**IN-PLANE CYCLIC SHEAR PERFORMANCE OF INTERLOCKING
COMPRESSED EARTH BLOCK WALLS**

A Thesis
presented to
the Faculty of California Polytechnic State University,
San Luis Obispo

In Partial Fulfillment
of the Requirements for the Degree
Master of Science in Civil and Environmental Engineering

by
David William Bland

2011

© 2011

David William Bland

ALL RIGHTS RESERVED

Committee Membership

TITLE: In-Plane Cyclic Performance of Interlocking
Compressed Earth Block Shear Walls

AUTHOR: David William Bland

DATE SUBMITTED: April 2011

COMMITTEE CHAIR: Daniel C. Jansen, Associate Professor CEEN

COMMITTEE MEMBER: Bing Qu, Assistant Professor CEEN

COMMITTEE MEMBER: Peter T. Laursen, Assistant Professor ARCE

Abstract

In-Plane Cyclic Performance of Interlocking Compressed Earth Block Shear Walls

David William Bland

This thesis presents results from testing of interlocking compressed earth block (CEB) masonry shear walls. CEBs are low strength earth masonry units sometimes stabilized with cement or lime. The interlocking compressed earth blocks (ICEBs) used in this experiment are dry stacked interlocking hollow units, which can be reinforced and grouted after they are laid. Although significant research has been undertaken to optimize the material properties of CEBs, little has been done to investigate the performance of structural systems currently being built using this technology.

Test results are reported for three 1800 mm x 1800 mm wall specimens constructed with cement stabilized ICEBs and subjected to cyclic in-plane lateral loading. Wall specifications were varied to identify the shear performance of partial and fully grouted walls, and to observe the performance of a flexure dominated wall panel. It was determined that the shear strength of fully grouted walls is significantly higher than that of partially grouted walls and calculation of capacity based on current ACI 530-08 masonry provisions significantly overestimates the shear strength of ICEB wall panels. Based on the observed performance, recommendations are made for limiting the calculated nominal shear strength in design. Results also indicate that calculations based on simple bending theory conservatively predict the flexural strength of a fully grouted ICEB wall. Discussion of ICEB material properties and recommendations for design and construction procedures are included.

Keywords: Interlocking Dry Stack Masonry, Compressed Earth, Shear Wall Testing

Acknowledgments

This project was influenced by the dedication and effort of many individuals, all of which have my deep gratitude and appreciation.

Thank you to Clayton Proto and Brad Stirling, who have dedicated countless hours to this project; from making blocks to building walls. Your hard work and friendship are truly appreciated.

I would like to thank my advisor, Daniel Jansen, for your guidance, helpful ideas, enthusiasm, and dedication. It is an honor to have worked on this project with you, and I will never forget your passion and commitment to excellence. Much thanks to Peter Laursen, Bing Qu, and Charles Chadwell for all of your help and valuable insight into structural testing and engineering principles.

Much thanks to my family for their support throughout my entire education.

To my beautiful wife, Carly: Thank you for your unconditional love, strength, and encouragement, I am truly blessed. I love you.

Table of Contents

List of Tables	viii
List of Figures.....	ix
Chapter 1. Introduction.....	1
1.1 Background.....	1
1.2 Objectives and Scope.....	4
1.3 Organization of contents	5
Chapter 2. Literature Review	7
2.1 Compressed Earth Blocks.....	7
2.1.1 <i>Compressed Earth Block Presses</i>	7
2.1.2 <i>CEB Material Properties</i>	13
2.1.3 <i>CEB Experimental Testing</i>	14
2.2 Masonry Shear Walls.....	17
2.2.1 <i>Building Code</i>	18
2.2.2 <i>Lateral Force Resistance Mechanisms</i>	18
2.2.3 <i>Experimental Testing of Shear Walls</i>	23
Chapter 3. Materials	27
3.1 ICEBs.....	27
3.1.1 <i>Materials</i>	28
3.1.2 <i>Manufacturing</i>	29
3.2 Grout	38
3.2.1 <i>Grout Preparation</i>	39
3.3 Tested Material Properties	39
3.3.1 <i>ICEB Compressive Strength</i>	39
3.3.2 <i>Grout Compressive Strength</i>	40
3.3.3 <i>Masonry Prism Strength</i>	42
3.3.4 <i>Reinforcing Steel</i>	45
Chapter 4. ICEB Shear Wall Construction and Test Setup	46
4.1 Construction of Walls	46
4.1.1 <i>Installation of Longitudinal Reinforcing Steel</i>	46
4.1.2 <i>Lower Bond Beam Installation</i>	47
4.1.3 <i>Intermediate Course Stacking</i>	48
4.1.4 <i>Grout Lifts</i>	49
4.1.5 <i>Installation of Transverse Reinforcing Steel</i>	50
4.1.6 <i>Upper Bond Beam Construction</i>	50
4.1.7 <i>Installation of Loading Beam</i>	52
4.1.8 <i>Quality Control and Wall Curing</i>	53
4.2 Test Setup.....	53
4.3 Instrumentation	54
4.3.1 <i>Strain Gages</i>	55
4.4 Data Reduction.....	56
4.4.1 <i>Shear Deformation</i>	56
4.4.2 <i>Bending Deformation</i>	57
4.4.3 <i>Rocking Deformation</i>	58
4.4.4 <i>Base Sliding</i>	58

4.5	Loading Protocol.....	59
4.6	Failure Criteria.....	61
4.7	Prediction of Wall Capacity.....	61
4.7.1	<i>Shear Capacity</i>	62
4.7.2	<i>Flexural Capacity</i>	62
Chapter 5.	ICEB Shear Wall Testing.....	63
5.1	Wall 1.....	64
5.1.1	<i>Wall 1 Testing Results</i>	65
5.2	Wall 2.....	68
5.2.1	<i>Wall 2 Testing Results</i>	70
5.3	Wall 3.....	72
5.3.1	<i>Wall 3(a) Testing Results</i>	73
5.3.2	<i>Wall 3 Repair</i>	75
5.3.3	<i>Wall 3(b) Results</i>	77
Chapter 6.	Experimental Results and Analysis.....	81
6.1	Introduction.....	81
6.2	Shear Behavior.....	83
6.2.1	<i>ICEB Masonry Shear Resisting Mechanisms</i>	85
6.2.2	<i>Effective Shear Area Correction Factor</i>	87
6.2.3	<i>Contribution of Transverse Reinforcement</i>	89
6.3	Flexural Capacity.....	89
6.4	Wall Displacement.....	90
6.4.1	<i>Wall Stiffness Calculation-Elastic Analysis</i>	90
6.4.2	<i>Measured Wall Displacement Components</i>	92
6.5	Static Non-Linear Analysis.....	94
6.5.1	<i>Moment-Curvature Analysis</i>	95
6.5.2	<i>Inelastic Static Pushover</i>	96
Chapter 7.	Conclusions and Recommendations.....	100
7.1	Conclusions.....	100
7.1.1	<i>ICEB Materials and Testing Conclusions:</i>	100
7.1.2	<i>ICEB Shear Wall Testing Conclusions</i>	101
7.1.3	<i>ICEB Wall Construction</i>	104
7.2	Recommendations for Future Work.....	105
References		107
Appendix A.	Shear and Flexural Displacement Components	110
Appendix B.	Correction of Lateral Displacement Data	118
Appendix C.	Material Testing Data	122
Appendix D.	Instrumentation Layout.....	125
Appendix E.	Wall Displacement Components	128
Appendix F.	Wall 2.....	132

List of Tables

Table 3.1: Soil plasticity per ASTM D4318-05	28
Table 3.2: ICEB batch proportions (8 Block)	31
Table 3.3: Block charge weights (wet)	34
Table 3.4: Model BP6 Press ICEB variations	35
Table 3.5: Material compressive strengths	39
Table 3.6: Grout cylinder dimensions	41
Table 4.1: Imposed experimental cyclic loading protocol	60
Table 5.1: ICEB Shear Wall Specifications	63
Table 5.2: ICEB shear wall testing experimental results	64
Table 6.1: Alternative ICEB Shear Areas	86
Table 6.2: Nominal masonry shear strength calculation alternatives (P=4.5 kN)	87
Table 6.3: Back calculated “effective shear area”	88
Table 6.4: Dimensions for displacement component calculations	93
Table C.1: ICEB prism compressive strength	122
Table C.2: Grout compressive strength	123
Table C.3: ICEB compressive strength	123
Table C.4: Rebar tensile test data	124

List of Figures

Figure 2.1: CINVA Ram press (from Maini 2010b).....	8
Figure 2.2: Aurum 3000 block variations (from Maini 2010b).....	10
Figure 2.3: Hydraform dry stacking system (Hydraform).....	11
Figure 2.4: ICEB block dimensions in cm (from Wheeler 2005).....	12
Figure 2.5: ICEB block variations (Wheeler 2005).....	13
Figure 2.6: Assumed flexural strain and stress.....	21
Figure 3.1: Grain size distribution for soil and sand.....	29
Figure 3.2: Soeng Thai Model SP3 pulverizer.....	30
Figure 3.3: Wet mixing ICEB batch.....	32
Figure 3.4: Drop test (from Proto et al. 2010).....	33
Figure 3.5: Loading Soeng Thai Model BP6 manual press.....	34
Figure 3.6: 1 st lift pre-compression by hand.....	36
Figure 3.7: Pocket penetrometer test.....	37
Figure 3.8: Initial curing rack.....	37
Figure 3.9: ICEB compressive failure mode.....	40
Figure 3.10: Grout cylinder (porous) compressive failure.....	41
Figure 3.11: Masonry prism test setup.....	43
Figure 3.12: Masonry prism compression stress vs. strain.....	43
Figure 4.1: Installation of vertical rebar.....	47
Figure 4.2: Lower bond beam rebar installation.....	48
Figure 4.3: Ratchet straps used to stabilize wall before grouting.....	49
Figure 4.4: Roofing nail used to improve vertical alignment of block courses.....	49
Figure 4.5: Partially grouted ICEB wall (Wall 1).....	50
Figure 4.6: Anchor installation with #3 and #4 longitudinal rebar.....	51
Figure 4.7: Upper bond beam with #3 longitudinal reinforcement.....	52
Figure 4.8: Loading beam installed at top of wall.....	53
Figure 4.9: Out-of-plane bracing frame.....	54
Figure 4.10: Wall instrumentation.....	56
Figure 4.11: Wall dimensions used for calculating deformation components.....	57
Figure 4.12: Rocking displacement.....	58
Figure 4.13: Imposed experimental cyclic loading protocol.....	60
Figure 4.14: Testing failure criteria.....	61
Figure 5.1: Wall 1 detail (units in mm).....	65
Figure 5.2: Force-displacement response of Wall 1.....	66
Figure 5.3: Initial failure of Wall 1 after 4.0 mm pull (0.22 % drift).....	67
Figure 5.4: Shear cracking initiated at upper corner after 8.0 mm cycle.....	68
Figure 5.5: Wall 1 damage.....	68
Figure 5.6: Wall 2 details (units in mm).....	69
Figure 5.7: Force-displacement response of Wall 2.....	70
Figure 5.8: Damage to Wall 2 after testing.....	71
Figure 5.9: Wall 2 damage.....	72
Figure 5.10: Wall 3 detail (units in mm).....	73
Figure 5.11: Force-displacement response of Wall 3(a).....	74
Figure 5.12: Local failure of Wall 3(a) at 20 mm pull (1.11% drift).....	75

Figure 5.13: Wall 3(a) local damage after ± 20 mm	75
Figure 5.14: Loose blocks removed in preparation for grouting repair	76
Figure 5.15: Grout repaired corner and epoxy surface coating	77
Figure 5.16: External wooden shear panel (both sides)	77
Figure 5.17: Force-displacement response of Wall 3(b)	78
Figure 5.18: Spalling of masonry due to buckling of longitudinal rebar	79
Figure 5.19: Wall 3(b) damage pattern	80
Figure 6.1: Force-displacement histories	82
Figure 6.2: Force-displacement envelopes	83
Figure 6.3: Shear cracking mechanism (Wall 2 at 16mm push)	84
Figure 6.4: Transformed elastic stiffness predictions	92
Figure 6.5: Measured displacement components	93
Figure 6.6: Moment-Curvature plot for ICEB wall specimen	96
Figure 6.7: Static pushover analysis dimensions	97
Figure 6.8: Non linear static pushover (Wall 1)	99
Figure 6.9: Non linear static pushover (Wall 2, Wall 3)	99
Figure A.1: Wall panel section	110
Figure A.2: Nodal displacement of a panel section	111
Figure A.3: Components of panel deformation	112
Figure B.1: Actuator to LVDT displacement “gap” vs actuator displacement	119
Figure B.2: Corrected Wall 1 force-displacement response	120
Figure B.3: Corrected Wall 2 force-displacement response	120
Figure B.4: Corrected Wall 3(a) force-displacement response	121
Figure B.5: Corrected Wall 3(b) force-displacement response	121
Figure C.1: Steel rebar tensile test data	124
Figure D.1: Instrumentation layout [Wall 1]	125
Figure D.2: Instrumentation layout [Wall 2 and Wall 3(a)]	126
Figure D.3: Instrumentation layout [Wall 3(b)]	127
Figure E.1: Decoupled force-displacement envelope [Wall 1]	128
Figure E.2: Decoupled force-displacement envelope [Wall 2]	129
Figure E.3: Decoupled force-displacement envelope [Wall 3(a)]	130
Figure E.4: Decoupled force-displacement envelope [Wall 3(b)]	131
Figure F.1: Wall 2 force-displacement history	132
Figure F.2: Initial shear crack [W2-1 st 8mm PULL cycle @ +43.0 kN, +5.25 mm]	133
Figure F.3: Initial shear crack [W2-1 st 8mm PUSH cycle @ -39.5 kN, -5.5 mm]	134

Chapter 1. Introduction

1.1 Background

Interlocking compressed earth block (ICEB) masonry is a building technology that has the potential to provide affordable and sustainable home building materials to many people throughout the globe. Worldwide, the high cost of materials and experienced labor has prevented many people from building with appropriate, safe building methods.

Earth is one of the oldest building materials, with documented use as far back as 2500 BC (Maini 2010a). Today, the most common uses of earth as a construction material are adobe, compressed earth masonry, and rammed earth. Historically, adobe construction has been implemented without the use of stabilizers, while both compressed earth and rammed earth construction often employ the use of soil stabilization with cement, lime, fly ash or other materials. Earth construction is used extensively around the world due to the sustainable and economical use of indigenous soils as well as the significant reduction in manufactured materials required. A typical stabilized compressed earth block consists of less than 10% portland cement by weight which

significantly reduces the amount of cement necessary to build a masonry structure (Walker 1999). From an environmental perspective, earth construction allows for a large reduction in embodied energy as compared to building with concrete or kiln fired clay masonry (Walker 1999).

Masonry construction is commonly used due to its inherent durability, aesthetic appeal and ability to resist gravity loads. However, it relies heavily on the use of skilled craftsmen, and in regions of moderate to high seismicity requires significant attention to reinforcing steel detailing to provide sufficient earthquake resistance. Hollow interlocking dry-stacked masonry provides a significant labor advantage over traditional mortared masonry because the interlocking mechanism of the blocks allow mortarless construction (Jaafar et al. 2006). Reductions in labor cost up to 80% are realized due to the increased output and constructability of hollow dry-stacked reinforced masonry systems (Anand and Ramamurthy 2005).

In developed countries, extensive research and testing have driven advances in masonry building codes which have led to improved earthquake performance and safety. ICEB masonry is progressing as a building technology, but has not been the subject of any significant structural testing to evaluate its overall performance as a building method. Although significant research has been done to optimize the material properties, manufacturing and availability of ICEB technology, little has been done to investigate the performance of structural systems currently being built using this technology. The underlying goal of this research is to observe and evaluate the in-plane performance of shear walls built with ICEBs. Cyclic testing of shear wall units provides very good insight into the expected seismic performance of current structures incorporating ICEB

technology and in addition, will contribute to more accurate and reliable prediction of the strength and performance of ICEB walls.

This research builds upon an existing relationship between the Cal Poly San Luis Obispo Engineers Without Borders (EWB) chapter and the Center for Vocational Building Technology (CVBT) in Thailand. Cal Poly students have worked with the CVBT from afar, and also in Thailand during internships. Previous students from various departments have worked with the CVBT to improve ICEB system, including:

Cal Poly Mechanical Engineering:

- Students worked on a project to improve the design of the Soeng Thai BP6 manual block press used to produce ICEBs, specifically the handle latching mechanism.
- Students worked on optimizing the design and operation of the Soeng Thai SP3 soil pulverizer, which is used to prepare soil for manufacturing.
- Students designed and manufactured low cost pocket penetrometers for more economical quality control of ICEB manufacturing.

Cal Poly Civil Engineering:

- Students worked to provide example structural engineering calculations and recommendations for several ICEB structures
- Students conducted research and performed experimental testing to quantify the basic material properties of ICEB masonry, including compressive strength, durability and bond strength (Bales et al. 2009).

- Students organized an ICEB manufacturing and construction manual aimed at helping promote safer building practices with ICEB masonry (Proto et al. 2010).

1.2 Objectives and Scope

The objective of this experimental program was to conduct cyclic in-plane testing of ICEB shear walls to observe the failure modes and performance. Recommendations for acceptable structural design and construction techniques should be based on experimental results from testing. The primary goals for the initial wall testing were to identify and quantify the lateral force resisting mechanisms and behavior of ICEB shear walls, namely flexure and shear.

The ICEB shear wall testing program was initially proposed to include the experimental testing of six ICEB shear walls. Phase 1, which is reported in this thesis, consisted of the first three walls and included:

- Development of a suitable compressed earth block mixture for manufacture of ICEBs for construction of test specimens, based on single block compressive strength, appearance, quality, and ease of manufacture.
- Development of an ICEB shear wall testing program, including necessary testing fixtures, procedures and instrumentation.
- Testing of three ICEB shear walls with 1:1 aspect ratio ($H/L=1.0$). The goal was to observe shear and flexural capacity and performance.
- Comparison of shear and flexural capacity to nominal strengths predicted by current masonry design methods.

1.3 Organization of contents

This thesis presents the results of the first phase of experimental testing of ICEB shear walls. It is organized to provide the reader with an understanding of the background, testing methods and results, and analysis procedures used for the initial experimental evaluation of the ICEB building system.

Chapter 2 provides background in the context of compressed earth masonry and masonry shear wall mechanics. A review of published literature is provided which covers previous research of compressed earth block material properties and testing, and experimental testing of in-plane masonry shear and flexural behavior.

Chapter 3 covers the material properties and processes used to prepare and manufacture the ICEBs and other materials used in this experiment. Results from material testing are presented and discussed in relation to previous experimental studies and their influence on structural performance.

Chapter 4 describes the shear wall test setup, instrumentation and testing procedure. Methods used for reduction of experimentally measured displacement data are discussed, in addition to the procedure used to initially predict the shear and flexural wall capacities. The procedures and details used for construction of the ICEB shear wall specimens are covered. Discussion of specific testing goals, details, specifications and testing results are provided for individual wall specimens.

Chapter 5 presents the experimental results of Phase 1 of the ICEB shear wall testing program, and provides discussion and analysis of general performance and behavior. Observed experimental results of shear dominated ICEB wall panels are

discussed and compared to predictions from common reinforced masonry design methods, and an alternative method for predicting the shear strength of ICEB wall panels is suggested. Also, observed flexural failure mechanisms and capacities are discussed and compared to conventional theory. Measured force displacement response is compared to conventional design methods for predicting initial and cracked wall stiffness. Wall panel shear, bending and rocking displacement components are calculated and compared to observed results, and effectiveness of the wall instrumentation scheme is discussed. Finally, the result of a non linear static “pushover” analysis is compared to the experimentally observed wall response.

Chapter 6 summarizes the conclusions and recommendations from Phase 1 of the ICEB shear wall testing experimental program, including comments on material properties and testing, ICEB shear wall testing and performance, ICEB wall construction and recommendations for future research with ICEB structural systems.

Chapter 2. Literature Review

This chapter presents background to the subjects of compressed earth masonry technology and masonry shear wall design. A review of published research relevant to the experimental testing of ICEB shear wall specimens is provided within the context of compressed earth block (CEB) masonry construction and conventional masonry shear walls.

2.1 Compressed Earth Blocks

Compressed earth block technology takes advantage of the natural properties of highly compressed or compacted soil. Depending on the type of soil, compaction method and use of binders like portland cement, compressed earth blocks can be produced which are suitable for construction of load bearing structures. The most common uses of CEBs today are for bearing or non bearing shear wall systems, and as infill in confined masonry type structures.

2.1.1 Compressed Earth Block Presses

The idea of compressed earth blocks (CEB) has been around since the early 19th century. French architect Francois Cointeraux developed pre-cast compacted earth bricks

in small wooden molds in 1803 (Maini 2010b). The first modern version of this technology, the CINVA Ram press, was developed in 1956 in Columbia by Paul Ramirez (Wheeler 2005). Figure 2.1 shows the CINVA Ram press.

Figure 2.1: CINVA Ram press (from Maini 2010b)

Since the invention of the CINVA Ram many modifications have been made to the original idea resulting in different variations of block sizes and types, as well as mechanical automation to increase manufacturing productivity. Manually operated presses are most commonly used for smaller production runs due to their mobility and ability to be used without a source of electricity or fuel. However, large scale manufacturing of compressed earth demands the higher productivity inherent with hydraulic powered presses. Several companies currently manufacture their own version of compressed earth block press. Aside from mechanical design, the major differences between the different press types are the size and style of blocks which can be pressed. For the purposes of this review, only three specific brands of block presses are discussed; Aurum Press 3000, Hydraform M7, and Soeng Thai Model BP6. Many other presses are

currently available worldwide, but these three presses produce the most commonly used types of interlocking CEBs.

Current compressed earth block presses use either vertical or horizontal press action to form and eject the blocks. This orientation of pressing is important due to the significant effect it has on finished product tolerances. Due to the inherent variability of soil as a material and the design of the press, finished pressed block dimensions can vary as much as 5mm. A vertically pressed block will have tight dimensions in the horizontal plane, but the block height can vary depending on the press type, quality control methods and soil type. Conversely, a horizontally pressed block will have tight dimensions in its vertical cross section, but the length of the block will vary.

The **Aurum 3000** is a manual vertical block press manufactured by Aureka in Tamil Nadu, India. It can be used with 15 different molds to produce about 75 different regular and interlocking block variations, some of which are shown in Figure 2.2. The Aurum block system has been designed to allow for installation of reinforcing steel for earthquake resistant construction. Most blocks are meant to be used with mortar between joints, but the Series 300 dry interlocking blocks are designed to be dry stacked and filled with fluid grout after.

The **Hydraform M7** series of block presses are manufactured by Hydraform in Johannesburg, South Africa. Different press models are available for different budgets and manufacturing goals. The Hydraform compressed earth blocks are horizontally pressed earth blocks which interlock in two planes. Shown in Figure 2.3, the Hydraform block system is designed to be mostly dry stacked. Hydraform dry stacking blocks are

commonly used as infill for confined masonry structures, but can also be constructed for use with vertical and horizontal reinforcing bars by means of a special press insert.

Figure 2.2: Aurum 3000 block variations (from Maini 2010b)

Figure 2.3: Hydraform dry stacking system (Hydraform)

The **Soeng Thai BP6** press is a modern descendent of the original CINVA Ram press. The BP6 is a manually operated vertical block press manufactured by the CVBT in Thailand. A typical building block is shown in Figure 2.4, and Figure 2.5 shows the 9 variations of block which can be manufactured using the BP6 press, and the common uses for the different types of blocks. The ICEB system is a combination between a typical CEB (or “CINVA Brick”) and an interlocking dry stack masonry unit. The interlocking dowels are intended to provide ease of block alignment during construction and resistance to lateral in and out-of-plane forces. The round reinforcement holes are provided to allow use of grouted vertical reinforcement, and the rectangular holes or “grout key channels” are provided to help ensure wall stability and proper load distribution. The grout key channels also help minimize cracking of blocks due to

uneven load distribution across gaps. This style of block was originally introduced in 1983 by the Thailand Institute of Scientific and Technological Research (TISTR) and later modified by the Asian Institute of Technology (Pathum Thani, Thailand) and Soil Block Development Company (Chiang Rai, Thailand) into the block which today is commonly called the “Rhino Block” or ICEB (interlocking compressed earth block)(Wheeler 2005). The ICEB variations allow for reinforced dry stacked masonry construction which can be grouted after stacking.

Figure 2.4: ICEB block dimensions in cm (from Wheeler 2005)

Figure 2.5: ICEB block variations (Wheeler 2005)

2.1.2 CEB Material Properties

Significant research has been done to understand and optimize the material properties of stabilized and un-stabilized CEBs. Although many resources are available to help determine the suitability of soil for CEB manufacturing, **Shildkamp (2009)** presents the most comprehensive guide to soil selection, stabilization and CEB manufacturing techniques. More geared to ICEB construction, **Proto et al. (2010)** describes manufacturing processes and tips which are specific to the Soeng Thai BP6

press. Both of these papers were consulted for development of a suitable soil mixture for this experimental program, further described in Section 3.1.

2.1.3 CEB Experimental Testing

In order to have a good amount of confidence in the safety and durability of structures built using CEBs, it is critical to understand the material properties and behavior as gathered from experimental testing of specimens. This section summarizes published research which is most relevant to the experimental testing of CEB material properties and performance.

Bei and Papayianni (2003) conducted experimental testing to determine the effect of soil to sand mixture proportions on the compressive strength of un-stabilized CEBs at a constant compaction level. Test specimens included single blocks, doublet and triplet block prisms with aspect ratio h/L of 1.4 and 2.2, respectively, and small walls with dimensions 770mm (height) x 800mm (width) x 120 mm (thickness). Block prisms and walls were constructed using a 10mm soil cement mortar. The experimental testing indicated that the mode of failure in compression of earth blocks is dependent on the h/L ratio. For doublet and triplet block prisms (stacked prisms of two or three blocks), the failure mode is similar to observed crushing of masonry walls, with noticeable hourglass shape. Most notable is that the measured stress strain response of compressed earth masonry under compressive loading is non-linear with very little elastic behavior. Measured strains of 0.007 at peak compressive stress were significantly high compared to typical strains for concrete or fired brick masonry, which commonly are closer to 0.0025 and 0.0035, respectively.

Perera and Jayasinghe (2003) performed experimental compressive strength testing to determine the performance of CEB wall panels of dimension 890mm (length) by 590mm (height) by 140mm (thickness). The testing was aimed at determining the load-deflection behavior, stress at first cracking and ultimate compressive strength of twenty axially loaded CEB wall panels. Experimental results were evaluated using limit state principals from BS 5628: Part 1 1992. The variables in the experiment were the fines content of the soil (20%-45%) and the cement content per block (2%-8%). Experimental results showed that the compressive strength of compressed earth blocks is dependent on the fines content of the soil used, the mixture cement content and the compaction ratio. The authors noted that for (tested) soils with fines percentages below 30% and cement content between 4% and 8%, walls constructed using Aurum blocks meet the requirements of BS 5628: Part 1 1992 based on serviceability and ultimate limit states for axial loading.

Heathcote (1991) performed compressive testing on cement stabilized CEBs in order to better quantify the effects of soil type, cement content and density on material strength. It was found that measured compressive strength of block units is dependent on confinement during testing, and an aspect ratio factor of 0, 0.5, 0.76 and 1.0 was used for height to thickness ratios of 0, 0.4, 1.0 and ≥ 5 , respectively. This factor is used to adjust the measured compressive strength to account for confinement due to platen effects. Based on the experimental results, a simple formula was created to predict the compressive strength of CEBs based on cement content and density. This formula cannot be extrapolated to earth block mixtures based on other soil types, and further work is necessary to introduce a formula which also considers soil clay content.

Pave (2007) conducted a series of experimental testing using the Hydraform dry stack CEB system. The author conducted testing to determine the influence of cement and moisture contents in addition to the general performance of the system. Several dry stack wall systems with dimensions 3000mm by 2500mm were tested under centric axial loading. Individual block units and small wallets, consisting of 4 or 8 blocks, were also tested.

In order to determine if the blocks were manufactured with consistent density, cube specimens were cut from various locations of the pressed blocks. The measured compressive strength of cube specimens cut from the top of the pressed earth block was approximately 78% of specimens cut from the bottom. Recommendations were made specific to the Hydraform dry stacking system regarding use of half and corner blocks which would be more susceptible to failures because of this non uniform strength.

The experimental results indicated that the wall panel strength of dry-stack systems is directly proportional to the strength of the masonry units with a ratio of panel compressive strength to unit compressive strength of 0.3 for the dry stacked system. Under uniform compression loading the Hydraform dry-stacked panels typically failed by development of vertical cracks through the center.

Out of plane flexural strength of dry-stack masonry/composite reinforced concrete beams was evaluated by testing. Beams were constructed with dry stack blocks and reinforcing steel, with or without reinforced concrete bond beam sections. Experimental results indicated that simple flexural theory accurately predicted the flexural capacity of the reinforced beams and lintels, and that more work is necessary to validate shear capacity calculations.

Bales et al. (2009) performed a series of experiments which were originally aimed at preceding experimental in-plane lateral testing of ICEB shear wall specimens. The experimental program was designed to examine properties of ICEBs which would be relevant to shear wall mechanics. The authors initially manufactured compressed earth blocks with varying soil types and cement stabilization to determine influences on durability, compaction and compressive strength of dry and soaked ICEB specimens.

Once a satisfactory earth block mixture was developed the authors conducted compressive testing of individual blocks to determine the unit compressive strength (f_{cb}), and testing of grouted and un-grouted masonry prisms consisting of 3 ICEBs stacked vertically. The masonry prisms were tested surrounded by a wooden form to provide lateral confinement as would be provided by surrounding masonry wall. This masonry prism testing was determined to be the most accurate way of determining the actual compressive strength of the masonry (f_{cp}). The panel strength reduction is attributed to platen effects for low aspect ratios, and is given by the ratio of unit compressive strength to prism compressive strength (f_{cp}/f_{cb}), which was an average of 0.43 and 0.37 for fully grouted and ungrouted prism specimens, respectively. This strength reduction is fairly consistent with previous compressive testing of dry stack panels by Pave (2007) and Jaafar et al. (2006). The typical compressive failure mode for ICEB masonry prisms was characterized by splitting of blocks around the grout column.

2.2 Masonry Shear Walls

Shear walls are generally used as part of a structural lateral force resisting system, often simultaneously resisting vertical gravity loads and lateral loads from horizontal diaphragms due to wind or earthquakes. Shear walls for buildings located in regions of

moderate to high seismicity require reinforcing steel to provide the strength and ductility necessary to perform well under seismic loading.

This section is arranged to provide an overview of the failure modes and resistance mechanisms of reinforced masonry shear walls subjected to lateral in-plane loading. Then, a review of previous experimental shear wall testing is provided. Although no research has been published regarding the in-plane lateral performance of ICEB shear walls, the mechanisms of resistance and building methods used are generally similar to conventional concrete masonry shear walls. Consequently, this chapter covers theory and research from a conventional reinforced concrete and masonry background.

2.2.1 Building Code

Analysis and design methods for reinforced masonry structures are dependent on the applicable building codes and specific requirements set forth by local government and building officials. Although codes and requirements for masonry design and construction vary depending on the locale, the methods of design are generally based on the same basic principles of mechanics. This thesis refers to the design requirements and theory provided by the Building Code Requirements and Specification for Masonry Structures (TMS 402-08/ACI 530-08/ASCE 5-08 and TMS 602-08/ACI 530.1-08/ASCE 6-08), a standard reported by the Masonry Standards Joint Committee (MSJC).

2.2.2 Lateral Force Resistance Mechanisms

Aside from failure of lap splices, debonding of rebar or bed joint sliding, cantilever masonry shear walls subject to lateral in plane loading and restrained from out of plane movement by support or connection conditions typically fail in one of two

different modes: flexure or shear (Voon 2007). This section covers the mechanisms of in-plane resistance provided by a concrete masonry shear wall.

2.2.2.1 Flexural Resistance

Flexural failure is characterized by yielding of longitudinal reinforcing steel in tension and/or crushing of masonry in compression due to bending stresses which result from overturning moment. Generally, flexural failure is preferred for seismic design due to the increased ductility and energy dissipation provided by tensile yielding of the steel (Voon and Ingham 2006).

Flexural resistance of member can be calculated according to basic principles of mechanics. Figure 2.6 shows the assumed strain and stress distributions which correspond with the following assumptions (MSJC 2008 Section 3.3.2):

1. The reinforcement, grout and masonry are assumed to resist loads in a composite action with strain continuity.
2. The nominal resistance of reinforced masonry to combined flexural and axial loads shall be based on equilibrium conditions.
3. Maximum usable strain in the extreme masonry compression fiber, ϵ_u , shall be taken as 0.0035 for clay masonry and 0.0025 for concrete masonry.
4. Strain in reinforcement and masonry are assumed to be directly proportional to the distance from the neutral axis (i.e. plane sections remain plane).
5. Compressive and tensile stress in reinforcement shall be taken as E_s multiplied by the steel strain, but no greater than yield f_y .
6. The tensile strength of masonry shall be neglected for calculation of flexural strength but shall be considered for calculation of deflections.

7. Masonry compressive stress shall be assumed to be $0.8f_m$ over an equivalent rectangular stress block with depth $a = 0.80c$, where c is the depth from neutral axis to extreme compressive fiber.

For a section as shown in Figure 2.6 with externally applied concentric axial load P , the nominal moment capacity can be calculated as follows:

$$M_n = C_m \left(c - \frac{a}{2} \right) + \sum_i^n C_{si} (c - d_i) + \sum_i^n T_{si} (d_i - c) + P \left(\frac{L}{2} - c \right) \quad (2-1)$$

where C_{si} and T_{si} represent the compressive and tensile forces in the longitudinal steel, with $F_{si} = A_s E_s \epsilon_{si} \leq A_s f_y$, and P is the applied axial load. For longitudinal steel which is unconfined, which is common for masonry shear walls, the contribution of steel in compression can be neglected. The compression in the masonry, C_m , can be calculated using an equivalent uniform rectangular stress block with width, b and depth, $a = \beta c$, where β is taken to be 0.8. The masonry compressive stress over this area is taken to be αf_m where α is assumed to be 0.8. The neutral axis depth c can be found by strain compatibility based on equilibrium of vertical forces. The nominal lateral strength, F_n of a masonry shear wall with effective height H_e can be expressed as:

$$F_n = \frac{M_n}{H_e} \quad (2-2)$$

Figure 2.6: Assumed flexural strain and stress

2.2.2.2 Shear Resistance

Shear failure is characterized by inclined cracking associated with tension splitting along a compression strut formed in a wall subjected to lateral loading. Shear failure is commonly more brittle than flexural failure, and is often accompanied by

sudden degradation of strength and stiffness (Voon and Ingham 2006). The shear strength of masonry is complex, and no theoretical method exists which predicts it. Research has shown that shear resistance is a combination of several mechanisms including tension of horizontal reinforcement, dowel action of vertical reinforcement, applied axial stress and aggregate interlocking (Voon and Ingham 2006). Based on empirical research, ACI 530-08 allows strength calculation of shear capacity by means of Equation 2-3 (MSJC 2008 Eqn. 3-19).

$$V_n = V_{nm} + V_{ns} \quad (2-3)$$

The contribution of masonry is calculated as:

$$V_{nm} = 0.083 \left[4.0 - 1.75 \left(\frac{M_u}{V_u d_v} \right) \right] A_n \sqrt{f'_m} + 0.25P \quad (2-4)$$

where A_n is the net cross sectional wall area (mm^2), M_u is the factored moment (N-mm), V_u is the factored shear force (N), d_v is the shear depth of the wall (mm), f'_m is the specified masonry compressive strength, and P is the vertically applied axial load (N).

The contribution of transverse reinforcing steel is calculated as:

$$V_{ns} = 0.5 \left(\frac{A_v}{s} \right) f_y d_v \quad (2-5)$$

where A_v is the area (mm^2), s is the spacing (mm), and f_y is the yield strength (MPa) of transverse reinforcing steel.

In order to reduce the chances of brittle shear failure, the nominal shear strength V_n is limited by Equations 2-6 and 2-7:

For $M_u/V_u d_v \leq 0.25$:

$$V_n \leq 6A_n \sqrt{f'_m} \quad (2-6)$$

Where $M_u/V_u d_v \geq 1.00$:

$$V_n \leq 4A_n \sqrt{f'_m} \quad (2-7)$$

For $0.25 \leq M_u/V_u d_v \leq 1.00$, linear interpolation between Equations 2-6 and 2-7 is permitted.

2.2.3 Experimental Testing of Shear Walls

This section contains a summary of published research which is relevant to the lateral in-plane experimental testing of shear wall performance and capacity. The following experimental studies were done using conventional CMU or clay fired brick masonry.

Shing et al. (1990) conducted in-plane cyclic testing of twenty-two 1830mm by 1830mm reinforced masonry walls in order to compare actual flexural and shear capacity to code based predictions. Sixteen specimens were constructed with hollow concrete blocks and six with hollow clay bricks. All walls were fully grouted with uniformly distributed horizontal and vertical reinforcement. All horizontal steel was hooked 180° around continuous vertical bars. Open face bond beam units were used throughout each wall for grout continuity. One specimen had wire mesh confinement in bed joints. The walls were constructed with varying reinforcing steel contents; 0.38 to 0.74% vertical reinforcing and 0.14 to 0.26% horizontal reinforcing. Uniform axial load, between 0 and 1.93 MPa, was applied with two servo controlled actuators.

The results indicated that simple flexural beam theory can closely predict the flexural capacity of reinforced masonry shear walls. The actual flexural strength can be increased by strain hardening of flexural steel due to large displacements. This effect is shown to reduce as the applied axial load increases. Also, the extent of post peak strength degradation for shear dominated wall specimens depends on the applied axial stress, the amount of vertical reinforcement and the compressive strength of the masonry.

Sucuoglu and McNiven (1991) investigated the experimental results from testing of eighteen 1400 mm high by 1200 mm wide single wythe concrete and clay brick masonry walls. The walls were chosen from a larger set of thirty experimental specimens because they exhibited a shear dominated failure mode. Wall testing parameters were varied to determine the influence of axial compressive stress and horizontal reinforcement. The authors concluded that axial compressive stresses have a significant effect on both the cracking and ultimate strengths of shear dominated shear panels. Also, no correlation was shown between the amount of shear reinforcement and the residual shear strength after diagonal cracking.

Voon and Ingham (2006) conducted in-plane cyclic testing of ten single-story reinforced concrete masonry cantilever shear walls in order to compare actual shear strength to predicted values based on code based requirements. The walls were constructed with standard production 15 series CMUs. All walls had uniformly distributed horizontal and vertical reinforcement. Horizontal steel was hooked 180° around continuous vertical bars. Open face bond beam units were used throughout each wall for grout continuity. The variables between walls were:

1. Horizontal reinforcing steel content between 0 and 0.14%.
2. Applied axial stress was varied from 0 to 0.50 MPa.
3. Eight walls were fully grouted, two partially grouted.
4. Eight walls were 1.8m x 1.8m ($H/L=1.0$), one was 3.6m x 1.8m ($H/L=0.5$) and one was 1.8m x 3.6m ($H/L=2.0$).
5. All walls except one were designed to fail in a shear dominated manner.

The authors noted that the ACI 530-08 method (Equation 2-3) for calculating nominal shear capacity reasonably predicted the shear strength of walls with aspect ratio (H/L) of 1 or less, but over-predicted the strength of the wall with an aspect ratio of 2. This indicates that masonry shear strength is inversely proportional to aspect ratio. Experimental results also showed that uniformly distributed shear reinforcement can increase the strength and ductility of shear failures by redistributing stresses. In addition, shear capacity increases with additional axial stress, but the post peak behavior is more brittle. These findings are consistent with previous research (Shing et al. 1990). Testing results from partially grouted walls showed significantly lower strength capacities than fully grouted walls. Calculations using the net shear area were shown to predict the lower strengths.

Shedid et al. (2009) conducted a series of experimental testing in order to quantify the ductility and energy dissipation of flexure dominated reinforced concrete masonry shear walls. Six walls, with H/L of 2.0, were subjected to reversed cyclic loading. Walls were subjected to varying vertical (0.29 to 1.31%) and horizontal (0.08 to 1.13%) reinforcing steel ratios. The testing program was designed so that ordinary, intermediate and special reinforced shear wall detailing could be represented and

evaluated. The experimental results were reduced to identify parameters such as shear and flexural displacement components, displacement ductility and energy dissipation.

For flexure dominated walls with H_e/L of 2.0, shear displacements were approximately 32% of total lateral displacement at first yielding of outer vertical reinforcement. At maximum load, shear displacements accounted for 20% of the total displacement. Tested walls reached 1.0% drift with little to no strength degradation, and about 1.5% drift at roughly 20% strength degradation. Increasing vertical reinforcement ratios resulted in decreasing drift capacities. It is noted that instrumentation set up to measure the strain profile along the wall showed that the strain was essentially linear for walls which were not subjected to axial stresses, but minor non linearity was shown for walls which were subjected to axial compressive stresses.

Chapter 3. Materials

This chapter covers the materials used for construction of the three ICEB shear wall specimens tested in this experimental program. It includes specifications, manufacturing processes, quality control methods, and measured material properties for the compressed stabilized blocks, grout material, and reinforcing steel used in the experimental program.

3.1 ICEBs

The Interlocking Compressed Earth Blocks used in this experiment (see Figure 2.4) were all constructed by student volunteers at Cal Poly using the following methods and materials. Determination of the specific mixture used for the blocks was done following procedures common to the current use of stabilized compressed earth technology. Presented here is the final iteration of several block mixtures, which was chosen based on ease of manufacture, block compressive strength and visual appearance.

3.1.1 Materials

3.1.1.1 Soil

The soil used for manufacturing of the ICEBs was procured from a local excavation site. The grain size distribution and plasticity of the soil was determined from a soil sample using applicable ASTM testing procedures. Figure 3.1 shows a soil grain size distribution based on sieve and hydrometer analyses conforming to ASTM D422-63. The soil consists of approximately 21% clay (particles finer than 0.002 mm). The plasticity of the soil was determined according to ASTM D4318-05, and is shown in Table 3.1.

Table 3.1: Soil plasticity per ASTM D4318-05

Liquid Limit (%)	Plastic Limit (%)	PI (%)
36	15	21

3.1.1.2 Sand

The sand used for ICEB manufacturing was medium-fine sand obtained Sisquoc river and had been cleaned and sieved for use in concrete. It was further screened to remove particles larger than 4 mm, and stored dry in bins. A grain size distribution based on an ASTM D422-63 sieve analysis is presented in Figure 3.1.

3.1.1.3 Cement

Commonly available Type I/II portland cement was used for manufacturing of all ICEBs, mortar, and grout.

Figure 3.1: Grain size distribution for soil and sand

3.1.2 Manufacturing

3.1.2.1 Material Preparation

Air dried soil was prepared for manufacture of compressed earth using a Soeng Thai Model SP3 soil pulverizer. The SP3 pulverizer, shown in Figure 3.2, uses rotating hammers to break down the chunky dry soil, allowing it to pass through a 4 mm screen. The pulverized soil was stored dry in bins. Sand was also air dried and stored in bins.

Figure 3.2: Soeng Thai Model SP3 pulverizer

3.1.2.2 Material Quantities

Soil batches were prepared for eight blocks at a time, which provided adequate time to finish pressing the blocks without excessive retention time. The batch proportions are presented in Table 3.2. In order to provide consistency, all materials were stored and weighed dry. Materials were weighed to the nearest 0.1 kg on digital scales. The proportion of water shown in Table 3.2 is approximate because the water was not added by mass, but rather to specific performance and visual specifications as discussed in Section 3.1.2.4.

Table 3.2: ICEB batch proportions (8 Block)

	Weight (kg)	% of Total
Soil	50.0	74.3
Sand	6.7	10.0
Cement	4.2	6.2
Water	~ 6.4	9.5
Total	67.3	100.0

3.1.2.3 Dry Mix

Once weighed, the materials were placed into a portable cement mixer for an initial dry mix. Use of the cement mixer allowed for uniform material consistency with a significant reduction in manual labor. Due to the cohesive nature of the clayey soil, it is not feasible to use a cement mixer with wet or moist soil mixtures.

3.1.2.4 Wet Mix

The uniformly blended dry mixture was dumped onto a flat concrete surface to begin wet mixing. The mixture was watered uniformly with a watering can while being blended with a shovel (See Figure 3.3). Care was taken to avoid watering too fast, as this tends to clump the soil and leads to non uniform soil moisture.

Figure 3.3: Wet mixing ICEB batch

The optimum water content for compressed stabilized blocks is dependent on many factors such as soil type, clay content and cement content. Previous testing with ICEBs manufactured using this soil type established the target moisture content based on the workability of the mixture. A common method used for establishing the best water content is called the “drop test.” Detailed in Figure 3.4, the drop test was used as the primary moisture control method. The soil-cement batch was mixed until it was homogeneous and satisfied the drop test criteria.

Drop Test

1. Take a handful of moistened soil and shape it into a ball using a moderate amount of pressure.
2. Drop the ball of soil onto a hard surface from a height of 1 m.
3. Observe the reaction of the ball.

Observed Result	Conclusion
Ball shattered or broke into more than 7 pieces	Soil is too dry
Ball broke into 4-6 pieces	Water content suitable for compression
Ball broke into 3 or less pieces, or didn't break at all	Soil is too wet

4. Repeat Drop Test at least 3 times to get an average result.

Figure 3.4: Drop test (from Proto et al. 2010)

Once mixed to optimal water content, the soil was weighed into individual buckets which contained one block “charges.” The wet charge weights of various blocks are listed in Table 3.3. The optimum block charge weight is most closely dependant on the calculated dry density of the block. Note that the Half and Half-Channel blocks are made two at a time as described in Section 3.1.2.5. The ICEBs were manufactured with a minimum calculated dry density of 1850 kg/m^3 .

Table 3.3: Block charge weights (wet)

Block	Charge Weight (kg)
Standard	8.0
Channel	7.3
Half (2)	8.0
Half-Channel (2)	7.3

3.1.2.5 Press Inserts

The ICEBs were compressed using a Soeng Thai Model BP6 manual block press, shown in Figure 3.5. Table 3.4 lists the 8 different block types used for the different walls. Manufacturing different block shapes is accomplished by simply adding or removing press inserts to change the form shape. Wall 1 was constructed using corner blocks at the ends for aesthetic purposes, which was determined to be unnecessary for subsequent walls.

Figure 3.5: Loading Soeng Thai Model BP6 manual press

Table 3.4: Model BP6 Press ICEB variations

Block	Description	Used for Wall
Standard	Full size, 5 holes	1,2,3
Channel	Full size, 5 holes- with channel insert	1,2,3
Half Block	Standard block with divider insert	2,3
Half Channel	Channel block with divider insert	2,3
Corner	Full size, 4 holes- remove one end insert and change bottom plate	1
Channel Corner	Full size, 4 holes- remove one end insert, change bottom plate and add channel insert	1
Half Corner	Change bottom plate, add divider insert, remove both end inserts	1
Half Channel Corner	Change bottom plate, add divider insert, remove both end inserts, add channel insert	1

3.1.2.6 Block Pressing

Once the soil cement mixture was weighed into buckets and the appropriate press inserts were installed, the charges were loaded into the BP6 press. For manufacturing of half blocks the press divider insert was sprayed with WD-40 to ensure clean block separation. In order to reach the target block dry density, the charges were poured into the block press in two separate lifts. This was done because the loose volume of soil would not fit into the press without initial compression by hand, shown in Figure 3.6. The compression stroke was a minimum of 2 seconds, and the compression was held at maximum compression for another 2 seconds.

Figure 3.6: 1st lift pre-compression by hand

3.1.2.7 Pocket Penetrometer

Care must be taken during manufacturing to ensure final product quality and consistency. One of the most common checks for consistency and proper charge weight is the use of the pocket penetrometer. Shown in Figure 3.7, the pocket penetrometer measures the bearing strength of the pressed soil block immediately after pressing. Although the bearing strength is not a very useful property for compressed earth block manufacturing, the pocket penetrometer test is a good indicator of material consistency and block density, and a good way to identify charges which were too small or big.

Figure 3.7: Pocket penetrometer test

3.1.2.8 Initial Cure

After ejection, the ICEBs were carefully moved from the press to the indoor curing rack (see Figure 3.8). The blocks were stored there for a maximum of one day, and watered at least two times. The benefit of this initial cure was a lower incidence of broken blocks because they had sufficient time to cure before handling and stacking.

Figure 3.8: Initial curing rack

3.1.2.9 Humid Covered Curing

Once allowed to cure indoors for 24 hours, the blocks were moved outside onto pallets and stacked 7 blocks high. The blocks were covered with plastic sheet, which was removed for watering daily for a minimum of 7 days. After humid curing, the blocks were transported to the structural testing lab to prepare for construction.

3.2 Grout

Due to the limited cavity space available for reinforcement and grouting it is important for the grout to be fine and have a very high slump. To produce a workable grout mixture at this pourable consistency, it is necessary to increase the fines content in order to prevent bleeding and segregation of aggregates. However, simply adding more cement to the mix decreases the water to cement ratio and will result in a grout which is much stronger than the soil block.

Previous compressive strength testing of ICEB prisms (Bales et al. 2009) indicates that large differences in material properties between cement/sand grout and stabilized compressed soil lead to a brittle failure mode dominated by splitting of the soil block around the grout at the interface and that the grout and soil were taking the load separately. For this testing program, it was decided to use a weaker grout mixture (closer in compressive strength to that of the blocks). The grout used for construction of the shear walls had approximate proportions of 1:0.4:2.6:4.2 (portland cement:lime:water:sand) by dry volume.

3.2.1 Grout Preparation

Grout was prepared to the above dry volume proportions in 20 liter batches. The dry materials were slowly blended with small amounts of water until a uniform consistency was reached. Water was added until the grout was a flowing consistency, which was necessary for pouring the grout through a funnel with an opening of 20 mm.

3.3 Tested Material Properties

Material compressive strengths were experimentally determined by compressive testing. Table 3.5 summarizes the compressive strengths for the individual ICEBs, grouted prisms and grout cylinder specimens, where prism samples were subjected to either load controlled (denoted σ) or strain rate compressive loading (denoted ϵ).

Appendix C presents the material testing data.

Table 3.5: Material compressive strengths

Sample Type	Number of Samples	Compressive Strength, MPa		Coefficient of Variation
		Gross	Net	
ICEB	8	6.6	7.6	9%
Partially Grouted Prism (σ)	2	3.3	2.5	19%
Partially Grouted Prism (ϵ)	4	2.1	1.6	5%
Fully Grouted Prism (σ)	4	4.2	-	9%
Fully Grouted Prism (ϵ)	4	3.0	-	7%
Grout Cylinder (Porous)	6	9.2	-	21%
Grout Cylinder (Non-Porous)	4	5.1	-	4%

3.3.1 ICEB Compressive Strength

ICEB compressive strength was determined by testing of individual ICEBs using a universal testing machine. Individual ICEBs were tested with the top and bottom press plates from the BP6 press used to distribute the load to the block. This was determined to be the most effective method of uniformly compressing the ICEBs. For stress

calculations, the net and gross areas of an ICEB are 39300 mm^2 and 45000 mm^2 , respectively. The common failure mode for single ICEBs in compression was a conical break where the sides fall off, as shown in Figure 3.9. This type of break is consistent with previous experimental compressive testing of ICEBs (Bales et al. 2009).

Figure 3.9: ICEB compressive failure mode

3.3.2 Grout Compressive Strength

Grout compressive strength was determined by testing of cylinders cast in plastic molds (denoted non-porous grout sample) as well as directly into the round reinforcement hole of spare blocks (porous grout sample). At the time of testing the porous grout sample, the cylinder was removed from the surrounding soil block. These two different types of samples were tested to determine the effect of the absorption of water by the surrounding blocks. Final grout strength is dependent on the amount of moisture

absorbed by the ICEBs at time of construction, and can be better determined by this quality control method. The values in Table 3.5 show the relative variability of the material strengths for the porous samples (COV 21%) compared to the non-porous grout samples (COV 4%). The higher variance in material strengths is indicative of the nature of CEBs. Varying water content and block density likely influenced the curing conditions for the grout cast into CEB material. Table 3.6 shows the dimensions of both the porous and non-porous cylinders. Cylinders were capped with a sulfur capping compound before testing. Figure 3.10 shows a porous grout cylinder compression test.

Table 3.6: Grout cylinder dimensions

Sample Type	Dimensions (mm)	Area (mm ²)
Porous	45 x 100	1590
Non-Porous	51 x 102	2043

Figure 3.10: Grout cylinder (porous) compressive failure

3.3.3 Masonry Prism Strength

The average masonry compressive strength (f'_m) was determined by testing of grouted prisms under stress rate compression loading (samples denoted as Fully Grouted Prism (σ) in Table 3.5). The prisms, consisting of three ICEBs stacked vertically, were constructed at the same time as the shear walls and subjected to the same curing conditions. Masonry prisms were also tested under strain rate compression as shown in Figure 3.11, with external confinement provided by thread rods which were hand tightened. The ratio of height to minimum width was 2.0, and reported compressive strength values are not modified for aspect ratio. The top and bottom surfaces were capped with plaster. Material strains were measured with both external extensometers and LVDTs. The extensometers provided the most accurate strain readings until compressive failure caused external cracking, at which point they were removed. The LVDTs were left on for the entire test.

The masonry prisms exhibited fairly large strains at ultimate failure. Figure 3.12 shows a plot of the stress strain curve for four fully grouted masonry prism (strain rate controlled) compression tests. As can be seen in the plot, the ICEB masonry prisms experience a very ductile compressive failure mode with strains above 0.01.

Figure 3.11: Masonry prism test setup

Figure 3.12: Masonry prism compression stress vs. strain

ACI 530-08 Section 1.8.2.2.1 recommends that the modulus of elasticity may be estimated from the chord modulus of elasticity taken between 0.05 and 0.33 of the maximum compressive strength from prism testing. Following this procedure, the mean ICEB masonry modulus of elasticity was calculated to be 575 MPa. This is approximately equal to $137 f'_m$, which is significantly lower than the values commonly assumed for clay and concrete masonry of $700 f'_m$ and $900 f'_m$, respectively. The measured stress-strain behavior does not differentiate between masonry strains and displacements caused by closing gaps at the dry stack interfaces. Further compressive testing of ICEB stacked prisms and panels should be conducted to better identify the

The compressive strength reduction from individual block (f'_{cb}) to prism (f'_{cp}), expressed as a ratio of f'_{cp}/f'_{cb} , was 0.43 to 0.55 for partially and fully grouted prisms, respectively. This strength reduction, associated with the increase in aspect ratio, is slightly lower than for values of 0.37 and 0.43 reported by Bales et al. (2009).

3.3.3.1 Modified Hognestad Model

For later non-linear analysis, a modified Hognestad model was fitted to the stress-strain data as shown in Figure 3.12. The modified Hognestad model uses a second-degree parabola with apex at strain ϵ_o , followed by a downward sloping line terminating at the maximum compressive strain limit, ϵ_{cu} (MacGregor and Wight 2005). The parameters of the model were modified to match the experimental data from the masonry prism testing.

$$f_m = f'_{mo} \left(\frac{2\epsilon_c}{\epsilon_o} - \left(\frac{\epsilon_c}{\epsilon_o} \right)^2 \right) \quad (\text{for } \epsilon_c \leq \epsilon_o) \quad (3-1)$$

For strain values between the strain at maximum stress and the maximum compressive strain limit

$$f_c = f'_0 - 0.25f'_{m0} \left(\frac{\epsilon_c - \epsilon_0}{\epsilon_{cu} - \epsilon_0} \right) \quad (\text{for } \epsilon_0 \leq \epsilon_c \leq \epsilon_{cu}) \quad (3-2)$$

The model depicted in Figure 3.12 was created for $f'_{m0} = 3.0$ MPa, assuming ϵ_0 to be 0.012 and ϵ_{cu} limited at 0.025.

3.3.4 Reinforcing Steel

Samples of the steel reinforcement used for wall construction were subjected to tensile testing, with average yield strength (f_y) for the #3 and #4 bars of 378 MPa and 565 MPa, respectively. Appendix C includes data from tensile testing of steel rebar.

Chapter 4. ICEB Shear Wall Construction and Test Setup

This chapter covers the specifics of the ICEB construction procedures used to build the specimens as well as the details of the test setup, instrumentation, and displacement controlled loading protocol. Also, a description of the processes used for data reduction of displacement measurements is provided. Finally, an overview of the methods used to predict the nominal shear and flexural strength capacities of the walls is detailed.

4.1 Construction of Walls

Walls were constructed to the design specifications following common ICEB building practices, which were determined from the “Interlocking Compressed Earth Blocks Volume II. Manual of Construction” (Wheeler 2005) published by Geoffrey Wheeler at CVBT. The chosen detailing and wall specifications, which are covered in Chapter 5, were based on previously established testing goals and objectives as well as interpretation of currently used construction details.

4.1.1 Installation of Longitudinal Reinforcing Steel

Vertical bars were installed to the moveable concrete footing using a common two part epoxy construction adhesive. 4 mm oversize diameter holes were drilled into the

reinforced concrete footing and cleaned out prior to installation per the epoxy manufacturer instructions. Figure 4.1 shows installed longitudinal reinforcing bars, which were intentionally left long to be cut after installation of the top block course. Strain gages were installed onto the rebar prior to installation into the footing.

Figure 4.1: Installation of vertical rebar

4.1.2 Lower Bond Beam Installation

The lower bond beam was installed to the foundation on a 20 mm bed of Type S mortar. Subsequent courses were not installed until the mortar had sufficient time to set up, which allowed for better ability to keep the wall level and plumb during construction. No. 3 transverse steel was installed into the bond beam typical as shown in Figure 4.2, with a 90 degree hook on each end. An alternate construction method would be a 180 degree hook around the longitudinal reinforcing bar, but minimum inner bend radius

requirements would prohibit proper fitment of the hook into the width of the ICEB channel block. Disadvantages of the “bent over” detailing as used are discussed in Chapter 5.

Figure 4.2: Lower bond beam rebar installation

4.1.3 Intermediate Course Stacking

ICEBs were stacked in running bond pattern, in low lifts of maximum 4 courses. Before installation of grout, blocks were clamped in place with ratchet straps after being leveled and adjusted to final position, as shown in Figure 4.3. Consistent with CVBT construction practice, nails were used to make minor adjustments to the height and alignment of block courses. Figure 4.4 shows a nail being used to adjust the height of an ICEB course.

Figure 4.3: Ratchet straps used to stabilize wall before grouting

Figure 4.4: Roofing nail used to improve vertical alignment of block courses

4.1.4 Grout Lifts

The low strength fine grout was installed in low lift application every 4 courses of block. Details on grout mixture, preparation and quality control are in Section 3.2. The grout was poured into the wall cavities through a funnel, and rodded quickly after pouring.

Wall 1 was partially grouted, which consisted of filling grout in every rectangular vertical grout channel, horizontal bond beam channel, and any vertical round hole which contained reinforcing steel. Wall 2 and Wall 3 were fully grouted, which consisted of

filling grout in every cavity. Figure 4.5 shows an intermediate course of Wall 1, which was partially grouted.

Figure 4.5: Partially grouted ICEB wall (Wall 1)

4.1.5 Installation of Transverse Reinforcing Steel

For Wall 3, transverse reinforcing steel was installed in intermediate bond beam courses in the same manner as the lower bond beam, as described in section 4.1.2. The intermediate bond beams were placed at the 5th, 9th, and 13th courses.

4.1.6 Upper Bond Beam Construction

The upper bond beam was constructed two different ways, depending on the size of the longitudinal reinforcing steel. Figure 4.6(a) shows the detail for anchor installation

used on Wall 1. With #4 (13mm) longitudinal bars, it was decided to not bend the bar due to the risk of damage to the wall; the transverse reinforcement was bent down at each end and installed with the anchors. 12.7 mm (1/2 in) anchors, provided for connection to the loading beam, were installed at each longitudinal rebar before the final grout lift.

Figure 4.6: Anchor installation with #3 and #4 longitudinal rebar

For Wall 2 and Wall 3, the #3 longitudinal bars can be bent in place without local damage to the blocks. Figure 4.6(b) shows the detail for anchor installation with #3 longitudinal bar bent 90 degrees into the upper channel. This installation method allows for better grout continuity in the upper channel. The wall was left so that the grout could harden overnight before bending the longitudinal rebar into the channel. The final grout lift was terminated at mid height of the upper bond beam in order to reduce the effect of a cold joint. Figure 4.7 shows the upper bond beam channel just prior to installation of the loading beam. Reinforcing bar tails were cut at a minimum of 200 mm for 90 degree bends.

Figure 4.7: Upper bond beam with #3 longitudinal reinforcement

4.1.7 Installation of Loading Beam

The steel loading beam, consisting of two steel channels welded together, was installed onto the upper bond beam with a 20 mm bed of Type S mortar (1.0:0.4:4.5). The four anchors cast into the upper bond beam were bolted through the loading beam as shown in Figure 4.8. The anchors were left hand tight while the mortar set, and fully tightened once the mortar had hardened.

Figure 4.8: Loading beam installed at top of wall

4.1.8 Quality Control and Wall Curing

After construction, the walls were watered twice daily for at least 3 days to maintain humidity. Samples of grout, mortar, and grouted prisms were constructed at the same time as the walls for testing of material properties. A summary of these material properties is included in Section 3.3.

4.2 Test Setup

The laboratory facilities for shear wall testing consisted of a +240/-365 kN servo controlled actuator mounted to a rigid concrete strong wall, which transferred lateral load to the top of the wall via a steel box section, herein referred to as the “loading beam.” The walls were built on a moveable reinforced concrete footing which was mounted to the rigid strong floor using eight 38.1 mm (1.5 inch) bolts. The walls were braced out of plane by four steel rollers attached to an adjustable steel reaction frame. A schematic of

the out of plane bracing system is shown in Figure 4.9. The axial load distributed to the wall was approximately 4.5 kN, which consisted of the self weight of the loading beam and actuator in addition to approximately 3.6 kN of steel angle which was stacked on the loading beam as shown in Figure 4.9.

Figure 4.9: Out-of-plane bracing frame

4.3 Instrumentation

The wall specimens were typically instrumented as shown in Figure 4.10. All data was collected using a computer data acquisition system. Although exact locations of some instruments were changed slightly for each wall, the general layout of the instrumentation was installed as shown in Figure 4.10. Dimensions of specific instrumentation layouts for different walls are provided in Appendix D. The instrumentation was generally intended to measure the following displacements:

- The applied lateral force and displacement at the actuator were measured by the integrated load cell and displacement transducer (denoted as instrument 0 in Figure 4.10.).
- In-plane lateral deflections at the top and mid-height locations were measured by both string potentiometers and LVDTs (instruments 1,2, and 3). Between Wall 1 and Wall 2 the upper LVDT location was changed to measure from the loading beam in order to reduce errors due to block spalling.
- Vertical panel displacement components were measured at each end (wall face) of the walls using string potentiometers (instruments 4 and 5).
- Diagonal panel displacements were measured using string potentiometers (instruments 6 and 7). The string potentiometers were all mounted to common studs that were mounted onto the face of the wall.
- Rocking or uplift at the wall base was measured by LVDTs (instruments 8 and 9)
- Relative slippage between loading beam and wall, wall and footing, footing and floor were measured with LVDTs (instruments 10,11, and 12)

4.3.1 Strain Gages

Strain gages were installed at the base of the outermost longitudinal rebar, approximately 100 mm (4 in) from the surface of the concrete footing.

Figure 4.10: Wall instrumentation

4.4 Data Reduction

The wall panel displacements for a cantilevered shear wall which is instrumented as shown in Figure 4.10 can be separated into 3 separate components: Rocking, flexural and shear/sliding deformations. Figure 4.11 shows the dimensions used for calculation of individual wall component deformations, which was done using a simplified procedure modified after Voon (2007); Appendix A provides a detailed description of the process used to separate the flexural and shear deformation components.

4.4.1 Shear Deformation

The shear deformation was calculated using measured relative displacements between points on the wall panel face as:

$$u_s = \frac{d(\delta_{d1} - \delta_{d2})}{2L} - \frac{h^2}{6(2d_u + h)} \frac{(\delta_{v1} - \delta_{v2})}{L} \quad (4-1)$$

where δ is the displacement measured at specific transducers, with extension corresponding to positive displacement, and compression as negative.

Figure 4.11: Wall dimensions used for calculating deformation components

4.4.2 Bending Deformation

The bending deformation is calculated based on the measured rotation of the wall panel by:

$$u_b = \frac{(\delta_{v1} - \delta_{v2})}{L} \left(h \frac{d_u + \frac{2h}{3}}{2d_u + h} + d_u \right) \quad (4-2)$$

4.4.3 Rocking Deformation

The rocking component of displacement was determined by calculating the rotation, θ_r of the wall, as shown in Figure 4.12:

$$\theta_r = \frac{\delta_{r1} - \delta_{r2}}{L_w + 2l_s} \quad (4-3)$$

where δ_{r1} and δ_{r2} are the deformations measured by LVDTs at the base of the wall (instruments 8 and 9 in Figure 4.10) and l_s is the distance from the end of the wall and the center of the instrument. The rocking displacement is calculated by:

$$u_r = \theta_r h_e \quad (4-4)$$

Figure 4.12: Rocking displacement

4.4.4 Base Sliding

The sliding deformation was measured by LVDTs, shown in Figure 4.10 as instruments 10-12. The foundation and loading beam were purposely roughened to reduce the occurrence of slippage.

4.5 Loading Protocol

The pseudo-static cyclic loading protocol used for the ICEB shear wall testing program is shown in Figure 4.13 and Table 4.1. This protocol consisted of two (pull/push) cycles to each target displacement. The actuator was programmed to start with a pull displacement for each cycle. The target displacements are based on the actuator displacement transducer, and do not necessarily reflect the actual displacement of the wall panel (See Appendix B for more information). This loading protocol is preferred for cyclic testing of masonry shear walls for two reasons (Voon 2007):

- To avoid a high level of dependency on instrument readings during the process of testing. This protocol can be run regardless of changes during testing.
- The non-ductile nature of the shear dominated wall specimens means that small displacement increments are necessary to avoid specimen failure at an early stage of testing. This was more important for the ICEB walls because no experimental testing results exist which could indicate possible structural response.

Figure 4.13: Imposed experimental cyclic loading protocol

Table 4.1: Imposed experimental cyclic loading protocol

# of Cycles	Deflection (mm)	Deflection (in)	Loading Rate (in/sec)	% Drift
2	0.5	0.0197	0.003	0.03%
2	1	0.0394	0.004	0.06%
2	2	0.0787	0.006	0.11%
2	4	0.1575	0.009	0.22%
2	6	0.2362	0.013	0.33%
2	8	0.3150	0.013	0.44%
2	10	0.3937	0.013	0.56%
2	12	0.4724	0.013	0.67%
2	14	0.5512	0.013	0.78%
2	16	0.6299	0.013	0.89%
2	20	0.7874	0.017	1.11%
2	24	0.9449	0.020	1.33%
2	28	1.1024	0.023	1.56%
2	32	1.2598	0.027	1.78%
2	36	1.4173	0.030	2.00%
2	40	1.5748	0.033	2.22%

4.6 Failure Criteria

This thesis defines failure at the point where wall lateral strength is reduced to 80% of the maximum measured lateral force. Figure 4.14 shows the definitions of the yield (δ_y) and ultimate (δ_u) displacement capacities which were reported in this thesis. For calculation of the yield displacement, an equivalent elastoplastic curve is fitted to the measured force-displacement envelope where area a_1 is equal to a_2 .

Figure 4.14: Testing failure criteria

4.7 Prediction of Wall Capacity

Predictions of wall strength were based on ACI 530-08 strength design methodology. The specified masonry strength, f'_m was taken as the average compressive strength determined by stacked masonry prism testing, covered in Section 3.3.3. Table 5.2 summarizes the predictions for the ICEB wall specimens. For calculation of shear

stresses, the effective shear areas used for partially grouted (Wall 1) and fully grouted (Wall 2 and Wall 3) walls were $246 \times 10^3 \text{ mm}^2$ and $259 \times 10^3 \text{ mm}^2$, respectively.

4.7.1 Shear Capacity

The lateral in-plane shear capacities of the ICEB shear walls were predicted according to Equation 2-4. For the wall geometry used, $(M_u/V_u d_v)$ is taken as 1.0. A_n is the width of the wall (150 mm) times d_v (1725 mm) less any un-grouted reinforcement holes. The steel area A_v is 71 mm^2 (0.11 in^2) and steel spacing s is 500 mm or 400 mm. The steel yield strength, f_y , was taken as the average measured yield strength for #3 bars of 378 MPa.

4.7.2 Flexural Capacity

The lateral in-plane flexural capacities, F_n , of the ICEB shear walls were predicted using classical flexural mechanics according to the requirements of ACI 530-08 as described in Section 2.2.2.1.

Chapter 5. ICEB Shear Wall Testing

Three 1800 mm x 1800 mm cantilever shear walls were constructed using ICEBs and subjected to cyclic in-plane loading. This chapter provides details, specifications, and testing results for individual walls. The variables between the three walls included full or partial grouting, longitudinal reinforcement content, and transverse reinforcement content. Table 5.1 summarizes the general specifications and intended failure modes of the three shear walls.

Table 5.1: ICEB Shear Wall Specifications

Wall	H (mm)	L (mm)	Intended Failure Mode	Vert. Rebar	Horz. Rebar	Grouting	Axial Load (kN)
1	1800	1800	Shear	(4) #4	-	Partial	4.5
2	1800	1800	Shear	(4) #3	-	Full	4.5
3	1800	1800	Flexure	(4) #3	(3) #3	Full	4.5

Table 5.2 presents the predicted nominal shear (V_n) and flexural (F_n) capacities for each wall, in addition to the experimental results. The masonry prism strength, f'_m , used for each calculation was taken as the average measured compressive strength determined by load controlled compressive testing of fully grouted ICEB prisms (see

Section 3.3.3), and A_n is the net shear area of the wall (see Section 4.7.1). The experimentally determined maximum lateral force (V_{max}) is reported for both pull (+) and push (-) directions, and δ_{vmax} is the corresponding displacement at maximum lateral force. The measured yield (δ_y) and ultimate (δ_u) displacements are also reported in mm and % drift. For comparative purposes, the ratio of experimentally determined strength to predicted nominal strength is reported for both shear and flexure. Wall 3 was subjected to two separate loading conditions, which are described further in Section 5.3. Unless otherwise noted, all displacements reported in this thesis are modified from the measured actuator (ACT) and LVDT displacements as described in Appendix B.

Table 5.2: ICEB shear wall testing experimental results

Wall	f'_m (MPa)	A_n (mm ²)	Prediction		Test Result										Failure Mode
			V_n (kN)	F_n (kN)	V_{max} (kN)	V_{max}/V_n	V_{max}/F_n	δ_y (mm) (%)		δ_{vmax} (mm) (%)		δ_u (mm) (%)			
1	4.2	246x10 ³	95.3	107	Pull	21.8	0.23	-	2.3	0.13	3.9	0.22	6.5	0.36	Shear
					Push	27.3	0.29	-	3.0	0.17	5.4	0.30	7.7	0.43	
2	4.2	259x10 ³	100.2	50.3	Pull	43.0	0.43	-	5.6	0.31	5.6	0.31	8.8	0.49	Shear
					Push	41.8	0.42	-	5.2	0.29	5.2	0.29	7.0	0.39	
3(a)	4.2	259x10 ³	146.5	50.3	Pull	51.6	-	1.03	9.8	0.54	12.2	0.68	15.7	0.87	Flexure
3(b)	4.2	259x10 ³	158.0	50.3	Pull	49.9	-	0.99	8.1	0.45	11.4	0.63	22.4	1.24	Flexure
					Push	48.1	-	0.96	11.0	0.61	11.0	0.61	28.1	1.56	

5.1 Wall 1

Wall 1 was constructed June 14, 2010 and tested 28 days later on August 11, 2010. The testing goal for Wall 1 was to produce a shear dominated failure of a partially grouted wall with 1:1 aspect ratio. Initial calculations for the flexural and shear capacity of the wall predicted (see Table 5.2 for a summary of predictions and results) that four #4 vertical bars would provide sufficient flexural strength to induce a shear failure for a wall

with no horizontal bars. Figure 5.1 shows a detail of Wall 1 as constructed. The general design specifications for Wall 1 as related to this experimental program are:

- Corner blocks were used at each end, resulting in no end grout channels
- Partially grouted as described in Section 4.1.4
- #4 Vertical (longitudinal) rebar (for detail see Figure 4.6 (a))
- No horizontal (transverse) rebar
- Upper and lower bond beam as described in Sections 4.1.2 and 4.1.6.

Figure 5.1: Wall 1 detail (units in mm)

5.1.1 Wall 1 Testing Results

Wall 1 was subjected to the cyclic displacement loading protocol up to a maximum actuator displacement of ± 12 mm (0.67% drift). The external LVDTs and string potentiometers were removed from the wall after the ± 10 mm cycles. Figure 5.2 shows the force displacement response for Wall 1.

Figure 5.2: Force-displacement response of Wall 1

Wall 1 was a shear dominated failure with minor diagonal cracking through blocks and large separation and sliding displacements along block joints. Minor flexural tension cracks in grout bed joints were first identified on the 2.0 mm cycles. Figure 5.3 shows an initial local failure, which consisted of the soil block spalling off near the end anchor bolt embedment at the upper connection to the loading beam, on the 4.0 mm pull cycle. Post test investigation revealed poor grouting coverage in the crowded vertical reinforcement hole. Figure 5.4 shows the initiation of diagonal shear cracking from the upper corner.

As seen in the force displacement response (Figure 5.2), post peak strength and stiffness degradation for Wall 1 were gradual. The maximum lateral force ($V_{\max}$) was significantly lower in the pull direction than the push direction even though the pull half of each displacement cycle occurred before the push half. It is likely that the local failure

due to improper grout installation significantly weakened the wall in the pull direction.

Figure 5.5 shows the damage Wall 1 upon completion of testing. The damage pattern is consistent with shear failure of wall panels, characterized by diagonal cracking associated with tension splitting along the diagonal compression strut. As seen in Table 5.2, the ultimate shear strength of Wall 1 was significantly below the shear strength predicted by ACI 530-08.

Figure 5.3: Initial failure of Wall 1 after 4.0 mm pull (0.22 % drift)

Figure 5.4: Shear cracking initiated at upper corner after 8.0 mm cycle

Figure 5.5: Wall 1 damage

5.2 Wall 2

Wall 2 was constructed August 20, 2010 and tested 21 days later on September 3, 2010. The testing goal for Wall 2 was to produce a shear dominated failure of a fully grouted wall with 1:1 aspect ratio. After testing of Wall 1, it was concluded that the ACI

530-08 method for predicting in plane shear strength significantly overestimates the nominal capacity. Consequently, the nominal flexural capacity required to force a shear failure is lower; for Wall 2, #3 longitudinal reinforcing bars were used. Figure 5.6 shows a detail of Wall 2 as constructed. The general design specifications for Wall 2 as related to this experimental program are:

- Standard blocks were used at the ends, resulting in end grout channels
- Fully grouted
- #3 Vertical (longitudinal) rebar (for anchor detail see Figure 4.6 (b))
- No horizontal (transverse) rebar
- Upper and lower bond beam as described in Sections 4.1.2 and 4.1.6

Figure 5.6: Wall 2 details (units in mm)

5.2.1 Wall 2 Testing Results

Wall 2 was subjected to the cyclic displacement loading protocol up to a maximum actuator displacement of ± 16 mm (0.89% drift). The external LVDTs and string potentiometers were removed from the wall after the ± 10 mm cycles. Figure 5.7 shows the force displacement response for Wall 2.

Figure 5.7: Force-displacement response of Wall 2

Wall 2 was dominated by extensive diagonal cracking through blocks and along joints. Minor flexural cracking was first observed on the 2 mm cycle, followed by sudden formation of diagonal shear cracking during the first 8 mm pull cycle. Strength degradation did not immediately occur, and very extensive diagonal shear cracking occurred on the first Figure 5.8 shows the damage to Wall 2 upon completion of testing, which is consistent on both wall faces. The damage pattern is consistent with shear

failure of wall panels, characterized by diagonal cracking associated with tension splitting along the diagonal compression strut. As shown in Figure 5.9, diagonal cracking was observed to initiate and terminate along lines of vertical reinforcement.

a.) Back

b.) Front

Figure 5.8: Damage to Wall 2 after testing

Figure 5.9: Wall 2 damage

5.3 Wall 3

Wall 3 was constructed September 10, 2010 and initially tested 21 days later on October 1, 2010. The testing goal for Wall 3 was to produce a flexure dominated failure of a fully grouted wall with 1:1 aspect ratio. The only difference from Wall 2 was the addition of 3 layers of #3 transverse reinforcing bars. Figure 5.10 shows a detail of Wall 3 as constructed. Due to the geometry of the wall, symmetrical uniform distribution of horizontal rebar was not possible. The lower four bars (including bond beam) were spaced at 400 mm with the top two bars spaced at 500 mm. The general design specifications for Wall 3 as related to this experimental program are:

- Standard blocks were used at the ends, resulting in end grout channels
- Fully grouted
- #3 vertical (longitudinal) rebar (for anchor detail see Figure 4.6)
- #3 horizontal (transverse) rebar (for detail see Section 4.1.5)

- Upper and lower bond beam as described in Sections 4.1.2 and 4.1.6

Figure 5.10: Wall 3 detail (units in mm)

5.3.1 Wall 3(a) Testing Results

Wall 3(a) was subjected to the cyclic displacement loading protocol up to the ± 1.0 mm cycles, at which point an input error resulted in a pull displacement of 20 mm (1.11% drift) at the actuator. The hydraulic actuator was paused at 13 mm while it was returning to zero displacement. At that point it was decided to manually control the wall displacement to 20 mm in the push direction in order to quantify the damage from the initial excursion. Figure 5.11 shows the force displacement response for Wall 3(a).

Figure 5.11: Force-displacement response of Wall 3(a)

On the initial 20 mm pull displacement, Wall 3(a) was determined to be a flexure dominated failure, characterized by tensile flexural cracking and yielding of vertical rebar, followed immediately by a local failure at the upper section of the wall between the bond beam and fourth layer of horizontal reinforcement. Figure 5.12 shows the cracking at the upper connection level at 20 mm displacement. Upon load reversal, the wall failure was characterized by shear sliding along the previously cracked section. Figure 5.13 shows the final damage to Wall 3(a) after completion of pull and push (± 20 mm) displacement cycles.

Figure 5.12: Local failure of Wall 3(a) at 20 mm pull (1.11% drift)

Figure 5.13: Wall 3(a) local damage after ± 20 mm

5.3.2 Wall 3 Repair

Aside from flexural cracking and steel tensile yielding, the damage to Wall 3 after the initial loading was concentrated at the very top and likely would have occurred if the original loading protocol would have been followed. It was decided that repairs could be made to the wall for further testing. At both upper corners of the wall, loose and broken

blocks which could be easily removed were pulled off from both upper corners of the wall. Figure 5.14 shows an upper corner of the wall after all loose material was cleaned out. Forms were installed at each end, and the voids at the corners were filled with concrete grout.

Figure 5.14: Loose blocks removed in preparation for grouting repair

Wooden shear panels were built to attach to both sides of the wall. Before installing the shear panels, a two part epoxy construction adhesive was applied to both sides of the wall in order to ensure proper shear transfer between the wooden shear panels and the damaged masonry wall panel. Figure 5.15 shows the surface epoxy application and grout filled corner voids of Wall 3 prior to installation of the wooden shear panels. The external shear panels were installed to the wall with six 12.7 mm (1/2 in) threaded rods which were thru bolted. Figure 5.16 shows the installed external shear panel.

Figure 5.15: Grout repaired corner and epoxy surface coating

Figure 5.16: External wooden shear panel (both sides)

5.3.3 Wall 3(b) Results

After repair, Wall 3 was subjected to the cyclic displacement loading protocol up to a maximum displacement of ± 40 mm (2.22% drift). The external LVDTs and string

potentiometers were removed from the wall after the ± 20 mm cycles. Figure 5.17 shows the force displacement response for Wall 3(b).

Figure 5.17: Force-displacement response of Wall 3(b)

The repaired Wall 3(b) failed in flexure, characterized by yielding of longitudinal steel followed by buckling of longitudinal steel in compression. The wall had existing flexural cracks along bed joints from previous testing, and separation of blocks along the existing cracks was identified but not measured. Wall 3(b) exhibited a much more ductile failure mode compared to previous walls, which is attributed to the tensile yielding of longitudinal reinforcing bars, until the first 20 mm (1.11% drift) push displacement, at which point buckling longitudinal steel in compression caused spalling of masonry at the base of the wall, shown in Figure 5.18(a). Figure 5.18(b) shows the buckled longitudinal rebar after the masonry spalled off completely. At further

displacement levels, the buckling and subsequent masonry spalling moved inwards towards the middle of the wall panel. Figure 5.19 shows a diagram of the damage to Wall 3(b) upon completion of testing. After spalling of the masonry a significant portion of wall panel displacement due to shear sliding at the third and fourth course of blocks was identified.

Figure 5.18: Spalling of masonry due to buckling of longitudinal rebar

Figure 5.19: Wall 3(b) damage pattern

Chapter 6. Experimental Results and Analysis

This chapter provides an overview of the behavior of the tested shear wall specimens and compares the observed behavior to predictions using conventional methods of analysis, including discussion of shear capacity and flexural capacity in the context of ICEB material properties. The experimentally observed force-displacement response is compared to predictions using common elastic analysis methods for cracked and uncracked sections. Also, experimentally measured shear and flexural displacement components are presented and related to observed results. Observations from material and structural testing indicate that the ICEB shear wall behaves in a non-linear manner under even low level loading, so the results from a non linear static analysis are compared to the experimental results.

6.1 Introduction

Shear wall test results are summarized in Table 5.2, and Figure 6.1 shows the force displacement histories for each wall specimen. Measured force displacement (loading beam level) envelopes for all wall specimens are shown in Figure 6.2.

Figure 6.1: Force-displacement histories

Figure 6.2: Force-displacement envelopes

6.2 Shear Behavior

The cyclic loading response of shear dominated ICEB wall panels is similar to that of partially and fully grouted CMU shear walls tested (Voon and Ingham 2006), characterized by in cycle strength and stiffness degradation in addition to rapid post peak strength loss. The diagonal cracking behavior is also consistent with observed shear failure modes for shear wall panels. A significant portion of the shear cracking was not directly visible at the finish of testing due to the nature of the ICEB system. Figure 6.3 shows that while some cracking occurs through the face of blocks, much of the cracking is internal to the wall, which is evident by dry stacked bed joint sliding. Upon load

reversal, diagonal cracks closed up, and only cracks which extended to the face of the blocks were still identifiable.

Figure 6.3: Shear cracking mechanism (Wall 2 at 16mm push)

Comparison of predicted shear strengths using the ACI 530 method (Equation 2-3) to experimental strength values, given in Table 5.2, indicates that this analysis method significantly over predicts the shear strength of ICEB shear walls. This is most likely due to the nature of the shear resisting behavior of dry stacked interlocking blocks as compared to traditional mortared joint masonry. Research has indicated that the shear resistance of conventional CMU masonry comes from many mechanisms, including tension of horizontal reinforcing steel, dowel action of vertical steel, applied axial stress and aggregate interlocking (Voon and Ingham 2006). It is likely that all of these factors apply to dry stacked ICEBs, but the absence of mortared joints leads to much different stress transfer behavior between courses. Additionally, it may not be valid to extrapolate Equation 2-3 to lower strength materials such as ICEB walls; Section 3.1.8.1.1 of ACI

530-08 requires a minimum specified masonry compressive strength of 10.3 MPa which is significantly higher than that of the ICEBs used in this investigation.

6.2.1 ICEB Masonry Shear Resisting Mechanisms

ICEB construction differs considerably from conventional mortared CMU masonry construction. Most significant is the dry stacked interlocking mechanism which is aimed at providing ease of installation and construction. The secondary intent is that the interlocking bumps will also provide resistance to in and out-of-plane lateral forces. Depending on the quality control of the ICEB manufacturing and construction processes the finished fit of an ICEB structure can vary, and continuous vertical grouting is installed to help overcome the inadequacies of dry stacked construction by providing a more “solid” or continuous core. Conventional design methods for mortared masonry would rely on the assumption that the net cross-sectional wall area provides shear resistance, but the mechanics of the ICEB system and results from this experimental testing program indicate otherwise.

It is suggested that the entire wall cross section does not provide shear resistance, but perhaps only the grouted core region with a lesser contribution from friction between dry stacked block interfaces and “interlocking action.” It should be noted that while the net area of partially grouted Wall 1 was 95% of fully grouted Wall 2, the shear strength was between 51 and 65 % of Wall 2. This suggests that the shear strength contribution of the ICEB masonry does not increase linearly with the net area.

For comparative purposes, Table 6.1 shows some alternative “effective shear areas” for use in calculating the nominal shear strength of the ICEB shear walls tested in

this experiment. Pre-testing predictions of shear strength reported in Table 5.2 were calculated using Method A, which assumes that the f'_m used in Equation 2-4 is the masonry prism compressive strength acting over the entire net cross-sectional area. Similarly, Method B uses the measured masonry prism compressive strength, but assumes that shear stresses are not resisted by the full width of the ICEB wall, but only a portion. The “effective width” (50 mm) used for Method B was arbitrarily chosen to be 1/3 of the total wall thickness. Alternatively, Method C assumes that only the vertically continuous grouting contributes to the nominal masonry shear strength. This approach uses the measured compressive strength of the porous grout samples.

Table 6.1: Alternative ICEB Shear Areas

Method	Description	A_n (mm ²)	
		Wall 1	Wall 2
A	Measured masonry prism strength acting over net "shear" area of	246027	258750
B	Measured masonry prism strength acting over net "shear" area of wall using "effective width", $b =$	73527	86250
C	Measured (porous) grout strength acting over net area of grout	21362	34085

Table 6.2 shows the calculated nominal shear strengths for Wall 1 and Wall 2 using Equation 2-4, for each alternative shear area. The experimental results compared in Table 6.2 indicate that the nominal shear strength of ICEB shear walls is not directly proportional to the net cross sectional area as assumed by Method A. In addition, use of an “effective width” by Method B does not capture the strength reduction due to partial grouting. This indicates that continuous vertical grouting does more than provide shear

strength in terms of providing “shear continuity.” A common way to calculate the nominal masonry shear strength for partially grouted conventional masonry is to only consider the contribution of the face shell thickness (Voon and Ingham 2006). The shear cracking mechanisms observed from experimental ICEB shear wall testing (see Figure 6.3) indicate that this assumption is not appropriate for partially grouted ICEB walls. This is due to the nature of the dry stacked interlocking system, which provides stress transfer primarily through the core of the wall (not the face). It should also be noted that while Method C provides the most conservative estimate of the shear strength, it introduces another variable (porous grout compressive strength) which is highly dependent on the grout mixture, quality control and construction techniques used.

Table 6.2: Nominal masonry shear strength calculation alternatives (P=4.5 kN)

Wall	V_{exp} (kN)		Method	A_n (mm ²)	f'_m (MPa)	V_{nm} (kN)	V_{exp}/V_{nm}	
	Pull (+)	Push (-)					min	max
1	21.8	27.3	A	246027	3.4	85.8	0.25	0.32
			B	73527	3.4	26.4	0.82	1.03
			C	21362	9	13.1	1.67	2.09
2	43.0	41.8	A	258750	3.4	90.2	0.46	0.48
			B	86250	3.4	30.8	1.36	1.39
			C	34085	9	20.2	2.07	2.13

6.2.2 Effective Shear Area Correction Factor

It is proposed that for the calculation of ICEB masonry shear contribution using Equation 2-4, the shear area (A_n) should be reduced by an appropriate correction factor to account for the different mechanisms of shear resistance inherent in the ICEB system. Back calculation of the “effective shear area” from experimental results can be accomplished by rearranging Equation 2-4:

$$A_{eff} = \frac{V_{exp} - 0.25P}{0.083 \left[4.0 - 1.75 \left(\frac{M_u}{V_u d_v} \right) \right] \sqrt{f'_m}} \quad (5-1)$$

where A_{eff} is the effective shear area, and $M_u/V_u d_v$ is taken as 1.0. Use of Equation 5-1 with the experimentally determined maximum shear forces for the pull and push directions results in effective shear areas shown in Table 6.3.

Table 6.3: Back calculated “effective shear area”

Wall	f'_m (MPa)		V_{exp} (kN)	A_{eff} (mm ²)	A_{eff}/A_n
1	3.4	Pull (+)	21.8	60041	0.24
		Push (-)	27.3	76013	0.31
2	3.4	Pull (+)	41.8	118121.1	0.46
		Push (-)	43.0	121606	0.47

Table 6.3 suggests that modification of A_n by correction factors of 0.2 and 0.4 for partially and fully grouted walls, respectively, would conservatively predict the shear strength for the experimental specimens from this testing program. Note that this is for calculation of nominal shear capacity, and results are still subject to further reduction by an applicable strength reduction factor, typically 0.8 for shear. It should also be noted that this area reduction is calculated for the specific block strengths, wall geometry, and imposed axial load used in this experimental program. Further testing should be completed to verify the correction factors for walls with different grout or CEB compressive strength properties.

6.2.3 Contribution of Transverse Reinforcement

Horizontal steel reinforcing bars are typically referred to as “shear” or “transverse” reinforcement. Building code commonly require transverse reinforcement to be hooked around the extreme longitudinal reinforcing bars with a 180 degree bend. Because of rebar bend radius requirements, the transverse steel installed in the ICEB wall specimens was not hooked 180 due to limited width in the channel blocks, but instead bent 90 degrees into the vertical reinforcing holes as described in Section 4.1.2. From the experimental results of the monotonic Wall 3 testing, it is noted that the local failure mode which initiated at the top of the wall was most likely due to the stresses resulting from this reinforcement detail. Consequently, it is suggested that horizontal reinforcing bars be hooked 180 degrees around extreme vertical rebar at the expense of not meeting minimum bend radius requirements. Further experimental shear wall testing with varying horizontal reinforcing steel contents is necessary to better understand the shear contribution of reinforcing steel for ICEB shear walls.

6.3 Flexural Capacity

Under monotonic loading Wall 3 initially failed in flexure, characterized by tensile yielding of longitudinal reinforcing bars until a sudden shear type failure was initiated at the upper section due to improper reinforcement detailing. After repair, the failure mode was again dominated by tensile yielding of longitudinal reinforcing steel. The force displacement response was significantly more ductile than the previous walls, with negligible in-cycle strength reduction until compression rebar buckling resulted in block spalling at the toes of the wall as shown in Figure 5.18. The buckling failure of rebar is likely due to the effects of reversed cyclic loading, which causes extensional yielding of

longitudinal rebar in tension. Upon load reversal, the lengthened section of rebar will be subject to buckling. Table 5.2 shows that the classical flexural theory used to predict the nominal flexural capacity of Wall 3 is fairly accurate and slightly conservative, consistent with previous experimental results for flexure dominated conventional masonry shear wall panels (Shing et al. 1990).

6.4 Wall Displacement

Modern design methodology of reinforced concrete or masonry requires the designer to check the structural component capacity versus ultimate loading demand, and then verify that the component can satisfy certain displacement or cracking limits under service loading conditions. This section covers displacement calculations which are commonly used for in-plane masonry shear wall design and compares the results of these analyses to experimental data from ICEB shear wall specimen testing. Also, displacement data for the ICEB shear wall specimens is reduced into component displacements.

6.4.1 Wall Stiffness Calculation-Elastic Analysis

The stiffness of shear walls is used for both deflection checks and distribution of lateral diaphragm forces to shear walls. Distribution of lateral forces to shear walls is dependent on the relative stiffness of the wall panels, and un-cracked section analysis is often used for simplicity. For computation of lateral drift or deflection, however, ACI 530-08 Section 3.1.5.3 specifically requires calculations to account for the effect of cracking and reinforcement on stiffness. According to the MDG-5 (TMS 2007), for the case of a solid cantilever shear wall subjected to a single point load at the tip, P , the deflection can be calculated as:

$$\Delta_c = P \left(\frac{h^3}{3E_m I} + \frac{1.2h}{E_v A} \right) \quad (5-2)$$

where the two terms refer to the flexural and shear deformation components. The shear modulus, E_v , is taken as $0.4E_m$. The wall stiffness can be represented as:

$$k_c = \frac{P}{\Delta_c} \quad (5-3)$$

The experimental results were compared to deflection predictions from an elastic analysis procedure done for Wall 2 and Wall 3, which have an identical solid cross section. The procedure for calculation of uncracked and cracked transformed moment of inertias is available in most reinforced concrete design textbooks. Transformed sections were created using the following parameters: $E_s=200$ GPa, $f_y=378$ MPa, $E_m=575$ MPa, $L=1800$ mm, $b=150$ mm. Because of the relatively low modulus of the masonry, the uncracked section moment of inertia was calculated using an uncracked transformed section which accounts for the effect of reinforcing steel on stiffness.

For simplicity, the cracked section moment of inertia was calculated without recognizing the contribution of compressive reinforcement. In addition, for calculation of cracked stiffness, the wall was assumed to be fully cracked for simplicity. Figure 6.4 shows the measured force displacement backbones and equivalent elastoplastic stiffness for each wall test in the pull direction with the calculated uncracked and cracked stiffness predictions. In Figure 6.4(d), comparison of the calculated cracked stiffness to the force displacement envelope for Wall 3(b) shows that this method over predicts the stiffness of the wall, which is most likely due to extensive stiffness degradation as a result of previous testing.

Figure 6.4: Transformed elastic stiffness predictions

6.4.2 Measured Wall Displacement Components

Measured displacement data for each wall was reduced to rocking, sliding, shear and bending deformation components following the procedures modified from Voon (2007), which are discussed in Section 4.4 and Appendix A. Displacement component plots corresponding to the force displacement envelope are shown in Figure 6.5.

Although the layout of wall instrumentation was generally the same for all walls, minor changes were made as necessary to improve the accuracy of the measurements; Table 6.4 lists the measurements used for calculation of each displacement component plot in

Figure 6.5 (see Figure 4.11 for instrumentation layout). In order to salvage the data from testing errors, the reference “tip” displacement was measured from either the upper LVDT (Wall 2 and 3) or upper string potentiometer as necessary (Wall 1). Appendix E presents decoupled force-displacement envelope plots.

Table 6.4: Dimensions for displacement component calculations

Wall	h (mm)	d_u (mm)	L (mm)	d (mm)
1	1700	150	1650	2369
2	1700	150	1600	2335
3(a)	1700	150	1600	2335
3(b)	900	1050	1600	1836

Figure 6.5: Measured displacement components

As shown in Figure 6.5, The displacement component calculations for Wall 2 and Wall 3(a) provided the best approximation of total displacement. The displacement measurements for Wall 1 were influenced by the local spalling failure which permanently offset the upper measuring point. The best method for measuring flexural curvature is by arranging multiple vertical transducers with shorter gage lengths at the ends of the wall, but limited availability of transducers resulted in the simple arrangement which was utilized. It is recognized that the large gage length of the vertical displacement transducers used for this experiment will probably not capture the true curvature distribution of the wall specimens, affecting the reliability of the presented displacement component plots. This is reflected in Figure 6.5 (c) and (d) for the flexurally dominated walls. However, the displacement component plot behavior for Wall 2 is consistent with observed specimen behavior: As the testing progressed towards larger imposed displacements, shear strength and stiffness degradation a resulted in increased shear sliding between block courses and widening of existing shear cracks. Figure 6.5 (b) shows the reduction in bending displacement contribution at larger displacements. For Wall 2, this behavior was most likely captured because shear failure occurred quickly before flexural curvature could increase due to non-linear bending response.

6.5 Static Non-Linear Analysis

Consistent with conventional reinforced concrete or masonry structures, the force displacement response of ICEB shear walls was observed to be non linear with very little linear elastic behavior. In order to check the validity of the measured prism compressive testing data, a static non linear moment curvature analysis was performed.

6.5.1 Moment-Curvature Analysis

A discretized layer or “fibre” model was created in Excel to represent the cross section of a fully grouted ICEB wall with dimensions identical to the tested specimens. Non linear material constitutive properties were assumed for the wall materials based on data from available testing and common values. The wall section was divided into 60 layers which were 30 mm thick by 150 mm wide. The ICEB masonry was modeled using a modified Hognestad stress strain relationship as discussed in Section 3.3.3. Typically, for static nonlinear analysis, masonry (or concrete) tensile capacity is conservatively assumed to be zero. In the interest of comparison to experimental data, the masonry tensile strength was assumed to be 0.4 MPa. This tensile strength value was based on best fit of experimental results, and would better be determined by indirect tensile testing. The reinforcing steel was modeled using an elastic-perfectly plastic stress strain curve which ignored the effects of strain hardening. #3 reinforcing bars ($f_y=378$ MPa) with area $A_s=71.0\text{mm}^2$ (0.11 in^2) or #4 bars ($f_y=565$ MPa) with area $A_s=123\text{ mm}^2$ (0.2 in^2) were distributed at spacing identical to that from the ICEB wall specimens (75, 675, 1125 and 1725 mm from extreme compression end).

A non linear moment curvature analysis was performed by systematically assuming a curvature (starting from zero) and solving for the moment capacity. For an assumed extreme compressive strain, a linear strain distribution was assigned to the section and material stresses were calculated based on the assumed material models. Once vertical equilibrium of forces was achieved by iteration of neutral axis depth, section curvature (κ) and the associated moment (M) were calculated and plotted as a point on the moment-curvature plot. In the interest of comparison to experimental results, a limiting

compressive strain was not set for the moment-curvature relationship. Figure 6.6 shows moment-curvature plots for two different wall sections; partially grouted with #4 longitudinal bars (Wall 1) and fully grouted with #3 longitudinal bars (Wall 2, Wall 3).

Figure 6.6: Moment-Curvature plot for ICEB wall specimen

6.5.2 Inelastic Static Pushover

For walls with the section moment curvature relationships shown in Figure 6.6, a distributed plasticity “static pushover” analysis was performed, as further described in Chen and Scawthorn (2003). As shown in Figure 6.7, the wall height was divided into 18 sections, each of which were subjected to a constant axial load comprised of the self weight or superimposed vertical axial load above. For increasing values of applied lateral load (F_v), the moment demand at each layer was calculated as:

$$M_i = F_v (H_e - H_i) \quad (5-4)$$

Curvature at each layer was computed from the moment-curvature relationship, and a corresponding rotation (θ_i) was calculated:

$$\theta_i = \kappa_i h_i \quad (5-5)$$

The total lateral bending deformation due to the static lateral force was calculated as:

$$\delta_{bn} = \sum_{i=1}^n \theta_i h_i \quad (5-6)$$

Total shear deformations were assumed to be elastic, and were calculated as:

$$\delta_v = \frac{1.2 F_v}{E_v A_n} \quad (5-7)$$

Figure 6.7: Static pushover analysis dimensions

Figure 6.8 shows the predicted static pushover curve for Wall 1 with the experimentally observed force displacement envelope in the pull (+) direction. Similarly, Figure 6.9 shows the pushover curve for the wall sections next to the experimentally observed force displacement envelopes for Wall 2, Wall 3 (monotonic) and Wall 3

(cyclic). For both figures, the nominal flexural strength (F_n) calculated by using the method from Section 2.2.2.1 is plotted as a horizontal line.

Initially, the predicted static pushover curve for Wall 1 is close to the experimentally observed wall displacement, but diverges at higher levels of lateral load. This is likely due to significant shear stiffness degradation as a result of the partial grouting. The results of the non linear analysis plotted in Figure 6.9 are very close to the experimentally observed wall panel lateral force displacement response for Wall 2 and Wall 3. This validates the use of the use of the stacked masonry prism compressive stress-strain data for the fully grouted wall. It is recognized that this simple model does not include more complex material behavior such as non-linear shear deformations or failure, masonry spalling, steel strain hardening or compressive buckling. However, the intent of this analysis was to match theoretical predictions to experimental behavior; it should be noted that this model can be applied to reasonably predict specimen behavior up to the limits of flexural tensile yielding for future ICEB shear wall specimens.

Figure 6.8: Non linear static pushover (Wall 1)

Figure 6.9: Non linear static pushover (Wall 2, Wall 3)

Chapter 7. Conclusions and Recommendations

This project was intended to provide valuable insight into the structural performance of ICEB shear wall systems subjected to cyclic in-plane loading, and also provide a basis for future research of this technology. It was shown that reinforced ICEB shear walls behave similarly to conventional reinforced CMU shear walls, and that some of the theory that is used for design conventional reinforced masonry or concrete can be applied to ICEB structures. However, experimental results also indicated some significant differences between the expected and actual shear capacity. Further research and testing can provide a strong foundation for the use of modern performance based engineering methods for the design of ICEB structures.

7.1 Conclusions

7.1.1 ICEB Materials and Testing Conclusions:

- Grout samples were subjected to compressive testing to determine the effect of the rapid rate at which water is absorbed from the grout by the soil blocks, and compressive strength of grout cast into the wall material was measured to be

approximately 80% higher than grout cast into non porous cylinder molds. This behavior should be noted when taking grout quality control samples

- The common failure mode for single ICEBs loaded in compression was a conical break where the sides fell off, which is consistent with previous material testing of ICEBs.
- Fully and partially grouted stacked masonry prisms were tested in compression to determine the masonry compressive strength. Measured behavior indicates a very ductile stress strain response, with very little linear elastic behavior.
- The compressive strength ratio of prism to individual block was measured to be 0.44 and 0.55 for partially and fully grouted ICEB prisms, respectively. This strength reduction due mainly to aspect ratio is consistent with previous research.
- From experimental testing of grouted ICEB prisms, the average modulus of elasticity was measured to be 575 MPa, which is significantly lower than the modulus predicted by common methods based on masonry compressive strength. It is likely that the dry-stacked mechanism influences this behavior.

7.1.2 ICEB Shear Wall Testing Conclusions

7.1.2.1 Shear Performance

- The hysteretic behavior of shear dominated ICEB wall panels is similar to that of shear dominated CMU wall panels, characterized by in-cycle strength and stiffness degradation in addition to brittle post peak strength loss
- Observed wall shear damage patterns indicate that much of the shear cracking occurs in the grouted core of the wall, and that sliding between block courses and

widening of head joints contributes significantly to overall displacements, but is not visible once the wall has been returned to “zero” displacement position.

- The ACI 530-08 method for predicting in-plane masonry shear strength significantly overestimates the capacity of ICEB shear walls. It is suggested that the shear resistance of ICEB masonry does not increase linearly with the net area, and that calculations which include a correction factor for the “effective shear area” can provide conservative estimates.
- The shear strength of fully grouted ICEB walls is significantly higher than that of partially grouted walls. This also suggests that ICEB shear strength is not directly proportional to net area, and is attributed to more continuous distribution of grouting, and better “shear continuity”.
- Calculations of ICEB masonry shear strength which ignore the compressed earth material and only consider the contribution of the continuous vertical grouting provide a very conservative prediction of capacity.
- The contribution of transverse reinforcing steel to shear strength was not properly determined from testing. The local shear type failure which occurred at the top of Wall 3 under monotonic loading was determined to be due to the reinforcement detail at the upper bond beam.

7.1.2.2 Flexural Performance

- Flexural failure of the ICEB shear wall specimen was characterized by tensile yielding of the longitudinal reinforcing steel. The observed hysteretic behavior was characteristic of a ductile tensile yielding failure with stiffness degradation, but no strength loss until compressive rebar buckling.

- Simple flexural theory, based on the assumptions that plane sections remain plane and an equivalent rectangular masonry stress block, provides a fairly close prediction of the flexural capacity of ICEB shear walls based on tensile yielding of longitudinal reinforcement.

7.1.2.3 Wall Stiffness and Displacement

- The displacement characteristics of ICEB shear walls under cyclic lateral in-plane loading are consistent with the mechanisms which resist lateral forces. The combination of the interlocking mechanism and lack of mortar leads to large sliding deformations once shear cracking or stiffness degradation occurs.
- Calculation of the stiffness of an ICEB shear wall panel using un-cracked transformed section properties underpredicts the stiffness. This could indicate that the measured masonry elastic modulus (E_m) is not accurate.
- Displacements were decoupled into shear, bending and rocking components. For flexure dominated walls, the method used underpredicts the bending displacement because the assumed distribution of curvature along the height of the wall is not accurate. For accurate evaluation of section curvature, additional instruments are necessary. However, for shear dominated panels the analysis method properly represents the observed behavior.

7.1.2.4 Static Non-Linear Analysis

- Results from a static analysis using non linear material properties for the ICEB masonry and steel reinforcing are reasonably close to the experimentally observed force displacement response of ICEB shear wall specimens. The masonry tensile stress strain behavior was assumed based on best fit of force displacement data.

This suggests that use of measured masonry prism compressive stress-strain response for bending analysis is validated.

7.1.3 ICEB Wall Construction

- The dry stacked interlocking mechanism of the ICEB system provides for simplified construction, but finished block dimensions vary as a result of the manufacturing process. This dimensional variance will negatively affect the constructability of the system and may result in uneven and inefficient load distribution along the wall. Full grouting of all cavities will improve the load carrying capabilities and reduce occurrence of cracking under service loads.
- Grout must be very fluid, but must not contain excessive water content which will result in bleeding of cement or other fines. For this experiment, lime was added to the grout mixture to reduce excessive bleeding without significantly increasing the strength of the grout. Other materials such as fly or rice husk ash may serve the same purpose.
- Although the dimensions of the ICEB channel blocks require tighter bending dimensions than allowed by many building codes, transverse reinforcement should be hook 180 degrees around the extreme longitudinal reinforcement. This construction detail should allow for better shear resistance by the horizontal reinforcement.
- Reinforced concrete bond beams installed at the top of walls will provide more space for proper reinforcement detail and installation, but will require use of external formwork.

7.2 Recommendations for Future Work

- Further material testing should be done to better determine ICEB masonry compressive stress-strain behavior at higher aspect ratios with varying grout strengths for partially and fully grouted walls. The differences between compressed earth and grout material properties probably contribute to the observed failure modes.
- Testing aimed at better quantification of ICEB shear resisting mechanisms needs to be pursued. This thesis suggests that the in-plane shear capacity of ICEB shear walls is proportional to an “effective shear area”, but further testing is necessary to better understand the individual contributions of grout shear strength, interlocking action, and sliding friction.
- The contribution of transverse reinforcing to in-plane shear strength needs to be evaluated by further testing of ICEB shear walls with varying horizontal reinforcing steel ratios. Experimental testing with a goal to observe “ductile” shear failure would provide valuable insight into the performance of shear dominated ICEB shear walls.
- Performance based seismic design of reinforced masonry shear walls depends on shear capacity design to guarantee a ductile flexural failure. The ductility capacity of flexurally dominated ICEB shear walls should be evaluated by further testing. Research should be also be done to identify a better experimental loading protocol for use with ductile specimens.
- The displacements from the transducer built into the actuator were up to 25% higher than the displacements measured by an LVDT at the same elevation. This

error is attributed to play within the actuator and its mounting system. Before further testing, a new upper LVDT mounting point for the loading beam should be fabricated which can be left in place even while all other instruments are removed from the wall.

References

-
- Anand, K.B., and Ramamurthy, K. (2005). "Development and Evaluation of Hollow Concrete Interlocking Block Masonry System." *The Masonry Society Journal*, 23(1), 11-19.
- Bales, C., Donahue, C.; Fisher, M, Mellbom, A., and Pearson, T. (2009). "Interlocking Compressed Earth Blocks: From Soil to Structure." Senior Project, Cal Poly San Luis Obispo, California.
- Bei, G., and Papayianni, I., (2003). "Compressive Strength of Compressed Earth Block Masonry." *Structural Studies, Repairs and Maintenance of Heritage Architecture VIII*, 15, 367-375.
- Chen, W., and Scawthron, C. (2003). *Earthquake Engineering Handbook*. Boca Raton, FL: CRC Press.
- Heathcote, K. (1991). "Compressive Strength of Cement Stabilized Pressed Earth Blocks." *Building Research and Information*, 19(2), 101-105.
- Jaafar, M.S., Thanoon, W.A., Najm, A.M.S., Abdulkadir, M.R., Ali, A.A.A. (2006). "Strength Correlation Between Individual Block, Prism and Basic Wall Panel for Load Bearing Interlocking Mortarless Hollow Block Masonry." *Construction and Building Materials*, 20, 492-498.
- MacGregor, J.G., and Wight, J.K. (2005). *Reinforced Concrete: Mechanics and Design*. 4th ed. Upper Saddle River, NJ: Prentice Hall.
- Maini, S. (2010a). "Earthen Architecture in the World," *Auroville Earth Institute* Retrieved from http://www.earth-auroville.com/?nav=menu&pg=earthworld&id1=26&lang_code=en Date accessed: 8/25/2010.

- Maini, S. (2010b). "Earthen Architecture in the World," *Auroville Earth Institute*
Retrieved from http://www.earth-auroville.com/?nav=menu&pg=technologies&id1=33&lang_code=en Date accessed: 8/25/2010.
- The Masonry Society (TMS) (2007). *Masonry Designers' Guide*. 4th ed. Boulder, CO.
- Masonry Standards Joint Committee (MSJC) (2008). "Building Code Requirements for Masonry Structures," *TMS 402-08/ACI 530-08/ASCE 5-08*, USA
- Pave, R.O., (2007). "Strength Evaluation of Dry-Stack Masonry." M.S. Thesis. University of the Witwatersrand, Johannesburg, South Africa. Retrieved from <http://wiredspace.wits.ac.za/handle/10539/5691> Date accessed: 1/25/2011.
- Perera, A., and Jayasinghe, C. (2003). "Strength Characteristics and Structural Design Methods for Compressed Earth Block Walls." *Masonry International*, 16(1), 34-38.
- Proto, C., Sanchez, D., Rowley, K., Thompson, R. (2010). "ICEB: Design and Construction Manual." Senior Project, Cal Poly San Luis Obispo, California.
- Shedid, M.T., El-Dakhkhni, W.W., Drysdale, R.G. (2009). "Behavior of fully grouted reinforced concrete masonry shear walls failing in flexure: Analysis." *Engineering Structures*, 31, 2032-2044.
- Shildkamp, M. (2009). *Suitability of Local Soil for Cost Saving Construction Techniques V.2*. Smart Shelter Foundation. Retrieved from <http://cvbt-web.org/uploads/Pubs/Suitability%20of%20Local%20Soil%20for%20Cost%20Saving%20Construction%20Techniques.pdf> Date accessed: 10/20/2010.
- Shing, P.B., Schuller, M., Hoskere, V.S. (1990). "In-Plane Resistance of Reinforced Masonry Shear Walls." *Journal of Structural Engineering*, 116(3), 619-640.
- Sucuoglu, H., McNiven, H.D. (1991). "Seismic Shear Capacity of Reinforced Masonry Piers." *Journal of Structural Engineering*, 117(7), 2166-2186.
- Voon, K.C. (2007). "In-plane Seismic Design of Concrete Masonry Structures." Ph.D. Thesis. University of Auckland: NZ. Retrieved from <http://hdl.handle.net/2292/580>
- Voon, K.C., and Ingham, J.M. (2006). "Experimental In-Plane Shear Strength Investigation of Reinforced Concrete Masonry Walls." *Journal of Structural Engineering (ASCE)*, 123(3), 400-408.
- Walker, P. (1999). "Bond Characteristics of Earth Block Masonry." *Journal of Materials in Civil Engineering*, 11(3), 249-256.

Wheeler, G. (2005). *Interlocking Compressed Earth Blocks Volume II. Manual of Construction*. Center for Vocational Building Technology, Thailand.

Appendix A. Shear and Flexural Displacement Components

The panel deformation components were separated using a simplified method originally developed by Voon (2007) after Brammer (1995) and Hirashi (1984). Data was collected during testing with the typical setup as shown in Figure A.1. This appendix describes the calculation of shear and flexural displacement components, following the procedure presented in Voon (2007).

Figure A.1: Wall panel section

Figure A.2 shows the considered deformation of a panel section. It is assumed that the two upper points, A and B, may transmit horizontally by u_l and u_r , and vertically by the amounts v_l and v_r . The lower points, C and D, are assumed to remain fixed in position because of the support condition at the foundation. The subscripts 'l' and 'r' refer to left and right sides. Extensional deformation components in the vertical and horizontal direction due to axial load are considered to be negligible for this situation. The sign

convention is assumed positive for displacements to the right and upward. From Figure A.1, δ_{d1} and δ_{d2} are the elongations of the diagonal transducers, while δ_{v1} and δ_{v2} are the elongations of the vertical transducers. The wall dimensions are defined by length, L , height, h , and diagonal length, d . The dimension from the top of the wall panel section to the top of the wall is given as d_u .

Figure A.2: Nodal displacement of a panel section

As shown in Figure A.3, the wall panel section deformation is assumed to consist of two components: shear and flexure. These deformations are represented by the coordinates u_l , u_r , v_l and v_r , where u and v represent the horizontal and diagonal deformation components, respectively. The subscripts “s” and “b” represent shear and flexural deformation components, respectively.

Figure A.3: Components of panel deformation

The primary purpose of the following derivation is to calculate the horizontal displacement at the top of the wall due to shear deformation, U_s , by relating the measured elongation (δ 's) to individual displacement components: u 's and v 's. The following assumptions are used:

1. The left and right horizontal shear deformation components are equal.
2. The left and right horizontal flexural deformation components are equal.

3. The vertical shear deformation components are zero.

The above assumptions are represented as follows:

$$\begin{aligned} \text{a)} \quad & u_{ls} = u_{rs} = u_s \\ \text{b)} \quad & u_{lb} = u_{rb} = u_b \end{aligned} \quad (\text{A-1})$$

The relationships between these displacements and those shown in Figure A.3 are as follows:

$$\begin{aligned} \text{a)} \quad & u_l = u_s + u_b \\ \text{b)} \quad & u_r = u_s + u_b \\ \text{c)} \quad & v_l = v_{lb} \\ \text{d)} \quad & v_r = v_{rb} \end{aligned} \quad (\text{A-3})$$

The measured relative deformations can be expressed in terms of global deformations by the following geometric relationships:

$$\begin{aligned} \text{a)} \quad & \delta_{d1} = \frac{L}{d}(-u_l) + \frac{h}{d}(v_l) \\ \text{b)} \quad & \delta_{d2} = \frac{L}{d}(u_r) + \frac{h}{d}(v_r) \\ \text{c)} \quad & \delta_{vl} = v_l \\ \text{d)} \quad & \delta_{vr} = v_r \end{aligned} \quad (\text{A-3})$$

Substituting Equation A-2 into A-3:

$$\begin{aligned} \text{a)} \quad & \delta_{d1} = \frac{L}{d}(-u_s - u_b) + \frac{h}{d}(v_{lb}) \\ \text{b)} \quad & \delta_{d2} = \frac{L}{d}(u_s + u_b) + \frac{h}{d}(v_{rb}) \end{aligned}$$

$$\begin{aligned}
 \text{c)} \quad \delta_{v1} &= v_{lb} \\
 \text{d)} \quad \delta_{vr} &= v_{rb}
 \end{aligned}
 \tag{A-4}$$

Inserting Equations A-4(c) and (d) into Equations A-4(a) and (b), and then subtracting A-4(a) from (b) gives:

$$\delta_{d2} - \delta_{d1} = \frac{L}{d}(2u_s + 2u_b) + \frac{h}{d}(\delta_{v2} - \delta_{v1})
 \tag{A-5}$$

Rearranging Equation A-5:

$$u_s = \frac{d}{2L}(\delta_{d2} - \delta_{d1}) + \frac{h}{2L}(\delta_{v1} - \delta_{v2}) - u_b
 \tag{A-6}$$

Equation A-6 can be solved by defining an equation relating the flexural deformation component to the measured relative displacement:

$$u_b = \theta h \alpha
 \tag{A-7}$$

where:

$$\theta = \frac{\delta_{v1} - \delta_{v2}}{L}$$

Equation A-7 states that the flexural deformation is equal to the rotation at the top of the panel section multiplied by the panel section height and by α . When taking α as 2/3, the equation represents the exact flexural displacement of an elastic prismatic cantilever with

a concentrated horizontal force applied at the top, with θ representing the rotation at the top of the wall. For reinforced concrete masonry and concrete walls, α is generally higher than $2/3$ since the wall flexural cracking tends to concentrate rotation towards the bottom of the wall, therefore resulting in higher $h\alpha$ and higher u_b .

The flexural deflection u_b for a section of wall was calculated rotation that occurs within the section. This rotation is calculated from the bending moment diagram. The bending moment at the top (M_{up}) and the bottom (M_{lw}) of a panel section is known to vary linearly according to the vertical location as shown in Figure A-1(b).

The moment (M)-curvature (ϕ) relationship for an elastic section is given by:

$$M = \phi EI \quad (A-8)$$

where E and I are the modulus of elasticity and moment of inertia. As the curvature is a linear function of the moment, the total rotation of the panel section between d_u and d_u+h can be calculated from the average bending moment:

$$\theta = \frac{h(M_{up} + M_{lw})}{2EI} \quad (A-9)$$

The panel flexural deformation, u_b , is evaluated by integration of curvature along the height of the panel section with the following result:

$$u_b = \frac{h^2}{EI} \left(\frac{M_{lw}}{3} + \frac{M_{up}}{6} \right) = \theta h \left(\frac{d_u + \frac{2h}{3}}{2d_u + h} \right) \quad (\text{A-10})$$

$$\text{where } \alpha = \left(\frac{d_u + \frac{2h}{3}}{2d_u + h} \right)$$

The α in Equation A-10 is defined with respect to the top of the investigated panel section.

u_b can be evaluated by incorporating Equation A-7:

$$u_b = \frac{h(\delta_{v1} - \delta_{v2})}{L} \left(\frac{d_u + \frac{2h}{3}}{2d_u + h} \right) \quad (\text{A-11})$$

Subsequently, the shear deformation for the panel section can be evaluated by substituting Equation A-11 into Equation A-6:

$$u_s = \frac{d}{2L} (\delta_{d2} - \delta_{d1}) + \frac{h}{2L} (\delta_{v1} - \delta_{v2}) - \frac{h(\delta_{v1} - \delta_{v2})}{L} \left(\frac{d_u + \frac{2h}{3}}{2d_u + h} \right) \quad (\text{A-12})$$

Rearranging Equation A-12 gives:

$$u_s = \frac{d}{2L}(\delta_{d2} - \delta_{d1}) - \frac{h^2}{6(2d_u + h)} \frac{(\delta_{v1} - \delta_{v2})}{L} \quad (\text{A-13})$$

In addition, the total flexural displacement can be evaluated as follows. The deformation at the top of the wall, u'_b , due to flexural deformation of the investigated panel section is evaluated as:

$$u'_b = \theta(\alpha h + d_u) = \frac{(\delta_{v1} - \delta_{v2})}{L} \left(h \frac{d_u + \frac{2h}{3}}{2d_u + h} + d_u \right) \quad (\text{A-14})$$

Appendix B. Correction of Lateral Displacement Data

This section includes a description of the process used to modify the force displacement hysteresis data to account for the discrepancy between the displacements measured by the Actuator displacement transducer and the upper LVDT measuring device, which should read the same measurement. Because the LVDT instrument was most recently calibrated, it is assumed to offer superior measurement accuracy and precision. However, the transducers were removed mid-testing to avoid any damage, and only partial data was recorded by the LVDTs. Consequently, the measured actuator displacement data is modified to correspond with the last measured LVDT data for each test.

From examination of the data, it was determined that the error in measurement was different for pull (+) and push (-) displacements. Figure B.1 shows a plot of the difference in actuator and LVDT displacement ($d_{ACT} - d_{LVDT}$) versus the total measured actuator displacement, d_{ACT} , for all wall testing results. The plot suggests that there are upper and lower limits to the measurement differences, which is indicative of slop or play within the actuator and/or its mounting points. From Figure B.1, the upper limits to the “slop” are approximately 2.5 mm and 0.9 mm in the pull (+) and push (-) directions, respectively. All lateral displacement data at the actuator level was modified to more closely represent the actual response of the walls. Figure B.2 through Figure B.5 are plots which show the original and modified force-displacement data. Generally, the initial LVDT displacement data is unmodified but data at further displacements is modified according to the following method:

1. Initial displacement data recorded from the LVDTs is used until the point at which the instruments were removed from the wall. This occurred at different drift levels for separate walls.
2. Actuator displacement data from after removal of the instruments was modified by a scalar (0.8 and 0.9 for pull (+) and push (-), respectively) to smoothly match the transition from pull to push “slop”. The modified displacements were limited to be within 2.5 and 0.9 mm of the measured actuator displacement for pull and push directions, respectively. The modified data can be identified in Figure B.1 through Figure B.5 as that fitting under the dashed “modified envelope” portion of the curves.

Figure B.1: Actuator to LVDT displacement “gap” vs actuator displacement

Figure B.2: Corrected Wall 1 force-displacement response

Figure B.3: Corrected Wall 2 force-displacement response

Figure B.4: Corrected Wall 3(a) force-displacement response

Figure B.5: Corrected Wall 3(b) force-displacement response

Appendix C. Material Testing Data

This appendix presents the measured compressive strength data from experimental testing. Table C.1 presents the data from testing of partially and fully grouted prisms, where samples were subjected to load or displacement controlled compression. Prisms were tested at a load controlled rate of (2.0 MPa/min), or displacement controlled rate of (0.45 mm/min). Loading type is denoted as σ or ϵ for load or displacement control, respectively. To provide a better representation of the material properties, tested material strengths are reported from additional walls which are part of a later project. Table C.2 includes compressive strengths of porous and non-porous grout samples, and Table C.3 includes measured compressive strengths of individual ICEBs.

Table C.1: ICEB prism compressive strength

					Compressive Strength (MPa)		
Wall	Grouting	Loading	Area (mm ²)		Mean	St Dev	COV
1	Partial	σ	41819	2.88	3.31	0.62	19%
1	Partial	σ	41819	3.75			
2	Partial	ϵ	41819	2.12	2.06	0.10	5%
6	Partial	ϵ	41819	2.14			
4	Partial	ϵ	41819	2.05			
6	Partial	ϵ	41819	1.91			
1	Full	σ	45000	4.57	4.16	0.38	9%
1	Full	σ	45000	3.67			
2	Full	σ	45000	4.31			
2	Full	σ	45000	4.09			
3	Full	ϵ	45000	2.88	3.00	0.21	7%
3	Full	ϵ	45000	3.19			
4	Full	ϵ	45000	2.77			
5	Full	ϵ	45000	3.16			
6	Full	ϵ	45000	2.37			

Table C.2: Grout compressive strength

				Compressive Strength (MPa)			
Wall	Sample	Porous or Non-Porous	Area (mm ²)		Mean	Std. Dev	COV
1	cube	NP	2581	4.61	4.94	0.39	0.080
1	cube	NP	2581	4.39			
1	cube	NP	2581	5.56			
1	cube	NP	2581	4.67			
2	cylinder	NP	2027	5.16			
2	cylinder	NP	2027	5.33			
2	cylinder	NP	2027	4.89			
2	cylinder	NP	2027	4.90			
1	cylinder	P	1590	6.85	9.17	1.88	0.205
1	cylinder	P	1590	9.38			
2	cylinder	P	1590	7.14			
2	cylinder	P	1590	11.58			
2	cylinder	P	1590	10.68			
2	cylinder	P	1590	9.39			

Table C.3: ICEB compressive strength

		Compressive Strength (MPa)			
Wall	Area (mm ²)		Average	Std. Dev	COV
1	39319	7.64	7.57	0.66	0.088
1	39319	7.42			
2	39319	6.96			
2	39319	7.80			
-	39319	7.39			
-	39319	7.36			
-	39319	9.03			
-	39319	6.93			

Table C.4 and Figure C.1 shows the results of rebar tensile testing. Only yield and ultimate strength values were recorded for sample 3A.

Table C.4: Rebar tensile test data

Sample	Specification	Diameter (mm)	<u>Measured Tensile Strength</u>	
			f_y (MPa)	f_u (MPa)
3A	#3 Gr. 40	10	407.4	574.3
3B	#3 Gr. 40	10	348.9	507.7
4A	#4 Gr. 60	13	479.6	1051.6
4B	#4 Gr. 60	13	651.3	1315.1

Figure C.1: Steel rebar tensile test data

Appendix D. Instrumentation Layout

Wall 1 was instrumented as shown in Figure D.1. During testing, the upper block which was connected to DTR0 and 2" LVDT-A spalled off, which effected the measured displacements. LVDT7 did not measure data during the test.

Figure D.1: Instrumentation layout [Wall 1]

Wall 2 and Wall 3(a) were instrumented as shown in Figure D.2. The 2" LVDT-A was moved to measure off of the loading beam, and rocking was measured with 0.5" LVDTs.

Figure D.2: Instrumentation layout[Wall 2 and Wall 3(a)]

After repair, Wall 3(b) was instrumented as shown in Figure D.3.

Figure D.3: Instrumentation layout [Wall 3(b)]

Appendix E. Wall Displacement Components

This section provides force-displacement plots with decoupled shear and bending components. The procedure used to decouple the components is presented in Appendix A. Figure E.1 through Figure E.4 show the force-displacement plots for W1, W2, W3(a), and W3(b).

Figure E.1: Decoupled force-displacement envelope [Wall 1]

Figure E.2: Decoupled force-displacement envelope [Wall 2]

Figure E.3: Decoupled force-displacement envelope [Wall 3(a)]

Figure E.4: Decoupled force-displacement envelope [Wall 3(b)]

Appendix F. Wall 2

This appendix reports the specific failure mode for Wall 2, which experienced sudden diagonal shear cracks initiated on the $\pm 8\text{mm}$ (actuator displacement) cycles. Figure F.1 indicates the individual failure points for the force-displacement history for Wall 2. Figure F.2 and Figure F.3 show the initial shear crack formation for the pull and push directions.

Figure F.1: Wall 2 force-displacement history

Figure F.2: Initial shear crack [W2-1st 8mm PULL cycle @ +43.0 kN, +5.25 mm]

Figure F.3: Initial shear crack [W2-1st 8mm PUSH cycle @ -39.5 kN, -5.5 mm]