6.19 k

Appendix D—Finite Element Specimens

***This section contains those walls that were built using Abaqus/CAE finite element software to extend the scope of the research

---For each respective wall, there are representations of the:

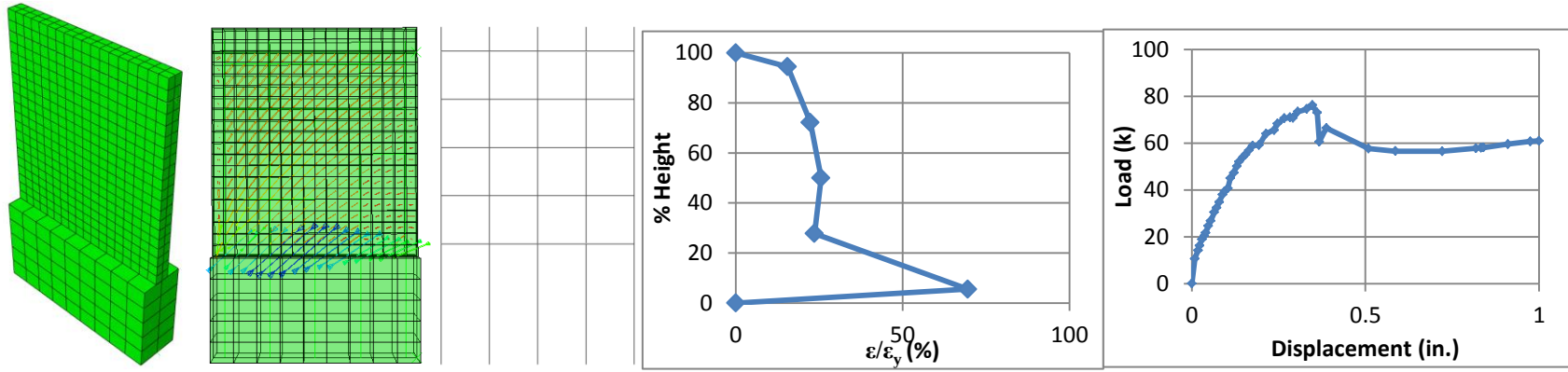
1. virtual wall
2. steel arrangement in the wall
3. view of regions of plastic compressive strain distribution at peak load (viewing regions of maximum and minimum plastic strain is the only way to observe “cracking” or “crushing” in the Concrete Damaged Plasticity model in Abaqus/CAE)

***Note—plastic compressive strain regions are displayed in terms of increasing values by the reverse rainbow spectrum (ROY G BIV), where the blue end of the spectrum contains higher values of plastic compressive strain

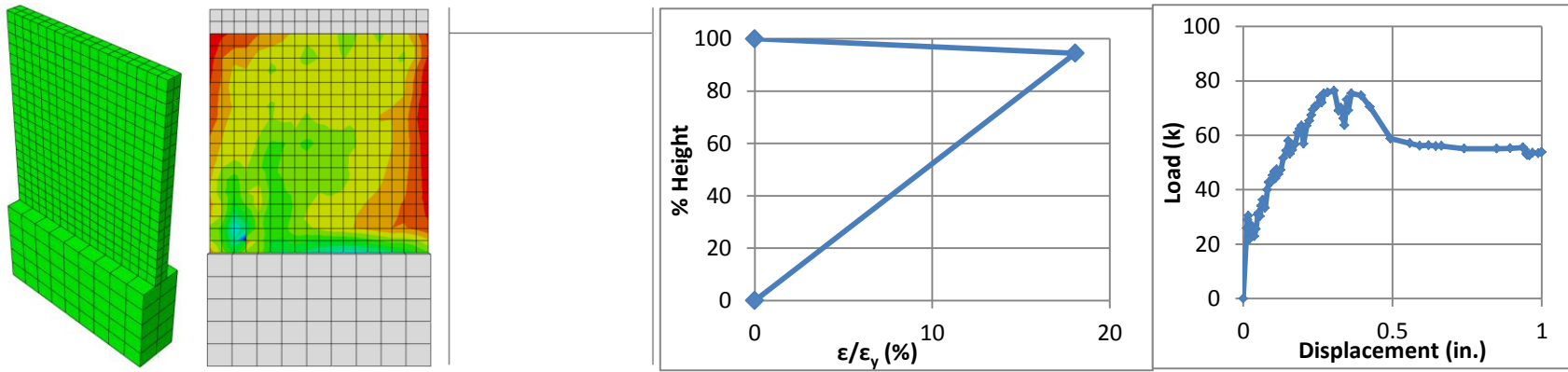
4. distribution of the maximum horizontal reinforcement strain at peak load (ϵ is the maximum strain in the bar at peak load)
5. Load vs. Displacement curve for the tested range of displacement

Figure D-1 Fully Grouted Finite Element Specimens

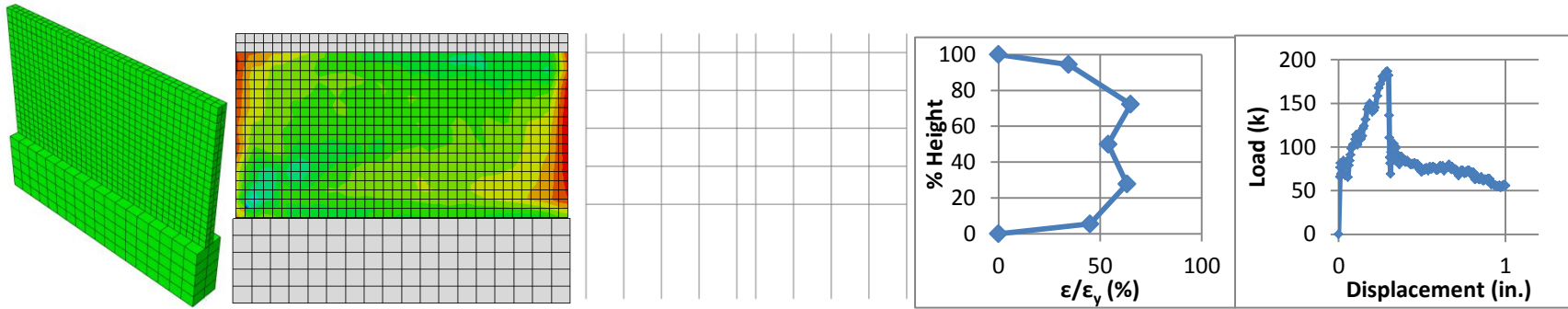
Wall #1--FG



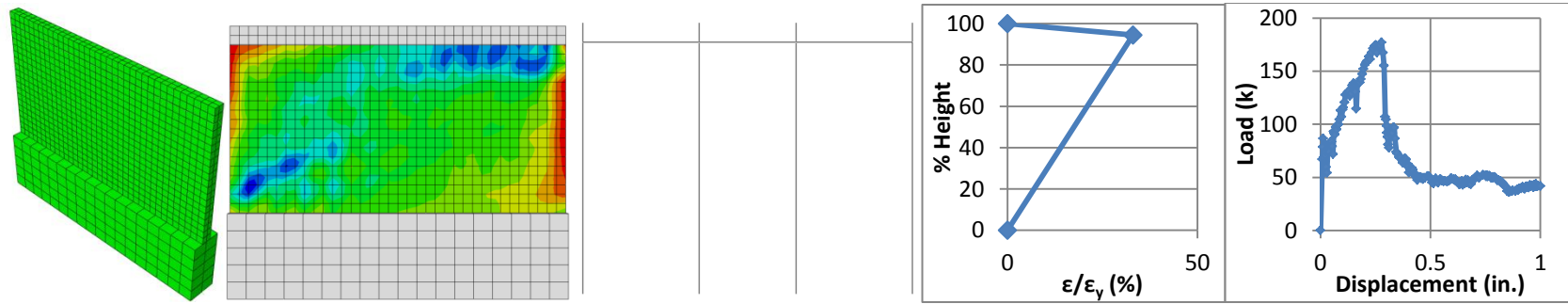
Wall #2—FG



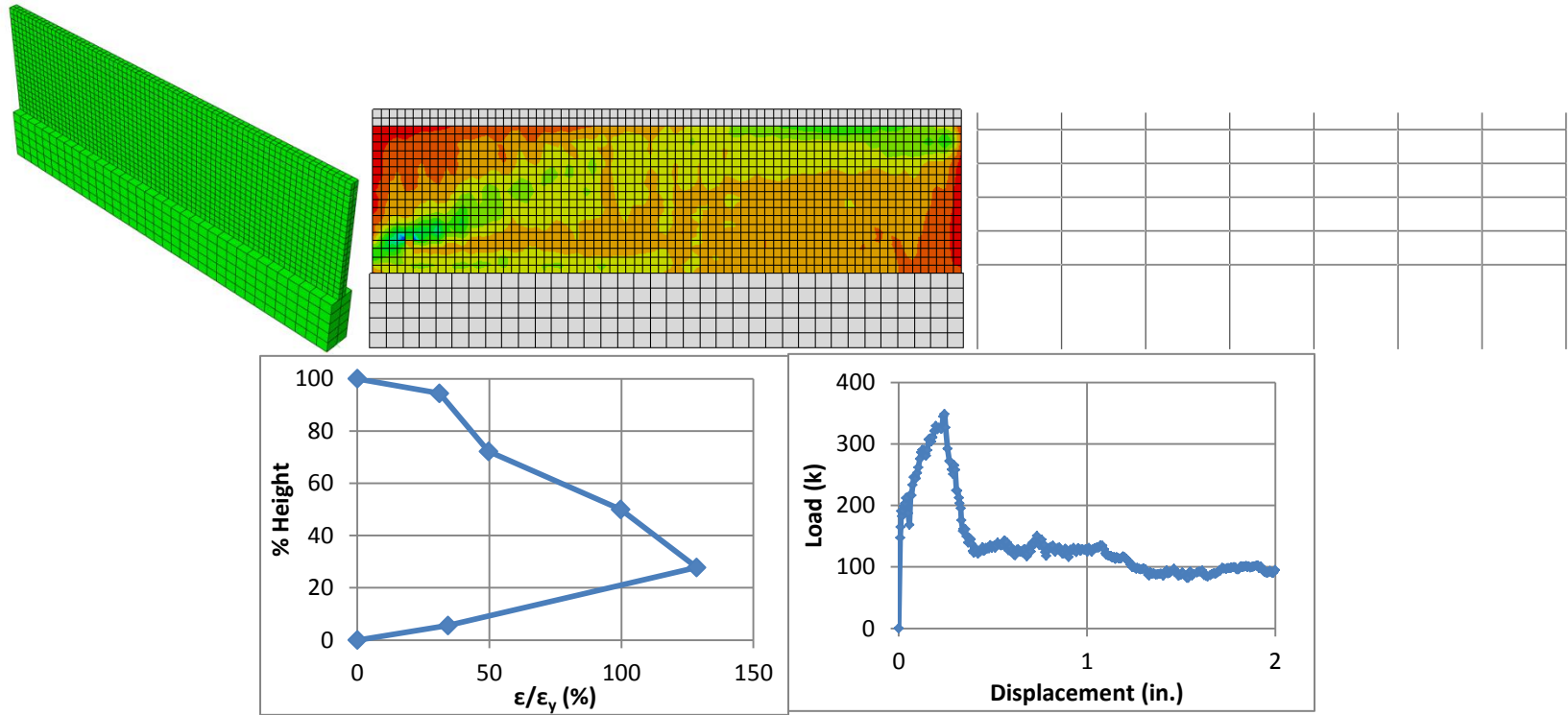
Wall #3—FG



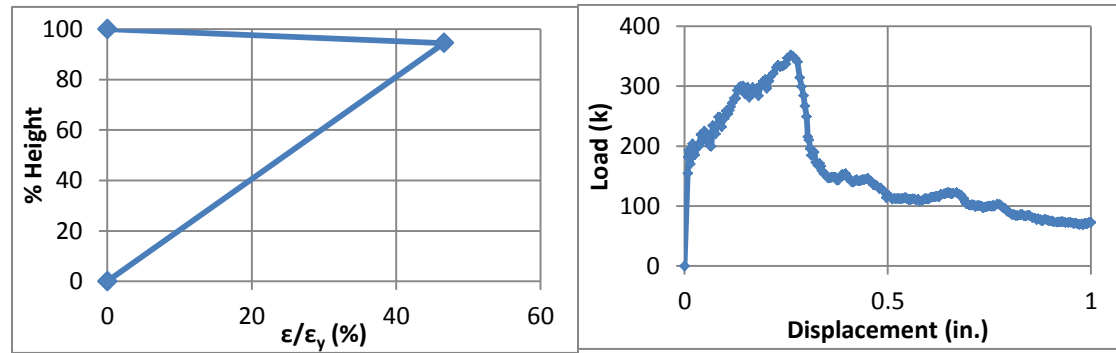
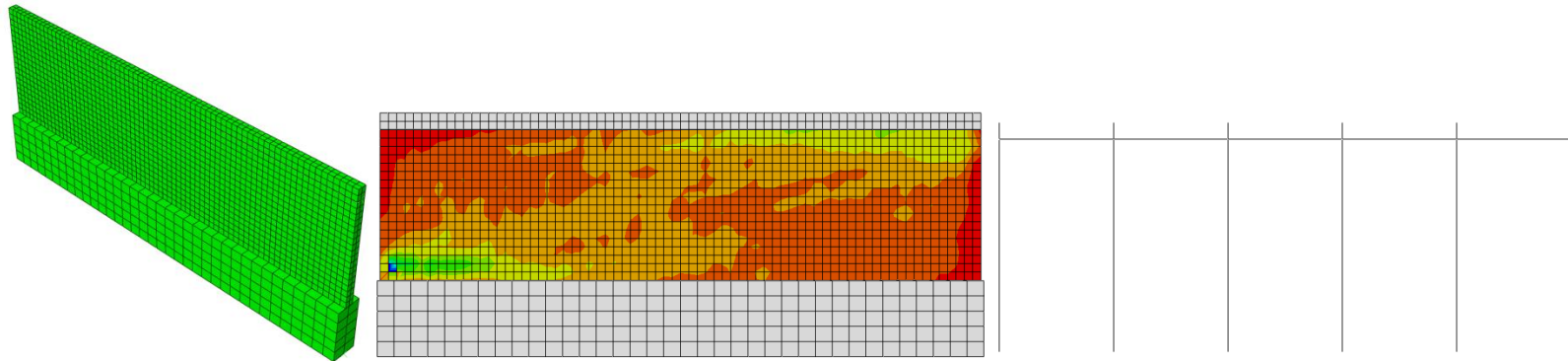
Wall #4--FG



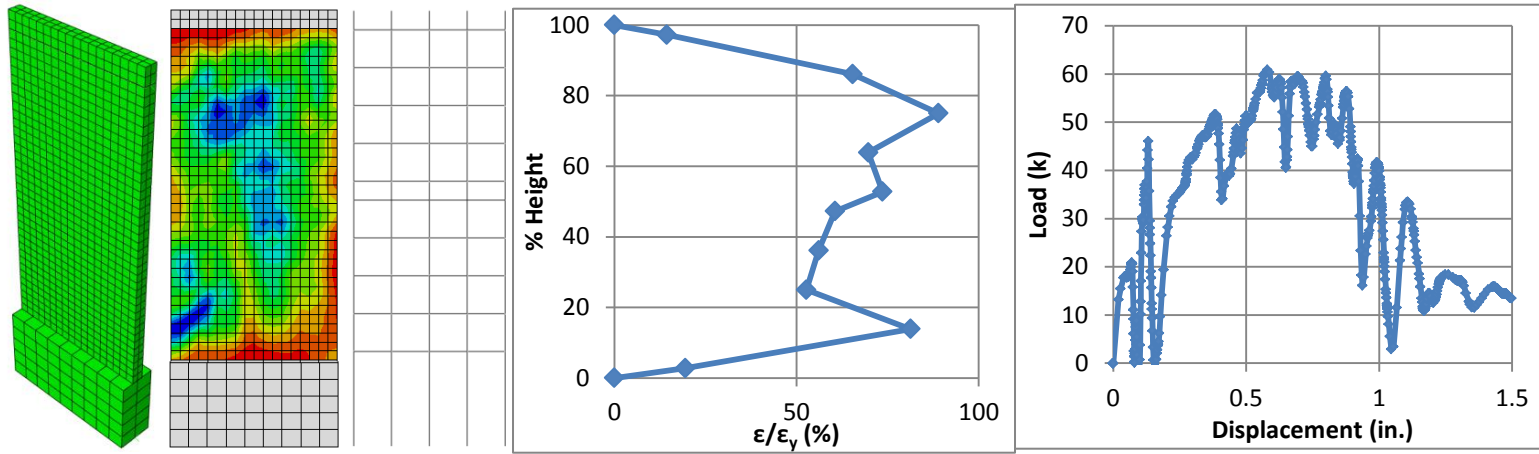
Wall #5--FG



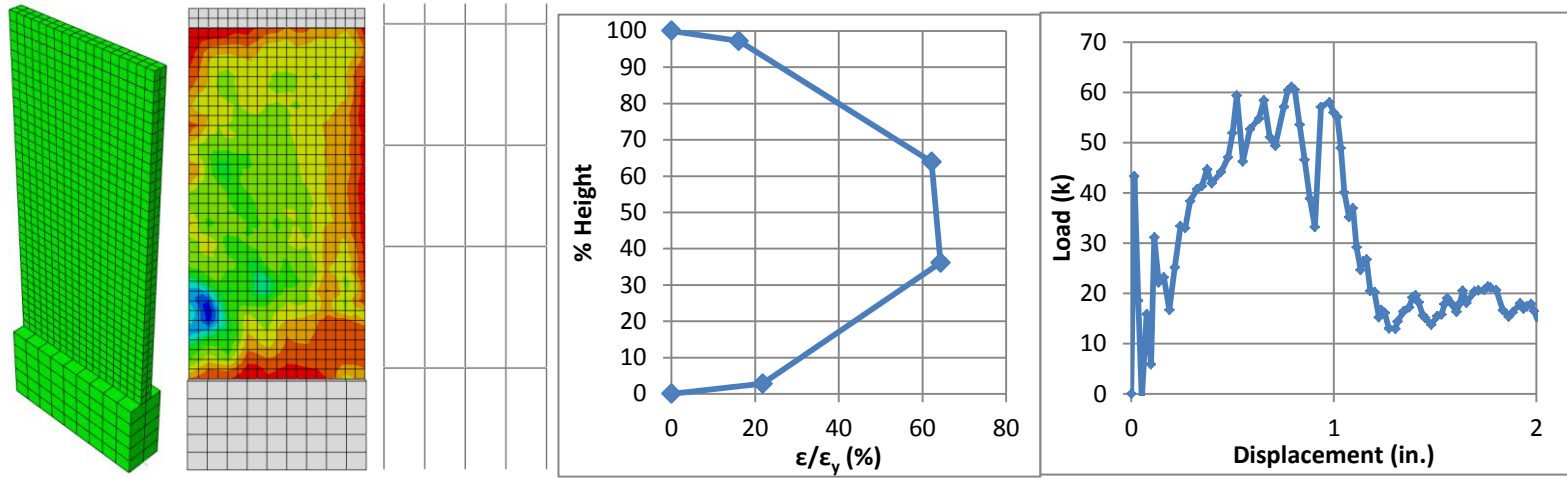
Wall #6--FG



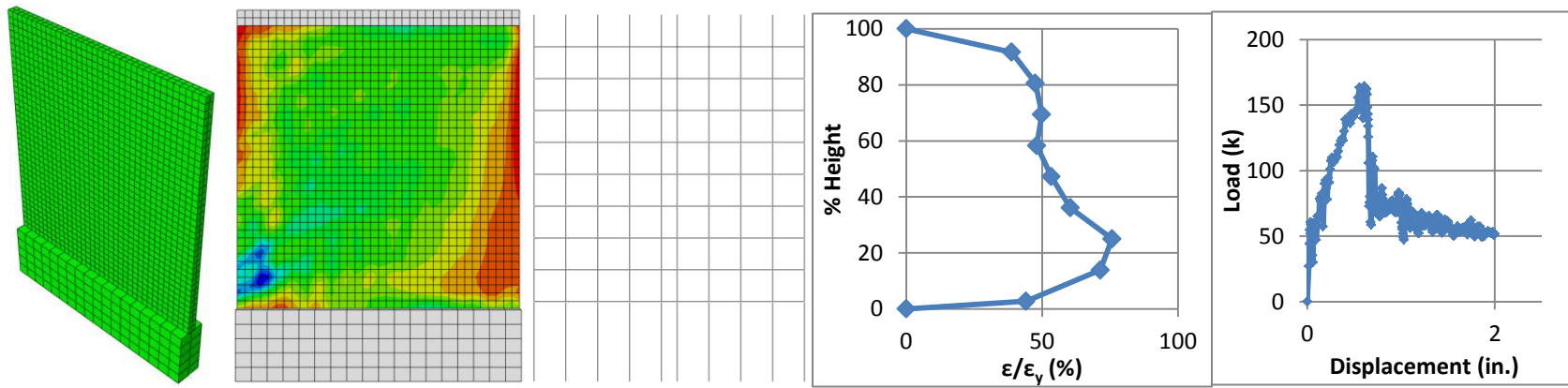
Wall #7--FG



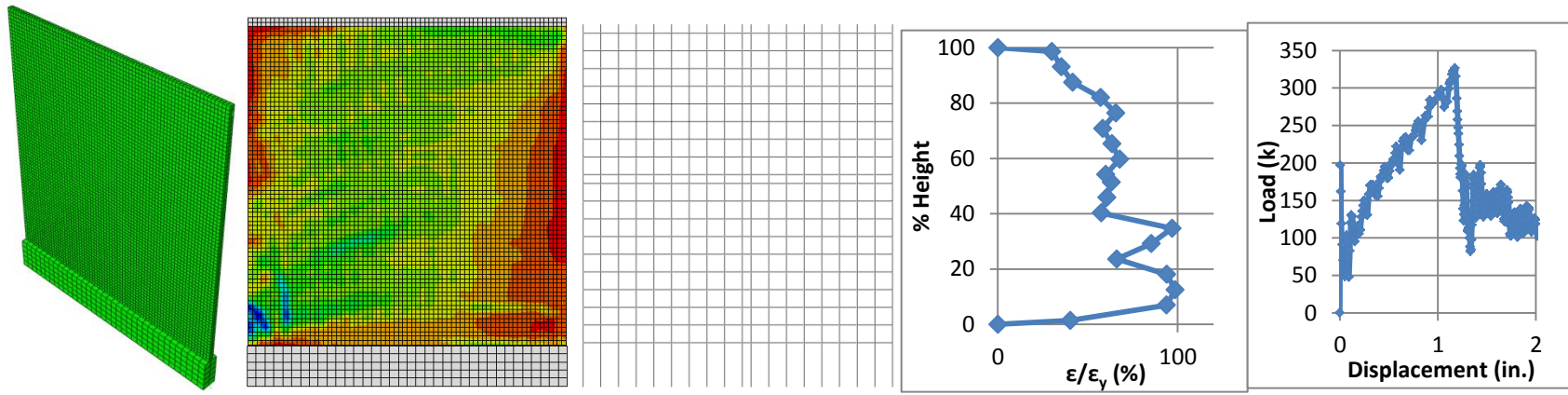
Wall #8--FG



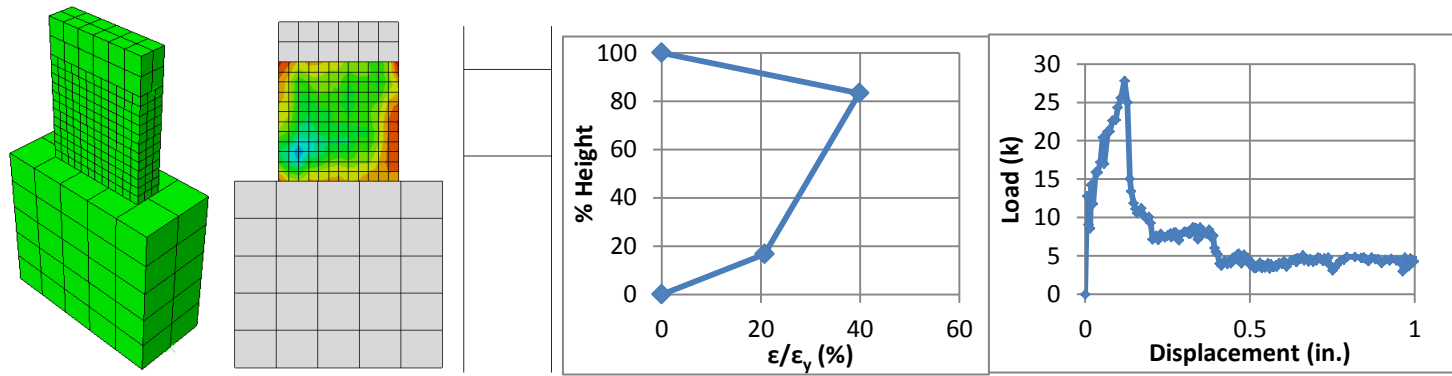
Wall #11--FG



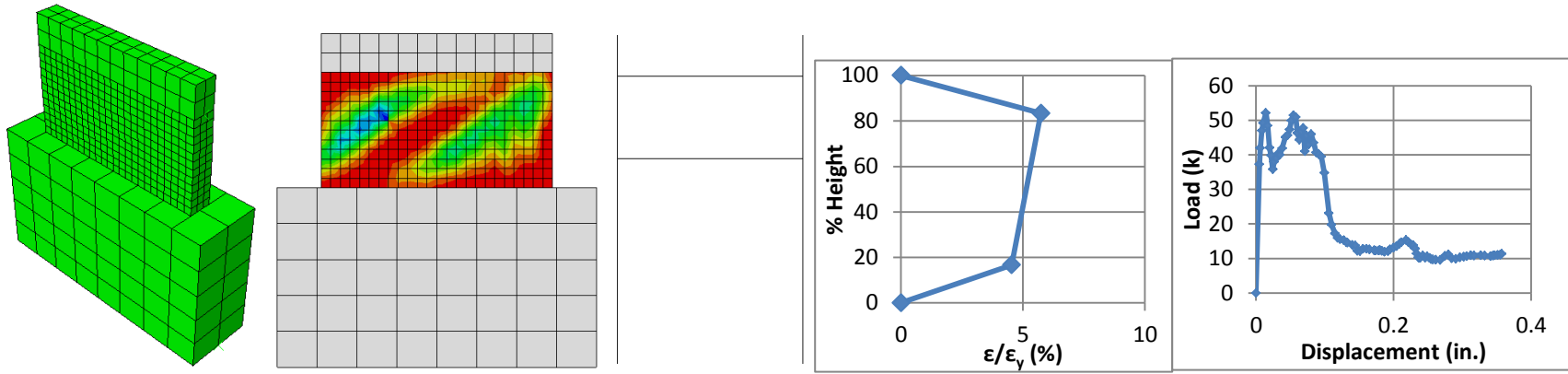
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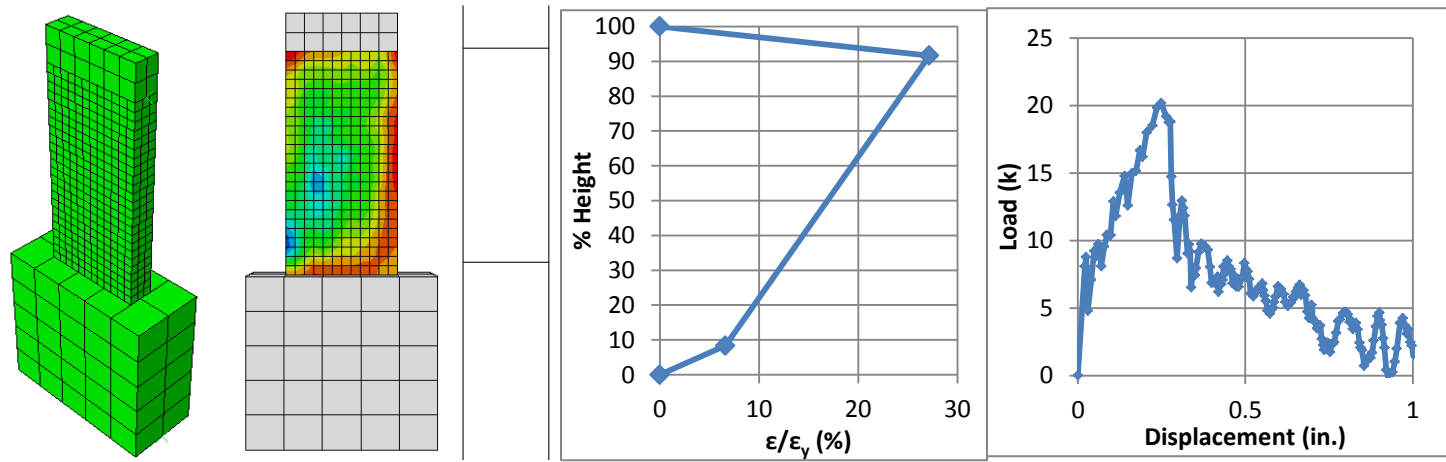
Wall #13—FG



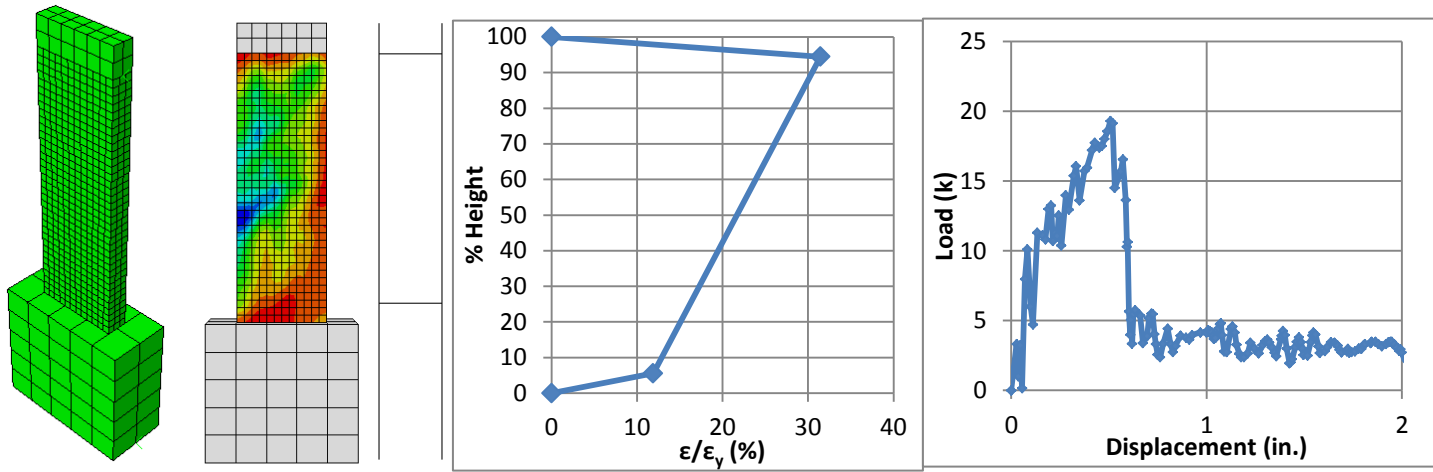
Wall #14--FG



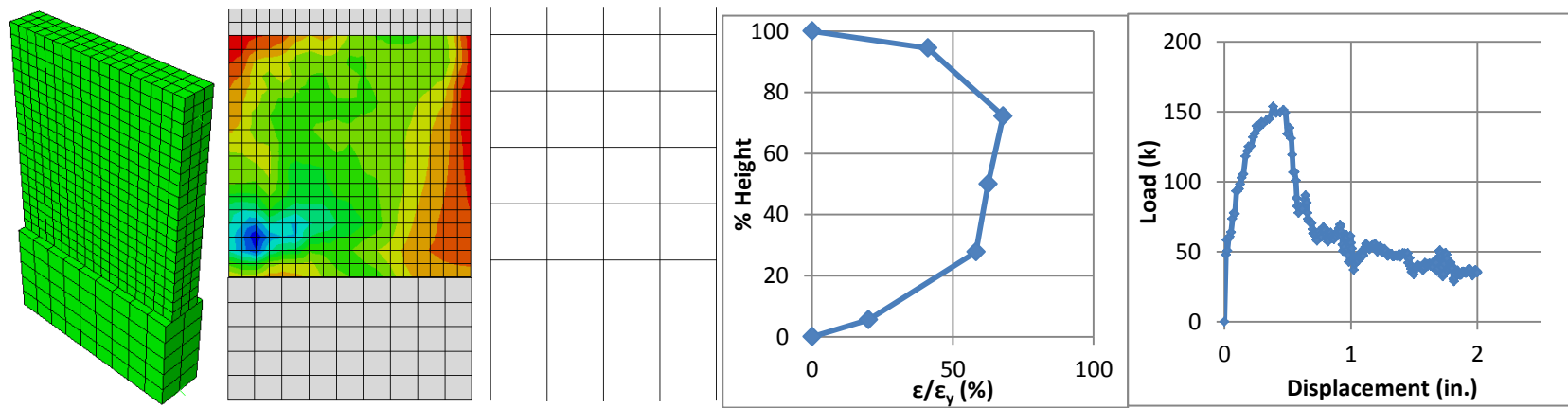
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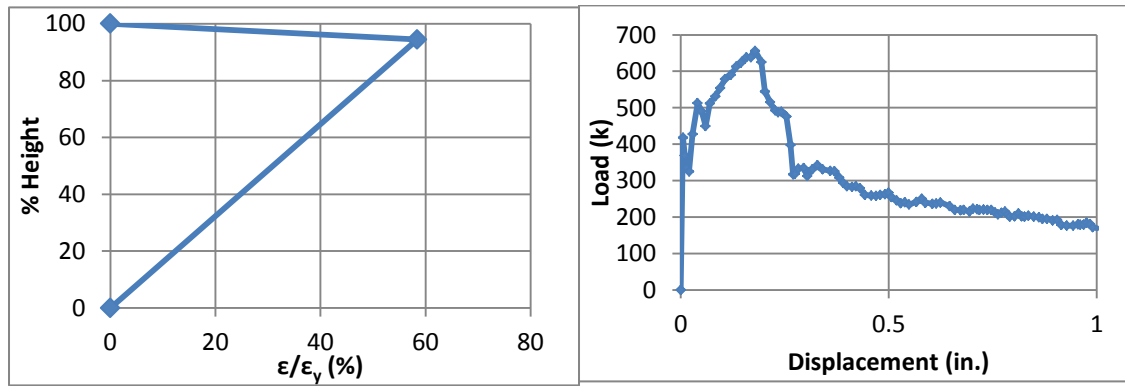
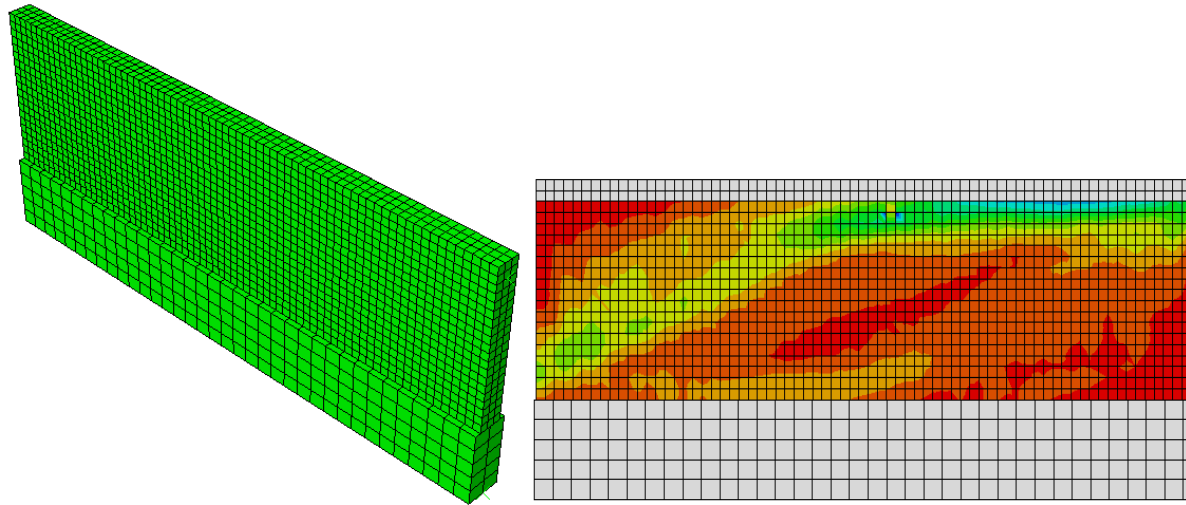
Wall #16—FG



Wall #17--FG



Wall #18—FG



Wall #19—FG

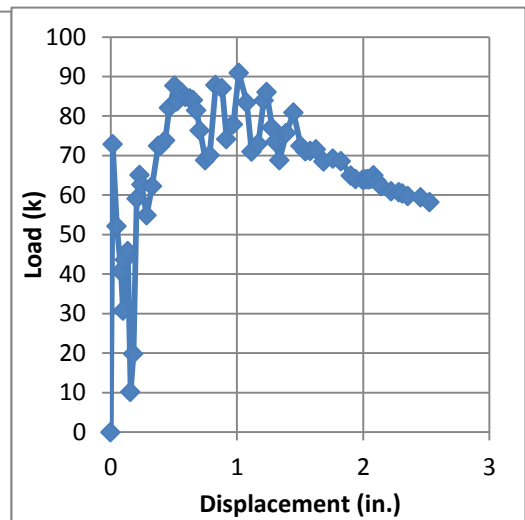
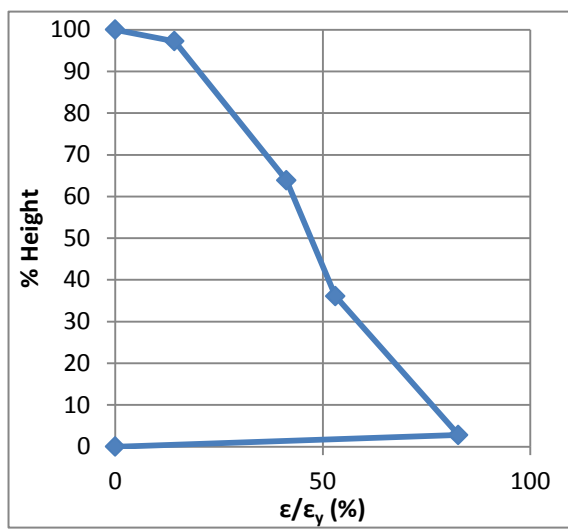
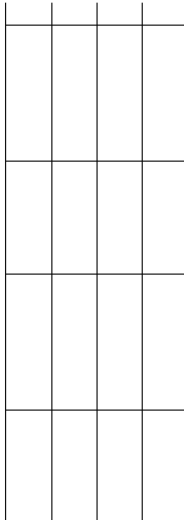
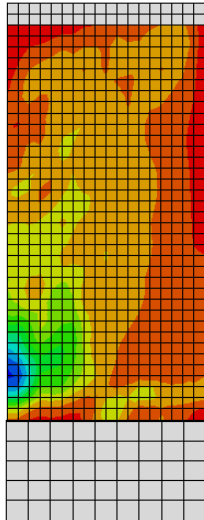
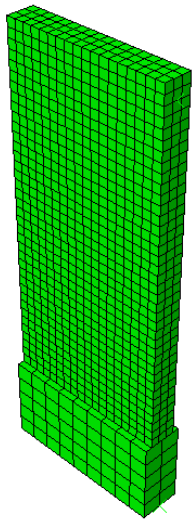
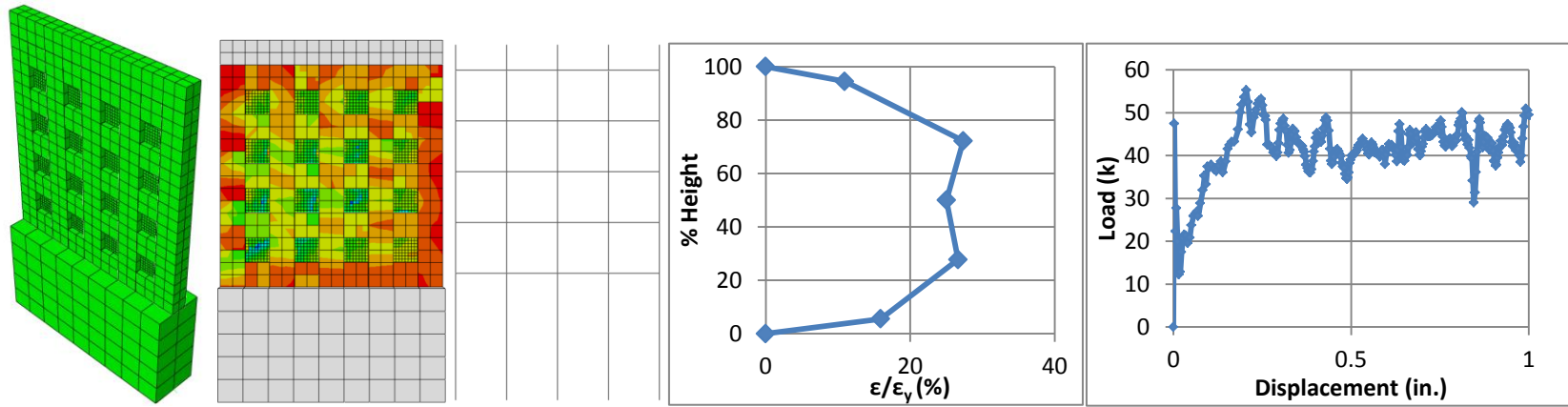
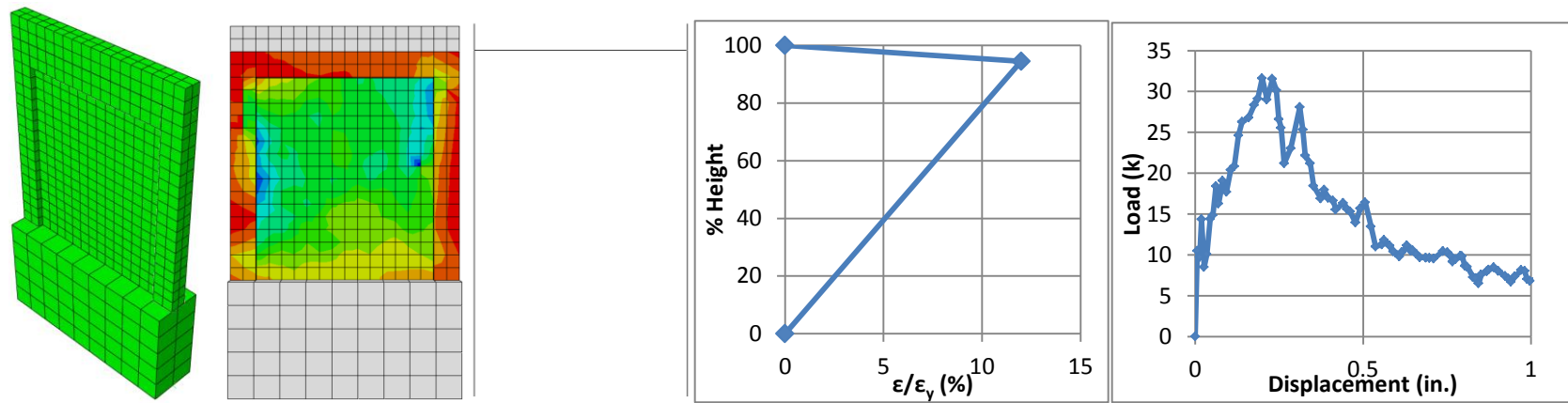


Figure D-2 Partially Grouted Finite Element Specimens

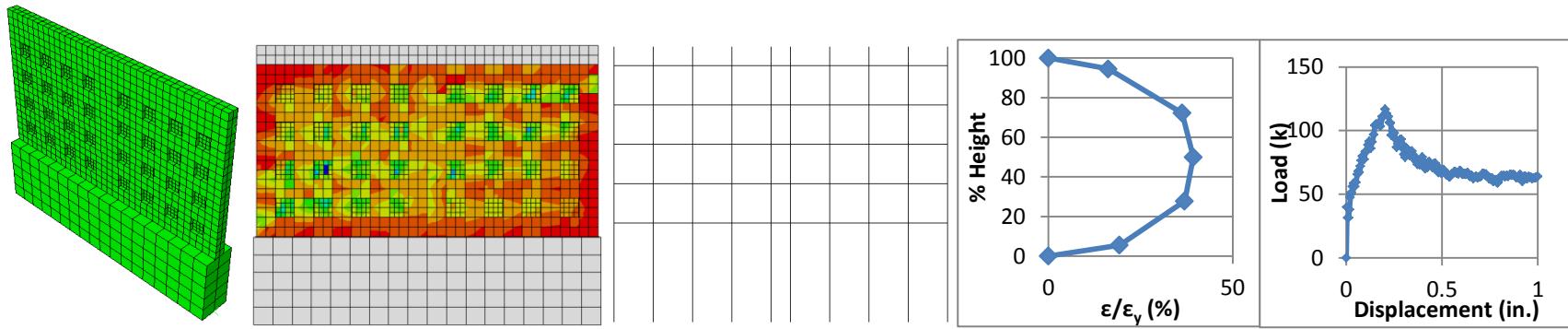
Wall #1—PG



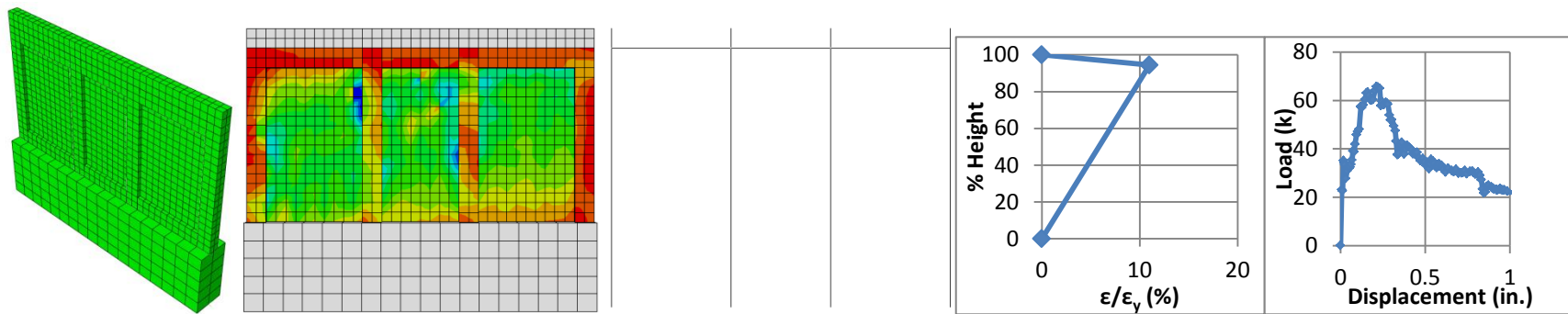
Wall #2--PG



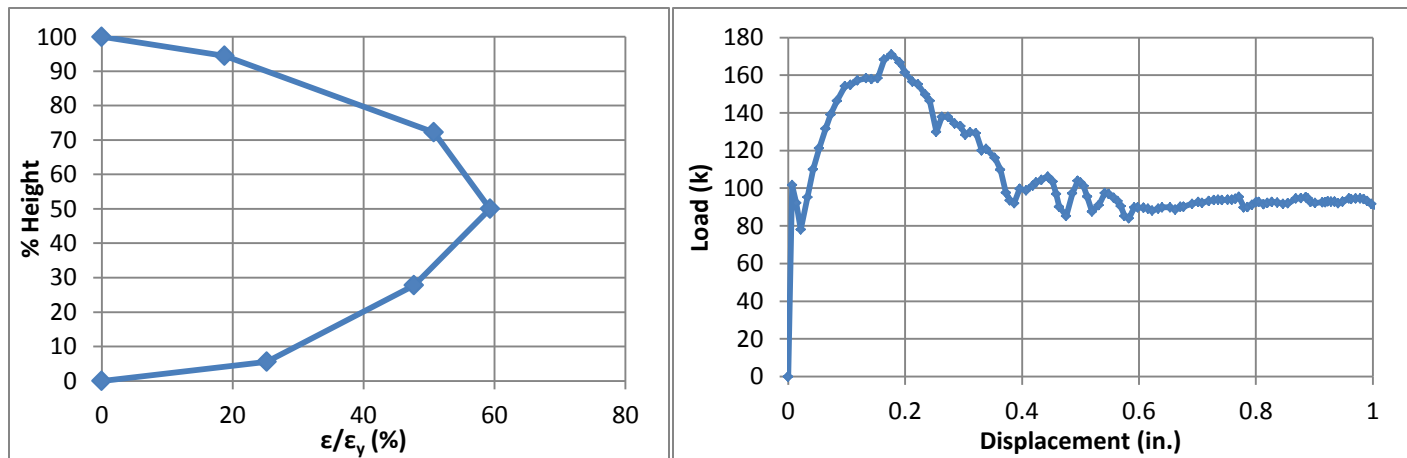
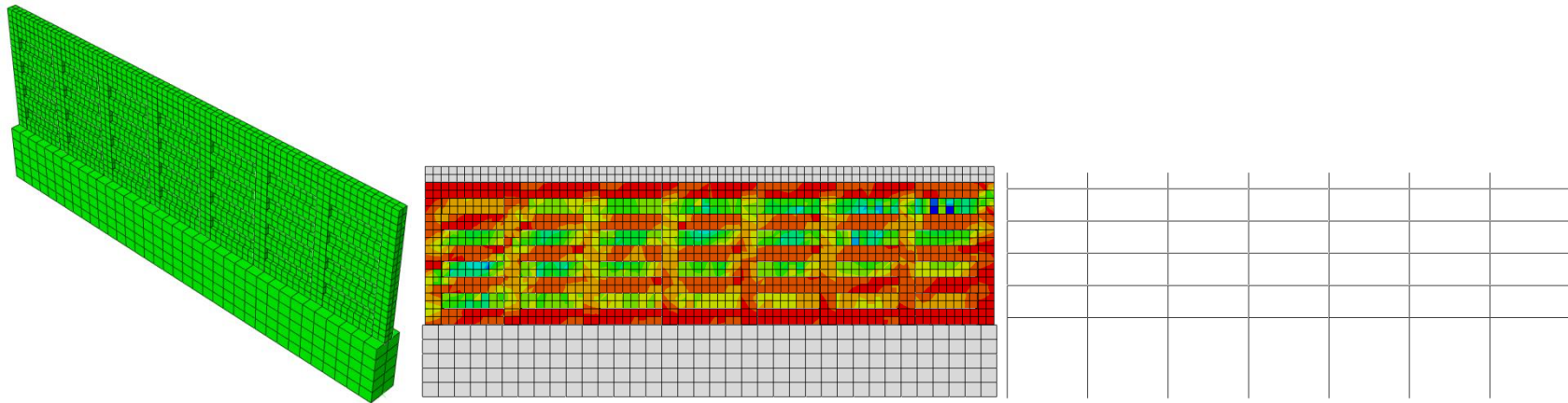
Wall #3--PG



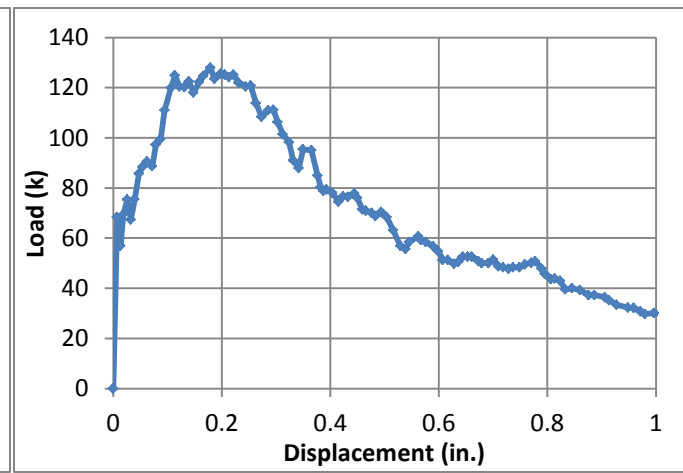
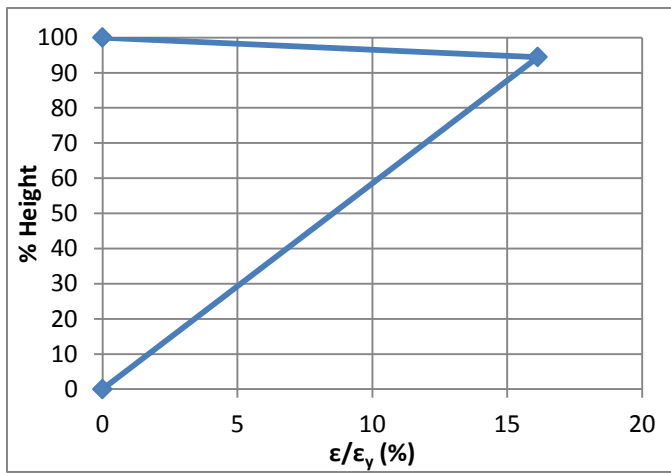
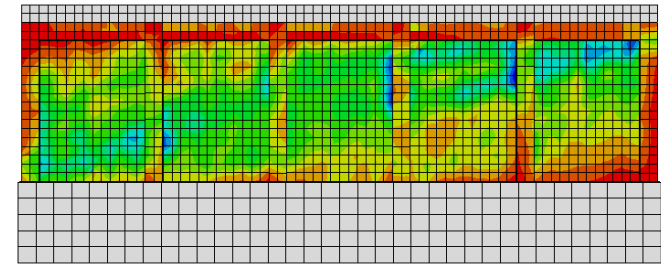
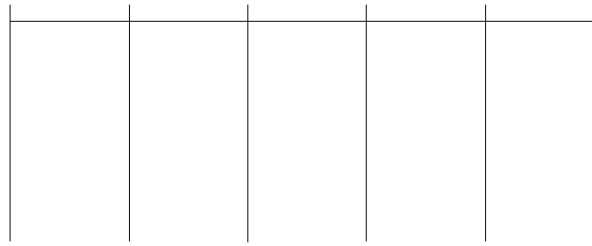
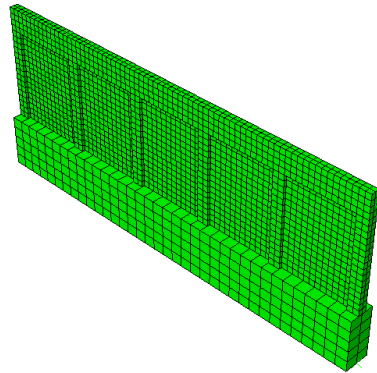
Wall #4--PG



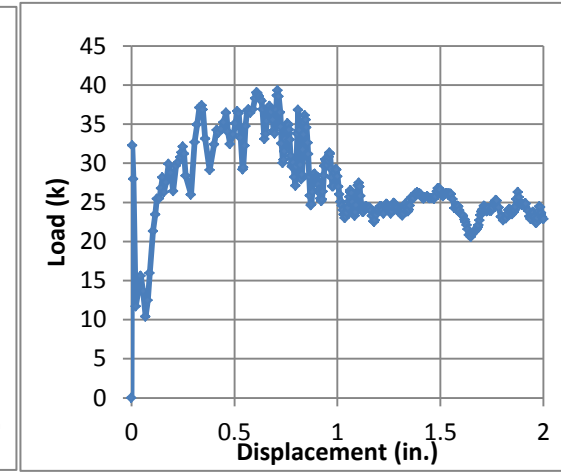
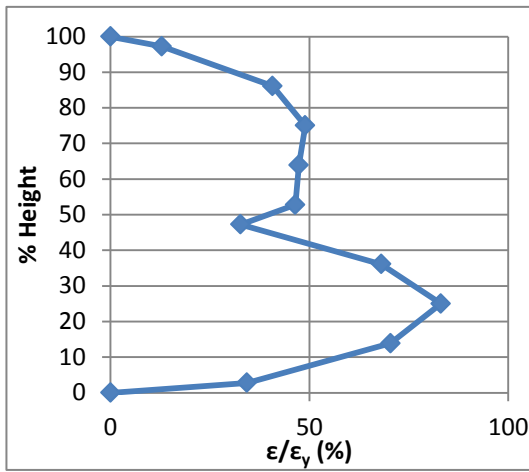
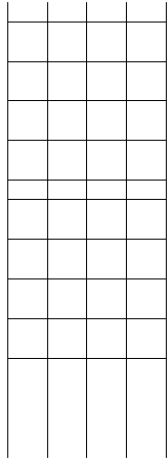
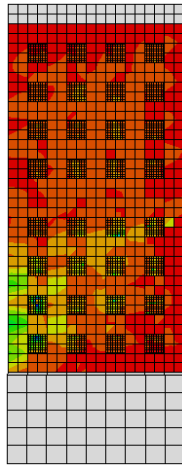
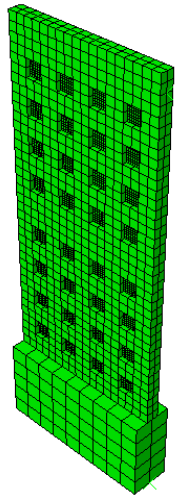
Wall #5--PG



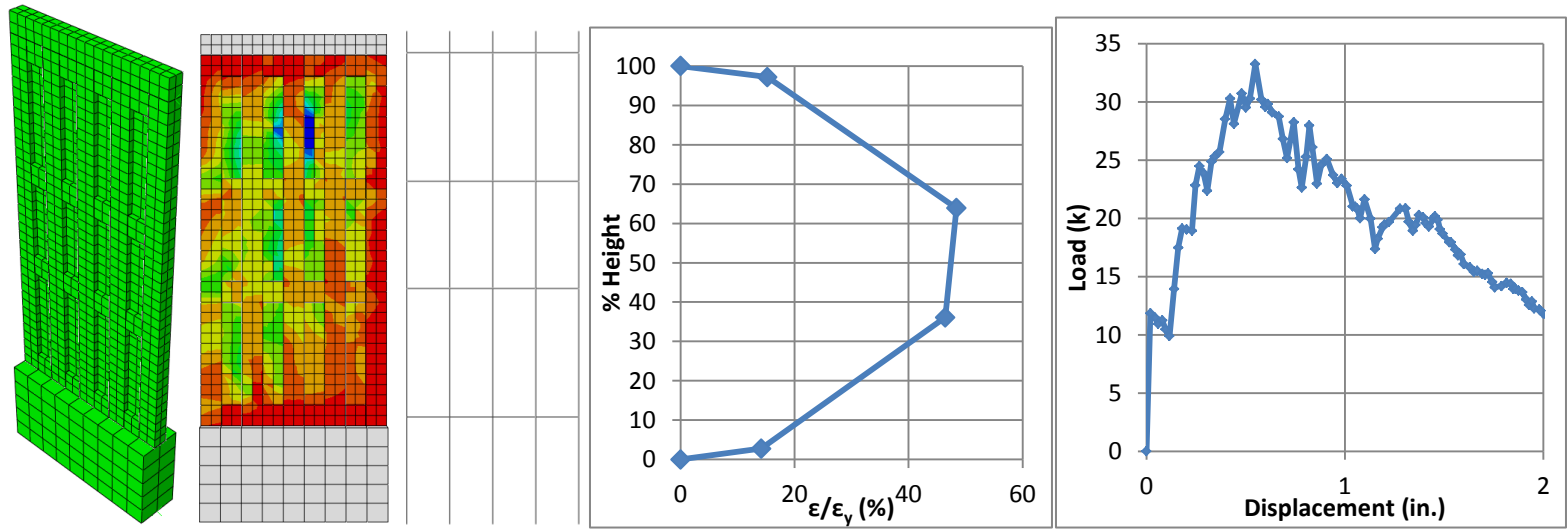
Wall #6--PG



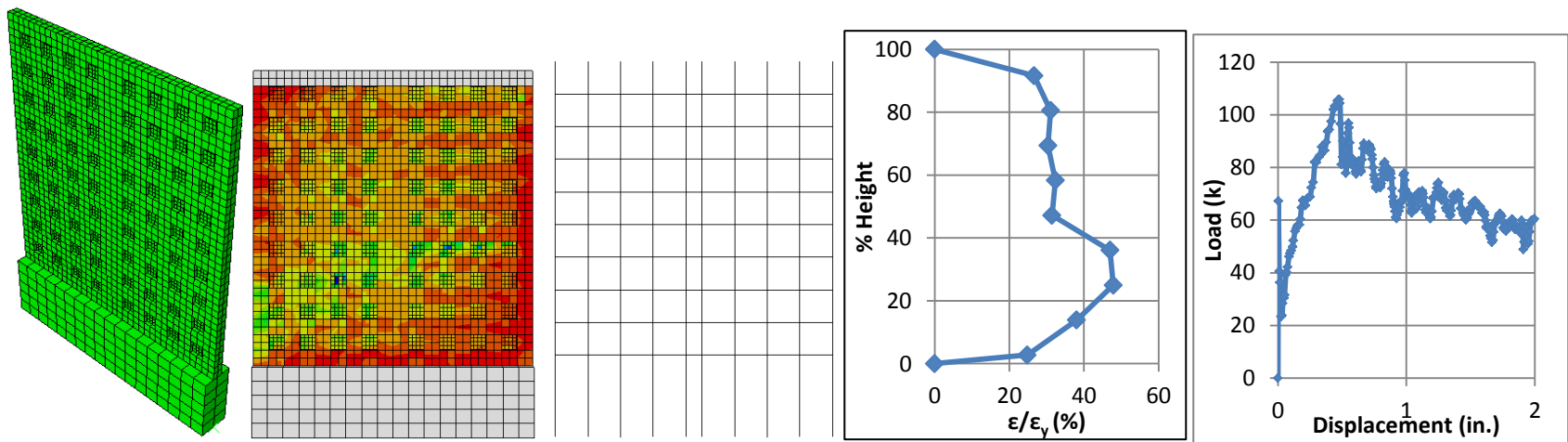
Wall #7--PG



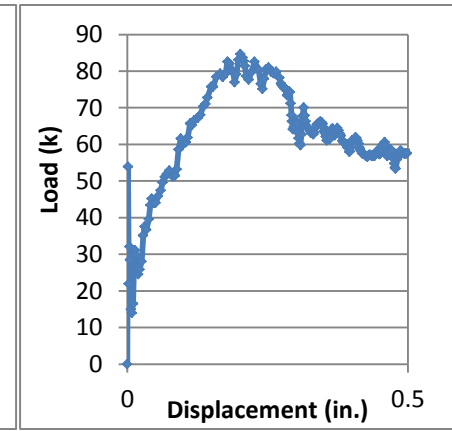
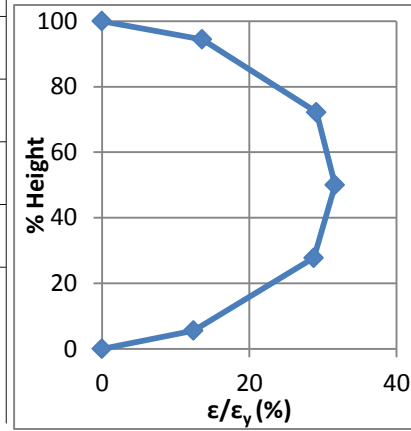
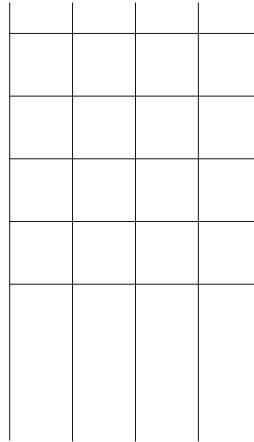
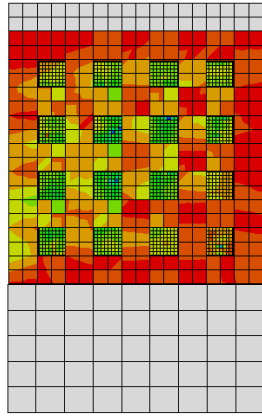
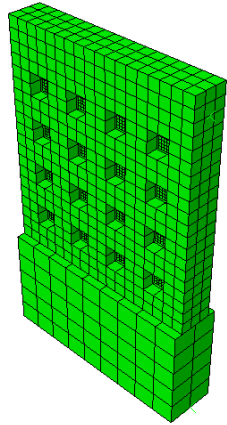
Wall #8--PG



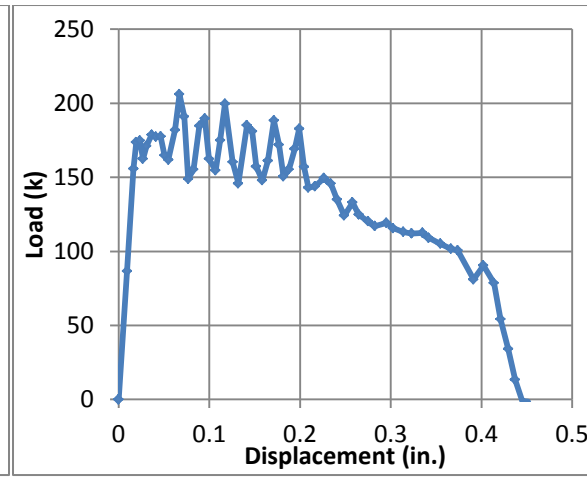
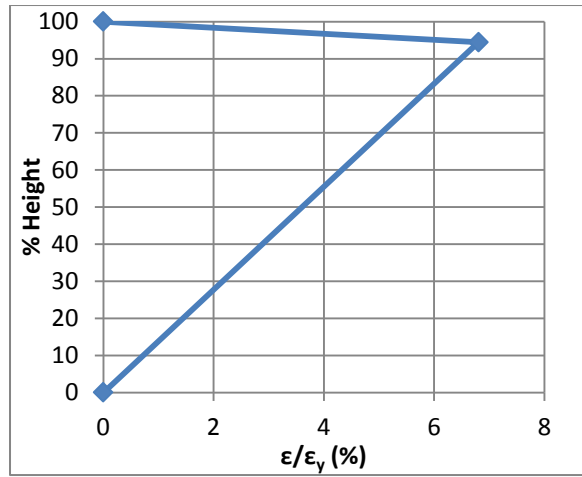
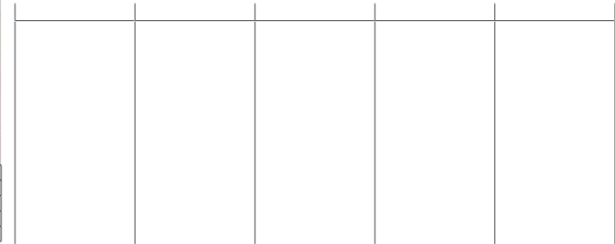
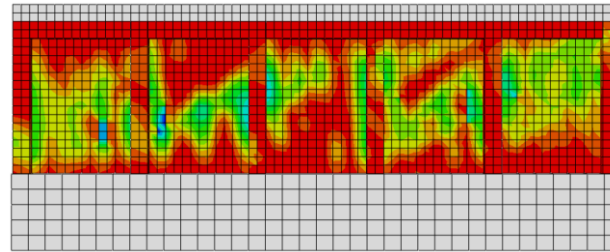
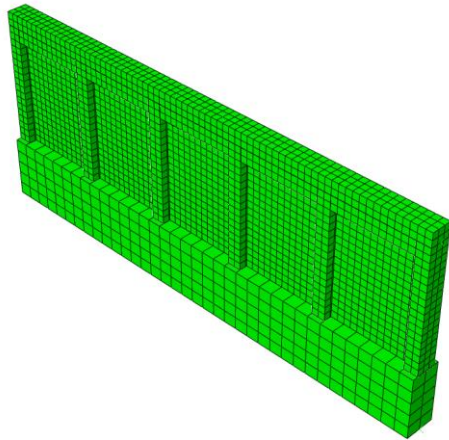
Wall #9—PG



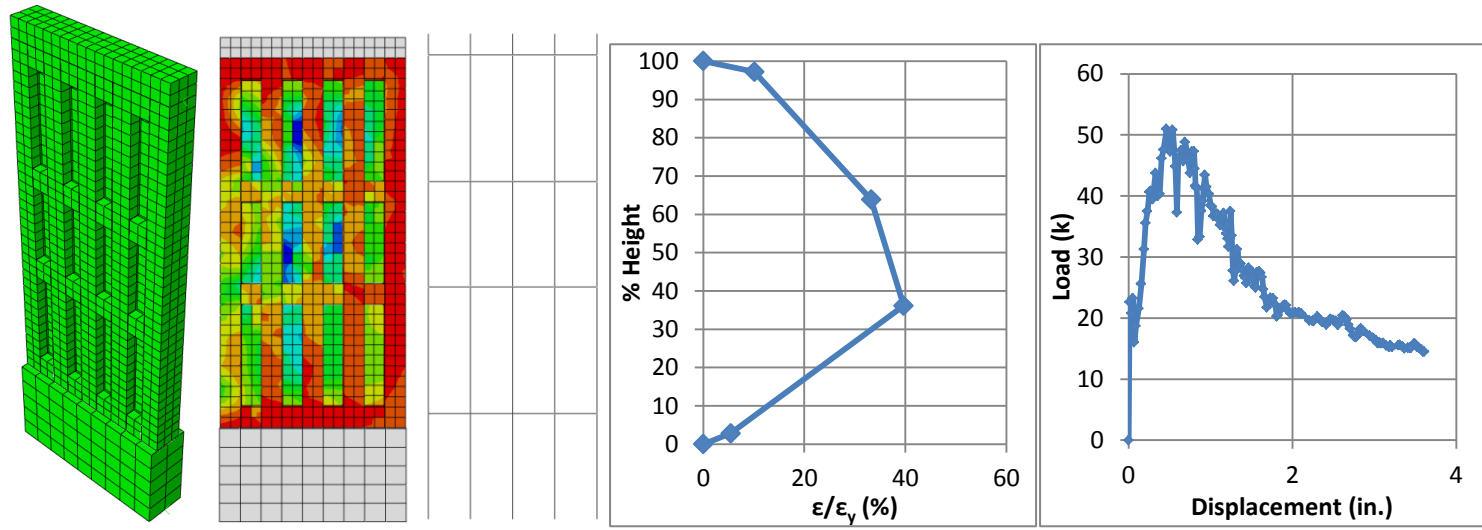
Wall #10—PG



Wall #11—PG



Wall #12—PG



Appendix E

Sliding Specimens

Table E-1—Wall Properties

Wall #	Researcher	Wall ID	Wall Size (in.)	Vertical Reinf.	# bars	ρ_v (%)	f_y (ksi)	Axial ³ Load (k)	Sliding Failure Type	Avg. ⁴ Vmax (k)
Concrete Block										
1	Koutalan	UT-PBS-05	72 x 72 x 8	#4 at 8in.	9	0.33	62.4	83.9	base sliding	139
2	Koutalan	UT-PBS-06	72 x 72 x 8	#4 at 16in.	5	0.18	62.4	71	base sliding	104
3	Koutalan	UT-PBS-09	72 x 72 x 8	#4 at 8in.	9	0.33	62.4	139.3	base sliding	184
4	Koutalan	UT-PBS-10	72 x 72 x 8	#4 at 16in.	5	0.18	62.4	139.3	base sliding	157
5	Shing	6	72 x 72 x 6	#5 at 16 in.	5	0.38	64	1.98	base sliding	49.5
6	Shing	8	72 x 72 x 6	#5 at 16in.	5	0.38	64	1.98	base sliding	48.5
7	Voon ¹	3	72 x 72 x 6	D20 at 16in.	5	0.6	46.1	1.98	base sliding	47
8	Hernandez ²	PBS-3	96 x 96 x 8	#4 at 8in.	12	0.33	61.4	4.8	base sliding	82
9	Hernandez ²	PBS-4	96 x 96 x 8	#4 at 16in.	6	0.16	61.4	4.8	base sliding	47
10	Hernandez ²	PBS-4G	96 x 96 x 8	#4 at 16in.	6	0.16	61.4	4.8	base sliding	42
11	Hidalgo	HCBL-12-4	40 x 80 x 8	#7 at 40 and 32in.	3	0.30	80.3	130.6	course ⁶ sliding	209.9
12	Hidalgo	HCBL-12-5	40 x 80 x 8	#7 at 40 and 32in.	3	0.30	80.3	132.1	course ⁶ sliding	220.2
Clay Brick										
13	Hidalgo	HCBR-12-1	40 x 78 x 8	#7 at 36in.	3	0.31	80.3	102.4	course ⁶ sliding	208.7
14	Hidalgo	HCBR-12-5	40 x 78 x 8	#7 at 36in.	3	0.31	80.3	101.9	course ⁶ sliding	223.8
15	Hidalgo	HCBR-12-6	40 x 78 x 8	#7 at 36in.	3	0.31	80.3	129.2	course ⁶ sliding	251.4

Table E-1 Continued

Wall #	Researcher	Wall ID	Wall Size (in.)	Vertical Reinf.	# bars	ρ_v (%)	f_y (ksi)	Axial ³ Load (k)	Sliding Failure Type	Avg. ⁴ Vmax (k)
16	Hidalgo ⁵	CBRC-12-1	40 x 78 x 10	#7 at 36in.	3	0.23	80.3	85.1	course ⁶ sliding	190.4
17	Hidalgo ⁵	CBRC-12-2	40 x 78 x 10	#7 at 36in.	3	0.23	80.3	100.1	course ⁶ sliding	186.3
18	Hidalgo ⁵	CBRC-12-3	40 x 78 x 10	#7 at 36in.	3	0.23	80.3	118.3	course ⁶ sliding	207.9
19	Hidalgo ⁵	CBRC-12-4	40 x 78 x 10	#7 at 36in.	3	0.23	80.3	97.3	course ⁶ sliding	227.1
20	Hidalgo ⁵	CBRC-12-5	40 x 78 x 10	#7 at 36in.	3	0.23	80.3	111	course ⁶ sliding	183

1—here, D20 signifies a bar 20 mm in diameter

2—Specimens are designated as failing in sliding shear, but this was a small contributor to failure (see Hernandez, 2012)

3—Axial load includes an assumed wall weight of 55 psf

4—Average failure load is given here for both loading directions as reported by the various referenced researchers

5—These specimens are double-wythe with 4 in. clay brick and a 3 in. grout space

6—Walls tested by Hidalgo et al (1979) were constructed with flanges top and bottom consisting of one course of masonry with shear keys. Therefore, it was determined that sliding failure had to occur between a course of masonry

Table E-2—Wall Analysis

Wall #	Researcher	Wall ID	Avg. ³ V _{max}	Eqn. 1 μ	V _{pred} (Eqn. 1 μ)	$\frac{V_{exp}}{V_{pred}}$ (Eqn. 1 μ)	V _{pred} Eq. 3	$\frac{V_{exp}}{V_{pred}}$ Eq. 3
Base Sliding								
1	Koutalan	UT-PBS-05	139	0.72	119	1.17	117	1.19
2	Koutalan	UT-PBS-06	104	0.79	81	1.29	84	1.23
3	Koutalan	UT-PBS-09	184	0.74	152	1.21	161	1.14
4	Koutalan	UT-PBS-10	157	0.78	123	1.28	139	1.13
5	Shing	6	49.5	0.49	61	0.81	46	1.08
6	Shing	8	48.5	0.48	61	0.79	46	1.06
7	Voon	3	47	0.41	70	0.67	52	0.90
8	Hernandez	PBS-3	82	0.54	93	0.88	70	1.17
9	Hernandez	PBS-4	47	0.61	47	1.00	36	1.30
10	Hernandez	PBS-4G	42	0.55	47	0.90	36	1.16
Average				0.61		1.00		1.14
COV				0.22		0.22		0.10
Course Sliding								
11	Hidalgo	HCBL-12-4	209.9	0.76	227	0.92	170	1.24
12	Hidalgo	HCBL-12-5	220.2	0.80	229	0.96	171	1.29
13	Hidalgo	HCBR-12-1	208.7	0.85	204	1.02	147	1.42
14	Hidalgo	HCBR-12-5	223.8	0.91	204	1.10	147	1.53
15	Hidalgo	HCBR-12-6	251.4	0.92	226	1.11	168	1.49
16	Hidalgo	CBRC-12-1	190.4	0.83	190	1.00	133	1.43
17	Hidalgo	CBRC-12-2	186.3	0.76	202	0.92	145	1.28
18	Hidalgo	CBRC-12-3	207.9	0.79	217	0.96	160	1.30

Table E-2 Continued

Wall #	Researcher	Wall ID	Avg. ³ Vmax	Eqn. 1 μ	V_{pred} (Eqn. 1 μ)	$\frac{V_{exp}}{V_{pred}}$ (Eqn. 1 μ)	V_{pred} Eq. 3	$\frac{V_{exp}}{V_{pred}}$ Eq. 3
19	Hidalgo	CBRC-12-4	227.1	0.94	200	1.14	143	1.59
20	Hidalgo	CBRC-12-5	183	0.72	211	0.87	154	1.19
Average				0.83		1.00		1.38
COV				0.09		0.09		0.10

Appendix F

Visual Analysis Strut-and-Tie Layouts

The ensuing message from Visual Analysis if the model became unstable:

Analysis Diagnostics

PROBLEM:

Analysis not complete - stiffness matrix is not positive definite.

The model is not stable.

SOLUTION:

This may happen during a p-delta analysis when compressive forces in plates or members are excessive. You should increase member sizes or reduce compressive loading.

The User's Guide (Help) has more trouble-shooting tips for analysis problems!

PROBLEM:

There is a possibility that your structure is unstable. It may have rigid body modes. The maximum stiffness diagonal is 8987.72 and occurred at N005 X-Translation. The minimum stiffness diagonal is 1506.55 and occurred at N001 Y-Translation. The minimum diagonal term may indicate a location of instability. The ratio of maximum to minimum diagonal is 5.96575. You might also want to analyze for mode shapes and look at the dynamic mode shapes of very high frequency for the rigid body mode.

SOLUTION:

If you are modelling a plane of plate elements this problem may be solved by constraining a single node's rotation about an axis perpendicular to the plane.

This problem might also be solved by using fewer pinned nodes and member releases. First try making all member end connections rigid which removes the releases.

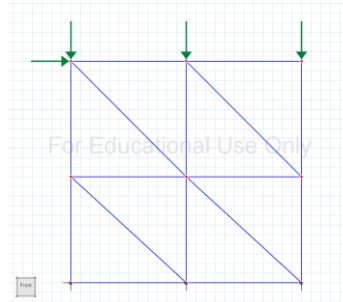
Then if analysis works, slowly put the releases back in until you come across the mechanism that is causing the instability.

If this process does not work, provide more nodal supports until you have removed the mechanism.

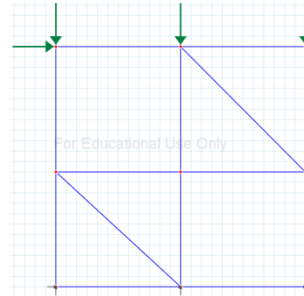
Figure F-1 Partially Grouted CMU Walls

Partially Grouted CMU Walls:

Nolph PG085-48 and PG120-48:

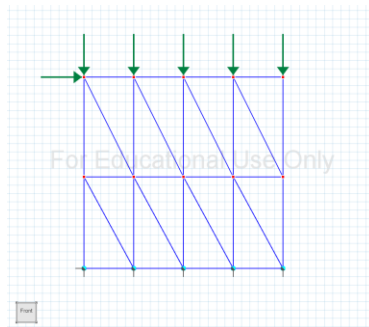


(a) Initial

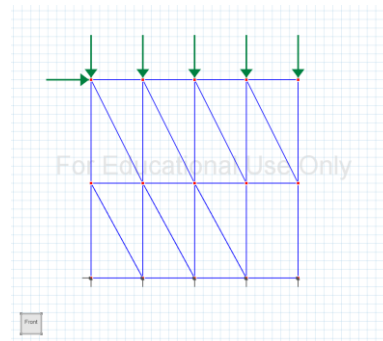


(b) Final

Nolph PG085-24:

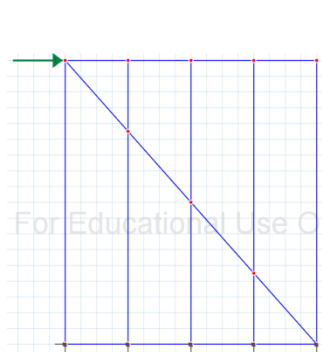


(a) Initial

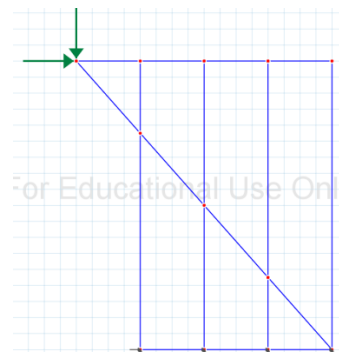


(b) Final

Voon A5:

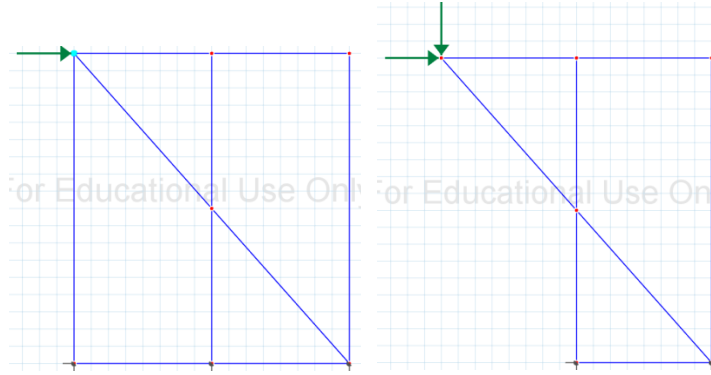


(a) Initial



(b) Final

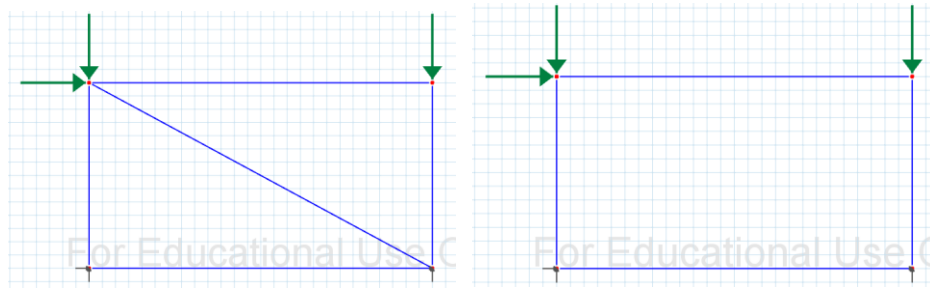
Voon A6:



(a) Initial

(b) Final

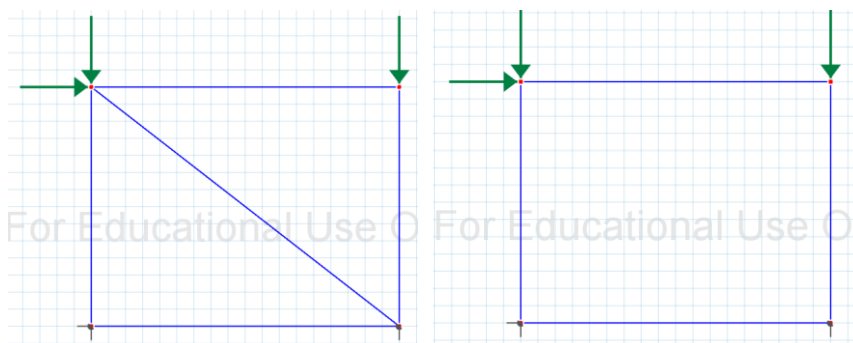
Schultz 2:



(a) Initial

(b) Final

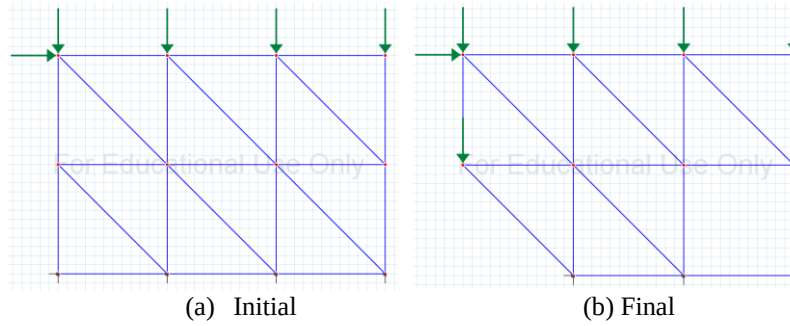
Schultz 4:



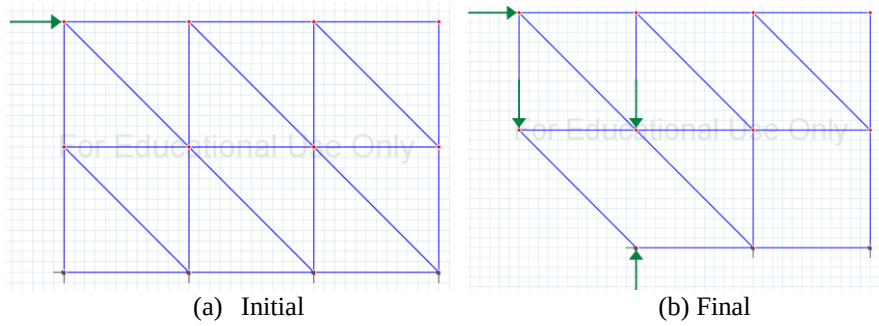
(a) Initial

(b) Final

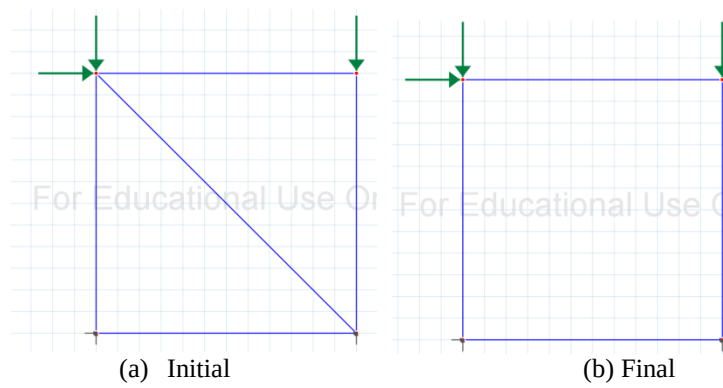
Minaie PCL 1:



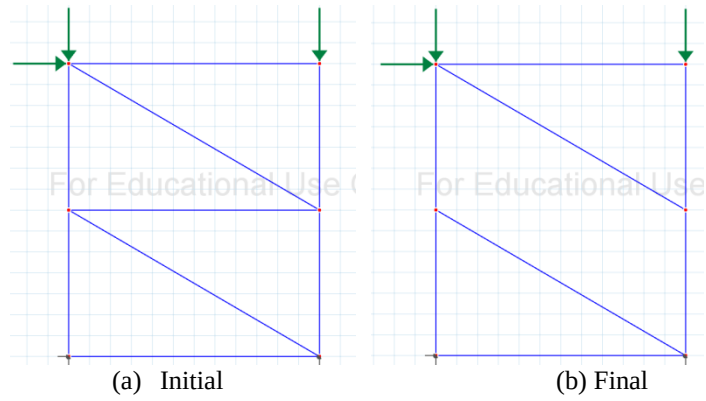
Minaie MC 2:



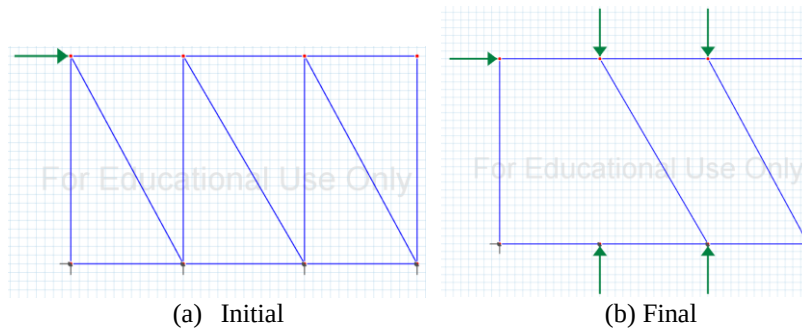
Yancey—R2:



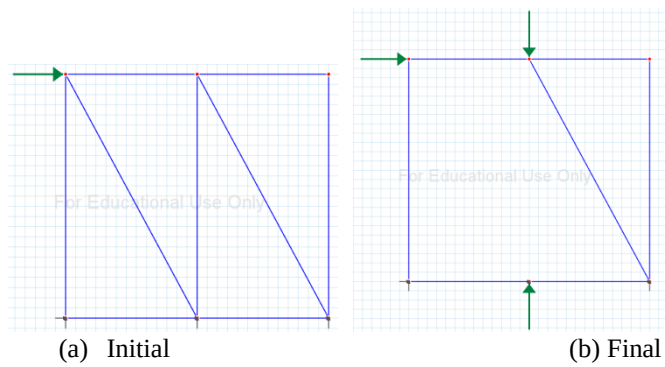
Yancey—R5:



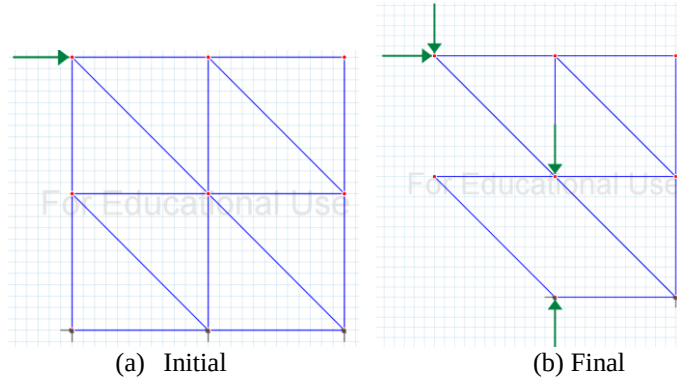
Baenzinger and Porter 4:



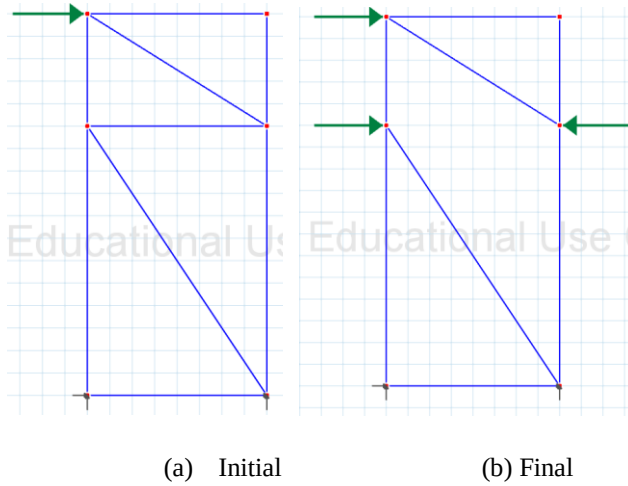
Baenzinger and Porter 6:



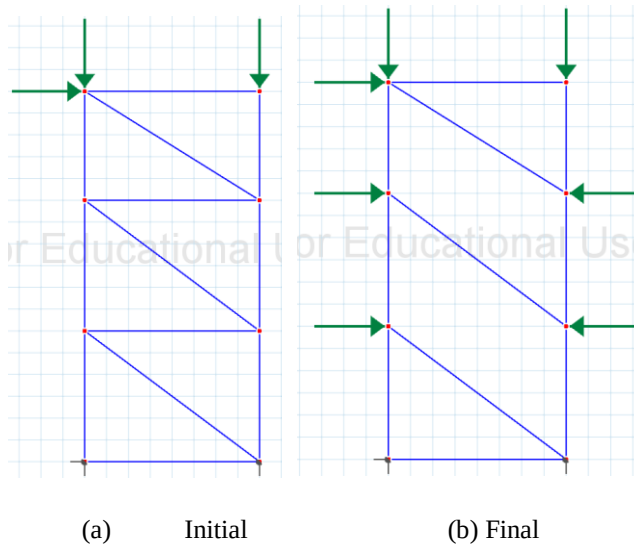
Baenzinger and Porter 7:



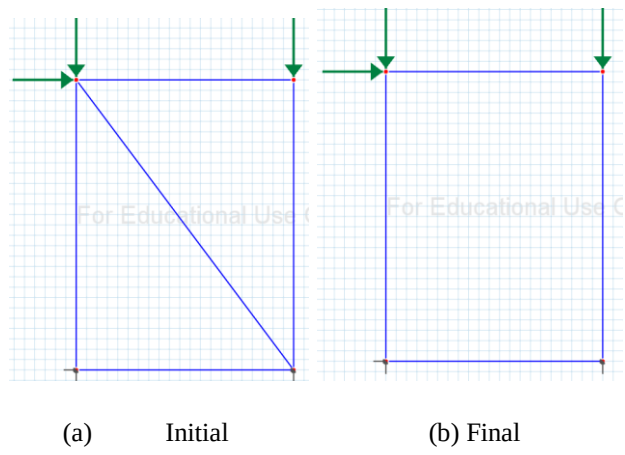
Matsumura CW2-1-1:



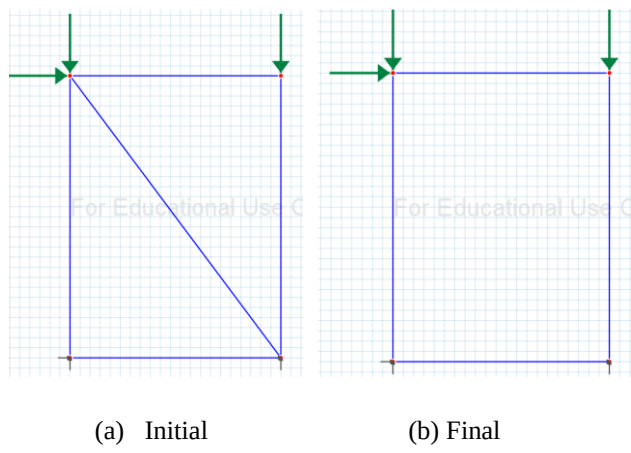
Matsumura CW2-2'-A2-1:



Chen HCBL-5:



Chen HCBL-8:



Chen HCBL-10:

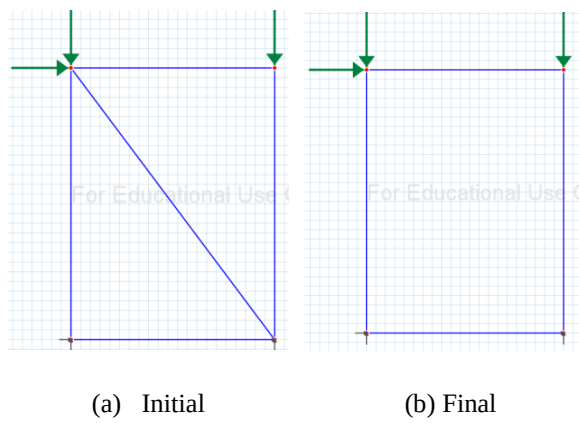
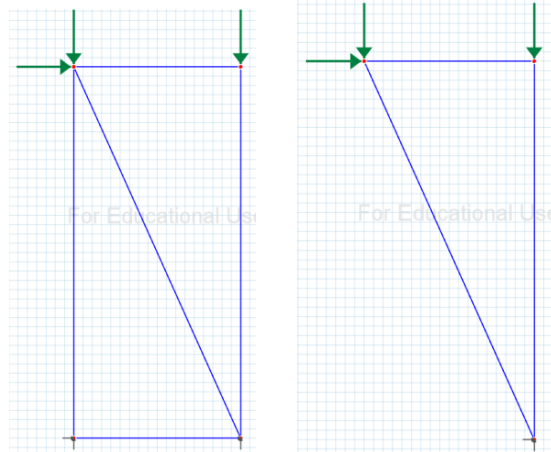


Figure F-2 Partially Grouted Clay Walls

Partially Grouted Clay Walls:

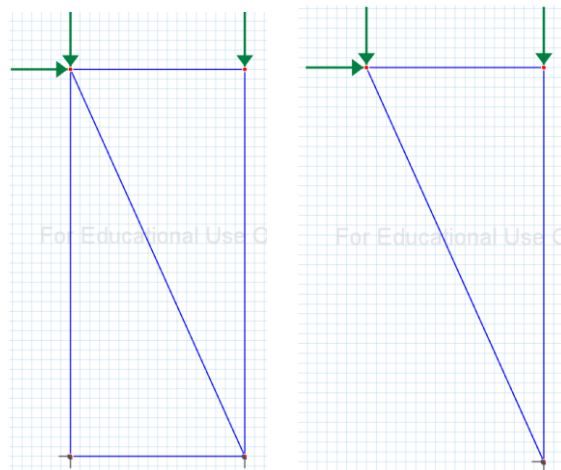
Hidalgo HCBR 21-3:



(a) Initial

(b) Final

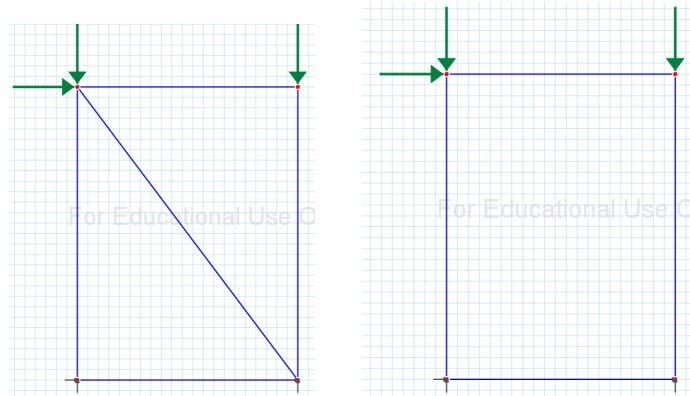
Hidalgo HCBR 21-5:



(a) Initial

(b) Final

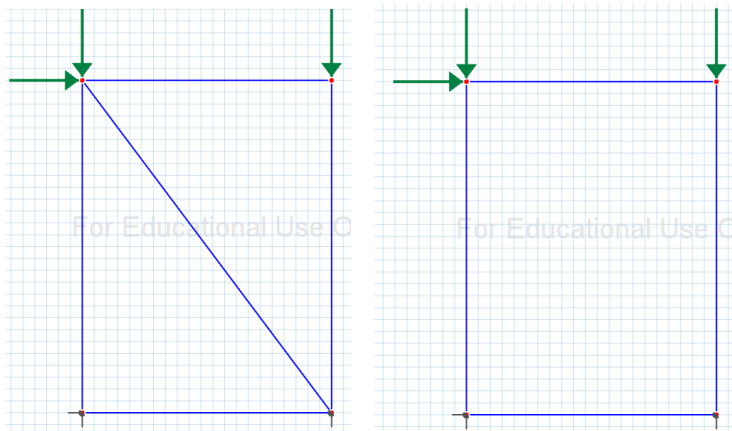
Chen HCBR 11-5:



(a) Initial

(b) Final

Chen HCBR 11-9:



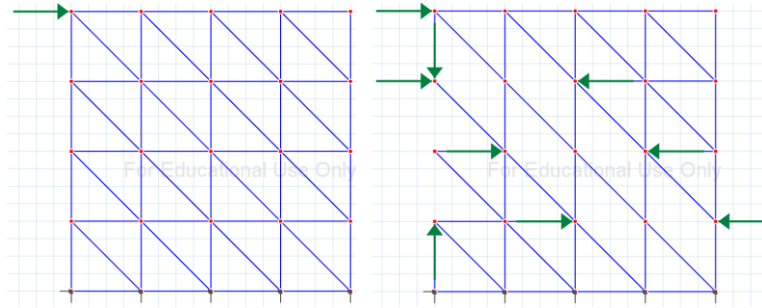
(a) Initial

(b) Final

Figure F-3 Fully Grouted CMU Walls

Fully Grouted CMU Walls:

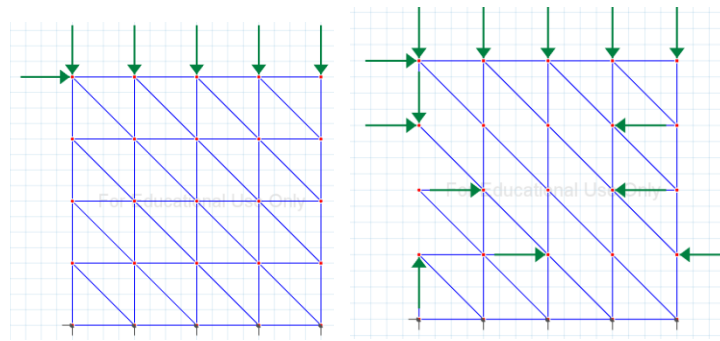
Shing 4:



(a) Initial

(b) Final

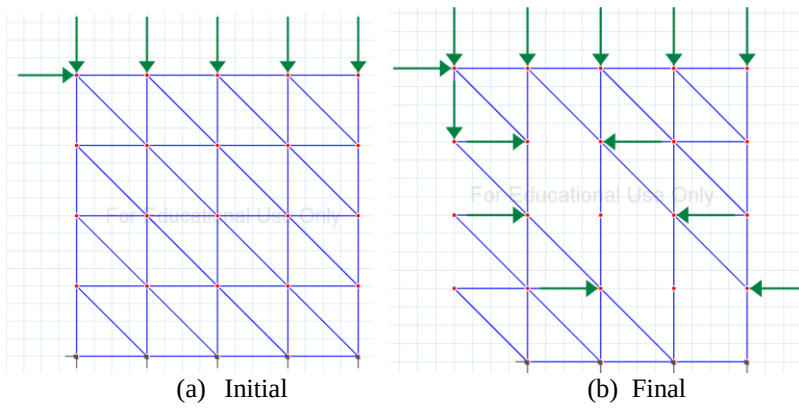
Shing 9:



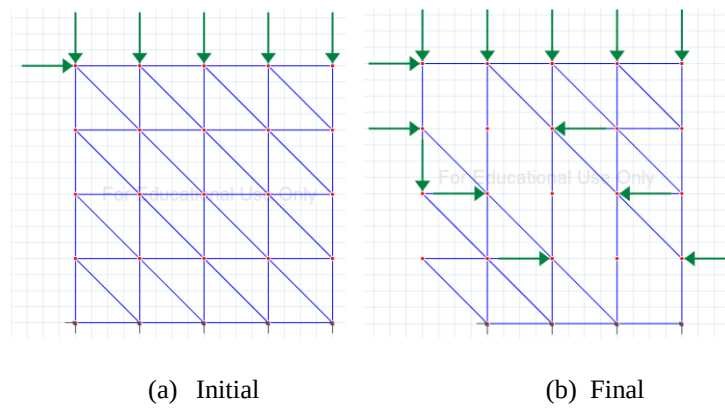
(a) Initial

(b) Final

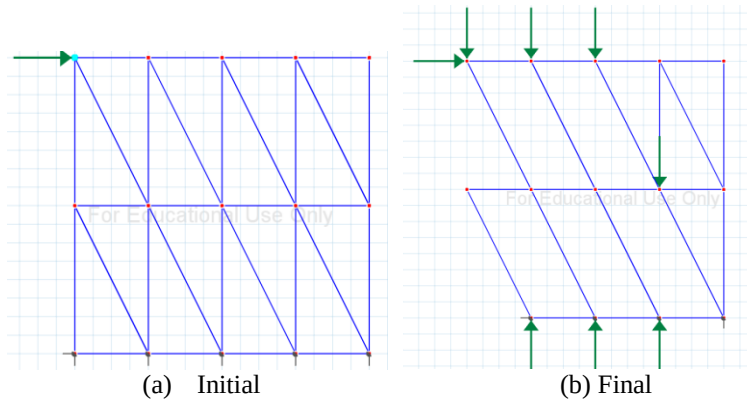
Shing 13:



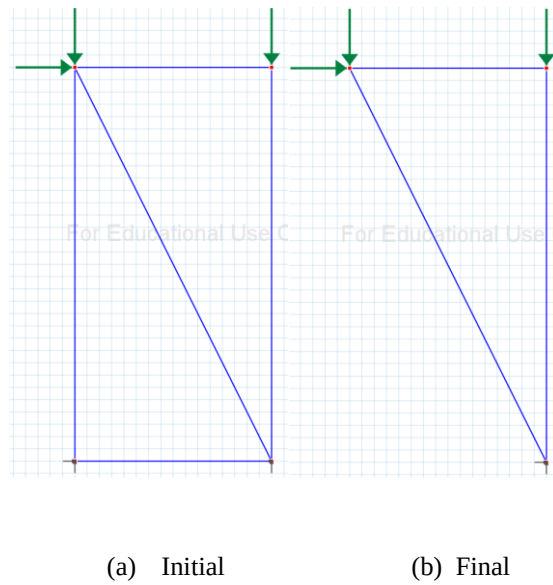
Shing 16:



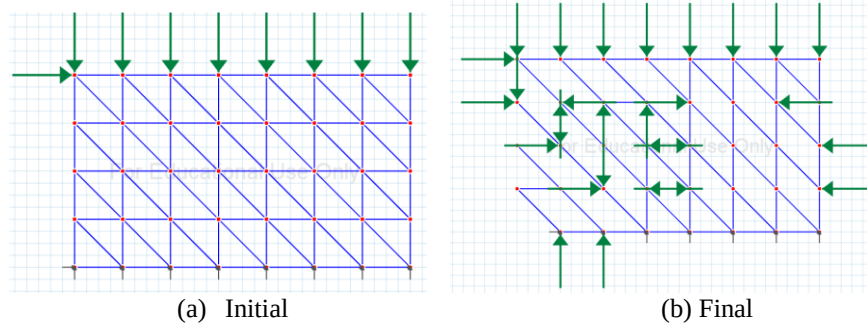
Voon A4:



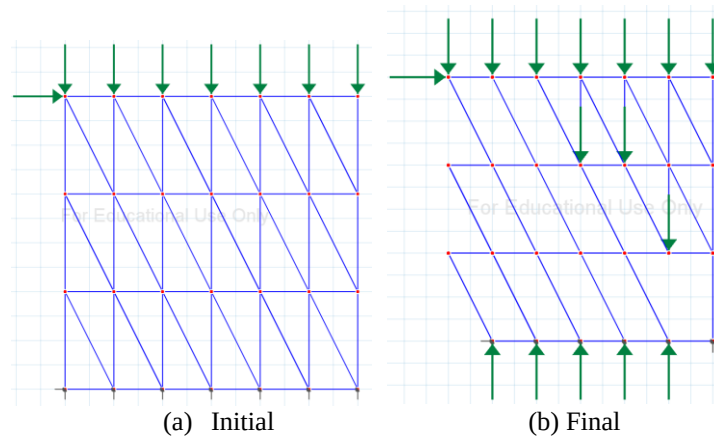
Voon A9:



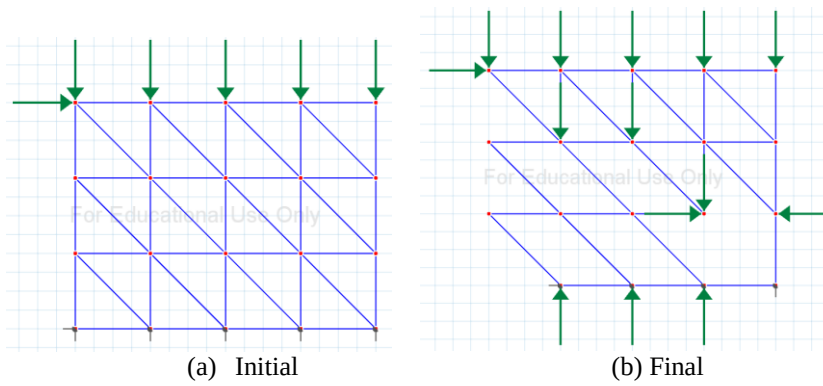
Voon A10:



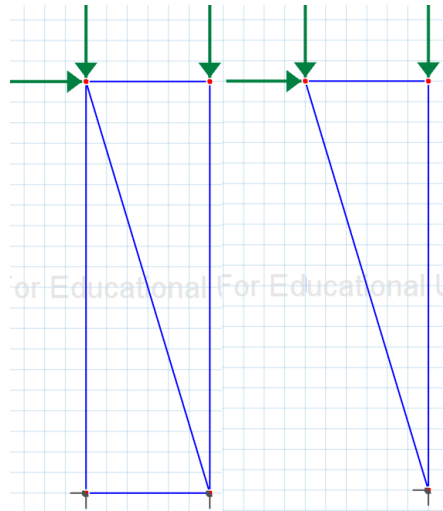
Eikanas 4:



Eikanas 7:



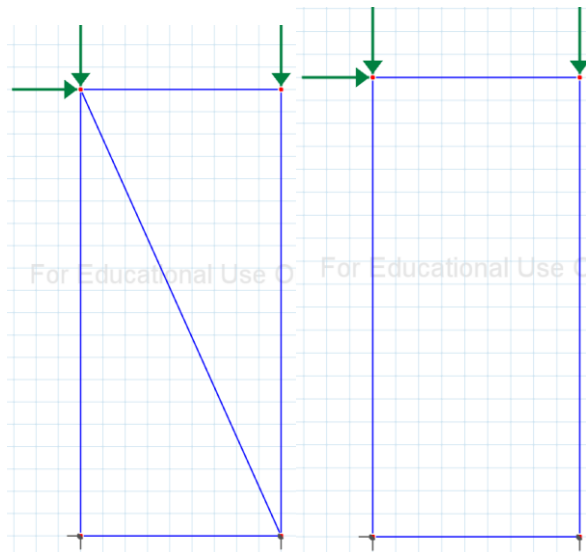
Matsumura KW2-1:



(a) Initial

(b) Final

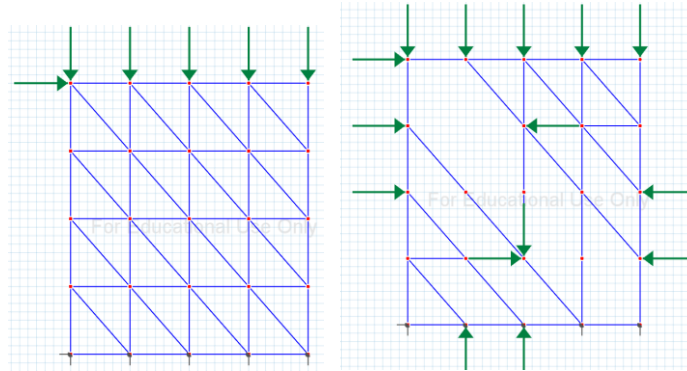
Matsumura KW3-1 and KW3S-1:



(a) Initial

(b) Final

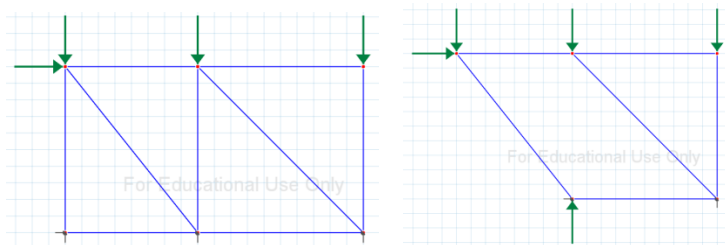
Matsumura KW4-1:



(a) Initial

(b) Final

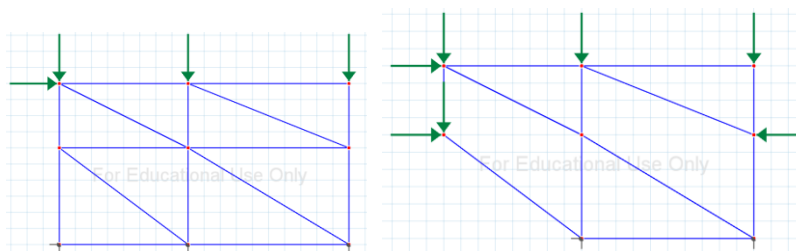
Hidalgo HCBL 12-1:



(a) Initial

(b) Final

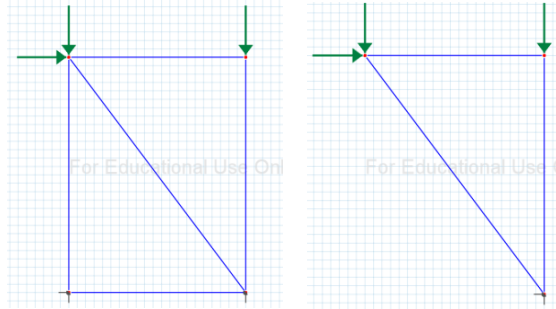
Hidalgo HCBL 12-6:



(a) Initial

(b) Final

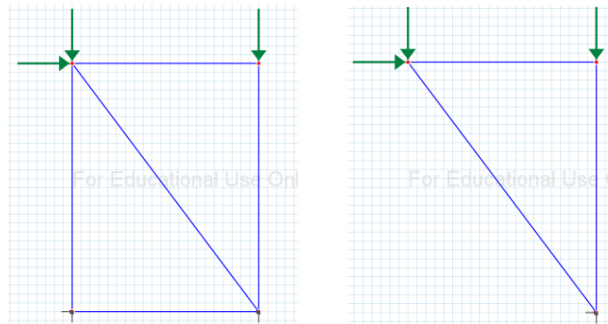
Chen HCBL 11-3:



(a) Initial

(b) Final

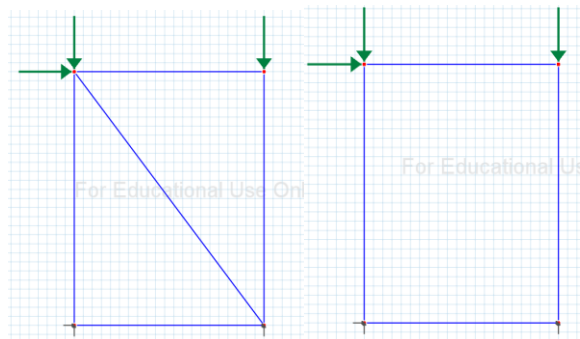
Chen HCBL 11-4:



(a) Initial

(b) Final

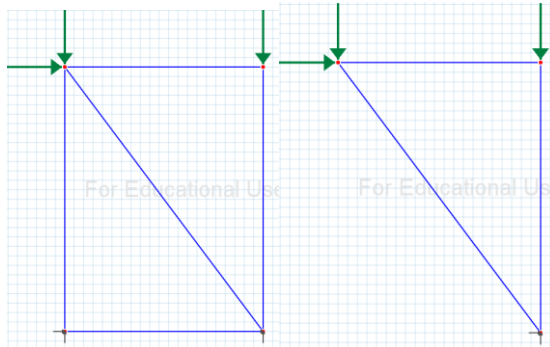
Chen HCBL 11-9:



(a) Initial

(b) Final

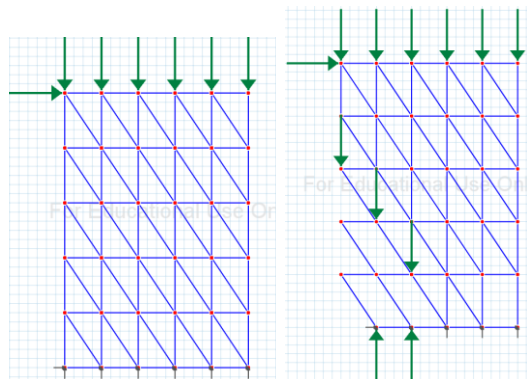
Sveinsson HCBL 11-13:



(a) Initial

(b) Final

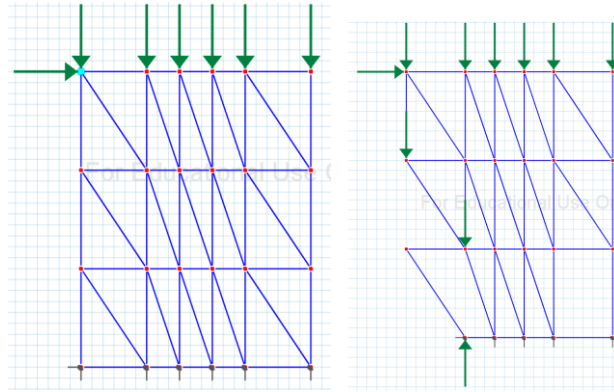
Sveinsson HCBL 11-18:



(a) Initial

(b) Final

Sveinsson HCBL 11-21:



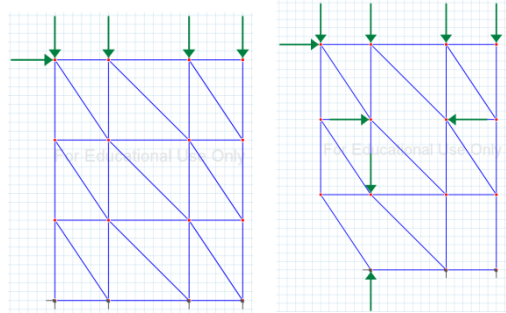
(a) Initial

(b) Final

Figure F-4 Fully Grouted Clay Walls

Fully Grouted Clay Walls:

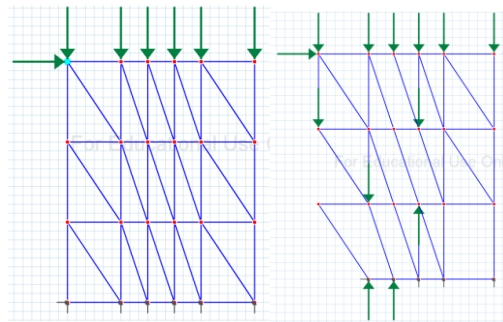
Sveinsson HCBR 11-21:



(a) Initial

(b) Final

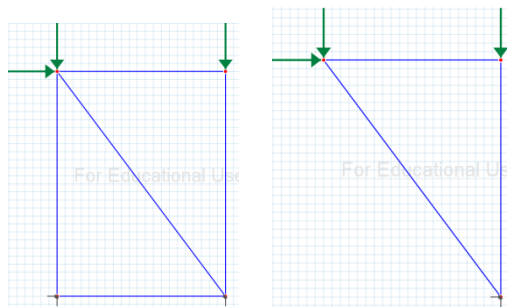
Sveinsson HCBR 11-23:



(a) Initial

(b) Final

Sveinsson CBRC 11-9:



(a) Initial

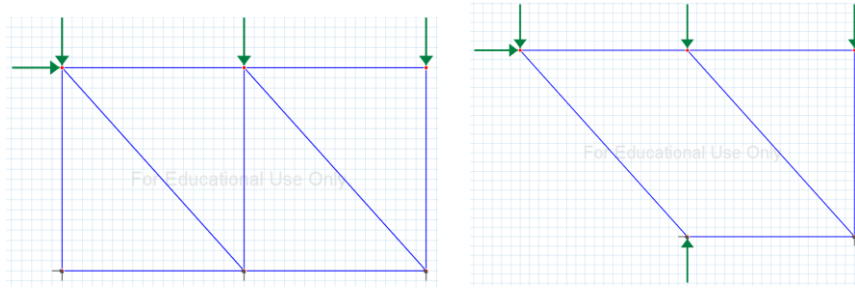
(b) Final

(b) Final

(b) Final

(b) Final

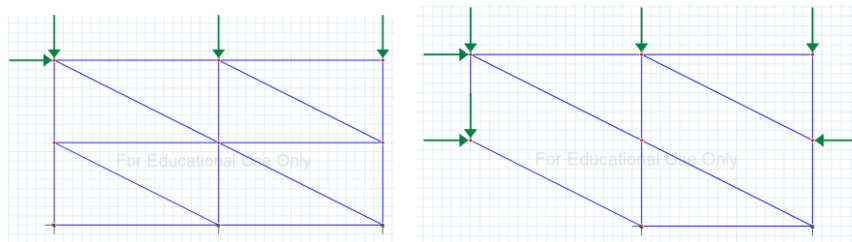
Hidalgo HCBR 12-3:



(a) Initial

(b) Final

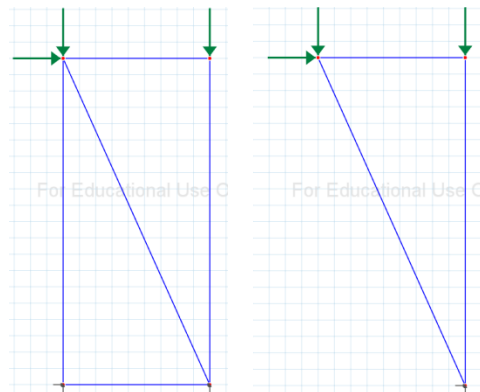
Hidalgo CBRC 12-6:



(a) Initial

(b) Final

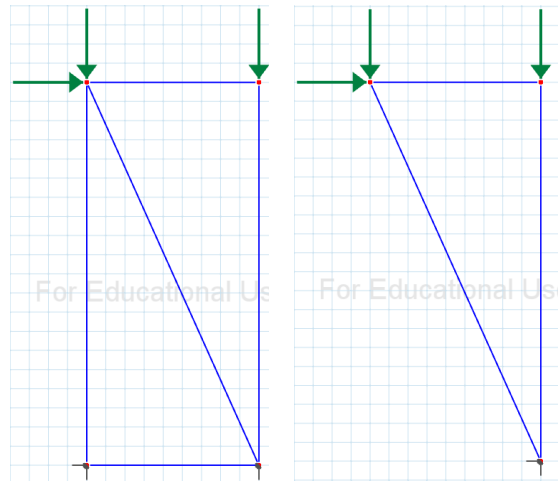
Hidalgo HCBR 21-2:



(a) Initial

(b) Final

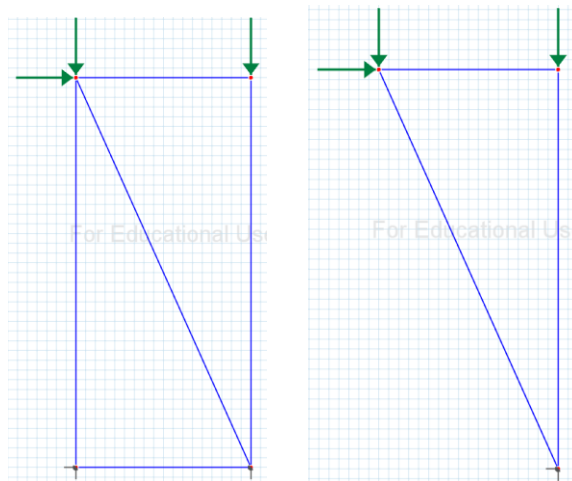
Hidalgo HCBR 21-6:



(a) Initial

(b) Final

Hidalgo CBRC 21-2:



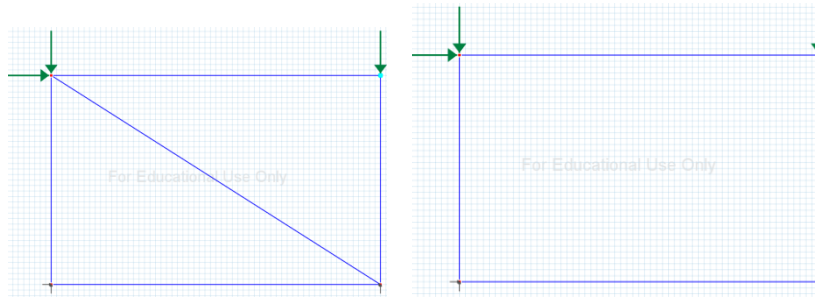
(a) Initial

(b) Final

Figure F-5 Fully Grouted AAC Walls

Fully Grouted AAC Walls:

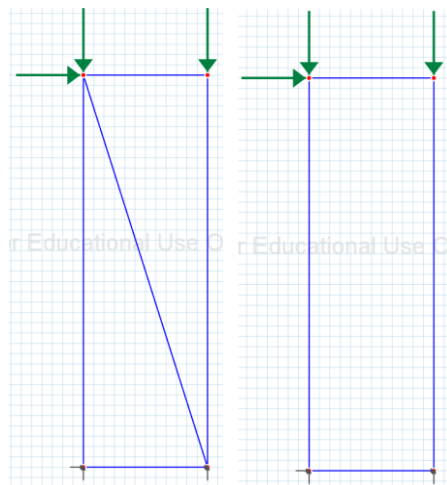
Tanner 3 and 5:



(a) Initial

(b) Final

Tanner 11:



(a) Initial

(b) Final

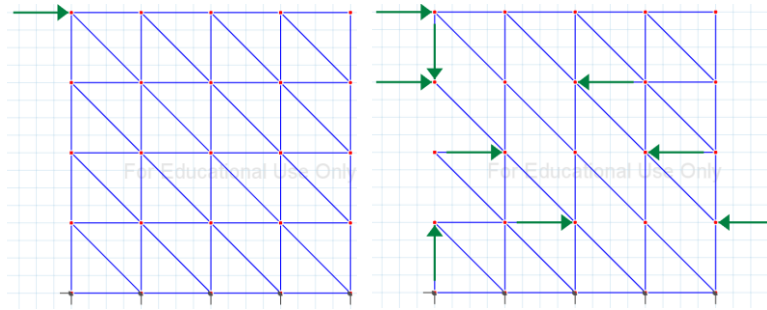
Appendix G

Finite Element Analysis Walls

Figure G-1 Fully Grouted Walls

Fully Grouted Walls:

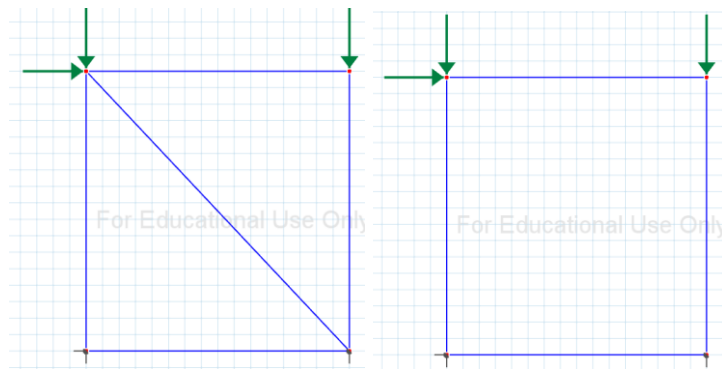
#1-FG:



(a) Initial

(b) Final

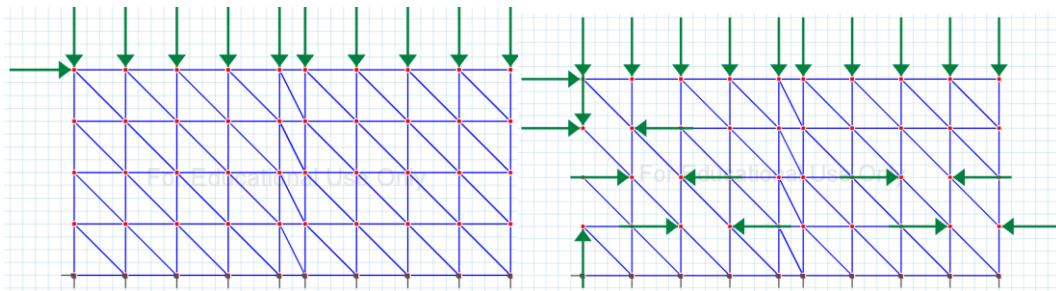
#2-FG:



(a) Initial

(b) Final

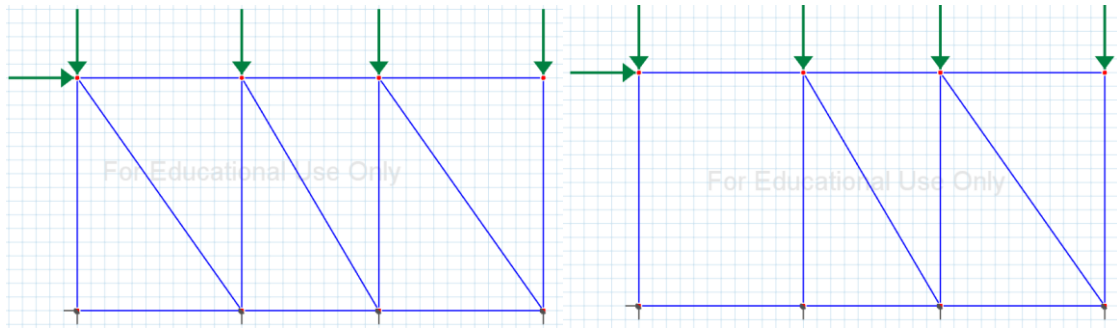
#3-FG:



(a) Initial

(b) Final

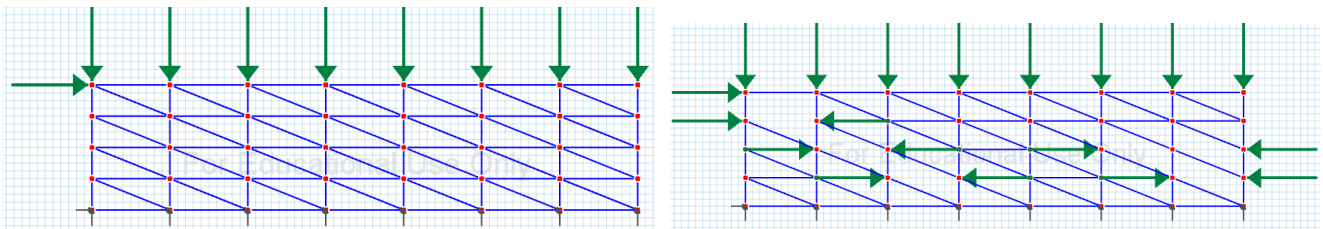
#4-FG:



(a) Initial

(b) Final

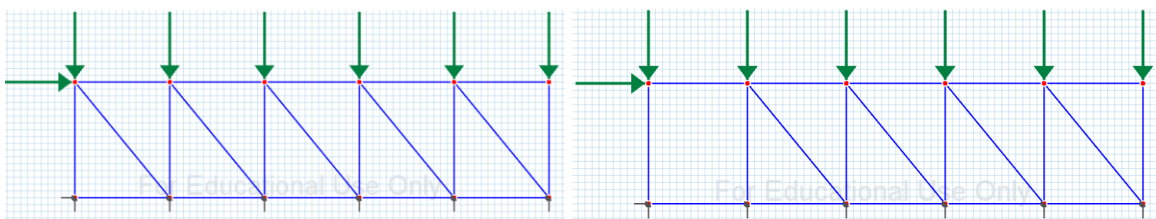
#5-FG:



(a) Initial

(b) Final

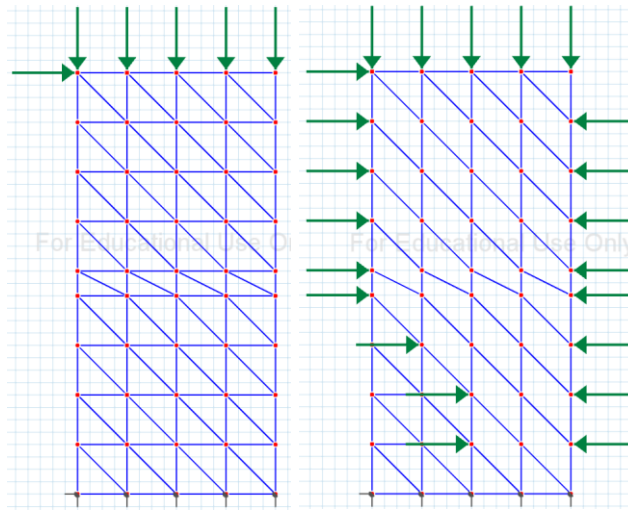
#6-FG:



(a) Initial

(b) Final

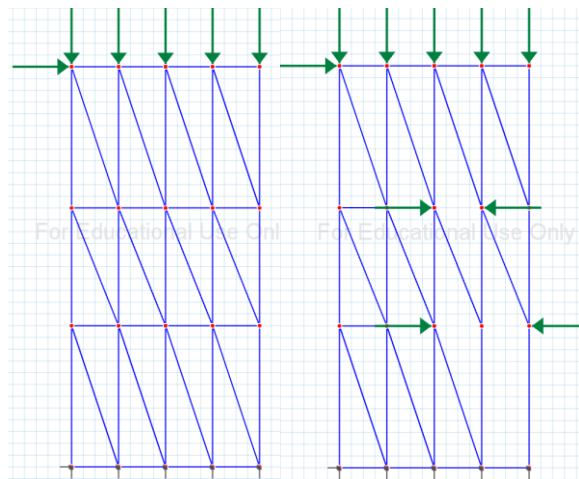
#7-FG:



(a) Initial

(b) Final

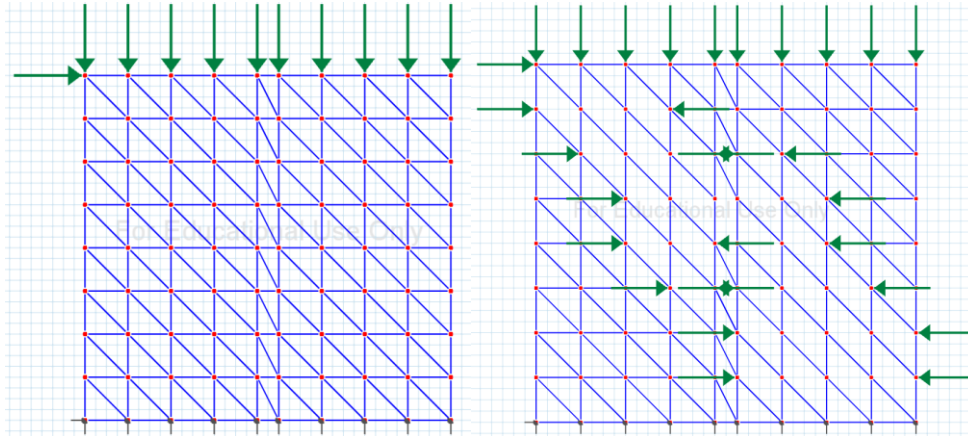
#8-FG:



(a) Initial

(b) Final

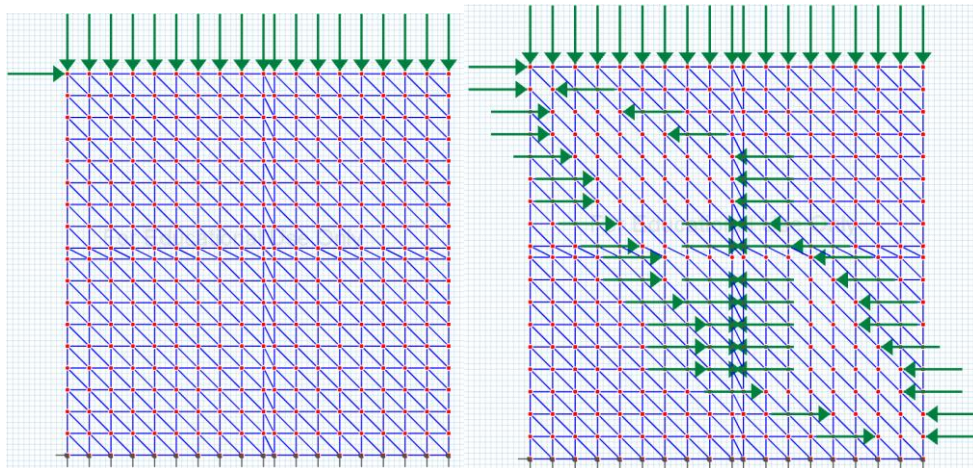
#11-FG:



(a) Initial

(b) Final

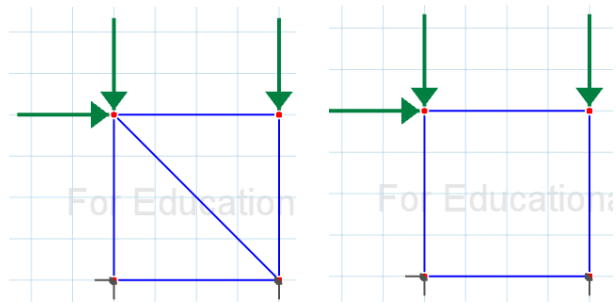
#12-FG:



(a) Initial

(b) Final

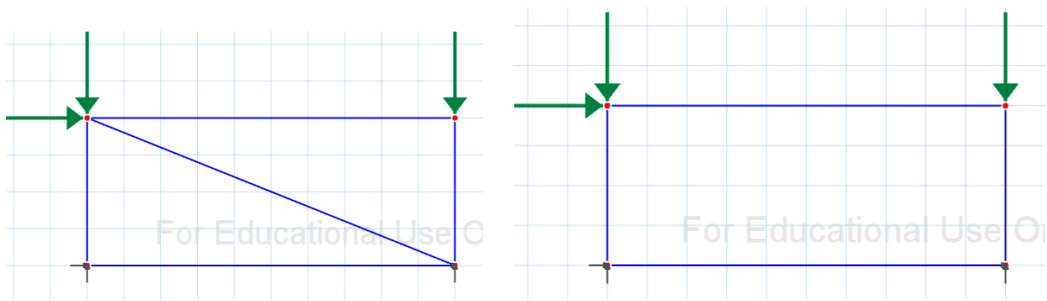
#13-FG:



(a) Initial

(b) Final

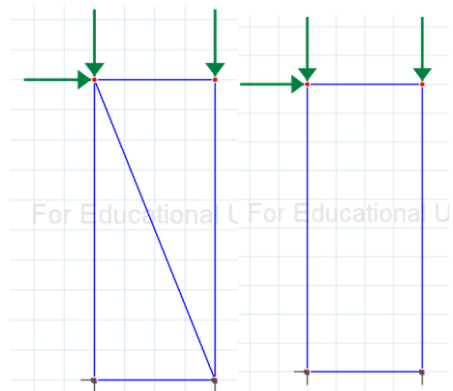
#14-FG:



(a) Initial

(b) Final

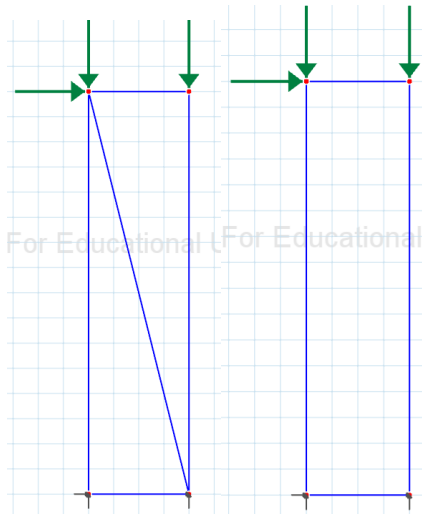
#15-FG:



(a) Initial

(b) Final

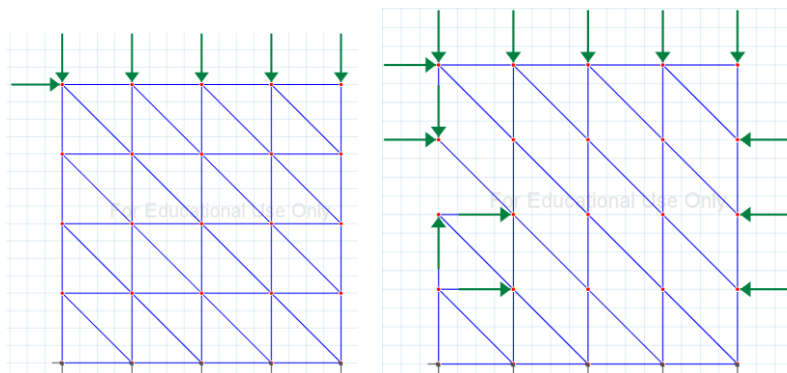
#16-FG:



(a) Initial

(b) Final

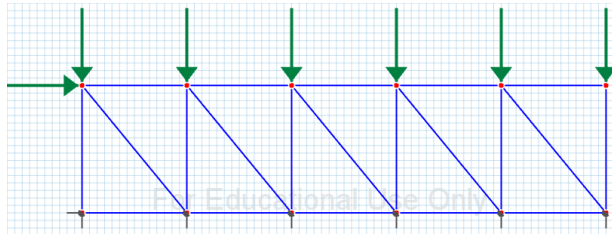
#17-FG:



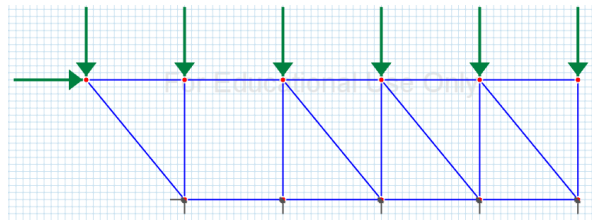
(a) Initial

(b) Final

#18-FG:

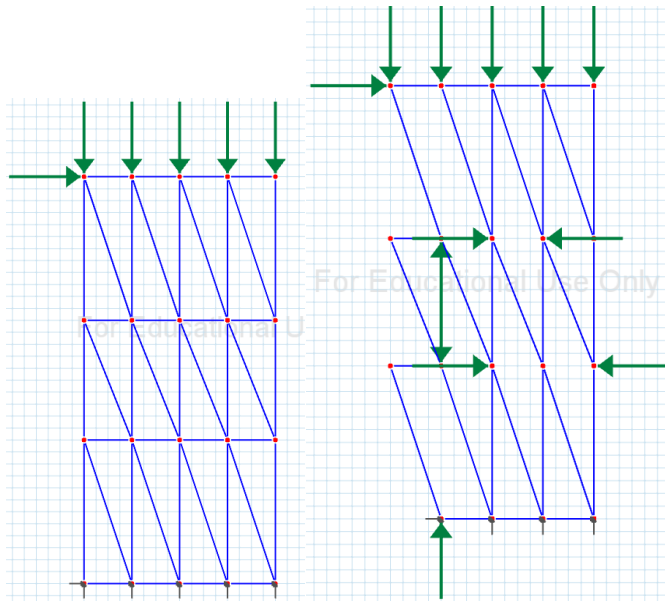


(a) Initial

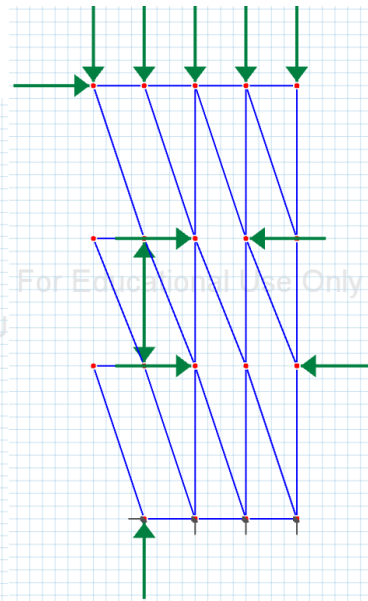


(b) Final

#19-FG:



(a) Initial

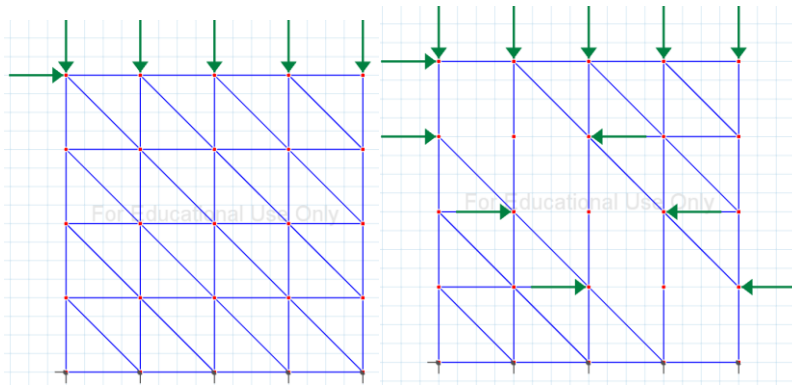


(b) Final

Figure G-2 Partially Grouted Walls

Partially Grouted Walls:

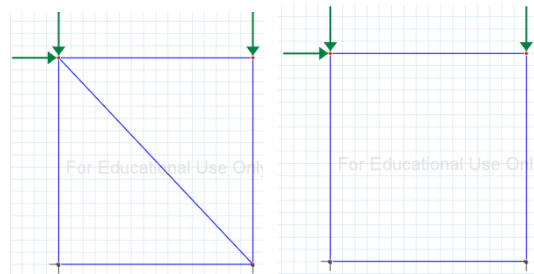
#1-PG:



(a) Initial

(b) Final

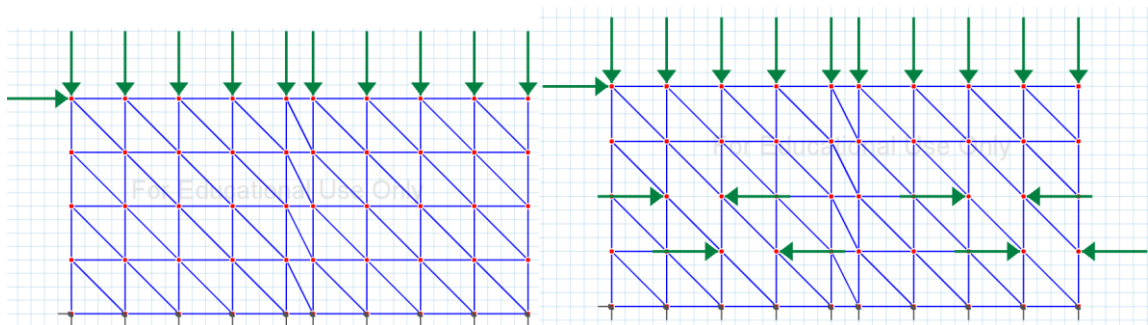
#2-PG:



(a) Initial

(b) Final

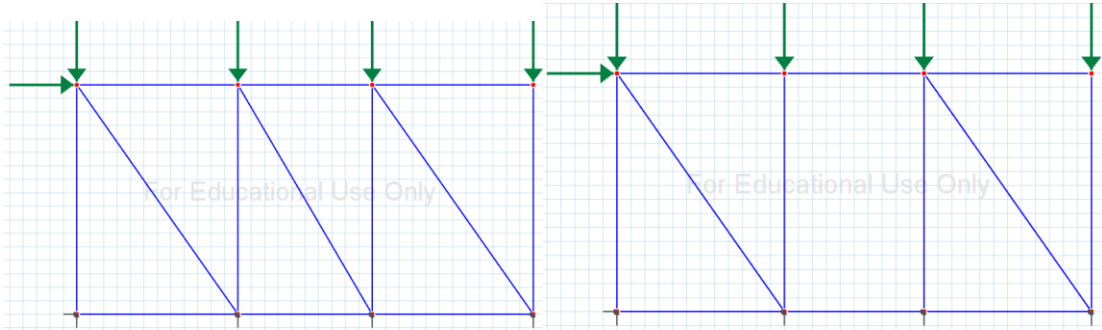
#3-PG:



(a) Initial

(b) Final

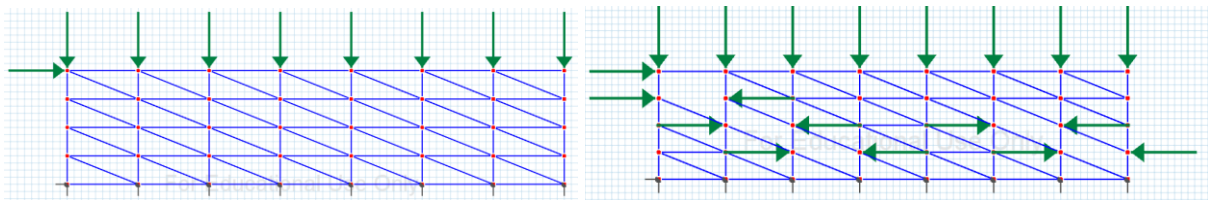
#4-PG:



(a) Initial

(b) Final

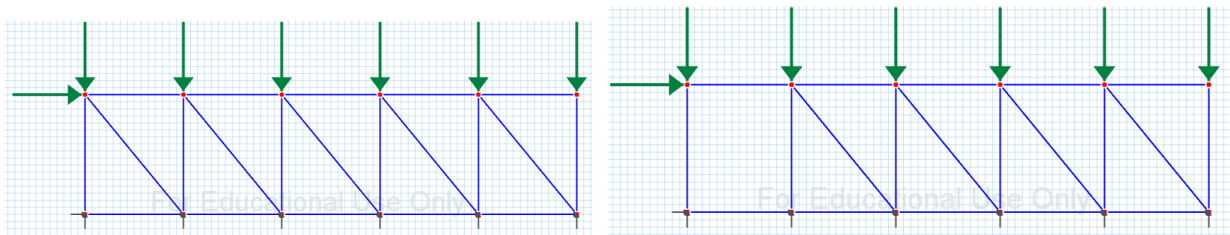
#5-PG:



(a) Initial

(b) Final

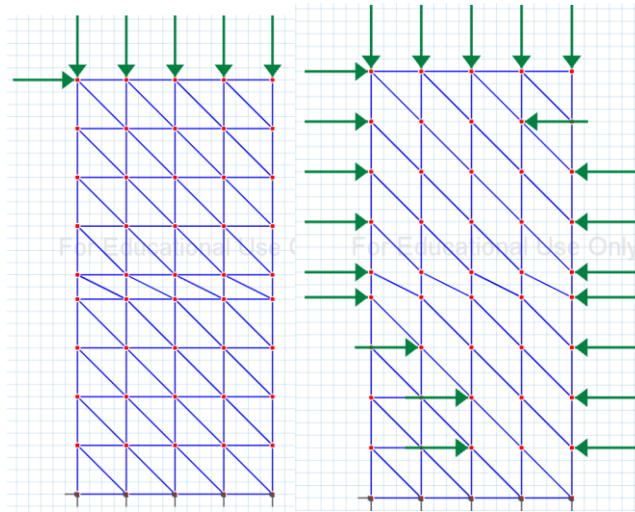
#6-PG:



(a) Initial

(b) Final

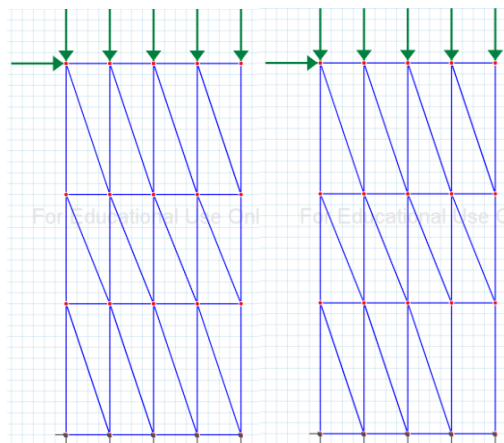
#7-PG:



(a) Initial

(b) Final

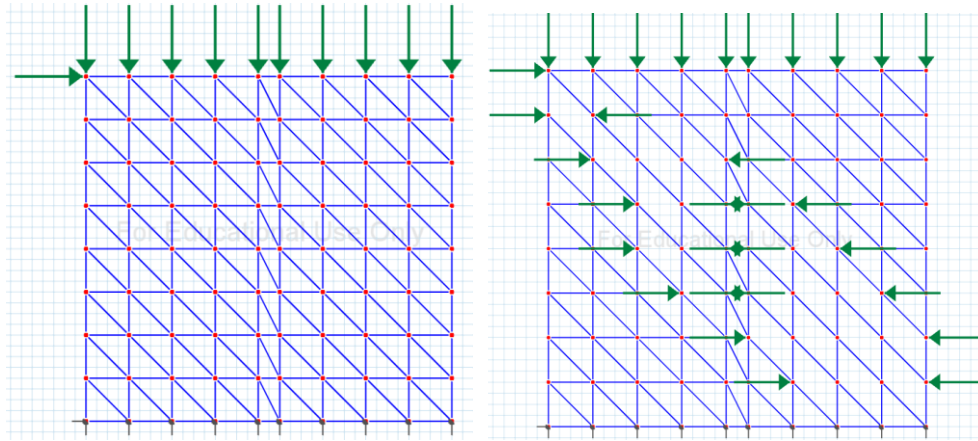
#8-PG:



(a) Initial

(b) Final

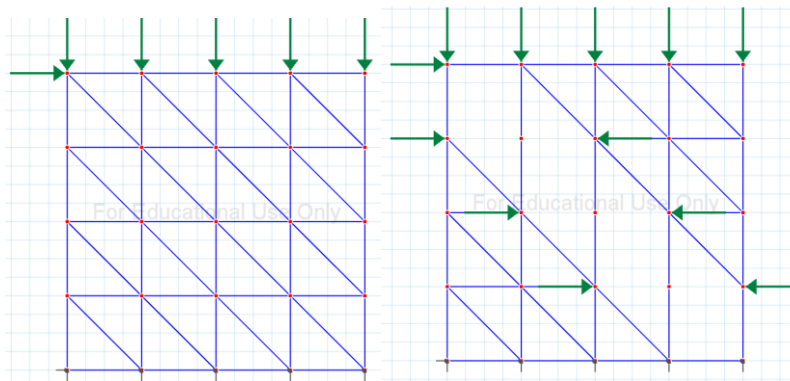
#9-PG:



(a) Initial

(b) Final

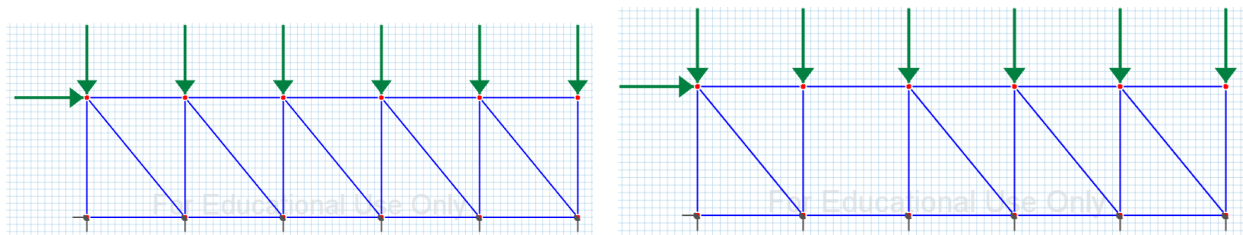
#10-PG:



(a) Initial

(b) Final

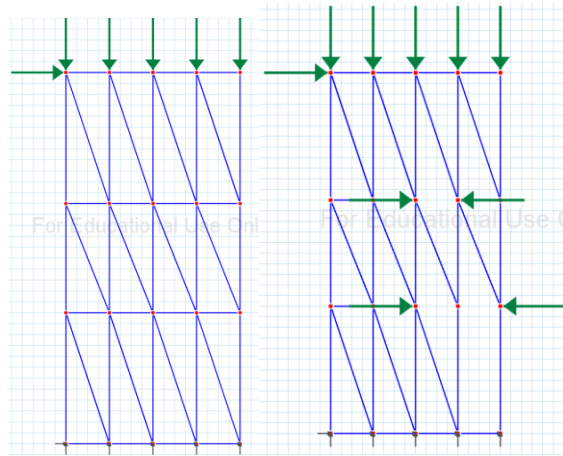
#11-PG:



(a) Initial

(b) Final

#12-PG:



(a) Initial

(b) Final

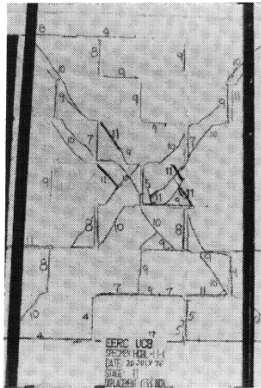
Appendix H

Available Failure Representations

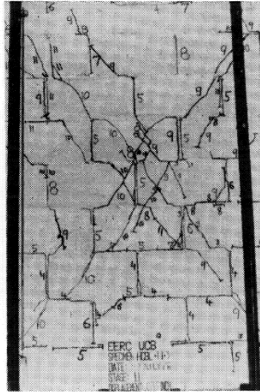
Figure H-1 Fully Grouted CMU Walls

Fully Grouted CMU Walls

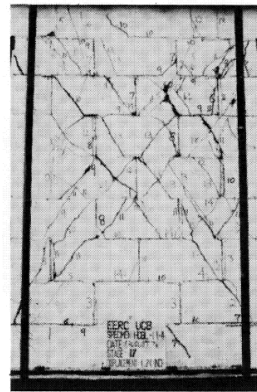
Chen et al. (1978)



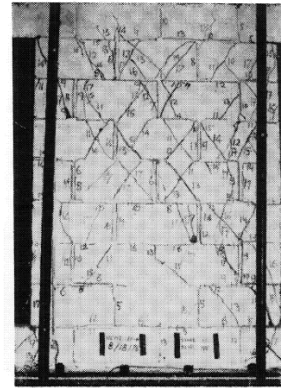
HCBL 11-1



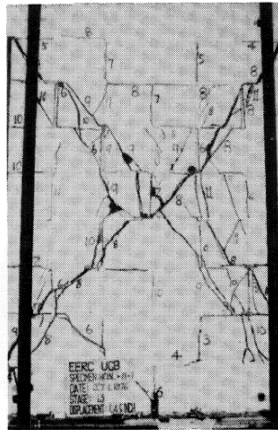
HCBL 11-3



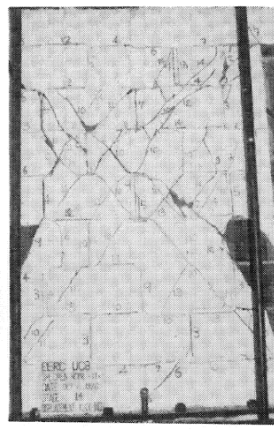
HCBL 11-4



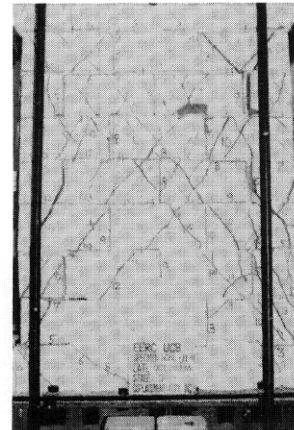
HCBL 11-6



HCBL 11-7

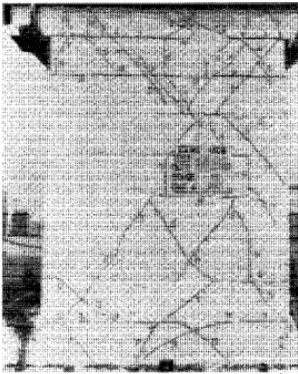


HCBL 11-9

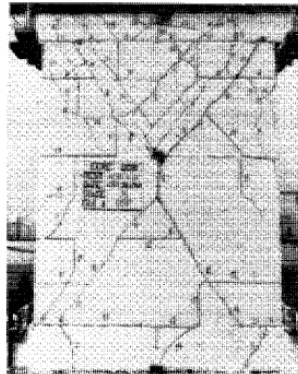


HCBL 11-11

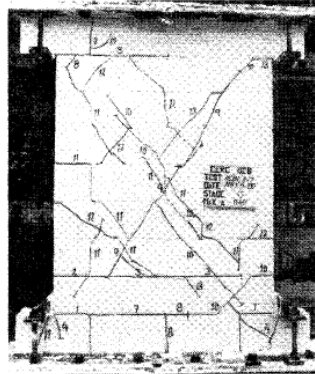
Sveinsson et al. (1985)



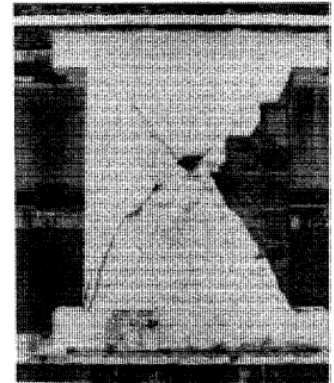
HCBL 11-13



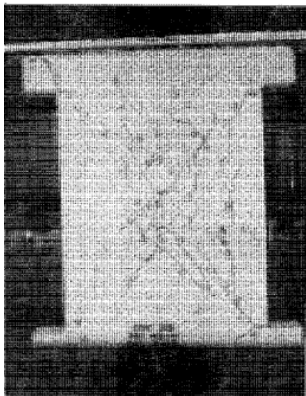
HCBL 11-15



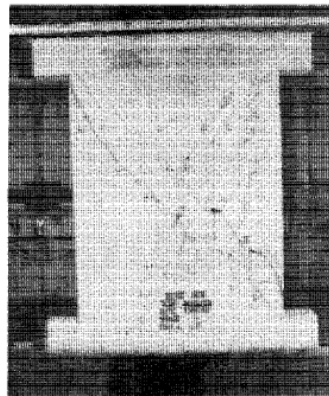
HCBL 11-17



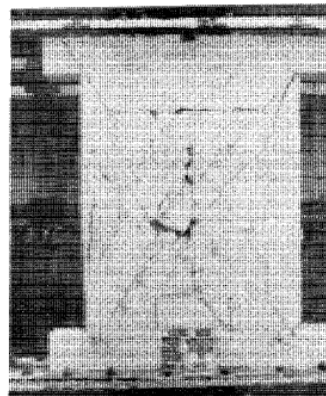
HCBL 11-20



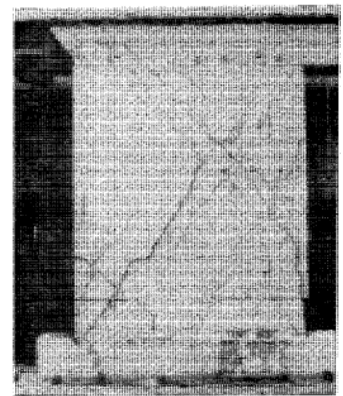
HCBL-11-21



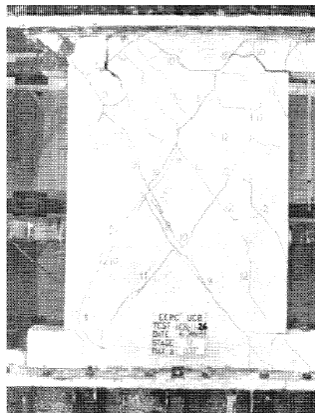
HCBL-11-23



HCBL 11-24

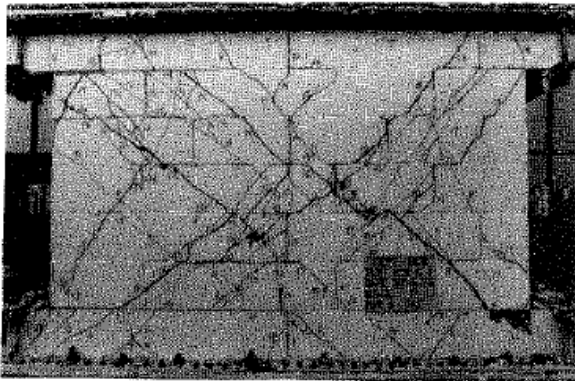


HCBL 11-25

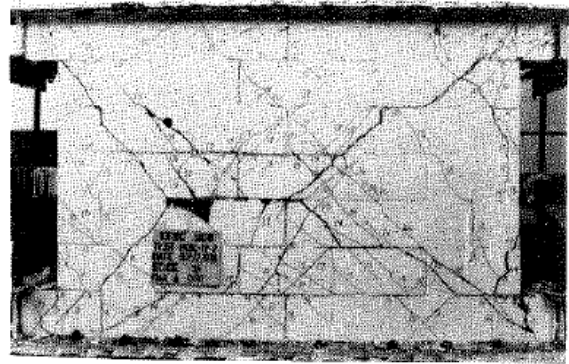


HCBL 11-26

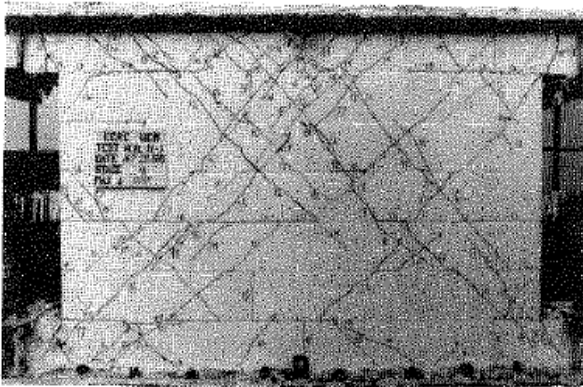
Hidalgo et al. (1979)



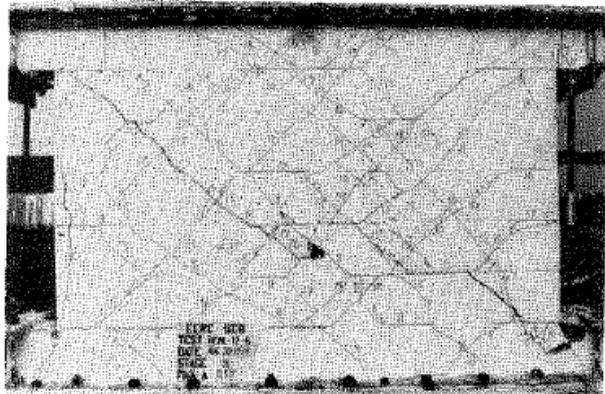
HCBL 12-1



HCBL 12-2

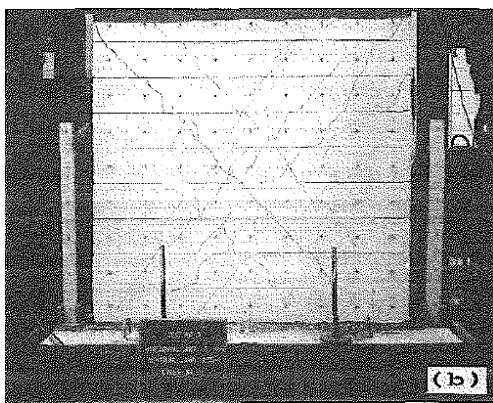


HCBL 12-3

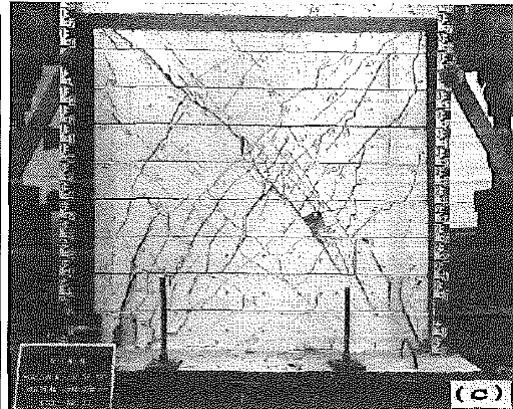


HCBL 12-6

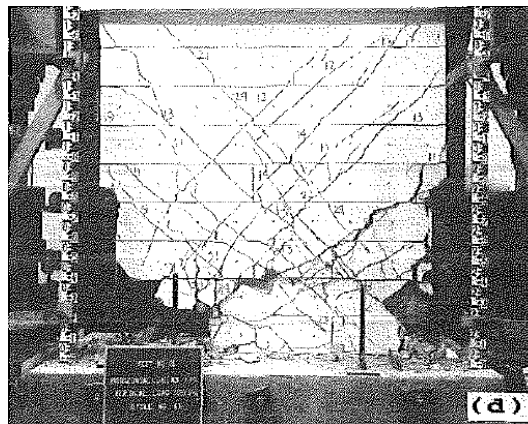
Shing et al. (1990)



Wall #3

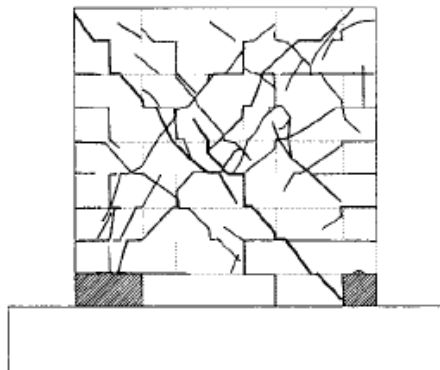


Wall #16

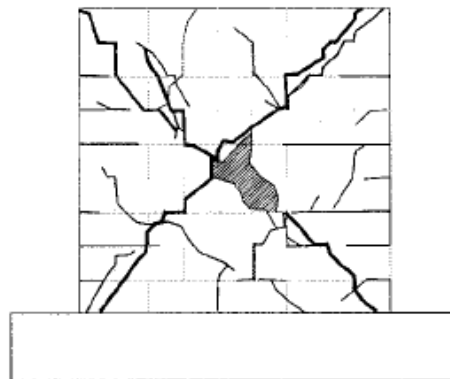


Wall #15

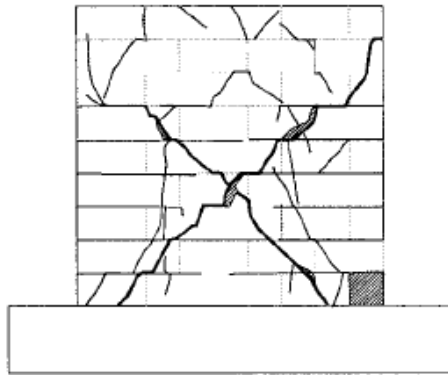
Voon and Ingham (2006)



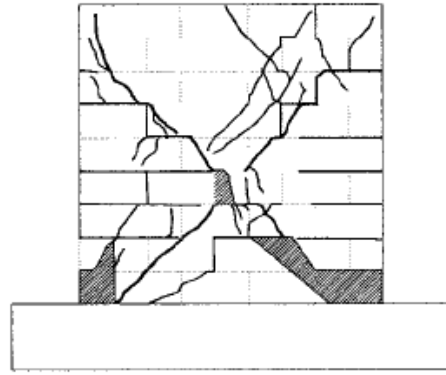
Wall #1



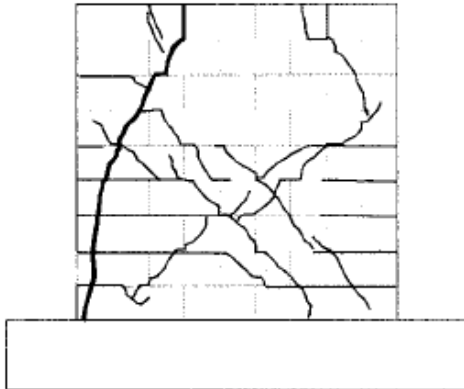
Wall #2



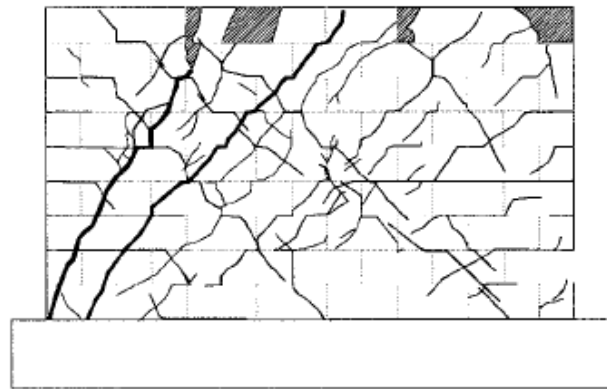
Wall #4



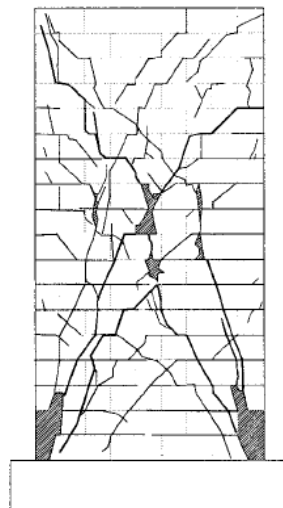
Wall #7



Wall #8



Wall #9

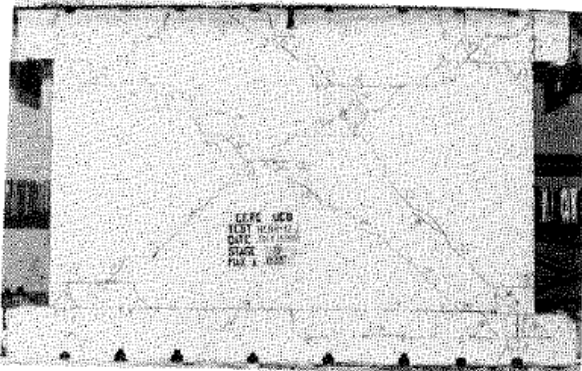


Wall #10

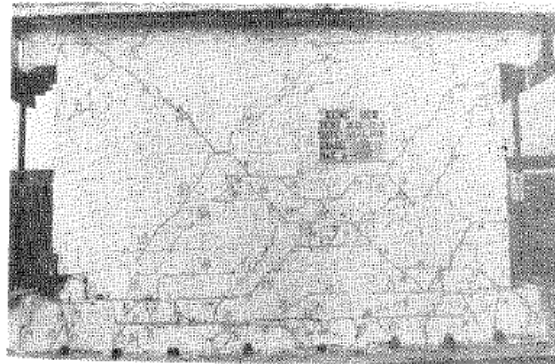
Figure H-2 Fully Grouted Clay Walls

Fully Grouted Clay Walls

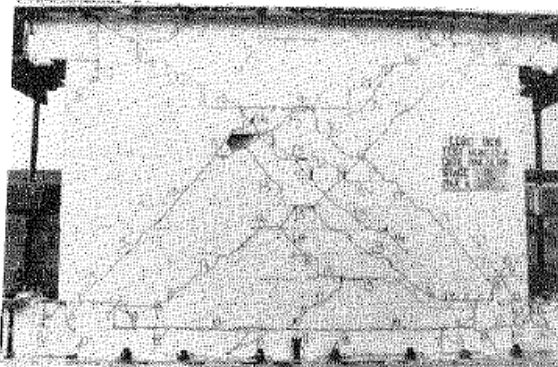
Hidalgo et al. (1978, 1979)



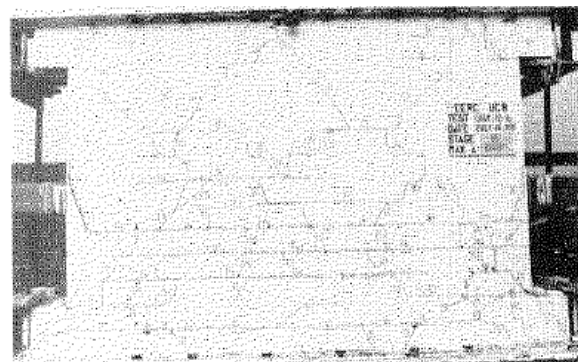
HCBR 12-2



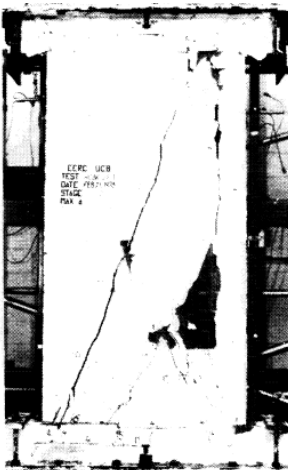
HCBR 12-3



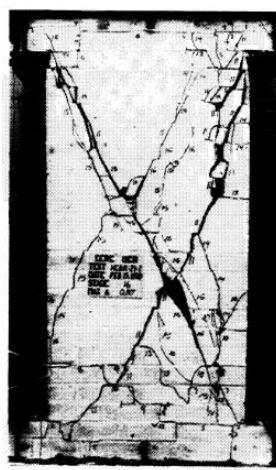
HCBR 12-4



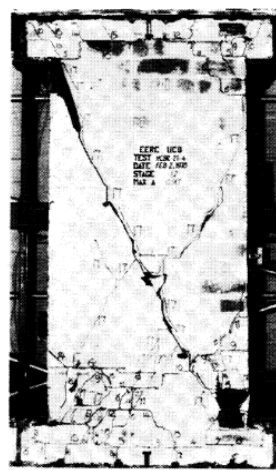
CBRC 12-6



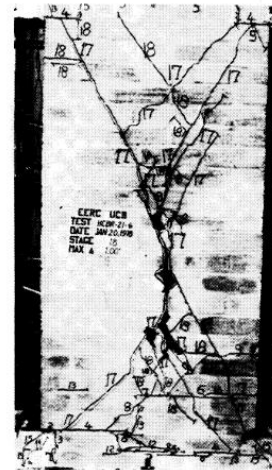
HCBR 21-1



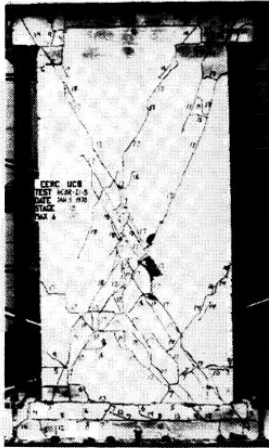
HCBR 21-2



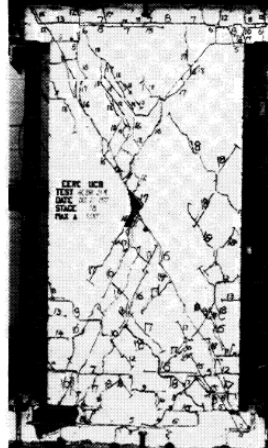
HCBR 21-4



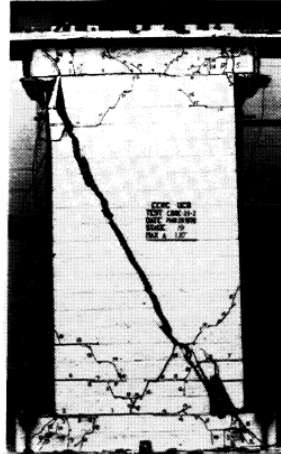
HCBR 21-6



HCBR 21-8



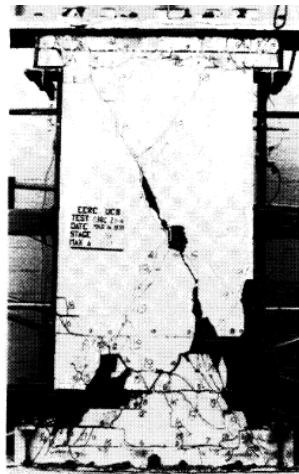
HCBR 21-9



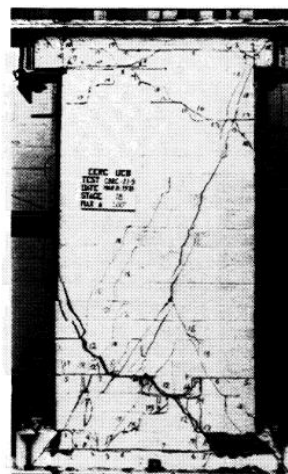
CBRC 21-2



CBRC 21-3

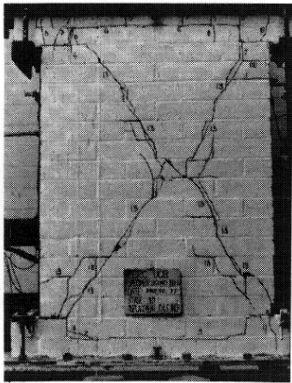


CBRC 21-4

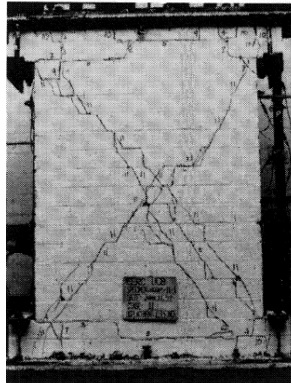


CBRC 21-5

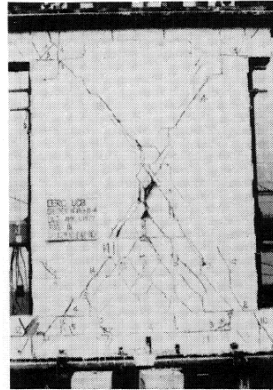
Chen et al. (1978)



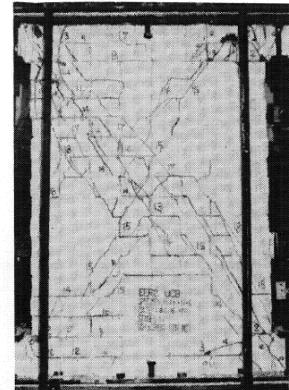
HCBR 11-1



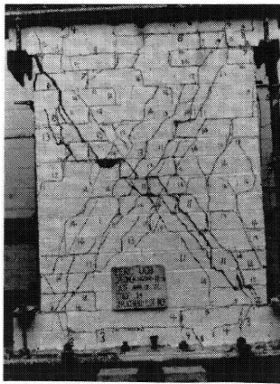
HCBR 11-3



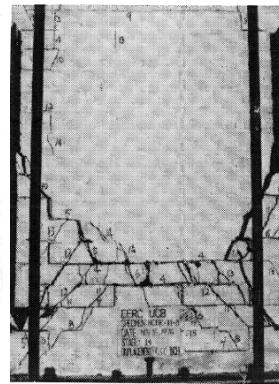
HCBR 11-4



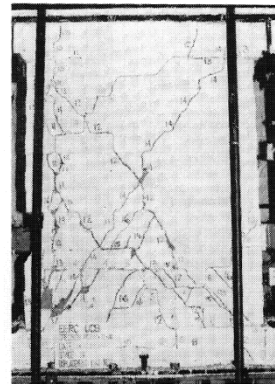
HCBR 11-6



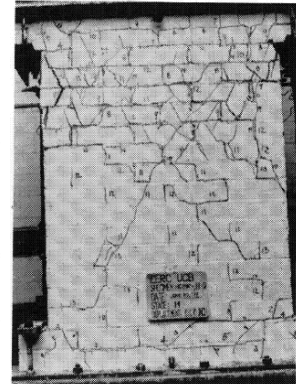
HCBR 11-7



HCBR 11-8



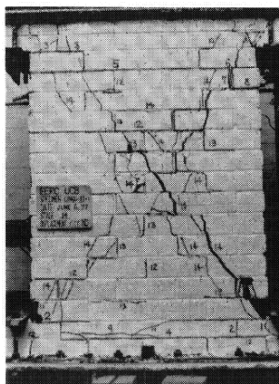
HCBR 11-10



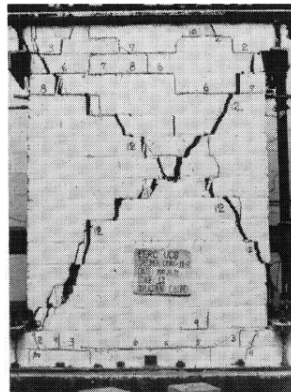
HCBR 11-12



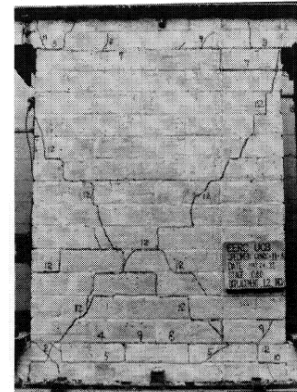
HCBR 11-13



CBRC 11-1



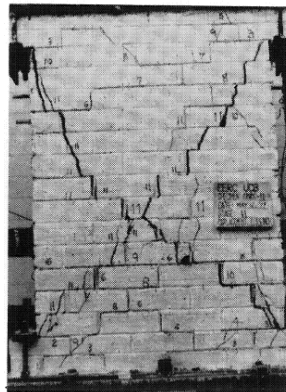
CBRC 11-2



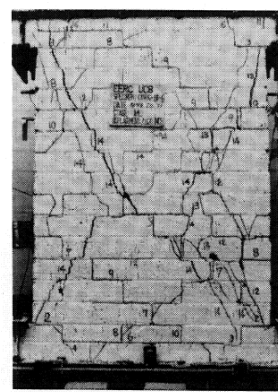
CBRC 11-3



CBRC 11-4

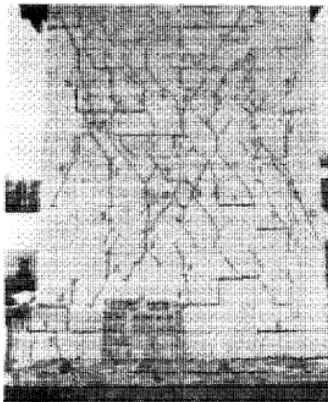


CBRC 11-5

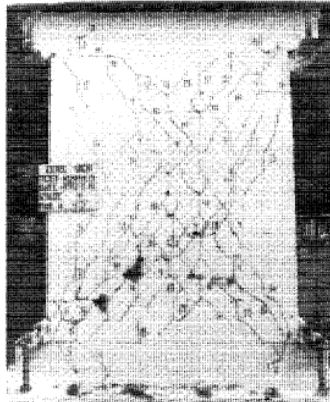


CBRC 11-6

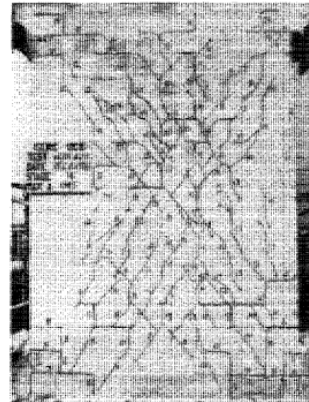
Sveinsson et al. (1985)



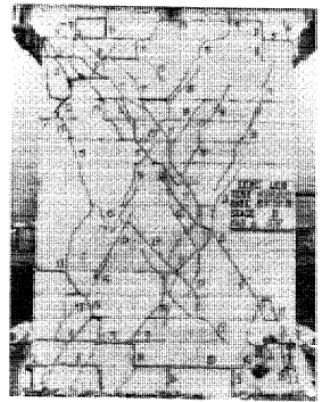
HCBR 11-19



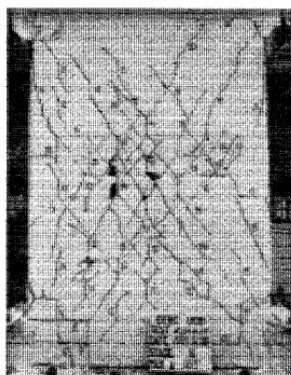
HCBR 11-21



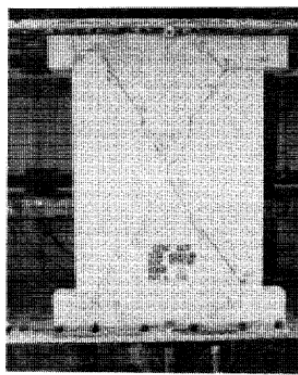
HCBR 11-23



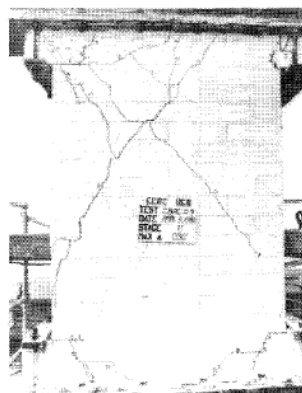
HCBR 11-25



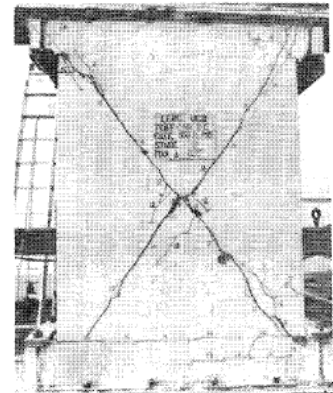
HCBR 11-27



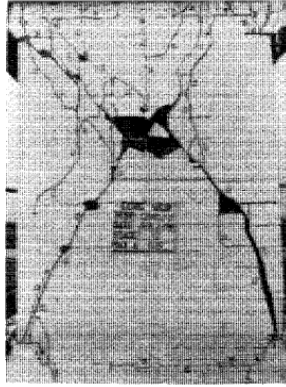
HCBR 11-30



CBRC 11-9



CBRC 11-10

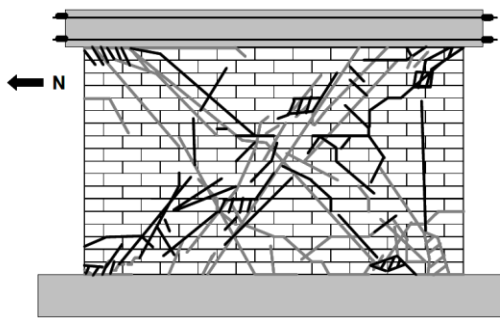


CBRC 11-12

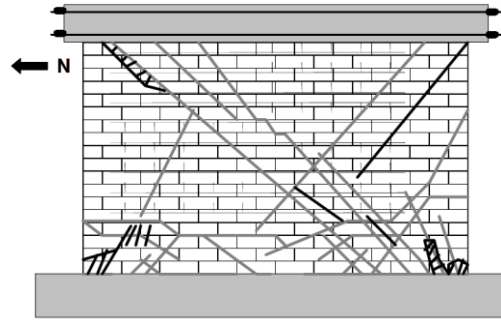
Figure H-3 Fully Grouted AAC Walls

Fully Grouted AAC Walls

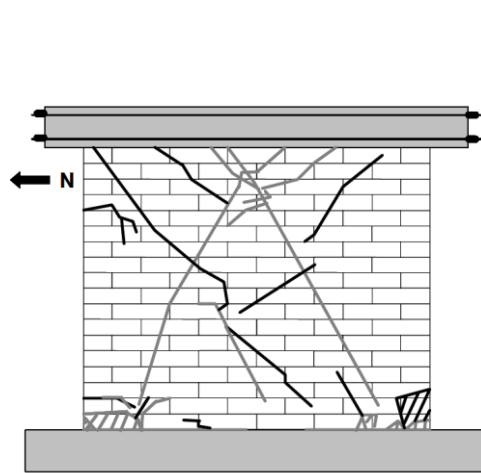
Tanner (2003)



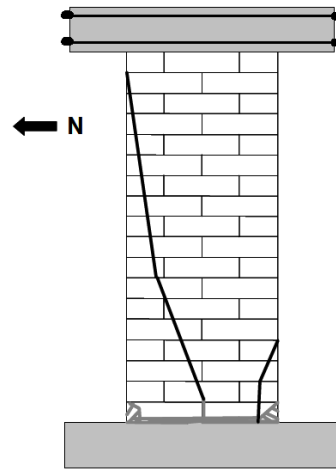
Wall #3



Wall #5

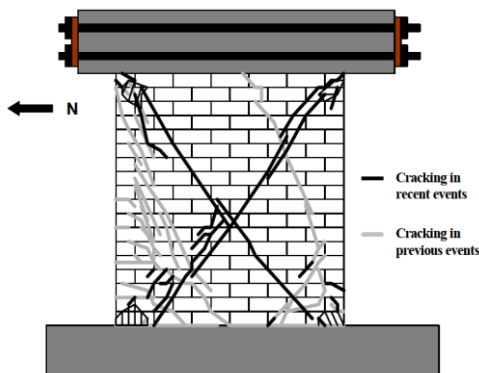


Wall #7

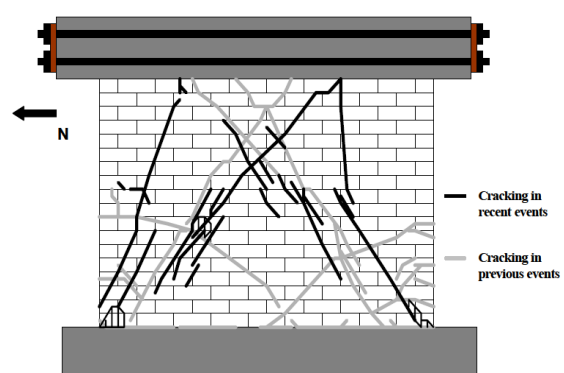


Wall #11

Cancino (2003)



Wall #18

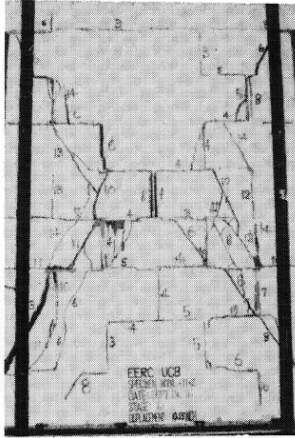


Wall #19

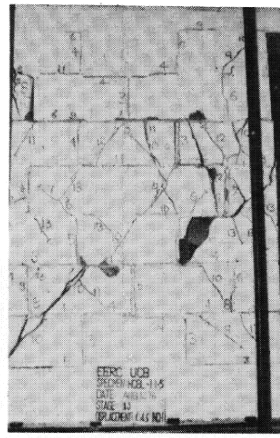
Figure H-4 Partially Grouted CMU Walls

Partially Grouted CMU Walls:

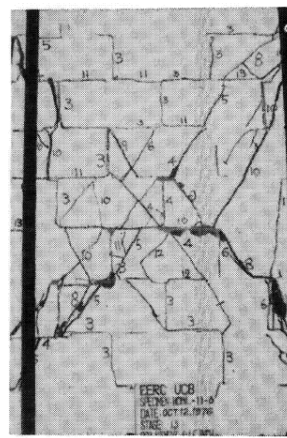
Chen et al .(1978)



HCBL 11-2



HCBL 11-5

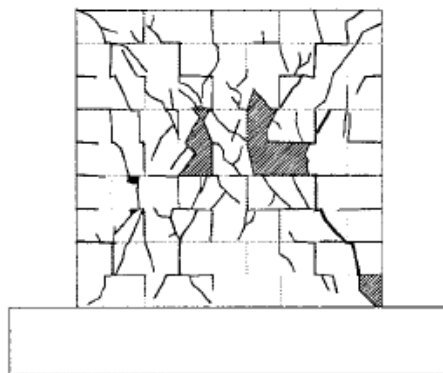


HCBL 11-8

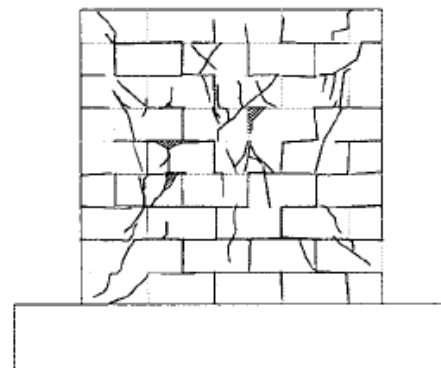


HCBL 11-10

Voon and Ingham (2006)



Wall #5

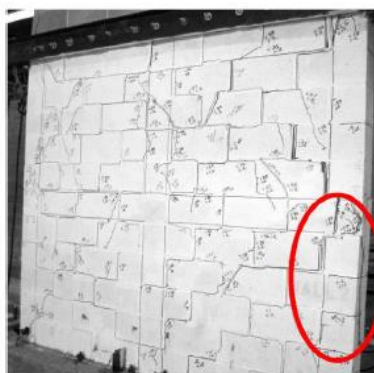
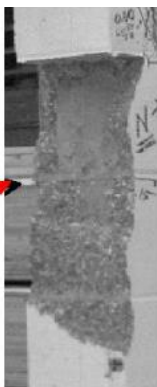


Wall #6

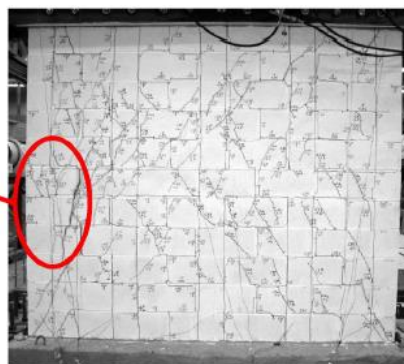
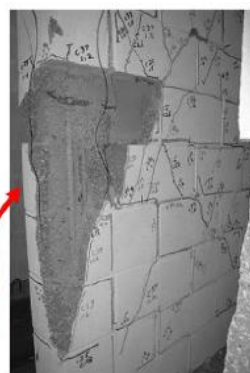
Nolph and ElGawady (2012)



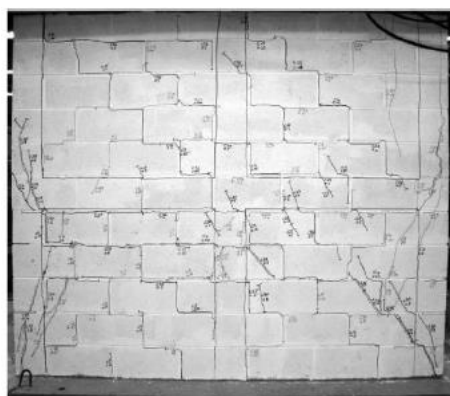
Wall PG085-48



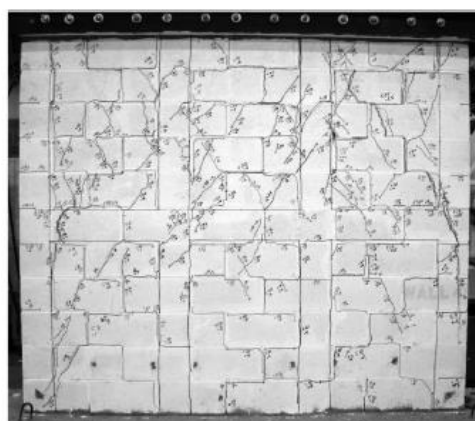
Wall PG120-48



Wall PG085-24

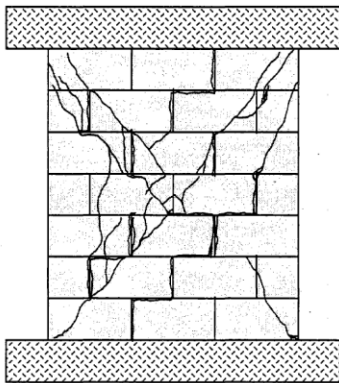


Wall PG169-48

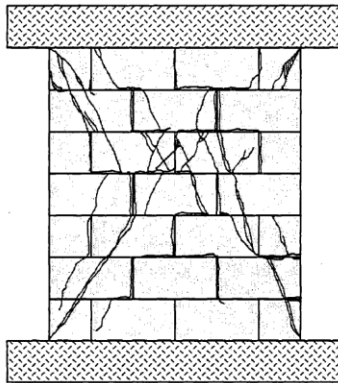


Wall PG085-32

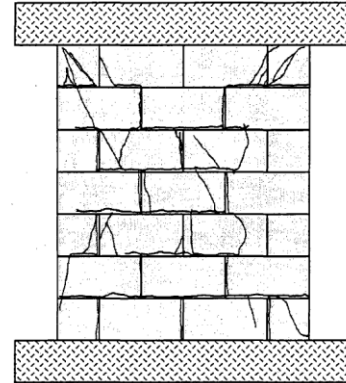
Yancey and Scribner (1989)



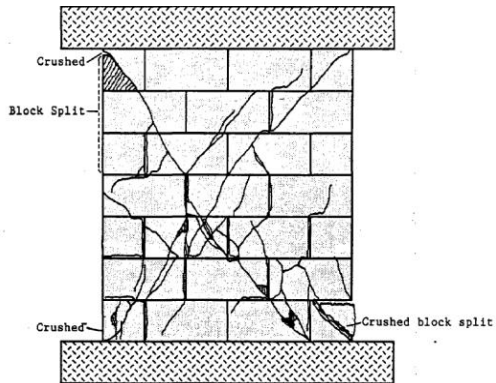
Wall R1



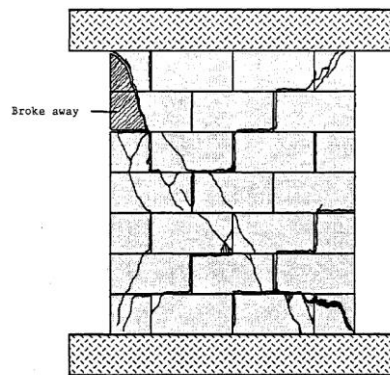
Wall R2



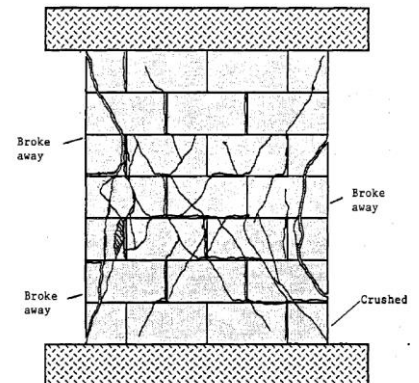
Wall R4



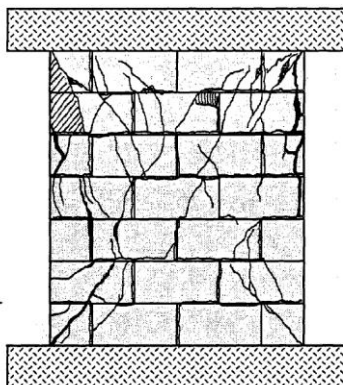
Wall R5



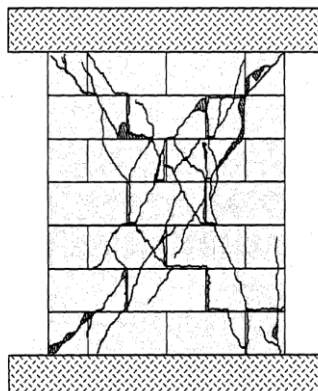
Wall R6



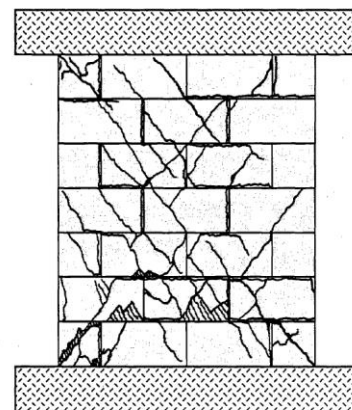
Wall R7



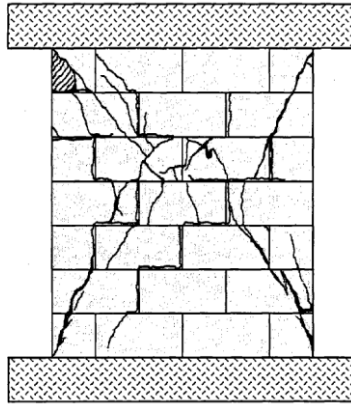
Wall R8



Wall R9

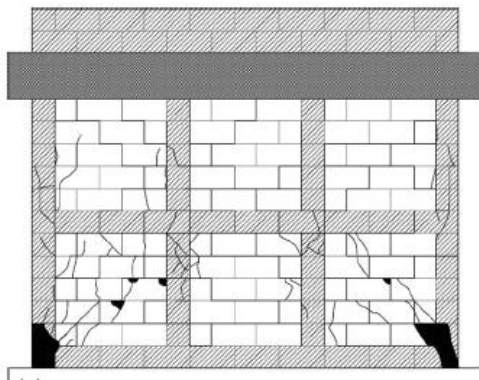


Wall R10

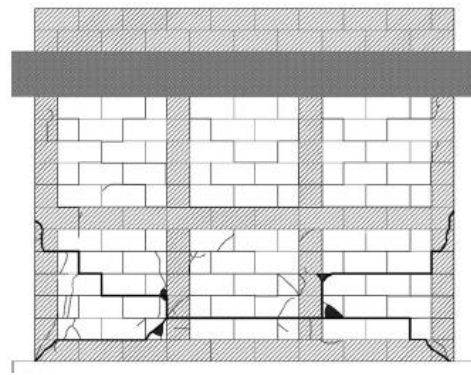


Wall R11

Minaie, et al. (2010)



Wall PCL-1 and MC-1

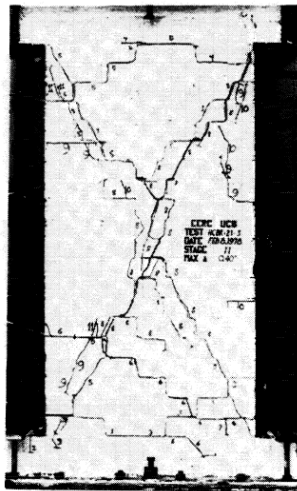


Wall PCL-2 and MC-2

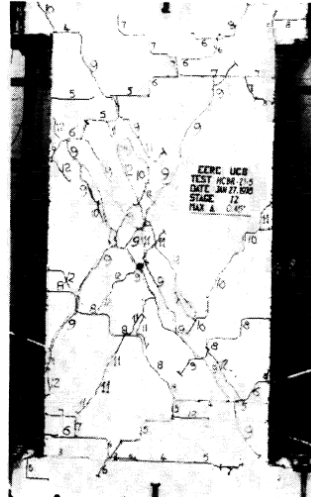
Figure H-5 Partially Grouted Clay Walls

Partially Grouted Clay Walls:

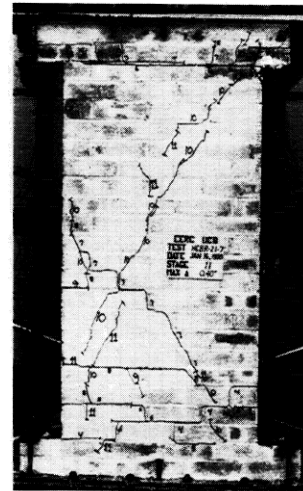
Hidalgo et al. (1978)



HCBR 21-3

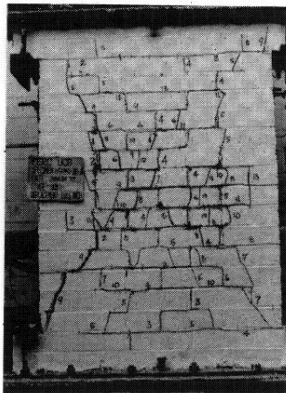


HCBR 21-5

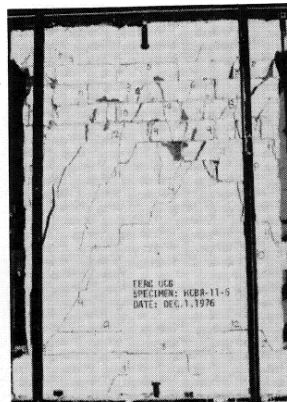


HCBR 21-7

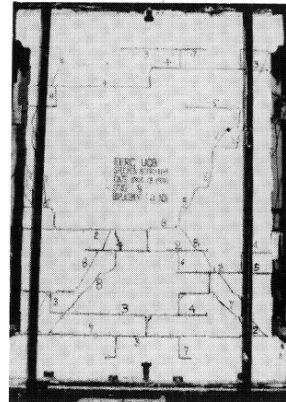
Chen et al. (1978)



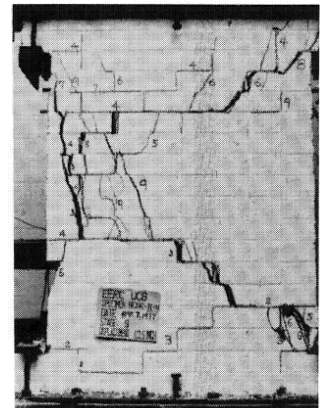
HCBR 11-2



HCBR 11-5



HCBR 11-9



HCBR 11-11

Appendix I

Sample Calculations Using Proposed Piecewise Equations:

Note--The sample calculations utilizing the proposed equations for in-plane shear strength of a masonry wall will use actual walls from the data set in Appendix A. All important parameters are shown here, as well as a representation of each wall. Only fully grouted walls will be shown in square, slender, and squat aspect ratios, as partially grouted walls are calculated in the same fashion with the exceptions noted in Chapter 6.

Fully Grouted Walls:

Wall ID--Shing #4

Aspect Ratio: 1.0 (72" x 72" x 6")

Nominal Block Size: 6 in

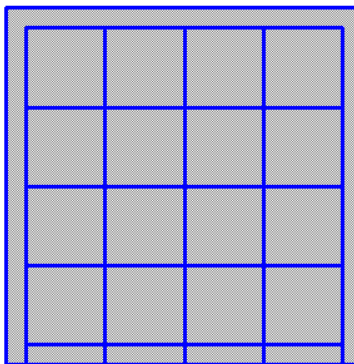
Vertical Reinforcement: #7 at 16 in. with yield stress of 72000 lb

Horizontal Reinforcement: #3 at 16 in. with yield stress of 56000 lb

$f'_m = 2600$ psi

Axial Load: 0.0 lb

Average Ultimate Strength: 79500 lb



Masonry Contribution:

$$V_m = 0.7(f'_m)(b_t^2)\left(\frac{l}{h}\right)(\zeta)$$

$$V_m = (0.7)(2600)(5.625)(5.625)(1)(0.65)$$

$$V_m = 37431 \text{ lb}$$

Steel Contribution:

$$V_s = \lambda((N) A_h f_y - \psi_s A_h f_y)$$

$$V_s = 0.6[(5)(0.11)(56000) - (2)(0.11)(56000)]$$

$$V_s = 11088 \text{ lb}$$

Total:

$$V_n = 37431 + 11088$$

$$V_n = 48519 \text{ lb}$$

$$\frac{V_{exp}}{V_{pred}} = 1.64$$

Wall ID—Voon A10

Aspect Ratio: 0.6 (72" x 120" x 6")

Nominal Block Size: 6 in

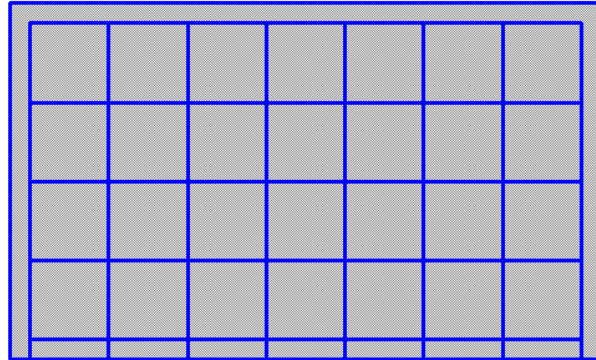
Vertical Reinforcement: #6 at 16 in. with yield stress of 46500 lb

Horizontal Reinforcement: 0.039 in² at 16 in. with yield stress of 46500 lb

$f'_m = 3523$ psi

Axial Load: 23600 lb

Average Ultimate Strength: 131460 lb



Masonry Contribution:

$$V_m = 0.7(f'_m)(b_t^2)\left(\frac{l}{h}\right)(\zeta)$$

$$V_m = (0.7)(3523)(5.625)(5.625)(120/72)(0.65)$$

$$V_m = 84532 \text{ lb}$$

Axial Load Contribution:

$$V_a = 0.25 P_n$$

$$V_a = 0.25 (23600)$$

$$V_a = 5900 \text{ lb}$$

Steel Contribution:

$$V_s = \lambda((N) A_h f_y - \psi_s A_h f_y)$$

$$V_s = 0.6[(5)(0.039)(46500)-(2)(0.039)(46500)]$$

$$V_s = 3264 \text{ lb}$$

Total:

$$V_n = 84532 + 3264 + 5900$$

$$\boxed{V_n = 93695 \text{ lb}}$$

$$\frac{V_{exp}}{V_{pred}} = \mathbf{1.40}$$

Wall ID—Voon A9

Aspect Ratio: 2.0 (144" x 72" x 6")

Nominal Block Size: 6 in

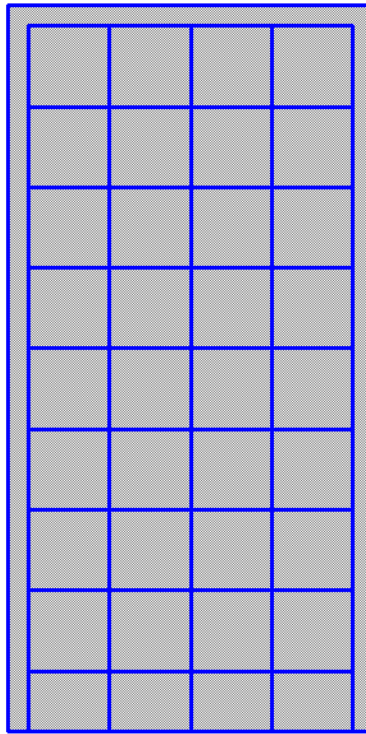
Vertical Reinforcement: #8 at 16 in. with yield stress of 46500 lb

Horizontal Reinforcement: 0.039 in² at 16 in. with yield stress of 71600 lb

$f'_m = 3523$ psi

Axial Load: 14157 lb

Average Ultimate Strength: 46179 lb



Masonry Contribution:

$$V_m = 0.7(f'_m)(b_t^2)\left(\frac{l}{h}\right)(\zeta)$$

$$V_m = (0.7)(3523)(5.625)(5.625)(72/144)(0.65)$$

$$V_m = 25359 \text{ lb}$$

Axial Load Contribution:

$$V_a = 0.25 P_n$$

$$V_a = 0.25 (14157)$$

$$V_a = 3539 \text{ lb}$$

Steel Contribution:

$$V_s = \lambda((N) A_h f_y - \psi_s A_h f_y)$$

$$V_s = 0.6[(9)(0.039)(46500) - (7)(0.039)(46500)]$$

$$V_s = 2176 \text{ lb}$$

Total:

$$V_n = 25359 + 2176 + 3539$$

$$V_n = 31074 \text{ lb}$$

$$\frac{V_{exp}}{V_{pred}} = 1.43$$

Vita

Shane Morrison was born May 16, 1986 to Bill and Brendia Morrison in Abingdon, Virginia.

Shane attended Abingdon High School, and holds a Bachelor of Science in Physics from King College in Bristol, Tennessee (Spring, 2008), a Master of Science in Civil Engineering from the University of Tennessee, Knoxville (Spring, 2011), and a Doctor of Philosophy in Civil Engineering from the University of Tennessee, Knoxville (Summer, 2013).