INVESTIGATION OF OUT-OF-PLANE PROPERTIES
OF INTERLOCKING COMPRESSED
EARTH BLOCK WALLS

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ABSTRACT

Out-Of-Plane Properties Of Interlocking Compressed Earth Block Walls

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Interlocking compressed earth blocks (ICEBs) are cement stabilized soil blocks that allow for dry stacked construction. The incomplete understanding of the inelastic performance of ICEB building systems limits widespread acceptance of this structural system in earthquake prone areas. This thesis presents results from an experimental program designed to explore the behavior of ICEB walls, built according to current design practice in Indonesia and Thailand, and subjected to out-of-plane loading. A total of five reinforced and grouted ICEB walls were constructed and tested.

Results from experimentation show the current masonry design code, ACI 530, adequately predicts the yield strength of these walls. However, ACI 530 grossly over-predicts the ICEB wall stiffness. All tests showed flexural behavior and failure, except for one wall. A brittle failure was observed in one wall before reaching the predicted flexural strength, prompting a suggested maximum shear tie spacing. The testing results provide useful data for developing analytical models that predicts the seismic behavior of ICEB walls under out-of-plane loading.

A moment-curvature relationship was developed that accurately predicts the behavior of these walls in the elastic range as well as the inelastic range. By comparing the data provided by two walls of similar sizes, one including a pilaster and one without a pilaster, insight into stiffener elements was gained. Analysis of these two walls provides a limit on the length and height of ICEB walls without stiffener elements to prevent significant structural damage during a seismic event. In all, conclusions based on experimental data from ICEB out-of-plane loading tests are aimed to provide suggestions for ICEB construction in areas of high-seismicity.

Keywords: Out-of-Plane, Interlocking Masonry, Compressed Earth Block, Dry-Stack

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TABLE OF CONTENTS

LIST OF FIGURES	ix
LIST OF TABLES	xi
LIST OF NOMENCLATURE	xii
1.0 Introduction	1
1.1 Background	2
1.2 Objective	4
1.3 Organization	5
2.0 Literature Review	7
2.1 Compressed Earth Blocks: Rhino Blocks	7
2.2 Masonry Wall Design	12
2.3 Out-of-Plane Wall Testing	14
3.0 Materials	22
3.1 Interlocking Compressed Earth Blocks	22
3.1.1 Soil	22
3.1.2 Sand	23
3.1.3 Cement	23
3.1.4 Material Preparation and Mixing	23
3.1.5 Pressing Procedure	26
3.1.6 Curing	28
3.2 Grout	29
3.3 Soil-Cement Plaster	29
3.3 Material Testing and Results	30
3.3.1 ICEB Compressive Strength	31
3.3.2 Grout Compressive Strength	32
3.3.3 ICEB Masonry Prism Compressive Strength	33
3.3.4 Soil-Cement Compressive Strength	36
3.3.5 Steel Reinforcement Tensile Testing	37
4.0 Test Setup and Procedure	38
4.1 ICEB Construction Methods	38

4.1.1 Laying ICEBs	39
4.1.2 Grouting	40
4.2 Cantilever Walls	40
4.2.1 Wall Layout and Construction	41
4.2.2 Loading Method and Instrumentation	43
4.2.3 Data Reduction Measures.....	46
4.3 Full Scale Walls.....	46
4.3.1 Wall Layout and Construction	47
4.3.2 Loading Method and Instrumentation	50
4.3.3 Data Reduction Measures.....	52
4.4 Wall Behavior Predictions.....	53
5.0 Experimental Results	58
5.1 Cantilever Walls	58
5.1.1 Wall 1: Basic Wall	59
5.1.2 Wall 2: Lap-Splice Wall.....	64
5.1.3 Wall 3: Plastered Wall.....	66
5.1.4 Comparison	71
5.2 Full Scale Walls.....	75
5.2.1 Wall 4: Full Scale Prismatic Wall	75
5.2.2 Wall 5: Full Scale Pilaster Wall	80
5.2.3 Comparison	86
6.0 Moment-Curvature and pilaster Analysis	87
6.1 Moment Curvature Analysis	88
6.1.1 Cantilever Wall Moment-Curvature Results.....	89
6.1.2 Moment-Curvature Relationship Computation	89
6.1.3 Moment-Curvature Spreadsheet Check	91
6.1.4 Calculated ICEB Moment-Curvature Relationship.....	93
6.1.5 Force-Displacement from Moment-Curvature.....	97
6.2 Spacing Determination of Stiffening Elements	101
6.2.1 Out-of-Plane Forces	101
6.2.2 Pilaster Performance	102

6.2.3 Deflection Based Calculation for Pilaster Spacing	106
7.0 Conclusion and Recommendations	112
7.1 Experimental Results.....	114
7.2 Analytical Results.....	116
7.3 Future Research Recommendations	117
References.....	118
Appendix A.....	120
Appendix B	123
Appendix C	128

LIST OF FIGURES

Figure A - ICEB block and variations	8
Figure B - Amrhein and Lee Test Set-up.....	15
Figure C - Moment Diagrams of Walls with Various End Restraints	16
Figure D - Compressed Stabilized Earth Block.....	19
Figure E - Jayasinghe and Mallawaarachchi Testing Directions.....	19
Figure F – Hydraform Blocks	20
Figure G – Example of Cracking Pattern and Loading.....	21
Figure H - Soeng Thai SP3 Pulverizer.....	23
Figure I - Drop Test	25
Figure J - Soeng BP6 with Finished ICEB	27
Figure K - Penetrometer Test.....	28
Figure L - Insufficient Plaster Mixture Results	30
Figure M - ICEB Compressive Failure	32
Figure N - Grout Cylinder Compressive Test.....	33
Figure O - ICEB Prism Compressive Test.....	34
Figure P - ICEB Compressive Stress vs. Strain.....	35
Figure Q - Cantilever Wall Layout	41
Figure R - Cantilever Wall Loading and Instrumentation	44
Figure S - Horizontal LVDTs for Block Rotations (Only One Side Shown)	45
Figure T - Pinned Connections	48
Figure U - Full Scale Wall Plan and Elevations Views	49
Figure V - Typical Pilaster Tie Installation	50
Figure W - Full Scale Wall Loading and Instrumentation.....	51
Figure X - Wall 1 Force Displacement Response and Strain vs. Displacement.....	60
Figure Y - Typical Horizontal Cracking	63
Figure Z - Wall 2 Force Displacement Response	65
Figure AA - Vertical Crack in Wall 2.....	65
Figure BB – Wall 2 Strain vs. Displacement.....	66
Figure CC – Wall 3 Force Displacement Response.....	69
Figure DD - Wall 3 Cracking Pattern	70
Figure EE - Enveloping Comparison of Hysteretic Behavior of Cantilever Walls	71
Figure FF – Experimental Moment-Curvature Envelope Curves.....	73
Figure GG - Wall 4 Force-Displacement Response	76
Figure HH - Wall 4 Rebar Strain vs. Displacement.....	77
Figure II - Vertical Crack in Wall 4.....	79
Figure JJ - Wall 4 Displaced Shape	79
Figure KK - Wall 5 Force Displacement Response.....	82

Figure LL - Wall 5 Cracks and Uplift.....	83
Figure MM - Wall 5 Rebar Strain vs. Displacement	84
Figure NN - Wall 5 Displaced Shape	85
Figure OO - Enveloping Comparison of Hysteretic Behavior of Full Scale Walls.....	86
Figure PP – Strain Profile for Moment Curvature Analysis	90
Figure QQ - Assumed Stress-Strain Plot for Steel Reinforcement.....	94
Figure RR - Moment-Curvature Comparison	95
Figure SS - Cracking Section Comparison	96
Figure TT - Force-Displacement from Moment-Curvature.....	98
Figure UU - Experimental vs. Moment-Curvature Force Displacement Response.....	100
Figure VV - Full Scale Walls with Drift Limit.....	101
Figure WW - Full Scale Walls with Drift Limit	104
Figure XX – Tributary Area of Typical Pilaster	105
Figure YY - Pilaster Tributary Area and Loading	105
Figure ZZ - Allowable Accelerations for Pilasters	106
Figure AAA - Deflected Shape Comparison of Wall Panel and Horizontal Strip.....	108
Figure BBB - Maximum Pilaster Spacing from Displacement Based Analysis.....	111

LIST OF TABLES

Table 1 - Soil Plasticity	22
Table 2 - ICEB Mixture Material Weights	24
Table 3 - ICEB Types and Weights per Block	26
Table 4 - Material Compressive Strengths.....	31
Table 5 - ICEB Wall Specifications.....	38
Table 6 - ACI 530 Predictions	57
Table 7 - Wall 1 & 2 Loading Sequence.....	59
Table 8 – Cantilever Walls ACI 530 vs. Experimental Results.....	62
Table 9 - Wall 3 Loading Sequence.....	67
Table 10 - Wall 4 Loading Sequence.....	75
Table 11 – Wall 4 ACI 530 vs. Experimental Results at Mid-Height	77
Table 12 - Wall 5 Loading Sequence.....	81
Table 13 – Wall 5 ACI 530 vs. Experimental Results at Mid-Height	82
Table 14 – Moment-Curvature Spreadsheet Check	92
Table 15 - Gap Size to Added Curvature.....	97
Table 16 – Finite Element Model Displacement Comparison.....	109

LIST OF NOMENCLATURE

A_s	effective cross-sectional area of reinforcement, mm^2 .
A_v	cross-sectional area of shear reinforcement, mm^2 .
b	width of section, mm.
c	distance from the fiber of maximum compressive strain to the neutral axis, mm.
d	distance from extreme compression fiber to center of tension reinforcement, mm.
d_b	nominal diameter of reinforcement, mm.
d_v	actual depth of a member in direction of shear considered, mm.
E_m	modulus of elasticity of masonry in compression, MPa.
E_s	modulus of elasticity of steel, MPa.
F_p	out-of-plane design force perpendicular to wall face, N/mm.
f'_m	specified compressive strength of masonry, MPa.
f_r	modulus of rupture, MPa.
f_y	specified yield strength of steel for reinforcement, MPa.
h	effective height of wall or pilaster, mm.
I	importance factor for lateral design (equal to 1.0 for this thesis).
I_{cr}	moment of inertia of cracked cross-sectional area of member, mm^4 .
I_g	moment of inertia of gross cross-sectional area of member, mm^4 .
K	dimension used to calculate reinforcement development, mm.
M_{cr}	nominal cracking moment strength, N-m.
M_n	nominal moment strength, N-m.
n	modular ratio, E_s/E_m , dimensionless.
S	spacing of wall stiffener elements, m.
S_{DS}	design spectral response acceleration parameter at short periods, m/s^2 .
S_n	section modulus of the net cross-sectional area of member, mm^3 .

s	spacing of reinforcement, mm.
V_s	nominal shear strength provided by shear reinforcement, N.
W	weight of structural wall, N/mm.
γ	reinforcement size factor.
Δ_{cr}	displacement at cracking moment, mm.
Δ_n	displacement at nominal moment, mm.
ϵ_c	masonry compressive strain, dimensionless.
ϵ_{cu}	ultimate masonry compressive strain, dimensionless.
ϵ_o	masonry compressive strain at the specified strength of masonry, dimensionless.
ρ	density of material, N/m ³

1.0 INTRODUCTION

Interlocking compressed earth block (ICEB) masonry has the potential to provide affordable construction around the world. Comprised of basic, inexpensive materials, such as soil, the blocks can provide homes and other facilities at low cost. By creating interlocking joints between layers of blocks, ICEBs allow for the blocks to be dry stacked, without the need for mortar.

While dry stacked ICEBs are currently being used in structures, little is understood about its behavior during an earthquake. Since there are many different forms of ICEBs, research done for one type is not directly applicable to another. The incomplete understanding of the inelastic performance of ICEB building systems limits the wide spread acceptance of this structural system in earthquake prone areas. The ICEBs used for this thesis are dry stacked and allow for both transverse and longitudinal reinforcement. The ICEBs used for this thesis are currently being used in Indonesia and Thailand, where earthquakes have the potential to cause significant damage.

Out-of-plane forces, created during an earthquake or by wind, can cause significant damage in to a structure. However, no out-of-plane experimental research has been completed on reinforced dry stacked ICEB walls. Therefore, it is the intent of this thesis to provide insight into the out-of-plane behavior of dry stacked, reinforced ICEB walls, constructed according to the current practices found in Indonesia and Thailand.

1.1 Background

Soil has been used as a building material for thousands of years. Adobe, rammed earth, and compressed earth masonry are examples of this building tradition in today's world. Using soil as the main material component of a building provides benefits such as the use of local materials, high thermal mass values, and increased workability (Maini, 2010). The soil used in earth buildings is taken from the surrounding areas or the excavation for the foundations (Maini, 2010). In cement stabilized earth construction, cement use is commonly kept to 5-10% by weight (Walker, 1999). In contrast, concrete and concrete masonry construction use anywhere from 10%-15% by volume of cement (Portland Cement Association, 2012). By eliminating the need for heating kilns and reducing the amount of cement, compressed earth blocks are energy efficient. ICEBs require anywhere from 1/5 to 1/15 of energy to make when compared to fired bricks and concrete masonry units (Maini, 2010). All of these facts about earth construction contribute to a decreased cost of construction and an increased availability in developing countries.

Dry stacked ICEB construction can lead to a faster construction time when compared to other types of masonry. Dry stacking does not rely on skilled labor such as masons. Instead, dry stacking can be done with little training and in a shorter amount of time than with traditional mortared masonry (Maini, 2010). Some researchers suggest the reduced need for skilled labor and the shorter construction time can reduce the cost of labor by as much as 80% (Anand and Ramamurthy 2005).

The materials and the forming method used in dry stacked ICEBs can have a very low carbon footprint when compared to traditional masonry, timber, and concrete. Depending on the location of the ICEB building, the importation of construction products is greatly reduced. Since indigenous soil is the main ingredient in ICEBs, a majority of supplies do not have to be shipped to the site (Maini, 2010). This reduction in transportation decreases the fossil fuel use. ICEB structures also use very little to no timber. Timber structures and the wooden forms used in concrete structures can lead to significant deforestation. Even other types of earth structures such as rammed earth and adobe construction require the use of forms during construction, using lots of wood members that eventually go to waste (Wheeler, 2005). Depending on the surrounding areas of the building, the wood products can end up being transported hundreds of miles to reach the site. Without the need for timber or timber forms, compressed earth block construction helps to limit deforestation around the world (Maini, 2010).

In developed countries, research, design, and construction improvements of masonry buildings have lead to better performance and safety of masonry buildings during an earthquake. However, little structural testing of dry stacked ICEB masonry has been done, leaving the masonry form vulnerable to significant damage or failure during a seismic event. California Polytechnic State University in San Luis Obispo (Cal Poly) has been involved in providing information for the use of ICEB masonry since 2008. The Engineers Without Borders chapter of Cal Poly has been working with the Center for Vocational Building Technology (CVBT) in Thailand on improving construction with

ICEB technology. Students from the mechanical engineering and civil engineering departments have worked with CVBT to improve the ICEB structural building design. Mechanical engineering students have worked with the Soeng Thai BP6 block press, the Soeng Thai SP3 soil pulverizer, and a pocket penetrometer. All of these items are critical to the creation of ICEB's and in making the blocks uniform in strength. Civil engineering students have begun to provide information on the basic properties of ICEB's as well as the in-plane shear wall capacities. It is the combined goal of the student efforts at Cal Poly to provide the research and evaluation of ICEB's as a structural system and offer insight into a better design manual for all future ICEB buildings.

1.2 Objective

The objective of this thesis project is to test ICEB walls in order to investigate the out-of-plane properties, performance, and failure mechanisms, as well as to use the recorded data to accurately predict the performance of each wall.

A total of five walls were tested in under out-of-plane loading. Three, one meter tall cantilever walls were built first. The first cantilever wall was built as a reference to predict the behavior of the full scale walls, as well as provide the basis for comparison of the other two cantilever walls. The second cantilever wall was designed to investigate the influence of a steel reinforcement lap-splice. The final cantilever wall experiment provided data used to determine the change in performance due to a plaster coating. The two full scale walls were constructed to investigate the behavior of ICEB walls loaded out-of-plane. By testing one full scale wall without a pilaster as well as one with a

pilaster, a direct comparison in performance was also formed using the data from the two full scale wall experiments. The hysteretic behavior of each wall was recorded and compared to the ACI 530 code predicted values. The results of this investigation will be used to create a suitable structural design for ICEB structures.

The experimental data from the experimental program also allowed for further analysis of ICEB wall behavior. The goal of the analysis portion of this thesis is to determine the following items:

- An accurate out-of-plane behavior analysis method for ICEB walls
- Maximum wall heights for individual seismic acceleration values
- Maximum pilaster spacing using strength based calculations
- Maximum pilaster spacing using displacement based calculations.

Furthermore, necessary recommendations for construction methods using ICEBs will be made as seen fit through analysis of the experimental results.

1.3 Organization

Chapter 2 offers a literature review of topics relating to Compressed Earth Block construction, testing, and results. Information regarding other types of masonry walls tested for out-of-plane performance and the masonry code is also discussed.

Chapter 3 describes the manufacturing process for ICEBs and the individual components for ICEB walls. Experimental material strength results are presented for each component.

Chapter 4 explains the wall construction and testing setup for each type of wall. The layout and purpose of instrumentation for each wall is discussed in relation to the overall goal of this thesis.

Chapter 5 summarizes the results from the testing. The observed behaviors from all experiments are discussed and compared against code predictions.

Chapter 6 details the process and findings for creating an advanced method for analyzing and designing ICEB walls for out-of-plane loading. The experimental results are also used to develop design requirements for pilaster shear strength and spacing.

Chapter 7 provides a conclusion of all the findings from experimentation and advancement of the analytical process for designing ICEB walls.

2.0 LITERATURE REVIEW

This section discusses relevant literature on the topics of compressed earth blocks, current masonry design guides, and experimentation on out-of-plane properties of walls.

2.1 Compressed Earth Blocks: Rhino Blocks

The Center for Vocational Building Technologies (CVBT) developed a manual of construction for ICEB buildings (Wheeler, 2005). The manual provides insight into how the construction of ICEB buildings is currently being designed. A rhino type block is used by the CVBT due to its ability to interlock and allow for steel reinforcement when necessary. The blocks can be made into nine different shapes, including channel blocks and half blocks. The rhino block dimensions in centimeters, as well as a diagram of each block type can be found in Figure A. Foundations are prescribed for all seismic areas as concrete footings with vertical reinforcement extending from the footing. Rebar splices are formed by one of three methods: a 15 cm weld, bend and hook, or a set of thin wire ties that are tightened around two overlapping bars. All vertical rebar is placed inside the large reinforcement holes in the rhino blocks, and grout is poured in every hole. Wheeler recommends no more than 10 layers of blocks be stacked before a grout pour takes place.

In designing ICEB buildings, it is recommended by Wheeler that no wall span be more than 4.5 meters long without a perpendicular wall, pilaster, or other stiffening member. This recommendation is based on knowledge that long, slender walls are vulnerable to collapse. It is one of the goals this thesis to investigate the span limit recommendation of the CVBT's manual.

Figure A - ICEB block and variations

Source: (Wheeler, 2005)

Bales et al. (2009) experimentally determined the structural properties of compressed earth blocks. The research aimed to create a consistent soil-to-water-to-cement mixing ratio to provide sufficient strength and consistent blocks. Blocks were made with various soil types and were allowed to cure under multiple conditions. Curing methods included submersion in water, under a tarp, or sun dried. Researchers viewed the effects to compressive strength, durability, and compactness.

The authors tested the block compressive strength, as well as the grouted and un-grouted prism strength. These prisms were confined in a wooden form to simulate the effect of confinement the blocks would have in an actual building. The average compressive strength of the fully grouted prisms was found to be 2.58 MPa. The prism compressive strength was found to be 0.43 for grouted prisms and 0.37 for un-grouted prisms times the single block compressive strength to prism compressive strength. The prisms failed by splitting down the grout plane. This failure was attributed to the difference in compressive strength from compressed earth blocks to the grout being used to fill them. Researchers provided detailed instructions for the forming of consistent ICEB's, which were to be used in this thesis.

Proto et al. (2010) wrote a construction manual that describes the manufacturing process for ICEB's using the Rhino blocks made from the Soeng Thai BP6. The Soeng Thai BP6 was used by Bales et al. The goal of the manual was to provide a detailed procedure and guidelines for making rhino blocks. First, the soil selection process was

established for consistent blocks. Next a block making procedure was given, involving six steps:

- Soil Preparation
- Measuring
- Mixing
- Pressing
- Curing
- Testing

The most critical section of Proto et al.'s manual is the description of the water to solids ratio for block making. It was found that the optimal moisture content for ICEB making is 10-14%. The easiest way to determine the moisture content during the block forming process was with a simple drop test. The soil mixture is ready to be pressed once a ball of mixture is dropped one meter and breaks into about 4 – 6 pieces. This drop test is shown in chapter three of this thesis under Figure I. The information presented in the manual will be used in the mixing and pressing of the ICEB's for this thesis.

Bland (2011) constructed walls of dry-stacking ICEB's in order to determine the in-plane properties of shear dominated shear walls. The same soil, press, and block properties determined by Bales et al. were used. Three, 1.8 meter by 1.8 meter walls were constructed and subjected to cyclic lateral loading. One wall was fully grouted without horizontal reinforcement, one was partially grouted without horizontal reinforcement, and one was fully grouted with horizontal reinforcement. The goal was to determine the appropriateness of using current concrete masonry design standards for shear walls constructed with ICEBs. In comparing the in-plane strength of the fully grouted wall

with the partially grouted wall, it was found that the partially grouted wall sustained about 50% of the strength of the fully grouted wall. The study also determined that the ACI 530-08 code predicted a shear strength that by far exceeded the experimentally determined shear strength. The third wall was designed to explore the shear strength contribution due to horizontal reinforcement.

As a companion thesis of Bland (2011), research was simultaneously being conducted to determine the behavior of flexural dominated ICEB shear walls under in-plane loads (Stirling, 2011). Stirling tested three walls: a slender wall, 1800 mm tall and 900mm wide, a 1800 mm by 1800 mm square wall with a 750mm wide flange at one end, and another 1800 mm by 1800 mm square wall with a 900mm square opening in the center. Stirling found that these three walls failed in flexure with tensile yielding of the vertical rebar. This is unlike the brittle failures found by Bland (2011). All walls showed significant differences in ductility due to varying dimensions and reinforcement.

Further analysis was performed of flexural dominated walls in order to more accurately predict the behavior of these types of walls during an earthquake. A non-linear lumped plasticity model and a plastic analysis model were made. Stirling determined that the current masonry code (ACI 530-08) was able to predict the flexural strength of the ICEB walls within an acceptable margin. The lumped plasticity model was found to predict the strength accurately but did not provide a stiffness that was comparable to the experimental results.

2.2 Masonry Wall Design

The design of masonry structures in the United States of America follows the *Building Code Requirements and Specifications for Masonry Structures* (TMS 402/ACI 530/ASCE 5, 2008), with lateral force design procedures from *Minimum Design Loads for Buildings and Other Structures* by the American Society of Civil Engineers (ASCE-7). These building codes prescribe loading, analysis, design, and detailing for masonry construction.

Results of multiple tests on simply supported walls were used to calculate strength and deflection formulas for out-of-plane masonry walls. The code allows the use of the principles of mechanics to determine the actual moments and deflections under different support conditions than those found in simply supported walls. ACI 530-08 presents the following assumptions for the design of reinforced masonry:

1. There is strain compatibility between the reinforcement, grout, and masonry so that loads are resisted in a composite manner.
2. Strain in reinforcement and masonry shall be assumed to be directly proportional to the distance from the neutral axis.
3. The tensile strength of masonry shall be neglected in calculating flexural strength but shall be considered in calculating deflection.

4. The masonry stress shall be taken constant as $0.8 f'_m$ over a length of 0.8 times the length from the extreme most compression fibers to the neutral axis at the strength limit state.

The ACI 530-08 code predictions will be a reference for comparison to the experimental behavior of the ICEB walls.

Brandow, Ekwueme, and Hart (2006) provide a detailed design guide for reinforced masonry structures. The guide outlines the process and equations needed for proper design of concrete masonry walls under out-of-plane loading, citing the ACI 530-08 code as the basis for design. Using the equations included in the design guide example, estimations of wall strength, stiffness, and cracking behavior can be made. The process for estimating deflections under a given loading includes P-delta effects from both self weight and additional axial loads. The out-of-plane lateral loads for pilaster design are shown to include the effect of the wall area tributary to the pilaster. This design assumption will be critical in developing the maximum spacing of stiffener elements in long spanning, slender ICEB walls.

The design guide for masonry structures also provides detailed guide to the use of moment-curvature analysis for flexurally dominated masonry members. By defining a cross-section and the material properties, a procedure can be followed to determine the curvature and moment at each stage of the beam's response to loads. By defining a stress-strain relationship for the reinforcement and the masonry and by applying the law of static equilibrium, a moment and corresponding curvature can be calculated. The

member's behavior is defined by three limit states; cracking, yield, and ultimate. The ultimate limit state is defined to be equal to the moment when the extreme compression fibers of the member fail due to crushing. Moment-curvature analysis and numerous design examples will be used in this thesis for analysis of the experiments.

2.3 Out-of-Plane Wall Testing

Amrhein and Lee (1984) performed experimental testing on load-bearing, reinforced, tall, slender masonry walls. The walls were tested for their out-of-plane performance. The intent was to test concrete and masonry walls that exceeded the code limitations of height to thickness ratio. A total of 32 panels were constructed for testing. The test set up, including the application of the vertical loads and airbag is shown in Figure B.

Vertical loading was applied using a simple loading technique using a lever arm and a barrel of water. The horizontal load was applied using an airbag in order to simulate the uniform loading assumed in out-of-plane loading. The end connections were chosen to be pin-pin in order to reduce the variables and unknown factors. The parameters for design were imposed by the Structural Engineers Association of Southern California (SEAOSC) and the American Concrete Institute – Southern California Chapter (ACI-SC). An initial serviceability limit was imposed where the lateral deflection in a slender wall could not be greater than 1% of the story height. A stricter limit of 0.7% of the story height was approved by the International Conference of Building Officials (ICBO) after code officials and structural engineers decided the permissible deflection

should be reduced. Examples of design for slender walls under these limits are plentiful in the report. These limits and design examples will be applied to the ICEB walls considered for this research program.

Figure B - Amrhein and Lee Test Set-up

Source: (Amrhein and Lee, 1984)

Ismail et. al. (2011) performed out-of-plane experimentation on an unreinforced masonry (URM) wall to study the retrofit technique of introducing post-tensioned tendons to the core of the URM walls. A wall was built using standard clay brick and mortar construction, 11 feet tall and 3 feet 6 inches wide. A single post-tensioning tendon was inserted down the middle of the wall. Special consideration was taken in determining the effective height to be used in this out-of-plane experimental research.

Figure C (a) shows the traditional simply supported beam model for out-of-plane analysis. Figure C (b-d) illustrates the change in moment diagram and the effective height between the inflection points. It was decided that most URM walls had little fixity at the base due to the nature of the masonry to rock on its foundation. Therefore, the experiment would be a simply supported wall with an effective height just short of the average story height of URM buildings. In order to simplify the uniform load experienced during an earthquake, four point loads were applied to the wall using a load spreader.

Figure C - Moment Diagrams of Walls with Various End Restraints

Source: (Lazzarini, 2009)

The testing was completed in the Architectural Engineering facilities at California Polytechnic University at San Luis Obispo, using a test setup that will be adopted for use in this thesis.

The out-of-plane behavior of unreinforced dry-stack masonry was studied by Vaculik, Griffith, Hogarth, and Todd (2004). A series of experiments were set up to study the failure modes of various walls when subjected to out-of-plane loading, as well as to quantify the behavior using airbag tests. All walls were simply supported at top and bottom, and had small wall returns on each side, creating the existence of double bending. It was concluded that the strength of these walls was directly proportional to the amount of axial load applied to the wall. The cracking occurred in a fairly predictable manner for double bending. The most regular wall was able to withstand loading equivalent to an acceleration of 0.42 g's. The cracking pattern will be used in this thesis to demonstrate the cracking patterns during two-way bending. The load-displacement behavior was studied in walls with three different aspect ratios. Each aspect ratio was tested under three different levels of axial compression. These walls showed that dry-stacked masonry walls all exhibited ductile behavior, where the wall gradually lost strength once the maximum load was applied without a sudden drop in stiffness. The ultimate strength and the amount of axial compression were found to be directly proportional, where the strength would increase with more load.

Jayasinghe and Mallawaarachchi conducted flexural strength tests of compressed stabilized earth masonry materials. The tests were designed to measure the flexural strength, both parallel and perpendicular to bed joints of compressed stabilized earth bricks, blocks and rammed earth. With relevance to this thesis, the blocks used were 290 mm long x 145 mm wide x 100 mm tall. An image of the blocks used can be found in Figure D. The blocks were bonded with a cement soil paste, meaning that there is some bond between courses of blocks. The voids in the block were also completely filled with a cement soil mix. None of the walls contained reinforcement.

Small wall samples were tested as shown in Figure E. The walls were able to reach 0.262 N/mm^2 as their average flexural resultant stress parallel to bed joints and 0.261 N/mm^2 perpendicular to bed joints. Thusly, both flexural resultant stresses were found to be about equal. The failure was brittle in nature and occurred when tension cracks started to form on the back side of the walls. The results of these tests provide insight into the flexural behavior relationship between earth block walls loaded in each out of plane direction.

Figure D - Compressed Stabilized Earth Block
(Source: Jayasinghe and Mallawaarachchi, 2009)

Figure E - Jayasinghe and Mallawaarachchi Testing Directions
(Source: Jayasinghe and Mallawaarachchi, 2009)

The Hydraform dry-stack block system was tested by Pave (2007). A complete testing program achieved values for compressive strength and flexural strength for the Hydraform blocks. Hydraform blocks are solid, compressed earth blocks that do allow for reinforcement parallel to the bed joint. The blocks form a shape that is pictured in Figure F, where two edges of the blocks are lowered, to form a dry-stacked, interlocking pattern. The tested compressive strength of a single Hydraform block, with a 5% cement content was found to be 3.0 MPa. The masonry compressive strength, as determined by prism testing was found to be 1.1 MPa for blocks with 5% cement content.

Figure F – Hydraform Blocks
(Source: Pave, 2007)

For the flexural strength tests, Pave decided to use composite beams made of reinforced concrete and dry stacked masonry. Multiple beams with 6 mm steel bar reinforcing were tested with different sizes and cross sections. Each beam was tested with loading perpendicular to the bed joints of the blocks, meaning the beams were tested across their minor axis (see Figure G). The tests showed that the concrete-masonry

beams were able to behave with composite action under flexural loading. A beam was tested without concrete, as shown in Figure G, and showed excessive deflections in the masonry. It should be noted that the Hydraform system does not have vertical grout holes or wide horizontal grout channels to resist this out-of-plane loading. Three out of the four beams tested did not meet the theoretical load capacity during experimentation. However, there were instances in shear cracks that could not be investigated completely at that time. The researchers recommended that the shear resistance of the dry-stack system should be heavily investigated.

Figure G – Example of Cracking Pattern and Loading
(Source: Pave, 2007)

3.0 MATERIALS

This section discusses the materials used in the creation of the ICEB walls. The discussion of relevant materials will include specifications, methods of construction, and measured structural properties.

3.1 Interlocking Compressed Earth Blocks

Interlocking Compressed Earth Blocks used in this experimentation program were formed using a predetermined mixture of materials based from the experiments of Bland (2011) and Stirling (2011). The material properties, process of manufacturing, and quality assurance of ICEB's will be discussed in this section.

3.1.1 Soil

The soil used in the manufacturing of the ICEB blocks was obtained from a local site and is identical to the soil used by Bland (2011) and Stirling (2011). ASTM testing procedures were used to determine the grain size distribution and soil plasticity. Results of ASTM D422-63 for grain size distribution showed the soil consists of approximately 21% clay particles finer than 0.002 mm (Bland, 2011). The plasticity of the soil was found using ASTM D4318-05 (Bland, 2011). Results are shown in the table below.

Table 1 - Soil Plasticity
Source: Bland 2011

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

3.1.2 Sand

Sand used in the ICEB mixture was a medium-fine sand for use in concrete. A screening was used to remove any particles larger than 6 mm. Screened sand was stored dry in a bin until ready for use.

3.1.3 Cement

Type I/II common portland cement was used in the ICEB and grout mixtures.

3.1.4 Material Preparation and Mixing

Soil was air dried and pulverized using a Soeng Thai Model SP3 soil pulverizer, pictured below. The pulverizer hammers the dried soil into grains that may only pass a screen in the machine when smaller than 4 mm. The pulverized soil was then stored in dry bins.

**Figure H - Soeng Thai SP3
Pulverizer**

Soil, sand, and cement were mixed in batches to form eight ICEB's at a time. The size of batches was selected in order to provide enough time for pressing between the addition of water and the mixture being dried out. The proportions of each material were weighed out to the nearest 0.1 kg to the mass specified in Table 2. A variation in the amount of water to each batch was necessary due to the difference in consistency of each batch.

Table 2 - ICEB Mixture Material Weights

	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

Weighed amounts of soil, sand, and cement were first dry mixed in a portable cement mixer. The materials were mixed until a uniform mixture had been achieved. After mixing, the materials were placed on the concrete floor and spread out evenly for wet mixing.

Water was added slowly to the mixture while being blended with shovels. The cement mixer was not used due to the fact that the water saturated soil sticks to the sides of the mixer and does not allow for a uniform blend. Since previous testing has shown water content of each batch to vary depending on clay content, cement content, and the

temperature of the air, water was added cautiously in between “Drop Tests” to check the performance of the batch. The details of the “Drop Test” are shown in Figure I.

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 I - Drop Test
Source: (Proto et. al., 2010)

3.1.5 Pressing Procedure

Once an optimal mixture was obtained, the mixture was separated and weighed into containers, each with enough mixture to make one ICEB. Depending on which type of ICEB was desired, different amounts of mixture were weighed per block. Block types and weights of wet mixture per block for each can be found in the table below.

Table 3 - ICEB Types and Weights per Block

Block Type	Description	Mixture Weight per Block (kg)
Standard	Full size, all 5 holes	8.0
Channel	Full size, all 5 holes, and channel insert	7.3
Pilaster	Full size, 3 holes- two end inserts were removed, and changed bottom press plate	8.2
Half-Standard	Standard block with standard divider plate	8.0
Half-Channel	Channel block with channel divider plate	7.3

The press used for each block was the Soeng Thai Model BP6. This press has the ability to construct each of the blocks needed for this thesis. For each block, the mixture was added to the BP6 press in two stages. Pouring half the mixture into the press at a time allowed for compaction of the mixture into the press at two stages. Without this separation, the mixture would overflow in the press, and the lid would not completely close. Once the mixture was in the press, the lid was closed, and the lever was pulled. After the lever was completely pulled down, it was held there at maximum compression

for a minimum of three seconds. The lever was then released and pulled down in the opposite direction, effectively ejecting the solid block out from the press, as shown in Figure J.

Figure J - Soeng BP6 with Finished ICEB

At least once per batch, blocks were tested for quality of mixture and density by the use of a pocket penetrometer (see Figure K). The test took place before ejecting the block from out of the press. The penetrometer tests the compactness of the block immediately after pressing. This test helped to immediately assure the quality of the mixture and block.

Figure K - Penetrometer Test

3.1.6 Curing

After pressing, each ICEB was carefully moved indoors to an initial curing rack. Each ICEB was allowed to cure on this rack for at least three days and watered at least four times. This initial cure time was used to allow each block to harden enough to significantly reduce the damage to the blocks by stacking.

Once the initial cure was completed, the blocks were moved outside and stacked closely together 8 blocks high. The ICEB stacks were then covered with plastic tarp and watered once a day for a minimum of 4 days. After this humid curing, the blocks were transported to the testing lab.

3.2 Grout

Grout mixtures for ICEB construction must be workable enough to pour into the small holes of the ICEB block. Therefore, a grout mixture with fine sands and a very high slump was used. An effort was also made to create a grout that would closely match the compressive strength of the ICEBs. Previous testing has shown that brittle failures occur in prisms where the grout has a significantly higher compressive strength when compared to ICEBs (Bales et al., 2009). For this thesis, the grout mixture was determined by the previous work of Bland, 2011. This mixture consisted of approximately 1:0.4:2.6:4.2 portions of portland cement to lime to water to sand; all measured by dry volume. The sand used in grout preparation was identical to the sand used in the ICEB construction.

Preparation of the grout consisted of dry mixing the ingredients in 15 liter batches. The dry mixture was then added slowly to a portion of the water and mixed until a homogeneous mixture was obtained. Water was then slowly added until a highly workable grout was achieved.

3.3 Soil-Cement Plaster

A plaster was to be applied to one wall for testing. A sustainable and cost effective mixture was desired using the materials that would already be on site during construction. A suitable mixture of soil, sand, and cement was deemed to be the best option for this experiment. Iterations of different plasters were made in order to find a mixture that would not crack once dried, and had a compressive strength that would

significantly affect the stiffness of the ICEB wall. One of the poor plaster mix is shown in

Figure L (a).

Figure L - Insufficient Plaster Mixture Results

The final mixture, shown in

Figure L (b) consisted of 1:6:0.25 parts of soil to sand to portland cement. The pulverized soil and sand were identical to that used in the pressing of ICEBs.

3.4 Material Testing and Results

Compressive strength testing was performed on samples of individual ICEBs, fully grouted prisms, grout cylinders extracted from the inside of blocks, grout cylinders

from plastic forms, and soil-cement plaster cylinders. Table 4 summarizes the average compressive strengths of each material.

Table 4 - Material Compressive Strengths

Material Type	Number of Samples	Compressive Strength, MPa	Coefficient of Variation
Individual ICEB	22	7.76	13.5 %
Grouted ICEB Prisms	12	2.81	10.7 %
Porous Grout Cylinders	19	9.19	16.4 %
Non-Porous Grout Cylinders	15	5.10	10.5 %
Soil-Cement Plaster Cylinders	3	0.85	13.3 %

3.3.1 ICEB Compressive Strength

Individual ICEB compressive strength was determined by using a universal compressive testing machine. The compressive strength of individual blocks is not used in design of ICEB walls, but does insure the uniformity of each batch of blocks. The BP6 press was dismantled for use of its top and bottom plates. Using these plates in the compressive testing of individual ICEBs was determined to be the most effective method of testing by previous experiments (Bales et al., 2009). Testing was conducted on standard and pilaster block types, their net areas being 39320 mm² and 40570 mm², respectively.

Failure of the individual ICEBs was a conical failure, shown in Figure M. This failure is identical to those found in the previous experiments of Bland, 2011 and Bales et al., 2009.

Figure M - ICEB Compressive Failure

3.3.2 Grout Compressive Strength

Grout samples were tested for each batch of grout poured. Due to the ICEB's inherent water absorbing properties, it was required to test grout samples that had been poured into the blocks. Grout cylinders were carefully removed from the blocks before testing. These samples formed in the blocks are titled porous grout cylinders. The dimensions of the porous grout cylinders were 45 mm in diameter, and 100 mm tall, with an area of 1590 mm^2 . Grout samples were also formed in plastic molds in order to test the relative strength between these non-porous samples and the porous samples. The dimensions of the non-porous cylinders were 76.2 mm in diameter and 152.4 mm tall, with an area of 4560 mm^2 . All types of cylinders were capped with a gypsum-cement capping agent before testing. Figure N shows a non-porous grout cylinder test.

The grout compressive strengths met the masonry code's minimum requirement of having a higher compressive strength than the prisms. The higher coefficient of variation in the porous cylinders compared to the non-porous cylinders shows the inherent variations in ICEB construction.

Figure N - Grout Cylinder Compressive Test

3.3.3 ICEB Masonry Prism Compressive Strength

Stacks of three ICEBs were subjected to compressive loading in order to determine the masonry prism strength (f'_m). The prisms were constructed with each wall, grouted concurrently along with the walls, and allowed to cure under the same conditions as each wall. The height to width ratio of the prisms was 2.0 and compressive strengths were not modified for the aspect ratio. Both the top and bottom of the prisms were capped with gypsum-cement plaster. Figure O shows the testing setup and typical failure

Out-of-Plane Properties of Interlocking Compressed Earth Block Walls

of ICEB prisms. Failure occurred under similar loads and strains to previous experiments, Bland, 2011. The stress-strain plot for three prism tests can be found in Figure P.

Figure O - ICEB Prism Compressive Test

As shown on the stress-strain plot in Figure P, the strain at peak strength is at about 0.014. When compared to ACI-530-08 peak strain values of 0.0025-0.003, the average ICEB peak strain is much larger. The difference can largely be attributed to the dry-stack system. Without mortar to bind the blocks together, there are inherent gaps between the blocks, which must close before compression failure occurs.

Figure P - ICEB Compressive Stress vs. Strain

For post experimental analysis, a modified Hognestad model was fitted to the stress-strain data. The Hognestad model uses a parabola from zero stress to the maximum compressive stress (MacGregor and Wight, 2009). At the maximum compressive stress, the slope of the parabola is zero, and the corresponding strain is ϵ_o .

$$f_m = f'_m \left(\frac{2\epsilon_c}{\epsilon_o} - \left(\frac{\epsilon_c}{\epsilon_o} \right)^2 \right) \quad (\text{for } \epsilon_c \leq \epsilon_o) \quad \text{Eq. 1}$$

For strains past ϵ_o , the stress degrades linearly.

$$f_c = f'_m - 0.25f'_m \left(\frac{\epsilon_c - \epsilon_o}{\epsilon_{cu} - \epsilon_o} \right) \quad (\text{for } \epsilon_o \leq \epsilon_c \leq \epsilon_{cu}) \quad \text{Eq. 2}$$

The final plot in Figure P used values $f'_m = 2.8$ MPa, $\epsilon_o = 0.014$, and $\epsilon_{cu} = 0.02$.

The modulus of elasticity (E) was determined through the analysis of the stress strain relationship using three of the prisms constructed for this thesis. From ACI 530 section 1.8.2.2.1, the modulus of elasticity can be taken as the slope between 0.05 and 0.33 of the maximum compressive strength of each prism. Using this procedure, the average calculated modulus of elasticity was 350 MPa. This is equal to $125 f'_m$, which is lower than Bland's (2011) modulus of $137 f'_m$, and significantly lower than the ACI 530 approximation of $700 f'_m$ for clay masonry. The displacement caused by the closing of gaps at the dry-stack joints in ICEBs is responsible for the majority of difference between the modulus of elasticity values from ACI 530 approximations and experimental results.

3.3.4 Soil-Cement Compressive Strength

Three soil-cement cylinders were tested for the compressive strength of the soil-cement plaster. Two cylinders were formed out of the scratch coat and one was formed out of the finish coat. The soil-cement mixture was formed in the same forms as the non-porous grout cylinders, having an area of 4560 mm^2 . Once the mixture was surface hardened, it was removed from the form in order to allow the mixture to air dry under similar conditions to the wall coating. Each cylinder was capped with a gypsum-cement

capping agent prior to testing. It should be noted that the plaster applied to the walls likely had a stronger compressive strength due to the high absorption rate of the ICEBs.

3.3.5 Steel Reinforcement Tensile Testing

Two sizes of steel reinforcement were used in the construction of the walls. Multiple samples of each rebar were tested under tensile loading. The average yield strength of the Grade 40 #3 bars, with a nominal area of 71 mm^2 , was 338 MPa. The average yield strength of the Grade 60 #2 smooth bars, with a nominal area of 32 mm^2 , was 435 MPa.

4.0 TEST SETUP AND PROCEDURE

The ICEB wall construction methods, test setup, loading protocol, and instrumentation are discussed in this section. Any data reduction measures are introduced and discussed. Finally, prediction methods and calculations are also presented in this section. Table 5 below presents an overall summary of each wall tested for this thesis.

Table 5 - ICEB Wall Specifications

	Boundary Conditions	Added Feature	Width (mm)	Height (mm)	Loading	Vertical Rebar	Horiz. Rebar
Wall 1	Cantilever	None	450	1100	Point Load	(1) #3	(4) #3
Wall 2	Cantilever	Lap Splice	450	1100	Point Load	(1) #3	(4) #3
Wall 3	Cantilever	Plaster Coating	450	1100	Point Load	(1) #3	(4) #3
Wall 4	Pin-Pin	None	1050	2400	4 Point Loads	(2) #3	(6) #3
Wall 5	Pin-Pin	Pilaster	1200	2400	4 Point Loads	(6) #3	(6) #3 & #2 Ties @ 400 mm

4.1 ICEB Construction Methods

All ICEB walls were constructed using methods standard to those found in Thailand and Indonesia. A manual has been produced by the Center for Vocational Building Technologies (CVBT) in order to standardize the construction of ICEB

buildings (Wheeler, 2005). All walls were built in the fashion this manual describes as well as current construction details.

4.1.1 Laying ICEBs

Every first course of ICEBs needed to be laid on a level surface to ensure a straight and vertical wall. A standard mason's mortar mix was used to create this level surface, and to bond the blocks to the base. Blocks in the first course were laid and leveled on a 10 mm to 20 mm thick mortar base and allowed to cure overnight. In order for horizontal reinforcement to be placed at the base of each wall, channel blocks were used as the first course in accordance with the CVBT manual.

Once the first layers of ICEBs were allowed to cure, additional lifts were stacked in groups of four courses. Three courses of standard blocks were laid and leveled, followed by one course of channel blocks. Each channel block layer included one horizontal #3 steel reinforcement bar. The horizontal #3 bars were added in order to simulate real life wall layouts and to ensure that the walls displace homogeneously along the length of each wall.

Due to variance in individual block heights and the mortarless dry-stacked system, there needed to be occasional adjustments to the vertical alignment of block courses. Following the CVBT manual, nails of various sizes were used to lift and align blocks. Once the finished walls were allowed to cure, each nail was removed in order to avoid any cracking caused by these nails. The process was repeated until the desired

height of each wall was met. The final course of blocks always contained channel ICEBs in order to provide a better connection or load path.

4.1.2 Grouting

Grout lifts were no more than five courses high and always ended on a channel block course. The grout pour was ended before the channel cavity was completely full. In order to allow grouting at the ends of the walls, timber members wrapped in plastic were tightened against the end of each wall with a ratchet strap system. This tightening also kept the ICEBs from moving during the grout pour.

Due to the ICEBs natural tendency to absorb water, water was poured into the grout holes just prior to grouting. Enough water was used to ensure that all surfaces were damp, but not overly saturated. Since the grout was a very fluid mix, it was very easy to pour into the grout keys and reinforcement holes. A funnel was very helpful in pouring both the water and grout. In some instances, grout was seen slowly passing through the larger gaps between blocks. This dripping was helpful in determining if the grout had filled all cavities and stopped shortly after the pour.

4.2 Cantilever Walls

Three cantilever walls were constructed using the methods described in Section 4.1. These three walls were designed to be small scale, easily replicable walls that would give information into the behavior of ICEB walls under out-of-plane loading. Along with this basic information, additional studies were designed to observe the effects of some common added features. Using the CVBT manual, two attributes were chosen for

experimentation, a reinforcement lap splice and a soil-cement plaster coating. These walls will hereby be referenced as Wall 1 for the basic wall, Wall 2 for the lap-splice wall, and Wall 3 for the plastered wall. The construction and test setup of all three walls was identical, except for the addition of each feature.

4.2.1 Wall Layout and Construction

The cantilever walls were constructed 1.1 (11 blocks) meters tall and 0.45 meters (1.5 blocks) wide onto pre-existing concrete pads. These concrete pads were clamped to a strong floor. Each wall was reinforced with one #3 steel rebar placed in the center reinforcing hole. The #3 bar was epoxied into a pre-existing concrete pad using Simpson Strong-Tie Set-XP Structural Anchoring Adhesive. Oversized holes were bored and cleaned per the adhesive manufacturer's recommendations.

Figure Q - Cantilever Wall Layout

ICEBs were dry-stacked per the methods described in Section 4.1. Before the placement of the final course, the vertical rebar was bent 90 degrees. The final horizontal reinforcement was made in a U shape in order to make sure there would be no localized failure or slipping between the blocks at the loading point. The U shape bar spanned 300 mm across the width of the wall and had legs of 250 mm inserted into the outer reinforcing holes.

Figure Q shows the typical layout for the cantilever walls.

Three grout lifts were poured for each wall containing the following amount of courses per lift: 5, 4, and 2. In order to provide extra grout cover for the reinforcement, the final pour included an overflow layer above the final course of blocks 25 to 30 mm tall.

For Wall 2, the lap splice was added after the initial vertical reinforcement was adhered to the concrete pad. The goal of including a lap splice was to determine whether the current masonry code provided an adequate calculation for minimum length. Using ACI 530 equation (2-12), the lap splice length was found to be 46 cm.

$$l_d = \frac{0.13d_b^2 f_y \gamma}{K \sqrt{f'_m}} \quad \text{Eq. 3}$$

where d_b is the bar diameter, 9.52 mm,

f_y is the steel nominal yield strength, 275 MPa,

γ is the reinforcement size factor, 1.0 for a No. 3 bar,

K is the minimum masonry clear cover, 70 mm,

f'_m is the masonry compressive strength, 2.8 MPa.

The first rebar was doveled into the concrete pad and cut 46 cm above the top of the concrete pad. The second #3 rebar began just above the concrete pad and was tied against the doveled bar using thin wire ties. Courses of ICEBs were then stacked as previously mentioned.

Wall 3 was identical to Wall 1, until the soil-cement plaster was added. Two layers of the plaster were pressed against both front and back of the wall to get a thickness of 20 mm on each side. Prior to adding the first layer of plaster, the ICEBs were dampened in order to avoid rapid absorption of water. The first layer was also scratched using standard plastering techniques in order to provide a rough surface for the second coat to adhere to.

4.2.2 Loading Method and Instrumentation

Each wall was loaded using the same equipment. Two hollow steel rectangular sections, each measuring 76.2 mm by 25.4 mm with a wall thickness of 6.35 mm were attached to each wall one meter above the concrete pad. A layer of masonry mortar less than 5 mm thick was applied to the inside face of each tube prior to being clamped to the wall. The mortar provided a constant surface for the load to be applied against. Two 20 mm threaded rods were used to clamp the tubes to the walls. The setup is pictured in Figure R.

The steel tubes were attached to a hydraulic jack for loading. The jack had a loading capacity of 17.8 kN and a stroke of 150 mm. The jack was connected to a strong frame that would not displace under loads from the experimentation. Each end of the jack was allowed to freely rotate as required during testing.

Figure R - Cantilever Wall Loading and Instrumentation

The instrumentation chosen for the cantilever walls was designed to provide data for four quantities: load, displacements, strains, and rotations. See Figure R for elevations of the instrumentation. A load cell was attached to the hydraulic jack. Four laser sensors and a turn pot measured displacements along the height of the walls, as well as any in-plane rotation at the top. These five instruments were attached to an instrument

stand. Inside the wall, one strain gauge was placed on the vertical reinforcement 40 mm above the concrete pad. For Wall 2, a strain gauge was also placed halfway up the lap splice on the #3 rebar that was not doveled into the concrete.

On the wall, three small linear variable differential transformers (LVDTs) were placed vertically against each side of the walls. Using small threaded rods inserted into the walls with epoxy adhesive, these LVDTs were stacked one above the other, in order to record the variation of displacements between blocks, see Figure S. These measurements would ultimately record the rotation and curvature of three heights along the walls.

Figure S - Horizontal LVDTs for Block Rotations (Only One Side Shown)

4.2.3 Data Reduction Measures

Displacements measured in the cantilever walls are absolute, and require no reduction measures. However, the three measurers at the top of the walls were checked to see if they presented the same data. By taking the two laser sensors at the top, and linearly interpolating to the center of the wall, the center turn pot could be checked for accuracy.

The six LVDT outputs required an equation to change a set of displacements to a single curvature at each height. By taking one LVDT reading and the LVDT reading at the same height on the opposite side of the wall, with the length each LVDT was away from the other, the following equation yields the curvature at that height of wall.

$$\Phi = \frac{\sin^{-1}\left(\frac{\Delta_{Front} - \Delta_{Back}}{W_{LVDT}}\right)}{H_{LVDT}} \quad \text{Eq. 4}$$

where Φ is the curvature of the wall,

Δ is the change in LVDT reading (mm),

H_{LVDT} is the height between LVDT measuring points (mm), and

W_{LVDT} is the width between LVDT measuring points (mm).

4.3 Full Scale Walls

Two full scale walls were designed for this research in order to observe the behavior of large walls with and without a pilaster. The first wall, the flat wall, will hereby be called Wall 4. The second wall was built with a pilaster, and will be named Wall 5. The CVBT manual issues a warning that no wall longer than 4.5 meters should be constructed without a corner, interior wall connecting it, or a pilaster. One goal of

these two experiments is to examine the CVBT's estimate of 4.5 meters in relation to the out-of-plane bending experienced during an earthquake.

4.3.1 Wall Layout and Construction

Both full scale walls were designed using typical building layouts in Thailand and Indonesia. Although in building details, the ICEB walls often have continuous reinforcement into a concrete footing below, in order to simplify the test and create a determinate structure, the top and bottom connections of each wall was pinned.

A steel plate connected to a steel channel section with a pin connection was used as the base. The steel plate was bolted to the strong floor with three 38 mm bolts. The channel section allowed the wall to be built on top of the steel web, with the flanges pointing up. This set up allowed the first course of blocks to be set on a mortar bed and be surrounded by mortar (see Figure T), allowing no sliding in the channel.

Figure T - Pinned Connections

A reinforced concrete beam was cast in place on the top of each wall. Two horizontal openings allowed for a set of steel plates to be attached to the wall (see Figure T). These plates created a pinned connection with two high strength steel tubes that would transfer load into a strong frame about 1 meter away from the wall. At both top and bottom of each wall, a #3 steel reinforcement was added in a U-shape from one vertical bar to another, to ensure that the blocks would not slide at the points of highest shear loads. Elevations of each wall showing the reinforcement layouts can be found in Figure U.

While both walls had a pin to pin height of 2.4 meters, the width of each wall varied slightly to accommodate symmetry. Wall 4 was 1.05 meters (3.5 blocks) wide,

while Wall 5 was 1.2 meters (4 blocks) wide. The plan views and alternating courses can be seen in Figure U, along with elevations and rebar layouts. Each wall contained five grout pours.

Figure U - Full Scale Wall Plan and Elevations Views

Pilasters in current construction details were shown to have shear reinforcement whenever horizontal rebar was placed. In order to accommodate the #2 smooth bars that were chosen for shear ties, openings needed to be cut into some ICEBs. A circular saw with a masonry blade was found to be the most effective method for these cuts. Figure V shows the cuts and layout for the shear reinforcement.

Figure V - Typical Pilaster Tie Installation

4.3.2 Loading Method and Instrumentation

During an earthquake, forces are approximately evenly distributed along the entire area of the wall. In order to simulate this even distribution of force, a loading tree was built to take the loading from one hydraulic jack, and distribute it evenly at four, equidistant heights along the wall, see Figure W. By using four point loads, the stresses and overall behavior of the wall found under uniform loading is closely replicated

without the aid of an airbag. Therefore, the results of these experiments are transferable to real world design and analysis of ICEB walls. The loading tree was designed with two hinges to accommodate the curvature of the wall during loading.

Figure W - Full Scale Wall Loading and Instrumentation

A hydraulic jack with a loading capacity of 44.5 kN and a stroke of 203 mm was used for both tests. It was attached with pinned connections to a strong frame and the loading tree. The pilaster wall was loaded so that the long face of the wall was in compression. In Figure U, the load would be coming from the bottom of the page.

A total of ten LVDTs were used on Wall 4 and eleven were used on Wall 5. The extra LVDT for Wall 5 was needed to see if the flanges of the wall were moving more or less than the pilaster core. As Figure W shows, displacements were measured at the base, at the top horizontally, at the top vertically, and at five different heights of each wall.

Two strain gauges were added to each wall at mid span. On Wall 4, both #3 reinforcement bars had strain gauges. On Wall 5, gauges were placed in the pilaster, one on a compression bar and one on a tension bar.

4.3.3 Data Reduction Measures

All displacement data, except for the data from the vertical displacement instrument at the top of the walls, will undergo two simple reduction measures in order to obtain the correct measurements. First, assuming the top of the wall will displace more than the base, all values will be subtracted by the displacement measured at the base. By removing any movement at the base, the displacements along the wall will be relative to the pinned base. Second, the average displacement of the two top horizontal displacement sources will be subtracted using linear interpolation down along the wall from the remaining outputs. The base and top points should be at relative zero displacement throughout the entire test using this method.

4.4 Wall Behavior Predictions

ACI 530 does require minimum properties for proper use of the design methods, some of which were not met with the ICEBs. These include a minimum compressive strength of 10.34 MPa, a minimum vertical reinforcing area of 0.002 multiplied by the gross cross sectional area, and, although it is never explicitly stated that dry-stacked masonry is not allowed in the ACI code, material and stack pattern lists do not include dry-stacked masonry. However, since current design in the US is done according to the ACI 530 and other similar codes elsewhere, the cracking and nominal flexural load and displacement values were calculated with this code.

Using the Strength Design method from ACI 530, both cracking moments and moments corresponding to the nominal flexural strength were calculated using the following equations. To make the flexural strength calculations relevant to experimental results, each predicted moment value was used to find the total load applied to the each wall. For Wall 5, the full wall was used to calculate the predicted values. Also, although there is no mortar between blocks, in order to follow the ACI 530 process, an ACI 530 standard modulus of rupture (f_r) of 0.69 MPa (Table 3.1.8.2.1) for tension normal to the bed joints in a stack bond was used for all walls. The actual steel reinforcement yield strength was used in all calculations. For Wall 3, the added plaster was assumed to have the same strength as the ICEBs, and was consequently given extra thickness, equal to the thickness of the plaster, when calculating the following values.

$$M_{cr} = S_n f_r \quad \text{Eq. 5}$$

$$M_n = A_s f_y \left[d - \frac{A_s f_y}{1.6 f'_m b} \right] \quad \text{Eq. 6}$$

where M_{cr} is the cracking moment strength (N-m),

S_n is section modulus (m^3),

A_s is the steel cross sectional area (m^2),

f_y is the steel yield strength (MPa),

M_n is the nominal moment strength (N-m),

d the distance from extreme compression to center of tensile area (m),

f'_m is the masonry compressive strength (MPa), and

b is the width of wall (m).

Anticipated deflections corresponding to the cracking moment and yield moment were also calculated using the equations below. Equation 7 is the standard cantilever deflection equation for the cracking displacement. By using Equation 8 and 9, the cantilever deflection after cracking can be found. These equations are found in the design guide produced by the Concrete Masonry association of California and Nevada (Brandow, Ekwueme, and Hart, 2006). For the full-scale walls, a different set of equations must be used. By substituting Equation 10 for Equation 7, and Equation 11 for Equation 8, the maximum deflection at the mid-height of each full-scale wall can be predicted. The full scale wall deflection calculations were derived for simply supported walls with four equal and equidistant point loads. For each type of wall, simply supported and cantilever, the load, shear, and moment diagrams can be found in Appendix C.

It should be noted that these equations, derived from standard concrete and masonry beam mechanics, may not be entirely applicable to the ICEB walls. For instance, when looking at Equation 9, the value of the modular ratio (n) is much larger for ICEBs than is commonly used in concrete or CMU design. Also, the assumptions in these formulas are inherently violated due to the dry-stacking construction method. However, these formulas must be used in order to compare the standard masonry design methods to those which would accurately predict the behavior of ICEB walls.

$$\Delta_{cr} = \frac{M_{cr} h^2}{3 E_m I_g} \quad \text{Eq. 7}$$

$$\Delta_n = \Delta_{cr} + \frac{(M_n - M_{cr}) h^2}{3 E_m I_{cr}} \quad \text{Eq. 8}$$

$$I_{cr} = n A_s (d - c)^2 + \frac{b c^3}{3} \quad \text{Eq. 9}$$

$$\Delta_{cr} = \frac{0.105 M_{cr} h^2}{E_m I_g} \quad \text{Eq. 10}$$

$$\Delta_n = \Delta_{cr} + \frac{0.105 (M_n - M_{cr}) h^2}{E_m I_{cr}} \quad \text{Eq. 11}$$

where M_{cr} is the cracking moment strength (N-m),

h is the height of the wall (m),

E_m is the masonry modulus of elasticity (MPa),

I_g is the gross moment of inertia (m^4),

M_n is the nominal moment strength (N-m),

I_{cr} is the cracked moment of inertia (m^4),

n is the modular ratio (E_s/E_m),

A_s is the steel cross sectional area (m^2),

d is the distance from extreme compression to center of tensile area (m),

c is the depth of compression zone (m),

b is the width of the member (m),

Δ_{cr} is the displacement at the cracking moment (m), and

Δ_n is the displacement at the nominal moment (m).

Pilasters add depth to the wall, stiffening the wall, and allow larger loads to be applied. With this extra depth, there is a possibility for shear dominated behavior. In order to predict the outcome of the testing of Wall 5, ACI 530 was used to determine the maximum applied shear force before shear failure. The ACI equation for maximum shear strength can be found in Eq. 12-14. Once the maximum shear force was acquired, it was translated into the resulting moment at mid-height of the wall for direct comparison to the maximum flexural strength.

$$V_n = V_{ns} + V_{nm} \quad \text{Eq. 12}$$

$$V_{ns} = 0.5 \left(\frac{A_v}{s} \right) f_y d_v \quad \text{Eq. 13}$$

$$V_{nm} = 0.083 \left[4 - 1.75 \left(\frac{M_u}{V_u d_v} \right) \right] A_n \sqrt{f'_m} \quad \text{Eq. 14}$$

where V_n is the nominal shear strength (N),

V_{nr} is the nominal shear strength due to steel reinforcement (N),

V_{nm} is the nominal shear strength due to masonry (N),

s is the spacing of the shear reinforcement (mm),

A_v is the cross sectional area of steel (mm^2),

f_y is the yield stress of the steel (MPa),

d_v is the shear depth of the wall (mm),

A_n is the net area of the member (mm^2),

f'_m is the masonry compressive strength (MPa), and

$\frac{M_u}{V_u d_v}$ need not exceed 1.0 for slender walls.

A sample calculation of Wall 1 and Wall 4, as well as shear strength calculations for Wall 5 are provided in Appendix A. A summary of all calculated predictions is found in Table 6.

Table 6 - ACI 530 Predictions

Wall Number	M_{cr} (N-m)	F_{cr} (N)	M_n (N-m)	F_n (N)	Δ_{cr} (mm)	Δ_n (mm)
Wall 1	582	582	1528	1528	4.4	14.8
Wall 2	582	582	1528	1528	4.4	14.8
Wall 3	915	915	2013	2013	3.5	9.4
Wall 4	1358	3772	3128	8689	7.9	23.0
Wall 5 Flexural	1553	4314	10440	29000	0.020	0.35
Wall 5 Shear	-	-	20831	57860	-	-

5.0 EXPERIMENTAL RESULTS

This chapter will provide the results for each test as well as details pertaining to the loading sequence, cracking locations, and overall performance of each wall. For each of the five walls, a separate loading sequence was selected in order to best observe the behavior of each at cracking and nominal strengths. Each sequence begins with target loads until a load equal to approximately $2/3$ of the nominal flexural strength. At $2/3$ the nominal flexural strength, the equivalent displacement is recorded. This displacement is used to continue the sequence with target displacements equal to a multiplier times the $2/3$ strength displacement. Each loading cycle was performed twice for each target. The load sequence is provided for each wall under its specified section below.

5.1 Cantilever Walls

A table of all the cantilever walls and their parameters can be found in Table 5. All cantilever wall tests were carried out successfully. However, each wall displaced more than anticipated during the testing. In order to perform cyclic testing on the wall (push and pull), the hydraulic jack was attached to the wall with half of its stroke extended. This way, each loading direction could displace equal maximum amounts. During the cyclic testing of the wall, displacements became larger than the maximum stroke of the hydraulic jack. Once the jack could not be extended any further, the jack was repositioned 75mm closer to the wall. Subsequently, the jack could only load the wall in the push direction. Each load sequence table shows when the jack was moved indicated by a “No” under the “Push and Pull” column, see Table 7.

Each experiment continued the loading sequence until the hydraulic jack's stroke was at its maximum. The walls were found to be able to displace increasingly large amounts past their observed maximum strength without increasing load, indicating that the wall had yielded and the testing could be concluded.

5.1.1 Wall 1: Basic Wall

Wall 1 was used to provide a basis for all other walls in this experimental program. The loading sequence used for Wall 1 is presented in Table 7. Note how the loading sequence changes from target load to target displacement at a load of 1000 N. Also, due to the aforementioned jack relocation, at 1000 N the wall was only loaded in the push direction. The Δ_N as described in Table 7 was found to be 48 mm for Wall 1.

Table 7 - Wall 1 & 2 Loading Sequence

Step	Push and Pull	Load (N)	Displacement
1	Yes	50	-
2	Yes	100	-
3	Yes	200	-
4	Yes	400	-
5	Yes	800	-
6	Yes	1000	$\Delta_{2/3}$
7	No	-	$\Delta_{2/3} \times 1.5 = \Delta_n$
8	No	-	$2 \Delta_n$
9	No	-	$3 \Delta_n$
10	No	-	$4 \Delta_n$

Figure X - Wall 1 Force Displacement Response and Strain vs. Displacement

The hysteretic behavior (a) and the strain data (b) of Wall 1 is presented in Figure X where the force represents the actuator force and the displacement is the horizontal displacement 1000mm above the base. An enveloping curve was formed out of the peak value for each of the push and pull cycles, see Figure EE. On the hysteresis loop there are also distinct points of initial stiffness loss due to cracking and significant stiffness loss due to yielding. These observed points of experimental cracking and nominal strengths are also shown in the Figure as a circle and a triangle, respectively. The yielding point on the hysteresis loop corresponds to a rebar strain value of 0.0036. This strain value is higher than the yield strain value of 0.0017 (1700 micro strains), which shows that the masonry failure and steel yielding did not occur simultaneously, and therefore the overall stiffness did not decline until higher loads were induced.

It can be seen that Wall 1 exhibited stable hysteretic behavior, a large displacement capacity, and show large amounts of pinching throughout the test. Table 8 shows the ACI code calculations accurately predicted the nominal flexural strength of Wall 1, but the predicted displacements were significantly lower than experimentally determined. The comparison shows that the strength design methods in the ACI translate well to ICEB design. However, the testing of Wall 1 shows that the inherent gaps between the ICEB courses, caused by the variance in the heights of blocks, creates a behavior that allows for more rotation and therefore more displacement than traditional masonry. The fact that the predicted nominal flexural strengths were surpassed for each cantilever wall experiments could be caused by numerous variations in ICEB

performance when compared to traditional masonry. Possibly the most direct variation could be the assumption that the steel reinforcement does not undergo strain hardening. This would cause the flexural strengths to be inaccurate in small amounts, similar to what was observed with the cantilever experiments.

Table 8 – Cantilever Walls ACI 530 vs. Experimental Results

		M_{cr} (N-m)	M_n (N-m)	Δ_{cr} (mm)	Δ_n (mm)
Wall 1	Predicted ACI 530	582	1528	4.4	14.8
	Experimental	213	1720	8.9	89.1
Wall 2	Predicted ACI 530	582	1528	4.4	14.8
	Experimental	352	1660	8.3	99.4
Wall 3	Predicted ACI 530	915	2013	3.5	9.4
	Experimental	797	2215	3.9	25.4

It should be noted that the values in Table 8 were determined by using the hysteresis loops for each wall to locate points of stiffness change. The peak values for each cycle were used to form an enveloping curve that more clearly indicates when a change in behavior was found. The points were determined by observation using these enveloping curves. In general, two of these points were found for each wall, indicating a cracking point and a nominal flexural strength point.

Figure Y - Typical Horizontal Cracking

Horizontal cracking in the grout cores at dry-stacked joints initiated under low force levels, see Figure Y. The cracks were observed starting above the base layer of ICEBs and slowly progressing up the wall above each layer of blocks. These horizontal cracks were limited to the dry-stack joints, and through post-experimental inspection, the grout cores in each block were found perfectly intact.

5.1.2 Wall 2: Lap-Splice Wall

Wall 2 was used to observe the behavior in a lap splice at the area for highest forces in a wall. The strength and deflection calculations were identical to Wall 1 and therefore, the loading sequence for Wall 2 was identical to that of Wall 1 (see Table 7). Again, the hydraulic jack was moved after the 1000 N cycles.

In comparison to the results from Wall 1, the lap splice wall behaved similarly. Wall 2 showed a slightly higher initial stiffness compared to Wall 1, but the flexural strength was similar in both, see Table 8. The observed experimental cracking and nominal strengths are noted with a circle and a triangle on the hysteresis loop in Figure Z. The hysteretic response of Wall 2 shows that the wall experienced a significant drop in stiffness at a load of 1250 N. At this time, a large vertical crack appeared on the face of the wall in the third course (Figure AA). This crack broke the block in half across both the length and the depth of the block. The stiffness was later regained in the wall, bringing the wall above the ACI determined nominal flexural strength. Similar to Wall 1, the ACI code was able to accurately predict the nominal flexural strength, but not the displacements. The yield force (marked as a triangle on Figure Z) was taken as the second instance of significant stiffness loss because of this vertical crack.

Figure Z - Wall 2 Force Displacement Response

Figure AA - Vertical Crack in Wall 2

The strain gauges on the steel reinforcement provided good insight into the behavior of the lap splice in Wall 2. Figure BB plots the strain vs. the displacement in both the starter bar (Strain at Base), doweled into the concrete footing, and the lap bar (Strain at Mid Splice). The Strain at the Base shows the steel reinforcement at the base was able to yield significantly, past the yield strain of 0.0017 (1700 micro strains), indicating that the lap splice was able to transfer load adequately into the footing. The strain a mid-splice did not yield significantly, even at high displacements.

Figure BB – Wall 2 Strain vs. Displacement

5.1.3 Wall 3: Plastered Wall

Wall 3 was designed to investigate the effect of a plaster coating on ICEB walls. Specifically, a plaster coating was chosen to limit the blocks from rotating due to the

inherent gaps between ICEB courses. The loading sequence used for Wall 3 is presented in Table 9. The jack relocation was able to be held off until later in the experiment for Wall 3 due to its increased stiffness compared to Walls 1 and 2. The jack was finally moved before the 7 Δ_n cycles.

Table 9 - Wall 3 Loading Sequence

Step	Push and Pull	Load (N)	Displacement
1	Yes	100	-
2	Yes	200	-
3	Yes	400	-
4	Yes	600	-
5	Yes	1000	-
6	Yes	1300	$\Delta_{2/3}$
7	Yes	-	$\Delta_{2/3} \times 1.5 = \Delta_n$
8	Yes	-	2 Δ_n
9	Yes	-	3 Δ_n
10	Yes	-	4 Δ_n
11	Yes	-	5 Δ_n
12	No	-	7 Δ_n
13	No	-	10 Δ_n

The hysteretic behavior and strain vs. displacement data of Wall 3 is presented in Figure CC. The observed experimental cracking and nominal strengths are also shown in the Figure again marked with a circle and a triangle, respectively. Wall 3 was similar to Wall 1 in that it exhibited stable hysteretic behavior and shows large amounts of pinching throughout the test. Wall 3 showed a drop in flexural strength past the 50 mm displacement mark which coincided with plaster spalling from the ICEB wall. Past the

50mm displacement, the Wall 3 plateaued at a force of 1800 N, which is similar to the observed yield strength of Wall 1 and Wall 2. The yielding point on the hysteresis loop corresponds to a strain value of 0.0020. This strain value is higher than the theoretical yield strain value of 0.0017, but smaller than the Wall 1 strain of 0.0036, indicating that the overall system was more balanced due to the added thickness of the plaster.

Table 8 shows that the ACI code calculations accurately predicted the nominal flexural strength of Wall 3. For Wall 3, the accuracy of the displacement predictions increased, but the cracking displacement and nominal displacement prediction errors were still 10% and 63% lower than the actual results respectively. This can be attributed to the plaster's ability to prevent the large rotations between blocks that were observed in the first two walls.

Although the cracking pattern in Wall 3 included horizontal cracks at the dry-stack joints, similar to Wall 1 and Wall 2, the plaster created more visible cracks with different behaviors. Figure DD shows cracking in the face of the plaster (upper picture) that initiated before any cracks in the grout could be observed. The lower picture of Figure DD shows a section view of Wall 3 near the end of the testing. Substantial rotation away from the mortar bed and the plaster spalling away from the ICEB face is shown.

Figure CC – Wall 3 Force Displacement Response

Figure DD - Wall 3 Cracking Pattern

5.1.4 Comparison of Force-Displacement Behavior

By using the load and displacement measured at each of the targets from each of the three hysteresis curves, an enveloping curve of each wall was created for comparison between results, Figure EE.

Figure EE - Enveloping Comparison of Hysteretic Behavior of Cantilever Walls

The comparison in Figure EE shows a slight increase of initial stiffness in Wall 2 compared to Wall 1. Despite the difference in stiffness, the flexural strengths were similar. Wall 3 showed a significant increase in stiffness, due to the plaster filling the gaps between the dry-stacked joints. Wall 3 also showed a significant increase in flexural capacity, due to the increased wall thickness provided by the plaster. Each wall consistently exceeded the ACI predicted nominal flexural strength prior to yielding, but

even with a plaster coating, each cantilever wall exhibited a stiffness that was far lower than predicted.

5.1.5 Moment-Curvature Response

Understanding the behavior of each cantilever wall is aided by analyzing the moment-curvature relationship. By using the three sets of vertical LVDTs set at different heights along the walls, see Figure R and Figure S, a total of nine Moment-Curvature relationships were calculated. The calculation of curvature is found in Eq. 4. The calculated moments that correspond to each curvature were taken as the measured load multiplied by the distance from the applied load to the mid-height between levels of sensors. For the bottom level of LVDTs, the moment was calculated as the measured load, multiplied by the distance from the applied load to half the height of the bottom LVDTs.

Figure FF – Experimental Moment-Curvature Envelope Curves

Out-of-Plane Properties of Interlocking Compressed Earth Block Walls

Figure FF, a), b), c) show the observed moment-curvature relationship in each wall as recorded by each of the vertical LVDTs previously mentioned. As shown, the results from Walls 1 and 2 are similar in nature. The relationships of Wall 2 are very similar along the height of the wall, meaning that at each height where measured, the moment-curvature relationship did not change. This similarity is ideal for beam mechanics. Wall 1, however, had varying relationships along its height. For Wall 1, the moment-curvature relationship shows that more rotation occurred at the top LVDTs when subjected to equal moments. Wall 3 shows more strength at each curvature amplitude than the other two walls. Again, the moment-curvature relationships varied along the height of the wall, with the curvature concentrated at the base. More rotation at the base is attributed to the cracking resistance achieved by the plaster.

5.2 Full Scale Walls

Each of the two full scale wall tests was accomplished without error. The loading patterns, hysteretic behaviors, observed cracking patterns, failure modes, and displaced shapes of each wall are presented and compared in the following sections.

5.2.1 Wall 4: Full Scale Prismatic Wall

Wall 4 was designed to provide data on the performance of prismatic ICEB walls (walls without stiffening members such as pilasters). The mid-height moment at any given total actuator load was calculated using a simply supported beam with four equal point loads applied, see Appendix C. The loading sequence was then calculated to best observe the expected performance of Wall 4. This loading sequence can be found in Table 10. For Wall 4, Δ_n in Table 10 was found to equal 93 mm.

Table 10 - Wall 4 Loading Sequence

Step	Actuator Force (N)	Mid-Height Moment (N-m)	Mid- Height Displacement
1	200	72	-
2	400	144	-
3	800	288	-
4	1500	540	-
5	2000	720	-
6	3000	1080	-
7	4000	1440	-
8	5000	1800	$\Delta_{2/3}$
9	-	-	$\Delta_{2/3} \times 1.5 = \Delta_n$
10	-	-	$2 \Delta_n$

Figure GG - Wall 4 Force-Displacement Response

The hysteretic behavior of Wall 4 is presented in Figure GG. The performance of Wall 4 was similar to the previous walls, in that the ICEB wall was able to withstand large displacements while continuing to resist load. Similar to the results from the cantilever wall tests, Wall 4 proved to be significantly more flexible than the ACI prediction. However, as Table 11 shows, Wall 4 was not able to meet or exceed the predicted nominal flexural strength. An asymmetric displacement in the loading tree was observed during the experiment and was confirmed by the displaced shape of the wall near the end of the test (See Figure JJ). This possibly resulted in uneven loading that

could have contributed to Wall 4 not being able to meet the predicted nominal flexural strength.

Table 11 – Wall 4 ACI 530 vs. Experimental Results at Mid-Height

	M_{cr} (N-m)	M_n (N-m)	Δ_{cr} (mm)	Δ_n (mm)
Predicted ACI 530	1358	3128	7.9	23.0
Experimental	542.5	2940	11.7	106

Figure HH - Wall 4 Rebar Strain vs. Displacement

Wall 4 was flexure dominated, with a cracking pattern that was consistent with that found in Wall 1 and Wall 2. Grout cracks formed in the dry-stack joint between ICEB courses. These cracks began at the mid-height of the wall and continued to propagate both above and below mid-height joints as the loading increased. The push

cycles were ended when a large crack created instability at the base of the wall. In the fourth ICEB course from the base, a vertical crack split through the ICEBs, separating the section of ICEB that was in tension from the rest of the ICEB and the grout keys. Figure II shows this vertical crack near the end of the test. As testing continued, the steel reinforcement inside the wall appeared to be slipping out of the bottom three courses. Once the wall began to slide away from these bottom courses, the testing was concluded. Analysis of the stain gauge data, Figure HH, showed that significant yielding occurred before failure. Therefore, it can be assumed that the observed flexural resistance of the wall would not have significantly increased had the test been continued.

The type of crack and the location of the crack indicate that the load distribution was weighted more heavily near the base of the wall. Also, the connection between the base and the wall was not adequate for this experiment. The crack in Figure II is located just below the final loading point, the steel hollow steel section. This location on the wall also was where the added U-shaped rebar, see Figure U, began. It should also be noted that the length of vertical U-bar legs were about 38 cm, shorter than the lap splice length determined for Wall 2. It is clear that the loading caused the two reinforcing bars to slide past each other, instead of working together. To mitigate this problem, a concrete beam with rebar embedded, similar to the detail used at the top of Wall 4, would have constrained the vertical rebar from slipping and would likely have prevented this failure.

Figure II - Vertical Crack in Wall 4

Figure JJ - Wall 4 Displaced Shape

The displaced shapes at the final six target displacements are shown in Figure JJ. The curves show a generally parabolic shape in the wall up until the mid-span displacement reached 98 mm. A comparison of the measured displacement at 400 mm high and 2000 mm high shows that at the 98 mm and above test cycles, the wall began to displace more at the base. Using simple beam mechanics, if the load distribution was equal throughout the experiment, the displacement readings at 400 mm and 2000 mm should be equal throughout the test. The final displaced shape shows the extent of the failure at the base. As shown the displacement reading at the 400 mm height is approximately equal to that of the 1600 mm height.

5.2.2 Wall 5: Full Scale Pilaster Wall

Wall 5 was designed to provide data on the performance of ICEB walls with a stiffening member, a pilaster. The loading sequence found in Table 12 was calculated to best observe the performance of Wall 5. For Wall 5, Δ_n in Table 12 was found to equal 16.5 mm.

Wall 5 was unable to achieve the predicted flexural strength. Again, the load distribution from the loading tree could have contributed to this behavior. However, another factor in the difference between the predicted nominal flexural strength and the experimental strength is the fact that Wall 5 exhibited shear failure or lap splice failure towards the base. As seen in Figure KK, in the excursion to a total force of 17499 N, the stiffness begins to degrade at an increasing rate. Visual inspection showed that the cracking started to form at a shearing force of 8.7 kN, or a total force of 17400 N, at a

mid-height displacement of 10.9 mm. Significant strength degradation occurred at a total force of 26940 N. A comparison between the actual and ACI predicted strengths and deflections is found in Table 13.

Table 12 - Wall 5 Loading Sequence

Step	Actuator Load (N)	Mid-Height Moment (N-m)	Mid-Height Displacement
1	500	180	-
2	1000	360	-
3	1500	540	-
4	2000	720	-
5	3000	1080	-
6	5000	1800	-
7	8000	2880	-
8	10000	3600	-
9	12000	4320	-
10	15000	5400	-
11	17300	6228	$\Delta_{2/3}$
12	-	-	$\Delta_{2/3} \times 1.5 = \Delta_n$
13	-	-	$1.5 \Delta_n$
14	-	-	$2 \Delta_n$
15	-	-	$2.5 \Delta_n$

Table 13 – Wall 5 ACI 530 vs. Experimental Results at Mid-Height

	M_{cr} (N-m)	M_n (N-m)	Δ_{cr} (mm)	Δ_n (mm)
Predicted ACI 530	1553	10440	0.020	0.35
Experimental – Flexural	2473	-	3.9	-
Experimental – At Failure	-	9700	-	22.3

Figure KK - Wall 5 Force Displacement Response

The cracks formed in the bottom courses of the pilaster. As shown in Figure LL, the cracks were predominantly vertical along the lines of the reinforcement holes. The cracks could have been caused by two different types of failure, shear failure or lap splice

failure. A shear failure could have been one type of failure most likely caused by inadequate spacing of the reinforcement ties. The current masonry codes prescribe tie spacing to be no more than the depth of the member. For Wall 5, the ties were spaced at 400 mm, while the depth of the pilaster was 300 mm. Therefore, the failure could have been attributed to this difference. Another possibility is that de-bonding occurred in the lap splice area at the bottom of the wall. A lap splice failure makes the grout core split as the reinforcement deforms and slides against each other, eventually causing the blocks to split as shown in Figure LL. Further research into the shear strength of ICEB walls and lap splice behavior is necessary for further ICEB development.

Figure LL - Wall 5 Cracks and Uplift

Figure MM - Wall 5 Rebar Strain vs. Displacement

Two strain gauges were placed in Wall 5 at mid-height, one on a tension rebar and one on a compression rebar. As shown in Figure MM, the compression strain gauge showed sporadic strains that showed no regular pattern of stress. The tension gauge did show strains in a regular pattern. However, the strain data for the pilaster test shows that yielding of the tension steel only just began at the point of stiffness degradation due to cracking near the base. Since no significant yielding was observed, Wall 5 was not flexure dominant.

Along with the cracks, uplift began between the first and second block layers. The vertical reinforcement and surrounding grout began to separate from the first layer of ICEBs. The uplift is shown in Figure LL. The wall began to slide away from the base, similar to in the final pushes of Wall 4. Once the uplift created instability in the wall, the

experiment concluded. This uplift shows that the construction technique used at the base could have been improved by restraining the rebar from lifting away from the base by welding or other connection.

Figure NN - Wall 5 Displaced Shape

The displaced shapes for the six largest displacement excursions are shown in Figure NN. Similar to Wall 4, a possibly uneven load distribution created asymmetric displaced shapes. At around the $\Delta = 24.8$ mm displaced shape, a significant difference between the 400 mm high and 2000 mm high displacement readings begin to show. The sliding effect due to the uplift previously mentioned subsequently created larger displacements at the bottom.

5.2.3 Comparison

Comparing the results from Wall 4 and Wall 5 provides insight into the effectiveness of adding a stiffening element to a prismatic wall. Enveloping curves of each wall's performance are compared in Figure OO. Comparing the effective stiffness of each wall, i.e. the slope of the line drawn from zero to the displacement at peak strength, the pilaster is shown to increase the stiffness by over 16 times the flat wall. Therefore, the pilaster in Wall 5 increased the out-of-plane wall stiffness significantly and confirms the beneficial effect of using pilasters to control the out-of-plane displacement.

Figure OO - Enveloping Comparison of Hysteretic Behavior of Full Scale Walls

6.0 MOMENT-CURVATURE AND PILASTER ANALYSIS

All of the five tested ICEB walls gave insight into the behavior under out-of-plane loading. The flexural strength of ICEB walls under out-of-plane loading was shown to be accurately predicted by the masonry code provisions set forth by ACI 530, but the stiffness and displacement calculations proved to be inaccurate, as expected, and unreliable for these walls. While strength calculations play a major role in the design of structures, displacement and stiffness calculations are equally important. In order to provide a better structural design of ICEB buildings, it is required to provide a better model to calculate the stiffness of ICEB walls loaded out-of-plane. If the walls are not able to meet their necessary stiffness, and stiffening elements must be added, a question of how to space these elements arises. Also, the strength of pilasters as a stiffening element was shown to be inaccurately estimated relative to shearing forces. In this section, the development of a more accurate strength and stiffness prediction method, an analysis of the shear strength of pilasters, as well as a guide to show how many stiffening elements must be added to a long spanning wall is described.

The analysis and findings in this chapter are only valid for the specific ICEB blocks and materials used in this thesis. By varying dimensions, material properties, or the reinforcement within the walls, the findings from this chapter could differ greatly. Therefore, it is incorrect and potentially unsafe to use the results for any other block types.

6.1 Moment Curvature Analysis

In order to design ICEB buildings, the behavior and performance of ICEB walls during out-of-plane loading must be accurately predicted. While the results from all five tested walls show that the nominal flexural strength can be accurately predicted using the ACI 530 masonry code, the stiffness estimations of these walls were proven to be significantly higher than the actual values. The out-of-plane stiffness must be correctly calculated in order to provide a safe structural design.

A major contribution to wall failure can be the P-Delta effect. This effect amplifies the flexural load in a wall that grows proportionally with the amount of deflection of the wall. Gravity loading creates an additional moment in the column or wall in addition to the lateral force causing the initial displacement. Displacements then increase from this P-Delta effect, which creates a compounding effect. In order to keep the P-Delta effect low, the displacement allowed in walls must be limited. ACI 530 suggests a drift limit of 0.7% in order to limit the lateral deflection to a position where the wall returns to its original vertical position when the lateral load is removed. This 0.7% limit will be applied to the out-of-plane ICEB walls for this thesis. The drift limit is applied per the ACI 530 standard, to the displacement at mid-height of the wall, and is calculated as the total height of wall multiplied by 0.007.

A more accurate prediction of deflections in a wall is achieved by understanding and quantifying the Moment-Curvature relationship for a given wall. For any given

material and cross-section, the Moment-Curvature relationship can be used to accurately predict the force displacement response for any given loading.

6.1.1 Cantilever Wall Moment-Curvature Results

As previously discussed in Chapter 5.0, the cantilever wall tests provide insight into the Moment-Curvature relationship for ICEB walls under out-of-plane loading. The three Moment-Curvature relationships for Wall 1, Wall 2, and Wall 3 are shown in Figure FF. Each plot is shown as enveloping curves for clarity. By using these relationships, a more accurate model of force displacement for ICEB walls can be calibrated.

6.1.2 Moment-Curvature Relationship Computation

The calculation of a Moment-Curvature relationship is derived from cross-sectional statics, see Figure PP. The strain profile of any given cross-section, under multiple strains allows for computational analysis forming a Moment-Curvature relationship. By using a spreadsheet program, the curvatures and corresponding moments can be calculated for any material. The process will be discussed in this section.

Figure PP – Strain Profile for Moment Curvature Analysis

The spreadsheet used for ICEB analysis allows the user to input the material and cross-sectional properties of the member to be analyzed. The depth of the member is divided into multiple slices, and the area and depth of the centroid of each slice is calculated. For any given strain at the extreme compression fiber, an average strain is applied through linear interpolation to each slice. Using a user-defined stress strain relationship, the strain of each section is turned into a stress. This stress is then multiplied by the area of the section to create a force. The forces from all these slices are used to determine the moment by multiplying the force by the depth of centroid. Appendix B contains sample calculations for one point on the moment-curvature relationship.

Concurrently, a similar process is occurring for the reinforcing steel in the member. The user inputs the steel properties and the location and size of each steel member. A strain, stress, force, and moment are then assigned to each reinforcement bar.

The spreadsheet uses an iterative process by changing the “c” value on Figure PP so that the sum of the axial forces is equal to the axial load applied to the member. This axial load is also user defined. The resisting moments and corresponding curvature is recorded for each strain step. This process is run for multiple extreme compression fiber strains to form the Moment-Curvature relationship.

A cracking moment and curvature was calculated using Equation 5. The curvature at the moment of cracking is calculated as the cracking moment divided by the modulus of elasticity (E) and the gross cross sectional moment of inertia (I). Appendix B also contains a sample calculation for this point on the moment-curvature relationship.

6.1.3 Moment-Curvature Spreadsheet Check

In order to verify the moment-curvature spreadsheet conforms to the theory set forth by similar, previous analyses, a moment-curvature design example from Brandow, Ekwueme, and Hart (2006) was input into the program. The moment and curvature values given by the program were then checked against the example’s values at various strain values for accuracy.

Example 5.2.2 and example 5.2.3 of Brandow, Ekwueme, and Hart (2006) show a beam 32 inches deep and 7.625 inches wide fully grouted with 0.4 in² of grade 60 reinforcement centered 28 inches down from the top of the beam. The beam is fully grouted, the masonry compressive strength (f'_m) is given as 1500 psi, and the modulus of elasticity (E) is equal to $900f'_m$. The moments and curvatures are found at six different loading patterns. These loading values are determined as at the maximum compressive

strain values equal to 0.0005, 0.0015, 0.002, and 0.0025 as well at the loading that causes cracking and then yielding in the reinforcement. The assumed stress strain relationship for these examples is a bi-linear function where the stress linearly climbs from the origin to f'_m at a strain equal to 0.0015. The stress then declines linearly to $0.5f'_m$ corresponding to a strain equal 0.0025. Finally, it is assumed that the rupture stress for cracking is set to 200 psi.

By using the moment-curvature spreadsheet developed for this thesis, the input parameters from the examples were entered. The moment-curvature relationship was then calculated for each of the maximum compressive strains as well as cracking and yielding loads. Cracking and yielding loads are defined per the masonry code, as the load that induce cracking across the section and the load that creates yielding in the extreme tension reinforcement, respectively. Table 14 below shows the comparison between the examples' relationships and the relationship found by the spreadsheet program.

Table 14 – Moment-Curvature Spreadsheet Check

Strain Value or Limit State	Brandow, Ekwueme, & Hart (2006)		Moment-Curvature Spreadsheet	
	Moment (k-ft)	Curvature (in^{-1})	Moment (k-ft)	Curvature (in^{-1})
Cracking	21.7	0.0000092	21.7	0.0000092
0.0005	31.29	0.0000639	31.03	0.0000628
Yield	50.77	0.000103	51.03	0.000103
0.0015	53.16	0.000358	53.20	0.000357
0.002	53.43	0.000567	53.43	0.000566
0.0025	53.11	0.000716	53.13	0.000716

As the results from the comparison show, the spreadsheet is able to calculate the moment-curvature relationship accurately for a given cross-section at any load or compressive strain value. The differences in the comparison are attributed to the two differences in stress-strain relationships. The masonry stress in the program increases at a faster rate than in the example problems, which is shown by the moment-curvature values at a strain of 0.0005. The strain hardening effect of the reinforcement comes in later for strain values of 0.002 and 0.0025. However, these effects are more accurate to actual conditions, and are needed for this thesis.

6.1.4 Calculated ICEB Moment-Curvature Relationship

By entering the material properties of ICEBs, and the cross sectional properties of Wall 1 and Wall 2, a Moment-Curvature relationship was calculated. The material properties include the width, thickness, reinforcement area, reinforcement depth, and modulus of elasticity of the masonry. For the ICEB walls, the stress-strain relationship determined in Figure P and Equations 1 and 2 was used. Figure QQ shows the stress-strain relationship for the vertical steel reinforcement in each ICEB wall. The relationship used the experimentally determined yield stress of 338 MPa, with a standard modulus of elasticity (E) of 200,000 MPa.

Figure QQ - Assumed Stress-Strain Plot for Steel Reinforcement

Other inputs for the ICEB walls included wall thickness (150 mm for Walls 1, 2, & 4, 150 mm plus 20 mm on each side for Wall 3 and the plaster, and 300 mm for Wall 5), wall length (450 mm for Walls 1-3, 1050 mm for Wall 4, and 1200 mm for Wall 5), and depth to tension bar (always assumed to be in the center of the reinforcement hole, 75 mm from the extreme tension fiber of the block). With these inputs, a moment-curvature relationship is calculated by the spreadsheet program. Again, a sample calculation can be found in Appendix B.

The experimental results from Wall 1 and the spreadsheet calculated relationships are shown in Figure RR. The calculated cracking moment and curvature is shown with a diamond point, while the calculated yield moment and curvature is shown with a square point on the graph.

Figure RR – Wall 1 Moment-Curvature Comparison

The comparison between the cantilever wall results and the initial spreadsheet calculation shows that the ICEB walls exhibit more curvature for a given moment. It is clear from this comparison that the ICEBs are more flexible than the moment-curvature analysis program determines.

In order to predict a more accurate moment-curvature relationship of dry-stacked ICEB walls for out-of-plane loading, two factors were investigated: the cracked section properties and the dry-stack joint gaps. As illustrated in Figure SS, the section properties

of the ICEB wall before cracking should not be taken from the gross section. Since the ICEBs are dry-stacked onto each other, there is no tensile restraint provided by the uplifting end of the blocks. In traditional mortared masonry, mortar creates a tensile restraint throughout the entire block. For this reason, when calculating the gross moment of inertia (I_g) and the gross section modulus (S_n), the depth of the wall is assumed to be 2/3 times the overall depth. For Wall 1 and Wall 2, this would mean the effective depth of the wall is 100 mm, instead of 150 mm.

Figure SS - Cracking Section Comparison

Additionally, the inherent gaps in the dry-stack joints allow for more rotation of each block, without adding more flexural restraint. By taking an estimated average gap thickness per joint, a constant additional curvature can be added to the curvatures found by the moment-curvature spreadsheet. A table of gap sizes for a 150 mm wide, 100 mm tall block and the additional curvatures they bring is found in Table 15.

Table 15 - Gap Size to Added Curvature

Gap Size (mm)	0.2	0.4	0.6	0.8	1.0
Curvature (1/mm)	0.000027	0.000053	0.000080	0.000107	0.000133

An average gap size of 0.4 mm per block, adding 0.000053 (1/mm) of curvature, was found to be adequate for matching the average experimental moment-curvature relationship. The additional curvature is added to every value found by the moment curvature spreadsheet except for zero. The final calculated moment-curvature relationship for Wall 1 and Wall 2 is shown in Figure RR. The gap size of the plastered wall, Wall 3, needed to be decreased to 0.1 mm, due to the stiffness of the plaster and the plaster's ability to fit into the inherent gaps between ICEB courses. The pilaster wall, Wall 5, was modeled with the pilaster and wall flanges, as well as all reinforcement.

6.1.5 Force-Displacement from Moment-Curvature

To bring the moment-curvature relationship to a force-displacement response for direct comparison to the experimental results, the following procedure was used. Figure TT can be used as a reference to the procedure, where a) refers to the loading pattern, b) shows the actual deflected shape, c) displays the moment diagram with the average moment a single block undergoes (M_i), and d) shows the rigid body rotation at that block (θ_i) that corresponds to the curvature (Φ_i).

In order to find the displacement at a given point, the wall must be broken down into separate levels. The moment at that level (M_i) will be calculated and used to provide

a corresponding curvature value (Φ_i). For all of the tested walls, the levels were made from each course of blocks. In this way, the cantilever walls all had 10 levels of calculated values. Using the a load value and the distance from beam end to the centroid of the selected block level, a moment can be calculated that equals the average moment in that level of ICEBs.

Figure TT - Force-Displacement from Moment-Curvature

With a moment value (M_i) and the moment-curvature relationship found by the programmed spreadsheet, a curvature value (Φ_i) can be found using linear interpolation between two found points on the spreadsheet. These curvatures are turned into rotations (Θ_i) by multiplying them by the height (h_i) of the layer they represent. In this case the height (h_i) is equal to 100 mm. Using beam mechanics, this rigid body rotation can be translated into a displacement (Δ_i) measured at the end of the cantilever walls, or the center of the simply supported, full scale walls. By summing these rigid body rotation

based displacements for each layer of ICEBs, a total displacement is found. By varying the load, this process can be done repeatedly for each wall and recorded as a predicted force-displacement response.

For walls with pin-pin boundary conditions, Wall 4 and Wall 5, the process is similar. The moment diagram changes with the different loading, and in order to find the displacement at mid-height, only half of the wall height is used when adding the rigid body rotations because of symmetric loading.

Figure UU shows the predicted and experimentally based force-displacement responses for all walls. Figure UU (a) graph shows the responses for the cantilever walls, while Figure UU (b) shows the responses for the full scale walls. These plots show close correlation between the predicted and the experimental responses. The displacements and the flexural strengths are predicted with acceptable accuracy. This indicates that using the moment-curvature relationship derived for ICEB walls suitably predicts the stiffness and strength of flexure dominated ICEB walls under out-of-plane loading.

Figure UU - Experimental vs. Moment-Curvature Force Displacement Response

6.2 Spacing Determination of Stiffening Elements

In the manual of construction for ICEB buildings, the author states that there should never be a wall longer than 4.5 meters without stiffening members (Wheeler, 2005). A stiffening member, a perpendicular wall or pilaster, would be required in any span longer than 4.5 meters. It is unknown how this number was determined. Using the data provided by the two full scale wall experiments, and assuming a flexural dominated wall and pilaster system, a pilaster spacing limit, see Figure VV can be calculated for ICEB walls.

Figure VV – Out-of-Plane Stiffening Element Plan

6.2.1 Out-of-Plane Forces

According to the current seismic design standard (ASCE-7, 2005), also shown in Brandow, Ekwueme, and Hart (2006), the out-of-plane earthquake load is calculated using Equation 11, but cannot be less than 10% of the weight (W) of the wall.

$$F_p = 0.4S_{DS}IW \quad \text{Eq. 15}$$

The density of fully grouted ICEBs was found to be an average of 18 kN/m^3 . The calculated load is distributed over the area of the wall. For all cases being investigated in this thesis, the wall thickness is one block or 0.15 m thick. The average weight (W) per area can therefore be calculated as 2.7 kN/m^2 . The importance factor (I) will be taken as 1.0 for the calculations in this thesis. The final values for load per area therefore depends on the value for the design spectral response acceleration parameter at short periods (S_{DS}), which is a value calculated using seismic data for the location of the structure. Values vary from location to location, therefore a range of S_{DS} values will be included in the spacing analysis of stiffening elements.

6.2.2 Pilaster Performance

In order to determine the spacing requirements of stiffening elements such as pilasters, the strength of the members themselves must first be checked against the demand. Where an intersecting wall acts as a stiffener, each intersecting wall must be checked to ensure adequate strength. In the case of a standard, one block pilaster, like the one built into Wall 5, analysis can be done to show that for various wall heights and various pilaster spacing, the pilaster can perform successfully during typical earthquake accelerations.

One or more limit states for the pilasters must be chosen to best provide a safe and stable structure during an earthquake. The results from Wall 4 and Wall 5 (see Figure WW) show that Wall 4 and Wall 5 could meet the deflection limit before the maximum flexural strength is exceeded. Therefore, the calculation for allowable load on a pilaster

will be based on the theoretical flexural strength and the allowable drift limit of 0.7% from the International Conference of Building Officials (ICBO) (see Section 2.3). The drift limit will ensure no point on the wall will be able to displace, in the out-of-plane direction, more than 0.7% of the wall height, while the flexural strength limit will constrain the pilaster to essentially elastic behavior.

Special consideration is needed for typical ICEB buildings due to methods of construction. At the top of most ICEB walls, a simple concrete beam is cast on top of the wall. Typical roofing members only connect to the ICEB at the pilasters or corners of the walls. Because of this condition, the tributary area of the pilaster must include the top portion of wall, between pilasters, see Figure XX. The tributary area is assumed to start at the base, at a 45 degree angle, until the two points meet. The spacing of the pilasters determines where the 45 degree angle lines will meet along the height of the wall, and will vary the pattern and magnitude of the load. For analysis of the pilaster's ability to withstand a seismic event, the height and spacing must both be variables. Also, a fixed-pinned support condition is assumed, removing any partial fixity the pilaster may have at the top of the wall while assuming a fully fixed base.

Figure WW - Full Scale Walls with Drift Limit

Figure XX – Tributary Area of Pilaster When $H = S$

Figure YY - Pilaster Tributary Area and Loading When $H = S$

The calculation of allowable load under the drift limit of 0.7% uses the moment-curvature spreadsheet developed in Section 6.1 to calculate the largest acceptable S_{DS} value for a given wall height. The pilaster was also restricted by its flexural strength. By using the moment-curvature relationship and the loading diagram shown in Figure YY, an allowable S_{DS} for each wall height is produced. Figure ZZ shows the distribution of

allowable S_{DS} values for the ICEB pilaster configuration used in this thesis, at various heights of walls at three spacing values. The spacing values used were equal to half the height, 1.5 times the height, and equal to the height. It can be seen that in order to achieve the drift limit or flexural strength of the pilaster, large S_{DS} values (larger than 1.0) must be applied. Today, ICEB walls in single story construction are typically about 2400 mm tall. Therefore, it is highly unlikely that the accelerations will be high enough to cause flexural failure or large displacements in a pilaster of the proportions investigated herein.

Figure ZZ - Allowable Accelerations for Pilasters

6.2.3 Deflection Based Limitation for Pilaster Spacing

As all five walls tested in this thesis has shown, the ICEB walls are much more flexible than other types of masonry. Due to this increased flexibility, the walls would likely meet the allowable deflection limit prior to yielding. Therefore, analysis was

performed to determine the maximum pilaster spacing in order to limit the deflections to 0.7% of the story height.

Using boundary conditions that consider the most conservative types of walls found in typical single story construction, a simplified model for deflection can be made. For a deflection based analysis for an ICEB wall panel, the top of the wall will remain free, while the sides and bottom of the wall will remain fixed. The wall is loaded with a uniform load across the entire area of the wall.

A comparison of displaced shapes between a wall and a horizontal strip is shown in Figure AAA using a finite element analysis program. The finite element model uses plate elements that are of a unit thickness. The pilasters on each side were modeled as an area of double thickness. As the deflected shapes shows, the largest displacement occurs at the top of the wall, mid-span between the two pilasters (designated by the brightest shade of white), and is adequately matched by the horizontal strip.

Figure AAA - Deflected Shape and Comparison of Wall Panel and Horizontal Strip

In order to simplify the analysis of a wall in double bending as shown in the figure below, some assumptions must be made. The finite element model was used to check various simplified models, in order to closely match the deflection found in the original analysis from Figure AAA . By using the computer analysis program, it was found that in all cases where the height to spacing ratio was between 1/1 and 1/2, the maximum displacement could be found within 25% of the displacement calculated in a full wall model, using a model of just a horizontally spanning strip of wall of equal stiffness. Various spacing vs. height displacements using the finite element models are compared in Table 16. Although most cases shown in Table 16 show the strip method is not conservative for displacement based design, this analysis method provides adequate insight into the wall displacement between pilasters. From this analysis, in order to calculate the maximum pilaster spacing using the 0.7% displacement limit, a beam analysis of a horizontally spanning wall section could be used. The 0.7% displacement limit will still be used to check the out-of-plane displacement against 0.7% of the total wall height.

Table 16 – Finite Element Model Displacement Comparison

Aspect Ratio (H/S)	Displacement		% Difference
	Full Wall	Strip (L=S)	
1/1	195	185.7	5.0
4/5	185	185.7	-0.4
3/5	169	185.7	-9.9
1/2	145	185.7	-22.7

An assumption must be made to determine the maximum spacing of pilasters using the model described above. It must be assumed that the moment-curvature relationship of a horizontal strip of ICEB wall is has the same basic behavior as a vertical strip of ICEB wall. The vertical relationship was experimentally tested during this thesis and analytically determined in Section 6.1. By using the moment-curvature spreadsheet for ICEB walls, and by providing the reinforcement spacing for a horizontal strip of wall, the maximum displacement of the horizontal strip can be determined for any given load. For the analysis of pilaster spacing, it is assumed that the end restraints of the horizontal strip of wall are fixed. Therefore, the moment diagram embedded into the spreadsheet was changed for this fixed-fixed support condition.

For the deflection based calculation of pilaster spacing, the height and earthquake acceleration values are both input parameters. Heights were varied from 2200 mm to 3000 mm in 200 mm steps. Earthquake acceleration, or S_{DS} values were varied from 0.25 to 1.5 in 0.25 increments. By running the moment-curvature spreadsheet for all variations of the two parameters, the maximum pilaster spacing for each was found. Results are summarized in Figure BBB.

Figure BBB - Maximum Pilaster Spacing from Displacement Based Analysis

The results from the displacement based analysis show only a slight difference in allowable spacing when varying the height. The height was varied to show that the shape of the maximum spacing curve remains the same as the maximum displacement value (0.7% of the height) is increased. In places of high seismicity, where S_{DS} values are equal to or above 0.75, the results show that a maximum spacing of 3.0 to 3.5 meters is necessary to limit the amount of displacement experienced during out-of-plane loading. From this analysis, it is recommended that for all ICEB structures in earthquake regions, there is no wall spanning 3.5 meters without a stiffening element.

6.3 Pilaster Failure

As discussed in the experimental results of the full scale pilaster wall, section 5.2.2, the pilaster was unable to meet its nominal flexural strength due to cracks forming at the bottom courses of the wall. These cracks could have been caused from failure in the anchorage of the reinforcement to the base, shear failure in the masonry, or a combination of the two.

A further look into the ACI 530 will be completed in this section to compare predicted values to experimental values. The ACI 530 equations for shear strength are shown in Section 4.4 as Eq. 12, 13, and 14.

By using equations 12, 13, and 14, the following values for nominal shear strength can be calculated. See Appendix A for values used in this calculation.

$$V_{nm} = 21100 \text{ N} \qquad V_{ns} = 7830 \text{ N}$$

Comparing these values to the 13470 N of shear that was observed at the failure of Wall 5, it is clear that the ACI 530 predicted does not adequately predict the shear behavior of an ICEB pilaster. The predicted values combine for a greater shear strength than what was observed.

For nominal shear strength of the steel reinforcement, the ACI 530 code requires the spacing of shear reinforcing ties to be no greater than the depth of the member. The pilaster was 300 mm in depth, so therefore the tie spacing should have been no greater than 300 mm. The full scale pilaster wall, Wall 5, was built with ties every 400 mm, and

was consequently not up to ACI 530 standards. Since the tie spacing did not meet the code standards, the contribution of shear strength due to the steel cannot be taken into effect. For all future ICEB buildings, the code limit of tie spacing should be enforced, at a minimum, in order to avoid shear dominated behavior.

With the shear strength of the masonry as the only contributor to nominal shear strength, the ICEB pilaster should have still been able to resist more force than the results of Wall 5 shows. Therefore, the ACI 530 calculation for masonry shear strength does not directly apply to ICEB walls. The masonry code over-predicts the contribution of shear strength due to the ICEB masonry. Therefore, it is a recommendation of this thesis that more research into the shear strength of ICEB walls and pilasters is needed to make a more accurate calculation for shear strength.

7.0 CONCLUSION AND RECOMMENDATIONS

This thesis conducted experimental and analytical research on ICEB walls. A total of five walls were tested in order to gain data on the performance of these walls while under out-of-plane loading. Three cantilever walls, 1100 mm tall by 450 mm wide, and two full scale, simply supported walls, 2400 mm tall by 1050 mm and 1200 mm wide were built. The first cantilever wall was built as a basis for the other two cantilever walls, as well as a method to predict the behavior of the full scale walls. The second and third cantilever walls were constructed to separately test the influence of a reinforcement lap-splice and a thick layer of plaster. The full scale walls were constructed to determine the difference in performance between a wall with and without a pilaster. The hysteretic behavior of each wall was recorded and compared to the ACI 530 code predicted values.

Analysis of the data was performed in order to meet the goals assigned for this thesis. The main goal of calculating or predicting the out-of-plane stiffness and strength of ICEB walls was met using moment-curvature analysis specifically made for the dry-stack construction present with ICEBs. The spacing of pilasters or other stiffening members was calculated using the aforementioned moment-curvature analysis combined with a drift limit set in place by the ACI 530 code.

7.1 Experimental Results

- Walls 1, 2, 3, and 4 all exhibited flexural dominated behavior. The steel reinforcement in each wall yielded before the conclusion of each experiment.

- Cracking in Walls 1, 2, 3, and 4 took place in the grout cores at the dry-stack joints between courses of ICEBs. These cracks were predominantly horizontal, straight across the plane of the joints.
- Comparing each wall's flexural strength predictions to the experimental results shows that the ACI 530 analysis techniques work reasonably well in determining the out-of-plane nominal flexural strength of ICEB walls.
- The ACI 530 analysis for predicting the out-of-plane stiffness of ICEB walls proved to be unsatisfactory for each wall. The code predicted a stiffness far higher than experimentally observed.
- The dry-stack joints left gaps in some cases between the courses of ICEBs. These gaps significantly decreased the overall stiffness of the walls.
- A comparison between the performance of Wall 1 and Wall 2 suggests the ACI 530 calculation for reinforcement lap-splice length is adequately transferable to use with ICEB walls.
- Wall 3, the cantilever wall with a plaster coating showed a strength and stiffness higher than in Wall 1. The plaster was applied into the gaps between the dry-stacked blocks, which allowed the plaster to control the rotations of the ICEBs due to the dry-stack joints.

- Comparing the full scale walls shows a dramatic increase of stiffness when a pilaster is added.
- Wall 5, the full scale wall with a pilaster developed cracks before the steel reinforcement could yield. The behavior of Wall 5 indicates that shear strength and lap bar strength must go under further review.

7.2 Analytical Results

- A moment-curvature program developed for use in concrete design can be used to predict the performance of ICEB walls loaded out-of-plane.
- Two additional factors were added to the moment-curvature program for ICEB design. The first took into account the fact that the blocks were dry-stacked by using an effective depth of wall to be half of the total depth. This property was used to find the cracking moment and corresponding curvature. The second factor added a constant amount of curvature by suggesting that each block was allowed to freely rotate by a set amount due to the inherent gaps in the dry-stack joints.
- The shear strength of ICEB pilasters could be lower than predicted using the ACI 530 code. It is recommended that the maximum spacing of shear reinforcement ties be limited to 0.3 m, or, in this case, one every three block courses. More shear reinforcement could be required.
- Although the maximum spacing of pilasters in a long spanning, slender wall is determined by the load applied and the height of the wall, for ICEB construction, it

is recommended that there is no span of wall longer than 3.5 meters without a pilaster or other stiffening member. In locations of high seismicity, a limit of 3.0 meters may be necessary to reduce damage.

7.3 Future Research Recommendations

It is the recommendation of this thesis that the following investigations take place to further the development of ICEBs as a structural system.

- The shear capacity of columns and pilasters must be investigated for members with varying reinforcement ratios and spacing of shear reinforcing ties. A ductile failure of ICEB walls under out-of-plane loading is preferred for these members.
- Explorations into methods to reduce the gap size at the dry-stack joints would reduce the amount of rotation added due to these joints. A decrease in rotation would reduce the displacement of a wall and ultimately raise the allowable load on ICEB walls.
- The connection detail at the pinned base of walls 4 & 5, for future reinforced walls, must be analyzed for uplift and for longitudinal reinforcement movement to avoid premature failure.

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APPENDIX A

Example Calculation of Predicted Behavior

Wall 1

Wall Width	450	mm
Wall Depth	150	mm
Wall Height	1000	mm
f_r	0.69	MPa
E_m	350	MPa
f'_m	2.8	MPa
d	75	mm
I_g	126562500	mm ⁴
S_n	1687500	mm ³
I_{cr}	4034032	mm ⁴
a	24	mm
C	30	mm
n	571	

Steel E	200000	Mpa
Steel F_y	24160	Newtons

$$= W \times D^3 / 12$$

$$= W \times D^2 / 6$$

$$\Rightarrow \text{EQ. 9}$$

$$= F_y / (0.8 \times f'_m \times W)$$

$$= a / 0.8$$

$$= E_s / E_m$$

Cracking Moment

$$\Rightarrow \text{EQ. 5}$$

582 N-m

Yield Moment

$$\Rightarrow \text{EQ. 6}$$

1522 N-m

Cracking Deflection

$$\Rightarrow \text{EQ. 7}$$

4.4 mm

Yield Deflection

$$\Rightarrow \text{EQ. 8}$$

14.8 mm

Wall 4

Wall Width	1050	mm	Steel E	200000	Mpa
Wall Depth	150	mm	Steel F_y	24160	Newtons
Wall Height	2400	mm			
f_r	0.69	MPa			
E_m	350	MPa			
f'_m	2.8	MPa			
d	75	mm			
I_g	295312500	mm ⁴	=	$W \times D^3 / 12$	
S_n	3937500	mm ³	=	$W \times D^2 / 6$	
I_{cr}	741086	mm ⁴	=>	EQ. 9	
a	10	mm	=	$F_y / (0.8 \times f'_m \times W)$	
C	13	mm	=	$a / 0.8$	
n	571		=	E_s / E_m	
Cracking Moment			=>	EQ. 5	
	1358	N-m			
Yield Moment			=>	EQ. 6	
	3128	N-m			
Cracking Deflection			=>	EQ. 10	
	7.9	mm			
Yield Deflection			=>	EQ. 11	
	23.0	mm			

Wall 5 Shear Calculation

A_v	64	mm ²
S	400	mm
d_v	225	mm
f_y	435	MPa
$M/(Vd)$	1	
a_n	67500	mm ²
f'_m	2.8	MPa

Steel Shear Strength

==> EQ. 13

7830

N

Masonry Shear Strength

==> EQ. 14

21102

N

Total Shear Strength

==> EQ. 15

28932

N

APPENDIX B

Example Calculation of Moment-Curvature Calculation

- Input for Wall 1 (Equal to Wall 2)

Wall 1 and Wall 2

Steel Rebar Properties

Steel Yield Stress:	0.34 Gpa
Steel Modulus of Elasticity:	200 GPa
ϵ Yield	0.0017
ϵ strain hardening	0.008
ϵ Ultimate	0.12

Block Properties

Concrete Compressive F'_c :	0.002758 Gpa
Concrete Modulus of Elasticity:	0.35 GPa
Strain maximum:	0.02
Strain at Yield:	0.014

Member Properties

Web Thickness:	450 mm
Beam Depth:	150 mm
Area:	67500 mm ²
Tension Depth:	75 mm
Area of Tension Steel:	71 mm ²
Ratio of Tension to Compression:	1

Slice Properties

Sections in Web:	70 sections
------------------	-------------

- Compressive Section Slice Properties and Lumped Mass Steel Properties

Section	Web (1=yes)	Thickness	Area	Y bar
1	1	2.14	964.3	1.07
2	1	2.14	964.3	3.21
3	1	2.14	964.3	5.36
4	1	2.14	964.3	7.50
5	1	2.14	964.3	9.64
6	1	2.14	964.3	11.79
7	1	2.14	964.3	13.93
8	1	2.14	964.3	16.07
9	1	2.14	964.3	18.21
10	1	2.14	964.3	20.36
11	1	2.14	964.3	22.50
12	1	2.14	964.3	24.64
13	1	2.14	964.3	26.79
14	1	2.14	964.3	28.93
15	1	2.14	964.3	31.07
16	1	2.14	964.3	33.21
17	1	2.14	964.3	35.36
18	1	2.14	964.3	37.50
19	1	2.14	964.3	39.64
20	1	2.14	964.3	41.79
21	1	2.14	964.3	43.93
22	1	2.14	964.3	46.07
23	1	2.14	964.3	48.21
24	1	2.14	964.3	50.36
25	1	2.14	964.3	52.50
26	1	2.14	964.3	54.64
27	1	2.14	964.3	56.79
28	1	2.14	964.3	58.93
29	1	2.14	964.3	61.07
30	1	2.14	964.3	63.21
Steel	Row 1		0	0
	2		0	0
	3		71	75
	4		0	0
	5		0	0

The thickness and area are functions of the amount of section cuts prescribed in the input section. The Y bar is the distance from the centroid of the section to the extreme compression fiber of the cross-section.

- Program calculation table example. Input maximum compressive strain of 0.01.

C = 36.5294994		Curvature = 0.0002738		
At Concrete Strain of: 0.01				
Y from d	Strain	Stress	Force	Moment
73.928571	0.00971	0.002499	2.41	178.12
71.785714	0.00912	0.002423	2.34	167.72
69.642857	0.00853	0.002338	2.25	156.98
67.5	0.00795	0.002242	2.16	145.96
65.357143	0.00736	0.002138	2.06	134.72
63.214286	0.00677	0.002023	1.95	123.33
61.071429	0.00619	0.001899	1.83	111.84
58.928571	0.00560	0.001765	1.70	100.31
56.785714	0.00501	0.001622	1.56	88.80
54.642857	0.00443	0.001469	1.42	77.38
52.5	0.00384	0.001306	1.26	66.10
50.357143	0.00325	0.001133	1.09	55.02
48.214286	0.00267	0.000951	0.92	44.21
46.071429	0.00208	0.000759	0.73	33.71
43.928571	0.00149	0.000557	0.54	23.61
41.785714	0.00091	0.000346	0.33	13.94
39.642857	0.00032	0.000125	0.12	4.78
37.5	0.00000	0	0.00	0
35.357143	0.00000	0	0.00	0
33.214286	0.00000	0	0.00	0
31.071429	0.00000	0	0.00	0
28.928571	0.00000	0	0.00	0
26.785714	0.00000	0	0.00	0
24.642857	0.00000	0	0.00	0
22.5	0.00000	0	0.00	0
20.357143	0.00000	0	0.00	0
18.214286	0.00000	0	0.00	0
16.071429	0.00000	0	0.00	0
13.928571	0.00000	0	0.00	0
11.785714	0.00000	0	0.00	0
75	0.01000	0.346017219	0.00	0
75	0.01000	0.346017219	0.00	0
0	0.01053	0.347597621	-24.68	0
75	0.01000	0.346017219	0.00	0
75	0.01000	0.346017219	0.00	0
Sum:			0.00	1526.51

The input strain in green is the extreme most compression strain. For each section, the strain, stress, and force are calculated. The moment for each slice is calculated about the center of the reinforcement. The sum of the forces must equal zero, as shown next to “Sum” at the bottom of the force column. The sum of the moments is shown as well. The curvature is found by dividing the maximum compressive strain by the distance to the neutral axis (C).

- Program calculation table example. Steel at point of yield.

Steel Yield

C = 57.2227736		Curvature = 0.0000956		
At Concrete Strain of: 0.0054721				
Y from d	Strain	Stress	Force	Moment
73.92857	0.00537	0.001710	1.65	121.90
71.78571	0.00516	0.001660	1.60	114.88
69.64286	0.00496	0.001608	1.55	107.99
67.5	0.00475	0.001555	1.50	101.23
65.35714	0.00455	0.001501	1.45	94.62
63.21429	0.00435	0.001446	1.39	88.16
61.07143	0.00414	0.001390	1.34	81.86
58.92857	0.00394	0.001333	1.28	75.72
56.78571	0.00373	0.001274	1.23	69.76
54.64286	0.00353	0.001214	1.17	63.97
52.5	0.00332	0.001153	1.11	58.38
50.35714	0.00312	0.001091	1.05	52.97
48.21429	0.00291	0.001028	0.99	47.77
46.07143	0.00271	0.000963	0.93	42.78
43.92857	0.00250	0.000897	0.87	38.01
41.78571	0.00230	0.000830	0.80	33.46
39.64286	0.00209	0.000762	0.74	29.14
37.5	0.00189	0.000693	0.67	25.06
35.35714	0.00168	0.000623	0.60	21.23
33.21429	0.00148	0.000551	0.53	17.65
31.07143	0.00127	0.000478	0.46	14.33
28.92857	0.00107	0.000404	0.39	11.27
26.78571	0.00086	0.000329	0.32	8.50
24.64286	0.00066	0.000253	0.24	6.00
22.5	0.00045	0.000175	0.17	3.80
20.35714	0.00025	0.000096	0.09	1.89
18.21429	0.00004	0.000016	0.02	0.29
16.07143	0.00000	0	0.00	0
13.92857	0.00000	0	0.00	0
11.78571	0.00000	0	0.00	0
75	0.00547	0.34	0.00	0
75	0.00547	0.34	0.00	0
0	0.00170	0.34	-24.14	0
75	0.00547	0.34	0.00	0
75	0.00170	0.34	0.00	0
Sum:			0.00	1332.62

See notes on previous page.

- Cracked Moment and Curvature input and calculation and ICEB gap thickness calculation.

Cracked Section Calculation

fr:	0.69 Mpa
I gross:	126562500
n:	571.43
Sn:	750000 mm ³
Mcr:	517.5 N-m
Curv. Cr:	0.00028038

The cracking moment and curvature values are calculated as prescribed in section 6.1.2.

The additional ICEB gap thickness curvature value is added to each curvature calculated value, by using two inputs: block height and average gap thickness.

Gap Thickness for Dry Stack Joints

Block Height:	100 mm
Average Thickness:	0.1 mm
Additional Curvature:	0.00001

- Typical Output summary.

Summary

Conc. Strain	Curvature (1/mm)	Moment (N-m)	C (mm)
0	0	0	0
Cracked	0.000294	517.50	---
Steel Yeild	0.00011	1332.62	57.22
0.006	0.0001266	1366.46	52.95
0.008	0.0002035	1452.03	42.06
0.01	0.0002871	1526.51	36.53
0.014	0.0004526	1662.94	31.87
0.017	0.0005635	1721.34	30.90
0.019	0.00063	1739.65	30.95
0.02	0.00066	1744.69	31.09

APPENDIX C

Loading, Shear, & Moment Diagrams

Cantilever Walls

Height = 1000 mm

Load Applied at 1000 mm above base

Full Scale Walls

Height = 2400 mm

Four Point Loads Applied at Equal Spacing