ABSTRACT

DURHAM, ADRIAN S. Influence of confinement plates on the seismic performance of reinforced clay brick masonry walls. (Under the direction of Dr. Meryvn Kowalsky.)

This thesis focuses on the behavior of clay masonry walls subjected to cyclic racking loading. It proposes that the seismic performance of clay masonry walls can be substantially improved if the section is adequately confined in the extreme compression zone at the toe of the wall to delay crushing of the masonry unit. This is accomplished by placing a 3.2mm thick galvanized steel plate in the mortar joint, of successive courses, in the plastic hinge region of the wall. The objective is investigated by conducting seven tests on full-scale clay masonry walls with various longitudinal and confining reinforcing ratios under seismic excitation. The results presented in this thesis show that adequately confining the grout of the clay masonry walls in the plastic hinge region may lead to substantially favorable seismic performance.
INFLUENCE OF CONFINEMENT PLATES ON THE SEISMIC PERFORMANCE OF REINFORCED CLAY BRICK MASONRY WALLS

by

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A thesis submitted to the Graduate Faculty of North Carolina State University in partial fulfillment of the requirements for the Degree of Masters of Science

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BIOGRAPHY

Adrian Semaj Durham was born on December 31, 1977 and raised in the town of Henderson, North Carolina. Upon receiving a high school diploma from Northern Vance High School, the author enrolled college at North Carolina Agricultural and Technical State University in Greensboro, North Carolina in 1996. In the spring of 2000, he earned a Bachelors of Science degree in Architectural Engineering. Afterwards he was accepted into the North Carolina State University Department of Civil Engineering in Raleigh, North Carolina to begin graduate studies in the area of structures and mechanics.

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1.0 Introduction

The utilization of masonry units for construction can be traced back several thousands years. Early civilizations developed primitive masonry units from a variety of readily available materials such as stones, soil deposits, and clay or mud. Stone is the first primitive masonry unit to be widely used. Rocks and rubble were crudely stacked, upon each other to erect primitive structures (2). As technology improved, units could be shaped into more efficiently and aesthetic elements. Concrete masonry units first appeared in the 1800s and have since evolved to incorporate several efficient and common shapes to ease handling and accommodate steel reinforcement. Ancient clay masonry was prominent in civilization rich in clay soils such as Egypt, Spain, and North and South America. These earlier clay units were hand-molded from the clay soils and allowed to sun-dry to increase its strength. Even today, the basic manufacturing process has not changed greatly. The only noticeable difference involves better control of raw materials and firing with the advancement of more comprehensive mechanization and automation of manufacturing plants.

The erection of advanced clay masonry structures is an art that began with the early Egyptian civilizations. Early pyramid construction centered around mastabas, which were to be tombs for early kings, that consisted of stepped sloping sides leading to a flat top, up to 30 ft. above the ground, which were constructed from mud brick units (2). Another ancient form of ancient clay masonry was the ziggurat, often referred to as mud, brick mountains. Several can be found in the Mesopotamia region, where they encompassed a diverse range of functions, including tombs for kings, religious
ceremonies and as place for trade and barter. They also consisted of stepped construction with a traditional ceremonial staircase leading to a divine temple at the zenith.

Figure 1.1 Ziggurat at Ur

Clay masonry was a dominant material in the United States in the early 20\textsuperscript{th} century, due to numerous attractive qualities, including aesthetic appeal, durability, relative ease of construction, cost effectiveness, and availability. Most of the construction in this era centered around unreinforced masonry building (URM) applications, which has suffered from notorious performance in moderate to high seismic regions, depicted in figure 1.2. Due to its poor seismic behavior and inherent brittle nature, many designers lost confidence in the material, relegating clay masonry to predominately façade applications (10).

Due to the numerous benefits provided by clay masonry, this study will examine the application of clay brick masonry as the primary lateral load resisting system.
Although many designers have abandoned the use of clay masonry systems, recently walls reinforced with steel rebar have shown promise of better lateral load performance. This application is achieved by constructing a double-wyth wall that allows for a center cavity, which could contain steel reinforcement. To achieve proper bond between the clay wall and the steel reinforcement the center cavity must be grouted with concrete. This application has achieved a more ductile failure mechanism, which is comparable to reinforced concrete structures.

![Figure 1.2 Franklin Junior High after 1933 Long Beach Earthquake](image)

To further advance the lateral load performance of clay masonry structures this investigation seeks to improve the deformation capacity of brick masonry through the use of confinement. It is well established that lateral reinforcement tends to improve the deformation capacity of cementitious members, by restraining the compressive reinforcement from buckling and placing the material in a state of triaxial compression.
This is clearly evident in reinforced concrete bridge columns, that use spiral reinforcement to increase ductility.

This concept has been applied to masonry by attempting to confine the extreme compression zone of the structures with galvanized steel reinforcement plates. The use of confinement plates in masonry structures was first proposed by Priestley and Bridgeman in 1974 (10). The plates are placed in the mortar joints of the structure and confine the core-grouted section to prevent premature spalling. It has been noted that this confinement technique can double the capacity of concrete masonry prisms (13). The aspect ratio of a clay masonry unit is less than half of typical concrete masonry units, leading to potentially higher confinement volumetric steel ratio, which could lead to a more significant increase in deformation capacity than the 100 percent increase in concrete masonry (13). Therefore this study will focus on the effect galvanized steel confinement plates has on the seismic behavior of reinforced clay brick masonry structures.

1.1 Scope and Research Objective

The primary goal of this investigation is to quantify the influence of galvanized steel confinement plates on the reversed cyclic response of clay brick masonry walls. This evaluation will determine if confining the extreme compression zone of the wall with steel confinement plates can have a substantially, favorable effect on the seismic performance from the perspective of increased deformation capacity, energy dissipation, and strength. To accomplish the targeted objective experimental studies consisting of reversed cyclic testing of large-scale walls was conducted in conjunction with extensive
analytical studies to determine optimum configuration and effectiveness of the confinement plates.

1.2 Research Significance

Even though clay masonry application is predominately centered around non-load bearing, façade applications, it remains the most extensively used masonry units for construction applications (2). This is primarily a function of several variables including its aesthetic appearance, the widespread availability of clays and shales, and its historic record of durability, with relative low maintenance required. Subsequently if the overall confidence in its lateral load performance can be re-established, which is the intent of this study, designers could potentially design substantially more cost effective structures that could also serve as an aesthetically pleasing façade.

1.3 Current State of Practice for Seismic Design of Masonry Walls

Analysis and design of masonry buildings is typically based on rational approaches developed through many years of research. The goal of any structural design procedure is to proportion an efficient structure that is able to safely resists loads placed upon without significant damage or loss of life. Currently, there are three traditional practices that are generally used in the contemporary masonry buildings and accepted by the Masonry Standards Joint Committee (7), sponsored by the American Concrete Institute (ACI), the American Society of Civil Engineers (ASCE), and the Masonry Society (TMS). Even though there are several other building codes that specify masonry design specification, the general design procedures typically remains the same. The
TMS Code (7) specifies that the all load resisting, reinforced masonry structures and their component members shall be designed in accordance to one of the following:

(1) Empirical Design of Masonry

(2) Allowable Stress Design

(3) Strength Design of Masonry

The empirical design is very simple, but extremely conservative technique in which arbitrary limitations such as a maximum diaphragm length-to-width ratio, an allowable compressive strength of a typical masonry unit, minimum thickness, and lateral support requirements are prescribed. Even though this technique is quite simple it provides no rational means of design, leads to highly conservative buildings, and is not allowed in certain highly seismic regions by the TMS code (10).

In 1978, ACI-Committee 531 published a masonry building code that introduced the working stress concept. The ACI 530 committee and ASCE committee 5 later jointly expanded upon the original concept and applied it to both concrete and clay masonry in 1988 (2). The reformed working stress procedure is presently the most widely used in the design of masonry structures. The allowable stress technique propose that stresses developed under normal service loading conditions be kept below an acceptable elastic limit to maintain a safe, functional structure. Allowable Stress Design (ASD) improved on the crude empirical design by providing a more rational approach, thus improving confidence in the behavior and design of masonry structures. The chief drawback to the procedure is it assumes linear elastic behavior up to the allowable limit, which also represents the ultimate loading scenario. This leaves no means to accurate account for the inelastic behavior of a system when it overloaded as in the case for most seismic
attacks. Also the procedure gives no rational way to evaluate the ductility of a system, a significant parameter in most seismic design procedure.

The inadequacies of the ASD procedure led to the most current design procedure accepted in the TMS code (7), Strength Design. In this technique, the structural members are designed taking into account the inelastic strain to reach ultimate strength when an ultimate load is applied to the structure, allowing the ability to accurately predict the inelastic response of a structure. Load factors are also incorporated to allow for variability in the applied load and to protect the structure from experiencing its ultimate condition.

1.4 Limit States Design

Even though the strength design procedure ensures satisfactory resistance against collapse, it does not account proper serviceability of a structure under normal design loads, such as excessive deflection, cracks, and vibrations. Therefore a recent technique, Limit States Design has been proposed, which identifies all modes of failure and disruption of functionality throughout the structure and determine acceptable levels of safety against the occurrence of each. The current 2002 Masonry Standards Joint Committee (7) does not include this procedure, but with its overall design scope, it is simple to understand the recent demand to adopt Limit State Design into the present code.

Since 1990, the limit states design procedure has been further extended by the SEAOC blue book document (16), which introduces the concept of Performance-Based Seismic Engineering (PBSE). The document outlines a set of procedures for design and construction of structures to achieve predictable levels of performance in response to
specified levels of earthquake, within definable levels of reliability. In contrast to the current Unified Building Code (UBC) (18) and the International Building Code (IBC) (5) which specify life safety as the only design criteria, PBSE allows for unlimited performance levels at the discretion of the owners and engineers. In this thorough procedure, the engineer must have extensive knowledge of structural performance of respective systems and is required to make rational decisions about the seismic performance of the system, thus providing the designer greater reliability in predicting and controlling seismic performance.
2.0 Literature Review

Several investigators have addressed the topic of the seismic response of masonry structures. Since most of the structural applications in the construction field incorporate the use of concrete masonry units (CMU), the majority of the previous research is concentrated on concrete masonry. Shear behavior of masonry has generated significant interest due to the fact most of the failures in previous earthquakes have been dominated by a shear mode. It has been noted that the masonry shear strength component can be several times higher than allowed by various codes. While, other experiments conclude that the introduction of increased amount of transverse steel in masonry walls has an insignificant effect on the inelastic behavior of the walls. Even though most of the previous seismic failures have been characterized by brittle shear failures, it has been demonstrated that a ductile, flexural mode exists for masonry walls, if properly detailed. This mode of failure will be further explored in this thesis.

2.1 Past Concrete Masonry Research

To withstand seismic forces generated in high seismic regions, masonry shear walls must be able to demonstrate sufficient, ductile characteristics. To compensate for the inherent brittle nature of the material several techniques have been adopted to improve the performance of masonry structures. The most influential advancement in masonry design was the incorporation of steel reinforcement within structural walls to improve ductility, a practice now required by most codes for high seismic regions.

It has been shown that properly detailed spiral or hoop reinforcement can significantly increase the deformation capacity of concrete structures by confining the
concrete. To simulate this concept in masonry structures, Priestley and Elder (10) placed stainless steel confining plates within the mortar joint of concrete masonry prisms to confine the section. Their study reports that the use of the stainless steel plates resulted in a more gradual decline in the stress-strain curve and thus over 100% increase in deformation capacity. With respect to the favorable results, they extended the concept to demonstrate how confinement plates could be used to design masonry shear walls that display exceptional deformation capacity. The increased ductility allows the structure to sustain significant inelastic excursions without significant degradation, thus enhancing its capability to dissipate the intense energy generated by potential earthquakes (12).

### 2.2 Past Clay Masonry Research

Due to fact there are very few modern buildings constructed with clay masonry as a primary load resisting system, there have been only a limited amount of research in this area. Even though CMU units dominate most of the masonry design, it has been shown that satisfactory behavior can be achieved from clay brick masonry if appropriately designed.

One of the most extensive evaluation of clay brick masonry was performed in the 1970s at the University of California at Berkeley (1,3,4,17). The investigators probed several parameters such as height ratio, transverse steel ratio, and grout techniques with respect to the shear mode of failure to determine if desirable inelastic behavior could be obtained.

Even though, shear has dominated most of the previous lateral load failures, Priestley and Bridgeman (10) were able to demonstrate that a flexural mode exists for the
clay masonry shear walls and satisfactory ductility could be obtained from the structures. Through a series of racking tests on large scale clay brick masonry walls, it was noted that confinement plates placed within the mortar bed joints restricted the lateral expansion of the joint and the differential expansion between the clay brick unit and the joint. As a result, the plates inhibited vertical splitting cracks caused by tensile forces introduced into the clay brick unit by the differential expansion of the mortar joint and brick. By providing shear steel to carry the full shear load and confining the compression region of the walls with stainless steel confinement plates, Priestley and Bridgeman (10) were able to obtain a flexural dominated response that demonstrated superior ductile characteristics.
3.0 Failure Mechanisms for Masonry Shear Walls

In general, three failure modes exist for masonry walls. Possible failure scenarios include a shear mode, flexural mode, and out of plane instability. In previous earthquakes, the majority of the failures were contributed to diagonal tension, which often can lead to sudden, catastrophic damage. The flexural mechanism is the most desirable mode of failure. It is characterized by significant yielding of reinforcement thus providing a favorable ductile response. The issue of lateral instability of masonry walls has only been recently addressed by a few researchers and not covered in most codes. However when thin, non-compact walls are subjected to compression strains, the danger of out of plane buckling may arise (8).

3.1 Shear

Shear failures should be avoided whenever possible due to potential catastrophic damage as demonstrated by previous seismic failures of masonry walls. Shear cracks are initiated when induced principal stresses become larger than the tensile stress of the masonry. As cracks propagate along the wall, the shear resistance of the masonry gradually diminishes. Therefore if induced stresses are larger than the masonry shear stress, transverse reinforcement must be placed within the cross section to transfer the shear load.

One well-established and commonly used model to represent the shear mechanism is the truss analogy, which is also used in reinforced concrete analysis. In the truss analogy, the transferring mechanism is represented by a truss that consist of a tension chord based on the longitudinal steel reinforcement and a compression chord comprised of the masonry. Compression in the masonry struts and tension in the
transverse steel form the diagonals, of the truss. Conservative approaches assumes that all the shear should be carried by the transverse steel, which leads to following commonly used expression for predicting the shear capacity provided by the transverse steel.

\[ V_s = \frac{A_v f_v d}{s}, \]  

\text{where} \hspace{1cm} \text{Equation 2.1}

\( V_s \) = shear capacity provided by the transverse steel  
\( A_v \) = area of reinforcement  
\( d \) = distance from extreme compression fiber to centroid of tension reinforcement  
\( s \) = spacing of reinforcement

In addition to the steel truss model, most codes allow for shear resistance provided by the masonry outside the truss analogy. In a well-established model proposed by Paulay and Priestley, two values are given for the shear strength of masonry to account for the drop in resistance in potential plastic hinge regions (8). The previously mentioned authors propose the following equations. Therefore the total resistance can be expressed by the sum of the masonry contribution \( V_m \) and the steel component \( V_s \).

In all regions except potential plastic hinges,

\[ v_m = 0.17 \sqrt{f'm} + 0.3 (P_u / A_g) \text{ (Mpa)} \]  

\text{Equation 2.2}

but not greater than

\[ v_m = 0.75 + 0.3 (P_u / A_g) \text{ (Mpa)} \text{ or 1.3 Mpa} \]  

\text{Equation 2.3}

In regions of plastic hinges
\[ v_m = 0.05 \sqrt{f'm} + 0.2(P_u/A_g) \text{ (Mpa)} \quad \text{Equation 2.4} \]

but not greater than

\[ v_m = 0.25 + 0.2(P_u/A_g) \text{ (Mpa)} \text{ or } 0.65 \text{ Mpa} \quad \text{Equation 2.5} \]

When a reversed cyclic load is applied to a structure, there is a possibility for the sliding to occur at the base of the wall or along flexural cracks of the structure generating shear stresses. This phenomenon is generally referred to as sliding shear and can significantly reduce stiffness and energy dissipation, and possible premature failure. Sliding can be minimized in a masonry wall by a well-distributed system of reinforcement due to the dowel action provided the steel reinforcement. To maintain proper seismic performance of masonry walls, Paulay and Priestley (8) propose the following equation to control the sliding shear in walls.

\[ A_{vf} = \frac{V_u - \phi \mu P_u}{\phi \mu f_y} \quad \text{Equation 2.6} \]

\( A_{vf} = \) area of reinforcement transverse to the potential sliding plane

\( V_u = \) applied shear

\( \phi = \) strength reduction factor

\( \mu = \) coefficient of friction

\( P_u = \) axial load

\( f_y = \) yield stress of the reinforcing bars
3.2 Flexure

The most desirable failure mechanism of masonry walls is the flexural mode. A flexural failure is usually characterized by a ductile response, allowing for extensive energy dissipation. Although the shear mode usually governs the behavior of masonry shear walls, it was reported by Priestley and Bridgeman (10), that if sufficient horizontal steel is provided to carry the full shear load that a flexural mode of failure can be achieved, thus providing sufficient ductility.

The flexure mechanism is characterized by significant yield of the reinforcement and extensive horizontal, flexural cracking leading to a nonlinear force-deformation response. Depending on the longitudinal reinforcement ratio and compressive strength of masonry, three possible secondary failure scenarios exist after significant yielding has occurred. The final failure due to flexure may be contributed to excessive crushing of the masonry at the extreme compressive toe of the wall, buckling of the longitudinal reinforcement after confinement from the masonry and grout is lost, or rupture of the extreme bars from accumulation of excessive strains within the bar. After the secondary failure is achieved a significant degradation of strength and stiffness will occur diminishing the reliability of the wall the transfer any significant forces.

3.3 Out-of-Plane Instability

Out-of-plane stability of masonry walls is a phenomenon that has not received as vast amount of consideration. Most design codes do not address the issue or provided any means to control the mechanism. It was first explored by Paulay and Priestley (9) around ten years ago. They demonstrated that instability could have a detrimental effect on the
ductile response of a masonry shear wall if not properly controlled. To monitor the phenomenon a conservative design approach was proposed to predict the out of plane buckling in masonry walls.

![Figure 3.1 Paulay-Priestley Out-of-Plane Buckling Model](image)

Figure 3.1 Paulay-Priestley Out-of-Plane Buckling Model
Thin non-compact walls masonry walls may be vulnerable to out-of-plane buckling when subjected to compression strains. However the study by Paulay and Priestley revealed that “the potential for out-of-plane buckling of thin sections of ductile walls depends more on the magnitude of the inelastic tensile strains imposed on a region of the wall that, on subsequent moment reversal, is subjected of compression.” (9)
When a masonry structural wall is subjected to large deformation demands such as that induced by severe earthquakes large cracks will develop across the width of the section. Upon unloading of the wall, the tensile strains in the longitudinal reinforcement is effectively reduced to zero while the cracks in the wall remains open due to substantial inelastic strains accumulated in the reinforcement. At this point, there is no reliable means to resist an out-of-plane forces generated by imperfections in the alignment of steel reinforcement and applied forces which may not fully coincide with the centroid of the section. When the cracks become closed on one the compression side of the wall due increased load, the wall eventually regain stability. Therefore during the load reversal phase a structure, before closure of the cracks occur, an out-of-plane buckling failure may be experienced before the full flexural strength of the wall is mobilized.
4.0 Experimental Design

In this investigation, seven cantilever, clay masonry walls were tested under in-plane, reversed, cyclic loading. The objective of the experiments was to evaluate how key variables such as longitudinal steel reinforcement, confinement steel ratio, and confinement plate configurations effect critical performance limit states of the structure. The results of the study were used to assess the influence of confinement plates on the seismic behavior of clay masonry structural walls and implement the influence into a practical analysis routine. This investigation will also provide insight into efficient application and design of galvanized steel confinement plates used in clay masonry walls.

4.1 Wall Design

Due to the apparent brittleness of clay masonry structures, special consideration must be given to allow a ductile, flexure failure to occur. Aspect ratio (the height of the wall divided by the length of the wall) is critical in evaluating the behavior of a structural wall. Walls with an aspect ratio of less than 2 are commonly referred to as squat walls. The flexural strength of such walls can be so much greater than the shear strength that a shear failure made be inevitable. To eliminate this phenomenon, walls with an aspect ratio of 2, commonly used in small industrial applications, were constructed to reduce the possibility of a shear failure. A double-wythe construction technique, in which a wall is constructed of two separate layers of bricks joined together in this case by a concrete grouted core, was adopted to allow the use of steel reinforcement for improved seismic behavior.

To accommodate limitations in laboratory space, the length of the multi-wythe wall was chosen to be 1219mm long with a corresponding effective height of 2438mm.
In compliance with ACI 530 and TMS 402 (7) masonry standards, a thickness of 254mm was adopted. In addition a minimum width of grout space for grouting between masonry wythes is also given by ACI 530 and TMS 402 (7). In fulfillment of this requirement and practical construction, a center cavity that will be fully grouted with concrete was design to be 70mm wide and 1035mm long.

![Figure 4.1 Construction of Test Specimens](image)

While the geometry of the clay masonry wall remained constant throughout the study, three primary variables were selected to conduct a more comprehensive evaluation. To insure that the results from the analysis would applicable to a wide range of masonry structures various steel reinforcing ratios were evaluated. In addition, different confinement ratios were assessed to achieve optimum performance and design.
Different plate configurations were also addressed to potentially reduce construction time.

Two practical reinforcing ratios were selected for this study. It is well-established that increasing reinforcing ratios generally results a reduction of ductility, so it would be worthwhile to investigate how effective confinement would be in a wall with low reinforcement ratio and one with a high reinforcing ratio. Therefore one set of walls was selected to have a high reinforcing ratio and the other set would contain a relatively low reinforcing ratio. Afterwards the initial section analysis performed with the computer program developed by King et al. (6) program, which is a concrete moment-curvature program. Practical steel ratios of 1.5% and .6% were selected for the investigation. To achieve the targeted longitudinal reinforcing ratios 13-%6 (D=19mm) bars and 5-%6 (D=19mm) bars were used respectively used.

After the longitudinal reinforcement was selected, the shear capacity of the wall was evaluated to determine if the wall had adequate strength to resist the external shear associated with a flexural failure. When the analysis was complete, it was determined that transverse steel should be placed within the section to resist the shear demand. Therefore 7-%4 (D=13mm) bars were inserted to help resist the applied shear. The bars were spaced at 152mm centers over the bottom 460mm (to account for the drop of shear resistance in the plastic hinge region) of the wall and 330mm centers over the rest of the wall. Even with the presence of the shear reinforcement, the first set of specimens experienced shear-induced failures. It is the belief of the investigators that this was a function of improper anchorage of the steel, not allowing the full capacity of the transverse steel to develop. Therefore to ensure a flexural mechanism for the final 3
specimens, U-shaped D5 bars spaced on 152mm centers throughout the entire wall. Bending the bars at a 90 degree angles creating the U-shape provided better anchorage for the reinforcement, thus allowing the transverse steel to fully develop its strength.

4.2 Confinement Plates

A flexural failure in masonry structures is generally characterized by crushing of the masonry due to extreme compressive stresses. By confining the region of maximum compression within the clay wall, higher strains can be achieved within the section allowing for increased ductility. In this project confinement is achieved by placing galvanized steel plates, shown in figure 4.3, in the mortar joint of the wall. As the masonry compression strain increases, the masonry dilates, and the tensile strain in the plates increases. In turn, the masonry is placed in a state of tri-axial compression, thus increasing the strength and ductility of the masonry. The galvanized plates, placed within the mortar bed joints, restrict the lateral expansion of the joint and the differential expansion between the clay brick unit and the joint. As a result, the plates inhibit vertical splitting cracks caused by tensile forces introduced into the clay brick unit by the differential expansion of the mortar joint and brick.

The dimensions of the standard confinement plate implemented were 380mm X 230mm X 3.2mm. A 230mm by 125mm centrally located hole was precut into plate to allow grout and reinforcing bars to pass. 12-9.5mm holes were punched throughout the plate to improve bond with the mortar. Also the plates were design to protrude 7mm into the grouted cavity to provide additional confinement to the core. The design and details of the confinement plates are shown in figure 4.2.
After evaluating the mechanism, it was concluded that placing galvanized steel confinement plates at every course over the entire height of the wall would be unnecessary, inefficient design. Therefore the confinement plates were only placed in mortar joints within the bottom 610mm of the wall corresponding with the theoretical plastic hinge region of the wall. Various galvanized, steel confinement plate ratios were implemented in respective specimens to better estimate the optimum utilization of the plates. Specimens 1 and 5 were control tests with no confinement steel. Specimens 2 and 6 had a volumetric confinement ratio of .015, which corresponds to spacing at every other course in the plastic hinge region or 140mm centers. Specimens: 3 and 7 contained a confinement steel volumetric ratio of .003, with specimen 4 containing a ratio of .034, both corresponding to confinement plates placed at every course of the wall in the plastic hinge region or 70mm centers. A complete testing matrix is show in table 4.2. In the matrix, the longitudinal and transverse are calculated as a function of area, while the confinement steel is shown as a function of volume. Equations 4.1-4.3 demonstrate how the respective steel ratios were calculated. The area of the wall was defined to be the gross area of the wall, in the direction being considered minus the displaced area from the steel reinforcement. Two values are given for the transverse steel within the wall for specimens 1-4 due to the different spacing used throughout the wall.

\[ \rho = \frac{A_l}{A_{wl}} \]

*Equation 4.1*

\[ \rho_l = \text{Longitudinal steel ratio} \]

\[ A_l = \text{Area of longitudinal steel reinforcement within the section} \]

\[ A_{wl} = \text{Cross sectional area of the wall} \]
\[
\rho_t = \frac{A_t}{A_{wt}} \quad \text{Equation 4.2}
\]

\(\rho_t\) = Transverse steel ratio

\(A_t\) = Area of transverse steel reinforcement within the effective section

\(A_{wt}\) = Effective height of the reinforcement * Width of the wall

\[
\rho_c = \frac{V_c}{V_w} \quad \text{or} \quad \rho_c = \frac{V_c}{L_c \cdot S_c \cdot b} \quad \text{Equation 4.3}
\]

\(\rho_c\) = Volumetric confinement steel ratio

\(V_c\) = Volume of confinement steel reinforcement

\(V_w\) = Effective volume of the wall containing the plates

\(L_c\) = Length of the confinement plate

\(S_c\) = Spacing of confinement

\(b\) = width of the wall

The current construction technique requires that the standard, rectangular confinement plate be threaded over the top of reinforcing steel and placed in its corresponding position between the transverse steel as depicted in figure 4.3. This current procedure can be quite awkward and time consuming, potentially increasing the construction cost of masonry structural walls. Therefore a different confinement plate configuration was implemented in specimen 4 to ease construction. The plate, referred to as the “Durham Plate”, shown in figure 4.4. has an open end, which allows for simple placement of the confinement plate in the desired location without any significant conflicts with any steel reinforcing bars. The plate style needed to be evaluated before implementation due to well established torsional theory, which states a closed section provides a significant increase in torsional rigidity than an open section. Therefore the
open plate may not confine the section as well the typical closed rectangular plate. To provide anchorage and add more torsional resistance the open section was shaped into a flanged segment.

![Diagram of confinement plates](image1)

**Figure 4.2 Confinement Plates Details**

![Implementation of standard rectangular confinement plate](image2)

**Figure 4.3 Implementation of Standard Rectangular Confinement Plate**
4.3 Footing Design

The dimensions of the footing were selected with respect to pre-fabricated forms that existed at the testing facility. The forms allowed footings 1219mm wide and 2438mm long to be cast. With the nominal footing dimensions selected, the reinforcement was designed to resist the load transmitted by the wall, as shown below in figure 4.5.

The flexural reinforcement consisted of 8-D25 on the top and bottom of the footing, in the direction of loading to resist the demand of transportation and induced loading during testing. The transverse steel consisted of 12-D16 bars on the top and bottom of the footing to resist the shear demand. To further increase the shear capacity of the footing, 11-D10, referred to as J-hook bars, were hooked around both sides of the flexural reinforcement. The longitudinal reinforcement from the masonry wall extended...
into the footing, resting along the bottom flexural reinforcement to provide anchorage for the steel and increase the stability of the wall.

Performing a section analysis on the footing demonstrated that the footing was adequately designed to accommodate forces transmitted by the wall, seen in figure 4.5, thus ensuring failure would occur in the clay masonry wall. Even though the reinforcement was designed to resist the applied forces, the footing was prestressed to 448kN, to increase the factor of safety to further avoid a footing failure.

![Figure 4.5 Footing Design](image)

**4.4 Cap Beam Design**

The load applying actuator was attached to a cap beam, resting atop of the structural wall, to transmit forces to the wall. The initial dimensions of the cap beam were 1372mm long X 457mm wide X 610mm deep. To ease construction, the cap beam for the first four specimens was cast on the ground. A 51mm void was created in the
center of the beam to allow the wall longitudinal reinforcement to pass through to obtain compatibility between the cap beam, wall, and footing. The void was formed by placing a styrofoam board wrapped in polyethylene to reduce bond with the concrete, in the desired location. The styrofoam board was held in place by steel reinforcement and accordingly removed after casting.

Minimal reinforcement was needed in the cap beam; to transmit the forces applied from the hydraulic actuator. The reinforcement cage was designed to control cracking and to help distribute the actuator loads into the wall. The reinforcement consisted of 6-D16 bars in the horizontal direction and D13 bars on 200mm centers in the vertical direction. 3-D10 were placed on each side of the styrofoam void material to keep it from moving.

When the forms were stripped, the cap beams were eventually placed on top of the respective walls. A small layer of mortar was placed in between the wall and the beam to create adequate bond. Approximately 51mm was intentionally left ungrouted in the wall so proper bond could be developed between the wall and the cap beam when the void in the cap beam was grouted, to eliminate lifting off the cap from the structural wall.

This process was abandoned for the final 3 specimens to minimize alignment imperfections from placing the cap on the wall after casting. Alignment imperfections were a major source of torsional rotations noticed in the first four tests. The cap for the latter specimens were cast in placed, also eliminating the need to create a central void. In addition, the length of the cap beam was extended to 2286 mm to allow the actuator to bolt directly to the cap without the use of an extension beam in an attempt to again minimize torsional effects, this modification will be further discussed in Chapter 5.
4.5 Material Strength

The compression strength of the clay masonry units was evaluated by a standard compression test. The prisms constructed were double wythed with a center cavity fully grouted with concrete to emulate the construction of actual specimens. Three classes of prisms were evaluated for the three confinement cases including no confinement, confinement at 140mm centers and confinement at 70mm centers. Three tests were performed for each case and then averaged providing the actual masonry strength used in this project. The average strength for each case is presented in the following table below.

<table>
<thead>
<tr>
<th>PRISM TYPE</th>
<th>$f_m'$ (Mpa)</th>
<th>$\varepsilon_{\text{ultimate}}$</th>
<th>$\varepsilon_{50%}$</th>
<th>$\varepsilon_{20%}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>28.352</td>
<td>0.00157</td>
<td>0.00366</td>
<td>0.00731</td>
</tr>
<tr>
<td>2</td>
<td>22.572</td>
<td>0.00189</td>
<td>0.00641</td>
<td>0.01037</td>
</tr>
<tr>
<td>3</td>
<td>26.786</td>
<td>0.00163</td>
<td>0.00487</td>
<td>0.00856</td>
</tr>
<tr>
<td>mean</td>
<td>25.903</td>
<td>0.00170</td>
<td>0.00498</td>
<td>0.00875</td>
</tr>
<tr>
<td>st. dev.</td>
<td>2.989</td>
<td>0.00017</td>
<td>0.00138</td>
<td>0.00154</td>
</tr>
<tr>
<td>Alternate Course Confined</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>30.483</td>
<td>0.00260</td>
<td>0.01354</td>
<td>0.02046</td>
</tr>
<tr>
<td>2</td>
<td>29.455</td>
<td>0.00355</td>
<td>0.01654</td>
<td>0.05878</td>
</tr>
<tr>
<td>3</td>
<td>31.124</td>
<td>0.00272</td>
<td>0.01535</td>
<td>0.02409</td>
</tr>
<tr>
<td>mean</td>
<td>30.354</td>
<td>0.00296</td>
<td>0.01514</td>
<td>0.03444</td>
</tr>
<tr>
<td>st. dev.</td>
<td>0.842</td>
<td>0.00052</td>
<td>0.00151</td>
<td>0.02115</td>
</tr>
<tr>
<td>Every Course Confined</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>35.655</td>
<td>0.00634</td>
<td>0.02679</td>
<td>0.04031</td>
</tr>
<tr>
<td>2</td>
<td>35.193</td>
<td>0.00564</td>
<td>0.03963</td>
<td>0.05850</td>
</tr>
<tr>
<td>3</td>
<td>35.972</td>
<td>0.00632</td>
<td>0.03663</td>
<td>0.04898</td>
</tr>
<tr>
<td>mean</td>
<td>35.607</td>
<td>0.00610</td>
<td>0.03435</td>
<td>0.04926</td>
</tr>
<tr>
<td>st. dev.</td>
<td>0.392</td>
<td>0.00040</td>
<td>0.00672</td>
<td>0.00910</td>
</tr>
</tbody>
</table>

A tension test was performed on the wall longitudinal reinforcement and confinement plates to obtain critical steel properties such as yield stress yield strain, ultimate stress, maximum elongation before rupture, and strain hardening characteristics. Figure 4.6 illustrates the actual stress-strain behavior the longitudinal steel used in the projects. The Mander model for predicting the stress-strain behavior of steel is also
plotted using the actual steel properties for confirmation of the test results. Figure 4.7 depicts the actual stress-strain response of the confining steel.

![Stress-Strain Curve for Longitudinal Reinforcement](image)

Figure 4.6 Stress-Strain Curve for Longitudinal Reinforcement
Figure 4.7 Stress-Strain Curve for Steel Confinement Plate
Table 4.2 Test Matrix

<table>
<thead>
<tr>
<th>Wall</th>
<th>Steel Reinforcement Ratio (%)</th>
<th>Volumetric Confinement Ratio (%)</th>
<th>Material Strengths</th>
<th>Masonry Prism</th>
<th>Longitudinal Steel</th>
<th>Confinement Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Longitudinal</td>
<td>Transverse</td>
<td>Longitudinal</td>
<td>Transverse</td>
<td>Longitudinal Steel</td>
</tr>
<tr>
<td>1</td>
<td>1.2</td>
<td>0.25 (#@152mm centers over lower 622mm of wall)</td>
<td>0.13 (#@330mm centers over upper 1512mm of wall)</td>
<td>-</td>
<td>25.9</td>
<td>429</td>
</tr>
<tr>
<td>2</td>
<td>1.2</td>
<td>0.25 (#@152mm centers over lower 622mm of wall)</td>
<td>0.13 (#@330mm centers over upper 1512mm of wall)</td>
<td>1.5 (@140mm centers over bottom 616mm of wall)</td>
<td>30.4</td>
<td>429</td>
</tr>
<tr>
<td>3</td>
<td>1.2</td>
<td>0.25 (#@152mm centers over lower 622mm of wall)</td>
<td>0.13 (#@330mm centers over upper 1512mm of wall)</td>
<td>3.0 (@70mm centers over bottom 816mm of wall)</td>
<td>35.6</td>
<td>429</td>
</tr>
<tr>
<td>4</td>
<td>1.2</td>
<td>0.25 (#@152mm centers over lower 622mm of wall)</td>
<td>0.13 (#@330mm centers over upper 1512mm of wall)</td>
<td>3.4 (@70mm centers over bottom 816mm of wall)</td>
<td>35.6</td>
<td>429</td>
</tr>
<tr>
<td>5</td>
<td>0.46</td>
<td>0.33 (#@152mm centers)</td>
<td>-</td>
<td>-</td>
<td>25.9</td>
<td>429</td>
</tr>
<tr>
<td>6</td>
<td>0.46</td>
<td>0.33 (#@152mm centers)</td>
<td>1.5 (@140mm centers over bottom 616mm of wall)</td>
<td>-</td>
<td>30.4</td>
<td>429</td>
</tr>
<tr>
<td>7</td>
<td>0.46</td>
<td>0.33 (#@152mm centers)</td>
<td>3.0 (@70mm centers over bottom 816mm of wall)</td>
<td>-</td>
<td>35.6</td>
<td>429</td>
</tr>
</tbody>
</table>

* Durham Confinement Plate
Figure 4.8 Elevation View of Test Specimen 1-4
Figure 4.9 Elevation View of Specimen 5-7
5.0 Instrumentation

All seven specimens in this investigation were instrumented to acquire an accurate record of the response of each wall under severe seismic distress. The measuring instruments used can be divided into two categories: internal instruments and externally applied instruments. The internal instruments consisted of strain gauges glued to the reinforcing and confining steel to measure the respective elongation of the steel. The external instruments consisted of a linear potentiometer, a string potentiometer, and a load cell. The potentiometers are used to measure a change in displacement over a given region, while the load cell measured the load applied to the wall from the actuator.

5.1 Strain Gauges

All strain gauges used in this study were made by Tokyo Kenkyujo Co, Ltd. The gauges had a gauge factor of $2.13 \pm 1\%$ with a resistance of $122\Omega$. The gauges for the first four test had wire leads soldered on them, while the leads had to be soldered to the gages for the final 3 specimens. The gauges were attached to the steel reinforcement in the procedure described below:

(1) A mechanical disk grinder was used to create a smooth surface on the steel to place the strain gauge.

(2) Sandpaper was then used to polish the surface to allow better bond between the surface and the steel.

(3) Excess dirt and grease was removed from the prepared surface with Methyl Ethyl Keytone (MEK).
(4) The gauges were then checked before application by measuring the resistance across the wire leads with a voltage-meter. If the gauge is function properly 122Ω should be read by the voltage-meter.

(5) After inspection, scotch tape was attached to the dummy end of the gauges, to keep the gauge in position until the subsequent applied glue began to set.

(6) The respective gauge with scotch tape still attached was glued to the reinforcement with Cyanoacrylate-Y glue made by Tokyo Sokki Kenkyujo Co., Ltd.

(7) A zip tie was positioned around the wire lead to hold the gauge in place and after the glue set, the scotch tape was removed from the gauge.

(8) Using a metal pick the metal lead from the gauge was lifted off the reinforcement to prevent the gauge from grounding out.

(9) Three separate coats of M-Coat D, manufactured by Micro Measurements Group LTD, was spread over the gauge for protection.

(10) After the M-Coat D dried sufficiently, the gauges were covered with Scotch Seal 2229 compound made by 3M providing an extra layer of protection.

(11) A final check was then done on the gauges by again reading the resistance across the wire leads with a voltage-meter.

(12) Each gauge was then labeled to identify its location.

(13) During construction of the walls, the gauges were routed out of the specimen through a specified 19mm gap, in the wall, 610mm above the footing in the center of the wall.

The strain gauges were placed at various locations on the reinforcing and confining steel to capture the response of the steel throughout the experiment. To track
the various gauges, labels were assigned to each gauge to describe the respective gauge according to location in the specimen. The various notations used for the different steel components within the masonry wall, such as the longitudinal reinforcement, transverse reinforcement, and confining steel is thoroughly described in the following paragraphs.

In the first four specimens, only the longitudinal bars at quarter points, i.e. bars 1, 5, 9, 13, were instrumented. Only two gauges on separate bars are actually needed to generate an accurate strain profile of the wall. Therefore placing gauges on all 13 bars was deemed inefficient and unnecessary, so it was decided that four gauged bars would be sufficient. Three gauges were placed at various vertical locations on each gauged bar. The first level corresponds to the top of the footing, the second level was 305mm above the footing, and the third location was 610mm above the footing level.

On the longitudinal steel, the notation consisted of a letter followed by two numbers. The letter represented the type of reinforcement. For example on the longitudinal steel the letter “L” was used to allocate the gauge. The first number designates the bar the gauge is attached to with respect to the direction of loading. As previous mentioned, only four bars at the quarter points were instrumented. The numbering system begins at the extreme north bar (closest to the load applying actuator) progressing to the extreme south bar. For instance the extreme north was designated by the number 1, the fourth bar from the north end was designated by the number 2, 3 was assigned to the bar at the three quarter location, and 4 to the extreme south bar. The last number refers to the vertical location of the gauge. The number 1 refers to a gage placed at the footing level, 2 refers to the second level of gauges at 305mm above the footing, and 3 refers to top gauge level.
In the final 3 specimens, only 5 longitudinal reinforcing bars were located within the section. Since only a small number of bars were present, all the longitudinal bars in these tests were instrumented. The bars were numbered in the same manner described in the above paragraph.

Gauges were placed on the transverse steel located in the bottom 610mm of the wall, with respect to the theoretical plastic hinge region of the wall. One gauge was placed in the center of the bar. The labeling notation is similar to the longitudinal steel but the last number is not needed because only one gauge is used per bar. Therefore a typical label for a transverse bar may read T1. In a similar manner the letter “T” represents a transverse bar with the number 1 corresponding to the first bar from the

Figure 5.1 Strain gauges attached to various steel components
footing in the plastic hinge region. In the first four tests, the first transverse bar, designated by the number 1 is located 152mm above the footing. In the final three specimens, a transverse bar was placed at the footing level and therefore denoted as bar 1.

Strain gauges were placed on both webs of the stainless steel confinement plates at 140mm and 280mm above the footing on both the north and south face of the wall. The labeling notation for the plates varies somewhat from the steel rebar. The notation consists of a letter, followed by a number, then two more letters. An example of a label corresponding to a plate is P2SW. Just as in the rebar the first letter stands for plate denoting the type of reinforcement. The number “either 1 or 2” corresponds with the location of the gauge above the footing. The example gauge has the number 2 representing a gauge 280mm above the footing. The next letter depicts the face of the wall the gauge is located. The plates were only placed in the regions with the highest compressive strains, which corresponds to either the north or south face depending on the direction of loading. Therefore the third letter in the example shows that gauge is placed on a plate on the south face of the wall. The fourth letter represents the web, east or west, of the gauge is positioned. In the example, the gauge is located in the middle of the west web. The nomenclature and positioning of all the gauges are shown in figures 5.4-5.8

5.2 Linear Potentiometers

A linear potentiometer (linear pot) is an instrument used to measure a change in displacement over a given gauge length. For the seven experiments conducted in this study 8 linear pots were attached to the north and south face of the wall to capture the change in displacement over 200mm cells. The displacements obtain from the linear pots
were then used to evaluate the curvature of the wall, which would provide a method to assess wall displacements.

As the walls were constructed, 6.4mm rods were placed at pre-defined locations, to give a means to attach the linear pots to the wall. The threaded rods used were approximately 305mm long with 153mm of the rod inserted inside the wall to allow sufficient development length to prevent the rods from pulling out and minimize localized movement. Four sets of rods, with each set consisting of two rods as seen in Fig 5.2 along with the linear pots, were placed on the north and south faces of the respective wall. The first set was placed in the mortar joint of the wall approximately 200mm above the footing, with the subsequent set placed 200mm above the previous.

The actual potentiometer was attached to pre-fabricated angle brackets, which were in turn mounted to the threaded rods. Each linear pot rested on the angle bracket below it, except for the extreme bottom pot, which rested on an aluminum strip attached to the footing. Wire leads from the potentiometer were connected to the data acquisition system, providing means to record and monitor the displacements.

From the recorded change in displacements of the linear potentiometers, the average angle of rotation of the structure over a given gauge length or cell was calculated from simple geometry principles using the following expression.

\[
\Theta_i = \frac{\Delta NP_i + \Delta SP_i}{L_i}
\]  

(Equation 5.1)

\(\Theta_i\) = Average angle of rotation

\(\Delta NP_i\) = North linear pot change in displacement

\(\Delta SP_i\) = South linear pot change in displacement
\( L_i \) = Horizontal distance between the north and south pots

\( i \) = Cell of consideration

After the respective average angle of rotation was obtained for each cell, the results could be used to attain the corresponding curvature for each cell by the following equation using geometry principles.

\[
\Phi_i = \frac{\dot{\varepsilon}_i}{G_i}
\]  

\( \Phi_i \) = Average curvature

\( G_i \) = Gauge length

The moment-area method was used to calculate the displacement after the curvature was obtained. Constant curvature was assumed over the entire gauge length of each cell. A linear curvature distribution was assumed in the region above the highest located cell (813mm above the footing). This assumption was deemed reasonable because the curvature at that level and higher is well above the inelastic plastic hinge region of the wall. The displacement was then calculated by summing the moment of areas of the composite curvature diagram about the effective height of the wall (at the center of the cap beam). Figure 5.3 illustrates this calculation.
5.3 String Potentiometers

Two string potentiometers were used in the experiments to measure in-plane and out-of-plane horizontal displacements. Pots of various strokes were used to record the in-plane displacements based on estimated displacements to get the most accurate possible reading, while a 1270mm was used to capture the out-of-plane displacements. The size of the out-of-plane pot was due to laboratory constraints not expected displacements. A leader line was used to connect the string pots to the center of the cap. In the first four tests, the out-of-plane was connected to the middle of the wall due to laboratory constraints and the maximum displacement was linearly estimated.
5.4 Load Cell

A 980kN load cell was attached to the hydraulic actuator used in all the experiments. The load cell monitored the force applied to the structure.

Figure 5.3 Curvature distribution assumed for deflection calculations
Figure 5.4 Location of Longitudinal Strain Gauges: Test 1-4

Figure 5.5 Location of Longitudinal Strain Gauges: Test 5-7
Figure 5.6 Location of Transverse Strain Gauges: Test 1-4

Figure 5.7 Location of Transverse Strain Gauges: Test 5-7
Figure 5.8 Location of Confinement Strain Gauges
6.0 Test Setup

After the specimens were constructed, the center cavity was fully grouted adding additional strength to the section and provided complete bond between the reinforcement and the clay masonry wall. The walls were painted with a flat latex paint to facilitate crack detection and marking. After the grout was granted sufficient time to cure and reach reliable strengths, the specimens were moved to the desired testing location, where it prepared for testing. The setup procedure consisted of leveling and anchoring of the footing, connecting the measuring instruments to a data acquisition system, mounting the actuator to the specimen, and for the majority test setting up a detailed lateral bracing system.

Four 51mm holes were placed in the corners of each footing to provide the capability to anchor the footing to the 762mm thick laboratory strong floor. The voids in the footing was designed to align with the hole pattern in the strong floor which allows test specimens and lab equipment to bolt through the strong floor. The voids in the footing were created by casting 51mm PVC tubes in the desired location. PVC tubes were cast with the cap beam to provide similar holes for the actuator to bolt through. 51mm tubes matching the bolt pattern of the actuator were used for the first 4 specimens, but the second set of walls used 76mm tubes to provide additional tolerance to better align the actuator at the center of the cap beam.

6.1 Leveling and Anchoring the Footing

The test specimen was placed on 4-13mm thick, 51mm square fiber-sheathing pads to level the structure. 51mm holes were pre-drilled in the sheathing pads and
aligned with the appropriate holes in the strong floor to allow subsequently inserted dywidag to pass and anchor the structure to the floor.

![Figure 6.1 Test setup with lateral stability bracing](image)

Hydrocal was used to create sufficient bond between the floor and the specimen. It also ensured that the footing is resting on a smooth, level surface preventing the footing from rocking during loading, minimizing unnecessary, unintentional footing damage. A simple form constructed from 51mm X 102mm wooden beams was placed on the floor around the base of the footing, to allow the hydrocal to set. A bead of silicone was placed around the inside and outside corners to prevent leakage of the hydrocal mixture. After the silicone dried, the hydrocal powder was mixed with water at a 1 to 1 volumetric ratio and subsequently poured into the form.

A dywidag bar was used to secure the footing against to the strong floor. A steel plate and nut were placed on both sides of the bar (one on the footing and the other
beneath the laboratory floor) to secure the bar. Due to imperfections at the footing surface, hydrocal was again used to create a smooth level surface for the plate to rest on the footing. This procedure was achieved by placing several layers of silicone around each plate, creating a small dam. Then the same hydorcal mixture used between the floor and footing was poured under the plates in the dam.

After the hydrocal set, a 35mm bar was place through the pre-cast holes at each corner of the footing and the floor, with a steel nut and plate on both end as previously mentioned. The nuts were hand tightened, then the bar was tensioned to 334kN using a .534mN ram, anchoring the footing to the strong floor.

### 6.2 Stability Bracing Set-up

Significant out-of-plane deformations were noticed during the first two experiments and in early inelastic cycles of specimen 3. To prevent this mechanism from having a significant effect on the remaining tests, a bracing frame was set up to restrain out-of-plane deformations. The first attempt was not successful because a steel extender beam, which was introduced to allow maximum actuator stroke capacity in both directions of loading, would not permit beams to be placed at the cap beam. The extender beam was removed for the final three specimens, so new bracing scheme was devise that prove to be more successful.

In the first bracing system, 4 - W10X60 steel columns were attached to the strong floor with a 25mm dywidag bar secured with a steel plate an nut attached to the column base and beneath the strong floor. The columns were located at each corner of the footing 914mm in front and 914mm over from the respective dywidag bars securing the footing to the floor. Two 102mm steel tubes were attached to the two sets of columns to
transfer the load to the columns. A C10x30 section, attached to the steel tubes, was
strung along each side of the wall to restrict the out-of-plane deformations, as shown in
figure 6.3. The channel was sufficiently lubricated with a grease compound to reduce
any frictional forces that may be generated between the wall and the channels that could
lead to a fictitious increase in strength capacity. A 3mm gap was placed in between the
wall and the channels so the channels would not restrict the natural deformation tendency
of the wall but merely act as a guide. The steel extender beam protruded from the cap
permitting bracing along the cap beam. Therefore channels were placed 25 mm below the
cap beam to accommodate in-plane rotations. Since, the bracing was not able to be
placed along the cap out-of-plane rotations were not constrained and hindered the results
of specimens 3 and 4.

No bracing was initially used in specimen 5 as seen in figure 6.4 because it was
felt that other revisions taken in the setup and construction phase would control the out-
of-plane response of the wall. As the wall was loaded deep into the inelastic range out-
of-plane deformation began to become significant, so the experiment was paused. Straps
were wrapped around each side of the wall and attached to a steel cable and pulley. The
steel cable and pulley system was attached to the floor one side, while the other side was
attached to a steel frame. The steel cranks provided sufficient resistance against out-of-
plane displacements, but were deemed unsafe because the tension strains induced to the
steel wire were unknown. So there was no way to anticipate a failure, if the maximum
strain were being approached.

Due to safety concerns of the steel cable and pulley, a new bracing system was
devised for specimens 6 & 7, as shown in figures 6.1 and 6.4. The column set up was
identical to the system used for specimens 3 and 4. With the removal of the extender beam, a W24x68 section, which attached directly to the column was strung along the cap beam with a 3mm gap as in the earlier set up. A 19mm teflon strip was attached to the east and west face of the cap beam. The W-section was positioned that if the beam came in contact the bracing, the flange of the beam would rest against the teflon strip reducing frictional forces. The set up proved to reliable means of limiting out-of-plane displacements.

6.3 Data Acquisition

All measuring instruments used in this study were connected to an Optim Megadac 31080AC Data Acquisition with a Windows based TCS program, which collected the respective data for each specimen. For the first five specimens the Megadac was calibrated to collect three data points per second. To reduce the size of the final data file for the specimen the Data Acquisition was re-calibrated to take one reading per second.

6.4 Mounting the Actuator

Reversed, cyclic horizontal force was applied to the clay masonry wall by a 980kN MTS hydraulic actuator. The actuator had a maximum stroke capacity of 1016mm with the ability to apply excitation with respect to specified loads or displacements. For the first four specimens a steel extender beam was attached to the cap beam to allow maximum stroke capacity in both loading directions. Therefore the actuator was subsequently bolted to the extender beam which would transfer the applied
force to the cap. A 51mm steel plate was positioned in between the actuator and extender beam to ensure the load was evenly distributed to the beam.

The steel extender beam was removed from the final three specimens because it introduced a slight misalignment between the actuator and the structure, introducing torsional effects. Consequently in the final set of specimens, the actuator was attached to the center cap beam through the holes cast in the cap beam that match the bolt pattern of the actuator head. The knuckle of the actuator was loosened to allow pivoting, permitting the actuator to close any gaps between the knuckle and cap ensuring the wall is evenly loaded. Four 38mm threaded rods were used to secure the actuator to the north side of clay masonry wall. The rods were attached to steel plates on the south face of the cap beam, to distribute and transfer the load to the cap.

Figure 6.2 Initial setup with steel extension beam and no lateral bracing
Figure 6.3 First setup revision: lateral bracing provided by C10x30 steel channel

Figure 6.4 Test setup for specimen 5: longer cap/no extension beam
Figure 6.5 Final setup with steel W24X68 along cap beam for lateral stability
7.0 Test Specimen 1

This study is broken down into two separate sets of specimens. The first set of specimens, consisting of the first four clay masonry walls, were constructed with a relatively high longitudinal reinforcement ratio of 1.5%. The second set consists of specimens 5-7 and contained a longitudinal reinforcement ratio of 0.6%. The first of the four specimens was tested was a control specimen with no confinement steel. The basis of the test was to provide a standard for comparison for the remaining specimens in the set, which contain various volumetric containing ratios. This chapter will cover the loading of the structure, observations of the experiment, the force-deformation response, and summary and suggestion for the subsequent experiment.

7.1 Loading of Specimen

The target-loading pattern consisted of elastic, force-based cycles up to the first yield; followed by inelastic cycles controlled by specified displacements until failure. Section analysis was performed on the masonry wall to obtain the yield force and displacements. The analysis provided the basis for the target force based cycles shown in figure 7.1.

The target inelastic loading pattern was based on the experimental equivalent yield point of the wall. Once the yield point was verified, the target inelastic loading pattern, as seen in figure 7.2 could be established. Due to a miscalibration of the load applying actuator, the wall was initial overloaded to 392kN leading to subsequent failure of the structure. Therefore the response under the target loading history could not be established and the specimen was subjected to the monotonic loading history shown in
However the unintentional mishap proved to be valuable in providing a monotonic response of the clay masonry structure, used in typical design applications.

Figure 7.1 Target force-controlled loading history

Figure 7.2 Target displacement-controlled loading history
Figure 7.3 Actual monotonic loading history for specimen 1

7.2 Observations

The experiment lasted approximately 27 seconds, therefore only brief observations were noted pertaining to the experiment. Figures 7.3 and 7.4 show a plot of force and displacements with respect to the elapsed time. From figures 7.5 and 7.6, it can be seen that at failure, the wall had experienced significant diagonal, shear cracking, out of plane buckling, and crushing in the extreme compressive toe region of the wall. Due to the nature of loading and various failure mechanisms present, the source of the failure is unknown.

Figure 7.4 depicts the force deformation response of the structure. The specimen reached a maximum force of 392kN at a displacement of 33.8mm with a corresponding drift of 1.4% drift. An immediate drop in strength was experienced after the maximum force was achieved.
Figure 7.4 Force vs. Time

Figure 7.5 Displacement vs. Time
Figure 7.6 Force-displacement response of specimen 1

7.3 Summary for the next test

As previously mentioned significant out-of-plane deformations were experienced. At this time, it was not known whether this response was an inherent characteristic of the wall or a function of the applied load. As a precaution, a string potentiometer was attached to the east face of the wall of subsequent test, to monitor out of plane deformations.

Another unexpected phenomenon was the significant diagonal shear cracking because shear steel was design to handle the shear capacity of the structure. Shear did not appear to govern the failure, so no significant action was taken or explored.
Figure 7.7 East Side of Specimen 1 after Test

Figure 7.8 West face of Specimen 1 after test
8.0 Test Specimen 2

Chapter 8 will focus on the second of seven specimens in this experimental investigation. The wall contained a longitudinal steel ratio of 1.5% and volumetric confinement ratio of 0.0023%. The confinement steel was spaced at 140mm centers over the first 610mm of the wall, which corresponds to one stainless, steel plate at every other course on the north and south faces of the clay, masonry wall. The 381mm long plate was placed in the mortar joint at the targeted locations to allow the development of sufficient bond.

8.1 Loading of Specimen

The specimen was loaded to the target loading history shown in figure 8.1, where the test specimen is respectively subjected to one cycle of force-control loading at 25%, 50%, 75%, and 100% of the theoretical first yield force. Afterwards the masonry wall was subjected to a displacement-controlled loading for inelastic range. Three cycles of increasing levels of displacement ductility were applied until apparent failure of the specimen as shown in figure 8.2.

8.2 Observations

Observations were noted throughout the entire experimental history. This section will cover the visual inspection of the behavior of the structure. The assessment will be broken down into two parts: Force controlled loading (elastic level), and Displacement controlled loading (inelastic level).
8.2.1 Force Controlled Loading

In the initial cycle of loading, a force of 49kN was applied to the specimen. An average displacement, from the push and pull directions, of 2.54mm was recorded. At
this time, small flexural cracks formed at locations where confinement steel was present. The cracks were observed at the base of the wall and over the first 560mm of the wall in the mortar joints.

The second cycle consisted of applying a force of 97.9kN to the wall. An average displacement of 5.96mm was measured. The behavior was characterized by the extension of the previously present flexural cracks.

On the third force-based cycle at a load of 147kN the base crack extended to span the entire length of wall. An average displacement of 6.86mm was recorded. Along with further extension of the present flexural cracks, new cracks formed over the height of the structure up to 978mm. Shear cracks also began at the present loading level. The cracks were noticed on the east face of the wall at height of 1524mm and propagated down the wall.

In the final force based cycle the structure was subjected to a 200kN load with a corresponding first yield displacement of 10.16mm. A significant drop in stiffness could be seen on the X-Y recorder with respect to the initial stiffness. The cracks were fairly, uniformly distributed at a distance of 140mm or at every course corresponding to the confinement location. A maximum crack width of .18mm was observed.

8.2.2 Displacement Loading

The first displacement-controlled cycle was at displacement ductility of 1. This value was determined from equation 8.1. The maximum displacements recorded in first yield cycle in the push and pull directions respectively were averaged to obtain the first yield displacement used to calculate the equivalent yield displacement. In specimen 2, the experimental first yield displacement was 10.2mm at a load of 200kN. The values
lead to an equivalent yield displacement of 15.2mm, which was then established as displacement ductility 1.

\[ \Delta_y = \Delta'_y \times \frac{F_u}{F'_y} \]  

---  

Equation 8.1

\( \Delta_y \) = Equivalent first yield

\( \Delta'_y \) = Experimental first yield

\( F_u \) = Theoretical ultimate force

\( F'_y \) = Experimental first yield force

The first displacement-controlled cycle was ductility 1, at displacement of 15.2mm. The maximum force associated with the displacement was 289kN. At this point, a maximum crack width of 1.52mm was located at the base of the wall. The majority of the cracks were located plastic hinge region with few an additional cracks forming outside the plastic hinge. The shear cracks continued to extend and widen as the cracks propagated down the wall.

On the second cycle, the wall was pushed to a displacement of 22.8mm corresponding to displacement ductility of 1.5. During the cycle a maximum force of 325kN was achieved. At the southeastern corner, crushing of the mortar began to initiate in the bottom course. The maximum wall crack width was measured at .51mm.

At the next cycle, the wall was pushed to 30.5mm at displacement ductility of 2. The maximum force attained during the cycle was 335kN. Noticeable crushing occurred at the extreme compressive toe on the north face of the wall. Wall crack widths were
measured at 1.27mm, while the base cracked opened to a width of 2.03mm. During the cycle shear cracking began became prominent throughout the structure.

During the following cycle, the wall displacements were increased to 45.7mm, a displacement ductility level of 3. The wall achieved its peak capacity of 365kN during the initial cycle at ductility 3. The face shell in the extreme compressive toe region of the south face began to crush and spall. The base crack opened to a width of 3.175mm. While the maximum wall crack was 1.524mm. By the third cycle noticeable crushing of the masonry began to occur on the north face of the wall.

The ensuing cycle was conducted at a displacement of 60.8mm, corresponding to a displacement ductility of 4. There was a slight decrease in recorded force capacity at 360kN. Huge tension cracks began to open within the wall and shear cracks up to 4.76mm were observed signaling a potential, imminent shear failure. Severe crushing continued to proceed in the compressive zones of wall on the north and south faces. Unconfined mortar became debonded and spalled off. By the third cycle, 97mm of the masonry face shell had spalled of the north and south faces of the wall, up to a height of 140mm.

On the last cycle, displacements were increased to 76mm. A maximum load of 298kN was obtained during the cycle. The load was 17% percent decrease from the peak load achieved at the ductility 3. Migration and widen of the initial shear cracks led to further wall degradation. By the second push cycle, the corresponding load recorded at 76m displacement was only reached 218kN (a 40% decrease from the peak force), therefore the test was ended. Even though significant crushing occurred due to flexural stresses, the ultimate failure was governed by a ductile sliding shear mechanism.
8.2.3 Curvature Distribution

The curvature at various levels of loading is shown in figures A-4.1 and A-4.2 for both directions of loading. Values were generated from the center height of each respective linear potentiometer. Readings for this specimen were prematurely stopped at ductility 2 due to technical complications.

The greatest concentration of curvature was experienced at the base of the wall. This should be expected because it is also the location of the peak strains and most intense damage. The curvature decreases over the height of wall as the moment demand is decreased.

8.3 Summary for the next specimen

Significant out-of-plane displacements were again noticed in the behavior of the specimen. The mechanism would begin at significant inelastic displacements. The maximum out-of-plane displacements would occur when the wall approached the zero
point for in-plane displacements and decreased as the wall was further loaded to the targeted in-plane deformation. From observations, the stability of the wall remained intact as long a slow, deliberate rate of loading was maintained. Therefore we would continue to monitor out-of-plane deformations with a string potentiometer and maintain a slow rate of loading. It was recognized that if the stability issue started to control the behavior of the wall, a bracing system would be needed to restrain the mechanism.

As previously stated, the final failure of the structure was due to a ductile shear mechanism. Even though, significant flexural deformations led to substantial crushing in the extreme compressive region of the wall. It was noted that a balanced flexural/shear failure was almost achieved. From the behavior of specimen 2, it was established that a shear mode would govern the next two specimens and that effectiveness of the confinement could not be completely evaluated because the intent of confinement is to delay the onset of crushing of masonry due to flexure. At this point, with the ensuing two specimens already constructed, no practical solution could be resolved to help resist the shear stresses. Therefore for the following test, the test objective was slightly modified to investigate if confinement steel could have an impact on the shear behavior on the clay masonry walls.
Figure 8.4  East Face of Specimen 2 at First Yield

Figure 8.5 West Face of Specimen 2 at Ductility 1-Cycle 3
Figure 8.6 West Face of Specimen 2 at Ductility 1.5-Cycle 3

Figure 8.7 First Crushing at Southeastern Corner at Ductility 1.5-Cycle 3
Figure 8.8 West Face of Specimen 2 at Ductility 2-Cycle 1

Figure 8.9 South Face of Specimen 3 at Ductility 3-Cycle 1
Figure 8.10 East Face of Specimen 2 at Ductility 3-Cycle 3

Figure 8.11 North Face of Specimen 2 at Ductility 4-Cycle 1
Figure 8.12 East Face of Specimen 2 at Ductility 4-Cycle 3
Figure 8.13 West Face of Specimen 2 at End of Test 2
9.0 Test Specimen 3

Chapter 9 is dedicated to the behavior of Specimen 3. The wall contained a longitudinal reinforcement steel ratio of 1.5% and a volumetric confinement ratio of 0.0046%. The rectangular confinement steel was spaced at 70mm centers over the first 610mm of the wall. Stainless, steel plates were placed in the mortar joint at every course on the north and south faces of the structure, at the targeted location.

9.1 Loading of Specimen

The targeted loading history, shown in figures 9.1 and 9.2 remained the same as specimen 2 and will be used throughout the entire experiment. The force-based loading followed the standard adopted loading history, but due stability complications the displacement-based loading followed the loading history shown in figure 9.3. In the loading history the structure was subjected to 3 cycles at increasing displacement ductilities up to a ductility level of 3. Afterwards the specimen was loaded monotonically in the pull direction until failure.

9.2 Observations

This section is devoted to the behavior observed in specimen 3. The assessment will follow the standard convention adopted in Chapter 8, in which the section will assess the step by step behavior of the wall throughout the entire test with respect to the force-based and displacement-based loading scenarios.
Figure 9.1 Target Force-Controlled Loading History

Figure 9.2 Target Displacement-Controlled Loading History
Figure 9.3 Actual Cyclic-Monotonic Loading History for Specimen 3

9.2.1 Force Controlled Loading

A force of 53.4kN was applied to the structure in the initial cycle of the structure. Under the applied force, the wall experienced an average displacement of 1.08mm. Small flexural cracks were detected at base of the structure.

On the ensuing cycle at 50% of the theoretical first yield, a force of 107kN was placed on the masonry wall. The wall displaced 3.26mm. More small flexural cracks were initiated. Cracks were observed over the first 838mm of the wall. The crack distribution appeared to be random, in contrast to specimen 2, where cracks developed at the location of confinement steel.

In the third cycle at 75% yield, a force of 160kN was applied. The average displacement recorded was 5.97mm. The behavior was characterized by the extension of the small flexural cracks. A few additional cracks were initiated over a height of 1327mm up the wall.
In the last force-based load, the wall was cycled at the theoretical yield of the structure. The applied force was 212kN with a corresponding first yield displacement of 9.01mm. The random crack distribution continued with the majority of the cracks taking place in the plastic hinge region of the wall. Crack widths up to .229mm were detected.

![Figure 9.4 Force-Displacement Response of Specimen 3](image)

**9.2.2 Displacement Based Loading**

15.24mm was established as displacement ductility 1 (the first displacement-based cycle) with respect to specimen 2 for comparison. The force at ductility 1 was 277kN. Shear cracking began to occur in the initial displacement-based loading. Cracks were now present over nearly the entire height of the wall.

At ductility 1.5, the wall displacement was increased to 22.9mm. The maximum force achieved during the cycles was 326kN. The flexural cracks were measured at .254mm, while shear cracks were measured at .1778mm. The base crack opened to 1mm.
at the loading level. The shear cracks began to become prominent throughout the structure, as they began to connect with flexural cracks.

The next cycles were conducted at ductility 2, with a displacement of 30.5mm. The maximum force achieved during the cycle was 306kN. The maximum shear crack was measured at 1.016 mm, in comparison to the flexural cracks at .330mm indicating the wall was becoming shear dominated. During the third cycle of loading, the first course of masonry began to crush from high compressive strains.

During the following cycles, the wall displacements were increased to 45.7mm, a displacement ductility of 3. The wall achieved a force 328kN in the present level of cycling. The maximum wall cracks were measured at 1.27mm.

After the second cycle at ductility 3 the experiment was paused due to severe out-of-plane deformations. A bracing system was established and utilized in an attempt to restrain the out-of-plane displacements of the wall. In an attempt to finish the current cycles at ductility 3, the wall attempted to twist within frames in the push direction causing premature crushing of the masonry coming in contact with the bracing frames. Therefore the structure was subsequently loaded monotonically in the pull direction until failure.

The first monotonic cycle was displacement ductility 4. A wall was pushed to a displacement of 60.8mm at a force of 287kN. The face shell continued to crush and spall on the south face.

At ductility 5 wall displacements were increased to 76.3mm. The corresponding force was 314kN. The severe cracking in the plastic region continued to progress.
The next targeted displacement was displacement ductility 6, at a displacement of 91.44. The structure carried a load of 325kN. Cracking continued to become more extensive but crushing was impeded by the implementation of confinement plates. The confined core remained intact delaying the onset of a flexural failure.

The next displacement level was 107mm corresponding to displacement ductility of 7. At the current level, the wall carried a force of 329kN. The shear cracks began to immensely widen. The crack widths were measured at 6.35mm. Only the face shell continued to spalled in the bottom courses of the north face as the core continued to resist significant damage.

At a displacement of displacement ductility of 8, the wall attained a force of 329kN. The shear cracks continued to widen. The mortar joints in the extreme compressive regions were crushed exposing the confinement plates.

The next level of loading was at a displacement ductility of 9. The corresponding load was 341kN. The behavior was characterized by extension and widening of the shear cracks and spalling of the masonry face shell with the concrete core remaining stiff.

At displacement ductility of 10, the wall achieved a displacement of 152.4mm. At the current displacement level, the demonstrated a drift of 6.1%. The force attained at the targeted displacement was 341kN. As the wall approached the target displacement the peak wall force of 345kN was achieved at 5.9% drift. The face shell of the masonry spalled up to a depth of 178mm over the first two courses. Shear cracks up to 12.7mm were measured.

Ductility 11 was the last cycle of the experiment. At a displacement of 167.64mm, the force dropped to 302kN, a 12.5% decrease from the maximum force.
Huge shear cracks caused the drop in strength and stiffness. In addition to the drop in strength, the massive cracks initiated wall instabilities so the test was ended.

**9.2.3 Curvature Distribution**

The curvature at varies levels of loading is shown in figures A-4.3 and A-4.4 for both directions of loading. Values were generated from the center height of each respective linear potentiometer. Readings for this specimen were stopped at ductility 3 in the positive direction of loading due to the testing complications that only allowed the specimen to be loaded in the pull direction.

The greatest concentration of curvature was experienced at the base of the wall. This should be expected because it is also the location of the peak strains and most intense damage. The curvature decreases over the height of wall as the moment demand is decreased.

**9.3 Summary for the next specimen**

When the wall is subjected to substantial out-of-plane displacements that it cannot recover from under its own strength, it becomes very difficult to properly re-align the structure introducing torsion into the specimen, as it is loaded. Therefore after the experiment, it was concluded that the wall must be braced from the beginning of the experiment to ensure that out-of-displacements will not impede the targeted testing procedure. Specimen four will be braced at the onset of testing to keep out-of-plane from occurring allowing the wall to remain in line with load applying actuator, reducing torsional effects.
Figure 9.5 East Face of Specimen 3 at First Yield

Figure 9.6 East Face of Specimen 3 at Ductility 1-Cycle 3
Figure 9.7 South Face of Specimen 3 at Ductility 1.5-Cycle 1

Figure 9.8 East Face of Specimen 3 Ductility 1.5-Cycle 3
Figure 9.9 East Face of Specimen 3 at Ductility 2-Cycle 1

Figure 9.10 South Face of Specimen 3 at Ductility 2-Cycle 3
Figure 9.11 East Face of Specimen 3 at Ductility 3

Figure 9.12 East Face of Specimen 3 at Ductility 4
Figure 9.13 East Face of Specimen 3 at Ductility 5

Figure 9.14 Northeastern View of Specimen 3 at Ductility 6
Figure 9.15 East Face of Specimen 3 at Ductility 7

Figure 9.16 East Face of Specimen 3 at Ductility 8
Figure 9.17 North Face of Specimen 3 at Ductility 8

Figure 9.18 East Face of Specimen 3 at Ductility 9
Figure 9.19 North Face of Specimen 3 at Ductility 9

Figure 9.20 East Face of Specimen 3 at Ductility 10
Figure 9.21 East Face of Specimen 3 at Ductility 11
10.0 Test specimen 4

Chapter 10 will focus on the behavior of Specimen 4. The wall contained a longitudinal reinforcement steel ratio of 1.5% and volumetric confinement ratio of .0046%. In this test, the Durham confinement plate was spaced at 70mm centers over the first 610mm of the wall. The plates were placed in the mortar joint at every course, just as specimen 3. The objective of this experiment was to compare the effectiveness of the modified open section plate to the closed, rectangular plate.

10.1 Loading of Specimen

The specimen was loaded with respect to the standard load history previously adopted shown in figures 10.1 and 10.2. However loading was prematurely terminated after the first cycle of displacement ductility 4 due to stability issues.

![Figure 10.1 Target Force-Controlled Loading History](image-url)
10.2 Observations

The observations on the behavior of specimen 4 will be presented in the following section. The assessment will follow a step by step account of the behavior of the wall throughout the entire loading history. The section will separated into force-based loading and displacement–based loading histories.

10.2.1 Force-Based Loading

On the first cycle, a force of 53.38kN was applied to the structure. The wall experienced an average displacement of 1.02mm. During the cycle a few, minor flexural cracks opened over the first 489mm of the masonry wall.

In the second cycle at 50% of the theoretical yield, a force of 107kN was placed on the specimen. The wall displaced 3.56mm. The previously present cracks were extended and a few new cracks were initiated.
In the third cycle at 75% yield, a force of 160kN was applied. An average displacement of 6.22 was recorded. More small flexural cracks were initiated. Cracks were observed over the bottom 1118mm of the wall.

At first yield, the wall was loaded to 212kN, with a corresponding displacement of 9.78mm. Previous cracks were extended, but no new cracks were initiated. At 1044mm up the structure, a flexural crack began to incline at a 45 degree signifying the first signs of shear cracking.

10.2.2 Displacement-Based Loading

The first displacement-based load was displacement ductility 1. The structure was pushed to a displacement of 15.24mm. The maximum load occurring during the cycles was 300kN. By the third cycle flexural cracks extended the entire length of the wall and shear cracking was becoming prominent.

At displacement ductility 1.5, the wall displacement was increased to 22.9mm. The maximum force achieved at the present level was 334kN. Cracks widths up to .254mm were measured. Initial signs of crushing of the bottom mortar joint were observed.

The next level of cycles was conducted at ductility 2. The displacement was increased to 30.5mm. The wall achieved a force of 327kN. Shear cracks up to .635mm were measured. On the third cycle the entire face shell of the bottom course of masonry had spalled.

Displacement ductility 3 was the last full cycle completed. The masonry wall was pushed to a displacement of 45.7mm. The maximum force obtained during the cycle was
324kN. Flexural cracks up to 1.27mm were observed throughout the wall, while shear cracks up to 1.524mm were measured. The second course of masonry on the south face of the specimen was crushed. On the north side the bottom layer of mortar was crushed and spalled up to a depth of 387mm. The mortar joint at a height 279mm also spalled over a depth of 290mm.

![Figure 10.3 Force-Displacement Response of Specimen 4](image)

**10.2.3 Curvature Distribution**

The curvature at varies levels of loading is shown in figures A-4.5 and A-4.6 for both directions of loading. Values were generated from the center height of each respective linear potentiometer. Readings for this specimen were stopped at ductility 2 in due to the testing complications.

The greatest concentration of curvature was experienced at the base of the wall. This should be expected because it is also the location of the peak strains and most
intense damage. The trend of curvature distribution showed a decrease in the level of curvature up the height of wall as the moment demand decreases.

**10.3 Summary for the next test**

After the test, it was concluded that a torsional effects induced the initial test set up must be eliminated to control the out-of-plane deformations. It was reasoned that if no forces are applied in the out-of-plane direction, no such deformations could occur. A longer cast-in-place cap beam was employed to eliminate the extender beam and reduce misalignment between the cap and the wall in the next set of walls. Also a laser would be used to properly align the structure and the actuator to reduce torsional effects and out-of-plane effects.

The different loading histories between specimens 3 and 4 coupled with the premature stoppage of test 4 made it impossible to compare the effectiveness of the two plate styles. Since no conclusion could be drawn on the effectiveness of the Durham plate, the standard rectangular plate will be used for the remainder of the test.
Figure 10.4 West Face of Specimen 4 at First Yield
Figure 10.5 West Face of Specimen 4 at Ductility 1-Cycle 3

Figure 10.6 East Face of Specimen 4 at Ductility 1.5-Cycle 3
Figure 10.7 South Face of Specimen 4 at Ductility 1.5-Cycle3

Figure 10.8 East Face of Specimen 4 at Ductility 2-Cycle 1
Figure 10.9 South Face of Specimen 4 at Ductility 2-Cycle 3

Figure 10.10 West Face of Specimen 4 at Ductility 3-Cycle 3
11.0 Test Specimen 5

Chapter 11 will focus on the behavior of Specimen 5. Specimen 5 is the first wall to be tested in the second set of experimental masonry walls. The second set of specimens consisted of three clay masonry walls containing 5-D19 bars grade 60 ASTM longitudinal reinforcement bars (a reinforcing ratio of 0.6%). The transverse reinforcement was increased to 14-D13, grade 60 ASTM, with a 90-degree hook at each to ensure a flexural failure would occur.

Specimen 5, the first of the three specimens to be tested, was a control specimen with no confining steel. The specimen will provide a standard for comparison for the final two walls with varying volumetric confinement ratios.

11.1 Loading of the specimen

The specimen was loaded in accordance with the standard loading history adopted in the previous tests. In the load histories, shown in figures 11.1 and 11.2, the specimen is subjected to one cycle of force-control loading at 25%, 50%, 75%, and 100% of the theoretical first yield lateral force. Afterwards, displacement control loading is employed, where the specimen is cycled three times at increasing levels of displacement ductility until failure.

11.2 Observations

The observations noted, with respect to the behavior of the specimen, will be presented in the following section. The observations will track the behavior of the wall throughout the entire test at each loading level. As in previous chapters the observations
will be presented in two separate sections corresponding to the force-based and displacement-based loading respectively.

Figure 11.1 Target Force-Controlled Loading History

Figure 11.2 Target Displacement-Controlled Loading History
11.2.1 Force Controlled Loading

At the first cycle at 25% yield, a force of 22.2kN was applied to the wall. Under the initial load the wall displaced 1.02mm. Small cracks were opened at the base and at respective heights of 140mm, 349mm, and 559mm up the wall.

On the second cycle conducted at 50% yield, a force of 46.5kN was placed on the specimen. An average displacement of 2.29mm was recorded. New cracks were opened on the north side or the wall, while the previous cracks were extended.

In the next cycle, at 75% yield, the specimen was subjected to 69.0kN force. The wall experienced a displacement of 3.43mm. The flexural cracks began to form a fairly uniform distribution pattern. Essentially the cracks were opening at a spacing of 210mm over the bottom 559mm of the wall.

At first yield the wall was loaded to 93.0kN with a corresponding displacement of 4.83mm. The behavior of the wall was characterized by the addition of a few new cracks and the extension of several previously present cracks.

11.2.2 Displacement-Based Loading

From the method established in section 8.2.2, displacement ductility 1 for specimens 5-7 were calculated to be 9.40mm. In the initial displacement-based ductility level a force of 132kN was obtained. The uniform distribution of cracking continued with a maximum crack width of 2.29mm. By the third cycle, the section experienced cracking spanning the entire length of the specimen up to a height of 768mm.

At displacement ductility 1.5, the wall displacements were increased to 14.1mm. The maximum force achieved at the current loading level was 149kN. Cracks widths of
.762mm were observed. The first sign of crushing of the bottom layer of mortar was witnessed.

The next level of cycles was conducted at ductility 2 with a wall displacement of 18.8mm. The highest load achieved during the cycling level was 154kN. Cracks were uniformly spaced at 210mm over a height of 1537mm. Cracks widths up to 1.02mm were measured. A diagonal crack began to initiate at a height of 1257mm on the north side of the wall.

During the ensuing cycles, the wall displacements were increased to 28.2mm, corresponding to a displacement ductility of 3. The wall obtained a force of 161kN. The cycles were characterized by the onset of more diagonal cracks that begin to propagate down the structure at a 45-degree angle. Crack widths up to 3.18mm were measured. During the current cycling instabilities issues began to arise. The structure was temporally braced by steel cable and pulley system for the remainder of specimen.

The next loading cycle was conducted at displacement ductility 4, where the wall was pushed to a displacement of 37.6mm. During the cycle the wall reached a peak load capacity of 165kN, for the entire experiment. The corresponding drift level for the peak capacity was 1.54%. Crack widths up to 4.76mm were observed. The bottom layer of mortar on the south face of the wall was completely crushed.

At ductility 5 wall displacements were increased to 47mm. The wall maintained a load of 162kN. Cracks widths of 6.35mm were observed. The face shell of the brick began to spall on the north and south faces.

The next targeted displacement was displacement ductility 6, with a corresponding wall displacement of 56.4mm. The wall was able to achieve a peak cycle
load of 151kN, 90% of the maximum load, during the first cycle. However, by the third cycle, the bottom two courses of masonry and concrete core had crushed and spalled over a depth of approximately 152mm, on the south face, exposing the extreme south longitudinal bar and transverse reinforcement.

Displacement ductility 7 was the last level of testing. The wall was pushed to a displacement of 65.8mm. The extreme south bar buckled during the initial push due to the lack of concrete cover, therefore only a peak load of 102kN, 62% of the peak, was achieved. The first cycle was completed and the structure was loaded in the push direction to obtain load degradation properties, before the test was stopped. The masonry continue to spall up to five courses on the south side, while the bottom two courses were crushed on the north face.

![Figure 11.3 Force-Displacement Response of Specimen 5](image)

Figure 11.3 Force-Displacement Response of Specimen 5
11.2.3 Curvature Distribution

The curvature at varies levels of loading is shown in figures A-4.7 and A-4.8 for both directions of loading. Values were generated from the center height of each respective linear potentiometer. Readings had to be stopped before the end of test because of the severe damage in the wall at the gauge locations, thus not allowing accurate readings at high inelastic displacements.

The greatest concentration of curvature was experienced at the base of the wall. This should be expected because it is also the location of the peak strains and most intense damage. The trend of curvature distribution was to decrease over the height of wall as the moment demand decreased.

11.3 Summary for the next specimen

The experiment went primarily as plan, with the exception again of the excessive out-of-plane deformations. The wall reached a peak load of 165kN at a drift of 1.54% and maintained 90% of the maximum load at a drift of 2.31% demonstrating reliable displacement capacity. The failure was induced by crushing of the masonry and concrete cover due to extreme compressive forces causing the extreme south bar to buckle. If the extreme compression region can be sufficiently confined by the stainless, steel plates, the section can achieve higher compressive strains introduction more displacement capacity. Therefore it will be noteworthy to examine the behavior of the specimen with various level of confinement steel.

In respect to the out-of-plane deformations, the steel cable did a sufficient job of restraining the displacements. However, it was deemed an unsafe bracing system because there was no way to monitor the stresses and strains being introduced into the
wire. Therefore it was no way to know if ultimate values were being approached, which could lead to catastrophic damage. Another bracing system was devised using a W24X68 steel beam strung along the cap beam, for the final two experiments.

Figure 11.4 East Face of Specimen 5 at First Yield

Figure 11.5 West Face of Specimen 5 at Ductility 1-Cycle 1
Figure 11.6 East Face of Specimen 5 at Ductility 1.5-Cycle 1

Figure 11.7 East Face of Specimen 5 at Ductility 2-Cycle 1
Figure 11.8 South Face of Specimen 5 at Ductility 2-Cycle 3

Figure 11.9 East Face of Specimen 5 at Ductility 3-Cycle3
Figure 11.10 South Face of Specimen 5 at Ductility 4-Cycle 1

Figure 11.11 West Face of Specimen 5 at Ductility 4-Cycle 3
Figure 11.12 West Face of Specimen 5 at Ductility 5-Cycle 3

Figure 11.13 South Face of Specimen 5 at Ductility 5-Cycle 3
Figure 11.14 West Face of Specimen 5 at Ductility 6-Cycle 3

Figure 11.15 West Face of Specimen 5 at Ductility 6-Cycle 3
Figure 11.16 West Face of Specimen 5 at Ductility 7
12.0 Test Specimen 6

Chapter 12 focuses on the behavior of Specimen 6. The wall contained a longitudinal reinforcement steel ratio of 0.6% and a volumetric confinement ratio of 0.0023%. The standard, rectangular confinement steel was spaced at 140mm centers over the first 610mm of the wall corresponding to a plate at every other course.

12.1 Loading of Specimen

The specimen was subjected to the standard loading history adopted for the previous test, shown in figures 12.1 and 12.2. In the target history the structure is cycled once at 25%, 50%, 75%, and 100% of theoretical yield under force-controlled loading. Subsequently the specimen is subjected increasing levels of displacement ductility under displacement-controlled loading.

![Figure 12.1 Target Force-Controlled Loading History](image)
12.2 Observations

The noted behavior of the specimen under the subjected loading history will be depicted in this section. As in previous chapter the assessment will be broken down into two parts: Force-controlled loads (elastic loads), and Displacement-controlled loads (inelastic loads).

12.2.1 Force-Controlled Loading

In the initial cycle of loading, a force of 23.1kN was applied to the specimen. Under the applied load the wall displaced .457mm. Two cracks were opened on the specimen. One was located at the base and the other at a height of 559mm on the north end of the wall.
The next cycle was conducted at 50% of yield. A force of 46.3kN was applied to the structure. The wall exhibited a displacement of 1.73mm. Additional cracks were opened at locations where the confinement was present over a height of 559mm.

At 75% of yield, a force of 68.7kN was placed on the masonry wall. An average displacement of 2.76 was recorded. Two new cracks were detected, with one at a height of 419mm on the south side of the wall and at 978 on the north side of the column.

At first yield, the wall was loaded with 92kN force. The wall displaced 4.01mm. A uniform crack distribution was formed, with cracks occurring at 140mm spacing. A few new cracks were initiated outside the plastic hinge region, but the majority activity was center around extension of previous cracks in the plastic hinge region.

12.2.2 Displacement-Based Loading

At displacement ductility 1, the structure was pushed to 9.4mm. The wall sustained a load of 142kN. Crack widths up to .41mm were observed. At the current displacement level wall crack lengths up to a height of 768mm spanned the entire length of the wall.

The next level of cycling was conducted at displacement ductility of 1.5. Wall displacements were increased to 14.1mm. The maximum force achieved during the cycles was 142kN. Crack widths up to .76mm were observed.

The ensuing cycles were performed at a displacement of 18.8mm corresponding to a displacement ductility of 2. The load of 142kN was maintained. The first sign of crushing of the bottom layer of mortar was witnessed on the south face. The maximum crack width observed was recorded at 1.27mm.
At a displacement ductility of 3, the wall was displaced 28.2mm. The sustained load increased to a force of 158kN. Crack widths had become sufficiently large to allow the stainless, steel plates to be viewed. Significant masonry crushing was initiated on the north face of wall, with the brick unit starting to spall.

The next cycling level was conducted at displacement level 4. The wall was displaced 37.6mm. The specimen maintained a constant load level at 159kN. The mortar joint at a height of 279mm on the south side became debonded from the specimen allowing the plate to clearly viewed from the east face of the wall.

During the subsequent level of loading, the wall was cycled at displacement ductility 5 corresponding to a displacement of 47mm. The load continued to remain constant with a force of 160kN. The cycle was characterized by further crushing of masonry in the extreme compression region of the wall.

The next cycles were performed at displacement ductility of 7. A peak experimental force of 173kN was reached at a displacement of 65.8mm, corresponding to a drift of 2.7%. By the third cycle the load capacity decreased to 134kN (77% of the peak load). Huge crack widths began to open from cracks initiated at confinement locations. The bottom two courses of masonry began to spall on the north face over a depth of 102mm.

At displacement ductility 8, the wall was cycled at a displacement of 75.2mm. The masonry wall managed to carry a load of 143kN. The force capacity decreased 21% from the peak obtained at ductility 7. On the north side of the wall bottom two courses of masonry and concrete core spalled off exposing the transverse steel and confining plates. The extreme north bar remained faintly covered. It was observed that the confinement
spacing at every course was not efficient because it allowed the masonry to spall two courses jointly, over various heights. To lose such a significant amount of masonry created instabilities in the compression zone and led to severe load degradation.

Displacement ductility 10 was the final level of loading. The specimen was subjected to 1.5 cycles at a displacement of 94mm with a drift of 3.9% before the experiment was ended. After the concrete cover was crushed the extreme north bar buckled, therefore the load capacity was reduced to 95.3kN (an 81% decrease in load capacity). At the end of test the first 6 courses of masonry on the north face had spalled. The first 4 courses of masonry spalled on the south face.

Figure 12.3 Force-Displacement Response of Specimen 6
12.2.3 Curvature Distribution

The curvature at varies levels of loading is shown in figure A-4.9 for both directions of loading. Values were generated from the center height of each respective linear potentiometer. The elastic curvature was not recorded due to information loss during a power outage in the laboratory. Readings had to be stopped before the end of test because of the severe damage in the wall at the gauge locations, thus not allowing accurate readings at high inelastic displacements.

The greatest concentration of curvature was experienced at the base of the wall. This should be expected because it is also the location of the peak strains and most intense damage. However a new phenomenon was observed as the curvature for the curvature at third cell above the base was greater than the second cell for most of the inelastic range. This would not be normally expected because of the decrease in moment over the height of the structure. However it can be rationalized by figure A-5.5 which shows the curvature profile at zero wall displacements. The plot indicates that the reinforcement experienced some buckling effects at the location of the second cell, which is reasonable in the lightly reinforced specimen. Therefore as the specimen was further loaded to its target displacement the bar begin to return to its underformed position not allowing the expected curvature.

12.3 Summary for the Next Specimen

The newly devise bracing system provided adequate restraint against significant out-of-plane deformations. By attaching the bracing to the cap beam torsional effects were also eliminated. With the successful behavior of the system, it will be implemented for the final test.
Confining the specimen at every other course increased the displacement capacity of the system. The maximum force was obtained at drift of 2.7% (75% increase from specimen 5). However, the confinement spacing allowed the masonry to spall in huge sections leading to rapid degradation of the compressive zone. By confining the section at every course of masonry, as in the final experiment, this behavior should be improved allowing for a substantial increase in deformation capacity with a less rapid degradation of the structure.

![Figure 12.4 West Face of Specimen 6 at First Yield](image-url)
Figure 12.5 East Face of Specimen 6 at Ductility 1-Cycle 3

Figure 12.6 East Face of Specimen 6 at Ductility 1.5-Cycle 1
Figure 12.7 East Face of Specimen 6 at Ductility 2-Cycle 3

Figure 12.8 North Face of Specimen 6 at Ductility 3-Cycle 3
Figure 12.9 East Face of Specimen 6 at Ductility 3-Cycle 3

Figure 12.10 East Face of Specimen 6 at Ductility 4-Cycle 3
Figure 12.11 West Face of Specimen 6 at Ductility 5-Cycle 3

Figure 12.12 West Face of Specimen 6 at Ductility 7-Cycle 1
Figure 12.13 North Face of Specimen 6 at Ductility 7-Cycle 3

Figure 12.14 East Face of Specimen 6 at Ductility 8-Cycle 1
Figure 12.15 North Face of Specimen 6 at Ductility 8-Cycle 1

Figure 12.16 North Face of Specimen 6 at Ductility 10-Cycle 1
Figure 12.17 East Face of Specimen 6 at Ductility 10-Cycle 1
13.0 Test Specimen 7

Chapter 13 focuses on the behavior of Specimen 7. The specimen is the last in this series of experiments. The wall contained a longitudinal reinforcement steel ratio of 0.6% and volumetric confinement ratio of 0.0046%. In this test, 3.2mm thick, stainless steel confinement plates were placed in every mortar joint on the north and south faces of the structure, over a height of 629mm.

13.1 Loading of Specimen

The specimen was subjected to the standard loading history adopted for the previous test, shown in figures 13.1 and 13.2. In the target history the structure is cycled once at 25%, 50%, 75%, and 100% of theoretical yield under force-controlled loading. Subsequently the specimen is subjected increasing levels of displacement ductility under displacement-controlled loading.

Figure 13.1 Force-Controlled Loading History
13.2 Observations

This section is devoted to the behavior observed in specimen 7. The assessment will follow the standard convention adopted in the previous chapters, in which the section will assess the step by step behavior of the wall throughout the entire test with respect to the force-based and displacement-based loading scenarios.

13.2.1 Force-Controlled Loading

The first load cycle was conducted at 25% of the theoretical first yield force, corresponding to 23.1kN. Under the applied load, the wall displaced .451mm. Only a small base crack was detected in the initial cycle.

In the next cycle, a force of 46.3kN (50% of yield force) was applied to the specimen. An average displacement of 1.31mm was observed. A few additional cracks
were opened over a height of 629mm. Localized splitting of the clay unit was observed at height of 489mm on the south side of the wall.

The third cycle was performed at 75% yield at a force of 68.9kN. The wall experienced a displacement of 2.82mm. Cracking was expanded to a height of 978mm on the specimen. At a height of 419mm, a crack was detected spanning the entire length of the wall.

At first yield, the wall was loaded with a 92.0kN force. The wall displaced 4.47mm. One new crack was detected at height of 629mm on the north side up the structure. The cycle was characterized by the extension of previous cracks, with cracks spanning the full length of the wall over a height of 629mm. No definitive crack distribution or spacing was present.

13.2.2 Displacement-Controlled Loading

In the initial displacement controlled cycles, the structure was cycled at displacement of 9.4mm (displacement ductility 1). The wall sustained a load of 135kN. Crack widths up to .51mm were observed. On the south side of the structure, at height of 978mm, vertical splitting of the brick unit was witnessed.

The next level of cycling was conducted at displacement ductility of 1.5. Wall displacements were increased to 14.1mm. The maximum force achieved during the cycles was 148kN. The first sign of crushing of the bottom layer of mortar was observed at the southeastern corner of the structure. Maximum crack widths were measured at .76mm.

At displacement ductility 2, the wall was cycled at a displacement of 18.8mm. The load remained steady at 146kN. The crack distribution continued to be randomly
spaced over the wall. Several shear cracks were initiated during the cycles. Further cracking of the bottom layer of mortar proceeded.

The ensuing cycles were conducted at a displacement of 28.2mm, a ductility level of 3. The load capacity was slightly increase to 154kN. The masonry face shell began to spall at the southeastern corner of the specimen. Crack widths were increased to 1.02mm.

At the next loading level, wall displacements were increased to 37.6mm at a displacement ductility level of 4. The load was further increased to 163kN. The majority of the behavior centered on the extension of shear cracks. Cracking became prevalent in the plastic hinge region of the specimen. The mortar joint, on the north side of the wall at a height of 279mm, began to crush.

At displacement ductility 5, the wall was cycled at a displacement of 47mm. The corresponding force at the present displacement was 164kN. Crushing of the masonry commenced on the south face of the wall. The mortar joint at 279mm up the wall on the south side was crushed as well. The bottom layer of mortar on the north face of the specimen began to spalling exposing the bottom confinement plate.

The next cycles were performed at displacement ductility 6. Wall displacements were increased to 56.4mm. The force remained stable at 161kN. No significant activity was witnessed other than the debonding and spall of several mortar joints.

At the next loading level, the wall was displaced 65.8mm, with a corresponding ductility level of 7. The force in the structure remained at 161kN. Other than crushing of several mortar joints, no significant damage was observed it the extreme compression zones of the clay, masonry wall.
At displacement ductility 8, the wall was cycled at a displacement of 75.2mm. The load capacity continued to remain at 161kN. Huge cracks began to open in several mortar joints in the plastic hinge region. The bottom layer of mortar began to severely crush up to a depth of 178mm on both sides of the wall.

The following cycles were conducted at displacement ductility 10. The wall displacements were increased to 94mm. A peak experimental load of 165kN was sustained. The current drift level of the structure was 3.9%. By the third cycle significant crushing transpired in the extreme compression zones of the walls. The bottom course of masonry was crushed up to a depth of 102mm. Vertical separation of the wall was witnessed at the edge of the plate 630mm up the wall on the south side.

Displacement ductility 12 was the last full cycling completed. The wall was displaced 113mm. The wall sustained a maximum load of 151kN in the push cycle of the first cycle. However, upon load reversal the extreme north bar buckled due to lack of confinement from the concrete cover. A load of only 122kN was achieved (a 35% decrease from the peak capacity). The cycles were characterized by extreme crushing and spalling of the masonry in the plastic hinge region. The masonry also appeared to be sliding along the mortar joint.

At displacement ductility 14, the structure was cycled 1.5 times at a displacement of 132mm. The present drift level was 5.4%. On the final cycle, with buckling of the extreme south bar and significant reductions of the compression zone, the wall only attained a force of 86.9kN and the test was subsequently ended.
13.2.3 Curvature Distribution

The curvature at varies levels of loading is shown in figures A-4.10 and A-4.11 for both directions of loading. Values were generated from the center height of each respective linear potentiometer. Readings had to be stopped before the end of test because of the severe damage in the wall at the gauge locations, thus not allowing accurate readings at high inelastic displacements.

The greatest concentration of curvature was experienced at the base of the wall. This should be expected because it is also the location of the peak strains and most intense damage. However a similar phenomenon as in specimen 6 was observed as the curvature for the curvature at third cell above the base was greater than the second cell for most of the inelastic range. This would not be normally expected but of the decrease in moment over the height of the structure. However it can be rationalized by figure A-5.6 which shows the curvature profile at zero wall displacements. The plot indicates that the reinforcement experienced some buckling effects at the location of the second cell, which is reasonable in the lightly reinforced walls. Therefore as the specimen was further loaded to its target displacement the bar begin to return to its undeformed position not allowing the expected curvature.

13.3 Final Summary

The implementation of the confining steel made a significant impact on the behavior of the clay, masonry wall. By placing confining plates at every course of the wall over the plastic hinge region the wall achieved over 150% increase in drift capacity before the occurrence of strength degradation. By placing plates at every other course the structure experience a 93% increase of drift capacity at the peak force. However, the
large spacing allowed huge sections of masonry in the compression zone to spall at high
strains. Therefore significant strength degradations occurred after the peak force was
achieved. Reducing the spacing to every course providing better confinement,
minimizing spalling. The closer space help to keep the compression zone in better
contact, preventing the sudden strength degradation, providing an overall improved
response.

Figure 13.3 Force-Displacement Response of Specimen 7
Figure 13.4 East Side of Specimen 7 at Ductility 1.5-Cycle 1

Figure 13.5 South Side of Specimen 7 at Ductility 2-Cycle 1
Figure 13.6 West Face of Specimen 7 at Ductility 2-Cycle 3

Figure 13.7 East Face of Specimen 7 at Ductility 3-Cycle 3
Figure 13.8 East Face of Specimen 7 at Ductility 4-Cycle 1

Figure 13.9 East Face of Specimen 7 at Ductility 5-Cycle 1
Figure 13.10 East Face of Specimen 7 at Ductility 6-Cycle 1

Figure 13.11 South Face of Specimen 7 at Ductility 6-Cycle 3
Figure 13.12 East Face of Specimen 7 at Ductility 7-Cycle 3

Figure 13.13 West Face of Specimen 7 at Ductility 8-Cycle 1
Figure 13.14 North Face of Specimen 7 at Ductility 10-Cycle 1

Figure 13.15 East Face of Specimen 7 at Ductility 10-Cycle 3
Figure 13.16 North Face of Specimen 7 at Ductility 12-Cycle 1

Figure 13.17 East Face of Specimen 7 at Ductility 12-Cycle 3
Figure 13.18 East Face of Specimen 7 at Ductility 14-Cycle 1

Figure 13.19 North Face of Specimen 7 at Ductility 14-Cycle 2
14.0 Analysis of Results

The results of the tests will be discussed in detail in this chapter. The analysis will be presented in two separate sections. The first section will cover general behavior: masonry strains, maximum force and displacement obtained, and the backbone force-displacement response of each section. The next section will focus on key performance limit states evaluation with respect to the data collected for each specimen. A comparison of the respective response of each specimen will provide a means to determining the effectiveness of the confinement plates and recommendation for analysis and design, which will be presented in the following chapter. The results will primarily focus on specimens 5-7 because the desired flexural failure was achieved and also structures were subjected to similar load histories, allowing for better comparison than the first set of experiments. However specimens 1-4 will be frequently referenced throughout the chapter.

14.1 Summary of Results

Most structural design applications are centered on force deformation responses and allowable strains. Both are essential tools in structural design to ensure safe resistance against vertical and lateral loads. Therefore this section will cover the force-displacement characteristics and masonry strains of each clay masonry structure in this series of experiments. Below in table 14.1 is a summary of the maximum reliable forces and displacements recorded in each specimen. In the table, the maximum displacement is defined as the displacement at the point when the structure load capacity has degraded to 80% of the peak load, which is obtained from figure 14.1.
Table 14.1 Failure Loads and Displacements

<table>
<thead>
<tr>
<th>No.</th>
<th>Method</th>
<th>Type</th>
<th>Force (kN)</th>
<th>Drift (%)</th>
<th>Force (kN)</th>
<th>Drift (%)</th>
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<th>Drift (%)</th>
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<th>Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Experimental</td>
<td></td>
<td>10.06</td>
<td>1.901</td>
<td>0.016</td>
<td>0.016</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Analytical</td>
<td></td>
<td>4.54</td>
<td>2.83</td>
<td>0.021</td>
<td>0.021</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Experimental</td>
<td></td>
<td>15.0</td>
<td>3.64</td>
<td>0.036</td>
<td>0.036</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Analytical</td>
<td></td>
<td>10.06</td>
<td>1.901</td>
<td>0.016</td>
<td>0.016</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 14.1 Strength Ratio vs. Displacement Ductility

14.1.1 Masonry Strains

Table 14.2 shows the masonry strains at various limits states for each of the specimens tested. From the results it can be seen that the confinement plates did not have a significant impact on the compression strains achieved at the various limit states besides at the maximum reliable force. The maximum reliable force is defined as the
point when the structure strength has degraded to 80% of the peak experimental load. The values obtained at this limit state demonstrate the effectiveness of the plates to increase the ductility of the walls. As the confinement ratio is increased the displacement capacity is enhanced.

It is important to note that the strain obtained in the walls and the compression prism test shown in table 14.3 are only compatible for the unconfined cases. Table 14.3 shows the experimental values compared with theoretical values derived from the modified Kent-Park Curve (13). The discrepancy of the strains obtained from the prism and wall can be reasonable justified because of the difference encountered from the different testing scenarios. The prism test places the section in pure compression with the majority of the damage occurring from only crushing of the masonry unit, therefore allowing the full resistance of the plates to be mobilized, which caused the confinement plates to rupture during the prism test.

Experiments on the member level introduce several other variables that effect the overall response of the wall. The specimens are subjected to degradation due to shear, tensile forces, and cyclic loading not experienced by the prisms. Also the capacity and performance of the longitudinal steel has a significant impact on the behavior of the walls, again not experienced in the prism test. Therefore the plates are not fully mobilized in the walls, so they did not rupture or allow the strain capacity as predicted and seen experimentally in the prisms test. The results show that results from the prism test are not directly applicable to the performance of the walls and accounts for the discrepancy in displacement capacity in the theoretical and experimental values in figures 14.2-14.8 and table 14.1. The theoretical values for the specimens were obtained from the modified
Kent-Park Curve, shown in equations 14.1 and 14.2 to predict the ultimate strain capacity of the specimens.

\[ f_m = f_m' \left[ 1 - Z_m (\varepsilon_m - 0.0015) \right] \quad \text{Equation 14.1} \]

\[ Z_m = \frac{1.17 \times 0.5}{\left( \frac{3 + 0.29 f_m'}{145 f_m' - 1000} \right) + \frac{3}{4} \rho_s \sqrt{h''} - 0.002 K_d} \quad \text{Equation 14.2} \]

- \( f_m \) = masonry compression stress
- \( f_m' \) = ultimate masonry compression stress
- \( \varepsilon_m \) = masonry compression strain
- \( \rho_s \) = volumetric confining ratio
- \( h'' \) = lateral dimension of confined core
- \( s_h \) = longitudinal spacing of confining steel

### Table 14.2 Experimental Masonry Strains

<table>
<thead>
<tr>
<th>Specimen</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Cracking</td>
<td>0.00013</td>
<td>0.00032</td>
<td>0.00013</td>
<td>0.00017</td>
<td>0.000098</td>
<td>0.000042</td>
<td>0.000086</td>
</tr>
<tr>
<td>First Yield of Steel Reinforcement</td>
<td>0.0011</td>
<td>0.0013</td>
<td>0.0012</td>
<td>0.0012</td>
<td>0.00096</td>
<td>0.00038</td>
<td>0.0004</td>
</tr>
<tr>
<td>Confinement Yield</td>
<td>0.0009</td>
<td>0.0009</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.0004</td>
</tr>
<tr>
<td>Initial Crushing of Masonry</td>
<td>-</td>
<td>0.0026</td>
<td>0.0001</td>
<td>-</td>
<td>0.0013</td>
<td>0.0014</td>
<td>0.0011</td>
</tr>
<tr>
<td>Maximum Force</td>
<td>0.0048</td>
<td>0.0066</td>
<td>0.0024</td>
<td>-</td>
<td>0.004</td>
<td>0.0052</td>
<td>0.0058</td>
</tr>
<tr>
<td>Maximum Reliable Force (min. 80% ( f_{max} ))</td>
<td>0.0067</td>
<td>0.0096</td>
<td>0.0025</td>
<td>-</td>
<td>0.0049</td>
<td>0.006</td>
<td>0.0075</td>
</tr>
</tbody>
</table>

### Table 14.3 Theoretical and Experimental Masonry Strains

<table>
<thead>
<tr>
<th>Limit States</th>
<th>Unconfined</th>
<th>Alternate Course Confined</th>
<th>Every Course Confined</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Initiation of Splitting Cracks</td>
<td>( 0.75 f_m )</td>
<td>( 0.00121 )</td>
<td>( 0.00139 )</td>
</tr>
<tr>
<td>2 Excessive Cracks/Spalling</td>
<td>( 0.95 f_m )</td>
<td>( 0.00161 )</td>
<td>( 0.00160 )</td>
</tr>
<tr>
<td>3 Yielding of Confinement Plates</td>
<td>( ----- )</td>
<td>( ----- )</td>
<td>( 0.00205 )</td>
</tr>
<tr>
<td>4 Maximum Dependable Compression Strain</td>
<td>( 0.5 f_m )</td>
<td>( 0.00305 )</td>
<td>( 0.00498 )</td>
</tr>
<tr>
<td>5 Ultimate Compression Strain</td>
<td>( 0.2 f_m )</td>
<td>( 0.00486 )</td>
<td>( 0.00875 )</td>
</tr>
</tbody>
</table>
14.1.2 Effect of Confinement on Displacement Capacity

It is difficult to rationally analyze the effectiveness of the confining plates in the first set of specimens due to the shear failures and various loading histories. However, specimens 5-7 clearly demonstrate the value of confining the wall with stainless, steel plates in figures 14.6 -14.8.

Although it is difficult to evaluate the true effectiveness of the confinement plates in the first set of tests, a comparison can be drawn in the first three specimens. Under monotonic loading Specimen 1 reached a peak load at 1.36% drift. Immediately after the specimen achieved its maximum force capacity there was a sudden drop in strength and stiffness. Under a more critical cyclic loading, specimen 2, confined at every other course in the plastic hinge region, demonstrates enhanced performance by achieving a peak load at 1.84% and maintained reliable strength capacity up to 3.06% drift before severe degradation. Specimen 3, confined at every course was cycled past the displacement achieved in specimen 1, then loaded in a similar monotonic nature. Specimen 3 achieved a displacement capacity 4 times greater than the control specimen, which can be witnessed in figure 14.10.

Specimen 7, which contains confinement steel at every course over the first 610mm of the wall, exhibited a 88% increase in displacement capacity over specimen 5 with no confining steel. Specimen 7 reached a drift of 4.55% before the onset of any significant strength degradation. Confining the section at every other course in the plastic hinge region of the wall, as in Specimen 6, also resulted in an increase in displacement capacity but not as effective as Specimen 7. Specimen 6 only exhibited a 34% increase in displacement capacity. The increased spacing of the plates in specimen 6, allowed
huge sections of a masonry to spall concurrently, thus permitting rapid degradation of the compression zone decreasing the ductility potential of the wall.

Figure 14.2 Backbone Force-Displacement Curve for Specimen 1
Figure 14.3 Backbone Force-Displacement Curve for Specimen 2

Figure 14.4 Backbone Force-Displacement Curve for Specimen 3
Figure 14.5 Backbone Force-Displacement Curve for Specimen 4

Figure 14.6 Backbone Force-Displacement Curve for Specimen 5
Figure 14.7 Backbone Force-Displacement Curve – Specimen 6

Figure 14.8 Backbone Force-Displacement Curve – Specimen 7
14.1.3 Effect of Confinement on Strength Capacity

Confinement did not appear to have a significant effect on the load capacity of the walls, as expected. This is easily justified because the plates are not fully mobilized until the structure has yielded; therefore there should be no significant increase in load capacity, other than strain hardening effects of the longitudinal reinforcement.

Most current seismic design approaches require the structure to withstand inelastic forces to dissipate the energy generated by a potential earthquake. Therefore trying to increase the force capacity is somewhat irrelevant, since the force capacity will be fully exhausted under inelastic excursions. Displacement capacity, which has been shown to be substantially increased by the confinement plates, allows the structure to achieve greater ductility permitting more energy dissipation capacity.

14.1.4 Effect of Loading History on Displacement Capacity

The loading history was shown to have a significant effect on the ductility of a structure. It is generally accepted that an increase in strength of a structure results in decreased ductility. However Specimen 3 which contained a longitudinal reinforcement ratio of 1.5% and confined at 70mm centers demonstrated greater ductility, than specimen 7 which only contained a reinforcing ratio 0.6% and also confined at 70mm centers as seen in figure 14.9. Specimen 3 was subjected to reversed, cycles up to a displacement ductility of 3 then loaded monotonically until failure. Conversely, specimen 7 was subjected to reversed cycles of loading until failure. Specimen 3 reached a drift capacity of 6.63% before the occurrence of any significant degradation, while specimen 7 was only able to achieve a drift of 4.55% before strength degradation. It is apparent that the monotonic loading does not initiate as much degradation in the wall as
experienced with cyclic loading history, therefore resulting in the increased displacement capacity of specimen 3.

![Figure 14.9 Force-Displacement Envelopes- Specimens 3 and 7](image)

**14.1.5 Effect of Loading History on Force Capacity**

Loading history was also shown to have a significant effect on the force capacity of the first three specimens, especially when the response is shear dominated. The cyclic loading history resulted in noticeable force degradations in the specimens. Referring to figure 14.10 it is shown that the specimen 1 (strictly monotonic loading) experienced a substantial increase in maximum force than the other two specimens. The elastic performance was also effected, as the initial stiffness of the specimens 2 and 3 were noticeable decreased.

In addition specimen 1 was the only wall to exceed its predicted force capacity as listed in table 14.1. The elastic response demonstrated good agreement with the
prediction, however the maximum force exceeded the predicted response by 7.69%, which can be attributed to dynamic amplification. The monotonic test was performed briskly, with failure been reached in 27 seconds. Using the strain history recorded from a strain gauge on the extreme north bar, the Mander model for dynamic amplification factors was generated from equation 14.3 and seen in figure 14.11. The model predicts a dynamic stress amplification of 6.58% at the maximum strain, which coincides closely with the experimental overstrength observed.

\[ f_{eh} = 0.953 \left( 1 + \frac{\varepsilon}{700} \right) \]

Equation 14.3

\( f_{eh} \) = stress dynamic amplification factor

\( \varepsilon \) = strain rate
Figure 14.10 Force Envelopes - Specimen 1-3

Figure 14.11 Stress Dynamic Amplification – Test 1
14.2 Performance Limit States

This section will attempt to define critical limit states of clay, masonry walls to further advance limit state design procedures, which gives engineers more control over the overall behavior of masonry structural walls. Several criteria will be evaluated to give a complete representation of the behavior of clay, masonry walls under various levels seismic loading.

14.2.1 Masonry Compression Strains

The compressive strain behavior is one the most critical parameters in proper design and analysis of masonry structures. By defining an ultimate strain limit, a design procedure can be developed to allow for safe, yet economical structures. Figures 14.13-14.19 show the masonry strains developed in each structure during the respective experiments.

Three sets of calculations were generated using the available strain gauges, linear potentiometers, and the string potentiometer respectively. Since the strain gauges stopped performing well before the end of the experiment, no strain calculations could be computed at significant inelastic displacements. The linear potentiometer readings became unreliable near the final cycles due to excessive rotations of the threaded rods and angle brackets. Therefore the string potentiometer was the only source of calculating the masonry strain throughout the entire experiment and is the instrument used to calculate the compressive strains in figure 14.20.

The masonry strain was generated from the strain gauges by using the strain recorded from two strain gauges at the same height on different longitudinal bars to generate a strain profile for the section as seen in figure 14.12. The masonry compression
strain can then be found from the equation 14.4. Equation 14.5 was used to generate the corresponding masonry strain from the linear potentiometer. The neutral axis depth was approximated using moment curvature analysis and the curvature was obtained from the experimental curvature profiles shown in figures (A4.1-A4.11). The masonry strain for the string potentiometer was also calculated using equation 14.5, but the elastic and inelastic curvatures were approximated by equations equation 14.6 and equation 14.7 respectively.

The maximum masonry compression strain, in this study, was achieved in Specimen 3, shown in figure 14.28, which was confined at every course in the plastic hinge region. Before the shear failure the structure reached a maximum compression strain of 2.9% at a drift of 6.63%. The strain is 8.3 times the value of .35 allowed by the TMS code (7). The ultimate strain value was 3.8 times higher than the strictly monotonic test of specimen 1, the unconfined, control specimen. It is noteworthy to consider that if a flexural failure could been achieved that the compression strain would have been further increased.

In the second set of experiments (specimens 5-7), it is clearly shown that confining the wall can have a substantial impact on the masonry strain. Since each specimen experienced the same cyclic loading history a direct comparison can easily be made. Confining the wall at every course resulted in a 48% increase in strain capacity over the unconfined wall and 24% increase over the specimen confined at every other course. Specimen 6 with confinement at every other course experienced a 19% increase in strain over the unconfined case. In addition from figures 14.20, the respective strain,
at the concurrent drift levels in specimens 5-7 is reduced by presence of confinement, therefore reducing damage potential.

\[
\varepsilon_m = \varepsilon_{si} - \left( \frac{\varepsilon_{si} - \varepsilon_{s2}}{d_1 - d_2} \right) \ast d_i
\]

**Equation 14.4**

\(\varepsilon_m\) = masonry compression strain

\(\varepsilon_{si}\) = steel strain from respective strain gauge

\(d_i\) = depth of longitudinal bar at which the gauge is located

\[
\phi = \frac{\varepsilon_m}{c}
\]

**Equation 14.5**

\(\phi\) = curvature

\(\varepsilon_m\) = masonry compression strain

\(c\) = theoretical neutral axis depth

\[
\Delta_i = \frac{\phi h_e^2}{3}
\]

**Equation 14.6**

\(\Delta_i\) = elastic displacement

\(\phi\) = elastic curvature

\(h_e\) = effective height

\[
\Delta_i = \Delta_y + (\phi_i - \phi_y) \ast L_p \left( h_e - \frac{L_p}{2} \right)
\]

**Equation 14.7**

\(\Delta_i\) = target displacement
\( \Delta_y = \) yield displacement

\( \phi = \) curvature at target displacement

\( \phi_y = \) yield curvature

\( L_p = \) theoretical plastic hinge length

\( h_e = \) effective height

Figure 14.12 Wall Section Strain Profile
Figure 14.13 Masonry Compression Strain – Specimen 1

Figure 14.14 Masonry Compression Strain – Specimen 2
Figure 14.15 Masonry Compression Strain – Specimen 3

Figure 14.16 Masonry Compression Strain – Specimen 4
Figure 14.17 Masonry Compression Strain for Specimen 5

Figure 14.18 Masonry Compression Strain – Specimen 6
Figure 14.19 Masonry Compression Strain – Specimen 7

Figure 14.20 Masonry Compression Strain: Specimens 1-7
14.2.2 Steel Tension Strains

According to the masonry strains obtained in the previous section, the corresponding steel tension strains are presented in figures 14.21-14.28. The increase in masonry compression strain by the confinement plates leads to increased steel tension strains, which could result in rupture of the longitudinal reinforcement. In specimens 3 and 7 where extremely high compressive strains were achieved the corresponding tension strains were well in excess of maximum strain of 0.08 found during material testing, and shown figure 5.2. The longitudinal bars did not rupture so that would indicate that reinforcement must experienced debonding from the concrete grout. This also manifested itself in the force displacement response. The experimental forces were well below the predicted response in the inelastic range which can be witnessed in backbone curves presented in figures 14.3-14.8. The force plateaued after yield indicating no significant strain hardening effects, even though the measured strains were beyond of strain hardening region of the steel again indicating debonding between the concrete grout and steel.
Figure 14.21 Steel Tension Strains – Specimen 1

Figure 14.22 Steel Tension Strains – Specimen 2
Figure 14.23 Steel Tension Strains – Specimen 3

Figure 14.24 Steel Tension Strains - Specimen 4
Figure 14.25 Steel Tension Strains – Specimen 5

Figure 14.26 Steel Tension Strains – Specimen 6
Figure 14.27 Steel Tension Strains – Specimen 7

Figure 14.28 Steel Tension Strains: Specimen 1-7
14.2.3 Confinement Strains

Strain gauges were placed on the confinement steel in the confined specimens to monitor the strains experienced by the stainless, steel plates. Gauges were placed on the east and west webs in specimens 2-4, while specimens 6 and 7 had an additional gauge on the outermost flange. Limited success was experienced with the gauges, but some important trends were captured. Figures A3.1-A3.3 shows after the confinement plate yields, there is a rapid increase in strain. This indicates that the confinement resistance diminishes after the onset of yield. Therefore the plates should be designed to behave elastically to further improve the displacement capacity of clay masonry walls.

A distinct behavior pattern for the plates can be witnessed from the various strain hysteresis shown in appendix three. The figures show that the plates are only active when the respective section of the specimen is placed in a state of compression. This is reasoned by the increase in strain of the plates when the specimen is loaded in the appropriate direction, which confirms that the plates are actively resisting lateral expansions of the masonry. Upon load reversal the plate strains decrease to as the section is placed in a state of tension. The plate strain as the respective section of masonry is placed in tension usually returns to approximate zero until the plate is well beyond its yield stress in which some residual strains is noticed in the opposite direction of loading.
14.2.4 Curvature Ductility

Although displacement ductility is the conventional method used to evaluate the ductility capacity of a system, curvature relationships can be a useful analysis tool. “The most common and desirable sources of inelastic structural deformation are rotations in potential plastic hinges. Therefore, it is useful to relate section rotations per unit length (i.e., curvature) to causative bending moments.”(8). Curvature ductility can be calculated in the following manner:

Evaluate the first yield curvature

\[
\phi_y = \frac{\varepsilon_y}{d - c_y}
\]  

**Equation 14.8**

\(\phi_y\) = first yield curvature

\(\varepsilon_y\) = steel tensile yield strain
\( d \) = depth of the section

\( c_y \) = neutral axis depth at first yield

Approximate equivalent yield curvature

\[
\phi_y = \phi_y \left( \frac{M_i}{M_y} \right)
\]

Equation 14.9

\( \phi_y \) = equivalent yield curvature

\( M_i \) = ideal moment capacity

\( M_y \) = yield moment

Calculate the target inelastic curvature

\[
\phi_t = \frac{\varepsilon_{mt}}{c_i}
\]

Equation 14.10

\( \phi_t \) = curvature at target displacement

\( \varepsilon_{mt} \) = masonry compression strain at target displacement

\( c_i \) = neutral axis depth at target displacement

Finally curvature ductility is evaluated by the following expression

\[
\mu_y = \frac{\Phi}{\phi_y}
\]

Equation 14.11

From the above equations, it can be seen there are several ways to increase the curvature ductility of a section such as increasing the section depth or compressive masonry strain. Also ductility can be increased by reducing the neutral axis depth at first yield or the targeted inelastic displacement. Confining the clay masonry wall allowed
higher compressive strains to be obtained, therefore resulted in a substantial increase in curvature ductility. Generally, as a section is further loaded into the inelastic range, the neutral axis depth will become more shallow increasing curvature ductility capacity. The increased amounts of confinement allow high drifts to be obtained again resulting in higher curvature ductility capacities. Figure 14.30 demonstrates the trend with the heavier confined walls (i.e. specimens 3 and 7) with the highest curvature ductility.

![Figure 14.30 Curvature Ductility](image)

### 14.2.5 Residual Drift

Residual drift is a performance criterion that has only recently received significant attention. It is an evaluation of the ability of a structure to return to initial position after being subjected to substantial loads. In this study, the residual drift was evaluated by
computing the ratio of the displacement when the structure is under a zero load to the effective height of the wall.

Even though a structure may be able to withstand lateral forces without catastrophic damage, severe residual displacement resulting from the applied load could render the structure unoccupiable. Prestress masonry walls usually demonstrate superior behavior in terms of residual drift through a self centering mechanism whichs nearly always bring the wall back to zero displacement upon unloading (15).

Regularly reinforced walls can experience a significant level of residual drift under inelastic loads. This mechanism increases the size of hystretic loops of the force dispalacement response. Therefore if properly designed, the structure can benefit somewhat from the residual displacements, because of the increased energy dissipation capacity.

Figure 14.31 depicts the residual drift ratios of the specimens that were subjected to a cyclic loading. It can be witnessed that as the structure is further loaded in the elastic range, the residual displacement also increases which is expected with the continued degradation of stiffness. The increased inelastic load and and loss of strength within the wall renders the structure inability to resume its original postion upon unloading. During specimens 5-7, in which the same loading history was applied, similar residual displacements were recorded at corresponding dirfts. Therefore the presence of confinement had no impact on the residual displacements, other than allowing the specimen to achieve higher drifts leading to an increase in residual displacements.
14.2.6 Plastic hinge lengths

The plastic hinge in a masonry wall is considered to be the length of the structure subjected to non-linear inelastic curvature. Non-linearities of the curvature distribution in the region and complexities of reinforcement strain penetrations into the footing make it extremely difficult in calculating inelastic displacements. Therefore Paulay and Priestley (8) propose a method in which the inelastic curvature profile is approximated by an equivalent constant curvature distribution in the plastic hinge region. The developed expressions for the plastic length, $L_p$, were derived from analytical and experimental components considering the shape of the moment-curvature curve and strain penetration. The plastic hinge of the wall should be approximated by the largest value from equations 14.12-14.14 (SI units).
\[ L_p = 0.2l_w + 0.044h_e \]
\[ L_p = 0.08h_e + 0.022f_yd_{bl} (Mpa) \]
\[ L_p = 0.044fyd_{bl} (Mpa) \]
\[ \Delta_t = \Delta_y + (\Phi_t - \Phi_y)L_p h_e \]

\( L_p \) = plastic hinge length

\( l_w \) = length of wall

\( h_e \) = effective wall height

\( f_y \) = longitudinal reinforcement yield stress

\( d_{bl} \) = diameter of longitudinal bar

\( \Delta_t \) = target inelastic displacement

\( \Delta_y \) = yield displacement

\( \Phi_t \) = curvature at target inelastic displacement

\( \Phi_y \) = yield curvature

In this study the plastic hinge length of each wall was calculated by the following equation 14.15, in which the target inelastic displacement is a function of the yield displacement and the plastic rotation of the wall. The results show an initial increase in plastic hinge length followed by a plateau region. This should be expected because as the initial plastic hinge develops, it initiates a redistribution of moment throughout the structure. The redistribution of moment couple with increased inelastic loading can lead to the spreading of the plastic hinge length or formation of new hinges to accommodate the substantial inelastic rotations. Therefore it can be shown one value can not accurate represent the plastic behavior of the wall. However the technique may be deemed
acceptable, since the hinge remains somewhat constant deep into the inelastic range, where the analysis is more critical. Eq 14.12-14.14 should yield plastic hinge lengths approximately equal to half the depth of the section according to Paulay and Priestley (8), which is close to, the experimental values obtained. However the equations results in lengths around .375 the depth of the section for the geometric properties of the section accounting for the difference in the experimental and theoretical results.

![Figure 14.32 Plastic Hinge Lengths](image)

**Figure 14.32 Plastic Hinge Lengths**

### 14.2.7 Hysteric Damping

Since damping is the process of dissipating energy, it is an extremely important concept in seismic design. There are several forms of damping, but masonry walls are dominated by hysteretic damping, in which energy is dissipated primarily from internal friction. Hysteretic damping in the specimens were obtained using the Jacobsen’s
approach. This method compares the area of an experimental hysteretic loop with a theoretical rigid-perfectly-plastic (RPP) loop enclosing the experimental loop. Since the damping relationship is only a function of the hysteretic shape, it is applicable to any material. The approach yields the equation 14.16, which estimates the level of hysteretic damping present within the structure.

\[
\zeta = \left( \frac{2}{\pi} \right) R
\]

*Equation 14.16*

\( \zeta = \) hysteretic damping

\( R = \frac{\text{area of hysteretic loop}}{\text{area of RPP loop}} \)

All the specimens showed the same damping characteristics. As the displacement ductility is initially increased there is a sharp rise in hysteretic damping. When the structure is loaded deep into its inelastic range the damping level begins to plateau with a gradual increases. The presence of confinement and different reinforcing ratios had no significant impact on the damping behavior of the walls, as figure 14.33 shows. The behavior of the specimens are closely bounded together with the only noticeable difference is the increase of displacement ductility, due to confinement, allowing a slight increase in damping during late cycles. The dotted line represents an reasonable approximation of the experimental damping levels using the Takeda degrading stiffness hysteretic response shown in equation 14.17 with the suggested parameters (\( \alpha = 0.43 \), \( \beta = 0.13 \)) to generate the curve.

\[
\zeta_{\text{hyst}} = \left( \frac{2}{\pi} \right) \left\{ 1 - \frac{3}{4} \mu^{\alpha-\gamma} - \frac{1}{4} \mu^{-1} + \frac{1}{4} \left[ \frac{\alpha^2}{\beta} \right] - \frac{1}{4} \left[ \frac{\beta^2}{\gamma} \right] \right\}
\]

*Equation 14.17*
\[ \zeta_{\text{hyst}} = \text{hysteretic damping} \]

\[ \mu = \text{ductility level} \]

\[ r = \text{second slope stiffness ratio} \]

\[ \gamma = r\mu - r + 1 \]

**14.3 Shear Behavior**

Even though shear was not the primary concern in this study, it is a significant parameter in the overall behavior of masonry walls. Many of the past failures are attributed to shear. In addition, the three of the first four specimens tested failed in shear, even though it was not the intended mode of failure. However the specimens provided an interest insight into the shear behavior of clay masonry.

A strain gauge was placed on the transverse steel in the plastic hinge region to monitor the strains induced by applied shear. The strains recorded throughout test were then multiplied by the appropriate stresses level to track the forces in the transverse bar. Once the force in the transverse steel was acquired, the masonry shear resistance was calculated by subtracting the force in the shear steel from the applied shear, allowing the figures 14.34-14.41 to be generated.
14.3.1 Total Shear Resistance

Figures 14.34 & 14.35 illustrate the total shear resistance provided by the masonry walls. The experimental results are compared to theoretical models provided by ACI 530 (7), and Paulay-Priestley (8). The Paulay-Priestley (PP) model offers two separate values, one for the elastic region of the wall and the other to calculate the resistance provided in the plastic hinge region of the wall. The latter model accounts for the fact that the shear resistance increases in the elastic region of the wall compared to the inelastic zones. The plastic hinge region is usually smaller than the elastic region, but is not the case for these specimens 1-4, due to the closer transverse steel spacing throughout the plastic hinge region.

Specimens 2 & 3 failed in a shear mode. They provided more resistance than the predicted ACI model but did not reach its full strength due to bond issues. Therefore the
shear resistance did not reach the resistance values allowed by the less conservative PP models. Specimen 1 resistance better agreed with the PP model. It was able to obtain higher shear capacity because of the monotonic loading history, reducing shear degradation.

Figure 14.35 shows that the shear resistance was well below all the models. This is due to the fact that the final three specimens the steel provided was well in excess of the amount needed to adequately resist the applied shear. Therefore very little shear cracking was detected and the walls were dominated by flexural as intended.

14.3.2 Masonry Shear Resistance

The masonry shear resistance is presented in figures 14.36 and 14.37. They demonstrate typical characteristics exhibited by concrete structures. The masonry components increase in the initial stages of loading and then began to decrease after significant cracking begins to transpire.

In figure 14.36 its seen that specimens showed a trend to reach values close or in excess of the elastic PP model and decreased to the values given by the PP plastic hinge and ACI model. The test demonstrates that one value not effective in analyzing the shear stress resistance as given by the current ACI code. Again it is difficult to draw any rational conclusions the test 5 and 7 shown in figure 14.37 because of the extremely conservative shear design, not allowing the full shear capacity to be mobilized.

14.3.3 Transverse Steel Resistance

Figures 14.38 and 14.39 depict the behavior of the transverse steel in the study. The steel also demonstrated behavior comparable to concrete structures. The steel stress
remained low until significant cracking occur, in which the steel then dominates the behavior.

The transverse steel strains recorded in specimen 2 were well in excess of that experienced in specimens 1 and 3. This account for the low masonry stress in specimen 2 depicted in figure 14.36. The steel stress in specimens 1 and 3 exceeded the conservative ACI model but did not reach values predicted by the PP models. This indicates the earlier conclusion that the steel was not able reach potential strength values due to improper anchorage. Specimens 5 and 7 showed steel stresses well below all the models, due to fact the applied shear force was not great enough to fully mobilize the transverse steel.

14.3.4 Masonry Shear Stress Coefficient

The coefficient K shown in figure 14.40 and 14.41 represents the coefficient used to empirically calculate the masonry shear stress, an approach used by the current ACI code and many other models and is shown in equation 14.18. The figures show the trend that one value may not be entirely appropriate for design purposes. The values are shown to be in excess of the value given in the ACI code well into the inelastic region. If a structure is designed to remain elastic, then the ACI model may be too conservative.

\[ v_m = K \sqrt{f'_m} \quad \text{Equation 14.18} \]
Figure 14.34 Total Shear Resistance: Test 1-3

Figure 14.35 Total Shear Resistance: Test 5&7
Figure 14.36 Masonry Shear Resistance: Test 1-3

Figure 14.37 Masonry Shear Resistance: Test 5&7
Figure 14.38 Transverse Shear Resistance: Test 1-3

Figure 14.39 Transverse Shear Resistance: Test 5&7
Figure 14.40 Masonry Shear Stress Coefficient: Test 1-3 (SI units)

Figure 14.41 Masonry Shear Stress Coefficient: Test 5&7 (SI units)
15.0 Conclusions

In this study, seven cantilevered, clay masonry walls were subjected to various seismic excitations to examine the effectiveness of a confinement technique first proposed by Priestley and Bridgeman (10). The technique consists of placing stainless steel plates in the mortar joints of the structure in attempt to improve the displacement capacity of the wall. From the results obtained in this study the following conclusions are drawn.

- **Effect of Confinement on Displacement Capacity**
  1. Providing galvanized steel confinement plates at every course over the plastic hinge region can increase displacement capacity of clay masonry walls up to 88%.
  2. Drift levels up to 6.6% may be achieved under monotonic, while under reversed cyclic loads reliable drifts up to 4.6% can be achieved with proper confinement.
  3. Confining clay, masonry walls at every other course over the plastic hinge region can also increase displacement capacity but is not as effective as placing plates at every course. The technique permits rapid degradation of the compressive zones at high strain levels leading to sharp drops in strength.

- **Effect of the Loading History on Clay, Masonry Walls**
  1. The effectiveness of confinement is enhanced under monotonic loading compared to cyclic loading. This monotonic loading does not introduce degradation into the section allowing more ductility.
• **Masonry Compressive Strain**
  1. By confining the section at every course compressive strains up to 2.9% can be achieved under monotonic loading, while strains up to .87% were obtained under cyclic loads.
  2. Properly confined walls may allow an increase up to 48% of compressive strain capacity.

• **Plastic Hinge Length**
  1. The length of the plastic hinge region of clay, masonry walls is independent of the presence confinement and longitudinal reinforcing ratios.
  2. In the early stage of inelastic loading the plastic hinge length gradually elongates until approximately 1.5%-2% drift where the lengths begins to remain somewhat constant.

• **Hysteretic Damping**
  1. The hysteretic damping is independent of reinforcing ratios and the presence of confinement.
  2. Damping levels can be reasonable predicted using the Takeda degrading stiffness model using the suggested parameters of $\alpha = .043$, $\beta = 0.13$.

**15.1 Design Recommendations**

In highly seismic regions a structure must be able to withstand several inelastic excursion to dissipate energy from a potential seismic attack. Therefore it is recommended that all clay masonry structures should incorporate the use of confinement
plates at every course in the plastic hinge region. If the wall is properly confined, the wall may be safely designed for drift levels up to 4% and compression strains up to 0.08.

15.2 Recommendations for Future Studies

1. Unintentionally different loading histories were applied to the walls in this study due to unfortunate circumstances. However it provided valuable insight that loading history can have a significant impact on the behavior of clay, masonry walls. Therefore it is a topic that needs to be addressed to provide engineers guidance on designing reliable, safe structures for various loading hazards.

2. In addition it was observed during the experiments that there were no noticeable deformations of the confinement plates in the upper portion of the plastic hinge region. This signifies that confinement is not needed that high in the structure, which is reasonable since the highest compressive strains are experienced at the bottom of the structure. Therefore more studies are needed to further examine different confinement ratios to allow more efficient design and implementation of confinement plates.

3. Also more test are needed on confined walls with different longitudinal reinforcing ratios. A successful evaluation was only completed on walls with a low reinforcement ratio of 0.6%, in which the effectiveness of the confinement was adequately confirmed. Therefore test are needed on heavier reinforced walls to confirm the results, for various reinforcement ratios.

4. Another area that needs to be addressed is walls with different configurations such as I-shaped and L-shape members. The performance of such configurations may lead to
a change in the response of the members. There it noteworthy to address the effectiveness of confinement in different shape walls.

5. Finally test are needed on the system level. In an actual housing unit many members are connected acting as one unit substantially changing the response of the individual members. Therefore more attention is needed on full housing units to improve the practice of designing and analyzing of masonry buildings.
REFERENCES


7. Masonry Standards Joint Committee, Building Code Requirements for Masonry Structures, ACI 530/ASCE 5/TMS 402, American Concrete Institute, Detroit, Michigan; American Society of Civil Engineers, New York; The Masonry Society, Boulder, Co.


APPENDIX
Appendix A-1

Figure A-1.1 Strain Gage Hysteresis : Test 1-L11

Figure A-1.2 Strain Gage Hysteresis : Test 1-L21
Figure A-1.3 Strain Gauge Hysteresis: Test 1-L31

Figure A-1.4 Strain Gauge Hysteresis: Test 2-L21
Figure A-1.5 Strain Gauge Hysteresis: Test 3-L11

Figure A-1.6 Strain Gauge Hysteresis: Test 3-L31
Figure A-1.7 Strain Gauge Hysteresis: Test 4-L21

Figure A-1.8 Strain Gauge Hysteresis: Test 5-L22
Figure A-1.9 Strain Gauge Hysteresis : Test 6 L42

Figure A-1.10 Strain Gauge Hysteresis : Test 6-L21
Appendix A-2

Figure A-2.1 Strain Gauge Hysteresis: Test 1-T1

Figure A-2.2 Strain Gauge Hysteresis: Test 2-T3
Figure A-2.3 Strain Gauge Hysteresis: Test 3-T1

Figure A-2.4 Strain Gauge Hysteresis: Test 5-T1
Figure A-2.5 Strain Gauge Hysteresis : Test 7-T2
Appendix A-3

Figure A-3.1 Strain Gauge Hysteresis : Test 2-P2SE

Figure A-3.2 Strain Gauge Hysteresis : Test 6-P1NE
Figure A-3.3 Strain Gauge Hysteresis : Test 7-P2SE
Appendix A-4

Figure A-4.1 Elastic Curvature Profile: Test 2

Figure A-4.2 Inelastic Curvature Profile: Test 2
Figure A-4.3 Elastic Curvature Profile: Test 3

Figure A-4.4 Inelastic Curvature Profile: Test 3
Figure A-4.5 Elastic Curvature Profile: Test 4

Figure A-4.6 Inelastic Curvature Profile: Test 4
Figure A-4.7 Elastic Curvature Profile: Test 5

Figure A-4.8 Inelastic Curvature Profile: Test 5
Figure A-4.9 Inelastic Curvature Profile: Test 6

Figure A-4.10 Elastic Curvature Profile: Test 7
Figure A-4.11 Inelastic Curvature Profile: Test 7
Appendix A-5

Figure A-5.1 Curvature Profile at Zero Wall Displacement: Test 2

Figure A-5.2 Curvature Profile at Zero Wall Displacement: Test 3
Figure A-5.3 Curvature Profile at Zero Wall Displacement: Test 4

Figure A-5.4 Curvature Profile at Zero Wall Displacement: Test 5
Figure A-5.5 Curvature Profile at Zero Wall Displacement: Test 6

Figure A-5.6 Curvature Profile at Zero Wall Displacement: Test 7