SEISMIC DESIGN MANUAL FOR

INTERLOCKING COMPRESSED EARTH BLOCKS

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Abstract

Seismic Design Manual for Interlocking Compressed Earth Blocks

Nicholas Edwards Kennedy

This thesis presents a comprehensive seismic design manual to be used to design and construct simple Interlocking Compressed Earth Block (ICEB) structures in seismically active regions. ICEBs are earth blocks made primarily of soil and stabilized with cement. They have female and male stud mechanisms designed to interlock when stacked, eliminating the need for mortar. The blocks can accept reinforcement and grout after they are placed. While ICEB construction is similar to conventional masonry construction, current design code standards for masonry only partially capture the actual behavior of ICEB structures. This thesis seeks to supplement the existing masonry design procedures and tailor them for use with ICEBs.

Additionally, this paper presents a preliminary design of ICEB shear walls for a disaster reconstruction project in the Philippines. While many structures in Southeast Asia and the Malay Archipelago are constructed from earthen blocks, very few are engineered. Of those that are, a lack of formal design guidance specific to ICEB construction leaves most engineers and designers with conventional concrete masonry design practices, some of which are not applicable for use with ICEBs.

Keywords: Interlocking Compressed Earth Blocks, Seismic Design, Masonry Design

Acknowledgments

This thesis is a summation of efforts by many individuals and organizations, all of whom I would like to thank. Clayton Proto, Brad Stirling, David Bland, Nick Herskedal, and Thomas Bowdey conducted experiments to determine different properties of ICEB systems, and their results and recommendations are mentioned in this manual.

Thank you to Geoffrey Wheeler and the Center for Vocational Building Technology in Udon Thani, Thailand for providing information on the production of ICEBs.

Thank you to Ivan Castro and the students of CE 555 who gave their time and energy to help procure materials and make blocks.

I would like to thank the National Collegiate Inventors and Innovators Alliance for funding a trip to Thailand to investigate the use of ICEBs as a sustainable, cost-effective alternative building material.

I would also like to thank my faculty advisor, Dr. Daniel Jansen, who went with me to Thailand and helped me make observations and collect data on many aspects of ICEBs, including the standard practice of construction and the economic advantages of the material.

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A note on structural seismic design and the use of this manual:

This manual attempts to consolidate published research on the seismic design of interlocking compressed earth block (ICEB) structures. It provides recommendations of engineering methods and equations to approximately quantify the seismic behavior of this material. This manual is based on the Load and Resistance Factor Design methodology and uses metric units. Although all the recommendations are based on the current concrete masonry code and are by nature conservative, they should be used with caution. The test results to date that this manual is based upon were primarily from cyclic pseudo-static loading protocols and not dynamic tests. This loading type was used to gain quantifiable information about the failure modes and failure behavior of ICEBs. These blocks may perform differently or unexpectedly under dynamic type loading. Also, until more tests are performed to supplement and verify the published results, the current pool of information is not sufficient to support codification of standards for ICEBs as a building material.

There is some anecdotal evidence that some existing ICEB structures have performed well in recent earthquakes. This information should be taken with caution. Even poorly designed structures can perform well in certain seismic events due to a number of variables, including fundamental building period, ground motion attenuation relationships, soil profiles, and reinforcement detailing.

Chapter 1: Introduction

In recent years, a need has been identified for a low-cost, sustainable housing alternative to aid in disaster reconstruction in developing countries where material costs are high and human capital is abundant. Interlocking compressed earth blocks (ICEBs) can meet this need. People in Southeast Asia and parts of Indonesia and the Philippines already use ICEBs as a building material, but there is no clear seismic design guidance for engineers and designers. This seismic design manual serves to unify existing research and tailor it for use with the Masonry Society Joint Committee (MSJC) building code (also known as ACI 530), modifying the code recommendations where necessary for use with ICEBs.

The scope of this manual is limited to design recommendations for one story compressed earth block structures in seismic regions. As such, the reinforcement recommendations are tailored for these cases. The requirements for special reinforced masonry shear walls in the 2008 MSJC are modified for use with ICEBs where necessary. These requirements may not apply for structures in low seismic areas.

To demonstrate the use of the seismic design recommendations, a sample design of the lateral earthquake force resisting system of a quadruplex in the Philippines is presented as part of a disaster reconstruction effort.

Chapter 2 summarizes the current research on ICEBs, including material property experimental results and shear wall loading test results. A more in-depth examination of the research results and how they can be applied in design situations is presented in Chapter 4.

Chapter 3 discusses the socioeconomic and environmental sustainability advantages of earthen construction, especially interlocking compressed earth blocks.

Chapter 4 examines current research results and provides modified equations based on the MSJC. Design principles are taken from ASCE 7 as well as seismic design documents such as FEMA P695 among others.

A discussion of seismic design criteria for the Philippines is presented in Chapter 5, with probabilistic seismic hazard data for the region provided by the United States Geological Survey.

Chapter 6 details the design calculations and drawings for the quadruplex, with supporting calculations in the Appendix.

Chapter 7 concludes the manual and identifies future research topics from a practical design perspective with the goal of eventual institutional acceptance of ICEBs and codification of design standards.

Chapter 2: Literature Review

This thesis unifies the knowledge that has been gathered recently at California Polytechnic State University concerning the seismic performance of ICEB structures. This includes the material properties of ICEB's, their flexural and shear performance in lateral load resisting walls, the strength of rebar lap splices, grout strength, and their flexural performance in out of plane loading cases. Additionally, there have been some papers written by other individuals concerning ICEB block deterioration over time, ICEB construction, and ICEB testing and production standards.

Large Scale Structural Performance Tests

Bland (2011) and **Stirling (2011)** authored companion theses summarizing test results for in-plane cyclic shear strength and in-plane cyclic flexural strength, respectively, of ICEB shear walls. Bland developed a stress strain curve for ICEB block and grout prisms that matched a modified Hognestad model. He used a grout mix with experimentally determined proportions of cement, lime, water, and sand to make it fluid and workable. Bland cast the grout into plastic test cylinders (non-porous samples) and also into spare blocks (porous samples) to study the effects of moisture absorption on the grout compressive strength. He found that grouted ICEB prisms had an average compressive strength of 3.0 MPa, the non-porous grout samples had an average compressive strength of 5.1 MPa, and the porous grout samples had an average compressive strength of 9.2 MPa.

Bland tested three 1.8 meter by 1.8 meter walls under cyclic in-plane loading. He did not put horizontal shear reinforcement in the first two walls with the intention of causing a shear failure, characterized by stiffness degradation and post-peak strength loss. His first wall was partially grouted, the second wall was fully grouted, and the third was fully grouted and reinforced with (3) #3 reinforcing bars intended to increase shear capacity to reach a flexural failure. Walls 1 and 2 exhibited shear failures characterized by diagonal cracking. The third wall experienced 20 mm of displacement (1.11% drift) initially due to an input error, resisting a maximum of about 50 kN during the loading. This caused a local failure at the top of the wall consistent with life safety damage. This was instructive based on ASCE 7-05 code drift limits for masonry shear walls of 0.7% (Table 12.2-1). The wall was flexible enough to sustain significantly more than code allowed drift while sustaining only local life safety damage.

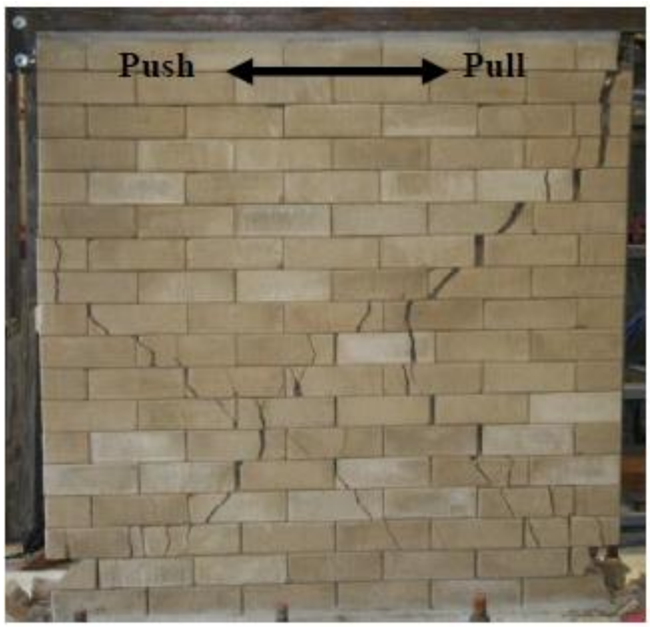

Figure 1: Wall 2 shear failure characterized by diagonal cracking and sliding displacement (Bland 2011)

Bland ultimately found that the steel reinforcing increased the lateral force capacity of the wall but the capacity of the masonry was much smaller than anticipated by the current masonry code. Bland modified the Masonry Standards Joint Committee (MSJC) equation 3-23 to reduce the contribution of ICEB masonry to shear strength. He theorized that the solid grout cores provided resistance to shearing motion but that the dry stack interlocking action did little to contribute to shear strength, unlike the block interface in regular CMU construction. Conventional masonry is assumed to provide shearing resistance in proportion to the net mortared cross sectional area. Bland also found that, as load increased, most of the displacement of the wall was primarily due to shear displacement and sliding between the block interfaces.

In the companion investigation, Stirling tested three walls, each of differing dimensions and features. The first wall was 0.9 meter by 1.8 meter, half the aspect ratio of Bland's third wall. Stirling's goal was to investigate the effect of aspect ratio on lateral in-plane performance. This wall failed in flexure as anticipated, taking a maximum of 13.2 kN in the pull direction and 14.5 kN in the push direction. The results showed that the current concrete masonry code (MSJC) accurately predicts the flexural capacity of reinforced ICEB walls.

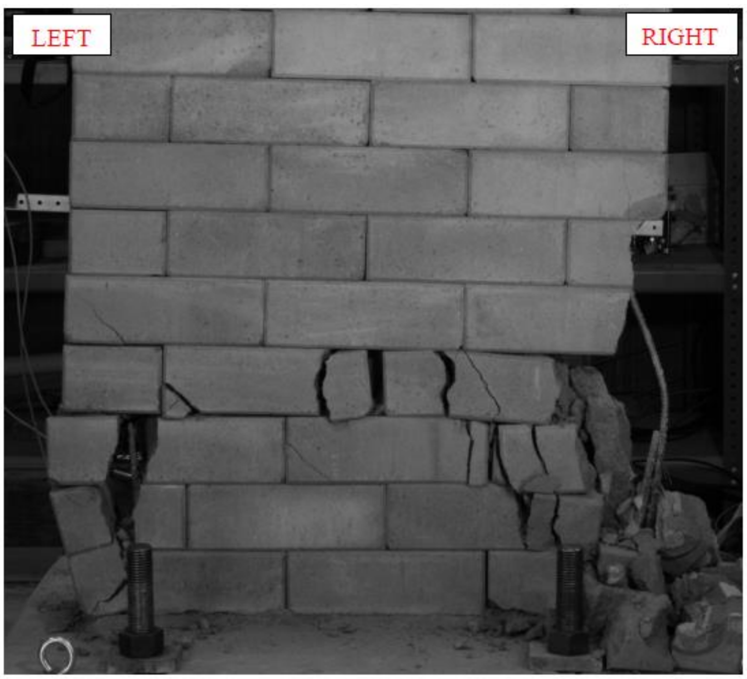

Figure 2: Wall 4 flexural failure characterized by spalling at the wall toe due to buckling of the vertical rebar (Stirling 2011)

The next wall was 1.8 meter by 1.8 meter with a 0.75 meter flange at one end to investigate strength contributions from flanges. The flange in tension exhibited approximately the same displacement ductility (defined as ultimate displacement over yield displacement) as the wall without the flange, but the flange in compression increased the displacement ductility by about 20%. Stirling tested one more wall, a 1.8 meter by 1.8 meter wall with a 0.9 meter by 0.9 meter opening in the center to investigate the formation of plastic hinges and the mechanism of shear and moment transfer through the opening. Stirling found that the development of the plastic hinge at each joint occurred at about 75% of the predicted plastic moment capacity using conventional reinforced concrete methods.

Herskedal (2012) investigated the out of plane flexural strength of ICEB walls and introduced pilaster requirements for out of plane stiffness. Herskedal found that ICEB walls are far more flexible than conventional concrete masonry, so he used the deformation limit of 0.7% found in ASCE 7-05 12.12.1 to determine the required stiffness. Adding stiffening elements such as pilasters increased the out of plane stiffness by over 16 times that of walls without stiffeners. Also, in seismically active areas, Herskedal recommended pilasters should be spaced no farther than 3.0 to 3.5 meters. Herskedal also found that plastering ICEB walls reduces some of the inherent out of plane rotation due to gaps in ICEB walls created by the bottom chamfers and variable geometry of the blocks. In some cases, this reduction in rotation translated to up to a 75% decrease in nominal displacement relative to unplastered walls.

Figure 3: Out of plane rotation of an unplastered wall due to gaps in blocks (Herskedal 2012)

Proto et. al (2010) assembled an “ICEB Design and Construction Manual” as part of a senior project at California Polytechnic State University, San Luis Obispo. It included geotechnical soil analyses, block production procedures and recommendations, general testing and quality control techniques, and simplified seismic design procedures. This manual provided the basis for the soil mixes and block production techniques used in later ICEB experiments. Proto and his team developed a drop test to determine soil mix suitability for pressing and a pocket penetrometer test to test the strength of freshly pressed blocks. They referenced information from previous studies, including target block compressive strength (Reddy and Gupta, 2005), block production statistics (Houben and Guillard, 1994), and block soil composition and clay content (Venkatarama et. al, 2007 and Burroughs 2006). According to Proto et. al, soil used to produce ICEB’s should meet several criteria, including linear shrinkage limits, minimum percentage of sand (65%) and recommended clay content (15%), maximum percentage of gravel (10%) and minimum percentage fines content (15%), medium plasticity and toughness, and no organic matter.

Kerali (2004) performed a study on the durability and deterioration rate of ICEB structures exposed to weather. He investigated two structures on the outskirts of Kampala, the capital of Uganda. Both had been abandoned without roofing for over 8 years. He identified two main categories of durability concerns: surface cracking and erosion. At the time of production, the blocks had an average compressive strength of 4.2 MPa, clay content of 12%, silt content of 12%, sand content of 68%, gravel content of 8%, cement content of 5%, and water absorption of 10.3%. He found a maximum of about 39% mean volume reduction in the two studied buildings due to rainwater exposure. Kerali determined that block elevation, block orientation on the façade, and age of the building all contributed to the loss in volume of the blocks. Blocks lower in the façade tended to lose more volume due to rainwater running down the walls.
Blocks on the east and west elevations tended to lose more volume for exposure reasons, and the fact that each building had been exposed to the elements for 8 years showed a level of deterioration consistent with this exposure.

Reddy and Gupta (2005) conducted experiments with blocks comprised of sandy soils to determine the effect of cement content on material properties. Reddy and Gupta used manually pressed non interlocking blocks that were 305 mm x 143 mm x 100 mm. The soil mix had 9% clay, 17.7% silt, and 73.3% sand. They tested blocks made with three cement contents: 6%, 8%, and 12% cement by weight for wet compressive strength, rate of water absorption, flexural strength, tensile strength, pore size, and stress-strain behavior. Of particular interest is the test for pore size. The authors obtained scanning electron microscopy images of block samples and used the magnified images to estimate pore size and porosity (taken as pore size area over whole area of image). They found that cement content influences the size of pores, with the 6% cement sample having larger pores than the 8% and 12% samples. However, the porosity ratio remained the same for all three cement contents because the blocks with higher cement contents have more pores that are smaller. Also, Reddy and Gupta found that wet compressive strength, flexural strength, and direct tensile strength increase with higher cement contents. The wet compressive strength results are shown in Figure 4:

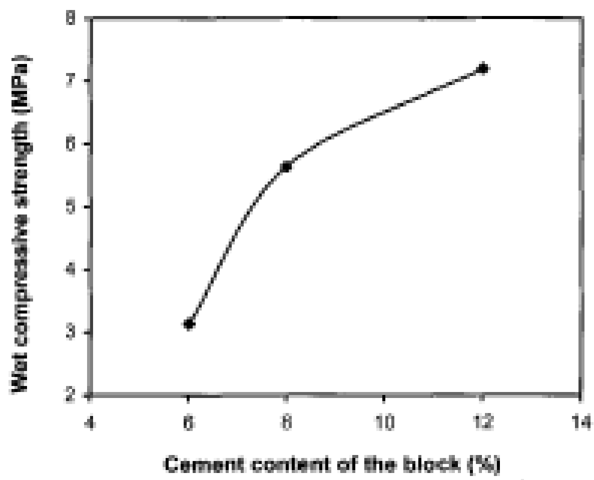

Figure 4: Wet compressive strength varying with cement content (Reddy & Gupta 2005)

The total water absorption did not depend on the cement content, with all three samples absorbing about 11-12% moisture during the experiments. The initial rate of absorption, however, is directly tied to cement content and decreases four times when doubling cement content from 6% to 12%.

The modulus of rupture of the blocks was found to be between 15 to 18% of the block compressive strength. This contrasts with the modulus of rupture of concrete masonry blocks as approximately 2.5% of compressive strength (based on Table 3.1.8.2 in the Building Code Requirements for Masonry Structures (ACI 530) and Table 2 in the Specification for Masonry Structures (ACI 530.1). The axial tensile strength was found to be between 5 and 6% of the block compressive strength, which contrasts with the axial tensile strength of concrete masonry as about 9.5% of compressive strength (based on Table 2 in the Specification for Masonry Structures (ACI 530.1) and concentric axial tensile tests performed by Hamid and Drysdale (1982).

Each block failed at about 0.003 strain, which is consistent with the strain limits imposed by the MSJC Section 3.3.2 (c) of 0.0025 strain. However, this limit is for mortared hollow masonry prisms, not for individual blocks as tested by Reddy and Gupta. Bland (2011) tested grouted prisms with interlocking rhino blocks of slightly different dimensions than Reddy and Gupta (300 mm x 150 mm x 100 mm) and found much more ductile strain limits, with prisms failing around 0.012 strain. The addition of grout and mortar increased the deformation capabilities of the prism specimens. The results of Reddy and Gupta's material properties study are summarized below in Table 1:

Table 1: Material properties test results (Reddy and Gupta 2005)

Block characteristics		Block designation		
		SCB1	SCB2	SCB3
Cement content (%) (by weight)		6	8	12
Compressive strength (MPa)	Mean value	3.13	5.63	7.19
	COV* (%)	16.2	10.2	9.6
	No. of specimen	20	20	20
Flexural strength (MPa)	Mean value	0.48	1.05	1.22
	Range	0.39-0.65	0.82-1.19	1.07-1.31
	No. of specimen	6	6	6
Tensile strength (Mpa)	Mean value	0.18	0.29	0.46
	Range	0.17-0.22	0.22-0.39	0.36-0.55
	No. of specimen	6	6	6
Initial Rate of Absorption (IRA) (kg/m^3/minute)	Mean value	6.5	4.9	1.6
	Range	4.2-8.5	3.2-7.0	1.3-1.8
	No. of specimen	6	6	6
Water Absorption (%)	Mean value	12.1	11.2	11.4
	Range	11.5-12.9	10.1-12.0	10.3-12.3
	No. of specimen	6	6	6

*Coefficient of variation

Reddy et. al (2007) performed experiments investigating the effect of soil grading, specifically clay content, on the characteristics of soil-cement blocks. They tested 305 mm x 143 mm x 96 mm blocks similar to the previous study on cement content. They used two cement contents, 4% and 8%. Compressive strength and flexural tensile strength tended toward a maximum at clay contents between 14 and 16% regardless of cement content as shown in Figure 5:

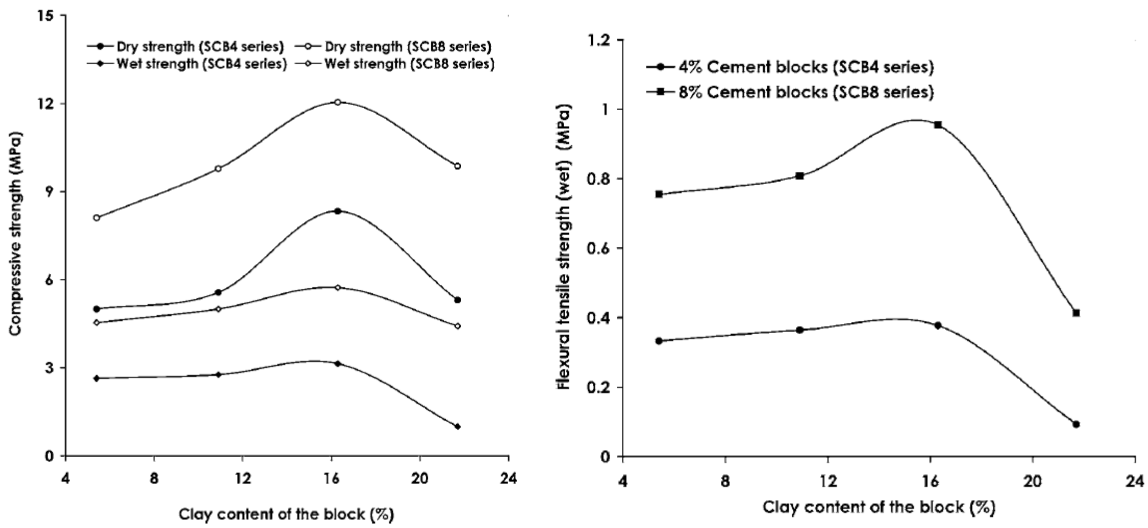

Figure 5: Maximum compressive and flexural strength at 14-16% clay content (Reddy et. al 2007)

Linear expansion and water absorption also increased linearly with clay content, as shown in Figure 6:

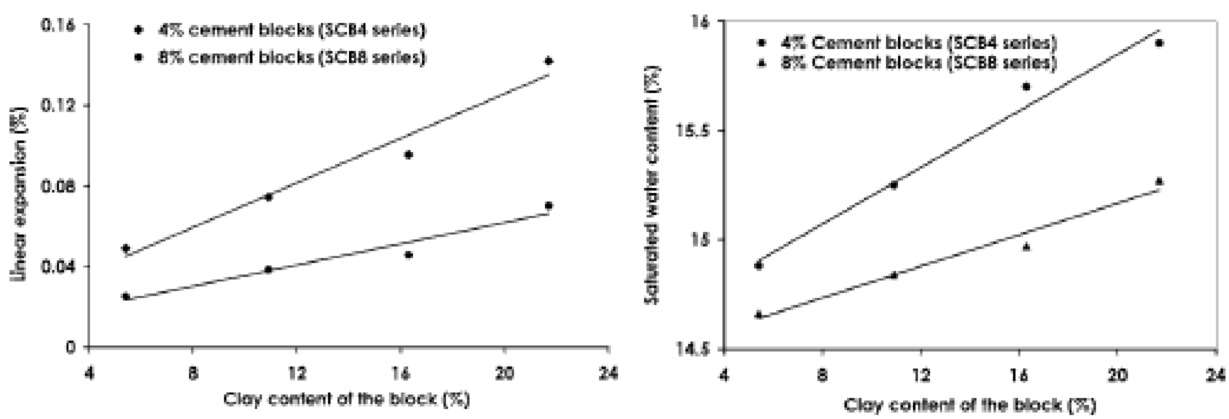

Figure 6: Linear expansion and water absorption increase with clay percentage (Reddy et. al 2007)

Construction Manuals and ICEB Standards

Adam and Agib (2001) authored a construction manual entitled “Compressed Stabilised Earth Block Manufacture in Sudan” as part of a larger project with the United Nations Educational, Scientific, and Cultural Organization (UNESCO). The UNESCO project, “Improvement of Educational Facilities in the Least Developed Countries of the Arab States”, was to construct a school in the Khartoum region of Sudan to demonstrate the potential of earth as a building material. Adam and Agib discussed the advantages and limitations of compressed earth block construction, noting that compressive strengths of the blocks can vary immensely (between 1 to 4 MPa) based on stabilizer content and as a consequence, buildings should be limited to one storey. However, they found that Sudanese black cotton soil with hydrated calcium lime as a stabilizer can produce blocks with compressive strengths of up to 8 MPa. The minimum British Standard requirement for precast concrete masonry units is 2.8 MPa, so compressed earth blocks compare favorably to conventional masonry units. For low income housing in rural areas, compressed earth blocks are ideal because they reduce material costs and they use local labor, which stimulates the economy. The blocks are fire resistant, have low environmental impact, and require very little equipment to manufacture. Their disadvantages are low resistance to impact and abrasion, reduced durability, and low tensile strength.

Adam and Agib assembled an exhaustive list of tests after Houben and Guillard (1994) to determine soil suitability for use in compressed earth blocks. This includes the smell test, nibble test, touch test, sedimentation test, wash test, linear shrinkage mold test, adhesion test, and dry strength test, to name a few. They also discuss soil stabilizing agents from cement to lime to bitumen, and they point out that the idea of soil stabilization has been around for many years, starting in the 1920’s in road construction.

Finally, the construction manual lists techniques for preparing and mixing soil for compressed earth block production, and it discusses methods and machines used to mold and compress the blocks. The authors also discuss mortar mixtures pertaining to compressed earth blocks.

In the back of the manual, the authors include the African Regional Standards for Compressed Earth Blocks, which are based on the Guide to Compressed Earth Block Standards (1998), more fully reviewed below. Of particular interest in this manual is the inclusion of several standard test procedures to determine several properties of interest pertaining to ICEB production and construction, including a soil gradation section, dry and wet compressive tests on hardened blocks, abrasion tests, and capillary absorption tests.

The **Guide to Compressed Earth Block Standards (1998)** is a comprehensive document that was created with the support of many different organizations in an effort to standardize compressed earth block construction. The main goal of the manual was to standardize earth block construction in three main regions: Africa, Caribbean, and Pacific (ACP) countries. The UK-based Centre for the Development of Industry (CDI) in partnership with enterprises in the European Union funded the project and consulted with the International Centre for Earth Construction, part of the School of Architecture in Grenoble, and the African Regional Organization for Standardization (ARSO) to gather the technical information in the manual. This represents the first standardization manual in the field of compressed earth block technology.

Although humans have been building with earthen materials for thousands of years, the idea of non-fired earth that has been mechanically compressed is relatively new. In the late 1700s, a French architect by the name of François Cointeraux invented a press for manually ramming earth into blocks, thereby creating the field of rammed earth construction. This particular manual acknowledges a breakthrough in compressed earth block technology appeared in 1952 in Bogota, Columbia. Raul Ramirez, an engineer at Bogota's CINVA center, invented the first manual CINVA-RAM. This helped spur startup enterprises using compressed earth block technology in countries all over the world, especially in Latin America and Africa. In 1983, the Thailand Institute of Scientific and Technological Research (TISTR) developed an interlocking block with top and bottom studs. Soon after, the Asian Institute of Technology in Pathum Thani, Thailand and the Soil Block Development Company in Chiang Rai, Thailand developed a larger interlocking block called the rhino block. Both are used today (Wheeler, 2005).

The manual has categorized compressed earth blocks in general shapes, principal dimensions, and nature of indentations into groups called primary formats. The most common primary format is a rectangular parallelepiped shape with length, width, and height. Secondary format shapes are fractions or multiples of the primary formats. The main multiples are ¾, ½, and ¼.

Compressed earth blocks are further categorized by type number. Type 1 is a full rectangular parallelepiped format with no indentations. Type 2 is a full rectangular parallelepiped with one or more indentations on one or both of the bed or laying surfaces (bottom and top). Type 3 is a full rectangular parallelepiped with one or more indentations on the stretcher or header (front or side faces) or on multiple faces. Type 4 is a rectangular parallelepiped with holes between the top and bottom faces. Perforated blocks are Type 4 blocks with a few small holes, hollow blocks are Type 4 with a few large holes, and alveolar blocks have many small holes. Type 5 blocks are commonly called channel blocks; they have indentations and holes in their top or bottom faces. Type 6 blocks are rectangular parallelepipeds with holes in the top and bottom faces and with indentations in the front and side faces.

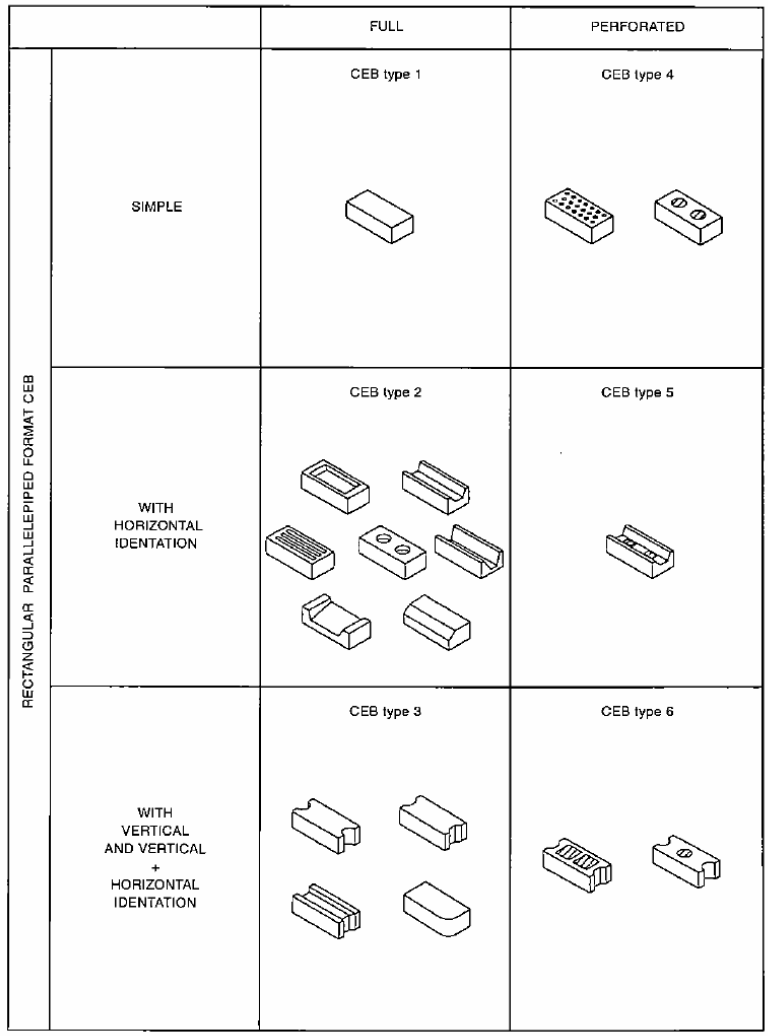

Figure 7: The 6 types of compressed earth blocks (1998 Standards)

It is important to note that the modern version of the CINVA block, the rhino block, is a Type 6 compressed earth block with two hollow interlocking “studs” on the top and bottom faces to allow blocks to be dry stacked and interlocked together (Wheeler, 2005). The rhino block also has grout channel keys on the header faces to fill with grout once the blocks have been laid.

Wheeler (2005) authored a construction manual specifically aimed toward CINVA rhino blocks made with the Soeng Thai BP6 block press as shown in Figure 8. This particular press can create nine different types of blocks.

Figure 8: Soeng Thai BP6 block press with 9 different block types (Wheeler 2005)

This manual describes techniques to lay out concrete or cyclopean foundations with cast in place vertical reinforcement to thread blocks over once the concrete has hardened. It also describes leveling techniques with nails and the importance ensuring walls are plumb. As compressed earth blocks are inherently variable, it is important to check the geometry of the wall early and often. Wheeler presents drawing details for the anchorage of a second floor slab into the first floor walls, but at this time there is no testing research to verify the seismic capacity of two storey and higher ICEB structures.

Wheeler also recognizes the importance of waterproofing compressed earth block structures. He recommends installing a capillary rise barrier under the first course of blocks to prevent water from rising from the foundation into the walls. He also recommends using long roof overhangs and drip strips above windows and doors to prevent water entry. Walls should be plastered or glazed with breathable sealants to prevent moisture entry. Glazed walls should have waterproofing plaster at least 2 courses high above the foundation.

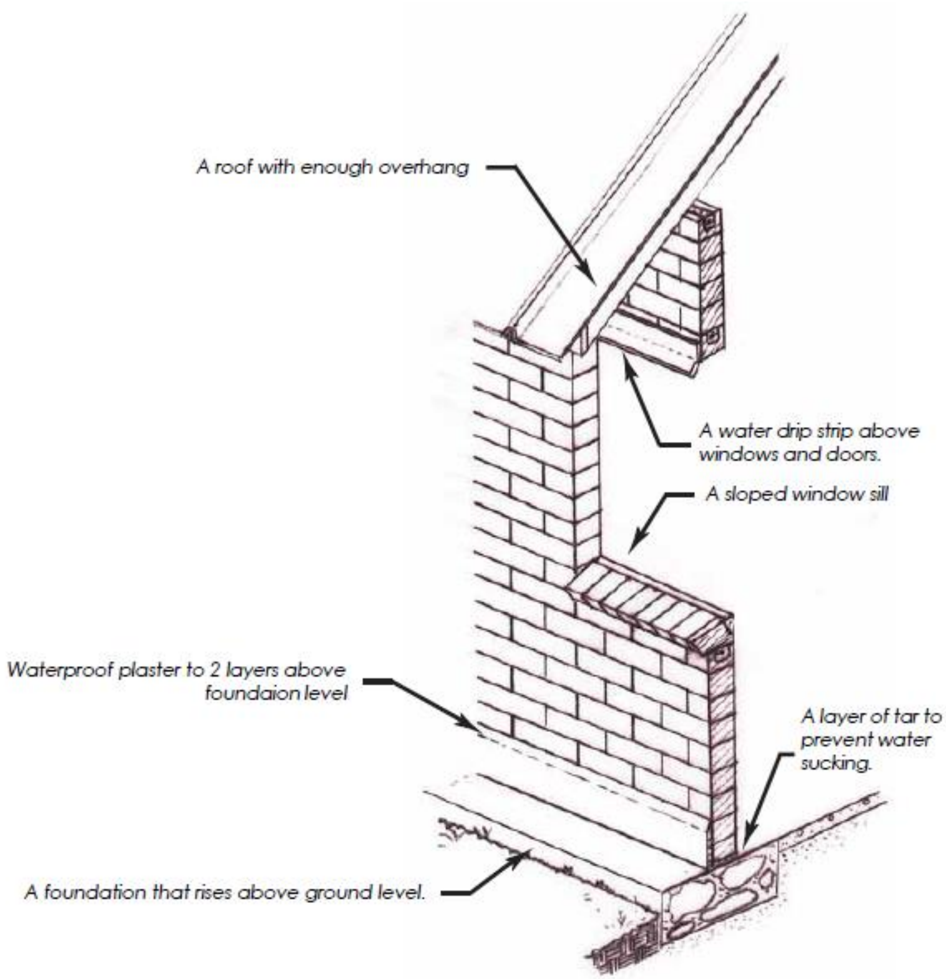

Figure 9: Waterproofing details (Wheeler 2005)

Compressed earth block homes are capable of supporting electrical wiring and plumbing systems. The manual shows how to place PVC pipe wire sleeves in empty grout holes and cut openings in blocks for switch boxes and plugs. The same concept applies for plumbing lines. Cells with electrical wiring or plumbing should not be grouted to facilitation replacement of faulty or aging components.

Chapter 3: ICEB Economic and Sustainability Factors

Earth as a building material lacks institutional acceptance in many parts of the world. However, the cost savings and low environmental impact of ICEBs warrant an in-depth socioeconomic and sustainability analysis to support their increased use worldwide. This chapter is based on the results of an economic observation and survey that was performed in rural Thailand at the Center for Vocational Building Technology in September of 2012. In rural Thailand and many other developing areas, the scarcity of construction grade timber makes it impractical as a building material. Reinforced concrete, clay fired bricks, and concrete masonry are more common options, but the high cost of cement makes these unreachable for many families. Compressed earth block masonry has many advantages over conventional materials. The most commonly cited advantage is the cost savings. The compressed earth block uses between half and three-quarters the amount of cement of conventional concrete masonry, making it a more affordable (and environmentally sustainable) option.

The compressed earth block economic model hinges on the relative costs of material and labor. In developing countries, labor accounts for a fraction of the cost of materials. Human capital is abundant, especially during the off months of the harvesting season. Part-time farmers sometimes move to the city during the offseason dry months, leaving their families for long periods. Labor intensive processes such as compressed earth block construction enable villagers to build housing and other infrastructure during a time when they would normally leave the area. Table 2 shows labor requirements based on Adam and Agib's Al Haj Yousif school in the Khartoum area of Sudan. High labor requirements create jobs while saving energy and expensive materials.

Table 2: Labor requirement comparisons (Adam and Agib 2001)

Products	Production method	Labour needed to make volume equivalent to 240 blocks/day
Compressed stabilised earth blocks	Brekpak press	6.00
Fired clay bricks	Traditional manual	2.50
	Intermediate technology	3.00
	Soft mud machine manual	1.00
	Moderately mechanised	0.33

The Al Haj Yousif prototype school building project in Sudan realized a cost per square meter savings of 40% compared to conventional masonry construction. The reduction in cost of the actual blocks was 70% compared to concrete masonry units.

In the rural area of Ban Then, Thailand, about 15 km north of Udon Thani, a 40 square meter, one story compressed earth block house complete with architectural finishings, indoor plumbing, and electricity can be built for about $3,000 total. This is based on an exchange rate of approximately 30 Thai baht per dollar at the time of the writing of this thesis. The construction estimate and plans can be found in Appendix D. The total skilled and unskilled labor to build the house accounted for about $220 of the total cost, while material costs accounted for about $2100. The ICEB’s and rebar cost $660 out of the $2100 for materials.

These cost analyses are based on the rhino block created with the Soeng Thai block press, a manual press that can be used to make up to 300 blocks per day with a skilled team of 4 laborers. To create a block producing enterprise, entrepreneurs need only to invest $2000 in a block press and $1000 in a soil pulverizer, plus the cost of cement and sand for the first few batches. A laborer in rural Thailand can expect to make $7-8 per day producing blocks. Blocks can also be purchased from local producers such as the Center for Vocational Building Technology for around 12¢ per block.

In some areas of Thailand, local entrepreneurs have invested in hydraulic block presses that manufacture thousands of blocks per month (one claimed to produce around 95,000 per month with laborers working around the clock). These hydraulic blocks are about three quarters of the size of the conventional rhino block and are of a slightly different interlocking design. They are mostly used in garden walls and other architectural applications, although some structures have been built with them.

Raw materials such as timber are difficult to obtain in Thailand and throughout Southeast Asia because of the depletion of forests long ago. However, cement production is on the rise (World Cement Magazine). Thai cement is known for its high quality and low price, so this makes compressed earth blocks using local cement a sustainable option. However, the production of cement is itself not environmentally friendly, so the fact that compressed earth blocks use up to half the amount of cement of regular concrete or concrete masonry adds to their appeal. The Desert Architecture Unit has estimated that the embodied energy in one cubic meter of soil is 36 MJ (10 kwh) while the embodied energy in one cubic meter of concrete is 3000 MJ (833 kwh) (Adam and Agib 2001). Also, the energy required for concrete production is primarily fossil fuel based, while the energy required for compressed earth block manufacture is made up largely of human energy when using manual methods of production, the only methods available in some remote areas of the world.

Chapter 4: ICEB Design Recommendations

Material Properties

Interlocking compressed earth blocks derive their strength from the amount of stabilizer present in the mix. Some common stabilizers include cement, lime, bitumen, pozzolanas, and gypsum. The different stabilizers have different advantages and disadvantages based on the sand and clay content of the soil mix (Adam and Agib 2001). The compressive strength of ICEB masonry, f'_{ICEB}, depends heavily on the amount of cement in the mix. Bland (2011) performed compressive tests to find the modulus of elasticity of ICEB masonry. It was found that the modulus of elasticity E_{ICEB} is 157 times the compressive strength f'_{ICEB} rather than 700 f'm for clay masonry or 900 f'm for concrete masonry as found in the 2008 MSJC Section 1.8.2.2.1. This increased flexibility is evident in the deflection behavior exhibited in ICEB shear walls, which can experience displacement ductilities (defined as ultimate displacement divided by yield displacement) up to 6.0 (Stirling 2011).

Bland performed strain controlled compression tests on ICEB prisms with f'_m = 3.0 MPa, Bland fit a modified Hognestad model to the data using ε_0 =0.012 (strain at peak stress) and ε_{cu} =0.025 (maximum usable strain) as shown in Figure 10: Modified Hognestad Model (Bland 2011). However, during tests, the ICEB walls began to spall on the compression side long before ε=0.012 because the compression rebar buckled. See the Flexural Capacity of Walls section for more discussion on compressive strain limits in ICEB walls.

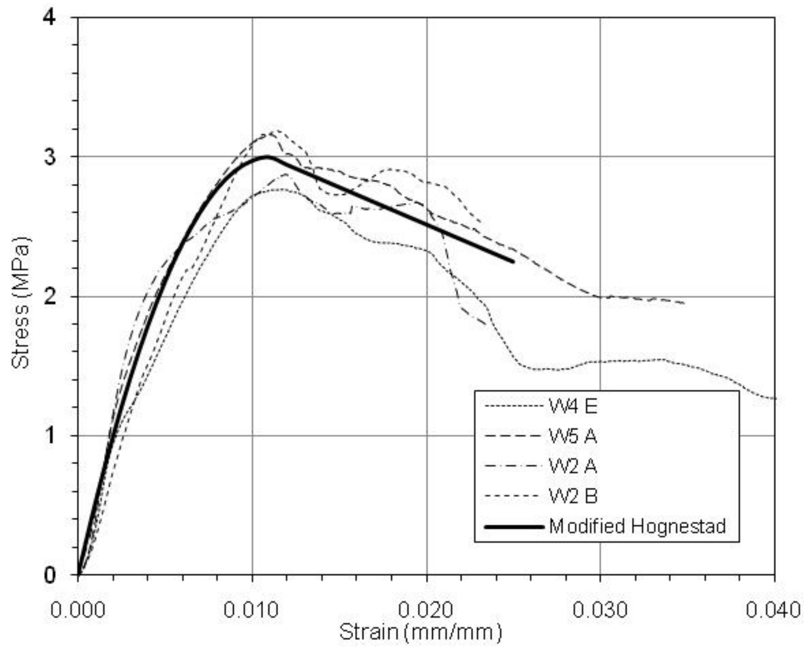

Figure 10: Modified Hognestad Model (Bland 2011)

Shear Capacity of Walls

The 2008 MSJC stipulates that masonry nominal shear strength be calculated using the following equation (MSJC Equation 3-19):

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

The contribution of the masonry (in metric units) is computed using MSJC Equation 3-22:

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

where M_u is the ultimate factored moment acting on the base of the wall in N-mm, V_u is the ultimate factored shear force acting on the wall in N, d_v is the shear depth of the wall in mm, A_n is the net cross sectional wall area in mm^2, f'_m is the prism compressive strength of the masonry in MPa, and P is the applied axial load on the wall in N. The quantity M_u/ V_u is known as the shear span. If the wall is significantly longer than it is tall, i.e. (M_u/ $V_u d_v$) ≤ 0.25, the total nominal shear strength V_n (in metric units) is limited by MSJC Equation 3-20:

$$V_n \leq 0.5 A_n \sqrt{f'_m}$$

If the wall is significantly taller than it is long, i.e. ((M_u/ $V_u d_v$) ≥ 1.00, the total nominal shear strength V_n (in metric units) is limited by Equation 3-21:

$$V_n \leq 0.33 A_n \sqrt{f'_m}$$

These limitations serve to protect against brittle shear failure. The contribution of the steel shear reinforcement, V_{ns}, is defined as (MSJC Equation 3-23):

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

where A_v is the area of shear reinforcement, s is the spacing in the vertical direction of the horizontal reinforcing steel, f_y is the yield strength of the shear reinforcement, and d_v is the depth from the outermost compressive fiber to the furthest tension flexural reinforcing bar. This equation has been empirically derived from

Recent research by Bland suggests using a corrected net cross sectional area A_n modified by correction factors of 0.2 for partially grouted walls and 0.4 for fully grouted walls. This is in addition to the capacity reduction factor φ (typically 0.8) applied to the sum of the masonry and shear reinforcement shear capacities. The modified equation becomes:

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

k = 0.20 for partially grouted walls

= 0.40 for fully grouted walls

Partially grouted walls mean grouting only the grout keys between blocks, all horizontal reinforcement channels, and reinforcement cells with longitudinal or vertical reinforcement (see Figure 11). Fully grouted walls have grout in every cavity.

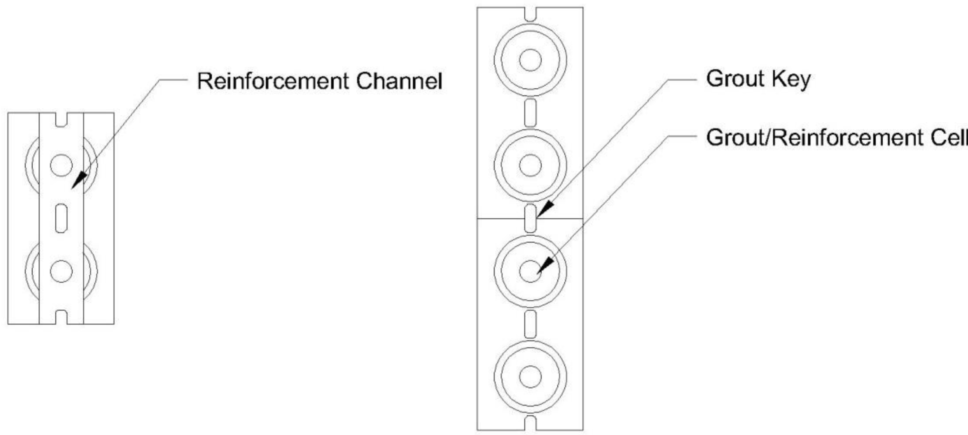

Figure 11: Grout Key, Grout Cell, and Reinforcement Channel Definition

These equations illustrate the beneficial effect axial loads can have on shear capacity. For conventional masonry, aggregate interlock is thought to improve with increased axial load (Brandow et al. 2011). It is not known whether the same phenomenon would occur for ICEB masonry. In ICEB structures, which are often single story in seismic zones, axial loads are quite low and the 0.25P term can be neglected.

The contribution of transverse steel reinforcement is calculated using the same equation as for conventional masonry. The modified total nominal shear capacity is now then:

$$V_{n'} = V_{nICEB} + V_{ns}$$

The $V_{n'}$ term indicates the nominal shear capacity of an ICEB wall to prevent confusion with V_n, the nominal shear capacity of a conventional masonry wall.

Concrete ring beams (or bond beams) should be installed at the top of shear walls to help transfer diaphragm loads into the shear walls and to provide shear continuity throughout the lateral force resisting system. They should be designed to transfer axial chord forces from the diaphragm into the shear walls. These ring beams should be deep enough to provide proper development of bars for roof anchorage. Design guidance can be found in Appendix D of ACI 318-08.

According to Bland, partially grouted walls have about 60% of the shear capacity of fully grouted walls. ICEB walls are thought to resist shear primarily through the continuous grouted core since they lack mortar. The interlocking studs add an additional shear resistance mechanism but this is considered to be smaller than the grouted core resistance. Shear strength of partially grouted walls traditionally considers only the contribution of the face shell thickness (Voon and Ingham 2006), but observed cracking patterns show this not to be the case with ICEBs. For design purposes, shear areas of partially grouted walls should be reduced by 80%, which will decrease the total shear capacity by about 60%.

Transverse shear reinforcement should be hooked 180° around the last longitudinal reinforcing bar in a shear wall as shown in Figure . The contribution of transverse steel remains the same as for conventional masonry. It is especially important to provide sufficient reinforcement to capacity protect against brittle shear failure. This can be accomplished by using the maximum probable moment M_{pr} to calculate design shear forces instead of designing based on the theoretical shear force resisted by the lateral force resisting system as calculated using simplified code procedures.

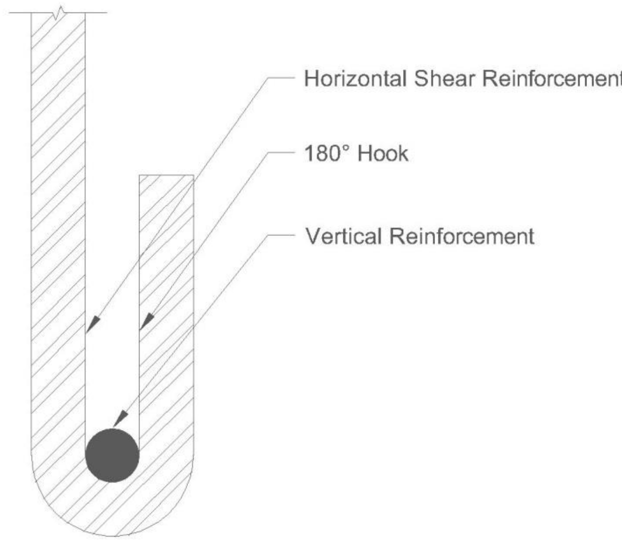

Figure 12: Horizontal Shear Reinforcement 180° Hook

The seismic design provisions of ACI 318-08 recommend calculating seismic shear demands based on the moment capacity of the member rather than the expected shears calculated from the main lateral force resisting system, provided the seismically induced shears are larger than the expected lateral shears. This is to capacity protect shear walls from brittle shear failure and allow them to reach ductile flexural deformation before ultimate failure. Bland and Stirling found that ICEB wall specimens that had transverse shear reinforcement designed to force flexural dominated behavior experienced higher deformations and ductility than wall specimens designed with minimum shear reinforcement. ACI 318-08 and the 2008 MSJC (1.17.3.2.6.1.1) require that reinforcing steel have a minimum overstrength factor of 1.25 when designing using LRFD methods. The maximum probable moment on the system, M_{pr}, can be calculated as:

$$M_{pr} = \frac{M_u(F.S.)(1.25)}{\Phi}$$

where M_u is the moment demand on the ICEB shear wall calculated from the lateral force resisting system demands, F.S. is the desired safety factor (typically 1.1 for this application) and φ is the flexural reduction factor, typically 0.9. Therefore, the seismic shear demand V_e on an ICEB shear wall can be calculated as:

$$V_e = \frac{M_{pr}}{h_w}$$

Flexural Capacity of Walls

The determination of the flexural capacity of a traditional masonry wall is governed by a few assumptions as listed in section 3.3.2 in the 2008 MSJC. Some of the assumptions that particularly apply to ICEBs are:

1. Strain continuity exists between reinforcement, grout, and masonry. Recent research suggests that certain low strength grout mixes may not bond well with the surrounding masonry.
2. The maximum usable strain, ε_{mu}, at the extreme masonry compression fiber shall be assumed to be 0.0025 for concrete masonry. It is difficult to quantify the amount of lateral restraint the unconfined ICEB blocks provide for the compression reinforcement, and the current masonry code assumes an empirically determined compressive limit strain of ε=0.0025. For this reason, the compressive strain limit of ε=0.0025 of masonry apply to ICEB construction as well even though ICEB prisms exhibit much higher ultimate strains.
3. Strains in reinforcement and masonry shall be assumed to be directly proportional to the distance from the neutral axis (plane sections remain plane).
4. Steel does not exhibit strain hardening properties, i.e. usable stress is limited to f_y. Testing by Bland and Stirling showed that ICEB shear walls exhibit high displacement ductility when strains exceed ε_y but this factor should neglected for

elastic seismic design of ordinary reinforced ICEB walls because of the uncertainty associated with the material.

5. Tensile strength of masonry shall be neglected when calculating flexural capacity but considered when calculating deflections.
6. Masonry compressive stress of $0.80f'_m$ shall be assumed uniformly distributed over an equivalent stress block bounded by the cross section limits and extending to a depth a = 0.80c where c is the depth of the neutral axis.

Bland and Stirling have suggested that ICEB walls exhibit flexural performance similar to that of conventional concrete masonry. As with conventional masonry, the compression side is not confined. This implies that ductility and system overstrength come entirely from the reinforcing steel. Based on static pushover analyses using the computer program XTRACT (Chadwell, 2004) on sample ICEB shear walls, ordinary reinforced ICEB walls should be considered to have a response modification factor R of 2.0. Response modification factors greater than 2.0 require special reinforcement detailing. See Appendix A for supporting calculations based on test results after Stirling (2011) and Bland (2011). The Federal Emergency Management Agency (FEMA) document P695 estimates the R factor using trial R factors of building systems in a computer model with incremental nonlinear dynamic analysis to determine the probability of local collapse or global instability. These simulations within FEMA P695 take six different deterioration or failure modes into account: flexural hinging of beam-column elements, column compressive failure due to amplified axial loads from earthquake induced moments, beam-column shear failure, joint shear failure, pull out and bond slip of rebar, and failure due to punching shear in the slab-column connection. This is far more comprehensive than simple flexural failure as was the goal in Stirling's tests. For this reason, the R-factors generated using Stirling's test results must be used with caution.

ICEB walls in seismic regions should be designed analogously to intermediate and special reinforced conventional masonry walls. ICEB specific requirements are as follows:

1. A maximum longitudinal steel reinforcing ratio ρ_l defined as:

$$\rho_l = \frac{A_{Sv}}{bd}$$

where b is the width of the wall in mm, and d is the depth to from the outermost compression fiber to the centroid of the farthest flexural (longitudinal) steel. A_{sv} is the total area of flexural steel in the wall. To achieve the required ductility for most ICEB shear walls with minimum grade steel (10 mm Gr 30 (206 MPa), ρ is limited by:

$$0.0007 \leq \rho_l \leq 0.007$$

This limit is to achieve axial equilibrium while preserving ductility based on the requirements of Section 3.3.3.5.1 of the 2008 MSJC. The maximum limit was determined based on an iterative code done in MATLAB based on wall aspect ratios (height to width) greater than or equal to 1.0 with equations found in the commentary on Section 3.3.3.5.1. See Appendix C for detailed calculations. The minimum limit is based on the total area of vertical reinforcement being at least 0.07% of the gross cross sectional area of the wall per Section 1.17.3.2.6 (c) (1). Spacing of longitudinal steel must not exceed 1200 mm for intermediate reinforced shear walls and the least of 33% of the length or height of the wall for special reinforced shear walls. Section 1.17.3.2.3.1 requires vertical (longitudinal) reinforcement of at least 113 mm^2 (12 mm bar) in diameter, however, for ICEB shear walls, 79 mm^2 (10 mm bar) is sufficient provided it is provided in all block cells at corners and ends of walls. Although this requirement is for special reinforced shear walls, by taking the more stringent requirement and applying it to intermediate reinforced shear walls, it is ensured that minimum longitudinal steel requirements are met while making it easier to obtain the necessary materials.

Stirling (2011) suggests that higher aspect ratio shear walls (height to width) have increased ductilities, but more testing should be completed to verify this result.

2. A minimum horizontal steel reinforcing ratio of ρ_h defined as:

$$\rho_h = \frac{A_{sh}}{bh}$$

where A_{sh} is the area of horizontal shear steel in the wall, b is the thickness of the wall, and h is the height of the wall. Per MSJC 1.17.3.2.6 (c) the horizontal steel ratio must be greater than 0.07% of the cross sectional area of the wall:

$$0.0007 \leq \rho_h$$

There is no maximum horizontal steel reinforcing ratio per Section 3.3.6.5. This applies to ICEB walls as well in order to capacity protect against brittle shear failure. The maximum spacing is the smallest of the longitudinal steel spacing and 610 mm for both intermediate and special reinforced ICEB shear walls per Section 1.17.3.2.6. The walls in Bland and Stirling's tests met the minimum horizontal reinforcement requirements.

Wall Deflections

ICEB shear walls should be elastically designed for simplicity and for the reasons described in the Flexural Capacity of Walls section. Deflections should be determined based on elastic analysis but with cracked section properties. Methods for this analysis can be found in most reinforced concrete textbooks.

The deflection amplification factor C_d has been shown to be significantly higher than that of conventional masonry. ICEB masonry is more flexible than conventional masonry, with displacement ductilities of up to 6.0 for 2:1 aspect ratio walls based on results from Stirling (2011). The deflection amplification factor for ordinary masonry shear walls is 1.75, but testing results from Stirling have shown it is different for ordinary reinforced ICEB shear walls. Varela, Tanner, and Klingner (2004) define the deflection amplification factor C_d as:

$$C_d = \frac{\Delta_u}{\Delta_y} \Omega_{system}$$

See Appendix A for calculations on determining the deflection amplification factor C_d. Based on test results, for 1:1 aspect ratio walls, C_d should be taken as approximately 3.0. It is conservative to increase C_d for higher aspect ratio walls. The deflection amplification factor is higher than conventional masonry because of the sliding and rocking action that occurs between the dry stacked blocks since ICEBs do not utilize mortar to prevent sliding.

The deflection amplification factor for ICEBs also applies in ASCE 7 equation 12.8-15 for calculating story drifts:

$$\delta_x = \frac{C_d \delta_{xe}}{I}$$

where δ_x is the story drift, C_d is the deflection amplification factor, δ_{xe} is the story deflection based on cracked section elastic analysis, and I is the importance factor found in ASCE 7-05 Table 11.5-1.

ICEB structures should not be higher than one story in Seismic Design Categories C through F as defined in ASCE 7-05 Table 11.6-1 and 11.6-2. According to ASCE 7-05 Table 12.2-1, conventional ordinary reinforced masonry shear walls are not permitted at all in Seismic Design Categories D through F. Because of these limits, the story drift Δ, calculated as the deflection δ_x at the top and bottom of the story in question, should be equal to δ_x calculated at the top of the wall. According to ASCE 7-05 Table 12.12-1, masonry cantilever shear wall structures should not be permitted to drift more than 0.7 percent of the story height below level x, or $0.007h_{sx}$. ICEB structures should be limited to these same criteria with h_{sx} equal to the height of the ICEB shear wall. In order to meet this requirement, the deflection amplification factor should be taken in the range of 3 to 6 to conservatively estimate drift for comparison to limits in ASCE 7-05 Table 12.12-1.

Long walls, as defined by having height to length aspect ratios less than 1.0, exhibit shear deflection behavior under lateral loads defined by Timoshenko beam theory after Ghugal et. al (2011):

$$\delta = \frac{PL^3}{3EI}\left[1 + 0.6(1 + \mu)\frac{h^2}{L^2}\right]$$

where δ is the wall deflection, P is the lateral load, L is the height of the wall, E is the modulus of elasticity of ICEBs, I is the moment of intertia of the wall about the strong axis, μ is a shape factor (0.2 for rectangular sections), and h is the shear depth of the wall. Bland found that for shear dominated wall behavior, shear deflection characterized by sliding of blocks and widening of shear cracks increased at higher applied loads due to shear stiffness degradation while flexural deflection decreased. If walls are sufficiently reinforced to prevent sudden shear failure, flexural deflection will most likely increase again due to strain hardening of flexural steel.

Plastic Behavior

Walls with openings, when analyzed as two separate piers for lateral resistance, do not fully develop plastic hinging before failure. The following equation defines the development of plastic moments at opening corners:

$$\sum M_{pi}\theta = 0.75HV_{eq}\,\theta$$

where M_{pi} is the plastic moment, H is the height of the opening, and V_{eq} is the shear force. Stirling (2011) suggests that plastic hinges develop to 75% of their nominal strength because of inadequate development length of rebar, local failures, and increased effective pier heights. This result should be verified with further testing as it is possible the underdevelopment of the plastic hinges occurred due to reinforcement detailing problems.

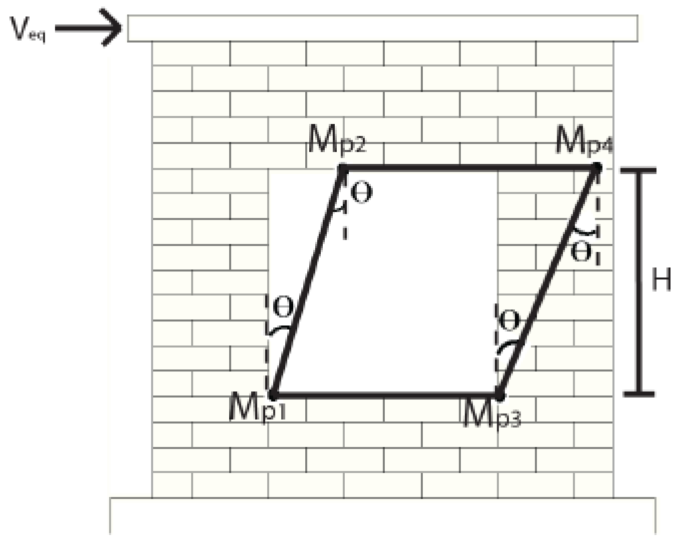

Figure 13: Plastic hinge formation in pier opening systems (Stirling 2011)

Out of Plane Flexural Capacity and Anchorage

A recent thesis by Herskedal (2012) investigated the strength of ICEB masonry when laterally loaded in the out of plane direction. Herskedal found that the flexural strength in the out of plane direction could be calculated using the current concrete masonry code, but classical mechanics based estimates of stiffness and displacements were incorrect. He performed moment curvature analyses on the five test walls and used a direct integration method with linear interpolation between block layers to predict displacements based on the curvatures. Herskedal recognized that ICEB structures should be limited to 0.7% drift as previously mentioned in the out of plane direction to limit the P-delta effect caused by increasing deflections. He determined that this deflection limit state would be reached long before flexural failure of the wall. He included a pilaster in his test walls to investigate the effect of pilasters in reducing deflections and increasing stiffness and flexural strength. Pilasters (designed in accordance with MSJC masonry column requirements) in the wall increased the nominal flexural strength by over three times and the stiffness by over sixteen times. Herskedal determined the required spacing of pilasters assuming a deflection limit state of 0.7% of the height and using the out of plane seismic force demand found in ASCE 7-05 Section 12.11.1:

$$F_p = 0.4 S_{DS} I W_p$$

where F_p is the seismic demand, S_{DS} is the design spectral acceleration, I is the importance factor (taken as 1.0), and W_p is the tributary weight of the wall. The minimum out of plane seismic demand is ten percent of the weight of the wall. Herskedal produced a chart with pilaster spacings for differing wall heights and differing S_{DS} values. He found that, for regions of high seismicity (defined as S_{DS} greater than or equal to 0.75), the maximum spacing of pilasters for most wall heights should be 3.0 meters.

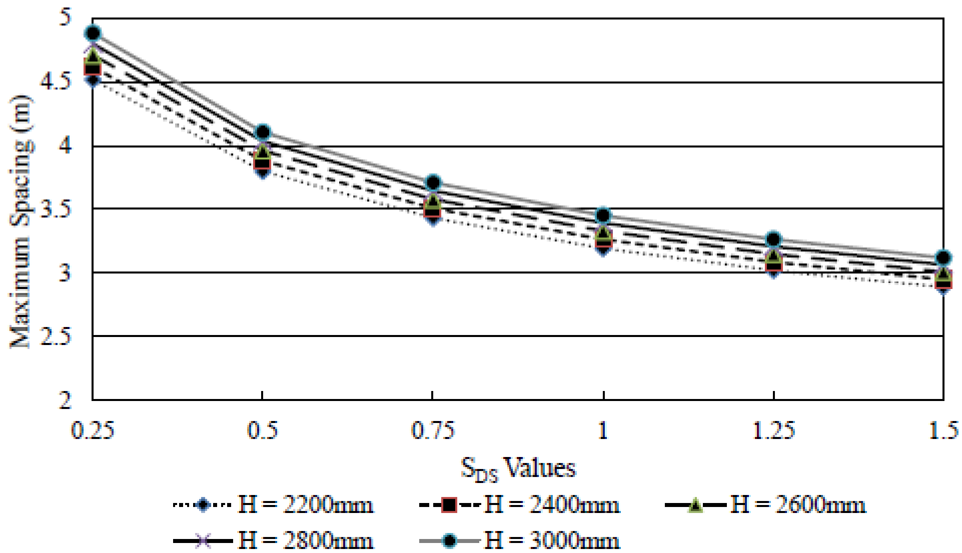

Figure 14: Maximum pilaster spacing vs. S_{DS} (Herskedal 2012)

ICEB walls should be properly anchored into the diaphragm to transmit forces throughout the system. ASCE 7-05 Section 12.11.2 stipulates that the anchorage should resist the greater of:

1. Twice the seismic out of plane demand from 12.11.1 (for Seismic Design Categories C through F)
2. A force of 5.84 S_{DS} I kN per meter of wall
3. 4.09 kN per meter of wall

Structural walls must also be designed to resist out of plane bending when the anchors exceed 1200 mm in spacing.

Roof to ICEB Wall Connections

Because the structures within the scope of this thesis are all one storey, the roof diaphragm collects inertial seismic loads and distributes them to the shear walls. The roof to wall connection is critical and must be designed with sufficient capacity to allow this transfer. Currently in Thailand, smooth rebar or steel hooks are embedded into the top of the wall, and the diaphragm structure is connected with welds as shown in Figure 15 and Figure 16. Sometimes bars are not welded to rafters but simply hooked to hold them in place as shown in Figure 19.

Figure 15:   Steel bearing plate with protruding bar welded to roof truss

Figure 19

Figure 16: Protruding bar welded to roof rafter with no bearing plate

Oftentimes, the roof diaphragm is not connected at all. A rafter may be keyed into the wall as shown in Figure 17 or it may be embedded in a layer of concrete at the top of the wall as shown in Figure 18. As a side note, blocks turned on their side faces can serve as ventilation and to let in natural light.

Figure 17: Rafter keyed into wall with no mechanical connection

Figure 18: Rafter embedded in concrete layer with sideways blocks used for ventilation (Photo credit Co-op Ville Rehabilitation Project)

Figure 19: Protruding bar hooked over rafter (Photo credit Co-op Ville Rehabilitation Project)

The roof to wall connection must be sufficiently designed to transfer lateral load from the roof diaphragm to the ICEB wall. This can be accomplished via a timber sill plate and bracket connection to the roof rafter or truss as shown in Figure 20:

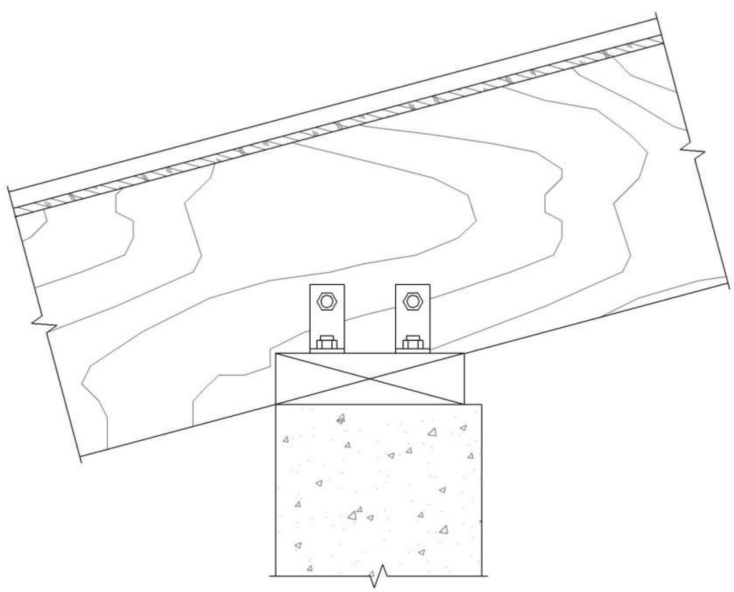

Figure 20: Timber rafter connected to wall with sill plate and through bolts

It can also be accomplished with a welded steel plate with connecting rods or brackets as shown in Figure 21:

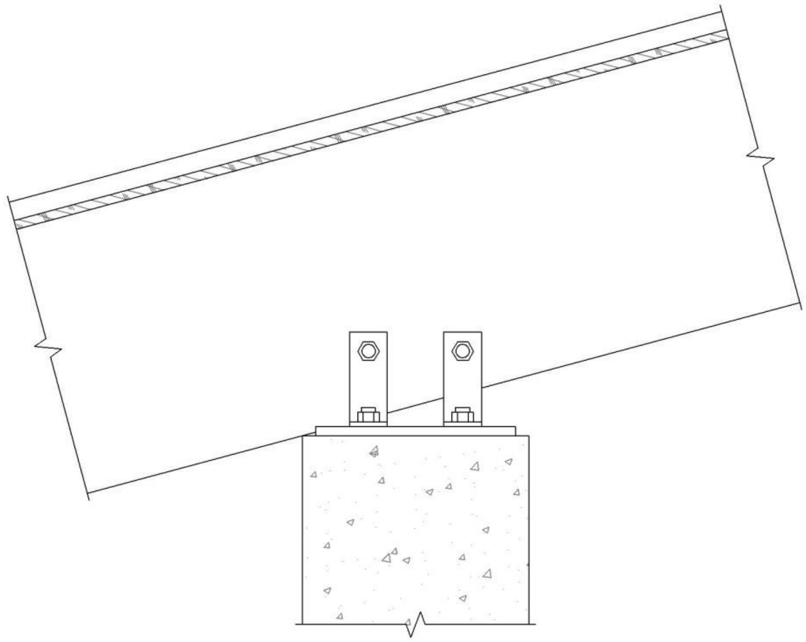

Figure 21: Steel plate on wall top connected with threaded rod or welded

ICEB Detailing Recommendations

ICEB structure foundations should have vertical reinforcement starter bars cast in place according to the layout of the walls. At a minimum, wall ends, corners, and all openings should have vertical reinforcement. It may be necessary to include more vertical reinforcement along the length of walls for increased flexural capacity. The vertical reinforcement should be as high as is practical to thread blocks over.

Horizontal shear reinforcement should be hooked around vertical reinforcement with 180° hooks. The reinforcement should be placed in channel blocks with rebar chairs to hold it above the surface of the block to ensure bonding with grout.

Grout can be similar to grout used with conventional masonry. Table 3 provides mixing proportions. The grout should be workable to ensure proper consolidation in walls.

Table 3: Recommended grout mix proportions and corresponding strengths (MSJC Tables SC-7 and SC-8)

Grout proportions by volume

Grout type	Cement	Lime	Aggregate damp, loose[1]	
			Fine	Coarse
Fine	1	0 to 1/10	2 1/4 to 3	-
Coarse	1	0 to 1/10	2 1/4 to 3	1 to 2

[1] Times the sum of the volumes of the cementitious materials

Grout strengths

Grout type	Location	Compressive strength, psi (MPa)			MSJC Reference
		Low	Mean	High	
Coarse	Lab	1,965 (13.55)	3106 (21.41)	4000 (27.58)	2.16
Coarse	Lab	3611 (24.90)	4145 (28.58)	4510 (31.10)	2.17
Coarse	Lab	5060 (34.89)	5455 (37.61)	5940 (40.96)	2.18

Grout should be poured in lifts no higher than 300 mm per MSJC Table 1.19.1 (based on 50 mm grout cells). This ensures proper consolidation of grout within the compressed earth block cells. Grout lifts higher than 300 mm may be approved by the engineer upon inspection of a test wall for proper grout consolidation and filling of void spaces. Grout should be consolidated as best as possible with mechanical or manual vibration. A grout pour should end at least 40 mm below the bed joint of a block (MSJC 3.5F). This creates a grout key which will prevent cold joints from forming at the joints between blocks. This is especially important if the grout has been left to set for more than an hour between pours.

The first layer of blocks should be laid with a layer of mortar, a layer of plastic film, and then another layer of mortar. The mortar should be of bitumen, rich cement, or have a silicone additive to help repel water intrusion. Alternatively, a comprehensive waterproofing system designed to prevent moisture intrusion manufactured by companies such as Sika, Dow Chemical, or BASF may be used. Exterior walls must be either plastered or glazed with a water-based acrylic coating for deterioration resistance.

Chapter 5: Seismic Design Criteria for the Philippines

This thesis relies upon the National Structural Code of the Philippines Volume 1, 6th ed.-Buildings, Towers, and Other Vertical Structures (NSCP 2010) for gravity loads and the United States Geological Survey (USGS) 'Worldwide Seismic "DesignMaps" Web Application' to estimate the equivalent lateral earthquake force sustained by ICEB structures of interest. The main seismic hazards in the Philippine archipelago are the 1200 km Philippine Fault that stretches from the northwest to the southeast of the country and the plate boundary subduction zones that make up the Philippine Mobile Belt, the Manila Trench to the west and the Philippine Trench to the east. In the map in Figure 22, the fault is represented by red lines and the two trenches are represented by the purple lines with triangles.

Although the Philippines Fault is broken into many different portions, the Philippine Institute of Volcanology and Seismology (PHIVOLCS) has designated almost the entire country as part of the Philippines Fault Zone (PFZ) meaning that most earthquakes in the country can be attributed to this predominant fault. This fault is responsible for the February 2012 M_w 6.9 Tayasan earthquake, the August 2012 M_w 7.6 Eastern Samar earthquake, and most recently, the February 2013 M_w 6.2 Davao del Sur earthquake (Phivolcs Seismological Observation and Earthquake Prediction Division). In fact, during the past 100 years, at least 10 significant earthquakes have been associated with the fault (Barrier et. al 1991). The quadruplex design in this thesis is part of the reconstruction efforts from earthquakes such as these.

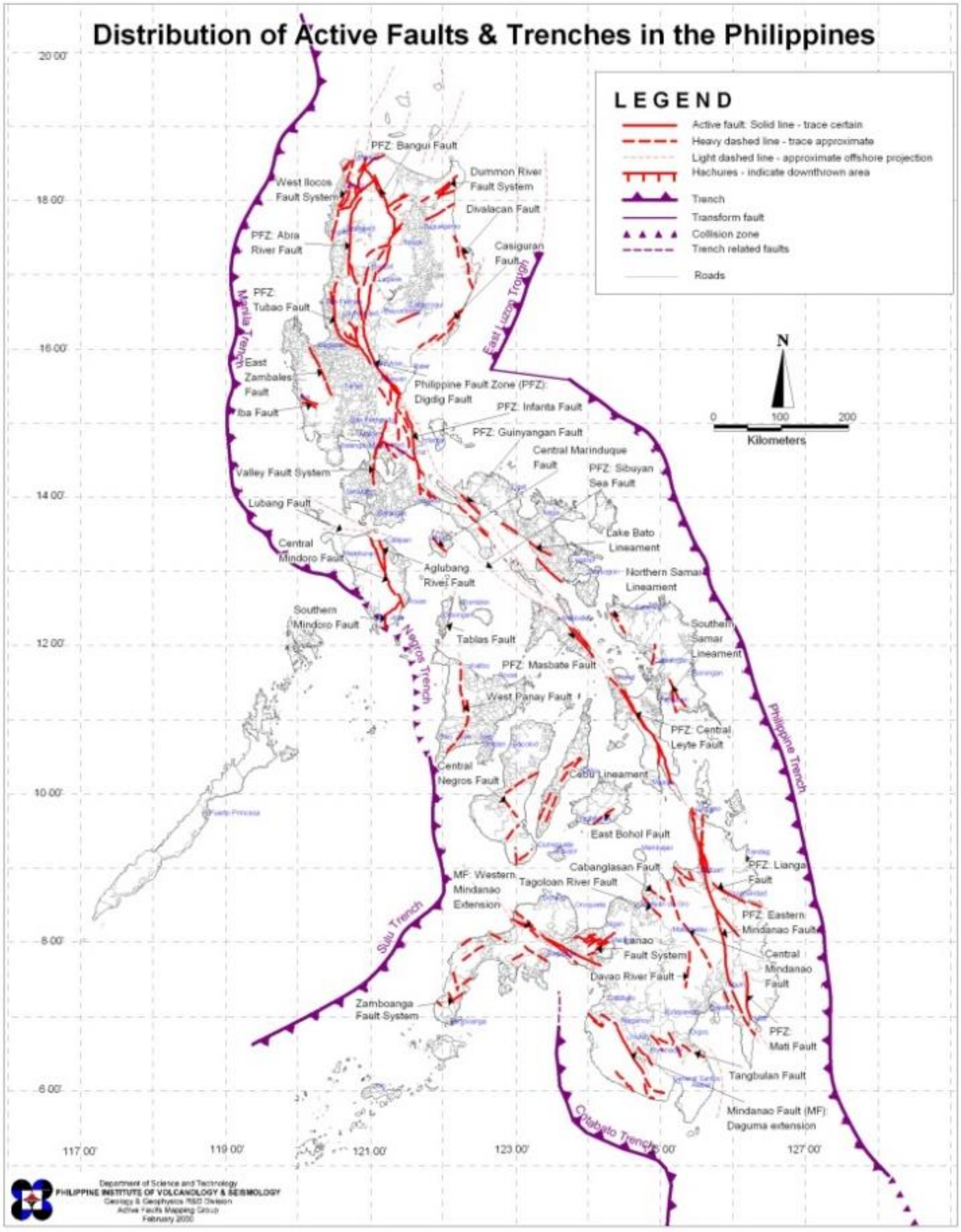

Figure 22: Philippines fault (Active Faults and Trenches in the Philippines)

The National Structural Code of the Philippines relies on seismic hazard analysis performed by Molas, Yamazaki, and Tomatsu (1992) from the Architectural Institute of Japan (AIJ) and the Japan Society of Civil Engineers (JSCE). Molas et. al noticed that although seismic design procedures had been borrowed from the Uniform Building Code from the United States, the data available from the Philippine Institute of Volcanology and Seismology (Phivolcs) were not sufficient to accurately assess the level of safety for design purposes. Molas et. al used USGS data and performed probabilistic seismic hazard analysis to propose four seismic zones throughout the Philippines. Each zone is based on a time period t of 100 years, but they all have differing return periods T and probabilities of exceedance Q. Molas et al. used this equation to relate return period to time period and probability of exceedance:

$$T = -t/\ln(Q)$$

Zone 1 is considered to be seismically inactive. Zone 2 has a mean return period in which the seismic design coefficients are exceeded of 2446.1 years, which corresponds to a probability of exceedance of 4%. Zone 3 has a mean return period of 97.9 years with 64% probability of exceedance, and Zone 4 has a mean return period of 7.9 years with a probability of exceedance of 98%.

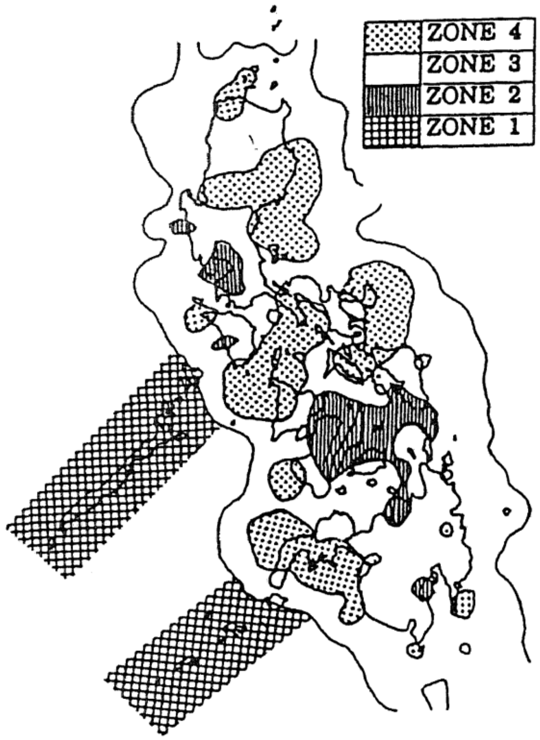

Figure 23: Philippines Zone Map (Molas et. al 1992)

The Zone factors Z for each zone are as follows:

Table 4: Zone factors for the Philippines (Molas et. al. 1992)

ZONE	Zone Factor, Z
1	0.5
2	0.7
3	1.0
4	1.5

According to Molas et al., base shear was calculated after the United States' Uniform Building Code (UBC) using the equation

$$V = Z \cdot C \cdot S \cdot I \cdot K \cdot W$$

where Z is the zone factor, C is the response factor (peak ground acceleration for the zone's return period), S is the soil profile factor, K is the structural factor (similar to the ASCE-7-05 response modification coefficient), and W is the seismic weight of the structure. The zone factor amplifies or diminishes the hazard according to its return period to arrive at a design basis earthquake base shear. This is inconsistent with the method found in ASCE 7-05. ASCE 7-05 uses USGS probabilistic seismic hazard maps that show the highest considered seismic hazard throughout a specific area. Next, factors to diminish or amplify that hazard based on soil conditions, occupancy categories, and response modifications due to structure ductility are applied. The NSCP (2010) now uses a similar simplified method to calculate base shear for ordinary structures:

$$V = \frac{3C_a}{R}W$$

where V is the base shear, C_a is an acceleration controlled seismic coefficient based on seismic zone and soil profile, and R is the "numerical coefficient representative of the inherent overstrength and global ductility capacity of lateral force resisting systems". This method may only be used for short period structures with standard occupancy.

The structural factor K used in the UBC method proposed in 1992 by Molas et al. and the numerical coefficient R used in the 2010 NSCP are different from the response modification coefficient R used in ASCE 7-05. This is because there are different methods of calculating this factor. Whittaker et al. (1990) suggests an R coefficient as the product of three components: a strength factor, a ductility factor, and a redundancy factor.

$$R = R_S R_\mu R_R$$

where R_S is the strength factor, R_μ is the ductility factor, and R_R is the redundancy factor. The strength factor R_S is based on the available nominal shear capacity V_b compared to the ultimate shear capacity V_u. We will assume R_S is equal to 1.0 based on shear wall tests from Bland (2011) which show shear dominated ICEB shear walls to show very little overstrength due to buckling of compression reinforcement. Also, we will assume most simple ICEB structures will be designed with few, if any, redundant members. We will therefore also assume R_R to be 1.0. We will solely rely on the ductility component R_μ to calculate R because it is related to the displacement ductility, which is readily available for ICEB shear walls. The ductility component can be calculated as follows:

$$R_\mu = [c(\mu - 1) + 1]^{1/c}$$

where μ is the displacement ductility and c is taken as 2.0.

The seismic forces in this thesis are calculated using procedures in ASCE 7-05 to convert the maximum considered earthquake hazard, an event with a 2% probability of exceedance in 50 years with a return period of 2500 years to the 10% in 50 years design basis earthquake with a return period of 475 years, accounting for soil profile factors and occupancy categories. The USGS map has been obtained from a probabilistic seismic hazard analysis for the Philippines. The seismic design acceleration is then used to determine an equivalent lateral force on the diaphragm of the structure, and the force is then distributed throughout the structure. This thesis does not utilize the NSCP (2010) procedure for the calculation of base shear to remain consistent with the estimation of seismic design parameters using methods proposed in U.S. based codes.

Chapter 6: Design of Multifamily Quadruplex Home in the Philippines

Applicable Codes and References

This design will utilize the 2010 edition of the National Structural Code of the Philippines (NSCP) to determine standard gravity loadings and then will use the 2% in 50 year Maximum Considered Earthquake (MCE) spectral accelerations from the USGS ‘Worldwide Seismic “DesignMaps” Web Application’ to estimate the equivalent lateral earthquake force. It will then follow provisions in ACI 318-08, 2008 MSJC, and ASCE 7-05 for capacity design.

Project Information

One-Storey Interlocking Compressed Earth Block Quadruplex Home

Occupancy: Residential (I=1.0)

Load and Resistance Factor (LRFD) Design

ICEB 28-day compressive strength:

6 MPa (block)

9 MPa (grout)

3 MPa (grout and block prism)

1 MPa (at 0.0025 maximum usable strain)

Blocks have 6% cement content by mass and soil has 15% clay content by mass

Steel yield strength f_y: 206 MPa (Grade 30) 10 mm bars

Building Length: 11850 mm

Building Width: 8250 mm

Average Building Height: 2600 mm

Roof Length: 13050 mm

Roof Width: 10050 mm

Roof Area: 131.15 m^2

Latitude, Longitude: 11.87°,122.86°

Two Percent in 50 Years Probability of Exceedance Short Period Spectral Acceleration

S_s: 1.44

Two Percent in 50 Years Probability of Exceedance Long Period Spectral Acceleration

S_1: 0.58

See Appendix B for renderings, design drawings and full detailed calculations.

Project Location Map

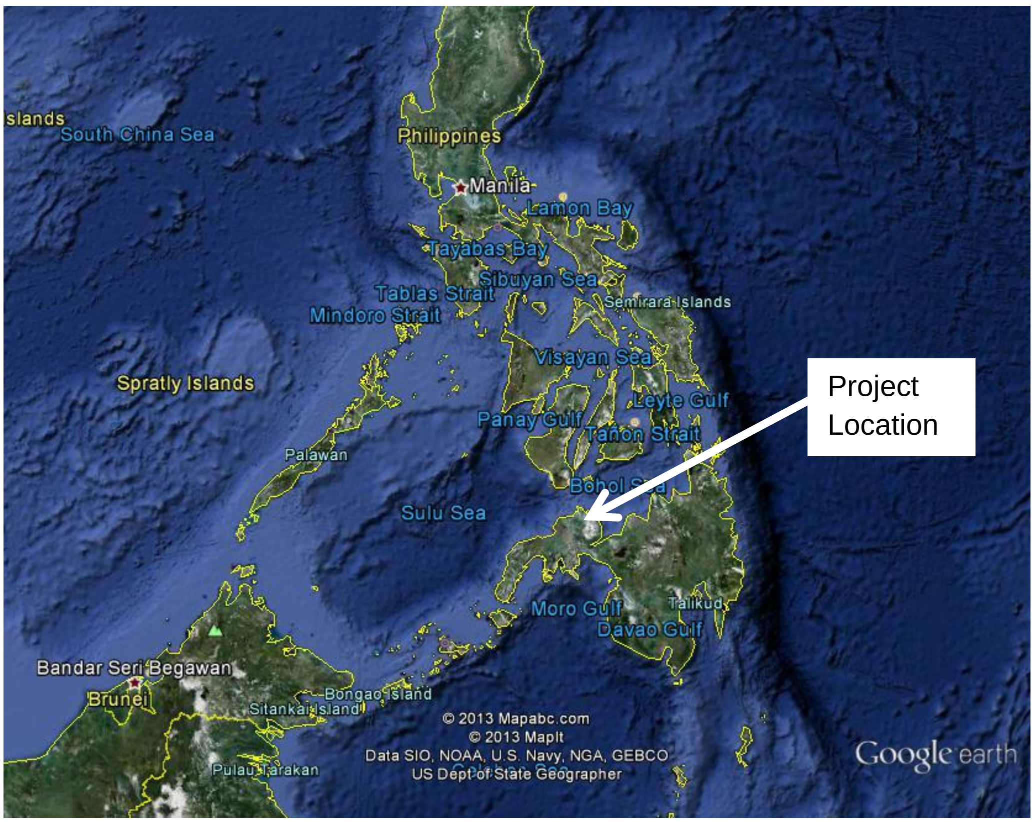

Figure 24: Philippines Location Map (Google Earth)

Floor Plan

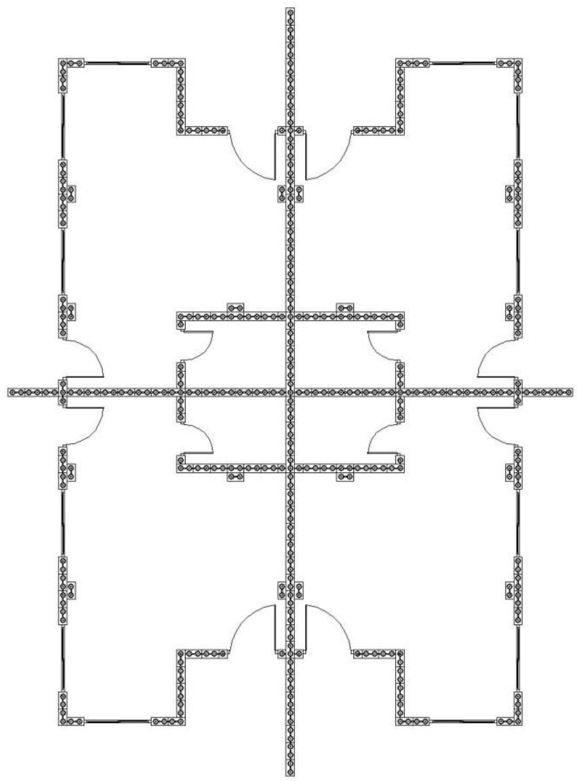

Figure 25: Floor Plan of Quadruplex

Gravity Loading

Standard dead and live loads for the Philippines were obtained from the National Structural Code of the Philippines (NSCP). The weight of the structure was calculated to be 428.79 kN based on the weight of ICEB walls, roofing, and 10% of the roof live load.

Table 5: Standard Loads

Dead Loads			Notes
Concrete	23.6	kN/m^3	NSCP Table 204-1
Structural Steel	77	kN/m^3	NSCP Table 204-1
ICEB Wall	2.62	kPa	Full Grouting, plaster
Steel Roof Trusses/Purlins	0.06	kPa	NSCP Table 204-2
Sheet Metal (Ga. 26)	0.1	kPa	NSCP Table 204-2
Ceiling Joists/Plywood	0.15	kPa	NSCP Table 204-1
Live Loads			
Residential Floor Live Load	1.9	kPa	NSCP Table 205-1
Residential Roof Live Load	0.6	kPa	NSCP Table 205-3

Table 6: Weight of ICEB Walls

Wall	Length (m)	Average Height (m)	Area (m^2)	Weight (kN)
1	0.9	2.2	1.98	5.19
2	1.2	2.2	2.64	6.92
3	0.75	2.2	1.65	4.32
4	0.75	2.2	1.65	4.32
5	1.2	2.2	2.64	6.92
6	0.9	2.2	1.98	5.19
7	1.8	2.5	4.50	11.79
8	13.65	2.8	38.22	100.14
9	1.8	2.5	4.50	11.79
10	0.9	2.2	1.98	5.19
11	1.2	2.2	2.64	6.92
12	0.75	2.2	1.65	4.32
13	0.75	2.2	1.65	4.32
14	1.2	2.2	2.64	6.92
15	0.9	2.2	1.98	5.19
16	1.8	2.5	4.50	11.79
17	1.8	2.5	4.50	11.79
18	1.2	2.5	3.00	7.86
19	1.2	2.5	3.00	7.86
A	0.9	2.65	2.39	6.25
B	0.9	2.65	2.39	6.25
C	3.9	2.65	10.34	27.08
D	10.05	2.5	25.13	65.83
E	3.9	2.65	10.34	27.08
F	0.9	2.65	2.39	6.25
G	0.9	2.65	2.39	6.25
			Total	373.70

Table 7: Weight of Roof Components

Component	Load (kPa)	Roof Area (m^2)	Weight (kN)	Notes
Trusses/Purlins	0.06	131.15	7.87	NSCP Table 204-2
26 Ga GI Sheet	0.1	131.15	13.12	NSCP Table 204-2
Ceiling Joists, Plywood, Finishes	0.2	131.15	26.23	NSCP Table 208-12 Note 7
		Total	47.21	

Seismic Loading

Table 8: Seismic Design Criteria ASCE 7-05 11.4

Seismic Design Criteria ASCE 7-05 11.4		
Category	**Value**	**Notes**
Site Class	D	Table 20.3-1
S_s	1.44	USGS
S_1	0.58	USGS
F_a	1	Table 11.4-1
F_v	1.5	Table 11.4-2
S_{MS}	1.44	11.4-1
S_{M1}	0.87	11.4-2
S_{DS}	0.96	11.4-3
S_{D1}	0.58	11.4-4
Seismic Design Category	D	Table 11.6-2
I	1	Table 11.5-1
R	2	
C_D	3	
T_a	0.11	12.8.2.1
C_s	0.48	12.8-2
Wtotal	428.79	kN
Vbase	205.82	kN
Mov	535129.92	kN-mm

A rigid diaphragm and flexible diaphragm analysis is presented to envelope the possible response of the structure during an earthquake event. For the rigid diaphragm analysis, walls were individual named 1-19 in the north-south direction and A-G in the east-west direction. The structure has stiff walls in the north-south (Wall 8) and east west directions (Wall D) which resist 89% of the direct shear load and 85% of the direct shear load respectively under a rigid diaphragm analysis. The structure is symmetric with the center of rigidity located exactly at the center of mass so it does not include any induced shears from torsional moments. For the flexible diaphragm analysis, wall lines were named A-C in the north-south direction and 1-5 in the east-west direction. The results of the flexible diaphragm analysis showed the stiff wall in the north-south direction (made up of wall line B) resists 50% of the load, with the other 50% distributed evenly to the outside walls (wall lines A and C). In the east-west direction, the load is redistributed to the bathroom walls and the recessed porch walls. The long, stiff wall (Wall line 3) resists 20% of the load, meaning the bathroom and recessed porch walls are required to share the remaining 80%.

The flexible diaphragm analysis requires the exterior walls to resist significantly more load. The actual behavior of the structure is somewhere between these two responses. In the north-south direction, the response can reliably be analyzed as rigid because of the aspect ratio of the diaphragm in that direction; it is much deeper than it is wide, making it very stiff. In the east-west direction, the response is most likely closer to the flexible diaphragm case. The diaphragm is more flexible in this direction. The diaphragm must be adequately connected to the walls in order to transfer the shear forces to the walls. Although outside the scope of this design, a possible roof rafter connection and its calculated demand to capacity ratio is presented in Appendix B.

The deflections in the walls were calculated using uncracked section properties for simplicity. The material models input into XTRACT (Chadwell, 2004) to determine wall capacities to compare to demands were limited in the linear elastic range because the pool of research on the non-linear overstrength properties of ICEB walls is not yet robust enough. The compressive strength of the masonry was 1 MPa at 0.0025 strain and the steel model was Grade 30 steel with yield strain at 0.001 and plastic thereafter until 0.002 maximum usable strain to be conservative. In reality, the steel is probably able to deform until at least the maximum usable strain of the ICEB (0.0025) or beyond.

The shear capacity design was performed using the equation recommended by Bland (2011) with partially grouted walls. The shear reduction factor was assumed to be 0.75 and the horizontal steel was assumed to be 10mm Gr. 30 (f_y=206 MPa).

Detailed spreadsheet verification calculations of the lateral force resisting ICEB shear walls can be found in Appendix B. Design drawings can be found in Appendix C.

Chapter 7: Conclusions and Recommendations for Further Research

The research conducted thus far has yielded information on the composition of ICEBs, the performance of flexural and shear dominated walls, and the out of plane performance of walls. Further research is needed to investigate the effects of varying steel ratios on shear wall performance, with special emphasis on attempting to gain strain hardening overstrength from following MSJC requirements for ordinary, intermediate, and special reinforced shear walls. Furthermore, ICEB columns and pilasters should be examined to compare their performance to conventional masonry column behavior, with the possibility of nonlinear behavior considered due to grout confinement and steel overstrength. It would then be possible to generate axial load versus moment capacity diagrams for columns, which could expand possible designs to include two storey buildings.

The important design recommendations from this seismic design manual are summarized below:

1. ICEBs have a much lower Young's Modulus E than conventional concrete masonry, $157f'_{ICEB}$ as opposed to $900f'_m$. This means ICEBs are more flexible and will deform more under applied loads.
2. ICEBs exhibit much higher strains at peak stress ε_0 and maximum usable strains ε_{su} than conventional concrete masonry. However, until research validates the use of higher strain limits, the same compressive strain limits for CMUs should be applied to ICEBs: ε_0=0.0025 and ε_{su}=0.004.
3. The shear capacity of ICEBs should be multiplied by factors of 0.2 for partially grouted walls (defined as only grouting keyways between blocks and cavities with reinforcement) and 0.4 for fully grouted walls (defined as grouting all cavities). This is to account for reduced shear area in the bed joint due to the

interlocking dry stack mechanism and the doweling action provided by grout cells.

4. The contribution of horizontal steel remains the same for ICEBs as conventional masonry.
5. The flexural capacity of walls can be reliably estimated using current masonry code techniques. The constraints imposed by the masonry code in Section 3.3.3.5 should be followed for ICEB design. When using masonry compressive strain limits (0.0025 compressive strain), ICEBs will remain linear elastic.
6. Wall deflections can be estimated based on elastic deflections. Care should be taken to account for block sliding in deflection calculations. One method is to amplify the deflections based on the deflection amplification factor C_d, approximately 3.0 for 1:1 aspect ratio walls.
7. Pilasters detailed per masonry requirements in Section 1.14 should be spaced based on the chart proposed by Herskedal (2012) to control out of plane deflections.

The design of the quadruplex is rooted in several assumptions, and has limitations. The walls are designed to remain primarily in the linear elastic range, with some plastic deformation allowed based on the 30 ksi (206 MPa) steel model input into XTRACT. The steel yield strain was set at 0.001 and the maximum usable strain was set at 0.002, only utilizing perfectly plastic effects. The ICEB material was limited to 0.0025 compressive strain with a prism compressive strength of 1 MPa. This corresponds to roughly 15% clay content and 6% cement, the parameters of the blocks used in the earlier referenced large scale experimental studies. The roof diaphragm is assumed to be adequately connected to all walls, including interior walls, in order to transfer load to shear walls. The walls were checked for out of plane capacity in

XTRACT and then pilasters were added to the walls if they did not pass deflection or flexural requirements.

Although material properties of interlocking compressed earth blocks are well understood, there is not yet a universally accepted testing procedure document to determine the basic material properties. Also, few studies have been conducted on the effect of moisture on block compressive strength, the addition of cementious materials other than Portland cement, the interaction between silt and clay particle content and cement content, and the effect of varying cement ratios in the mix for interlocking rhino blocks such as those pressed with the Soeng Thai BP6 block press.

The resurgence of earthen construction in the form of interlocking compressed earth blocks is still in progress. As a result, it has a long way to go before design standards are codified and the technology gains institutional acceptance. With the globalization of the economy and the rising of median incomes throughout the world, compressed earth block technology may experience a reduction in interest in favor of less labor intensive materials. However, although they have not been institutionally accepted, interlocking compressed earth blocks used within the context of a properly designed structural system still remain a viable material for many developing countries around the world because of their cost, durability, and aesthetic qualities.

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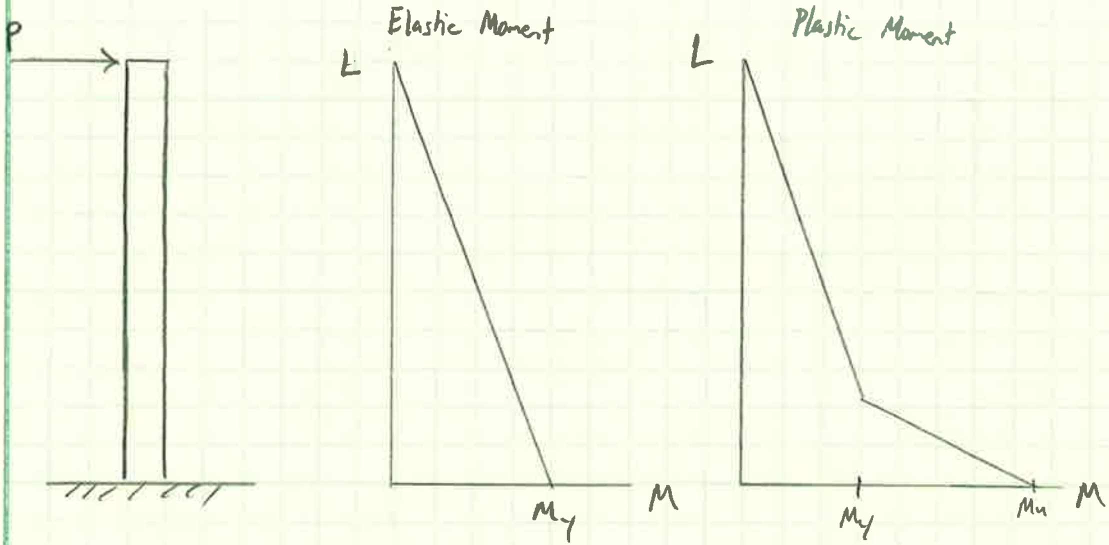
Appendix A: Estimation of Response Modification Factor and Displacement Amplification Factor

Estimation of Response Modification Factor From Test Results

	Wall 3 (Bland)	Wall 4 (Stirling)	Wall 5 (Stirling)
Wall Height (m)	1.8	1.8	1.8
Effective Wall Height $L_{effective}$ (m)	1.8	1.8	0.9
Shear Depth d (m)	1.8	0.9	1.8
Length of Plastic Hinge L_p (m)	0.9	0.45	0.9
XTRACT Output			
Overstrength Factor Ω_0	1.18	1.08	1.06
Yield Curvature φ_y (1/m)	1.74E-03	3.47E-03	2.25E-03
Ultimate Curvature φ_u (1/m)	5.60E-03	3.09E-02	3.32E-02
Curvature Ductility μ_φ	3.22	8.90	14.77
ACI Lumped Plasticity Method			
Effective Yield Curvature φ_y' (1/m)	1.89E-03	3.60E-03	2.31E-03
Effective Yield Displacement Δ_y' (m)	2.04E-03	3.89E-03	6.24E-04
Plastic Curvature φ_p (1/m)	3.71E-03	2.73E-02	3.09E-02
Plastic Rotation θ_p (rad)	3.34E-03	1.23E-02	2.78E-02
Plastic Displacement Δ_p (m)	4.51E-03	1.93E-02	1.25E-02
Total Displacement Δ_u (m)	6.54E-03	2.32E-02	1.31E-02
Calculated Displacement Ductility μ_Δ	3.21	5.97	21.04
Actual Displacement Ductility μ_Δ	2.63	6.06	N/A
Element Response Modification Coefficient R	2.33	3.31	6.41
Displacement Amplification Factor C_d	3.77	6.43	22.24

Test results from Bland and Stirling were used to calibrate a model in XTRACT. Material model inputs were as closely modeled to actual tested material models as possible. An ICEB model with strain at peak stress of 0.0025 was used to simulate the behavior of the earth masonry, and 40 ksi steel with yield strain of 0.001 and strain at onset of strain hardening of 0.008 was input for the behavior of the flexural reinforcement. The displacement ductility (ultimate displacement divided by yield displacement) from three flexurally dominated test walls was compared to displacement ductility calculated using XTRACT yield curvature and ultimate curvature outputs with the ACI lumped plasticity method with the length of the plastic hinge taken as half the wall height and the effective height of the wall as equal to the full wall height. As shown, the calculated displacement ductility matched fairly well with the actual displacement ductility for Walls 3 and 4, with a 22% difference for Wall 3 and a 1% different for Wall 4. This model did not account for shear deformation or block sliding in its calculation of ultimate and yield displacement.

Deriving ductility-based response modification coefficient using lumped plasticity.



$\phi'_y = \frac{1}{2}\left(\frac{M_u}{M_y} + 1\right)\phi_y$ (effective curvature)

$\Delta'_y = \frac{1}{3}\phi'_y L^2$ (effective yield displacement)

Length of plastic hinge = $h/2$ (h is height of section)

$\phi_p = \phi_u - \phi'_y$ (plastic curvature)

$\theta_p = \phi_p L_p$ (plastic rotation)

$\Delta_p = \theta_p \left(L - \frac{L_p}{2}\right)$ (plastic displacement)

$\Delta_u = \Delta'_y + \Delta_p$ (ultimate displacement)

$\frac{\Delta_u}{\Delta'_y} = \mu_\Delta$ (displacement ductility)

$R = \sqrt{2\mu_\Delta - 1}$ (response modification for ductility)

See Estimation of Response Modification Factor table for test results and calculated results using moment curvature data from XTRACT.

Wall 3: $L = 1.8\,m$ $\phi_y = 1.74\,E{-03}$ (1/m)
$h = 1.8\,m$ $\phi_u = 5.60\,E{-03}$ (1/m)
$L_p = 0.9\,m$
$\Omega_0 = 1.18$ $\mu_\phi = 3.22$

$\phi'_y = 1.99\,E{-03}$ (1/m) $\phi_p = 3.71\,E{-03}$ (1/m) $\Delta_u = 6.54\,E{-03}$ (m)
$\Delta'_y = 2.04\,E{-03}$ (1/m) $\theta_p = 3.34\,E{-03}$ rad $\frac{\Delta_u}{\Delta'_y} = 3.21$
$\Delta_p = 4.51\,E{-03}$ (m)

Estimation of Cd from Bland and Stirling test results for Wall 3, 4, 5

Displacement Amplification Factor

$$C_d = \frac{\Delta_u}{\Delta'_y} \Omega_o$$

Wall 3: XTRACT output: $\Omega_o = 1.18$

Using ACI Lumped Plasticity method,

$$\left.\begin{aligned} \Delta_u &= 6.54E\text{-}03\ m \\ \Delta'_y &= 2.04E\text{-}03\ m \end{aligned}\right\}\ \text{derived previously in this Appendix}$$

$C_d = \frac{\Delta_u}{\Delta'_y} \Omega_o = 3.77$ for Wall 3 (Bland) (1:1 aspect ratio)

$C_d = 6.43$ for Wall 4 (Stirling) (2:1 aspect ratio)

Use $C_d = 3.0$ for ICEB walls as conservative estimate.

This is reasonable for $R = 2.0$. Due to the uncertainty of sliding deflection and variable block geometry, design walls to primarily remain in the linear elastic range.

XTRACT Material Report - Educational

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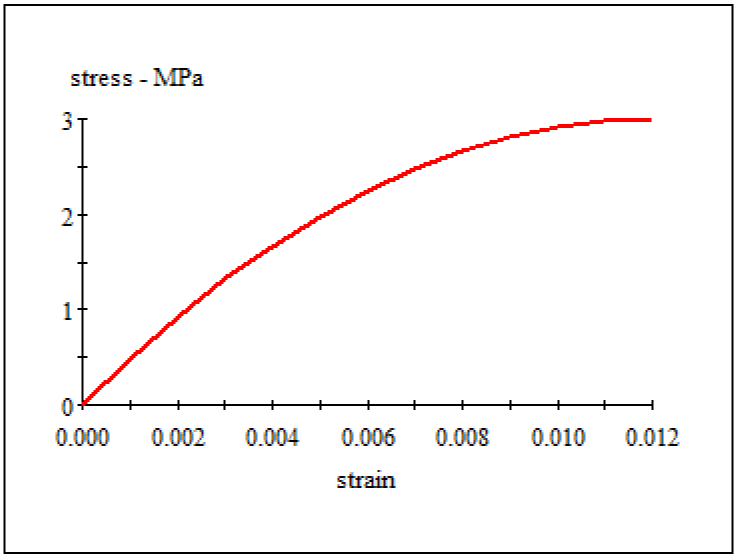
Material Name: ICEB

Material Type: User Defined

Input Parameters:

Ultimate Compresive Strain:	2.500E-3
Compression Yield Strain:	2.000E-3
Tensile Yield Strain:	1.0000
Ultimate Tensile Strain:	1.200
Additional Information:	'Strain' 'Stress'



Material Color States:

Yield

Stress Strain Points:

Strain	Stress (MP
0	0
1.000E-3	.4792
2.000E-3	.9167
3.000E-3	1.313
4.000E-3	1.667
5.000E-3	1.979
6.000E-3	2.250
7.000E-3	2.479
8.000E-3	2.667
9.000E-3	2.813
10.00E-3	2.917
11.00E-3	2.979
12.00E-3	3.000

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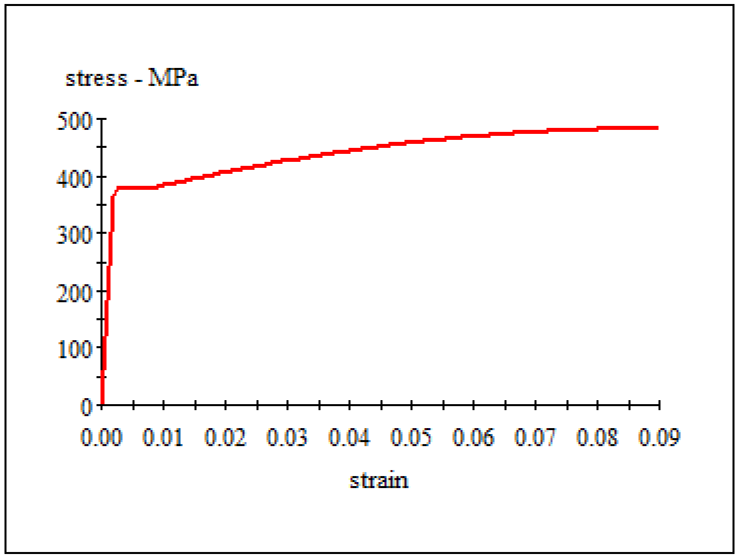
Material Name: Grade 40

Material Type: Strain Hardening Steel

Input Parameters:

Yield Stress:	378.0 MPa
Fracture Stress:	482.6 MPa
Yield Strain:	1.891E-3
Strain at Strain Hardening:	8.000E-3
Failure Strain:	90.00E-3
Elastic Modulus:	199.9E+3 MPa
Additional Information:	Symetric Tension and Comp.



Model Details:

For Strain - $\varepsilon < \varepsilon_y$ $\quad fs = E \cdot \varepsilon$

For Strain - $\varepsilon < \varepsilon_{sh}$ $\quad fs = f_y$

For Strain - $\varepsilon < \varepsilon_{su}$ $\quad fs = f_u - (f_u - f_y) \cdot \left(\dfrac{\varepsilon_{su} - \varepsilon}{\varepsilon_{su} - \varepsilon_{sh}} \right)^2$

ε = Steel Strain

fs = Steel Stress

f_y = Yield Stress

f_u = Fracture Stress

ε_y = Yield Strain

ε_{sh} = Strain at Strain Hardening

ε_{su} = Failure Strain

E = Elastic Modulus

Material Color States:

- Tension force after onset of strain hardening
- Tension force after yield
- Initial state
- Compression force after yield
- Compression force after onset of strain hardening

XTRACT Material Report - Educational

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Material Name: Concrete

Material Type: Unconfined Concrete

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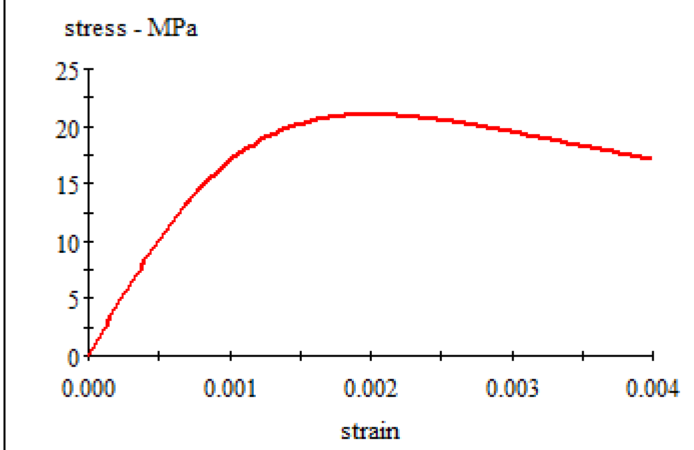
Input Parameters:

Tension Strength:	0 MPa
28 Day Strength:	21.00 MPa
Post Crushing Strength:	0 MPa
Tension Strain Capacity:	0 Ten
Spalling Strain:	6.000E-3 Comp
Failure Strain:	4.000E-3 Comp
Elastic Modulus:	21.69E+3 MPa
Secant Modulus:	1523 MPa



Model Details:

For Strain - $\varepsilon < 2 \cdot \varepsilon_t$ $\quad fc = 0$

For Strain - $\varepsilon < 0$ $\quad fc = \varepsilon \cdot Ec$

For Strain - $\varepsilon < \varepsilon_{cu}$ $\quad fc = \dfrac{f'_c \cdot x \cdot r}{r - 1 + x^r}$

For Strain - $\varepsilon < \varepsilon_{sp}$ $\quad fc = f_{cu} + \left(f_{cp} - f_{cu}\right) \cdot \dfrac{\left(\varepsilon - \varepsilon_{cu}\right)}{\left(\varepsilon_{sp} - \varepsilon_{cu}\right)}$

$$x = \frac{\varepsilon}{\varepsilon_{cc}}$$

$$r = \frac{Ec}{Ec - E_{sec}}$$

$$E_{sec} = \frac{f'_c}{\varepsilon_{cc}}$$

ε = Concrete Strain

fc = Concrete Stress

Ec = Elastic Modulus

E_{sec} = Secant Modulus

ε_t = Tension Strain Capacity

ε_{cu} = Ultimate Concrete Strain

ε_{cc} = Strain at Peak Stress = .002

ε_{sp} = Spalling Strain

f'_c = 28 Day Compressive Strength

f_{cu} = Stress at ε_{cu}

f_{cp} = Post Spalling Strength

Material Color States:

- Tension strain after tension capacity
- Tension strain before tension capacity
- Initial state
- Compression before crushing strain
- Compression before end of spalling
- Compression after spalling

Reference:

Mander, J.B., Priestley, M. J. N., "Observed Stress-Strain Behavior of Confined Concrete", Journal of Structural Engineering, ASCE, Vol. 114, No. 8, August 1988, pp. 1827-1849

XTRACT Section Report - Educational

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Section Name: ICEB David Wall 3

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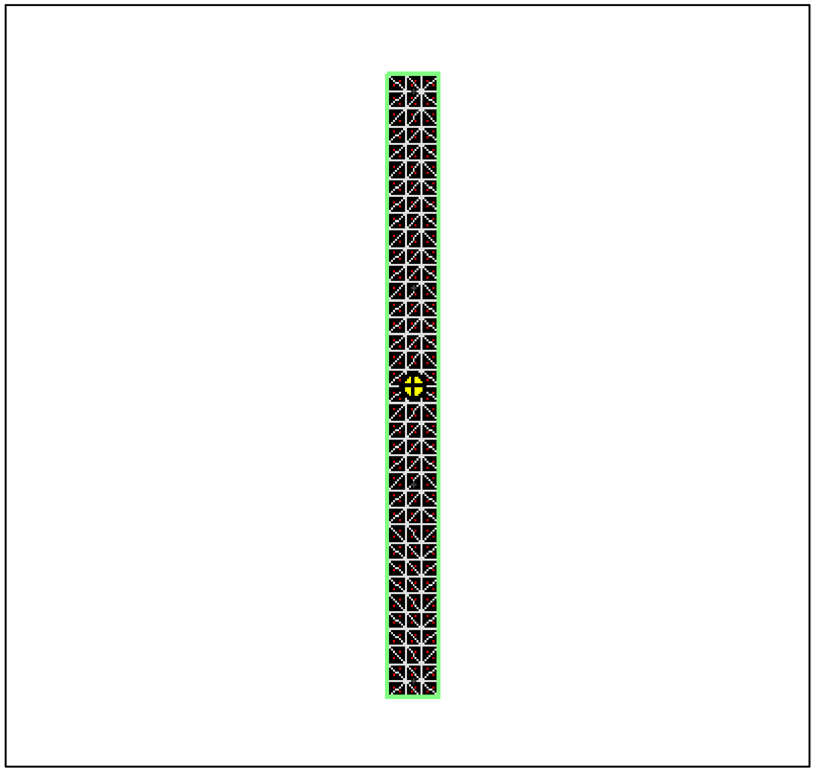
Section Details:

X Centroid:	-5.22E-15 mm
Y Centroid:	1.11E-14 mm
Section Area:	270.0E+3 mm^2
EI gross about X:	60.06E+6 N-m^2
EI gross about Y:	224.6E+3 N-m^2
I trans (ICEB) about X:	12.53E+6 cm^4
I trans (ICEB) about Y:	46.87E+3 cm^4
Reinforcing Bar Area:	314.2 mm^2
Percent Longitudinal Steel:	.1164 %
Overall Width:	150.0 mm
Overall Height:	1800 mm
Number of Fibers:	216
Number of Bars:	4
Number of Materials:	2



Material Types and Names:

User Defined:	ICEB
Strain Hardening Steel:	Grade 40

Comments:

User Comments

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Section Name: ICEB David Wall 3

Loading Name: Moment Curvature

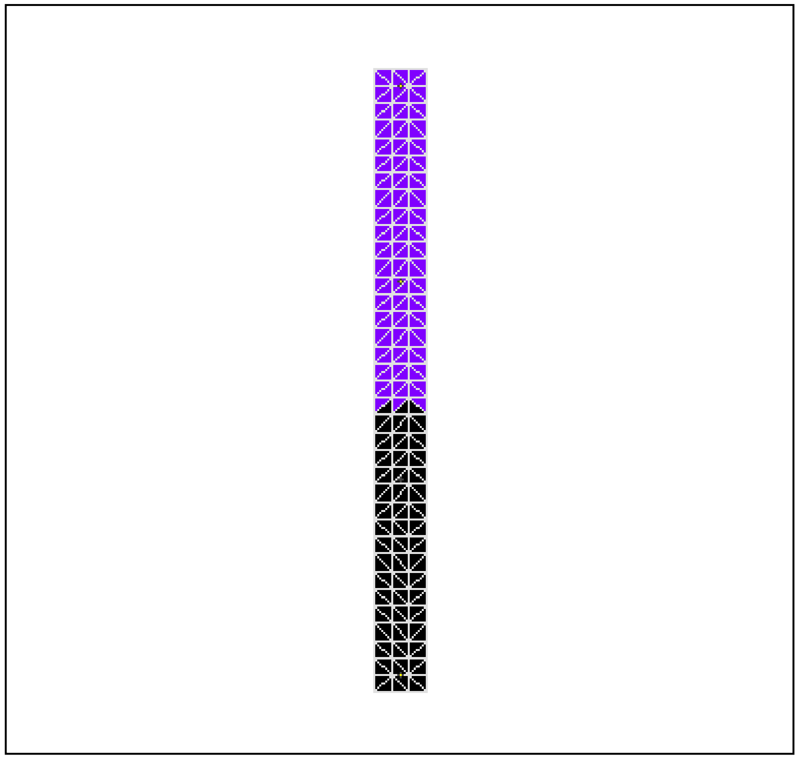
Analysis Type: Moment Curvature

Section Details:

X Centroid: -5.22E-15 mm

Y Centroid: 1.11E-14 mm

Section Area: 270.0E+3 mm^2



Loading Details:

Incrementing Loads: Mxx Only

Number of Points: 29

Analysis Strategy: Displacement Control

Analysis Results:

Failing Material: ICEB

Failure Strain: 2.500E-3 Compression

Curvature at Initial Load: 0 1/m

Curvature at First Yield: 1.735E-3 1/m

Ultimate Curvature: 5.595E-3 1/m

Moment at First Yield: 62.08E+3 N-m

Ultimate Moment: 85.86E+3 N-m

Centroid Strain at Yield: .4164E-3 Ten

Centroid Strain at Ultimate: 2.442E-3 Ten

N.A. at First Yield: 240.0 mm

N.A. at Ultimate: 436.5 mm

Energy per Length: 356.9 N

Effective Yield Curvature: 2.042E-3 1/m

Effective Yield Moment: 73.08E+3 N-m

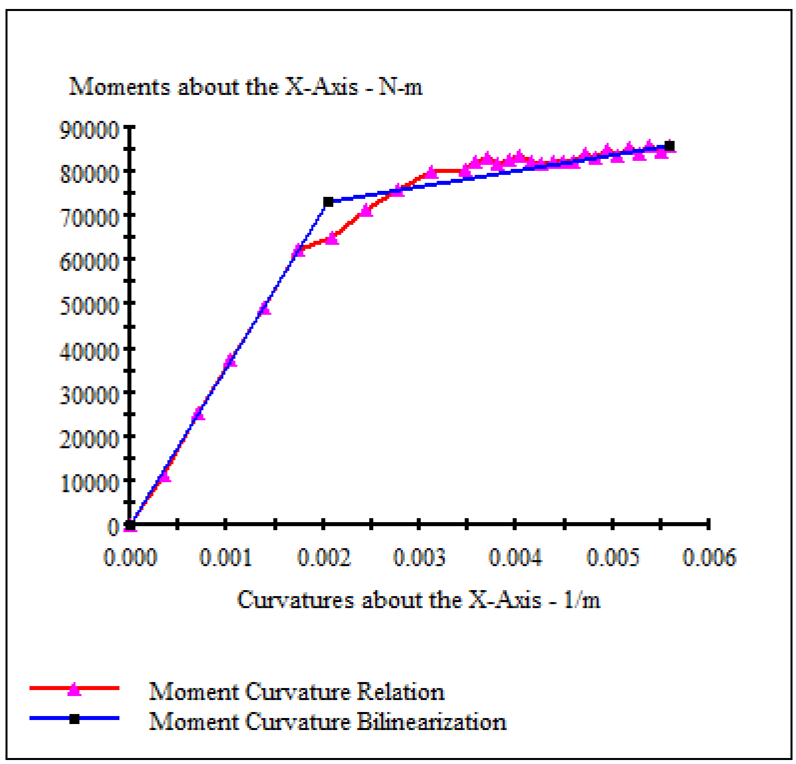
Over Strength Factor: 1.175

EI Effective: 35.78E+6 N-m^2

Yield EI Effective: 3.597E+6 N-m^2

Bilinear Harding Slope: 10.05 %

Curvature Ductility: 2.739



Comments:

User Comments

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Section Name: ICEB Brad Wall 4

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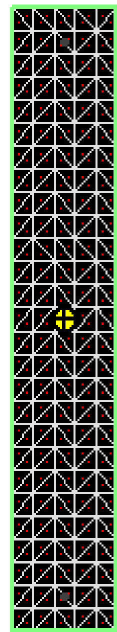
Section Details:



X Centroid:	-1.84E-15 mm
Y Centroid:	-4.69E-15 mm
Section Area:	135.0E+3 mm^2
EI gross about X:	9.375E+6 N-m^2
EI gross about Y:	118.1E+3 N-m^2
I trans (ICEB) about X:	1.956E+6 cm^4
I trans (ICEB) about Y:	24.64E+3 cm^4
Reinforcing Bar Area:	157.1 mm^2
Percent Longitudinal Steel:	.1164 %
Overall Width:	150.0 mm
Overall Height:	900.0 mm
Number of Fibers:	270
Number of Bars:	2
Number of Materials:	2

Material Types and Names:

User Defined:	ICEB
Strain Hardening Steel:	Grade 40

Comments:

User Comments

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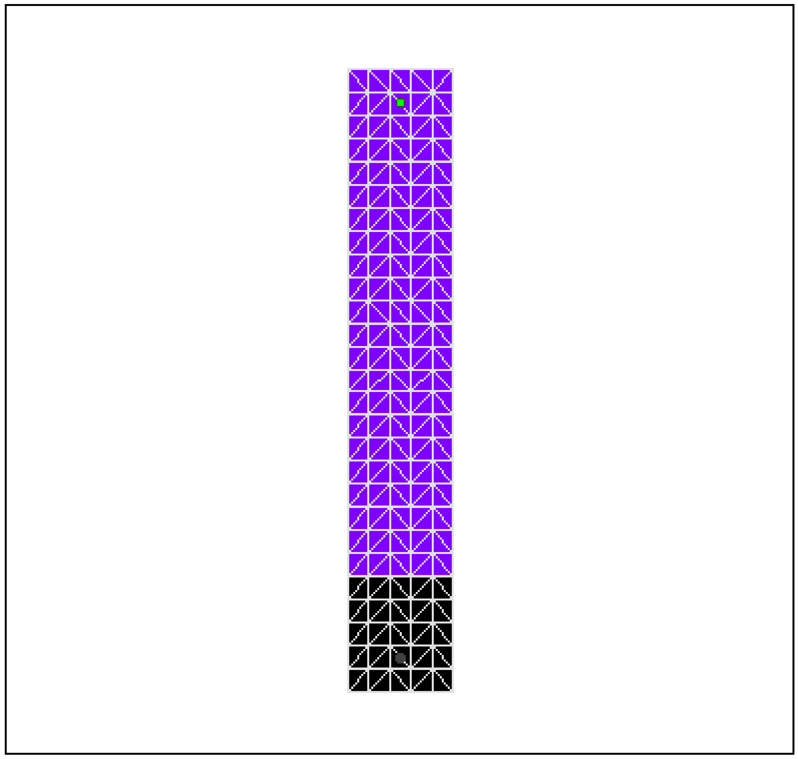
Section Name: ICEB Brad Wall 4

Loading Name: Moment Curvature

Analysis Type: Moment Curvature

Section Details:

X Centroid:	-1.84E-15 mm
Y Centroid:	-4.69E-15 mm
Section Area:	135.0E+3 mm^2

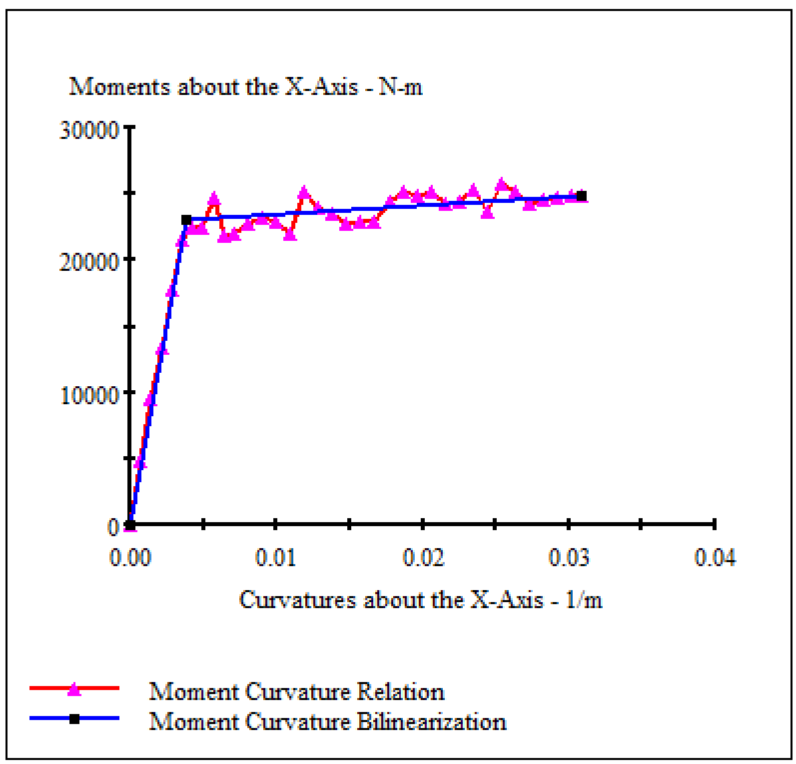


Loading Details:

Incrementing Loads:	Mxx Only
Number of Points:	35
Analysis Strategy:	Displacement Control

Analysis Results:

Failing Material:	ICEB
Failure Strain:	2.500E-3 Compression
Curvature at Initial Load:	0 1/m
Curvature at First Yield:	3.469E-3 1/m
Ultimate Curvature:	30.88E-3 1/m
Moment at First Yield:	21.19E+3 N-m
Ultimate Moment:	24.86E+3 N-m
Centroid Strain at Yield:	.5032E-3 Ten
Centroid Strain at Ultimate:	11.05E-3 Ten
N.A. at First Yield:	145.1 mm
N.A. at Ultimate:	357.9 mm
Energy per Length:	693.3 N
Effective Yield Curvature:	3.778E-3 1/m
Effective Yield Moment:	23.08E+3 N-m
Over Strength Factor:	1.077
EI Effective:	6.109E+6 N-m^2
Yield EI Effective:	65.92E+3 N-m^2
Bilinear Harding Slope:	1.079 %
Curvature Ductility:	8.175



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Section Name: ICEB Brad Wall 5

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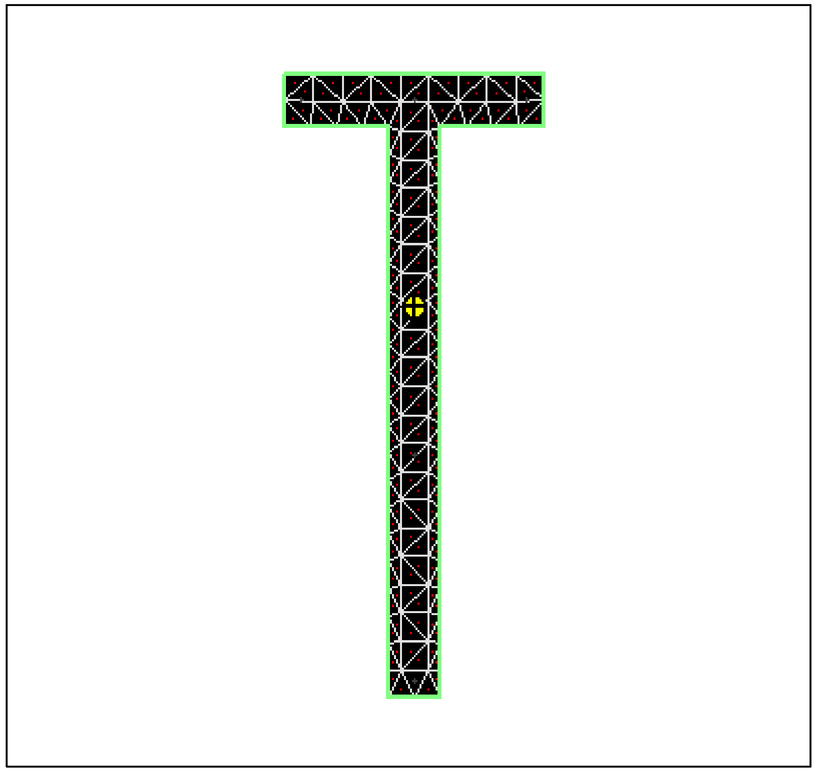
Section Details:

X Centroid:	-2.87E-15 mm
Y Centroid:	-521.0 mm
Section Area:	360.0E+3 mm^2
EI gross about X:	94.88E+6 N-m^2
EI gross about Y:	6.011E+6 N-m^2
I trans (ICEB) about X:	19.80E+6 cm^4
I trans (ICEB) about Y:	1.254E+6 cm^4
Reinforcing Bar Area:	471.2 mm^2
Percent Longitudinal Steel:	.1309 %
Overall Width:	750.0 mm
Overall Height:	1800 mm
Number of Fibers:	159
Number of Bars:	6
Number of Materials:	2



Material Types and Names:

User Defined:	ICEB
Strain Hardening Steel:	Grade 40

Comments:

User Comments

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Section Name: ICEB Brad Wall 5

Loading Name: Moment Curvature

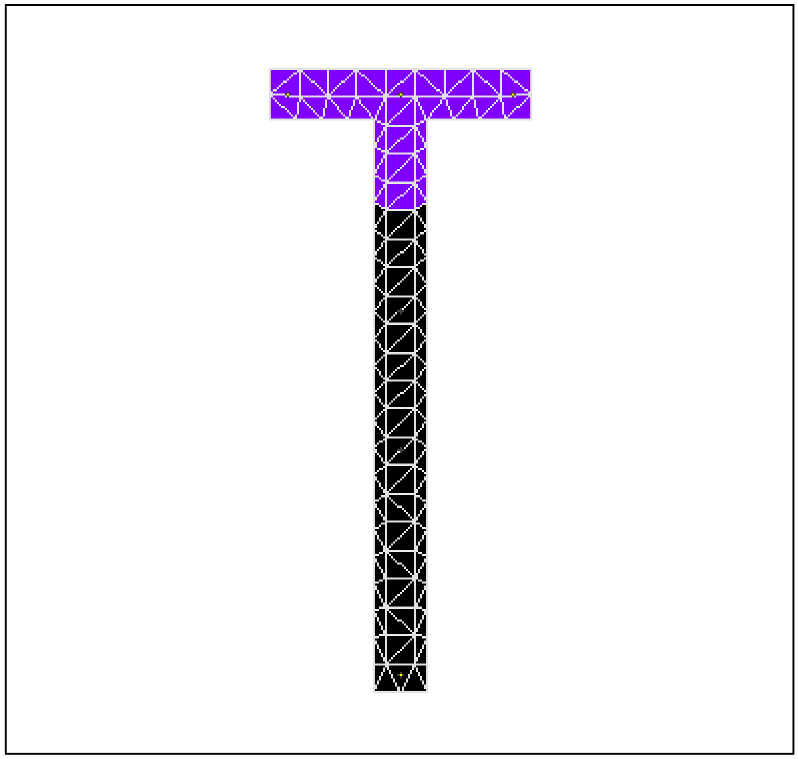
Analysis Type: Moment Curvature

Section Details:

X Centroid: -2.87E-15 mm

Y Centroid: -521.0 mm

Section Area: 360.0E+3 mm^2



Loading Details:

Incrementing Loads: Mxx Only

Number of Points: 8

Analysis Strategy: Displacement Control

Analysis Results:

Failing Material: ICEB

Failure Strain: 2.500E-3 Compression

Curvature at Initial Load: 0 1/m

Curvature at First Yield: 2.247E-3 1/m

Ultimate Curvature: 3.319E-3 1/m

Moment at First Yield: 138.5E+3 N-m

Ultimate Moment: 146.7E+3 N-m

Centroid Strain at Yield: .5515E-3 Ten

Centroid Strain at Ultimate: 1.156E-3 Ten

N.A. at First Yield: 245.4 mm

N.A. at Ultimate: 348.4 mm

Energy per Length: 308.6 N

Effective Yield Curvature: 2.254E-3 1/m

Effective Yield Moment: 138.9E+3 N-m

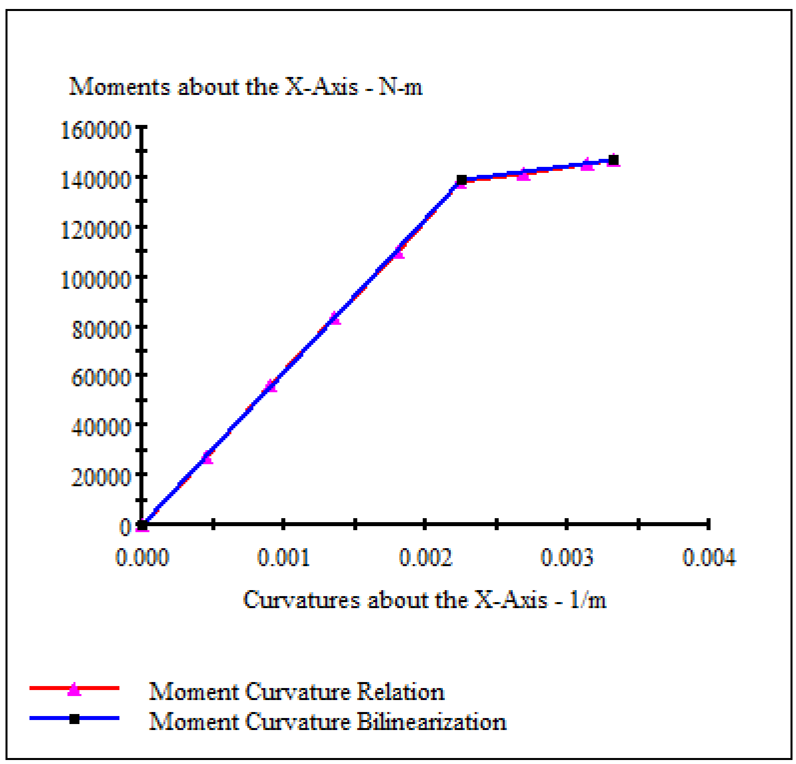
Over Strength Factor: 1.057

EI Effective: 61.61E+6 N-m^2

Yield EI Effective: 7.376E+6 N-m^2

Bilinear Harding Slope: 11.97 %

Curvature Ductility: 1.472



Comments:

User Comments

XTRACT Analysis Report - Educational

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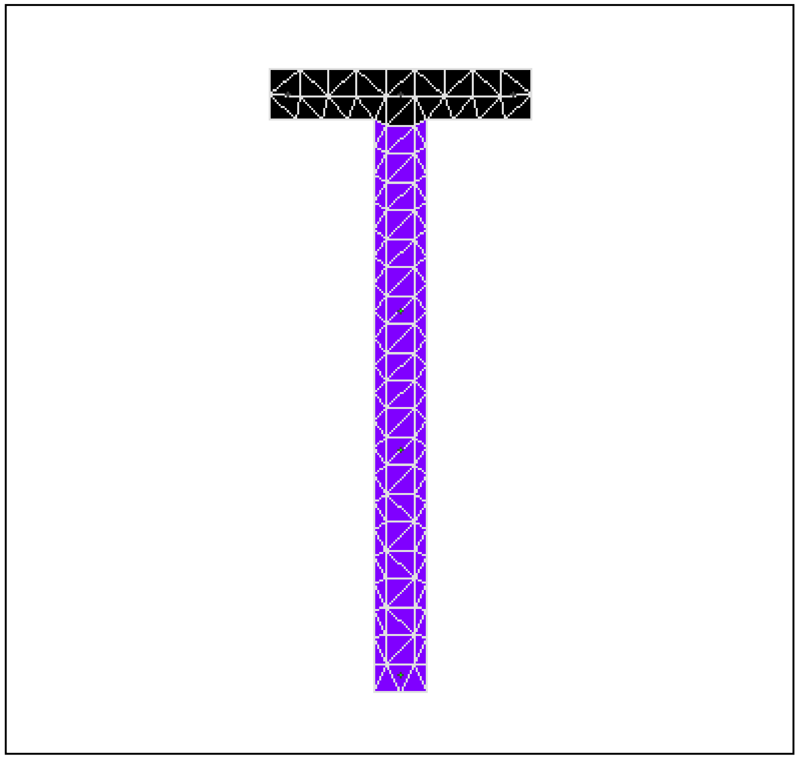
Section Name: ICEB Brad Wall 5

Loading Name: Flange Tension

Analysis Type: Moment Curvature

Section Details:

X Centroid:	-2.87E-15 mm
Y Centroid:	-521.0 mm
Section Area:	360.0E+3 mm^2

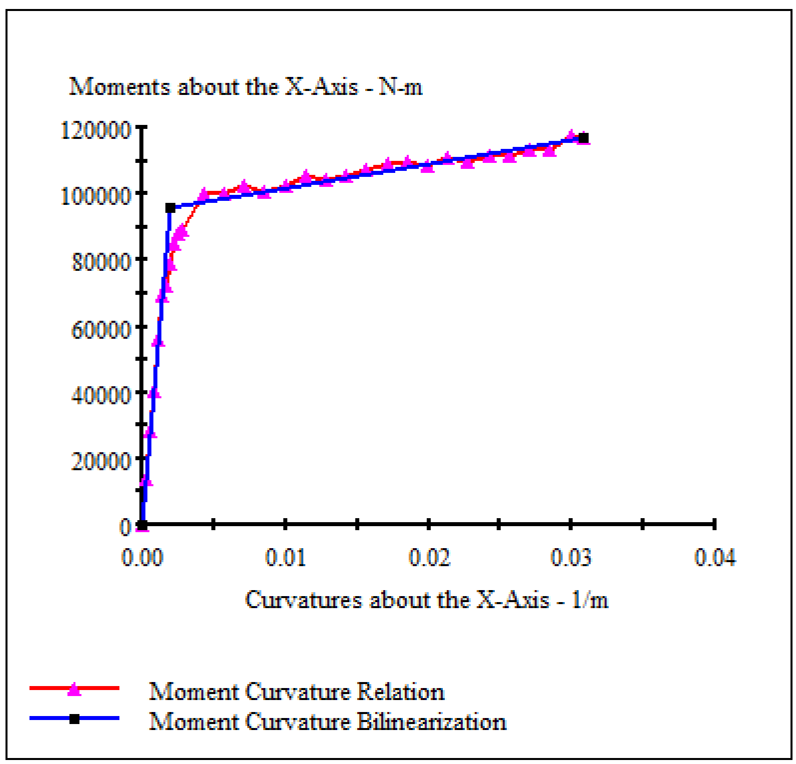


Loading Details:

Incrementing Loads:	Mxx Only
Number of Points:	30
Analysis Strategy:	Displacement Control

Analysis Results:

Failing Material:	ICEB
Failure Strain:	2.500E-3 Compression
Curvature at Initial Load:	0 1/m
Curvature at First Yield:	-1.413E-3 1/m
Ultimate Curvature:	-30.77E-3 1/m
Moment at First Yield:	-68.71E+3 N-m
Ultimate Moment:	-116.8E+3 N-m
Centroid Strain at Yield:	.3666E-3 Ten
Centroid Strain at Ultimate:	17.37E-3 Ten
N.A. at First Yield:	-259.5 mm
N.A. at Ultimate:	-564.7 mm
Energy per Length:	3156 N
Effective Yield Curvature:	1.970E-3 1/m
Effective Yield Moment:	95.81E+3 N-m
Over Strength Factor:	-1.219
EI Effective:	48.64E+6 N-m^2
Yield EI Effective:	730.3E+3 N-m^2
Bilinear Harding Slope:	1.501 %
Curvature Ductility:	15.62



Comments:

User Comments

Appendix B: Philippines Quadruplex Design Spreadsheets and Verification Calculations

Verification Calculations for Quadruplex Design Spreadsheet

Lateral force resisting system calculations referenced from Caritas Core House Calculations by Daniel Mostrales, attached to these calculations.

Seismic Weight of Structure: NSCP 208.5.1.1

$$\underset{\text{ICEB walls + components}}{373.7\ kN} + \underset{\text{Roof components}}{47.21\ kN} + \underset{\text{Roof live load}}{0.1(78.69\ kN)} = 428.79\ kN$$

$C_s = 0.48$ (ASCE 7 12.8-2)

$\boxed{V_{base} = C_s W = 205.82\ kN}$

$f'_m = 3\ MPa$ (Bland 2011 Figure 3.12)

$f_y = 206\ MPa$ (Gr. 30 steel)

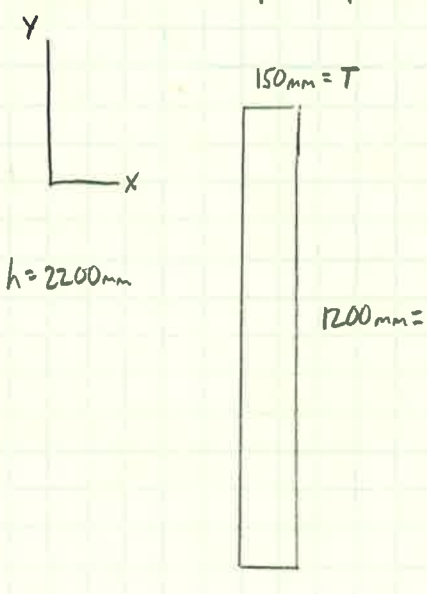
Elastic modulus $E_m = 137 f'_m$ (Bland 2011 p.44)

$E_m = 411\ MPa$

Shear modulus $E_v = 0.4 E_m = 164\ MPa$

Determination of Wall Stiffnesses and Percentage of Total Stiffness

Wall 2: Loading in y direction (north-south)



Plan view

$A = 1200mm \times 150mm = 180000\ mm^2$

$I_{xx} = \frac{1}{12} T L^3 = 2.16 \times 10^{10}\ mm^4$

$I_{yy} = \frac{1}{12} L T^3 = 3.38 \times 10^8\ mm^4$

$K_{ix} = \frac{3EI}{h^3} + \frac{E_v(A)}{1.2h}$ (Fix-free condition)

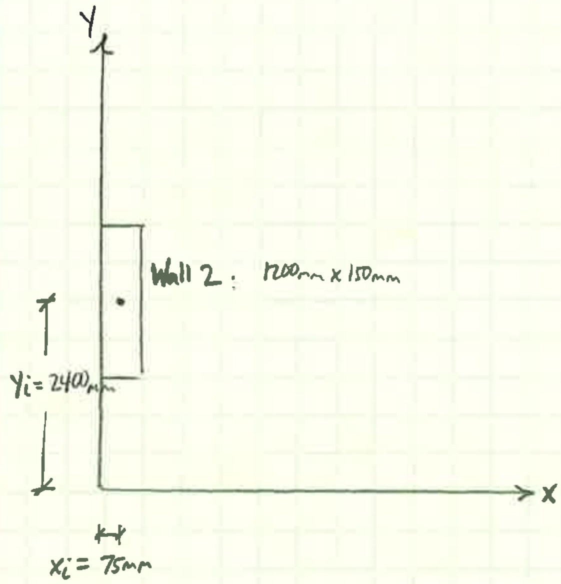
$$= \frac{3(411\ MPa)(2.16 \times 10^{10}\ mm^4)}{(2200\ mm)^3} + \frac{164\ MPa\ (180000\ mm^2)}{1.2(2200\ mm)} = 13.71\ \frac{kN}{mm}$$

Neglect out of plane stiffness (K_{iy})

Total stiffness in Y direction (xx local stiffness) $= 2105.47\ \frac{kN}{mm}$

$\frac{K_{ix}}{\Sigma K_{ix}} = 0.0065$ i.e. this wall contributes 0.65% of total stiffness.

Determination of Center of Rigidity



$(K_{iy})(x_i) = 0$ (Out of plane stiffness K_{iy} neglected)

$(K_{ix})(y_i) = 13.71 \frac{kN}{mm} \cdot 2400mm = 32904.7\ kN$

$\Sigma K_{ix} = 2105.97 kN/mm$

$\Sigma K_{iy} = 1272.96\ kN/mm$

$\frac{\Sigma K_{iy}(x_i)}{\Sigma K_{iy}} = x_r$ (x coordinate of center of rigidity)

$= 4125\ mm$

$\frac{\Sigma K_{ix}(y_i)}{\Sigma K_{ix}} = Y_r$ (Y coordinate of center of rigidity)

$= 5925mm$

X_m (x coordinate of center of mass)

$= 4125mm$

Y_m (y coordinate of center of mass)

$= 5925mm$

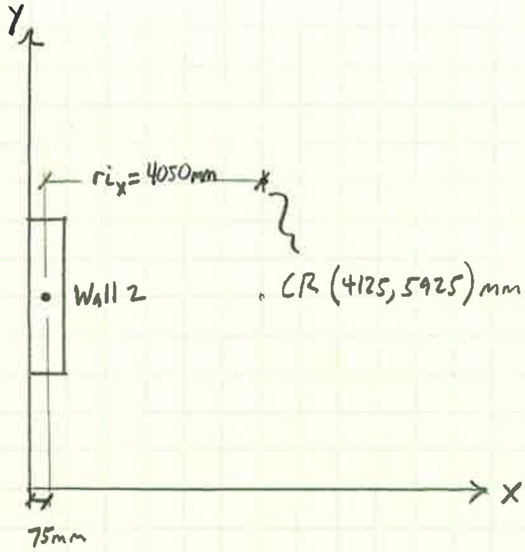
The Center of Rigidity and Center of Mass are concurrent.

$e_x = |x_m - x_r| = 0$

$e_y = |y_m - y_r| = 0$

$\therefore$ no additional torsional moment on the structure.

Determination of Rotational Stiffness J_r



$(r_{ix}^2)(K_{ix}) = 224883081.1$ kN-mm

$\Sigma(r_{ix}^2)(K_{ix}) = 2356289619$ kN-mm

$\Sigma(r_{iy}^2)(K_{iy}) = 937463382$ kN-mm

$J_r = \Sigma(r_{ix}^2)(K_{ix}) + \Sigma(r_{iy}^2)(K_{iy})$

$= \underline{3293753002 \text{ kN-mm}}$ (Rotational stiffness)

Torsional Moments: Vbase (x direction) = Vbase (y direction) = 205.82 kN

$M_{Tx} = V_{base}(x\ direction)\, e_y = 0$

$M_{Ty} = V_{base}(y\ direction)\, e_x = 0$

$(e_x = e_y = 0$, no eccentricity$)$

Determination of Seismic Force Distribution

Wall 2: Torsional Shear: $\frac{r_i \cdot K_i \cdot M_i}{J_r} = 0$ ($M_{Ty} = 0$, torsional moment caused by V_{base} (y direction))

Direct Shear: $V_{base} \cdot \frac{K_i}{\Sigma K_i} = 0.0065 \cdot 205.82\,kN = 1.34\,kN$

Total = Torsional Shear + Direct Shear = 1.34 kN

Determination of Elastic Deflection

- Timoshenko beam theory for cantilever beam with point load P at end
- Assume uncracked section properties

$$\delta_{xe} = \frac{PL^3}{3EI}\left(1 + 0.6(1+\mu)\frac{h^2}{L^2}\right)$$

For wall 2:

$P = 1.34\,kN$

$L = 2200\,mm$

$E = 411\,MPa$

$I = I_{xx} = 2.16 \times 10^{10}\,mm^4$

$\mu = 0.2$ (rectangular section)

$h = 1200\,mm$

$\delta_{xe} = 0.65\,mm$

Story Drift Deflection: ASCE 7-05 12.8-15

$$\delta_x = \frac{C_d\,\delta_{xe}}{I}$$

$C_d = 6.5$ (See estimate in Appendix)

$\delta_{xe} = 0.65\,mm$ (Wall 2)

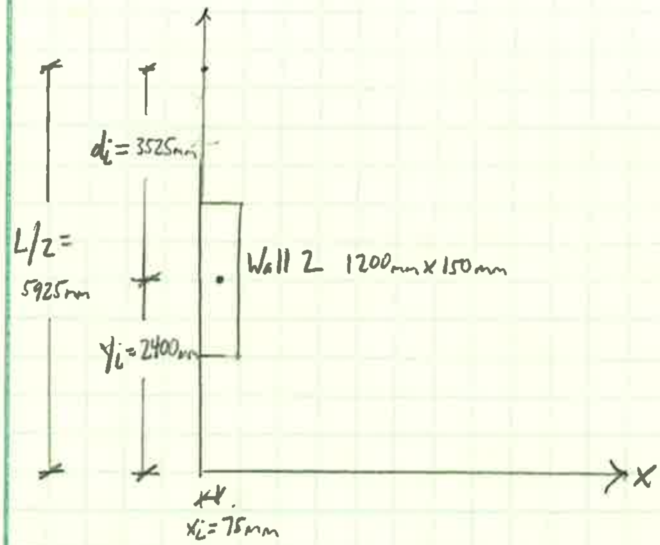
$I = 1.0$ ASCE 7-05 Table 11.5-1

$\delta_x = 4.23\,mm$

$\frac{\delta_x}{h_x} = \frac{4.23\,mm}{2200\,mm} = 0.0019 < 0.007$ ok ✓ ASCE 7-05 Table 12.12-1

↖ height of wall @ story x

Additional Axial Force on Walls from Overturning Moments



For Wall 2:

$I_i = 2.16 \times 10^{10} mm^4$

$A_i = 180000 \, mm^2$

$A_i(d_i)^2 = 2.23 \times 10^{12} mm^4$

$I_i + A_i(d_i)^2 = 2.26 \times 10^{12} mm^4$

$V_{base} = 205.82 \, kN$ (base shear)

$M_{ov} = V_{base}(H)$ (overturning moment due to base shear)

$= 205.82 \, kN \, (2600 mm)$

avg. wall height

$M_{ov} = 535129.92 \, kN\text{-}mm$

$$P_{ov} = \frac{M_{ov} \cdot A_i \cdot d_i}{\Sigma(I_i + A_i(d_i)^2)}$$

$= 3.73 \, kN$

Determination of Gravity Loads

Wall 2: Length = 1200 mm Height = 2200 mm

Tributary length: 900 mm left
1950 mm right
2850 mm

Roof dead load = 0.06 kPa Steel roof trusses / purlins (NSCP Table 204-2)
0.1 kPa Sheet metal (Ga. 26) (NSCP Table 204-2)
0.15 kPa Ceiling joists / plywood (NSCP Table 204-1)
0.31 kPa

Roof live load = 0.6 kPa (NSCP Table 205-3)

Dead load = 0.31 kPa · 2.85 m = 0.88 kN/m

Live load = 0.6 kPa · 2.85 m = 1.71 kN/m

Earthquake load = 1.34 kN
P_{ov} = 3.73 kN

Load cases: ASCE 7-05 2.3.2

$\underline{1.4D} = 1.4(0.88\,kN/m)(1.2m) = 1.48\,kN$ axial 0 moment

$\underline{1.2D + 1.6L_r} = 1.2(0.88\,kN/m)(1.2m) + 1.6(1.71\,kN/m)(1.2m) = 4.56\,kN$ axial 0 moment

$\underline{1.2D + 1.0E + 0.5L_r} = 1.2(0.88\,kN/m)(1.2m) + 0.2S_{DS}D + 0.5(1.71\,kN/m)(1.2m) + P_{ov}$

$= 1.27 + 0.2(0.96)(0.88\,kN/m)(1.2m) + 1.03 + 3.73\,kN$

$= 6.23\,kN$ axial ← governs

Moment: $1.34\,kN(2.2m) \doteq 2.95\,kN\text{-}m$ ← governs

Note: – Account for vertical acceleration $0.2S_{DS}D$ per ASCE 7-05 12.4.2.2
– ρ in 12.4.2.3 (5) and 12.4.2.1 = 1.0 per Table 12.3-3

$\underline{0.9D - 1.0E} = 0.9(0.88\,kN/m)(1.2m) - 0.2S_{DS}D - P_{ov}$

$= 0.9(1.06\,kN) - 0.2(0.96)(1.06\,kN) - 3.73\,kN$

$= -2.98\,kN$ (tension) ← tension governing case

Moment: $1.34\,kN(2.2m) = 2.95\,kN\text{-}m$

Compare with P-M interaction diagrams generated in XTRACT

Shear Design of Walls

Wall 2: Length = 1200m Height = 2200mm $f'_m = 3\ MPa$

Width = 150mm

$A_g = 180000\ mm^2$

3 flexural bars (10mm) = $235.62\ mm^2$

$A_n = A_g - 235.62\ mm^2$

$= 179764.38\ mm^2$

$d_v = L - 75mm = 1200mm - 75mm = 1125mm$

$M_u = 2947.84\ kN\text{-}mm$ (governing moment)

$$V_u = \frac{\left(\frac{(M_u)(F.S.)(1.25)}{\phi}\right)}{h}$$

← Overstrength factor MSJC 1.17.3.2.6.1.1

$\phi = 0.9$ (Flexural reduction factor for maximum probable moment)

$F.S. = 1.10$ (Designer choice)

$h = 2200mm$

$$V_u = \frac{\left(\frac{(2947.84\ kN\text{-}mm)(1.1)(1.25)}{0.9}\right)}{2200mm}$$

$= 2.05\ kN$ (to capacity protect against shear failure)

$P_u = -2.98\ kN$ ← smallest axial load case for shear design

$\phi V_n \geq V_u$ $\phi = 0.75$ (shear reduction factor)

$V_n = V_{nICEB} + V_{ns}$ (based on MSJC Eq. 3-19)

$$V_{nICEB} = k\left[0.083\left[4.0 - 1.75\frac{M_u}{V_u d_v}\right]A_n\sqrt{f'_m} + 0.25P_u\right]$$

$k = 0.4$ fully grouted

$k = 0.2$ partially grouted

$V_{nICEB} = 8947.65\ N$ for partially grouted wall

Shear Design of Walls continued

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

$A_v = 78.54\,mm^2$ (10 mm bar)

$s = 600\,mm$ (every 6 courses)

$f_y = 206\,MPa$ (Gr. 30)

$d_v = 1125\,mm$

$V_{ns} = 15168\,N$

Check max V_n: $\frac{M_u}{V_u d_v} \geq 1.00$ (MSJC Eq. 3-21)

$\frac{M_u}{V_u d_v} = 1.69$

$V_n \leq 0.33 A_n \sqrt{f'_m}$

$V_n \leq 102749\,N$

$V_{n_{ICEB}} + V_{ns} = 24115.65\,N < 102749\,N$ ok ✓

$\phi V_n \geq V_u$

$(0.75)(24.115\,kN) \geq 2.05\,kN$

$18.09\,kN \geq 2.05\,kN$ ok

Check min. steel ratio (MSJC 1.17.3.2.6 (c))

Horizontal: $\frac{A_V}{bh} \geq 0.0007$

$\frac{78.54\,mm^2 \text{ (4 bars horizontally)}}{(150\,mm)(2200\,mm)} = 0.0009 \geq 0.0007$ ok ✓

Vertical: $0.0007 \leq \frac{A_s}{bd} \leq 0.007 \leftarrow$ Max from Section 3.3.3.5.1

$\frac{78.54\,mm^2 \text{ (3 bars vertically)}}{(150\,mm)(1125\,mm)} = 0.0013$ ok ✓

Design of Anchors ACI 318-08 Appendix D.6

For anchors at bottom and top of wall

Wall 2: V_u (non capacity protection) = 1.34 kN

3 bars, 12 mm diameter for flexural reinforcement

$f_y = 206$ MPa (Gr. 30) $\quad f_{uta} = 1.9 f_y = 391.4$ MPa $\quad f'_c = 22$ MPa (3190 psi)

$$V_{sa} = n A_{se,v} f_{uta} \qquad (D-19)$$

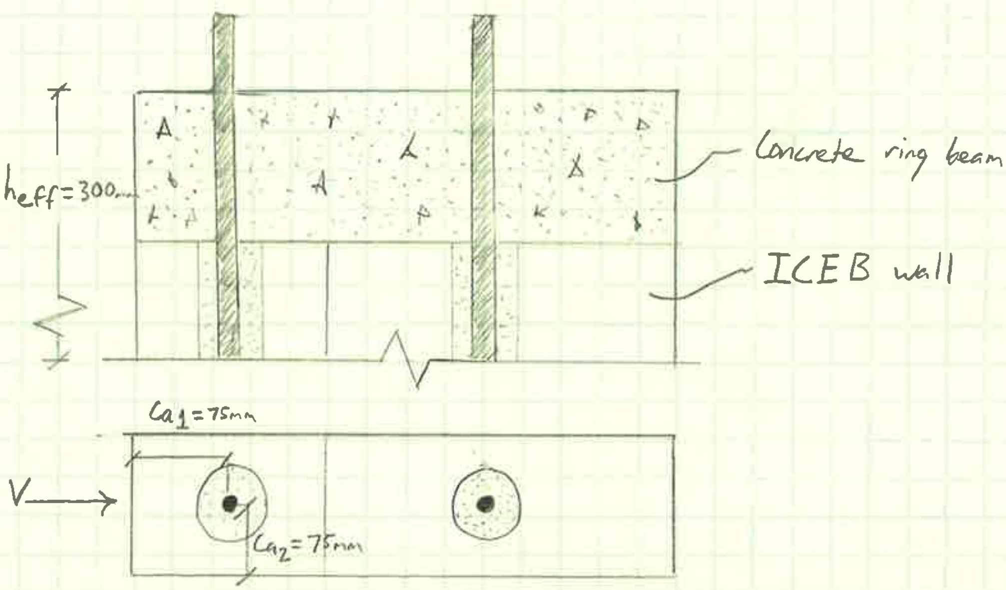
$$= (3)(0.25\pi\, 12^2)(391.4\text{ MPa})$$

$$= 132798.89 \text{ N}$$

$$\phi V_{sa} \geq V_u$$

$$0.75(132798.89) = 99599 \geq 1340 \text{ N ok}$$

Basic Concrete Breakout Strength of a Single Anchor in Cracked Concrete



$$V_b = n\left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right)\lambda\sqrt{f'_c}\,(c_{a1})^{1.5} \qquad (D-24)$$

$l_e = h_{eff} = 300$ mm $\quad n = 3$ bars $\quad$ Note: Multiply by 4.448 for Newtons and input in U.S. units: inches, lbs

$d_a = 12$mm

$f'_c = 22$ MPa

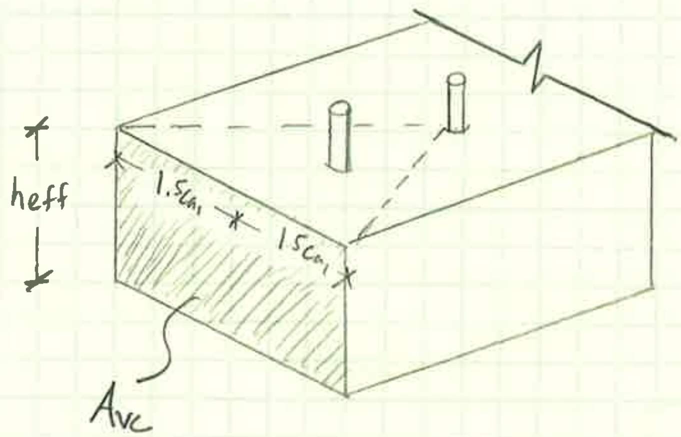
$c_{a1} = 75$ mm

$V_b = 35029.90$ N

Nominal Concrete Breakout Strength of a Group of Anchors

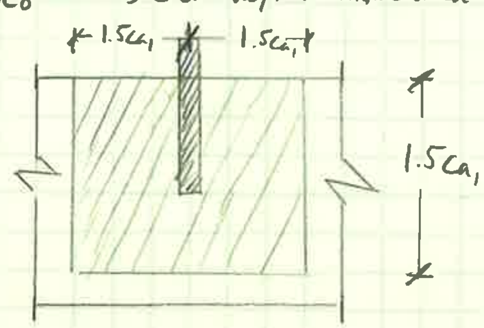
$$V_{cbg} = \frac{A_{Vc}}{A_{Vco}} \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b \quad (D-22)$$

A_{Vc} = projected failure area of group of anchors



$$A_{Vc} = 3c_{a1}(h_{eff})$$
$$= 3(75\text{mm})(300\text{mm})$$
$$= 67500\text{mm}^2$$

A_{Vco} = Projected depth failure area of a single anchor



$$A_{Vco} = 4.5c_{a1}^2$$
$$= 4.5(75\text{mm})^2 = 25312.5\text{ mm}^2$$

Nominal Concrete Breakout Strength of a Group of Anchors continued

$\psi_{ec,V}$: modification factor for anchor groups loaded eccentrically in shear

$$\psi_{ec,V} = \frac{1}{\left(1+\frac{2e'_V}{3c_{a1}}\right)} \qquad (D\text{-}26)$$

$e'_V = 0$ per Fig. RD.6.2.5

$\psi_{ec,V} = 1.0$

$\psi_{ed,V}$: modification factor for edge effects for a group of anchors

$$\psi_{ed,V} = 0.7 + 0.3\frac{c_{a2}}{1.5c_{a1}} \qquad (D\text{-}28)$$

$c_{a1} = c_{a2} = 75\text{mm}$

$\psi_{ed,V} = 0.9$

$\psi_{c,V}$: Concrete cracking modification factor

$\psi_{c,V} = 1.0$ (anchors in cracked concrete with no supplementary reinforcement)
(conservative) (D.6.2.7)

$\psi_{H,V}$: Modification factor for thin members, $h_a < 1.5c_{a1}$

$\psi_{H,V} = 1.0$ $(h_a \geq 1.5c_{a1})$ (D-29)

$$V_{cbg} = \frac{67500\text{mm}^2}{25312.5\text{mm}^2}(1.0)(0.9)(1.0)(1.0)(35029.90\text{N})$$

$$= 84071.77\ \text{N}$$

$$\phi V_n = \min(V_{cbg}, V_b)\phi$$

$$= 0.75(35029.90\text{N})$$

$$= 26272\ \text{N}$$

$V_u = 1.34\text{ kN} \implies \phi V_n = 26.2\text{ kN} \geq V_u = 1.34\text{ kN}$ ok ✓

Tension Breakout of Rebar

Basic concrete breakout strength of a single anchor in tension in cracked concrete:

$$N_b = k_c \lambda \sqrt{f'_c}\, h_{ef}^{1.5} \quad (D\text{-}7)$$

$k_c = 24$ for cast anchors

Wall 2: $f'_c = 3000$ psi

$h_{ef} = 12''$

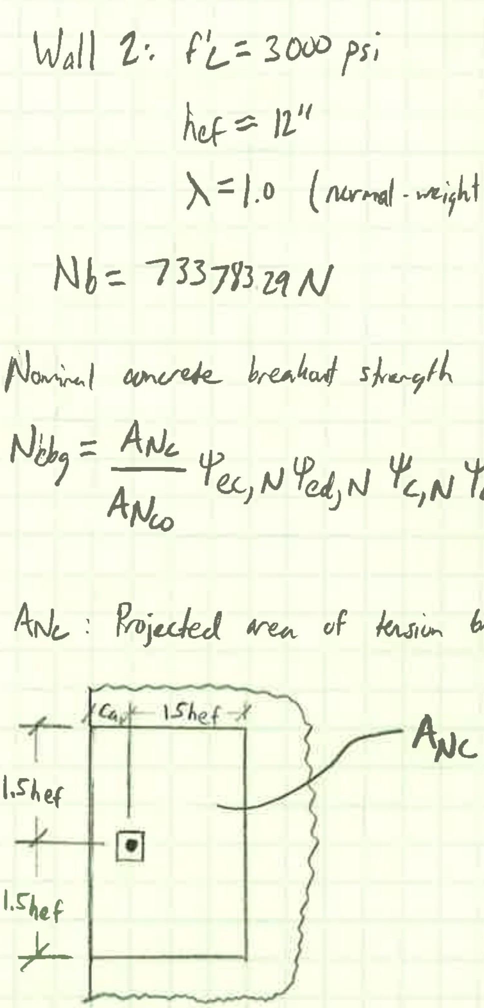
$\lambda = 1.0$ (normal-weight concrete)

$N_b = 733783.29\ N$

Nominal concrete breakout strength of a group of anchors:

$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b \quad (D\text{-}5)$$

A_{Nc}: Projected area of tension breakout of one anchor ($c_{a1} < 1.5 h_{ef}$)

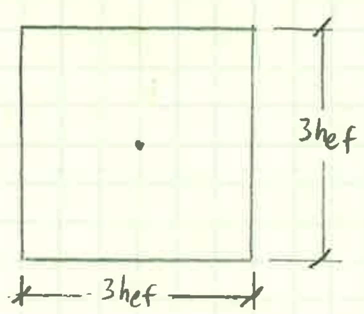


$c_{a1} = 75$ mm

$h_{ef} = 300$ mm

$A_{Nc} = 472500\ mm^2$

A_{Nco}: Projected area of tension failure cone



$A_{Nco} = 810000\ mm^2$

Tension Breakout of Rebar continued

$\Psi_{ec,N}$: Modification factor for anchor groups loaded eccentrically in tension.

$$\Psi_{ec,N} = \frac{1}{\left(1 + \frac{2e'_N}{3h_{ef}}\right)} \quad (D\text{-}9)$$

$e'_N = 0$ so $\Psi_{ec,N} = 1.0$

$\Psi_{ed,N}$: Modification factor for edge effects for single anchors or anchor groups loaded in tension.

$$\Psi_{ed,N} = 0.7 + 0.3\frac{c_{a,min}}{1.5h_{ef}} \quad (\text{for } c_{a,min} < 1.5h_{ef}) \quad (D\text{-}11)$$

$c_{a,min} = 75\text{ mm}$

$h_{ef} = 300\text{ mm}$

$\Psi_{ed,N} = 0.75$

$\Psi_{c,N}$: Modification factor for anchors in tension in uncracked areas

$\Psi_{c,N} = 1.0$ (cracked) (D.5.2.6)

$\Psi_{cp,N}$: Modification factor for post installed anchors designed for uncracked concrete. (D.5.2.7)

$\Psi_{cp,N} = 1.0$ (cast in anchors)

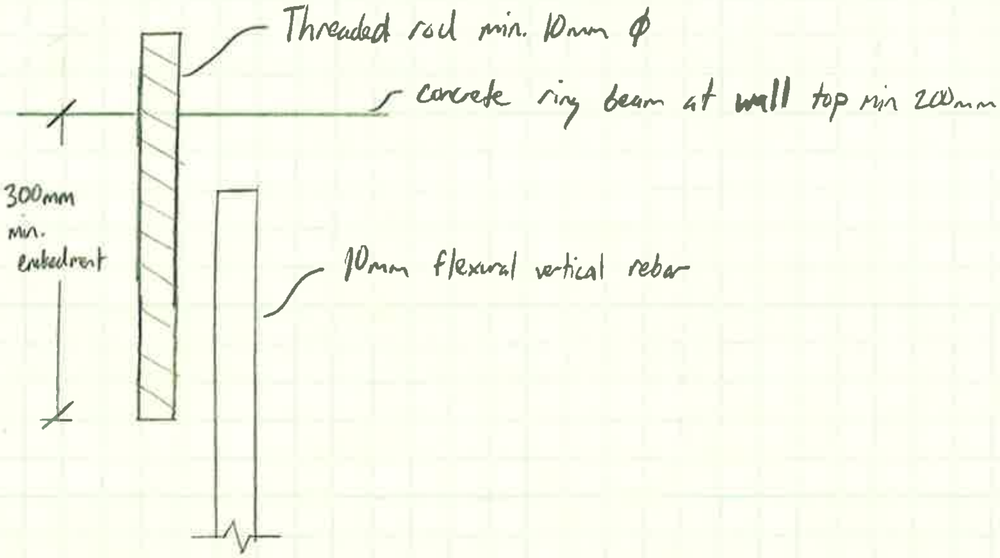
$$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \quad (D\text{-}5)$$

$N_{cbg} = 321030.19\text{ N}$

$$N_u = \frac{M_u}{L} = \frac{2947.84\text{ N·m}}{1.2\text{ m}} = 2456.53\text{ N}$$

$\phi N = 0.75(321030.19\text{ N}) = 240773\text{ N} \geq N_u = 2456.53\text{ N}$ ok ✓

Final Anchor Design:



Use this detail to connect roof diaphragm to wall with steel brackets at each rafter.

Development Length into foundation of hooked rebar

ACI 12.5

$$l_{dh} = \left(\frac{0.02\,\psi_e\, f_y}{\lambda\sqrt{f'_c}}\right) d_b$$

$\psi_e = 1.0$ (no epoxy)

$\lambda = 1.0$ (normal weight concrete)

$f'_c = 3190$ psi (22 MPa)

$f_y = 30$ ksi (206 MPa)

$d_b = 3/8''$ (10mm)

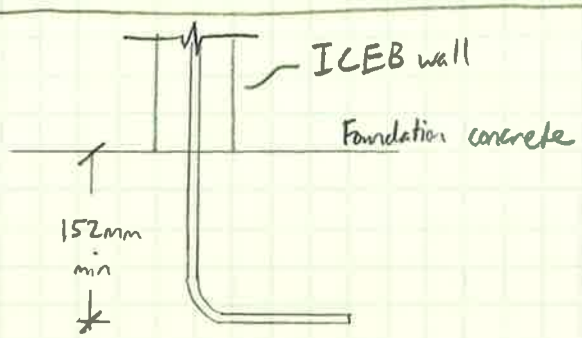
$l_{dh} = 3.98''$

ACI 12.5.3

Use 0.7 reduction factor

$l_{dh} = 2.79''$ but must be minimum $6''$ or 152.4mm

Use 152 mm min for all foundation hook embedments.



Design for out of plane bending

Wall 2:

Length = 1200mm Height = 2200mm

$F_p = 0.4 W_p I S_{DS}$

W_p = Tributary weight of wall

$= 450mm \times 2200mm \times 2.62\ kN/m^2$

$= 0.99m^2 \times 2.62\ kN/m^2$

$= 2.59\ kN$

$S_{DS} = 0.96$

$I = 1.0$ (Residential occupancy)

$F_p = 0.996\ kN$

$M = 0.996\ kN\ (2.2m)$

$= 2.19\ kN\text{-}m$

Compare with XTRACT output for out of plane loading

Rigid vs. flexible diaphragm

Flexible diaphragm analysis

$V_{base} = 205.82$ kN

Diaphragm Length: 11850 mm

Diaphragm Width: 8250 mm

Diaphragm Area: 97762500 mm^2

Force in XX direction = $V_{base} \cdot \dfrac{Width}{Area} = 0.0174$ kN/mm

Force in YY direction = $V_{base} \cdot \dfrac{Length}{Area} = 0.0249$ kN/mm

Example for Wall Line 1:

Length of diaphragm in considered direction: 8250 mm

Force: 600 mm x 0.0174 kN/mm = 10.42 kN

Unit shear on diaphragm = Force / Length = 10.42 kN / 8250 mm = 1.26 kN/m

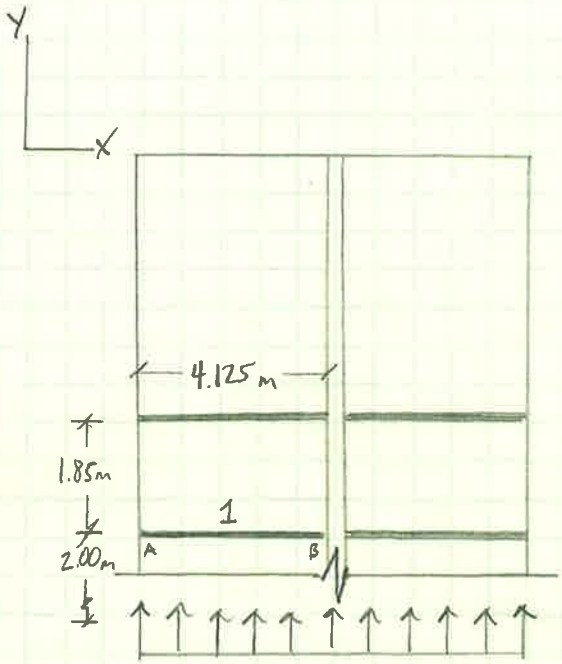
Individual wall forces

Wall 2: 1200 mm long x 4.34 kN/m diaphragm shear =

2.61 kN

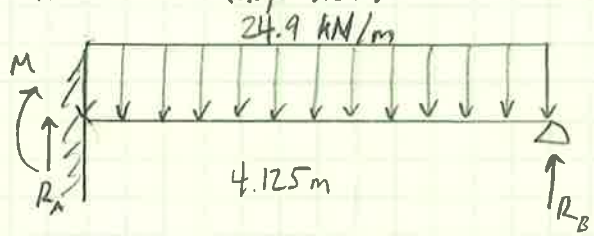
Design walls for shear and flexure similar to rigid diaphragm case.

Possible roof diaphragm connection:



24.9 kN/m (from flexible diaphragm analysis)

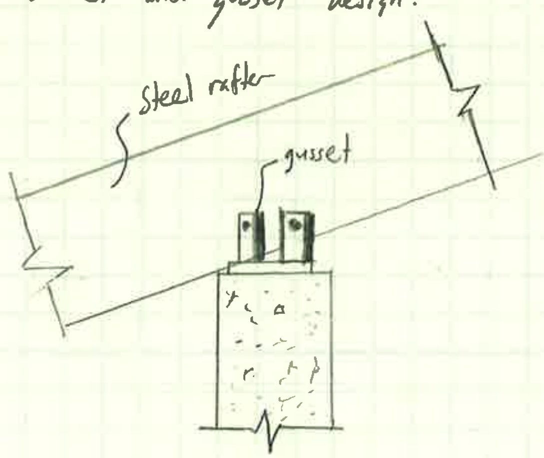
Rafter 1 (top view)



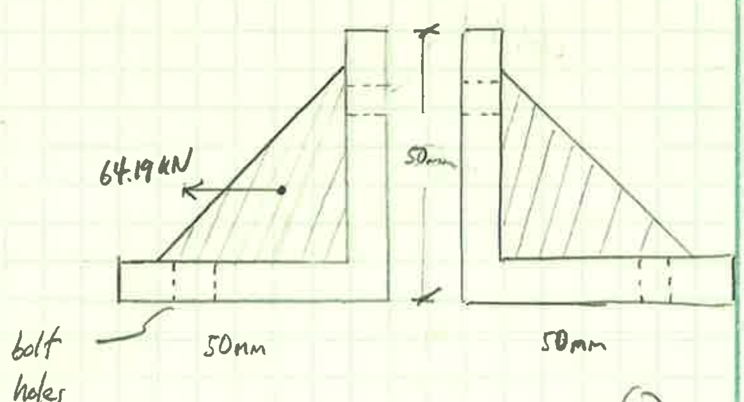
$$M = \frac{wl^2}{8} = 53 \text{ kN-m}$$

$R_A = \frac{5}{8}wl$ $R_B = \frac{3}{8}wl$ $R_A = 64.19$ kN $R_B = 38.52$ kN

Bracket and gusset design:



Section view of connection (rafter not shown)



Roof connection continued:

Check local buckling (AISC Specification Table B4.1 Case 14)

$b/t \leq 1.49\sqrt{E/F_y}$

$b/t \leq 32.75$

$b = 40mm$

$t = 5mm$

$b/t = 8 \leq 32.75$ ok ✓

Check gusset weld:

Shear:

$R_n = 0.6 F_{Exx} \times \frac{\sqrt{2}}{2} \times \frac{D}{16} \times l$ (J.2-3)

F_{Exx} = weld metal strength = 30 ksi

D = weld size in sixteenths of an inch = 4/16"

l = length of weld in inches ≒ 4"

$R_n = 10.81\ k = 48.11\ kN/\text{gusset} \times 2\ \text{gussets} = 96.2\ kN$

$\phi = 0.75$ so $0.75(96.2) = 72.17\ kN \geq 64.19\ kN$

Tension / Compression: Not necessary to check per Table J2.5

Check base metal (J.2-2)

Shear: $R_n = F_{BM} A_{BM}$

F_{BM} = yield stress of base metal

$= f_{uta} = 391.6\ MPa$

A_{BM} = area of base metal

$= 40mm\ (6mm)$

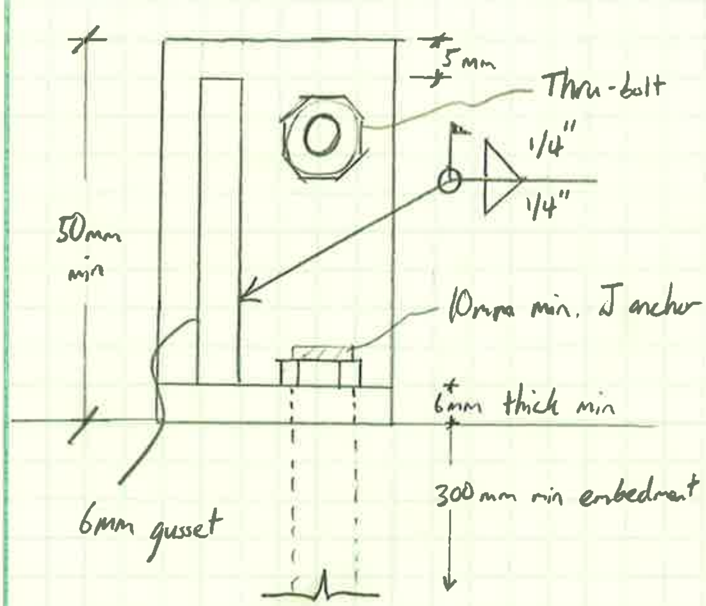
$= 240mm^2$

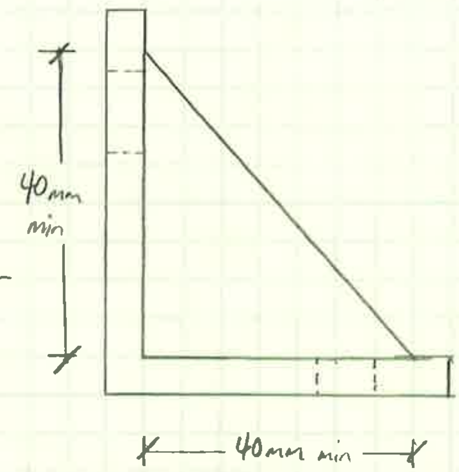
$R_n = 93.9\ kN$

$\phi R_n = 0.75(93.9\ kN) = 70.48\ kN \geq 64.19\ kN$ ok ✓

Final design for possible roofing connection

Rafter not shown





Use two gusset connections per rafter, one each side.

Rigid Diaphragm Analysis

Determination of Wall Stiffnesses and Percentage of Total Stiffness

Wall	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	A	B	C	D	E	F	G	Total
L (mm)	600	1200	750	750	1200	600	1200	13650	1200	600	1200	750	750	1200	600	1200	1200	1200	1200	900	900	3900	10050	3900	900	900	
T (mm)	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	150	
h (mm)	2200	2200	2200	2200	2200	2200	2500	2800	2500	2200	2200	2200	2200	2200	2200	2500	2500	2500	2500	2650	2650	2650	2500	2650	2650	2650	
A (mm^2)	135000	180000	112500	112500	180000	135000	270000	2047500	270000	135000	180000	112500	112500	180000	135000	270000	270000	180000	180000	135000	135000	585000	1507500	585000	135000	135000	
I_{xx} (mm^4)	4.30E+09	2.16E+10	5.27E+09	5.27E+09	2.16E+10	4.30E+09	5.35E+10	3.18E+13	5.35E+10	4.30E+09	2.16E+10	5.27E+09	5.27E+09	2.16E+10	4.30E+09	5.35E+10	5.35E+10	2.16E+10	2.16E+10	2.53E+08	2.53E+08	1.10E+09	2.83E+09	1.10E+09	2.53E+08	2.53E+08	
I_{yy} (mm^4)	2.02E+09	3.38E+08	2.11E+08	2.11E+08	3.38E+08	2.02E+09	6.08E+09	3.84E+09	6.08E+09	2.02E+09	3.38E+08	2.11E+08	2.11E+08	3.38E+08	2.02E+09	6.08E+09	6.08E+09	3.38E+08	3.38E+08	9.11E+09	9.11E+09	7.41E+11	1.27E+13	7.41E+11	9.11E+09	9.11E+09	
Kix (kN/mm)	8.90	13.71	7.62	7.62	13.71	8.90	19.02	1885.83	19.02	8.90	13.71	7.62	7.62	13.71	8.90	19.02	19.02	11.57	11.57								2105.97
Kiy (kN/mm)																				7.58	7.58	79.37	1083.88	79.37	7.58	7.58	1272.96
Kix/ΣKix	0.0042	0.0065	0.0036	0.0036	0.0065	0.0042	0.0090	0.8955	0.0090	0.0042	0.0065	0.0036	0.0036	0.0065	0.0042	0.0090	0.0090	0.0055	0.0055								1
Kiy/ΣKiy																				0.0060	0.0060	0.0624	0.8515	0.0624	0.0060	0.0060	1

Elastic Modulus E_m (Mpa)	411
Shear Modulus E_v (Mpa)	164.4
Weight of Building (kN)	428.79

Determination of Center of Rigidity

Wall	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	A	B	C	D	E	F	G	Total
yi (mm)	225	2400	4575	7275	9450	11625	11325	5925	11325	11625	9450	7275	4575	2400	225	525	525	5925	5925	1425	1425	4575	5925	7425	10425	10425	
xi (mm)	150	75	75	75	75	150	2175	4125	6150	8100	8175	8175	8175	8175	8100	2100	6150	2175	6075	2625	5625	4125	4125	4125	2625	5625	
Kiy*xi	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	19905.38	42654.39	327407.10	4471009.55	327407.10	19905.38	42654.39	5250943.32
Kix*yi	2003.57	32904.70	34844.70	55408.78	129562.27	103517.63	215376.49	11173565.06	215376.49	103517.63	129562.27	55408.78	34844.70	32904.70	2003.57	9984.34	9984.34	68543.36	68543.36	0	0	0	0	0	0	0	12477856.73
ΣKiy	1272.96																										
ΣKix	2105.97																										

x coor. of CR	xr	4125
y coor. of CR	yr	5925
x coor. of CM	xm	4125
y coor. of CM	ym	5925
eccentricity x +5% accidental torsion	ex	412.5
eccentricity y +5% accidental torsion	ey	593
Along XX	Mtx (kN-mm)	121948
Along YY	Mty (kN-mm)	84900.42

Determination of Rotational Stiffness J_r

Wall	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	A	B	C	D	E	F	G	Total
ri_x	3975	4050	4050	4050	4050	3975	1950	0	2025	3975	4050	4050	4050	4050	3975	2025	2025	1950	1950								
ri_y																				4500	4500	1350	0	1500	4500	4500	
ri^2	15800625	16402500	16402500	16402500	16402500	15800625	3802500	0	4100625	15800625	16402500	16402500	16402500	16402500	15800625	4100625	4100625	3802500	3802500								
																				20250000	20250000	1822500	0	2250000	20250000	20250000	
ri^2*Kix	140700499.7	224883081.1	124926801.4	124926801.4	224883081.1	140700500	72315154.08	0	77984833.32	140700499.7	224883081.1	124926801.4	124926801.4	224883081.1	140700499.7	77984833.32	77984833.32	43989218.21	43989218.21								2356289619
ri^2*Kiy																				153555819.2	153555819.2	144654412	0	178585694	153555819	153555819	937463382.3
Jr (kN-mm)																											3293753002

Distribution of Seismic Forces

Wall	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	A	B	C	D	E	F	G
ri*Ki*Mi/Jr	0.91	1.43	0.80	0.80	1.43	0.91	0.96	0.00	0.99	0.91	1.43	0.80	0.80	1.43	0.91	0.99	0.99	0.58	0.58	1.26	1.26	3.97	0.00	4.41	1.26	1.26
Vbase*Ki/ΣKi	0.87	1.34	0.74	0.74	1.34	0.87	1.86	184.31	1.86	0.87	1.34	0.74	0.74	1.34	0.87	1.86	1.86	1.13	1.13	1.23	1.23	12.83	175.25	12.83	1.23	1.23
Sum	1.78	2.77	1.54	1.54	2.77	1.78	2.81	184.31	2.85	1.78	2.77	1.54	1.54	2.77	1.78	2.85	2.85	1.71	1.71	2.49	2.49	16.80	175.25	17.24	2.49	2.49
Force*Height	3.92	6.10	3.39	3.39	6.10	3.92	7.04	516.05	7.13	3.92	6.10	3.39	3.39	6.10	3.92	7.13	7.13	4.28	4.28	6.60	6.60	44.52	438.12	45.69	6.60	6.60

Elastic Deflection of Walls

Wall	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	A	B	C	D	E	F	G
$(PL^3/3EI)*(1+0.6*(1+\mu)*h^2/L^2)$	3.77	1.35	2.73	2.73	1.35	3.77	0.78	1.87	0.79	3.77	1.35	2.73	2.73	1.35	3.77	0.79	0.79	1.17	1.17	4.47	4.47	0.88	2.21	0.90	4.47	4.47
$\delta=\Delta*Cd$	11.32	4.04	8.20	8.20	4.04	11.32	2.33	5.61	2.36	11.32	4.04	8.20	8.20	4.04	11.32	2.36	2.36	3.51	3.51	13.40	13.40	2.63	6.63	2.69	13.40	13.40
δ/h	0.0051	0.0018	0.0037	0.0037	0.0018	0.0051	0.0009	0.0020	0.0009	0.0051	0.0018	0.0037	0.0037	0.0018	0.0051	0.0009	0.0009	0.0014	0.0014	0.0051	0.0051	0.0010	0.0027	0.0010	0.0051	0.0051
$\delta/h < 0.007$	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK	OK

Additional Axial Force from Overturning Moment

Wall	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	A	B	C	D	E	F	G	Total
L (mm)	600	1200	750	750	1200	600	1200	13650	1200	600	1200	750	750	1200	600	1200	1200	1200	1200	900	900	3900	10050	3900	900	900	
A (mm^2)	135000	180000	112500	112500	180000	135000	270000	2047500	270000	135000	180000	112500	112500	180000	135000	270000	270000	180000	180000	135000	135000	585000	1507500	585000	135000	135000	
Ii (strong axis)	4.30E+09	2.16E+10	5.27E+09	5.27E+09	2.16E+10	4.30E+09	5.35E+10	3.18E+13	5.35E+10	4.30E+09	2.16E+10	5.27E+09	5.27E+09	2.16E+10	4.30E+09	5.35E+10	5.35E+10	2.16E+10	2.16E+10								
																				9112500000	9112500000	7.415E+11	1.2688E+13	7.415E+11	9.113E+09	9.113E+09	
yi (mm)	225	2400	4575	7275	9450	11625	11325	5925	11325	11625	9450	7275	4575	2400	225	525	525	5925	5925	1425	1425	4575	5925	7425	10425	10425	
xi (mm)	150	75	75	75	75	150	2175	4125	6150	8100	8175	8175	8175	8175	8100	2100	6150	2175	6075	2625	5625	4125	4125	4125	2625	5625	
di (mm)	5700	3525	1350	1350	3525	5700	5400	0	5400	5700	3525	1350	1350	3525	5700	5400	5400	0	0								
																				1500	1500	0	0	0	1500	1500	
A*di^2 (mm^4)	4.38615E+12	2.23661E+12	2.05031E+11	2.05031E+11	2.23661E+12	4.3862E+12	7.8732E+12	0	7.8732E+12	4.38615E+12	2.23661E+12	2.05031E+11	2.05031E+11	2.23661E+12	4.38615E+12	7.8732E+12	7.8732E+12	0	0								
																				3.0375E+11	3.0375E+11	0	0	0	3.038E+11	3.038E+11	
Ii+A*di^2 (mm^4)	4.39E+12	2.26E+12	2.10E+11	2.10E+11	2.26E+12	4.39E+12	7.93E+12	3.18E+13	7.93E+12	4.39E+12	2.26E+12	2.10E+11	2.10E+11	2.26E+12	4.39E+12	7.93E+12	7.93E+12	2.16E+10	2.16E+10								9.10E+13
																				3.12863E+11	3.12863E+11	7.415E+11	1.2688E+13	7.415E+11	3.129E+11	3.129E+11	1.54E+13
Pov (kN)	4.53	3.73	0.89	0.89	3.73	4.53	8.58	0.00	8.58	4.53	3.73	0.89	0.89	3.73	4.53	8.58	8.58	0.00	0.00	7.03	7.03	0.00	0.00	0.00	7.03	7.03	

Determination of Gravity Loads

											Load Cases							
											1.4D		1.2D + 1.6Lr		1.2D + E + 0.5Lr		0.9D - E	
Wall	Trib Left (mm)	Trib Right (mm)	Roof Dead Load (kPa)	Roof Live Load (kPa)	DL (kN/m)	Lr (kN/m)	E (kN)	Pov (kN)	Length (mm)	Height (mm)	Axial (kN)	Moment (kN-m)	Axial (kN)	Moment (kN-m)	Axial (kN)	Moment (kN-m)	Axial (kN)	Moment (kN-m)
1	900	1050	0.31	0.6	0.60	1.17	1.78	4.53	600	2200	0.51	0	1.56	0	5.38	3.92	-4.27	3.92
2	900	1950	0.31	0.6	0.88	1.71	2.77	3.73	1200	2200	1.48	0	4.56	0	6.23	6.10	-2.98	6.10
3	900	1050	0.31	0.6	0.60	1.17	1.54	0.89	750	2200	0.63	0	1.95	0	1.96	3.39	-0.57	3.39
4	900	1050	0.31	0.6	0.60	1.17	1.54	0.89	750	2200	0.63	0	1.95	0	1.96	3.39	-0.57	3.39
5	900	1950	0.31	0.6	0.88	1.71	2.77	3.73	1200	2200	1.48	0	4.56	0	6.23	6.10	-2.98	6.10
6	900	1050	0.31	0.6	0.60	1.17	1.78	4.53	600	2200	0.51	0	1.56	0	5.38	3.92	-4.27	3.92
7	975	900	0.31	0.6	0.58	1.13	2.81	8.58	1200	2500	0.98	0	3.00	0	10.22	7.04	-8.08	7.04
8	1950	1950	0.31	0.6	1.21	2.34	184.31	0.00	13650	2800	23.10	0	70.91	0	38.94	516.05	11.68	516.05
9	975	900	0.31	0.6	0.58	1.13	2.85	8.58	1200	2500	0.98	0	3.00	0	10.22	7.13	-8.08	7.13
10	900	1050	0.31	0.6	0.60	1.17	1.78	4.53	600	2200	0.51	0	1.56	0	5.38	3.92	-4.27	3.92
11	900	1950	0.31	0.6	0.88	1.71	2.77	3.73	1200	2200	1.48	0	4.56	0	6.23	6.10	-2.98	6.10
12	900	1050	0.31	0.6	0.60	1.17	1.54	0.89	750	2200	0.63	0	1.95	0	1.96	3.39	-0.57	3.39
13	900	1050	0.31	0.6	0.60	1.17	1.54	0.89	750	2200	0.63	0	1.95	0	1.96	3.39	-0.57	3.39
14	900	1950	0.31	0.6	0.88	1.71	2.77	3.73	1200	2200	1.48	0	4.56	0	6.23	6.10	-2.98	6.10
15	900	1050	0.31	0.6	0.60	1.17	1.78	4.53	600	2200	0.51	0	1.56	0	5.38	3.92	-4.27	3.92
16	975	900	0.31	0.6	0.58	1.13	2.85	8.58	1200	2500	0.98	0	3.00	0	10.22	7.13	-8.08	7.13
17	975	900	0.31	0.6	0.58	1.13	2.85	8.58	1200	2500	0.98	0	3.00	0	10.22	7.13	-8.08	7.13
18	975	900	0.31	0.6	0.58	1.13	1.71	0.00	1200	2500	0.98	0	3.00	0	1.65	4.28	0.49	4.28
19	975	900	0.31	0.6	0.58	1.13	1.71	0.00	1200	2500	0.98	0	3.00	0	1.65	4.28	0.49	4.28
A	0	0	0.31	1.6	0.00	0.00	2.49	7.03	900	2650	0.00	0	0.00	1	7.03	6.60	-7.03	6.60
B	0	0	0.31	2.6	0.00	0.00	2.49	7.03	900	2650	0.00	0	0.00	2	7.03	6.60	-7.03	6.60
C	0	0	0.31	3.6	0.00	0.00	16.80	0.00	3900	2650	0.00	0	0.00	3	0.00	44.52	0.00	44.52
D	0	0	0.31	4.6	0.00	0.00	175.25	0.00	10050	2500	0.00	0	0.00	4	0.00	438.12	0.00	438.12
E	0	0	0.31	5.6	0.00	0.00	17.24	0.00	3900	2650	0.00	0	0.00	5	0.00	45.69	0.00	45.69
F	0	0	0.31	6.6	0.00	0.00	2.49	7.03	900	2650	0.00	0	0.00	6	7.03	6.60	-7.03	6.60
G	0	0	0.31	7.6	0.00	0.00	2.49	7.03	900	2650	0.00	0	0.00	7	7.03	6.60	-7.03	6.60

Shear Design

General Parameters

f'm (MPa)	3
Shear Bars (mm)	10
Flexural Bars (mm)	10
As (mm^2)	78.54
Av (mm^2)	78.54
fy (MPa)	206
Grout	Partially Grouted
ϕ (shear)	0.75
Safety Factor	1.1
ϕ (flexure)	0.9

								Seismic Shear Demand														
Wall	L (mm)	T (mm)	Height (mm)	Ag (mm^2)	Flexural Bars	An (mm^2)	dv (mm)	Mu (kN-mm)	Vu (kN)	Pu (N)	Vert. Spacing (mm)	VnICEB (N)	Vs (N)	Mu/(Vudv)	Vn (kN)	ϕVn (kN)	ϕVn > Vu	F.S.	Vertical Steel Ratio Check		Horizontal Steel Ratio Check	
1	600	150	2200	135000	3	134764.38	525	3921.84	2.72	-4269.43	600	0.00	7078.40	4.19	7.08	5.31	OK	1.95	0.003	OK	0.00095	OK
2	1200	150	2200	180000	3	179764.38	1125	6096.63	4.23	-2981.52	600	8947.65	15168.00	1.96	24.12	18.09	OK	4.27	0.001	OK	0.00095	OK
3	750	150	2200	112500	2	112342.92	675	3386.79	2.35	-572.34	600	832.74	9100.80	3.26	9.93	7.45	OK	3.17	0.002	OK	0.00095	OK
4	750	150	2200	112500	2	112342.92	675	3386.79	2.35	-572.34	600	832.74	9100.80	3.26	9.93	7.45	OK	3.17	0.002	OK	0.00095	OK
5	1200	150	2200	180000	3	179764.38	1125	6096.63	4.23	-2981.52	600	8947.65	15168.00	1.96	24.12	18.09	OK	4.27	0.001	OK	0.00095	OK
6	600	150	2200	135000	3	134764.38	525	3921.84	2.72	-4269.43	600	0.00	7078.40	4.19	7.08	5.31	OK	1.95	0.003	OK	0.00095	OK
7	1200	150	2500	180000	3	179764.38	1125	7036.35	4.30	-8082.16	600	7113.85	15168.00	2.22	22.28	16.71	OK	3.89	0.001	OK	0.00105	OK
8	13650	150	2800	2047500	31	2045065.27	13575	516054.64	281.58	11684.02	600	221891.19	183027.22	0.21	404.92	303.69	OK	1.08	0.001	OK	0.00093	OK
9	1200	150	2500	180000	3	179764.38	1125	7128.26	4.36	-8082.16	600	7113.85	15168.00	2.22	22.28	16.71	OK	3.84	0.001	OK	0.00105	OK
10	600	150	2200	135000	3	134764.38	525	3921.84	2.72	-4269.43	600	0.00	7078.40	4.19	7.08	5.31	OK	1.95	0.003	OK	0.00095	OK
11	1200	150	2200	180000	3	179764.38	1125	6096.63	4.23	-2981.52	600	8947.65	15168.00	1.96	24.12	18.09	OK	4.27	0.001	OK	0.00095	OK
12	750	150	2200	112500	2	112342.92	675	3386.79	2.35	-572.34	600	832.74	9100.80	3.26	9.93	7.45	OK	3.17	0.002	OK	0.00095	OK
13	750	150	2200	112500	2	112342.92	675	3386.79	2.35	-572.34	600	832.74	9100.80	3.26	9.93	7.45	OK	3.17	0.002	OK	0.00095	OK
14	1200	150	2200	180000	3	179764.38	1125	6096.63	4.23	-2981.52	600	8947.65	15168.00	1.96	24.12	18.09	OK	4.27	0.001	OK	0.00095	OK
15	600	150	2200	135000	3	134764.38	525	3921.84	2.72	-4269.43	600	0.00	7078.40	4.19	7.08	5.31	OK	1.95	0.003	OK	0.00095	OK
16	1200	150	2500	180000	3	179764.38	1125	7128.26	4.36	-8082.16	600	7113.85	15168.00	2.22	22.28	16.71	OK	3.84	0.001	OK	0.00105	OK
17	1200	150	2500	180000	3	179764.38	1125	7128.26	4.36	-8082.16	600	7113.85	15168.00	2.22	22.28	16.71	OK	3.84	0.001	OK	0.00105	OK
18	1200	150	2500	180000	3	179764.38	1125	4280.20	2.62	493.83	600	7542.65	15168.00	2.22	22.71	17.03	OK	6.51	0.001	OK	0.00105	OK
19	1200	150	2500	180000	3	179764.38	1125	4280.20	2.62	493.83	600	7542.65	15168.00	2.22	22.71	17.03	OK	6.51	0.001	OK	0.00105	OK
A	900	150	2650	135000	4	134685.84	825	6597.05	3.80	-7026.18	600	890.45	11123.20	3.21	12.01	9.01	OK	2.37	0.003	OK	0.00099	OK
B	900	150	2650	135000	4	134685.84	825	6597.05	3.80	-7026.18	600	890.45	11123.20	3.21	12.01	9.01	OK	2.37	0.003	OK	0.00099	OK
C	3900	150	2650	585000	10	584214.60	3825	44521.12	25.67	0.00	600	53859.35	51571.21	0.69	105.43	79.07	OK	3.08	0.001	OK	0.00099	OK
D	10050	150	2500	1507500	23	1505693.58	9975	438121.09	267.74	0.00	400	160738.81	201734.43	0.25	362.47	271.85	OK	1.02	0.001	OK	0.00147	OK
E	3900	150	2650	585000	10	584214.60	3825	45689.23	26.34	0.00	600	53859.35	51571.21	0.69	105.43	79.07	OK	3.00	0.001	OK	0.00099	OK
F	900	150	2650	135000	4	134685.84	825	6597.05	3.80	-7026.18	600	890.45	11123.20	3.21	12.01	9.01	OK	2.37	0.003	OK	0.00099	OK
G	900	150	2650	135000	4	134685.84	825	7000.00	4.04	-7026.18	600	890.45	11123.20	3.21	12.01	9.01	OK	2.23	0.003	OK	0.00099	OK

Design of Diaphragm and Wall Anchors ACI 318-08 D.6

General Parameters

Parameter	Value	Units	Parameter	Value	SI Units	Value	English Units
fy	206	MPa	ca1	75	mm	2.95	inches
f_{uta} (shear)	391.4	MPa	ca2	75	mm	2.95	inches
ϕ (shear)	0.75	-	hef	300	mm	11.81	inches
ϕ (tension)	0.90	-	f'c	22	Mpa	3.19	ksi

Shear Breakout Parameters

Parameter	Value	English Units (in^2)	ACI Reference
$\psi_{ec,V}$	1	-	D.6.2.5
$\psi_{ed,V}$	0.9	-	D.6.2.6
$\psi_{c,V}$	1	-	D.6.2.7
$\psi_{h,V}$	1	-	D.6.2.8
Avc (mm^2)	67500	104.63	D.6.2.1
Avco (mm^2)	25312.5	39.23	D.6.2.1

Tension Breakout Parameters

Parameter	Value	English Units (in^2)	ACI Reference
$\psi_{ec,N}$	1	-	D.5.2.4
$\psi_{ed,N}$	0.75	-	D.5.2.5
$\psi_{c,N}$	1	-	D.5.2.6
$\psi_{cp,N}$	1	-	D.5.2.7
Anc (mm^2)	472500	732.38	D.5.2.1
Anco (mm^2)	810000	1255.50	D.5.2.1

			Steel Strength in Shear ACI D.6.1				Shear Breakout ACI D.6.2				
Wall	Vu (kN)	# Bars	Bar Dia (mm)	Bar Dia (inches)	ϕVsa (N)	Check	Vb (N)	Vcbg (N)	ϕVn (N)	Check	F.S.
1	1.78	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	14.0
2	2.77	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	9.0
3	1.54	2	10	0.39	46110.73	OK	22110.23	53064.56	16582.67	OK	10.8
4	1.54	2	10	0.39	46110.73	OK	22110.23	53064.56	16582.67	OK	10.8
5	2.77	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	9.0
6	1.78	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	14.0
7	2.81	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	8.8
8	184.31	31	10	0.39	714716.26	OK	342708.61	822500.67	257031.46	OK	1.4
9	2.85	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	8.7
10	1.78	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	14.0
11	2.77	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	9.0
12	1.54	2	10	0.39	46110.73	OK	22110.23	53064.56	16582.67	OK	10.8
13	1.54	2	10	0.39	46110.73	OK	22110.23	53064.56	16582.67	OK	10.8
14	2.77	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	9.0
15	1.78	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	14.0
16	2.85	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	8.7
17	2.85	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	8.7
18	1.71	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	14.5
19	1.71	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	14.5
A	2.49	4	10	0.39	92221.45	OK	44220.47	106129.12	33165.35	OK	13.3
B	2.49	4	10	0.39	92221.45	OK	44220.47	106129.12	33165.35	OK	13.3
C	16.80	10	10	0.39	230553.63	OK	110551.17	265322.80	82913.37	OK	4.9
D	175.25	23	10	0.39	530273.35	OK	254267.68	610242.43	190700.76	OK	1.1
E	17.24	10	10	0.39	230553.63	OK	110551.17	265322.80	82913.37	OK	4.8
F	2.49	4	10	0.39	92221.45	OK	44220.47	106129.12	33165.35	OK	13.3
G	2.49	4	10	0.39	92221.45	OK	44220.47	106129.12	33165.35	OK	13.3

	Tension Breakout ACI D.5.2								Development Length into Foundation		
Wall	Nb (N)	Ncbg (N)	Mu (N-m)	L (m)	T (N)	ϕN (N)	Check	F.S.	ldh (ft)	ldh (inches)	ldh (mm)
1	733783.29	321030.19	3921.84	0.6	6536.41	288927.17	OK	44.20	0.24	2.92	74.05
2	733783.29	321030.19	6096.63	1.2	5080.52	288927.17	OK	56.87	0.24	2.92	74.05
3	489188.86	214020.13	3386.79	0.75	4515.72	192618.11	OK	42.66	0.24	2.92	74.05
4	489188.86	214020.13	3386.79	0.75	4515.72	192618.11	OK	42.66	0.24	2.92	74.05
5	733783.29	321030.19	6096.63	1.2	5080.52	288927.17	OK	56.87	0.24	2.92	74.05
6	733783.29	321030.19	3921.84	0.6	6536.41	288927.17	OK	44.20	0.24	2.92	74.05
7	733783.29	321030.19	7036.35	1.2	5863.62	288927.17	OK	49.27	0.24	2.92	74.05
8	7582427.37	3317311.98	516054.64	13.65	37806.20	2985580.78	OK	78.97	0.24	2.92	74.05
9	733783.29	321030.19	7128.26	1.2	5940.22	288927.17	OK	48.64	0.24	2.92	74.05
10	733783.29	321030.19	3921.84	0.6	6536.41	288927.17	OK	44.20	0.24	2.92	74.05
11	733783.29	321030.19	6096.63	1.2	5080.52	288927.17	OK	56.87	0.24	2.92	74.05
12	489188.86	214020.13	3386.79	0.75	4515.72	192618.11	OK	42.66	0.24	2.92	74.05
13	489188.86	214020.13	3386.79	0.75	4515.72	192618.11	OK	42.66	0.24	2.92	74.05
14	733783.29	321030.19	6096.63	1.2	5080.52	288927.17	OK	56.87	0.24	2.92	74.05
15	733783.29	321030.19	3921.84	0.6	6536.41	288927.17	OK	44.20	0.24	2.92	74.05
16	733783.29	321030.19	7128.26	1.2	5940.22	288927.17	OK	48.64	0.24	2.92	74.05
17	733783.29	321030.19	7128.26	1.2	5940.22	288927.17	OK	48.64	0.24	2.92	74.05
18	733783.29	321030.19	4280.20	1.2	3566.83	288927.17	OK	81.00	0.24	2.92	74.05
19	733783.29	321030.19	4280.20	1.2	3566.83	288927.17	OK	81.00	0.24	2.92	74.05
A	978377.73	428040.25	6597.05	0.9	7330.06	385236.23	OK	52.56	0.24	2.92	74.05
B	978377.73	428040.25	6597.05	0.9	7330.06	385236.23	OK	52.56	0.24	2.92	74.05
C	2445944.31	1070100.64	44521.12	3.9	11415.67	963090.57	OK	84.37	0.24	2.92	74.05
D	5625671.92	2461231.47	438121.09	10.05	43594.14	2215108.32	OK	50.81	0.24	2.92	74.05
E	2445944.31	1070100.64	45689.23	3.9	11715.19	963090.57	OK	82.21	0.24	2.92	74.05
F	978377.73	428040.25	6597.05	0.9	7330.06	385236.23	OK	52.56	0.24	2.92	74.05
G	978377.73	428040.25	7000.00	0.9	7777.78	385236.23	OK	49.53	0.24	2.92	74.05

Out of Plane Wall Forces

Wall	Unbraced Length L (mm)	T (mm)	H (mm)	Wp (kN)	Fp (between pilasters) (kN)	Pu (kN)	Mu (kN-m)
1	450	150	2200	2.59	1.00	5.38	2.19
2	450	150	2200	2.59	1.00	6.23	2.19
3	300	150	2200	1.73	0.66	1.96	1.46
4	300	150	2200	1.73	0.66	1.96	1.46
5	450	150	2200	2.59	1.00	6.23	2.19
6	450	150	2200	2.59	1.00	5.38	2.19
7	1050	150	2500	6.88	2.64	10.22	6.60
8	1950	150	2800	14.31	5.49	38.94	15.38
9	1050	150	2500	6.88	2.64	10.22	6.60
10	450	150	2200	2.59	1.00	5.38	2.19
11	450	150	2200	2.59	1.00	6.23	2.19
12	300	150	2200	1.73	0.66	1.96	1.46
13	750	150	2200	4.32	1.66	1.96	3.65
14	450	150	2200	2.59	1.00	6.23	2.19
15	450	150	2200	2.59	1.00	5.38	2.19
16	1050	150	2500	6.88	2.64	10.22	6.60
17	1050	150	2500	6.88	2.64	10.22	6.60
18	450	150	2500	2.95	1.13	1.65	2.83
19	450	150	2500	2.95	1.13	1.65	2.83
A	750	150	2650	5.21	2.00	7.03	5.30
B	750	150	2650	5.21	2.00	7.03	5.30
C	750	150	2650	5.21	2.00	0.00	5.30
D	1950	150	2500	12.77	4.90	0.00	12.26
E	750	150	2650	5.21	2.00	0.00	5.30
F	750	150	2650	5.21	2.00	7.03	5.30
G	750	150	2650	5.21	2.00	7.03	5.30

Flexible Diaphragm Analysis

General Parameters

Seismic Base Shear (kN)	Length of Diaphragm (mm)	Width of Diaphragm (mm)	Area of Diaphragm (mm^2)	Diaphragm Load (kN/mm)	
				Along XX	Along YY
205.82	11850	8250	97762500	0.017	0.025

Wall Line	Force (kN)	Diaphragm Length (mm)	Diaphragm Unit Shear (kN/m)
7	10.42	8250	1.26
6	37.78	8250	4.58
5	37.78	8250	4.58
4	20.84	8250	2.53
3	37.78	8250	4.58
2	37.78	8250	4.58
1	10.42	8250	1.26
A	51.47	11850	4.34
B	102.93	11850	8.69
C	51.47	11850	4.34

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	A	B	C	D	E	F	G
600	1200	750	750	1200	600	1200	13650	1200	600	1200	750	750	1200	600	1200	1200	1200	1200	900	900	3900	10050	3900	900	900
0.0043	0.0043	0.0043	0.0043	0.0043	0.0043	0.0013	0.0087	0.0013	0.0043	0.0043	0.0043	0.0043	0.0043	0.0043	0.0013	0.0013			0.0046	0.0046	0.0046	0.0025	0.0046	0.0046	0.0046
2.61	5.21	3.26	3.26	5.21	2.61	1.52	118.57	1.52	2.61	5.21	3.26	3.26	5.21	2.61	1.52	1.52			4.12	4.12	17.86	25.39	17.86	4.12	4.12

Determination of Gravity Loads (Flexible Diaphragm)

										1.4D		1.2D + 1.6Lr		1.2D + E + 0.5Lr		0.9D - E	
Trib Left (mm)	Trib Right (mm)	Roof Dead Load (kPa)	Roof Live Load (kPa)	DL (kN/m)	Lr (kN/m)	E (kN)	Pov (kN)	Length (mm)	Height (mm)	Axial (kN)	Moment (kN-m)	Axial (kN)	Moment (kN-m)	Axial (kN)	Moment (kN-m)	Axial (kN)	Moment (kN-m)
900	1050	0.31	0.6	0.60	1.17	2.61	9.56	600	2200	0.51	0	1.56	0	10.41	5.73	-9.23	5.73
900	1950	0.31	0.6	0.88	1.71	5.21	9.56	1200	2200	1.48	0	4.56	0	12.06	11.47	-8.60	11.47
900	1050	0.31	0.6	0.60	1.17	3.26	9.56	750	2200	0.63	0	1.95	0	10.62	7.17	-9.15	7.17
900	1050	0.31	0.6	0.60	1.17	3.26	9.56	750	2200	0.63	0	1.95	0	10.62	7.17	-9.15	7.17
900	1950	0.31	0.6	0.88	1.71	5.21	9.56	1200	2200	1.48	0	4.56	0	12.06	11.47	-8.60	11.47
900	1050	0.31	0.6	0.60	1.17	2.61	9.56	600	2200	0.51	0	1.56	0	10.41	5.73	-9.23	5.73
975	900	0.31	0.6	0.58	1.13	1.52	3.16	1200	2500	0.98	0	3.00	0	4.80	3.79	-2.53	3.79
1950	1950	0.31	0.6	1.21	2.34	118.57	24.32	13650	2800	23.10	0	70.91	0	63.26	332.00	-9.47	332.00
975	900	0.31	0.6	0.58	1.13	1.52	3.16	1200	2500	0.98	0	3.00	0	4.80	3.79	-2.53	3.79
900	1050	0.31	0.6	0.60	1.17	2.61	9.56	600	2200	0.51	0	1.56	0	10.41	5.73	-9.23	5.73
900	1950	0.31	0.6	0.88	1.71	5.21	9.56	1200	2200	1.48	0	4.56	0	12.06	11.47	-8.60	11.47
900	1050	0.31	0.6	0.60	1.17	3.26	9.56	750	2200	0.63	0	1.95	0	10.62	7.17	-9.15	7.17
900	1050	0.31	0.6	0.60	1.17	3.26	9.56	750	2200	0.63	0	1.95	0	10.62	7.17	-9.15	7.17
900	1950	0.31	0.6	0.88	1.71	5.21	9.56	1200	2200	1.48	0	4.56	0	12.06	11.47	-8.60	11.47
900	1050	0.31	0.6	0.60	1.17	2.61	9.56	600	2200	0.51	0	1.56	0	10.41	5.73	-9.23	5.73
975	900	0.31	0.6	0.58	1.13	1.52	3.16	1200	2500	0.98	0	3.00	0	4.80	3.79	-2.53	3.79
975	900	0.31	0.6	0.58	1.13	1.52	3.16	1200	2500	0.98	0	3.00	0	4.80	3.79	-2.53	3.79
975	900	0.31	0.6	0.58	1.13	0.00	0.00	1200	2500	0.98	0	3.00	0	1.65	0.00	0.63	0.00
975	900	0.31	0.6	0.58	1.13	0.00	0.00	1200	2500	0.98	0	3.00	0	1.65	0.00	0.63	0.00
0	0	0.31	1.6	0.00	0.00	4.12	12.13	900	2650	0.00	0	0.00	0	12.13	10.92	-12.13	10.92
0	0	0.31	2.6	0.00	0.00	4.12	12.13	900	2650	0.00	0	0.00	0	12.13	10.92	-12.13	10.92
0	0	0.31	3.6	0.00	0.00	17.86	12.13	3900	2650	0.00	0	0.00	0	12.13	47.32	-12.13	47.32
0	0	0.31	4.6	0.00	0.00	25.39	6.32	10050	2500	0.00	0	0.00	0	6.32	63.47	-6.32	63.47
0	0	0.31	5.6	0.00	0.00	17.86	12.13	3900	2650	0.00	0	0.00	0	12.13	47.32	-12.13	47.32
0	0	0.31	6.6	0.00	0.00	4.12	12.13	900	2650	0.00	0	0.00	0	12.13	10.92	-12.13	10.92
0	0	0.31	7.6	0.00	0.00	4.12	12.13	900	2650	0.00	0	0.00	0	12.13	10.92	-12.13	10.92

Shear Design (Flexible Diaphragm)

f'm (MPa)	3
Shear Bars (mm)	10
Flexural Bars (mm)	10
As (mm^2)	78.54
Av (mm^2)	78.54
fy (MPa)	206
Grout	Partially Grouted
ϕ (shear)	0.75
Safety Factor	1.1
ϕ (flexure)	0.9

							Seismic Shear Demand								
Wall	L (mm)	T (mm)	Ag (mm^2)	Flexural Bars	An (mm^2)	dv (mm)	Mu (kN-mm)	Vu (kN)	Pu (N)	Vert. Spacing (mm)	VnICEB (N)	Vs (N)	ϕVn (kN)	ϕVn > Vu	F.S.
1	600	150	135000	3	134764.38	525	5733.06	3.98	10410.98	600	0.00	7078.40	5.31	OK	1.33
2	1200	150	180000	3	179764.38	1125	11466.13	7.96	12056.90	600	9699.57	15168.00	18.65	OK	2.34
3	750	150	112500	2	112342.92	675	7166.33	4.98	10624.95	600	1392.60	9100.80	7.87	OK	1.58
4	750	150	112500	2	112342.92	675	7166.33	4.98	10624.95	600	1392.60	9100.80	7.87	OK	1.58
5	1200	150	180000	3	179764.38	1125	11466.13	7.96	12056.90	600	9699.57	15168.00	18.65	OK	2.34
6	600	150	135000	3	134764.38	525	5733.06	3.98	10410.98	600	0.00	7078.40	5.31	OK	1.33
7	1200	150	180000	3	179764.38	300	3789.54	2.32	4803.87	600	0.00	4044.80	3.03	OK	1.31
8	13650	150	2047500	31	2045065.27	13575	331996.49	181.15	63264.55	600	224470.22	183027.22	305.62	OK	1.69
9	1200	150	180000	3	179764.38	300	3789.54	2.32	4803.87	600	0.00	4044.80	3.03	OK	1.31
10	600	150	135000	3	134764.38	525	5733.06	3.98	10410.98	600	0.00	7078.40	5.31	OK	1.33
11	1200	150	180000	3	179764.38	1125	11466.13	7.96	12056.90	600	9699.57	15168.00	18.65	OK	2.34
12	750	150	112500	2	112342.92	675	7166.33	4.98	10624.95	600	1392.60	9100.80	7.87	OK	1.58
13	750	150	112500	2	112342.92	675	7166.33	4.98	10624.95	600	1392.60	9100.80	7.87	OK	1.58
14	1200	150	180000	3	179764.38	1125	11466.13	7.96	12056.90	600	9699.57	15168.00	18.65	OK	2.34
15	600	150	135000	3	134764.38	525	5733.06	3.98	10410.98	600	0.00	7078.40	5.31	OK	1.33
16	1200	150	180000	3	179764.38	300	3789.54	2.32	4803.87	600	0.00	4044.80	3.03	OK	1.31
17	1200	150	180000	3	179764.38	300	3789.54	2.32	4803.87	600	0.00	4044.80	3.03	OK	1.31
18	1200	150	180000	3	179764.38	1125	0.00	0.00	1645.92	600	N/A	N/A	N/A	N/A	N/A
19	1200	150	180000	3	179764.38	1125	0.00	0.00	1645.92	600	N/A	N/A	N/A	N/A	N/A
A	900	150	135000	4	134685.84	825	10920.97	6.30	12134.41	600	1848.48	11123.20	9.73	OK	1.55
B	900	150	135000	4	134685.84	825	10920.97	6.30	12134.41	600	1848.48	11123.20	9.73	OK	1.55
C	3900	150	585000	10	584214.60	3825	47324.21	27.28	12134.41	600	54466.07	51571.21	79.53	OK	2.91
D	10050	150	1507500	23	1505693.58	9975	63474.74	38.79	6315.89	600	161054.61	134489.62	221.66	OK	5.71
E	3900	150	585000	10	584214.60	3825	47324.21	27.28	12134.41	600	54466.07	51571.21	79.53	OK	2.91
F	900	150	135000	4	134685.84	825	10920.97	6.30	12134.41	600	1848.48	11123.20	9.73	OK	1.55
G	900	150	135000	4	134685.84	825	10920.97	6.30	12134.41	600	1848.48	11123.20	9.73	OK	1.55

Design of Diaphragm and Wall Anchors ACI 318-08 D.6 (Flexible Diaphragm)

General Parameters

Parameter	Value	Units	Parameter	Value	SI Units	Value	English Units
fy	206	MPa	ca1	75	mm	2.95	inches
f_{uta} (shear)	391.4	MPa	ca2	75	mm	2.95	inches
ϕ (shear)	0.75	-	hef	300	mm	11.81	inches
ϕ (tension)	0.90	-	f'c	22	MPa	3.19	ksi

Parameter	Value	English Units (in^2)	ACI Reference
$\psi_{ec,V}$	1	-	D.6.2.5
$\psi_{ed,V}$	0.9	-	D.6.2.6
$\psi_{c,V}$	1	-	D.6.2.7
$\psi_{h,V}$	1	-	D.6.2.8
Avc (mm^2)	67500	104.63	D.6.2.1
Avco (mm^2)	25312.5	39.23	D.6.2.1

Parameter	Value	English Units (in^2)	ACI Reference
$\psi_{ec,N}$	1	-	D.5.2.4
$\psi_{ed,N}$	0.75	-	D.5.2.5
$\psi_{c,N}$	1	-	D.5.2.6
$\psi_{cp,N}$	1	-	D.5.2.7
Anc (mm^2)	472500	732.3764648	D.5.2.1
Anco (mm^2)	810000	1255.502511	D.5.2.1

			Steel Strength in Shear				Concrete Breakout				
Wall	Vu (kN)	# Bars	Bar Dia (mm)	Bar Dia (inches)	ϕVsa (N)	Check	Vb (N)	Vcbg (N)	ϕVn (N)	Check	F.S.
1	2.61	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	9.5
2	5.21	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	4.8
3	3.26	2	10	0.39	46110.73	OK	22110.23	53064.56	16582.67	OK	5.1
4	3.26	2	10	0.39	46110.73	OK	22110.23	53064.56	16582.67	OK	5.1
5	5.21	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	4.8
6	2.61	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	9.5
7	1.52	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	16.4
8	118.57	31	10	0.39	714716.26	OK	342708.61	822500.67	257031.46	OK	2.2
9	1.52	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	16.4
10	2.61	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	9.5
11	5.21	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	4.8
12	3.26	2	10	0.39	46110.73	OK	22110.23	53064.56	16582.67	OK	5.1
13	3.26	2	10	0.39	46110.73	OK	22110.23	53064.56	16582.67	OK	5.1
14	5.21	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	4.8
15	2.61	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	9.5
16	1.52	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	16.4
17	1.52	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	16.4
18	0.00	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	N/A
19	0.00	3	10	0.39	69166.09	OK	33165.35	79596.84	24874.01	OK	N/A
A	4.12	4	10	0.39	92221.45	OK	44220.47	106129.12	33165.35	OK	8.0
B	4.12	4	10	0.39	92221.45	OK	44220.47	106129.12	33165.35	OK	8.0
C	17.86	10	10	0.39	230553.63	OK	110551.17	265322.80	82913.37	OK	4.6
D	25.39	23	10	0.39	530273.35	OK	254267.68	610242.43	190700.76	OK	7.5
E	17.86	10	10	0.39	230553.63	OK	110551.17	265322.80	82913.37	OK	4.6
F	4.12	4	10	0.39	92221.45	OK	44220.47	106129.12	33165.35	OK	8.0
G	4.12	4	10	0.39	92221.45	OK	44220.47	106129.12	33165.35	OK	8.0

	Tension Breakout D.5.2								Development Length into Foundation		
Wall	Nb (N)	Ncbg (N)	Mu (N-m)	L (m)	T (N)	ϕN	Check	F.S.	ldh (ft)	ldh (inches)	ldh (mm)
1	733783.29	321030.19	5.73	0.6	9.56	288927.17	OK	30237.99	0.24	2.92	74.05
2	733783.29	321030.19	11.47	1.2	9.56	288927.17	OK	30237.99	0.24	2.92	74.05
3	489188.86	214020.13	7.17	0.75	9.56	192618.11	OK	20158.66	0.24	2.92	74.05
4	489188.86	214020.13	7.17	0.75	9.56	192618.11	OK	20158.66	0.24	2.92	74.05
5	733783.29	321030.19	11.47	1.2	9.56	288927.17	OK	30237.99	0.24	2.92	74.05
6	733783.29	321030.19	5.73	0.6	9.56	288927.17	OK	30237.99	0.24	2.92	74.05
7	733783.29	321030.19	3.79	1.2	3.16	288927.17	OK	91492.09	0.24	2.92	74.05
8	7582427.37	3317311.98	332.00	13.65	24.32	2985580.78	OK	122751.83	0.24	2.92	74.05
9	733783.29	321030.19	3.79	1.2	3.16	288927.17	OK	91492.09	0.24	2.92	74.05
10	733783.29	321030.19	5.73	0.6	9.56	288927.17	OK	30237.99	0.24	2.92	74.05
11	733783.29	321030.19	11.47	1.2	9.56	288927.17	OK	30237.99	0.24	2.92	74.05
12	489188.86	214020.13	7.17	0.75	9.56	192618.11	OK	20158.66	0.24	2.92	74.05
13	489188.86	214020.13	7.17	0.75	9.56	192618.11	OK	20158.66	0.24	2.92	74.05
14	733783.29	321030.19	11.47	1.2	9.56	288927.17	OK	30237.99	0.24	2.92	74.05
15	733783.29	321030.19	5.73	0.6	9.56	288927.17	OK	30237.99	0.24	2.92	74.05
16	733783.29	321030.19	3.79	1.2	3.16	288927.17	OK	91492.09	0.24	2.92	74.05
17	733783.29	321030.19	3.79	1.2	3.16	288927.17	OK	91492.09	0.24	2.92	74.05
18	733783.29	321030.19	0.00	1.2	0.00	288927.17	OK	N/A	0.24	2.92	74.05
19	733783.29	321030.19	0.00	1.2	0.00	288927.17	OK	N/A	0.24	2.92	74.05
A	978377.73	428040.25	10.92	0.9	12.13	385236.23	OK	31747.42	0.24	2.92	74.05
B	978377.73	428040.25	10.92	0.9	12.13	385236.23	OK	31747.42	0.24	2.92	74.05
C	2445944.31	1070100.64	47.32	3.9	12.13	963090.57	OK	79368.54	0.24	2.92	74.05
D	5625671.92	2461231.47	63.47	10.05	6.32	2215108.32	OK	350719.67	0.24	2.92	74.05
E	2445944.31	1070100.64	47.32	3.9	12.13	963090.57	OK	79368.54	0.24	2.92	74.05
F	978377.73	428040.25	10.92	0.9	12.13	385236.23	OK	31747.42	0.24	2.92	74.05
G	978377.73	428040.25	10.92	0.9	12.13	385236.23	OK	31747.42	0.24	2.92	74.05

Design Summary (for Rigid and Flexible Diaphragms)

Wall	Horizontal Steel Dia (mm)	Spacing (mm)	Vertical Steel Dia (mm)	# of bars (evenly spaced)
1	10	600	10	3
2	10	600	10	3
3	10	600	10	2
4	10	600	10	2
5	10	600	10	3
6	10	600	10	3
7	10	600	10	3
8	10	600	10	31
9	10	600	10	3
10	10	600	10	3
11	10	600	10	3
12	10	600	10	2
13	10	600	10	2
14	10	600	10	3
15	10	600	10	3
16	10	600	10	3
17	10	600	10	3
18	10	600	10	3
19	10	600	10	3
A	10	600	10	4
B	10	600	10	4
C	10	600	10	10
D	10	400	10	23
E	10	600	10	10
F	10	600	10	4
G	10	600	10	4

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Material Name: ICEB

Material Type: User Defined

Input Parameters:

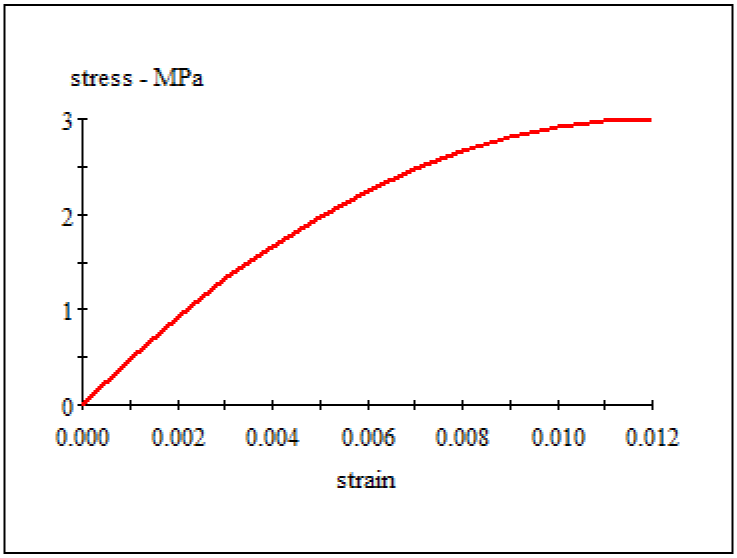
Ultimate Compresive Strain: 2.500E-3

Compression Yield Strain: 2.000E-3

Tensile Yield Strain: 1.0000

Ultimate Tensile Strain: 1.200

Additional Information: 'Strain' 'Stress'



Material Color States:

Yield

Stress Strain Points:

Strain	Stress (MP
0	0
1.000E-3	.4792
2.000E-3	.9167
3.000E-3	1.313
4.000E-3	1.667
5.000E-3	1.979
6.000E-3	2.250
7.000E-3	2.479
8.000E-3	2.667
9.000E-3	2.813
10.00E-3	2.917
11.00E-3	2.979
12.00E-3	3.000

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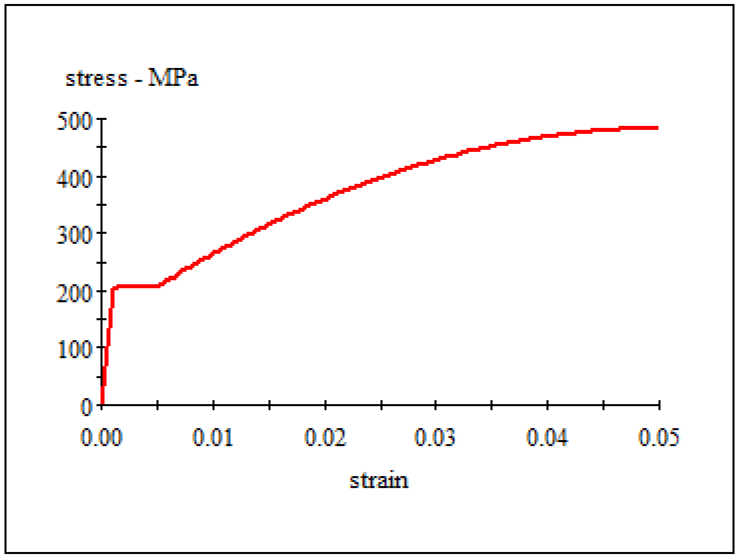
Material Name: Grade 30

Material Type: Strain Hardening Steel

Input Parameters:

Yield Stress:	206.0 MPa
Fracture Stress:	482.6 MPa
Yield Strain:	1.031E-3
Strain at Strain Hardening:	5.000E-3
Failure Strain:	50.00E-3
Elastic Modulus:	199.9E+3 MPa
Additional Information:	Symetric Tension and Comp.



Model Details:

For Strain - $\varepsilon < \varepsilon_y$ $\quad fs = E \cdot \varepsilon$

For Strain - $\varepsilon < \varepsilon_{sh}$ $\quad fs = f_y$

For Strain - $\varepsilon < \varepsilon_{su}$ $\quad fs = f_u - \left(f_u - f_y\right) \cdot \left(\frac{\varepsilon_{su} - \varepsilon}{\varepsilon_{su} - \varepsilon_{sh}}\right)^2$

ε = Steel Strain

fs = Steel Stress

f_y = Yield Stress

f_u = Fracture Stress

ε_y = Yield Strain

ε_{sh} = Strain at Strain Hardening

ε_{su} = Failure Strain

E = Elastic Modulus

Material Color States:

- Tension force after onset of strain hardening
- Tension force after yield
- Initial state
- Compression force after yield
- Compression force after onset of strain hardening

XTRACT Material Report - Educational

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Material Name: Concrete

Material Type: Unconfined Concrete

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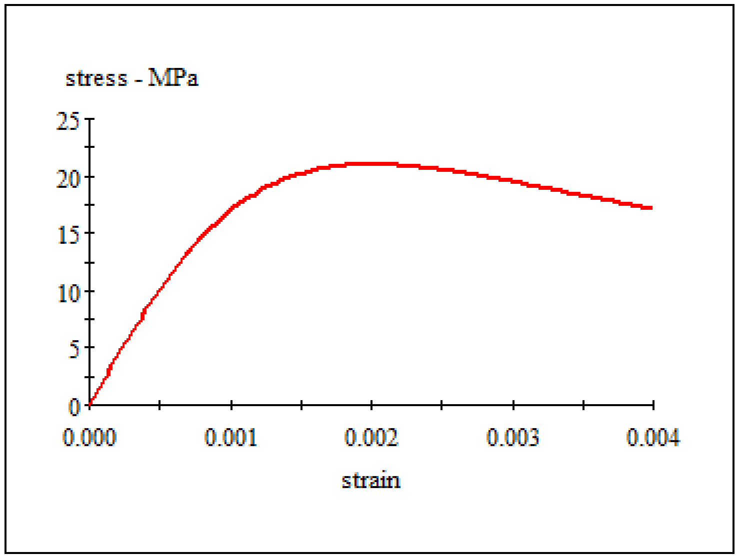
Input Parameters:

Tension Strength:	0 MPa
28 Day Strength:	21.00 MPa
Post Crushing Strength:	0 MPa
Tension Strain Capacity:	0 Ten
Spalling Strain:	6.000E-3 Comp
Failure Strain:	4.000E-3 Comp
Elastic Modulus:	21.69E+3 MPa
Secant Modulus:	1523 MPa



Model Details:

For Strain - $\varepsilon < 2 \cdot \varepsilon_t$ $\quad fc = 0$

For Strain - $\varepsilon < 0$ $\quad fc = \varepsilon \cdot Ec$

For Strain - $\varepsilon < \varepsilon_{cu}$ $\quad fc = \dfrac{f'_c \cdot x \cdot r}{r - 1 + x^r}$

For Strain - $\varepsilon < \varepsilon_{sp}$ $\quad fc = f_{cu} + \left(f_{cp} - f_{cu}\right) \cdot \dfrac{\left(\varepsilon - \varepsilon_{cu}\right)}{\left(\varepsilon_{sp} - \varepsilon_{cu}\right)}$

$$x = \frac{\varepsilon}{\varepsilon_{cc}}$$

$$r = \frac{Ec}{Ec - E_{sec}}$$

$$E_{sec} = \frac{f'_c}{\varepsilon_{cc}}$$

ε = Concrete Strain

fc = Concrete Stress

Ec = Elastic Modulus

E_{sec} = Secant Modulus

ε_t = Tension Strain Capacity

ε_{cu} = Ultimate Concrete Strain

ε_{cc} = Strain at Peak Stress = .002

ε_{sp} = Spalling Strain

f'_c = 28 Day Compressive Strength

f_{cu} = Stress at ε_{cu}

f_{cp} = Post Spalling Strength

Material Color States:

- Tension strain after tension capacity
- Tension strain before tension capacity
- Initial state
- Compression before crushing strain
- Compression before end of spalling
- Compression after spalling

Reference:

Mander, J.B., Priestley, M. J. N., "Observed Stress-Strain Behavior of Confined Concrete", Journal of Structural Engineering, ASCE, Vol. 114, No. 8, August 1988, pp. 1827-1849

XTRACT Section Report - Educational

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Section Name: PWall1

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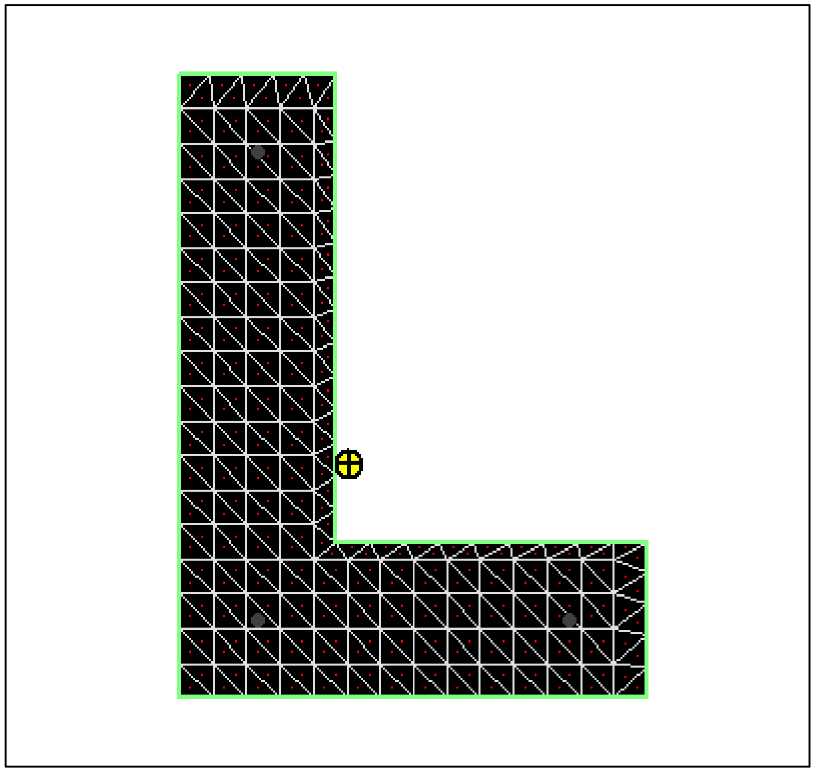
Section Details:

X Centroid:	162.8 mm
Y Centroid:	225.0 mm
Section Area:	135.0E+3 mm^2
EI gross about X:	17.09E+9 N-m^2
EI gross about Y:	613.2E+3 N-m^2
I trans (ICEB) about X:	1.065E+6 cm^4
I trans (ICEB) about Y:	488.5E+3 cm^4
Reinforcing Bar Area:	339.3 mm^2
Percent Longitudinal Steel:	.2513 %
Overall Width:	450.0 mm
Overall Height:	600.0 mm
Number of Fibers:	271
Number of Bars:	3
Number of Materials:	2



Material Types and Names:

User Defined:	ICEB
Strain Hardening Steel:	Grade 30

Comments:

User Comments

3*(P/Pu)^2 + -38.55E+3*(P/Pu)^3

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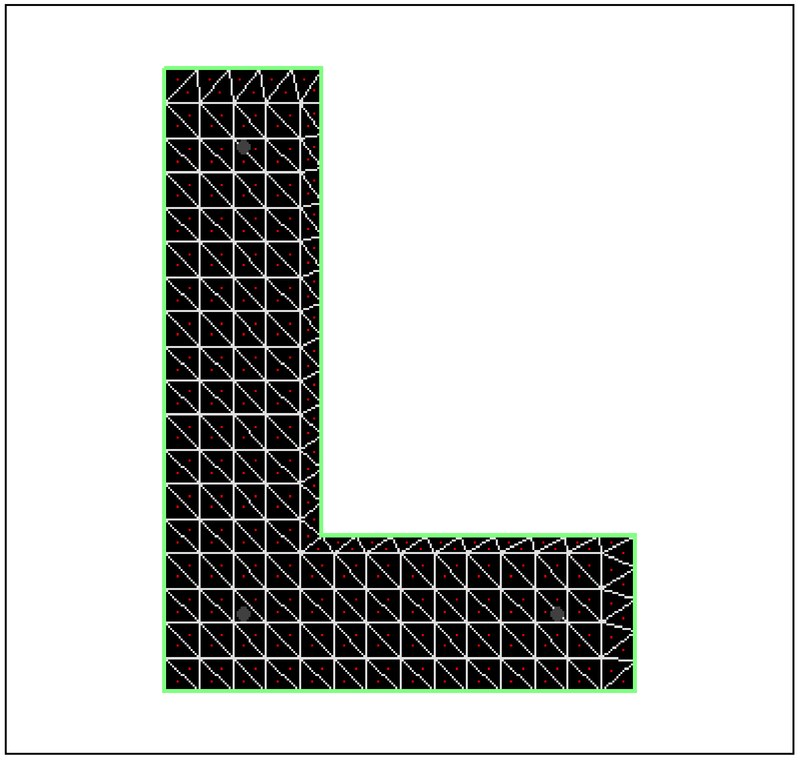
Section Name:	PWall1
Loading Name:	Interaction
Analysis Type:	PM Interaction

Section Details:

X Centroid:	162.8 mm
Y Centroid:	225.0 mm
Section Area:	135.0E+3 mm^2

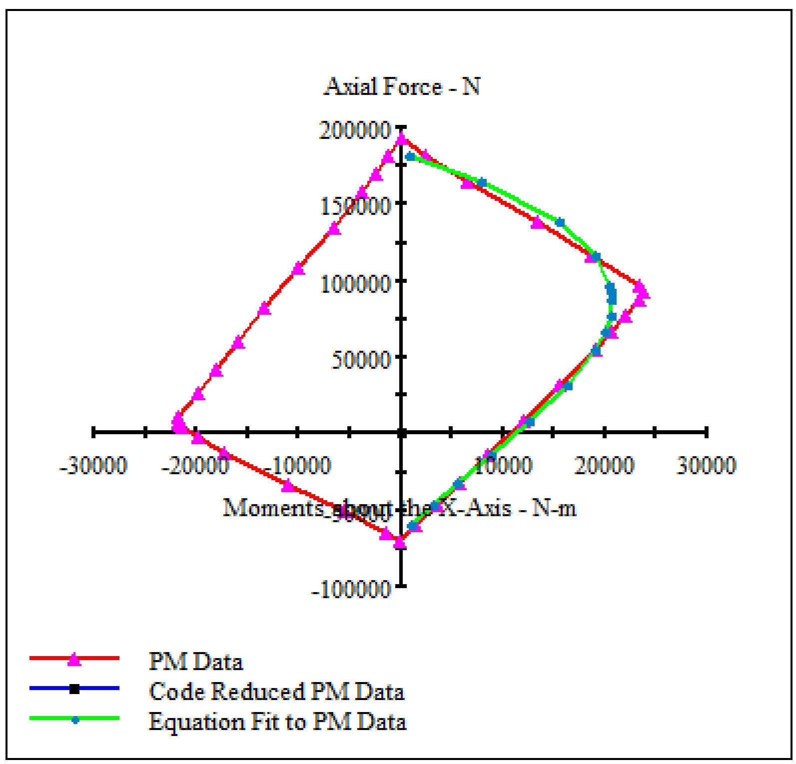


Loading Details:

Angle of Loading:	0 deg
Number of Points:	40
Min. ICEB Strain:	2.500E-3 Comp
Max. ICEB Strain:	1.0000 Ten
Min. Grade 30 Strain:	2.000E-3 Comp
Max. Grade 30 Strain:	2.000E-3 Ten

Analysis Results:

Max. Compression Load:	193.3E+3 N
Max. Tension Load:	-69.90E+3 N
Maximum Moment:	23.65E+3 N-m
P at Max. Moment:	92.91E+3 N
Minimum Moment:	-21.84E+3 N-m
P at Min. Moment:	7294 N
Moment (Mxx) at P=0:	10.76E+3 N-m
Max. Code Comp. Load:	0 N
Max. Code Ten. Load:	0 N
Maximum Code Moment:	0 N-m
P at Max. Code Moment:	0 N
Minimum Code Moment:	0 N-m
P at Min. Code Moment:	0 N
PM Interaction Equation:	Units in N-m



Comments:

User Comments

(P/Pu)^2 + 38.09E+3(P/Pu)^3

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Section Name: PWall1

Loading Name: Out of Plane

Analysis Type: PM Interaction

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Section Details:

X Centroid: 162.8 mm

Y Centroid: 225.0 mm

Section Area: 135.0E+3 mm^2

Loading Details:

Angle of Loading: 90 deg

Number of Points: 40

Min. ICEB Strain: 2.000E-3 Comp

Max. ICEB Strain: 1.0000 Ten

Min. Grade 30 Strain: 2.000E-3 Comp

Max. Grade 30 Strain: 2.000E-3 Ten

Analysis Results:

Max. Compression Load: 193.3E+3 N

Max. Tension Load: -69.90E+3 N

Maximum Moment: 12.66E+3 N-m

P at Max. Moment: -5692 N

Minimum Moment: -14.83E+3 N-m

P at Min. Moment: 84.11E+3 N

Moment (Myy) at P=0: 12.38E+3 N-m

Max. Code Comp. Load: 0 N

Max. Code Ten. Load: 0 N

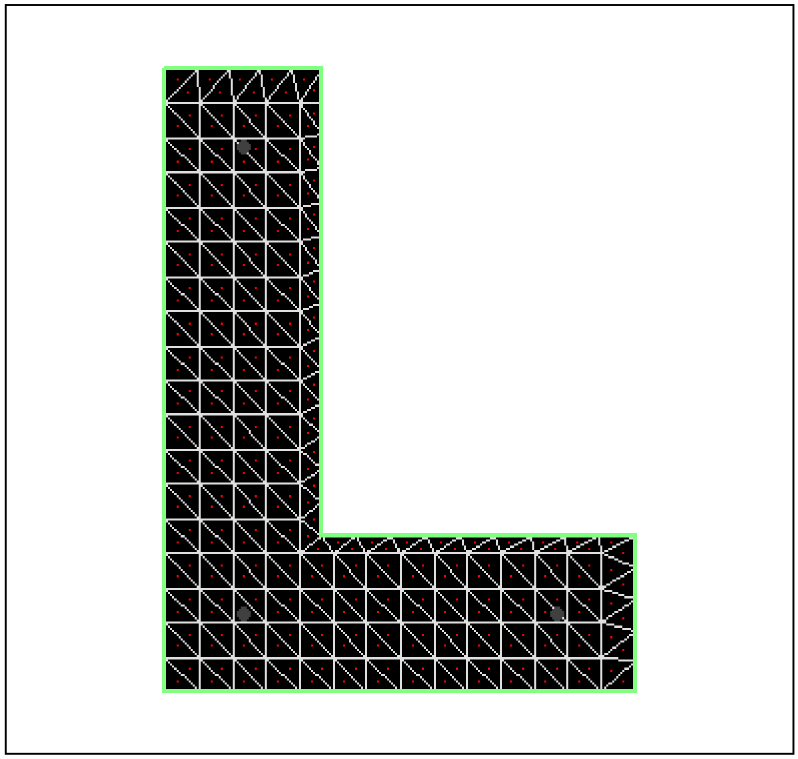
Maximum Code Moment: 0 N-m

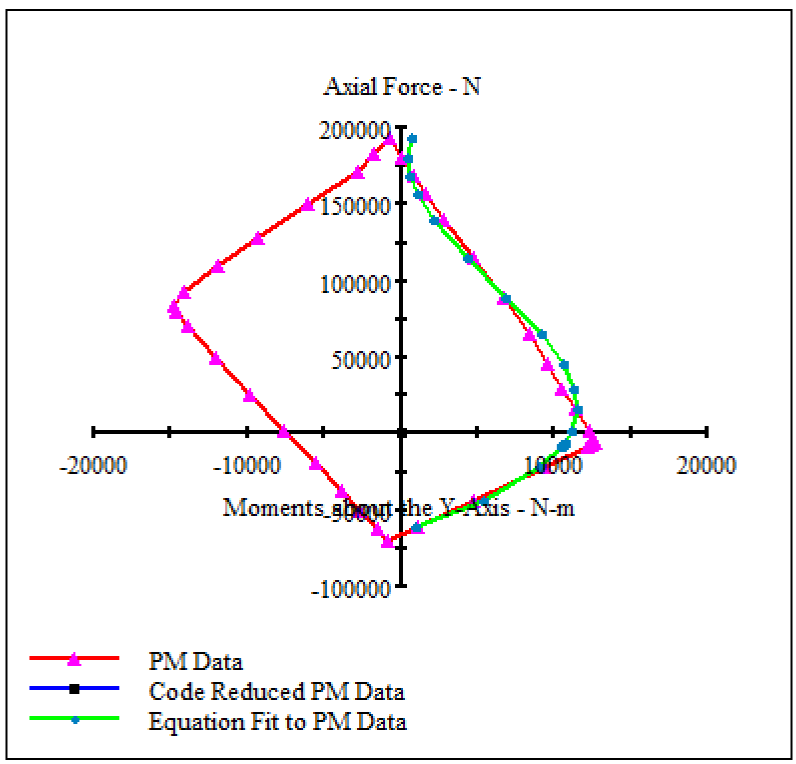
P at Max. Code Moment: 0 N

Minimum Code Moment: 0 N-m

P at Min. Code Moment: 0 N

PM Interaction Equation: Units in N-m





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Section Name: PWall2

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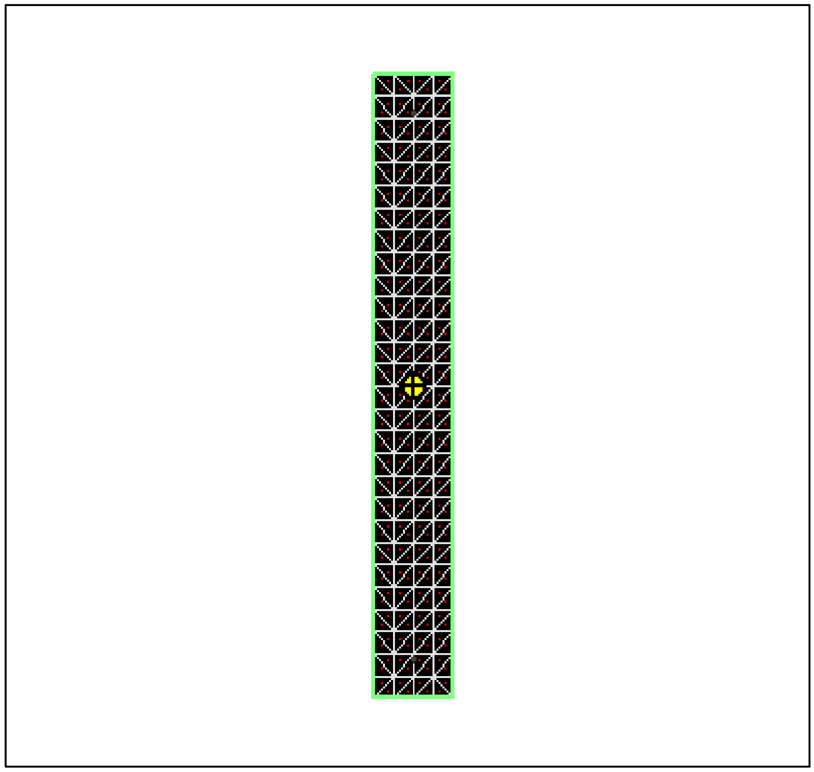
Section Details:

X Centroid:	2.69E-15 mm
Y Centroid:	1.02E-14 mm
Section Area:	180.0E+3 mm^2
EI gross about X:	17.09E+9 N-m^2
EI gross about Y:	613.2E+3 N-m^2
I trans (ICEB) about X:	3.960E+6 cm^4
I trans (ICEB) about Y:	32.34E+3 cm^4
Reinforcing Bar Area:	235.6 mm^2
Percent Longitudinal Steel:	.1309 %
Overall Width:	150.0 mm
Overall Height:	1200 mm
Number of Fibers:	224
Number of Bars:	3
Number of Materials:	2



Material Types and Names:

User Defined:	ICEB
Strain Hardening Steel:	Grade 30

Comments:

User Comments

-3*(P/Pu)^2 + 67.14E+3*(P/Pu)^3

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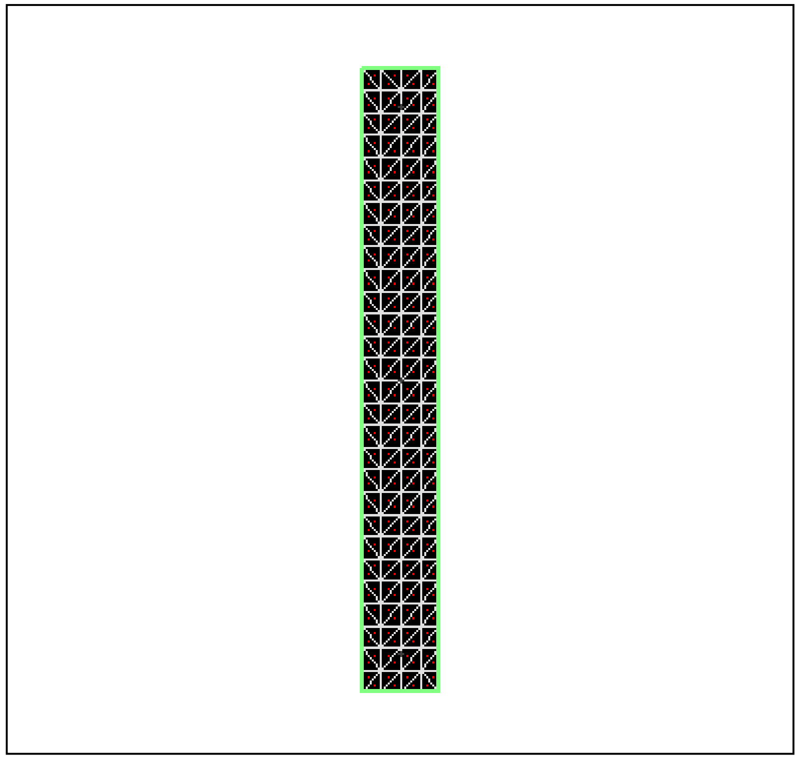
Section Name:	PWall2
Loading Name:	Interaction
Analysis Type:	PM Interaction

Section Details:

X Centroid:	2.69E-15 mm
Y Centroid:	1.02E-14 mm
Section Area:	180.0E+3 mm^2

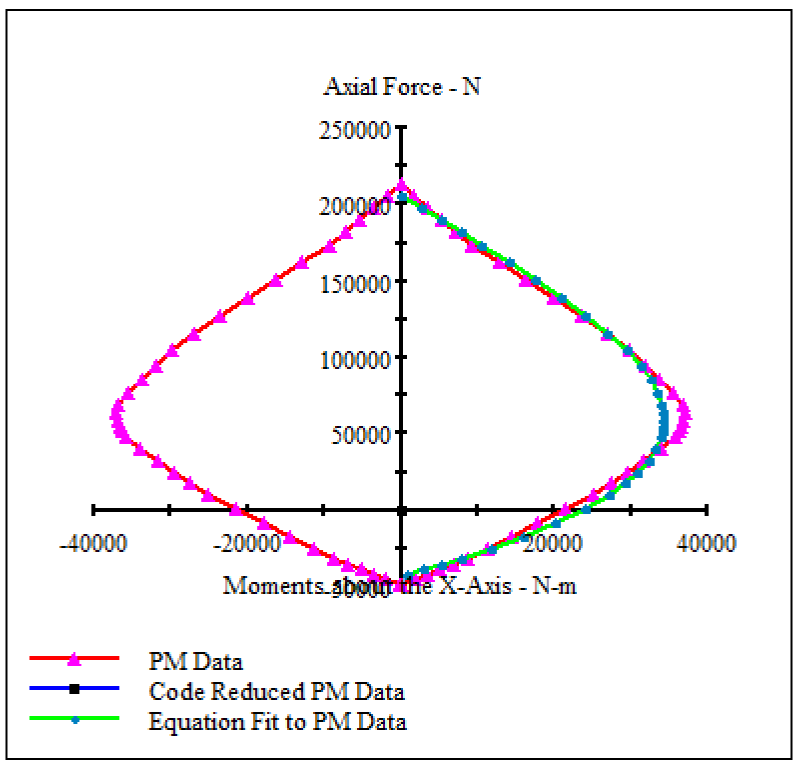


Loading Details:

Angle of Loading:	0 deg
Number of Points:	80
Min. ICEB Strain:	2.500E-3 Comp
Max. ICEB Strain:	1.0000 Ten
Min. Grade 30 Strain:	2.000E-3 Comp
Max. Grade 30 Strain:	2.000E-3 Ten

Analysis Results:

Max. Compression Load:	213.3E+3 N
Max. Tension Load:	-48.54E+3 N
Maximum Moment:	37.05E+3 N-m
P at Max. Moment:	63.16E+3 N
Minimum Moment:	-37.05E+3 N-m
P at Min. Moment:	63.16E+3 N
Moment (Mxx) at P=0:	21.31E+3 N-m
Max. Code Comp. Load:	0 N
Max. Code Ten. Load:	0 N
Maximum Code Moment:	0 N-m
P at Max. Code Moment:	0 N
Minimum Code Moment:	0 N-m
P at Min. Code Moment:	0 N
PM Interaction Equation:	Units in N-m



3*(P/Pu)^2 + 4088*(P/Pu)^3

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Section Name: PWall2

Loading Name: Out of Plane

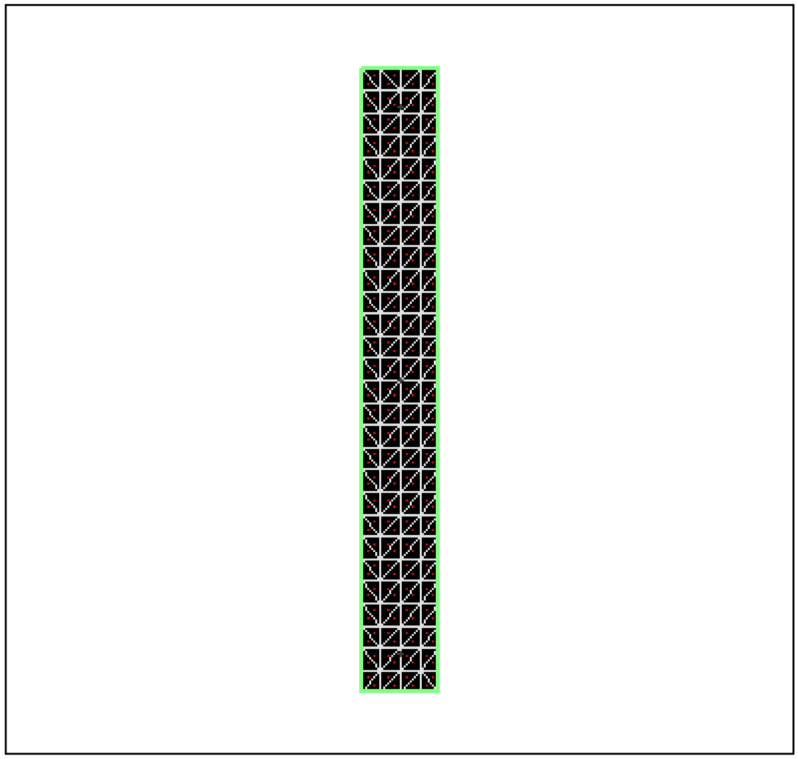
Analysis Type: PM Interaction

Section Details:

X Centroid: 2.69E-15 mm

Y Centroid: 1.02E-14 mm

Section Area: 180.0E+3 mm^2



Loading Details:

Angle of Loading: 90 deg

Number of Points: 60

Min. ICEB Strain: 2.500E-3 Comp

Max. ICEB Strain: 1.0000 Ten

Min. Grade 30 Strain: 2.000E-3 Comp

Max. Grade 30 Strain: 2.000E-3 Ten

Analysis Results:

Max. Compression Load: 213.3E+3 N

Max. Tension Load: -48.54E+3 N

Maximum Moment: 3125 N-m

P at Max. Moment: 107.3E+3 N

Minimum Moment: -3125 N-m

P at Min. Moment: 107.3E+3 N

Moment (Myy) at P=0: 2597 N-m

Max. Code Comp. Load: 0 N

Max. Code Ten. Load: 0 N

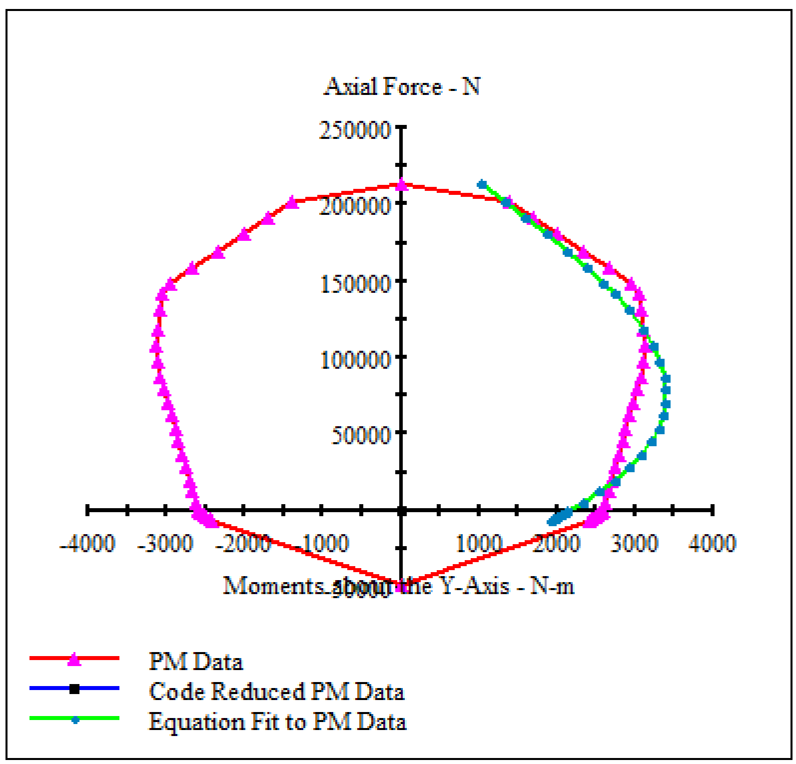
Maximum Code Moment: 0 N-m

P at Max. Code Moment: 0 N

Minimum Code Moment: 0 N-m

P at Min. Code Moment: 0 N

PM Interaction Equation: Units in N-m



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Section Name: PWall3

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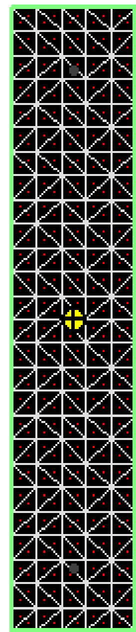
Section Details:



X Centroid:	-8.68E-15 mm
Y Centroid:	8.62E-16 mm
Section Area:	112.5E+3 mm^2
EI gross about X:	17.09E+9 N-m^2
EI gross about Y:	613.2E+3 N-m^2
I trans (ICEB) about X:	1.115E+6 cm^4
I trans (ICEB) about Y:	20.53E+3 cm^4
Reinforcing Bar Area:	157.1 mm^2
Percent Longitudinal Steel:	.1396 %
Overall Width:	150.0 mm
Overall Height:	750.0 mm
Number of Fibers:	260
Number of Bars:	2
Number of Materials:	2

Material Types and Names:

User Defined:	ICEB
Strain Hardening Steel:	Grade 30

Comments:

User Comments

-3*(P/Pu)^2 + 37.88E+3*(P/Pu)^3

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Section Name: PWall3

Loading Name: Interaction

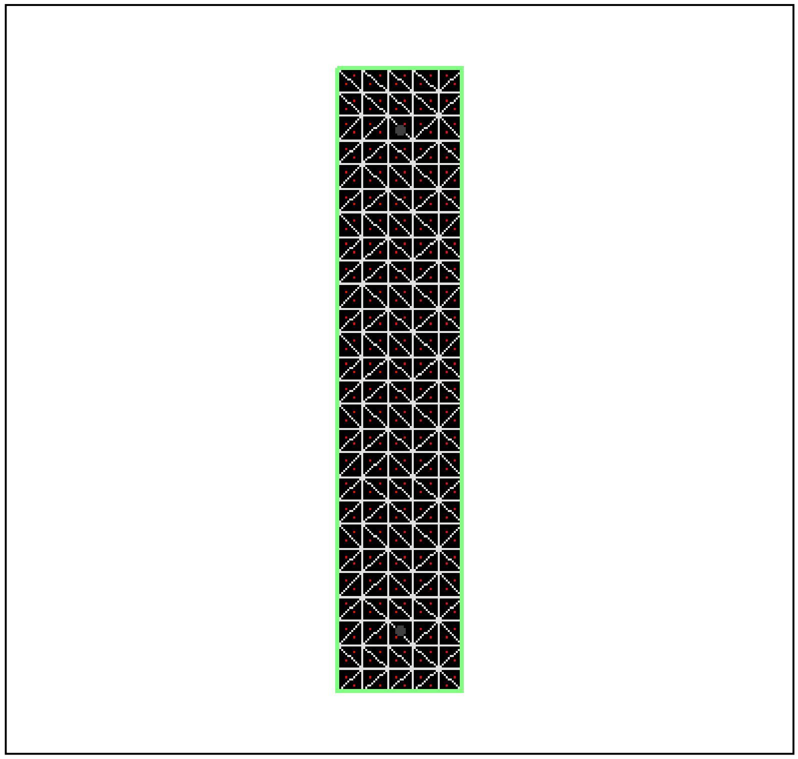
Analysis Type: PM Interaction

Section Details:

X Centroid: -8.68E-15 mm

Y Centroid: 8.62E-16 mm

Section Area: 112.5E+3 mm^2



Loading Details:

Angle of Loading: 0 deg

Number of Points: 40

Min. ICEB Strain: 2.500E-3 Comp

Max. ICEB Strain: 1.0000 Ten

Min. Grade 30 Strain: 2.000E-3 Comp

Max. Grade 30 Strain: 2.000E-3 Ten

Analysis Results:

Max. Compression Load: 135.3E+3 N

Max. Tension Load: -32.36E+3 N

Maximum Moment: 18.05E+3 N-m

P at Max. Moment: 37.49E+3 N

Minimum Moment: -18.05E+3 N-m

P at Min. Moment: 37.49E+3 N

Moment (Mxx) at P=0: 9711 N-m

Max. Code Comp. Load: 0 N

Max. Code Ten. Load: 0 N

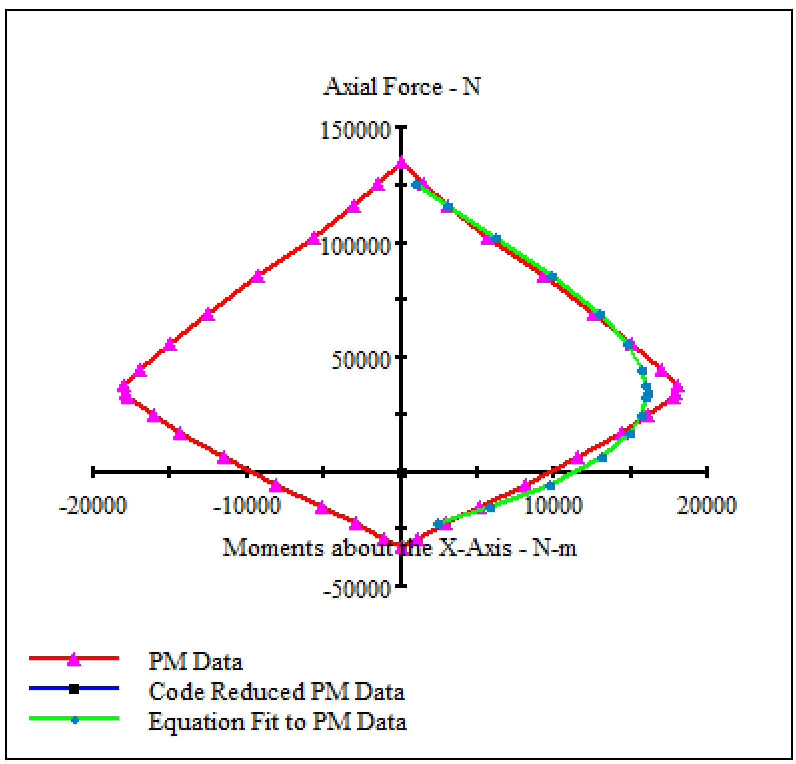
Maximum Code Moment: 0 N-m

P at Max. Code Moment: 0 N

Minimum Code Moment: 0 N-m

P at Min. Code Moment: 0 N

PM Interaction Equation: Units in N-m



Comments:

User Comments

(P/Pu)^2 + 1612*(P/Pu)^3

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Section Name: PWall3

Loading Name: Out of Plane

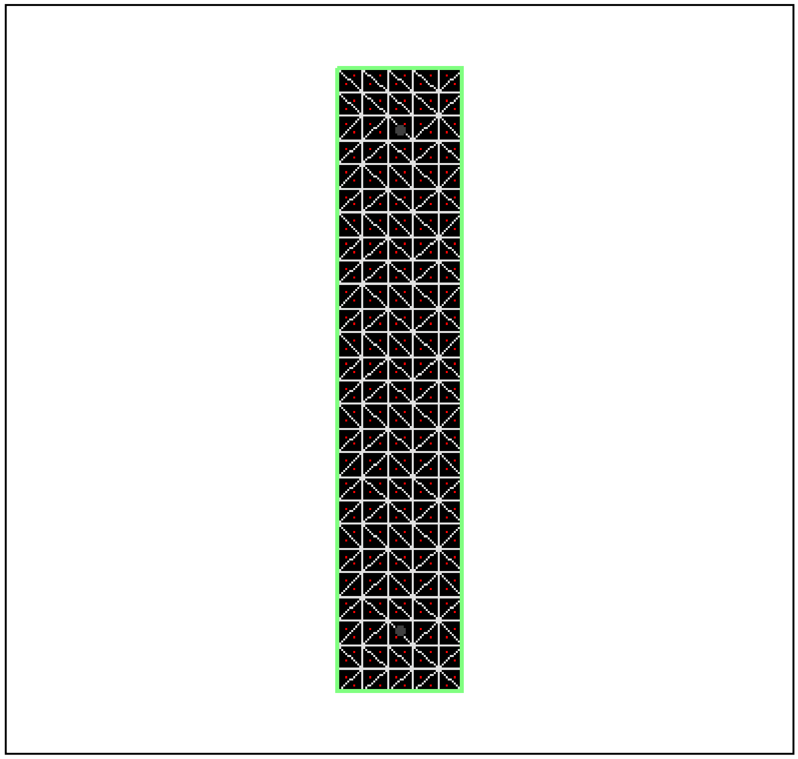
Analysis Type: PM Interaction

Section Details:

X Centroid: -8.68E-15 mm

Y Centroid: 8.62E-16 mm

Section Area: 112.5E+3 mm^2



Loading Details:

Angle of Loading: 90 deg

Number of Points: 40

Min. ICEB Strain: 2.500E-3 Comp

Max. ICEB Strain: 1.0000 Ten

Min. Grade 30 Strain: 2.000E-3 Comp

Max. Grade 30 Strain: 2.000E-3 Ten

Analysis Results:

Max. Compression Load: 135.3E+3 N

Max. Tension Load: -32.36E+3 N

Maximum Moment: 1931 N-m

P at Max. Moment: 74.93E+3 N

Minimum Moment: -1931 N-m

P at Min. Moment: 74.93E+3 N

Moment (Myy) at P=0: 1597 N-m

Max. Code Comp. Load: 0 N

Max. Code Ten. Load: 0 N

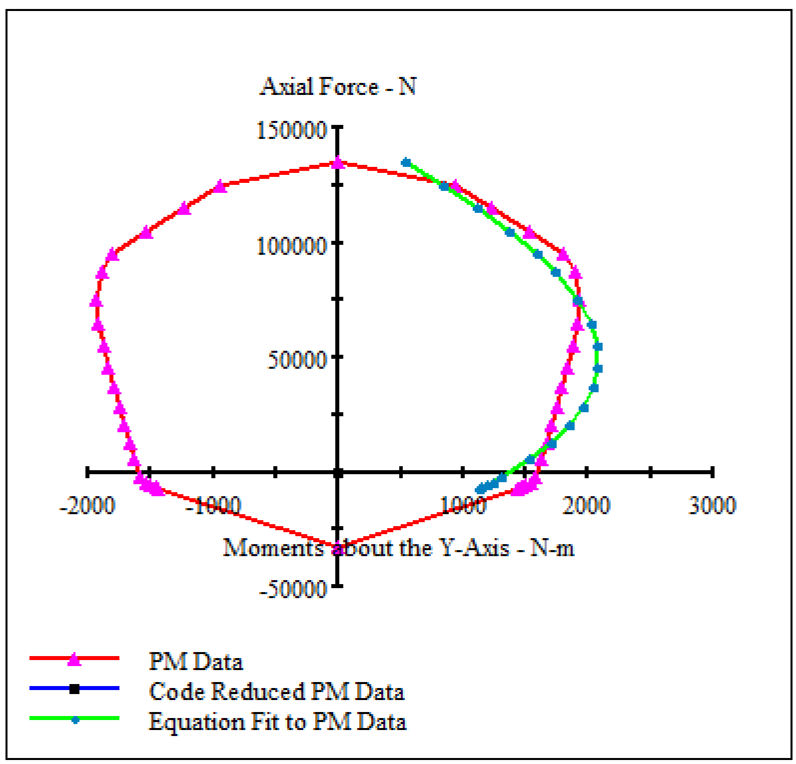
Maximum Code Moment: 0 N-m

P at Max. Code Moment: 0 N

Minimum Code Moment: 0 N-m

P at Min. Code Moment: 0 N

PM Interaction Equation: Units in N-m



XTRACT Section Report - Educational

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Section Name: PWall8

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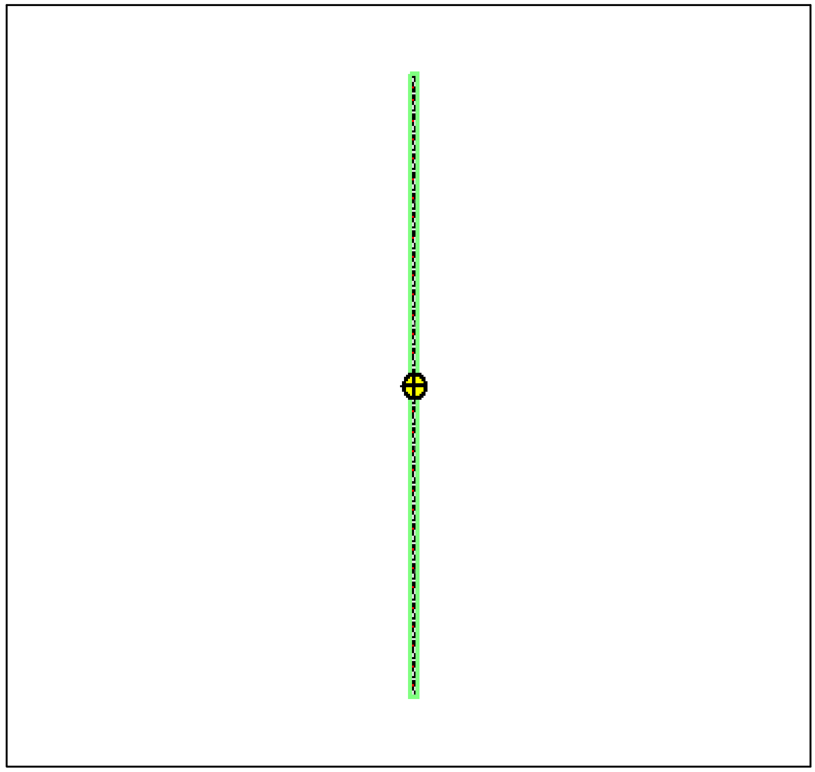
Section Details:

X Centroid:	1.76E-16 mm
Y Centroid:	-4.499 mm
Section Area:	2.048E+6 mm^2
EI gross about X:	17.09E+9 N-m^2
EI gross about Y:	613.2E+3 N-m^2
I trans (ICEB) about X:	3.566E+9 cm^4
I trans (ICEB) about Y:	128.0E+3 cm^4
Reinforcing Bar Area:	735.0 mm^2
Percent Longitudinal Steel:	35.90E-3 %
Overall Width:	150.0 mm
Overall Height:	13.65E+3 mm
Number of Fibers:	64
Number of Bars:	26
Number of Materials:	2



Material Types and Names:

User Defined:	ICEB
Strain Hardening Steel:	Grade 30

Comments:

User Comments

(P/Pu)^2 + -336.0E+3(P/Pu)^3

XTRACT Analysis Report - Educational

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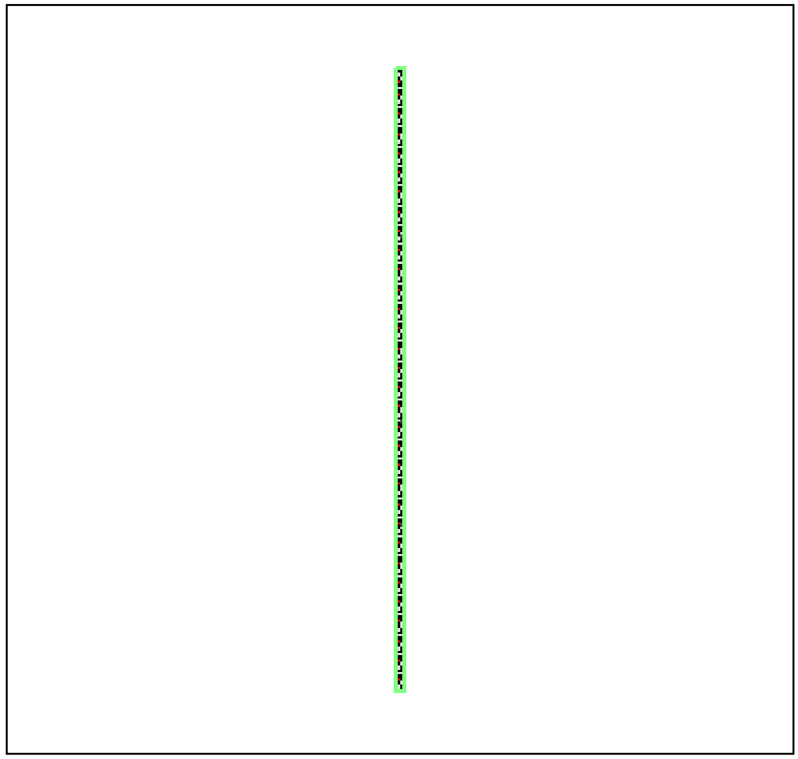
Section Name:	PWall8
Loading Name:	Out of Plane
Analysis Type:	PM Interaction

Section Details:

X Centroid:	1.76E-16 mm
Y Centroid:	-4.499 mm
Section Area:	2.048E+6 mm^2

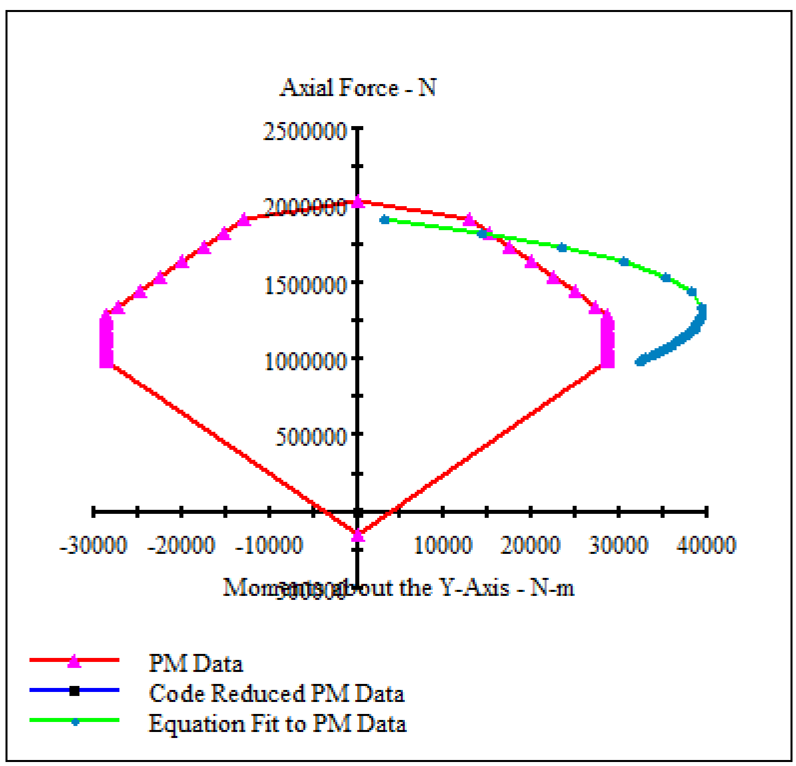


Loading Details:

Angle of Loading:	90 deg
Number of Points:	80
Min. ICEB Strain:	2.500E-3 Comp
Max. ICEB Strain:	1.0000 Ten
Min. Grade 30 Strain:	2.000E-3 Comp
Max. Grade 30 Strain:	2.000E-3 Ten

Analysis Results:

Max. Compression Load:	2.028E+6 N
Max. Tension Load:	-151.4E+3 N
Maximum Moment:	28.53E+3 N-m
P at Max. Moment:	1.292E+6 N
Minimum Moment:	-28.53E+3 N-m
P at Min. Moment:	1.292E+6 N
Moment (Myy) at P=0:	3785 N-m
Max. Code Comp. Load:	0 N
Max. Code Ten. Load:	0 N
Maximum Code Moment:	0 N-m
P at Max. Code Moment:	0 N
Minimum Code Moment:	0 N-m
P at Min. Code Moment:	0 N
PM Interaction Equation:	Units in N-m



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Section Name: PWall 17

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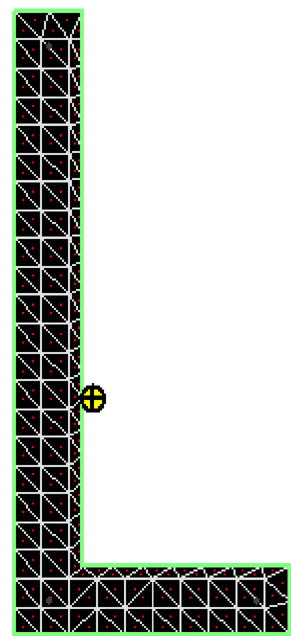
Section Details:

X Centroid:	170.0 mm
Y Centroid:	511.7 mm
Section Area:	270.0E+3 mm^2
EI gross about X:	5.103E+6 N-m^2
EI gross about Y:	2.341E+6 N-m^2
I trans (ICEB) about X:	8.061E+6 cm^4
I trans (ICEB) about Y:	1.084E+6 cm^4
Reinforcing Bar Area:	235.6 mm^2
Percent Longitudinal Steel:	87.27E-3 %
Overall Width:	600.0 mm
Overall Height:	1350 mm
Number of Fibers:	173
Number of Bars:	3
Number of Materials:	2



Material Types and Names:

User Defined:	ICEB
Strain Hardening Steel:	Grade 30

Comments:

User Comments

3*(P/Pu)^2 + -46.29E+3*(P/Pu)^3

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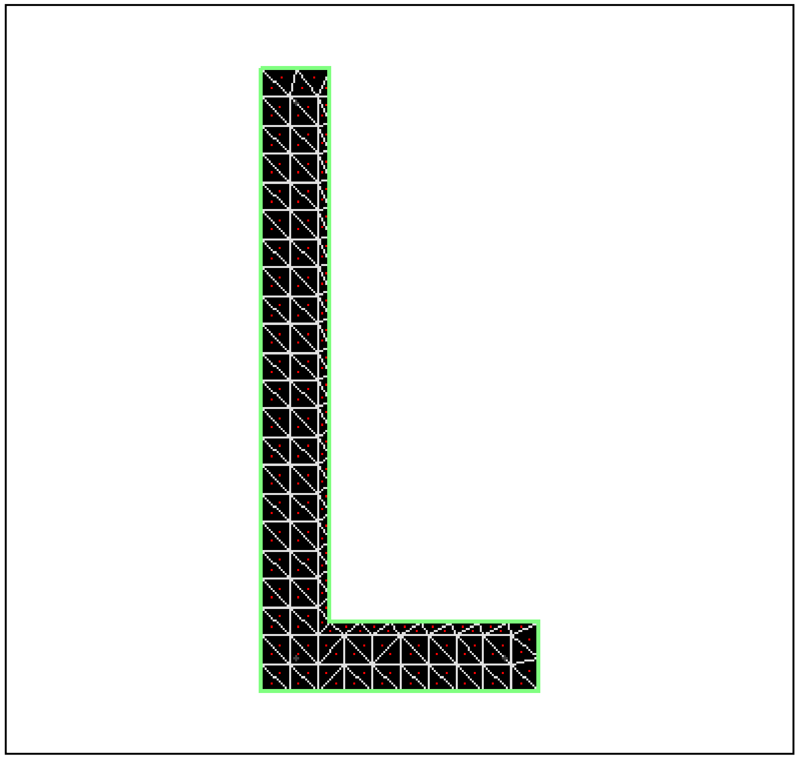
Section Name:	PWall 17
Loading Name:	Interaction
Analysis Type:	PM Interaction

Section Details:

X Centroid:	170.0 mm
Y Centroid:	511.7 mm
Section Area:	270.0E+3 mm^2

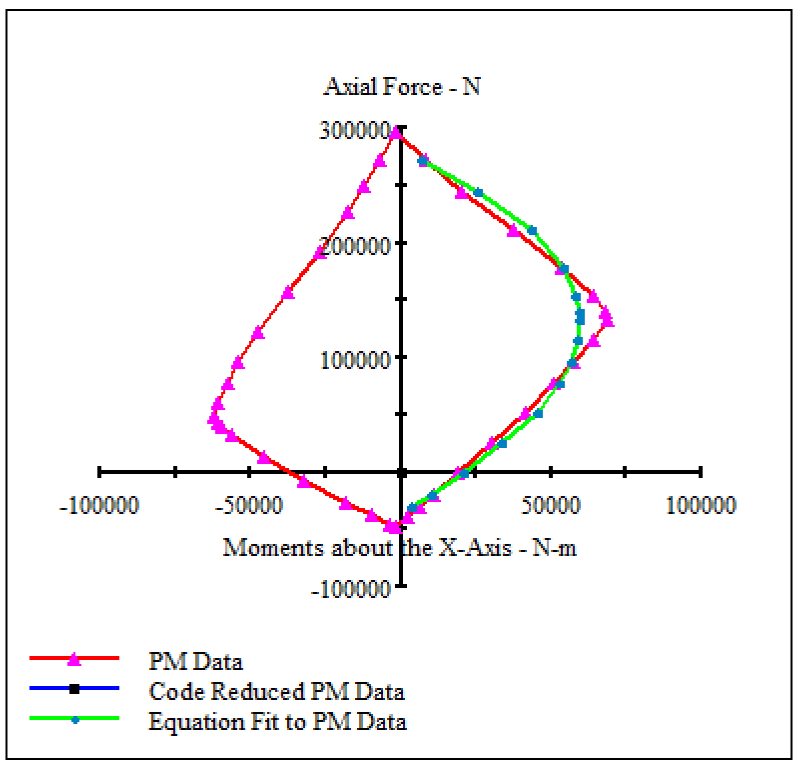


Loading Details:

Angle of Loading:	0 deg
Number of Points:	40
Min. ICEB Strain:	2.500E-3 Comp
Max. ICEB Strain:	1.0000 Ten
Min. Grade 30 Strain:	2.000E-3 Comp
Max. Grade 30 Strain:	2.000E-3 Ten

Analysis Results:

Max. Compression Load:	295.8E+3 N
Max. Tension Load:	-48.54E+3 N
Maximum Moment:	68.58E+3 N-m
P at Max. Moment:	132.4E+3 N
Minimum Moment:	-62.17E+3 N-m
P at Min. Moment:	48.14E+3 N
Moment (Mxx) at P=0:	19.53E+3 N-m
Max. Code Comp. Load:	0 N
Max. Code Ten. Load:	0 N
Maximum Code Moment:	0 N-m
P at Max. Code Moment:	0 N
Minimum Code Moment:	0 N-m
P at Min. Code Moment:	0 N
PM Interaction Equation:	Units in N-m



Comments:

User Comments

-3*(P/Pu)^2 + 98.11E+3*(P/Pu)^3

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Section Name: PWall 17

Loading Name: Out of Plane

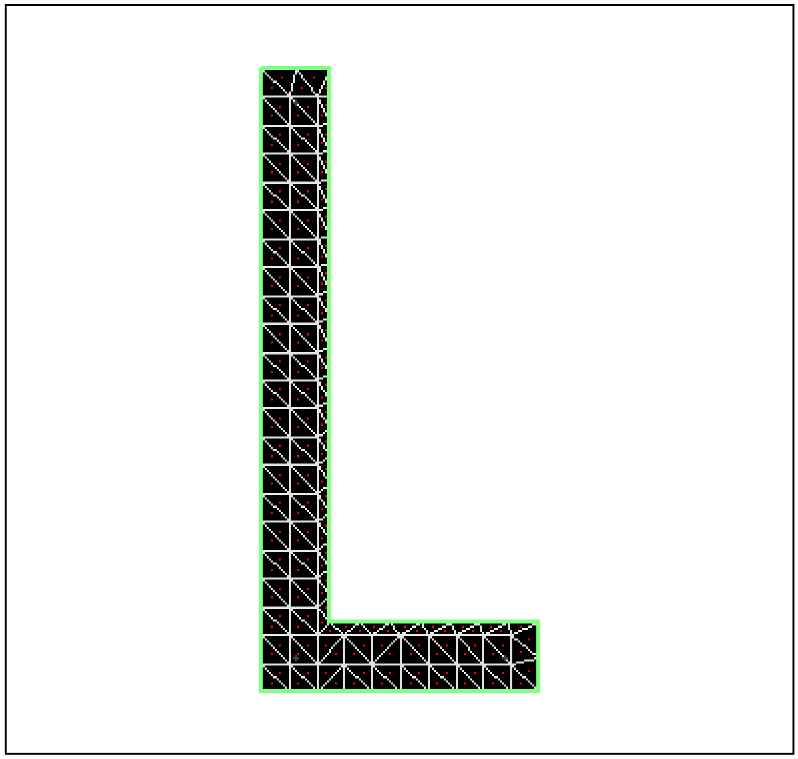
Analysis Type: PM Interaction

Section Details:

X Centroid: 170.0 mm

Y Centroid: 511.7 mm

Section Area: 270.0E+3 mm^2



Loading Details:

Angle of Loading: 90 deg

Number of Points: 40

Min. ICEB Strain: 2.500E-3 Comp

Max. ICEB Strain: 1.0000 Ten

Min. Grade 30 Strain: 2.000E-3 Comp

Max. Grade 30 Strain: 2.000E-3 Ten

Analysis Results:

Max. Compression Load: 295.8E+3 N

Max. Tension Load: -48.54E+3 N

Maximum Moment: 18.58E+3 N-m

P at Max. Moment: 15.98E+3 N

Minimum Moment: -27.24E+3 N-m

P at Min. Moment: 208.3E+3 N

Moment (Myy) at P=0: 14.28E+3 N-m

Max. Code Comp. Load: 0 N

Max. Code Ten. Load: 0 N

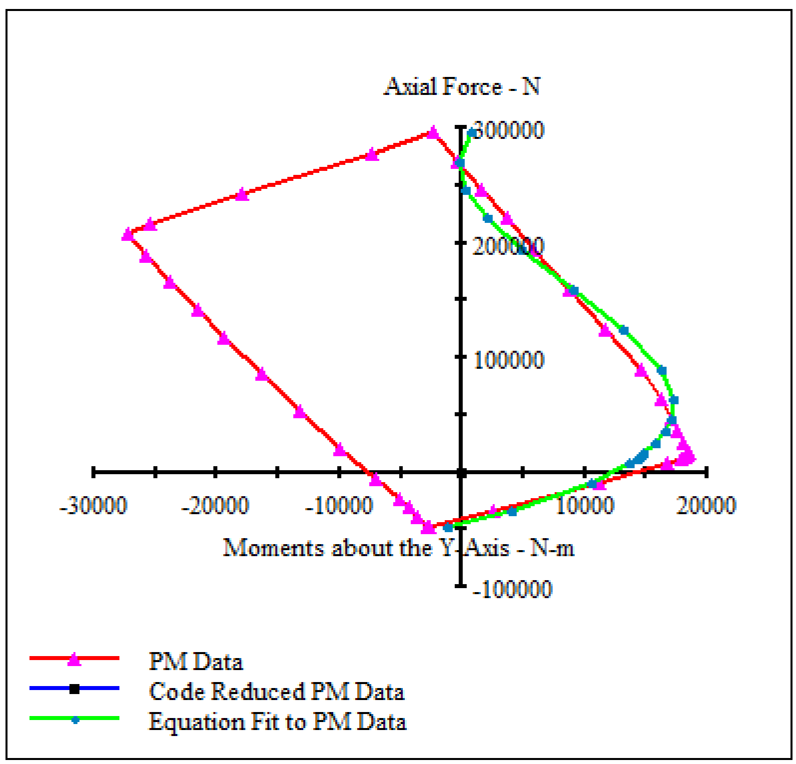
Maximum Code Moment: 0 N-m

P at Max. Code Moment: 0 N

Minimum Code Moment: 0 N-m

P at Min. Code Moment: 0 N

PM Interaction Equation: Units in N-m



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Section Name: PWall18

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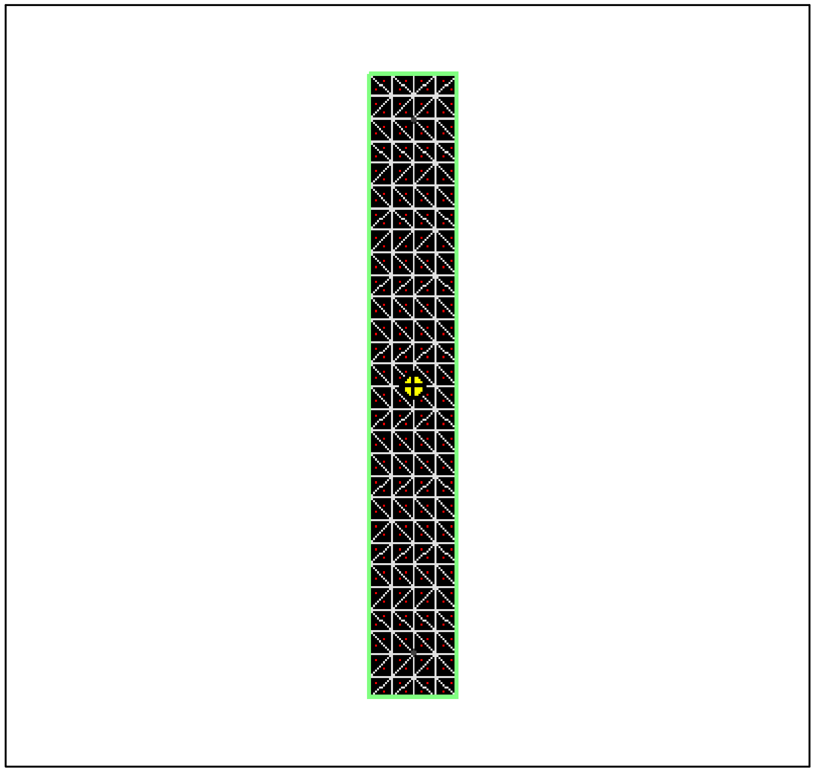
Section Details:

X Centroid:	5.80E-15 mm
Y Centroid:	-5.81E-15 mm
Section Area:	157.5E+3 mm^2
EI gross about X:	13.27E+6 N-m^2
EI gross about Y:	135.6E+3 N-m^2
I trans (ICEB) about X:	2.770E+6 cm^4
I trans (ICEB) about Y:	28.30E+3 cm^4
Reinforcing Bar Area:	235.6 mm^2
Percent Longitudinal Steel:	.1496 %
Overall Width:	150.0 mm
Overall Height:	1050 mm
Number of Fibers:	224
Number of Bars:	3
Number of Materials:	2



Material Types and Names:

User Defined:	ICEB
Strain Hardening Steel:	Grade 30

Comments:

User Comments

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Section Name: PWall18

Loading Name: Interaction

Analysis Type: PM Interaction

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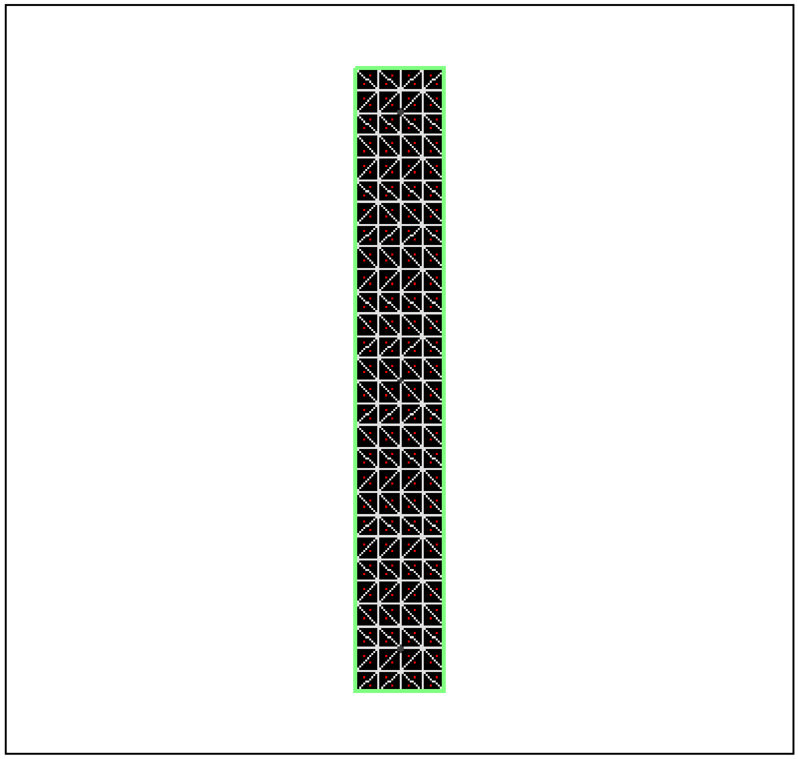
Section Details:

X Centroid: 5.80E-15 mm

Y Centroid: -5.81E-15 mm

Section Area: 157.5E+3 mm^2



Loading Details:

Angle of Loading: 0 deg

Number of Points: 40

Min. ICEB Strain: 2.500E-3 Comp

Max. ICEB Strain: 1.0000 Ten

Min. Grade 30 Strain: 2.000E-3 Comp

Max. Grade 30 Strain: 2.000E-3 Ten

Analysis Results:

Max. Compression Load: 192.7E+3 N

Max. Tension Load: -48.54E+3 N

Maximum Moment: 30.14E+3 N-m

P at Max. Moment: 56.65E+3 N

Minimum Moment: -30.14E+3 N-m

P at Min. Moment: 56.65E+3 N

Moment (Mxx) at P=0: 18.25E+3 N-m

Max. Code Comp. Load: 0 N

Max. Code Ten. Load: 0 N

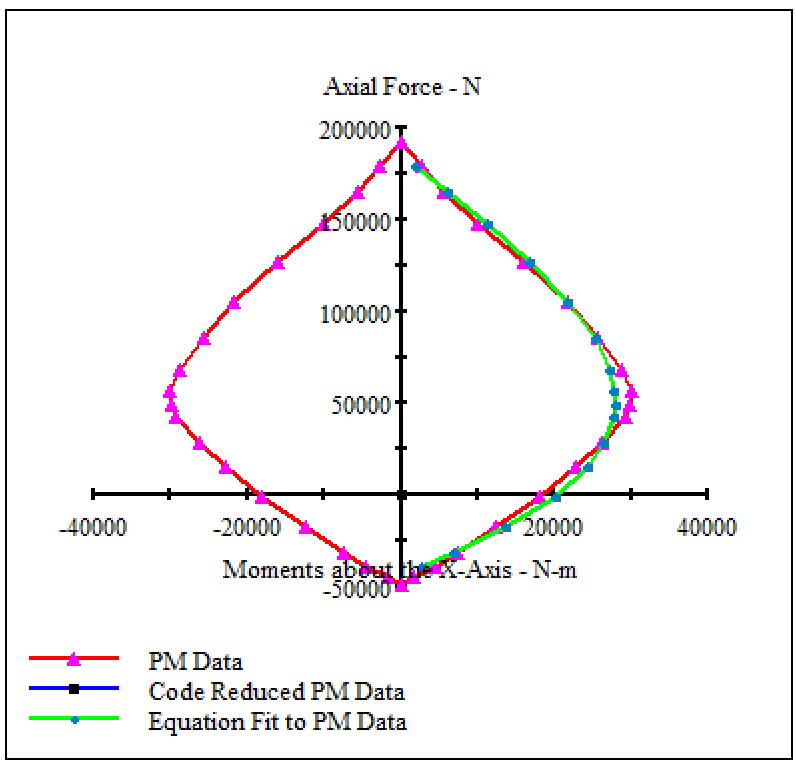
Maximum Code Moment: 0 N-m

P at Max. Code Moment: 0 N

Minimum Code Moment: 0 N-m

P at Min. Code Moment: 0 N

PM Interaction Equation: Units in N-m



Comments:

User Comments

(P/Pu)^2 + 2466*(P/Pu)^3

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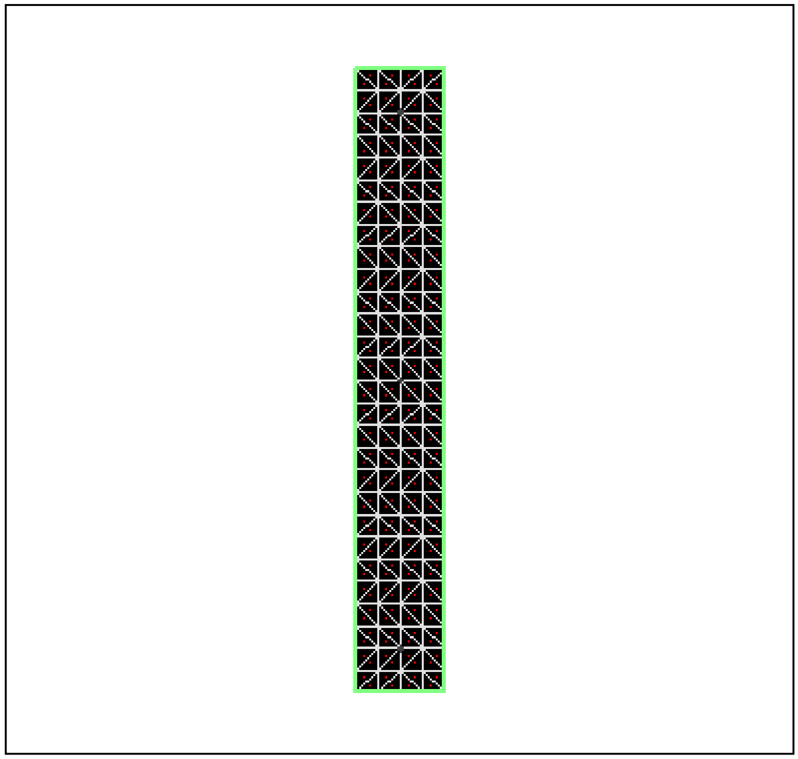
Section Name:	PWall18
Loading Name:	Out of Plane
Analysis Type:	PM Interaction

Section Details:

X Centroid:	5.80E-15 mm
Y Centroid:	-5.81E-15 mm
Section Area:	157.5E+3 mm^2

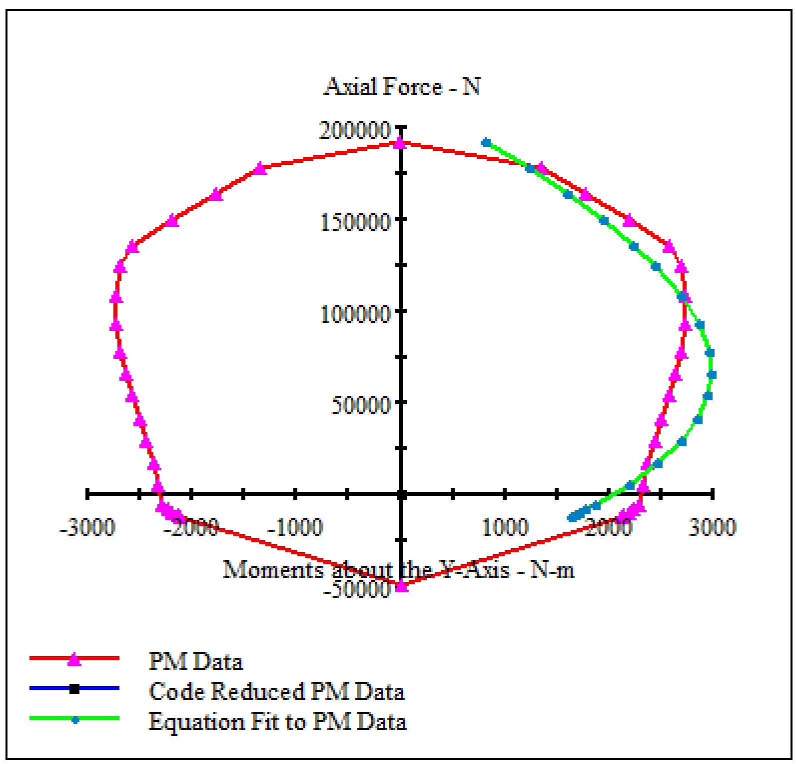


Loading Details:

Angle of Loading:	90 deg
Number of Points:	40
Min. ICEB Strain:	2.500E-3 Comp
Max. ICEB Strain:	1.0000 Ten
Min. Grade 30 Strain:	2.000E-3 Comp
Max. Grade 30 Strain:	2.000E-3 Ten

Analysis Results:

Max. Compression Load:	192.7E+3 N
Max. Tension Load:	-48.54E+3 N
Maximum Moment:	2726 N-m
P at Max. Moment:	108.5E+3 N
Minimum Moment:	-2726 N-m
P at Min. Moment:	108.5E+3 N
Moment (Myy) at P=0:	2303 N-m
Max. Code Comp. Load:	0 N
Max. Code Ten. Load:	0 N
Maximum Code Moment:	0 N-m
P at Max. Code Moment:	0 N
Minimum Code Moment:	0 N-m
P at Min. Code Moment:	0 N
PM Interaction Equation:	Units in N-m



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Section Name: PWallA

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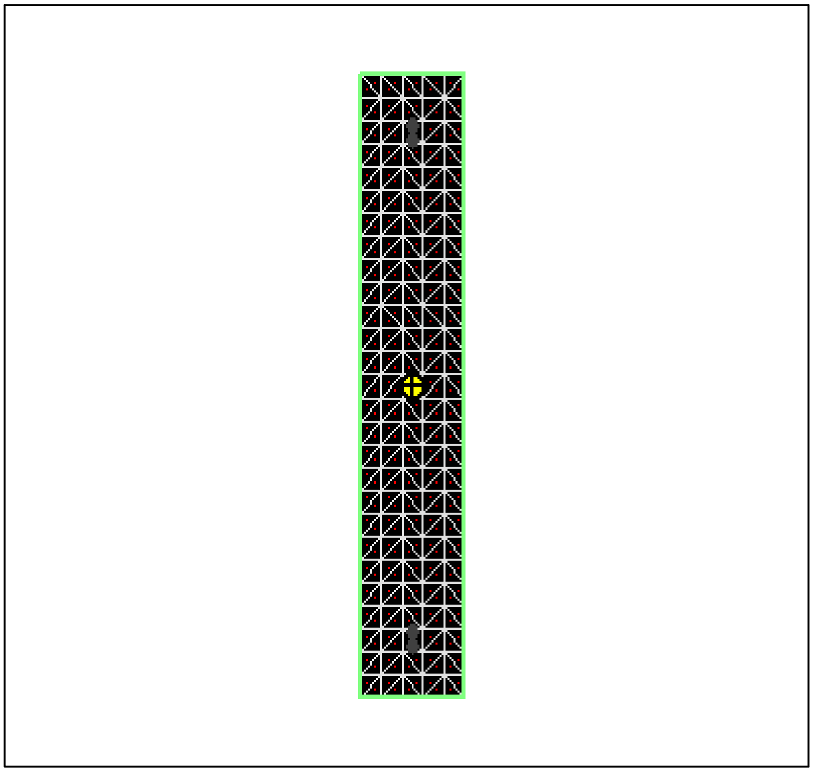
Section Details:

X Centroid:	-5.60E-16 mm
Y Centroid:	4.05E-15 mm
Section Area:	135.0E+3 mm^2
EI gross about X:	13.27E+6 N-m^2
EI gross about Y:	135.6E+3 N-m^2
I trans (ICEB) about X:	7.884E+6 cm^4
I trans (ICEB) about Y:	24.64E+3 cm^4
Reinforcing Bar Area:	1257 mm^2
Percent Longitudinal Steel:	.9310 %
Overall Width:	150.0 mm
Overall Height:	900.0 mm
Number of Fibers:	270
Number of Bars:	4
Number of Materials:	2



Material Types and Names:

User Defined:	ICEB
Strain Hardening Steel:	Grade 30

Comments:

User Comments

-3*(P/Pu)^2 + 61.01E+3*(P/Pu)^3

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Section Name:	PWallA
Loading Name:	Interaction
Analysis Type:	PM Interaction

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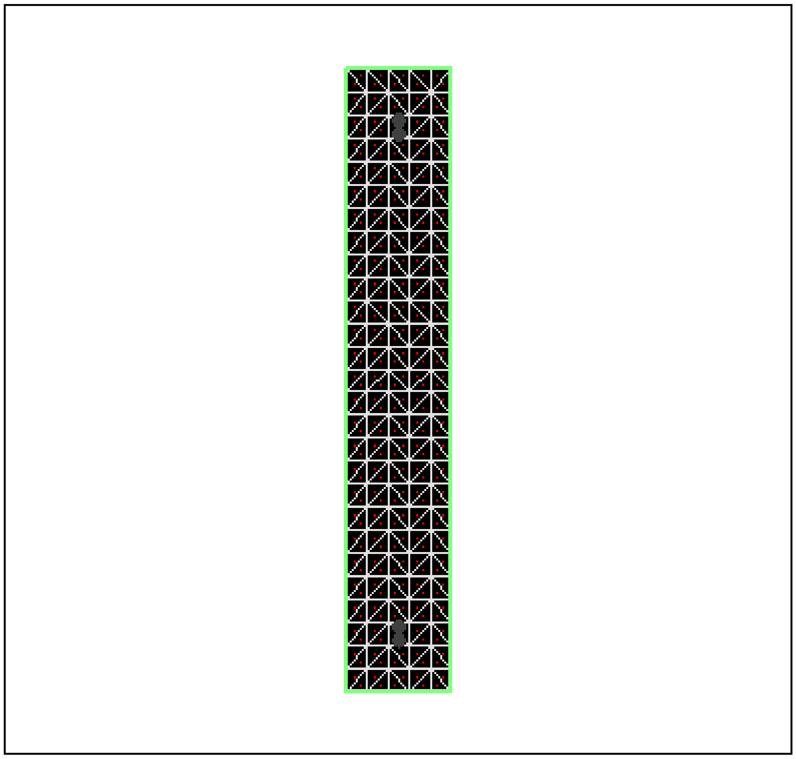
Section Details:

X Centroid:	-5.60E-16 mm
Y Centroid:	4.05E-15 mm
Section Area:	135.0E+3 mm^2

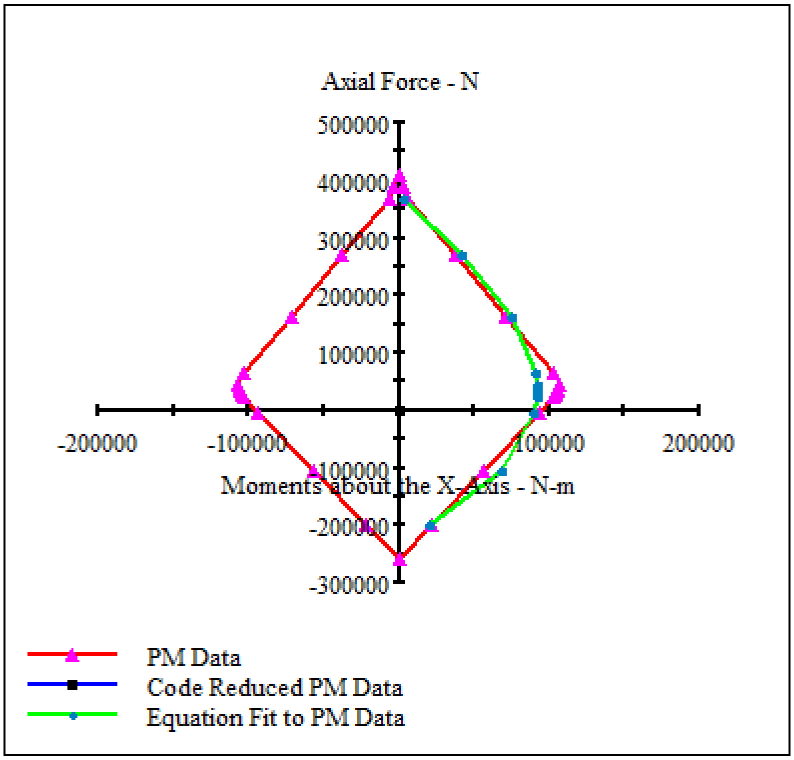


Loading Details:

Angle of Loading:	0 deg
Number of Points:	40
Min. ICEB Strain:	2.500E-3 Comp
Max. ICEB Strain:	1.0000 Ten
Min. Grade 30 Strain:	5.000E-3 Comp
Max. Grade 30 Strain:	5.000E-3 Ten

Analysis Results:

Max. Compression Load:	408.0E+3 N
Max. Tension Load:	-258.9E+3 N
Maximum Moment:	106.7E+3 N-m
P at Max. Moment:	43.56E+3 N
Minimum Moment:	-106.7E+3 N-m
P at Min. Moment:	43.56E+3 N
Moment (Mxx) at P=0:	94.74E+3 N-m
Max. Code Comp. Load:	0 N
Max. Code Ten. Load:	0 N
Maximum Code Moment:	0 N-m
P at Max. Code Moment:	0 N
Minimum Code Moment:	0 N-m
P at Min. Code Moment:	0 N
PM Interaction Equation:	Units in N-m



(P/Pu)^2 + -349.6*(P/Pu)^3

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Section Name: PWallA

Loading Name: Out of Plane

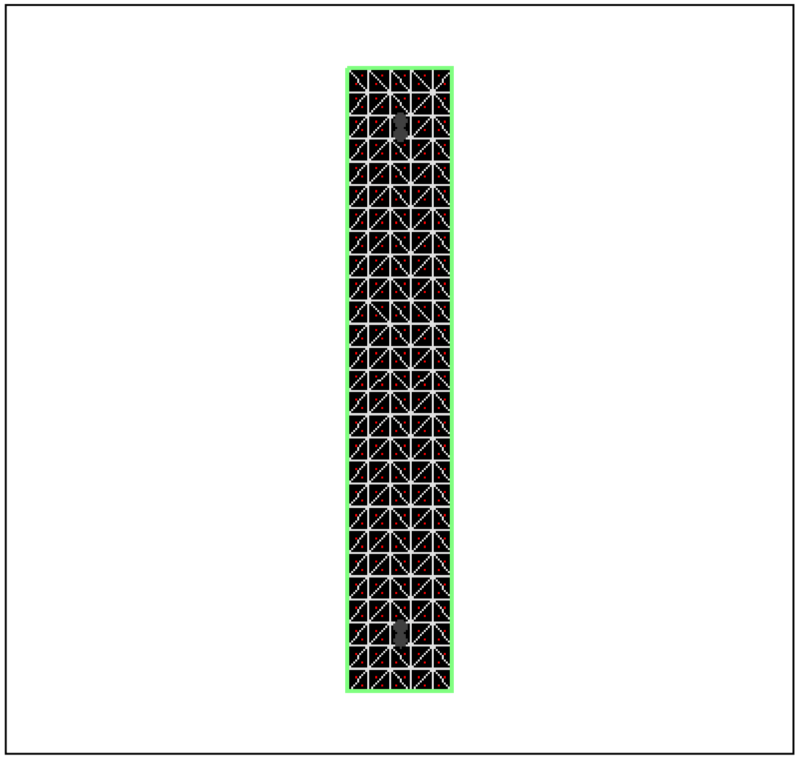
Analysis Type: PM Interaction

Section Details:

X Centroid: -5.60E-16 mm

Y Centroid: 4.05E-15 mm

Section Area: 135.0E+3 mm^2



Loading Details:

Angle of Loading: 90 deg

Number of Points: 40

Min. ICEB Strain: 2.500E-3 Comp

Max. ICEB Strain: 1.0000 Ten

Min. Grade 30 Strain: 2.000E-3 Comp

Max. Grade 30 Strain: 2.000E-3 Ten

Analysis Results:

Max. Compression Load: 381.5E+3 N

Max. Tension Load: -258.9E+3 N

Maximum Moment: 2317 N-m

P at Max. Moment: 260.3E+3 N

Minimum Moment: -2317 N-m

P at Min. Moment: 260.3E+3 N

Moment (Myy) at P=0: 2108 N-m

Max. Code Comp. Load: 0 N

Max. Code Ten. Load: 0 N

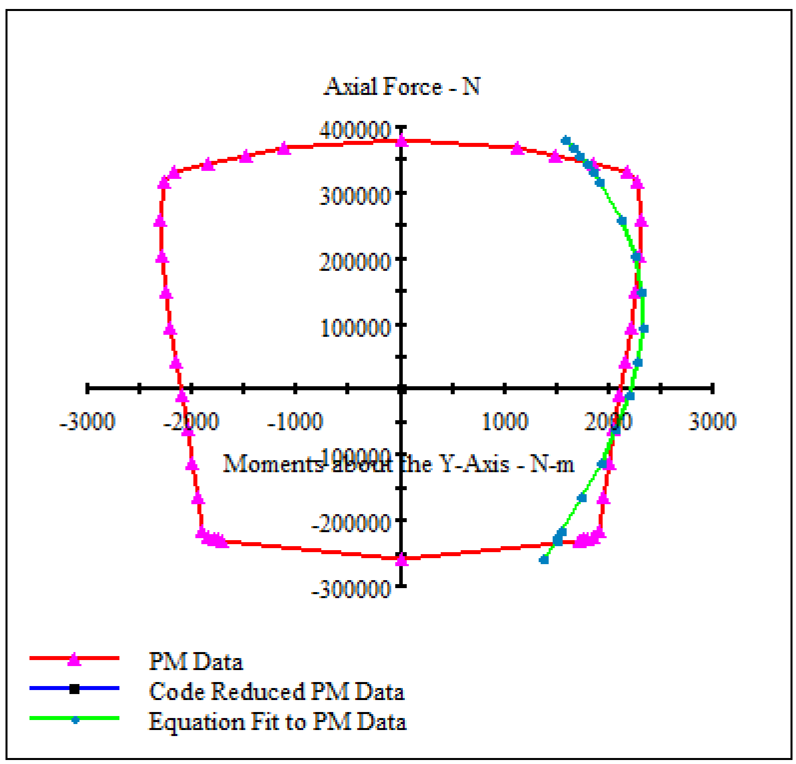
Maximum Code Moment: 0 N-m

P at Max. Code Moment: 0 N

Minimum Code Moment: 0 N-m

P at Min. Code Moment: 0 N

PM Interaction Equation: Units in N-m



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Section Name: PWallC

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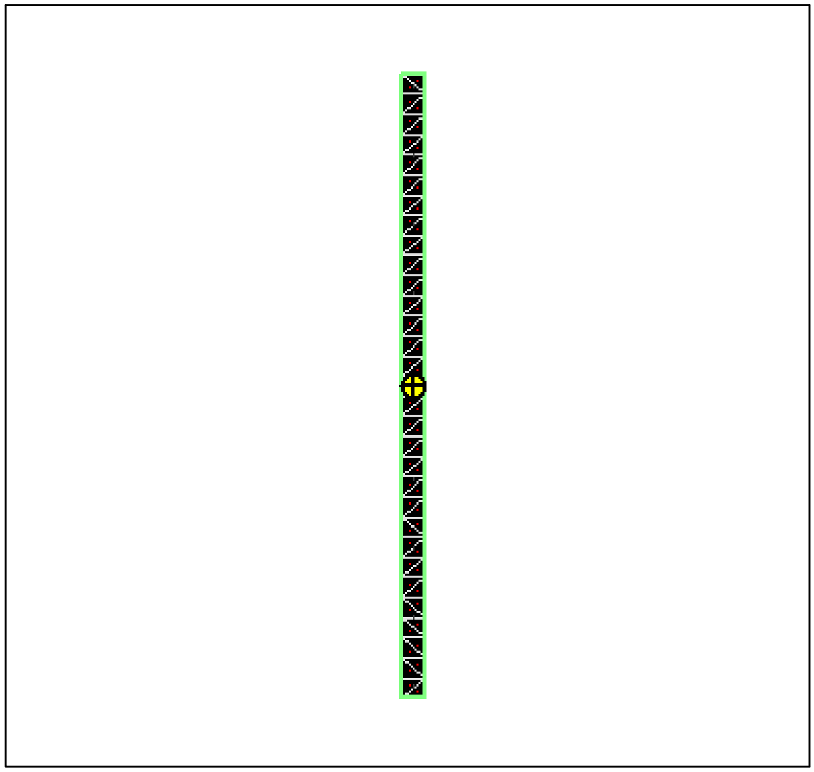
Section Details:

X Centroid:	-2.33E-18 mm
Y Centroid:	-3.41E-15 mm
Section Area:	607.5E+3 mm^2
EI gross about X:	13.27E+6 N-m^2
EI gross about Y:	135.6E+3 N-m^2
I trans (ICEB) about X:	132.25E+6 cm^4
I trans (ICEB) about Y:	37.97E+3 cm^4
Reinforcing Bar Area:	785.4 mm^2
Percent Longitudinal Steel:	.1293 %
Overall Width:	150.0 mm
Overall Height:	4050 mm
Number of Fibers:	62
Number of Bars:	10
Number of Materials:	2



Material Types and Names:

User Defined:	ICEB
Strain Hardening Steel:	Grade 30

Comments:

User Comments

(P/Pu)^2 + -11.40E+3(P/Pu)^3

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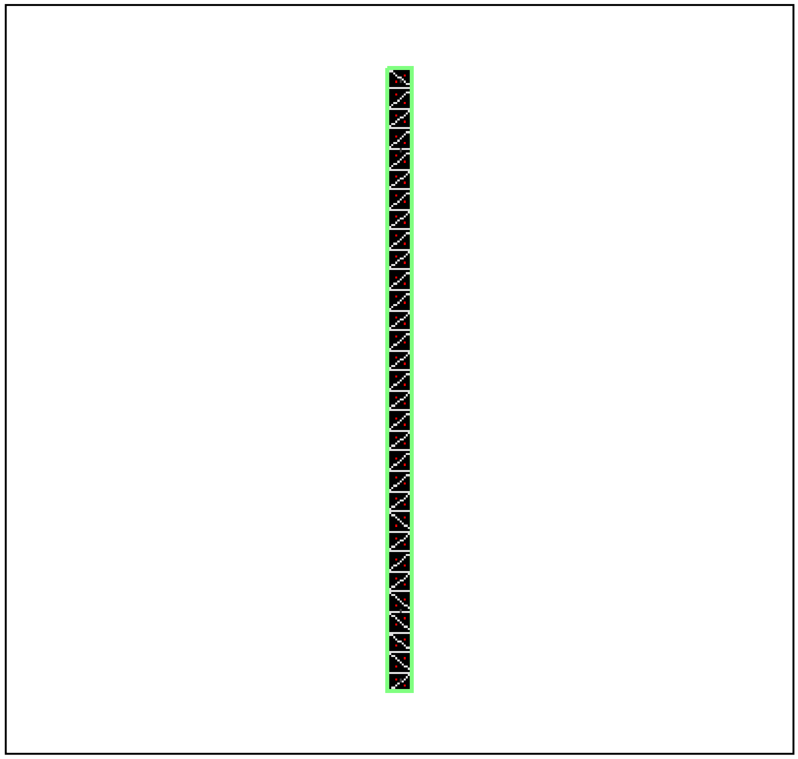
Section Name:	PWallC
Loading Name:	Out of Plane
Analysis Type:	PM Interaction

Section Details:

X Centroid:	-2.33E-18 mm
Y Centroid:	-3.41E-15 mm
Section Area:	607.5E+3 mm^2

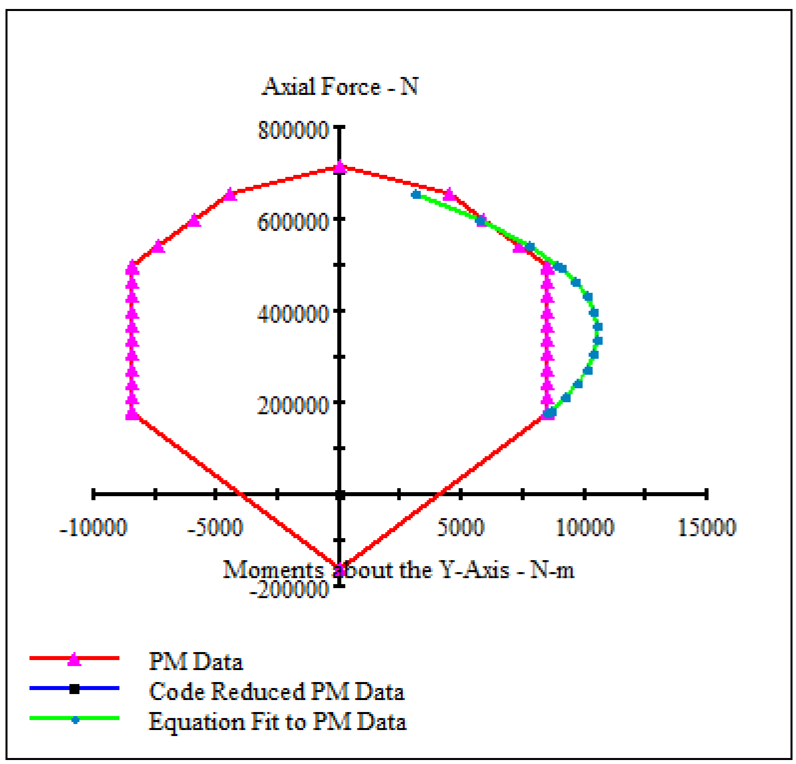


Loading Details:

Angle of Loading:	90 deg
Number of Points:	40
Min. ICEB Strain:	2.500E-3 Comp
Max. ICEB Strain:	1.0000 Ten
Min. Grade 30 Strain:	2.000E-3 Comp
Max. Grade 30 Strain:	2.000E-3 Ten

Analysis Results:

Max. Compression Load:	718.0E+3 N
Max. Tension Load:	-161.8E+3 N
Maximum Moment:	8466 N-m
P at Max. Moment:	500.0E+3 N
Minimum Moment:	-8466 N-m
P at Min. Moment:	500.0E+3 N
Moment (Myy) at P=0:	4045 N-m
Max. Code Comp. Load:	0 N
Max. Code Ten. Load:	0 N
Maximum Code Moment:	0 N-m
P at Max. Code Moment:	0 N
Minimum Code Moment:	0 N-m
P at Min. Code Moment:	0 N
PM Interaction Equation:	Units in N-m



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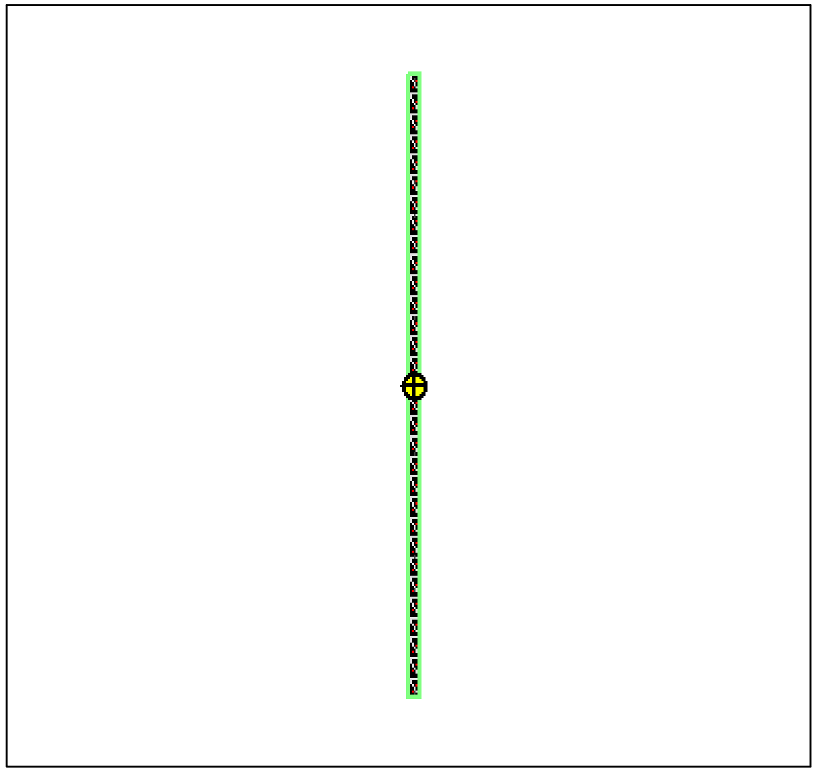
Section Name: PWallD

Section Details:



X Centroid:	-7.22E-16 mm
Y Centroid:	-8.93E-15 mm
Section Area:	1.238E+6 mm^2
EI gross about X:	17.09E+9 N-m^2
EI gross about Y:	613.2E+3 N-m^2
I trans (ICEB) about X:	1.245E+9 cm^4
I trans (ICEB) about Y:	77.34E+3 cm^4
Reinforcing Bar Area:	2149 mm^2
Percent Longitudinal Steel:	.1736 %
Overall Width:	150.0 mm
Overall Height:	8250 mm
Number of Fibers:	62
Number of Bars:	19
Number of Materials:	2

Material Types and Names:

User Defined:	ICEB
Strain Hardening Steel:	Grade 30

Comments:

User Comments

E+3*(P/Pu)^2 + 5238*(P/Pu)^3

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Section Name: PWallD

Loading Name: Out of Plane

Analysis Type: PM Interaction

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Section Details:

X Centroid: -7.22E-16 mm

Y Centroid: -8.93E-15 mm

Section Area: 1.238E+6 mm^2

Loading Details:

Angle of Loading: 90 deg

Number of Points: 40

Min. ICEB Strain: 2.500E-3 Comp

Max. ICEB Strain: 1.0000 Ten

Min. Grade 30 Strain: 2.000E-3 Comp

Max. Grade 30 Strain: 2.000E-3 Ten

Analysis Results:

Max. Compression Load: 1.575E+6 N

Max. Tension Load: -442.7E+3 N

Maximum Moment: 17.25E+3 N-m

P at Max. Moment: 1.131E+6 N

Minimum Moment: -17.25E+3 N-m

P at Min. Moment: 1.131E+6 N

Moment (Myy) at P=0: 11.07E+3 N-m

Max. Code Comp. Load: 0 N

Max. Code Ten. Load: 0 N

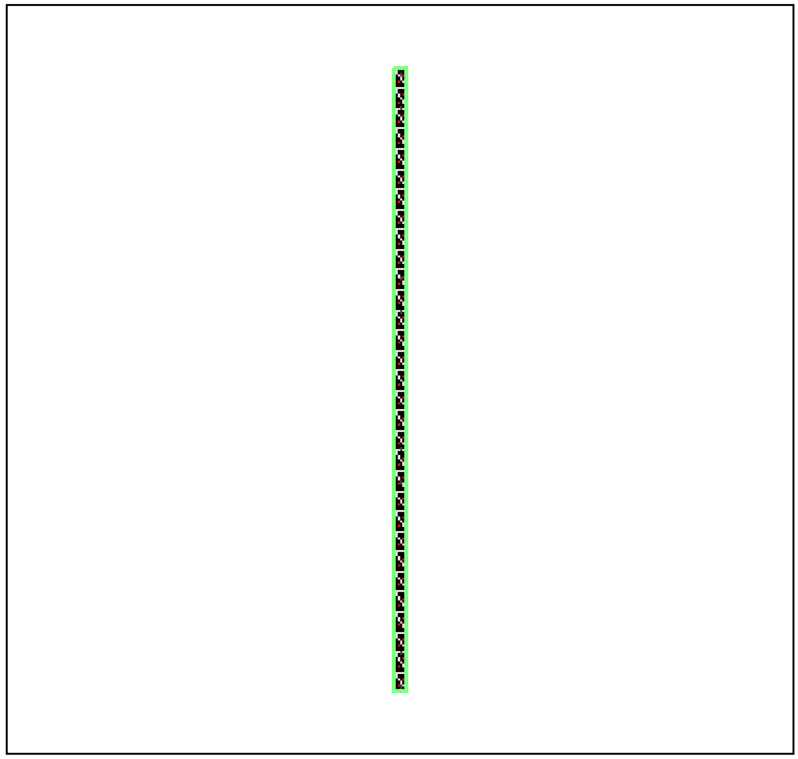
Maximum Code Moment: 0 N-m

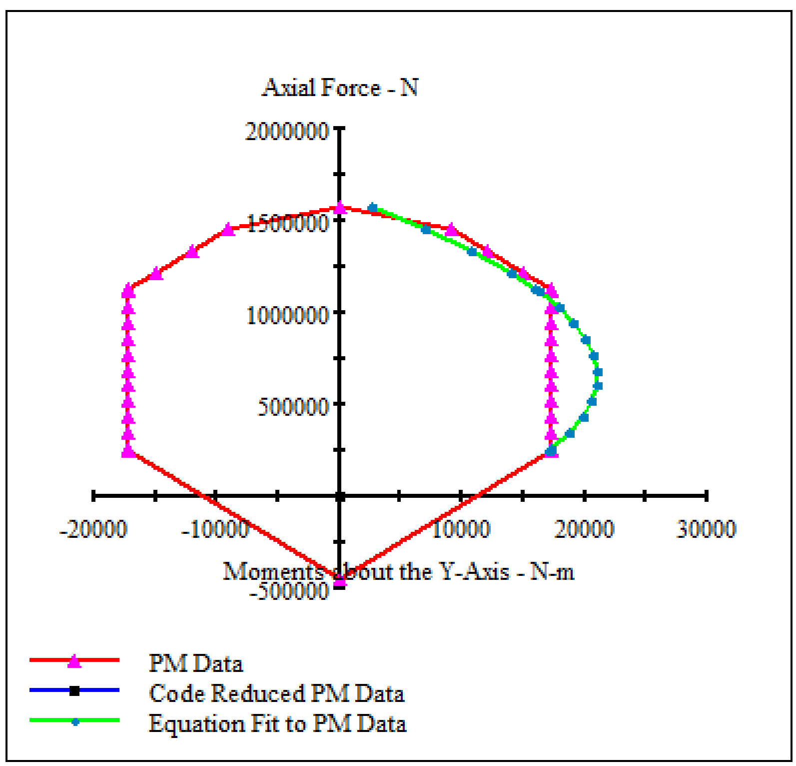
P at Max. Code Moment: 0 N

Minimum Code Moment: 0 N-m

P at Min. Code Moment: 0 N

PM Interaction Equation: Units in N-m





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Section Name: Ring Beam

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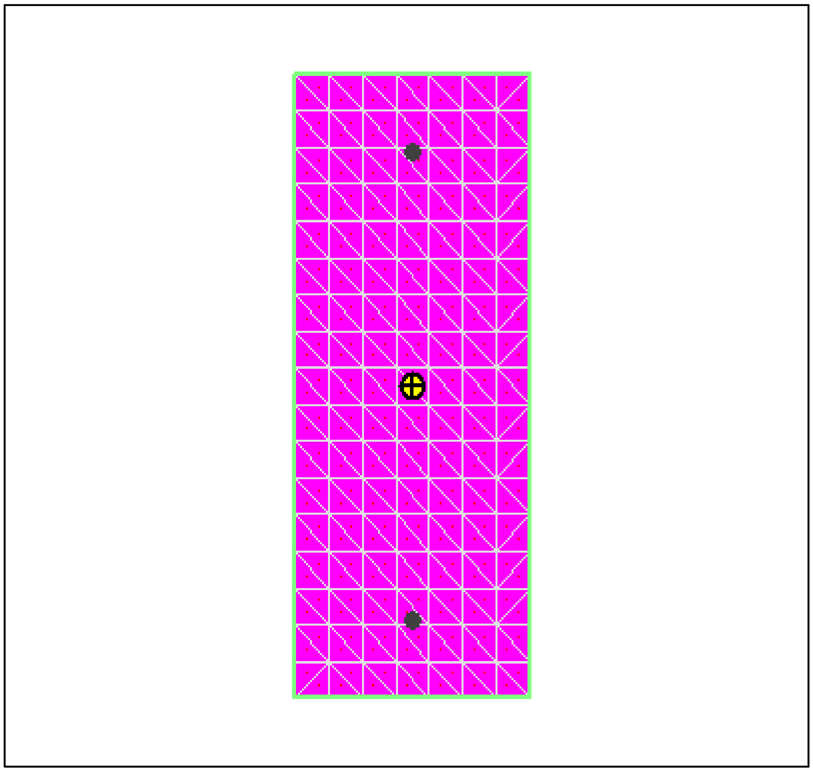
Section Details:

X Centroid:	-1.96E-14 mm
Y Centroid:	1.49E-16 mm
Section Area:	60.00E+3 mm^2
EI gross about X:	13.27E+6 N-m^2
EI gross about Y:	135.6E+3 N-m^2
I trans (Concrete) about X:	82.72E+3 cm^4
I trans (Concrete) about Y:	11.10E+3 cm^4
Reinforcing Bar Area:	157.1 mm^2
Percent Longitudinal Steel:	.2618 %
Overall Width:	150.0 mm
Overall Height:	400.0 mm
Number of Fibers:	238
Number of Bars:	2
Number of Materials:	2



Material Types and Names:

Strain Hardening Steel:	Grade 30
Unconfined Concrete:	Concrete

Comments:

User Comments

(P/Pu)^2 + 110.5E+3(P/Pu)^3

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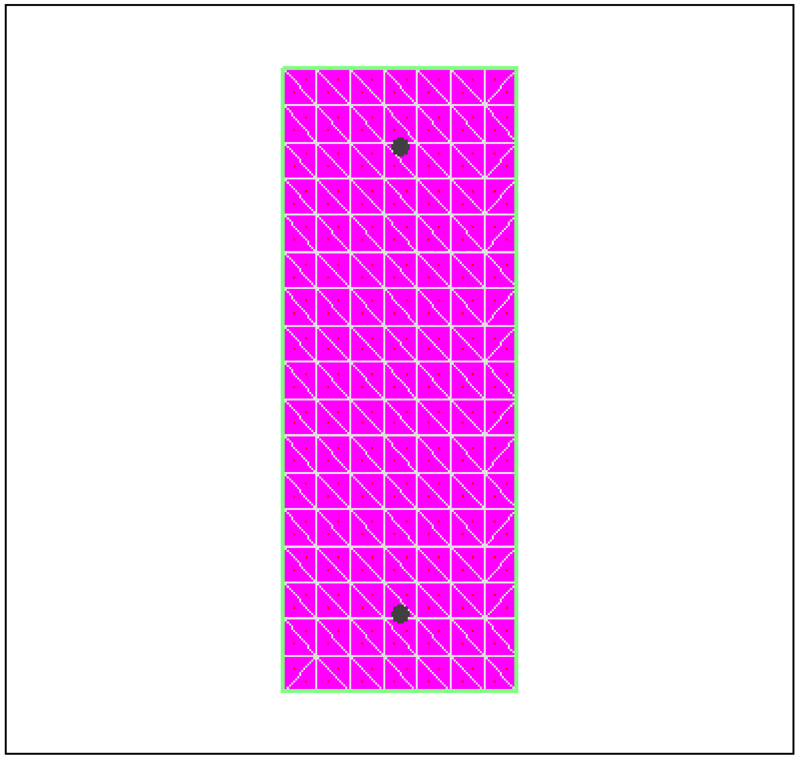
Section Name:	Ring Beam
Loading Name:	Interaction
Analysis Type:	PM Interaction

Section Details:

X Centroid:	-1.96E-14 mm
Y Centroid:	1.49E-16 mm
Section Area:	60.00E+3 mm^2

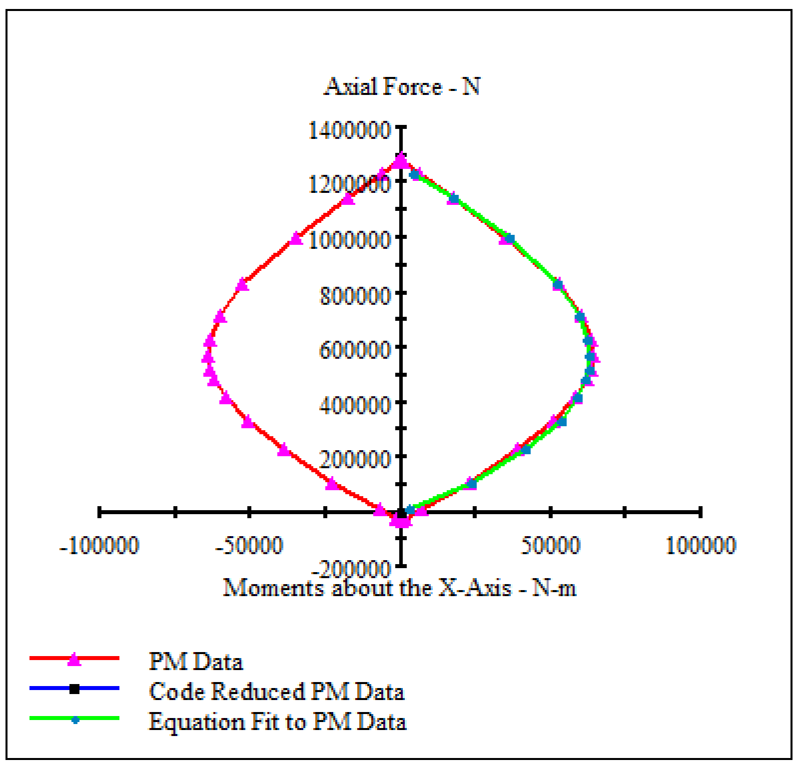


Loading Details:

Angle of Loading:	0 deg
Number of Points:	40
Min. Grade 30 Strain:	2.000E-3 Comp
Max. Grade 30 Strain:	2.000E-3 Ten
Min. Concrete Strain:	3.000E-3 Comp
Max. Concrete Strain:	1.0000 Ten

Analysis Results:

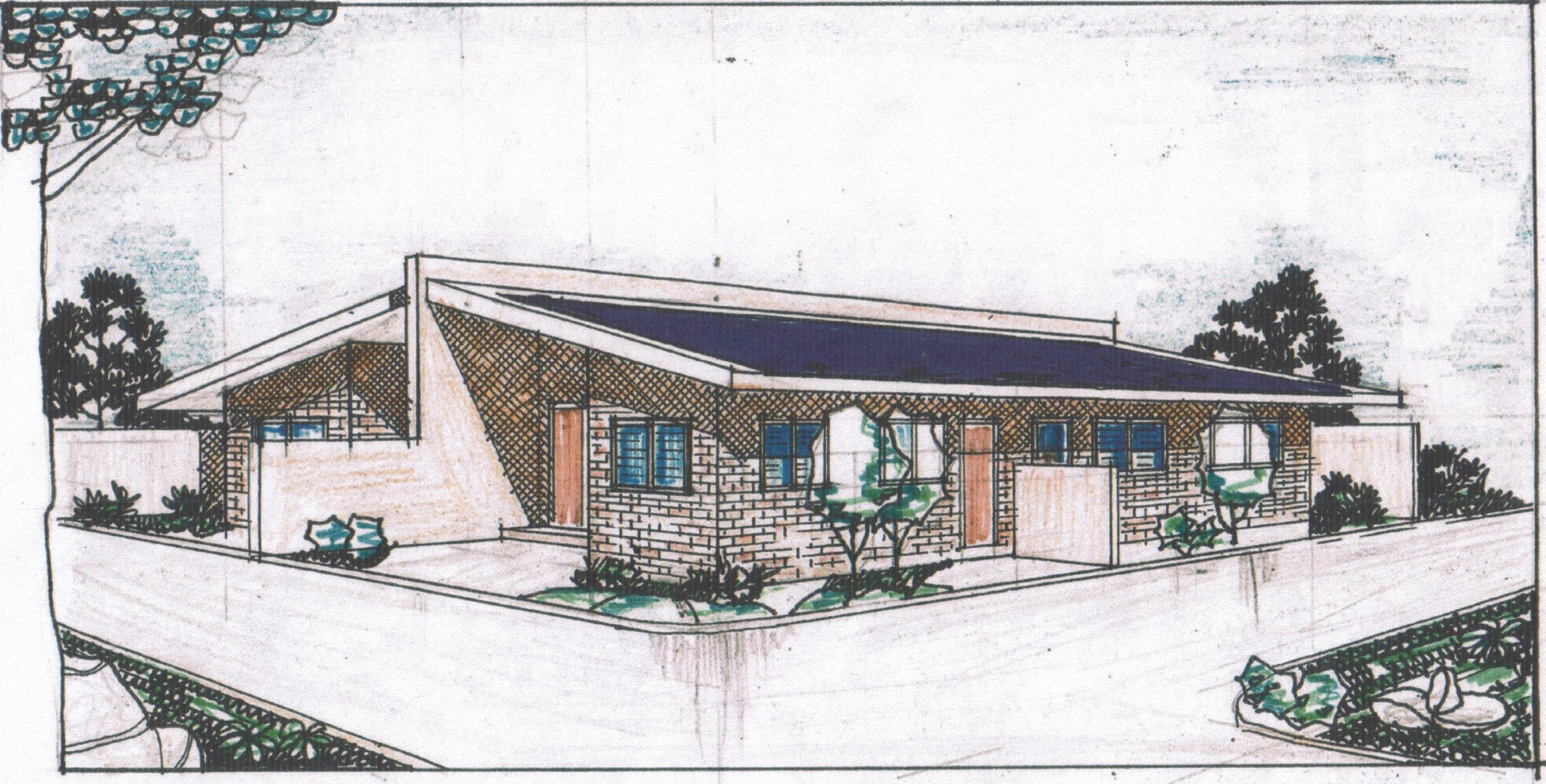
Max. Compression Load:	1.289E+6 N
Max. Tension Load:	-32.36E+3 N
Maximum Moment:	64.13E+3 N-m
P at Max. Moment:	567.6E+3 N
Minimum Moment:	-64.13E+3 N-m
P at Min. Moment:	567.6E+3 N
Moment (Mxx) at P=0:	5439 N-m
Max. Code Comp. Load:	0 N
Max. Code Ten. Load:	0 N
Maximum Code Moment:	0 N-m
P at Max. Code Moment:	0 N
Minimum Code Moment:	0 N-m
P at Min. Code Moment:	0 N
PM Interaction Equation:	Units in N-m



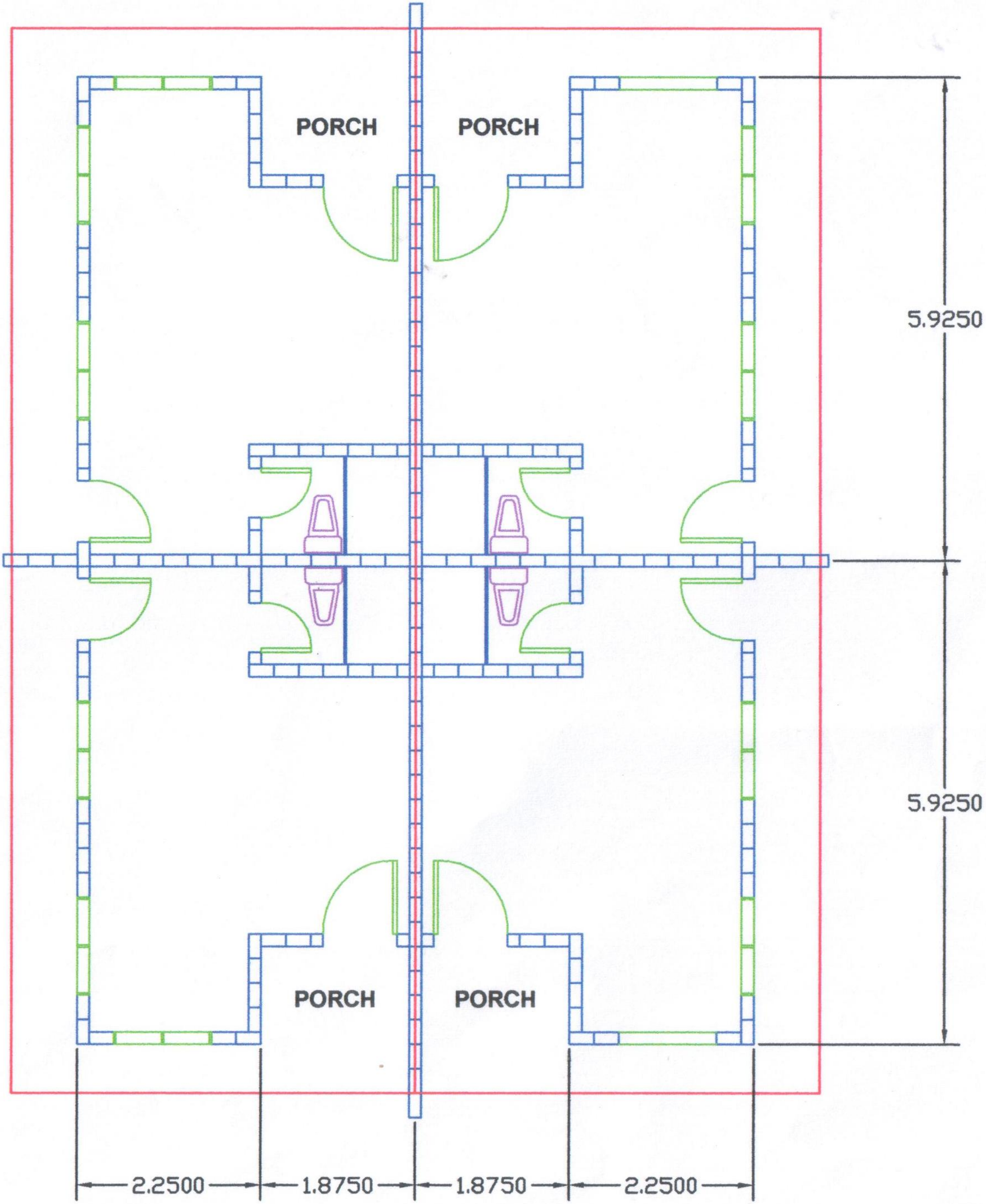
Comments:

User Comments

Appendix C: Quadruplex Design Drawings

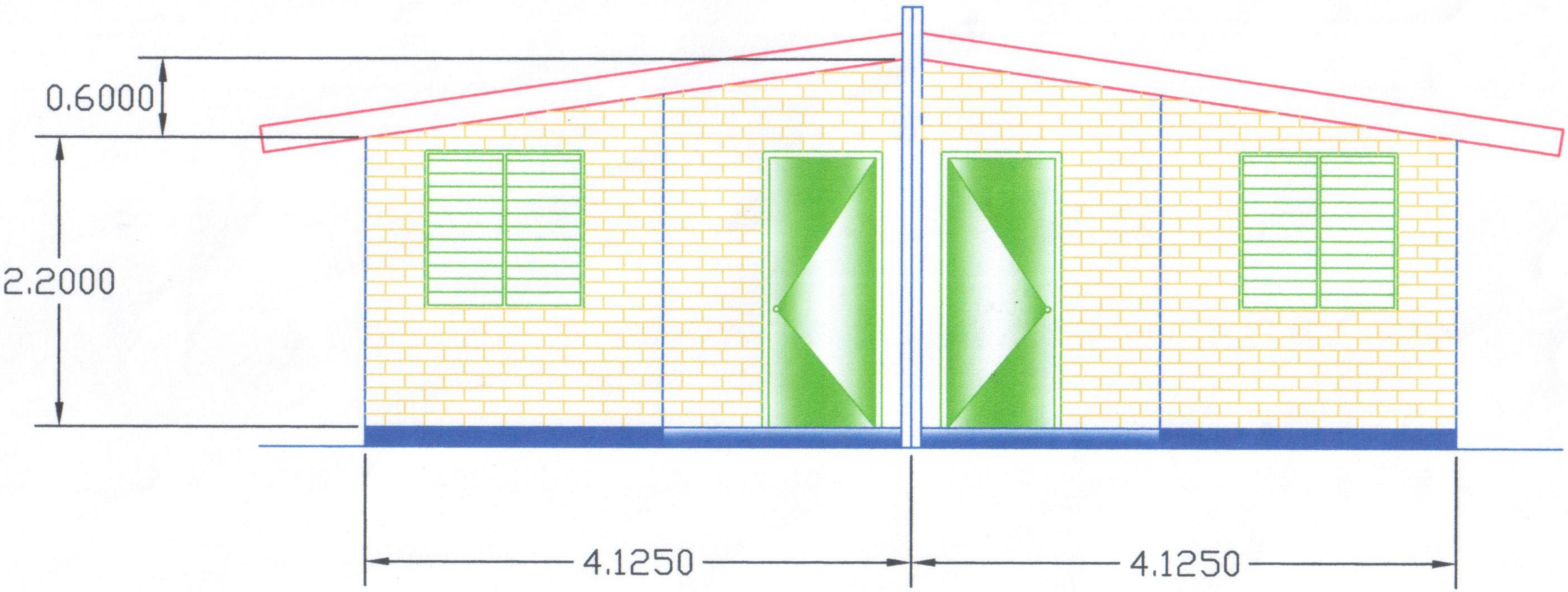


PERSPECTIVE
QUADROPLEX

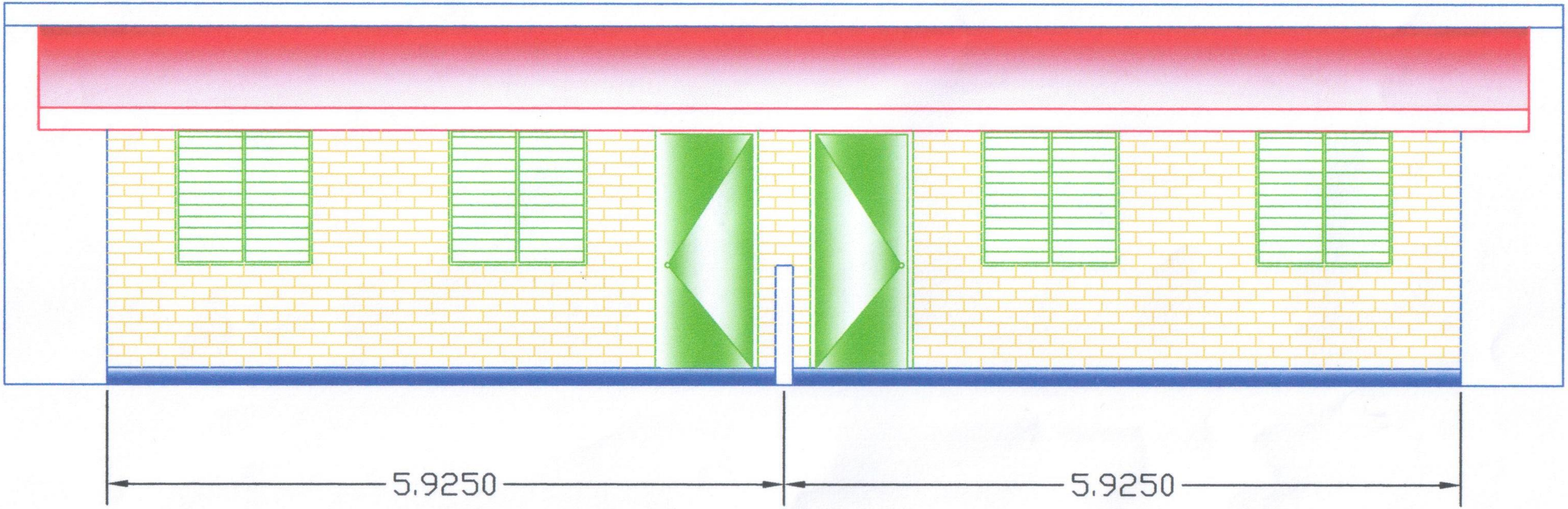


PORCH
PORCH
5.9250
5.9250
PORCH
PORCH
2.2500
1.8750
1.8750
2.2500

FLOOR PLAN



FRONT ELEVATION



SIDE ELEVATION

General Notes

1. Material Properties

Compressed earth blocks shall have a minimum 28-day compressive strength of 3 MPa (435 psi). Soil mixture shall have approximately 15% clay content (particles less than 0.005mm in diameter) with 6% minimum cement content by weight.
Grout shall have a minimum 28-day compressive strength of 5 MPa (725 psi). Grout mixture shall be 1.0 : 0.4 : 2.6 : 4.2 (portland cement : lime : water : sand) by dry volume.
Concrete for foundations and ring beams at tops of walls shall have a minimum 28-day compressive strength of 21 MPa (3000 psi).
Vertical steel shall be minimum 12 mm bar with fy=206 MPa (Gr. 30). Horizontal steel shall be minimum 10 mm bar with fy=206 MPa (Gr. 30). Vertical steel shall be placed per plan with a maximum spacing of 1200 mm and at all wall ends and door jambs. Horizontal steel shall be placed per plan with a maximum spacing of 600 mm.

f'm (ICEB) = 3 MPa (435 psi)
f'c (Foundations and ring beams) = 21 MPa (3000 psi)
fy = 206 MPa (Gr. 30)

2. Foundations

All walls shall be founded on strip foundations per plan. The foundation plan is to be designed by others.

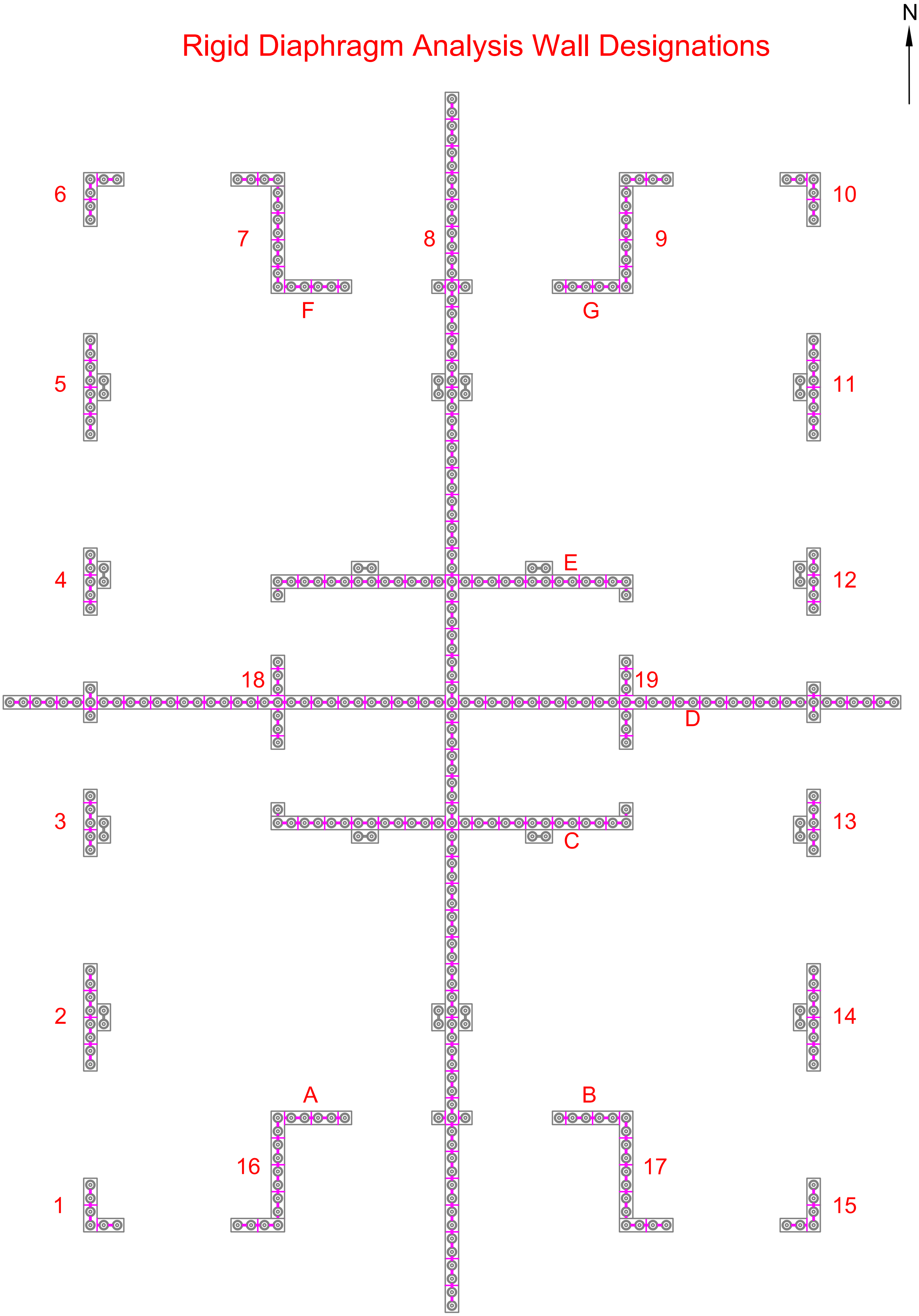
3. Roofing

All walls shall be sloped to match the roofing system. This system is to be designed by others.

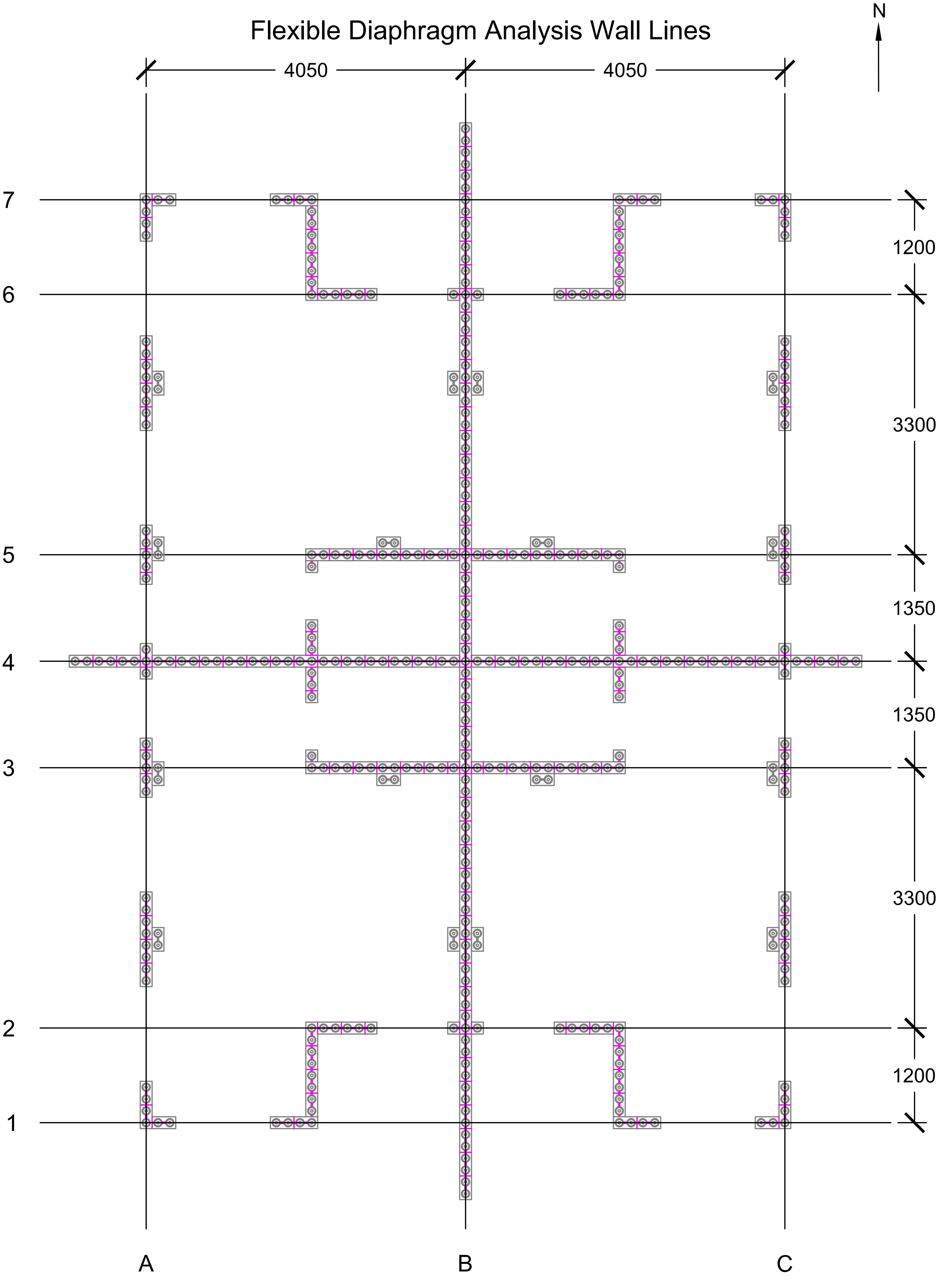
4. Construction Techniques

Grout is to be poured in lifts no higher than 300mm. Each grout lift shall be ended 40mm below the bed joint of a block to create a grout key at the cold joint. Channel blocks shall be grouted to 10mm below the bed joint to create a grout key in the channel. Horizontal rebar must be completely bonded to grout. Walls shall be partially or fully grouted with each grout channel cell grouted and each cell with reinforcement grouted. Channel blocks shall be fully grouted. Horizontal rebar shall be hooked 180° around vertical rebar at ends of walls and where necessary.

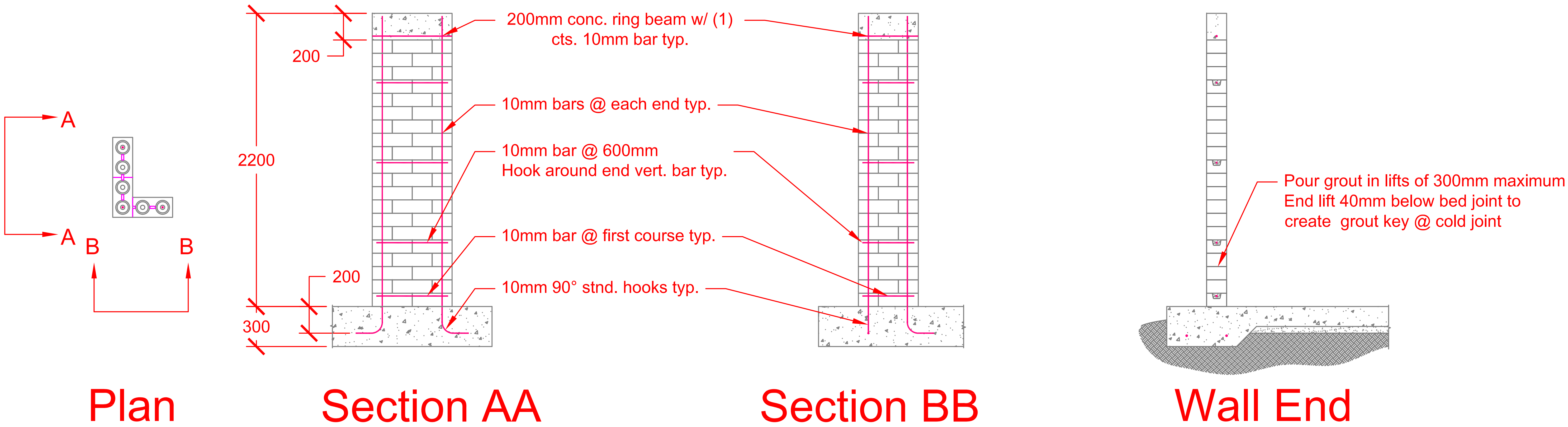
Rigid Diaphragm Analysis Wall Designations



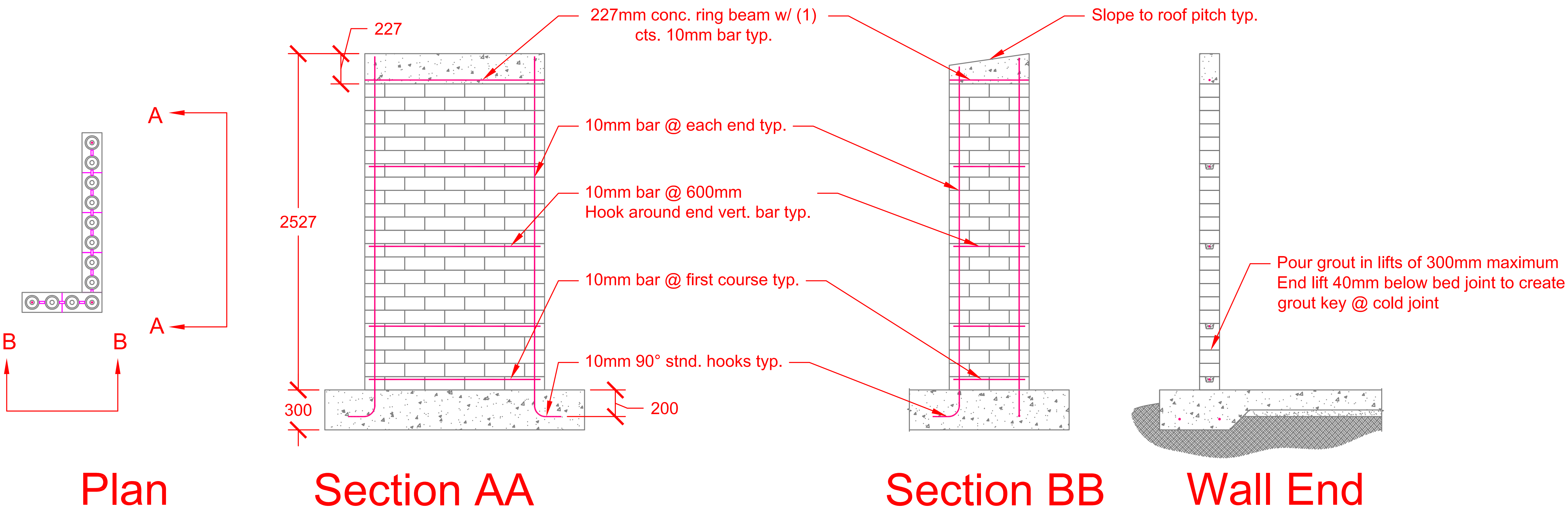
Flexible Diaphragm Analysis Wall Lines



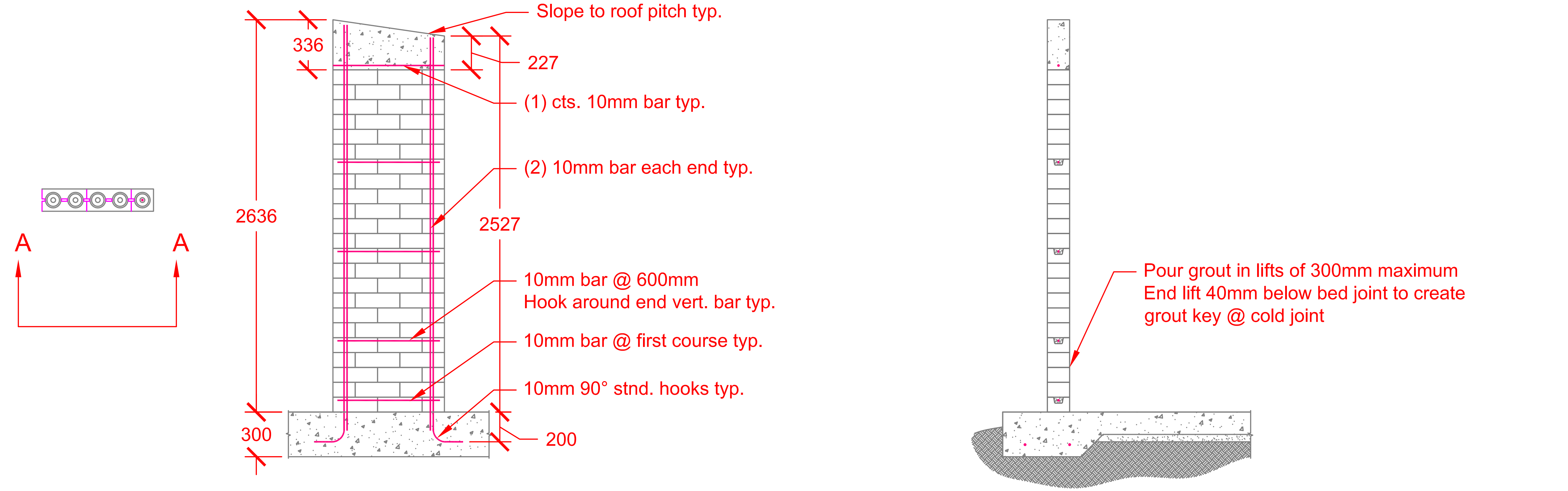
Walls 1,6,10,15 1:30



Walls 7,9,16,17 1:30

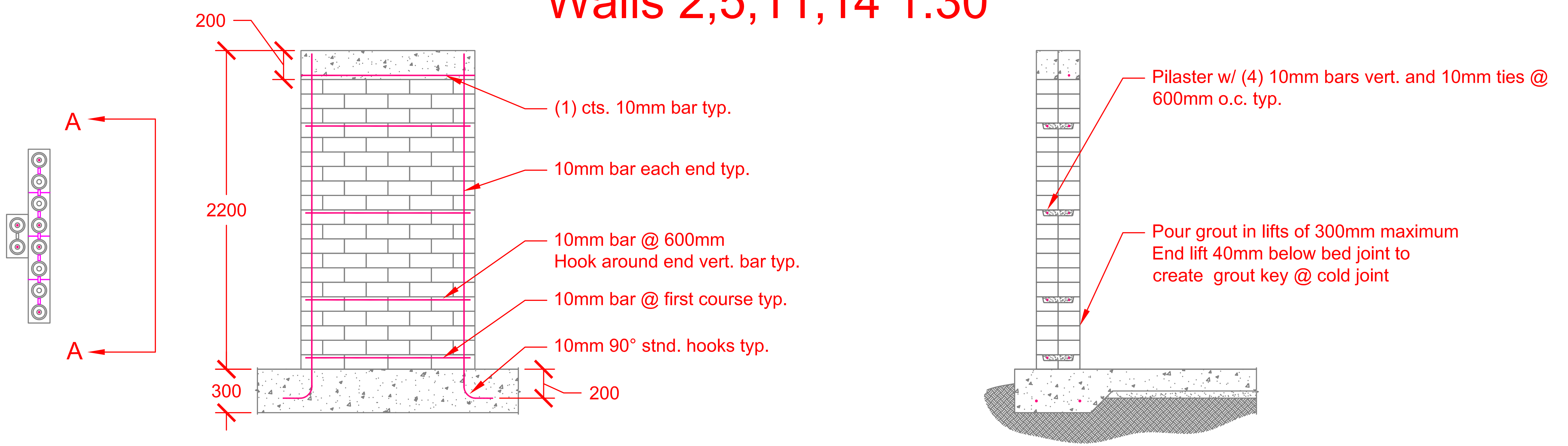


Walls A,B,F,G 1:30



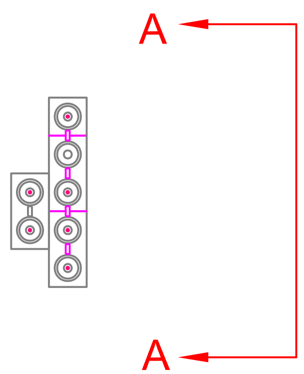
Plan Section AA Wall End

Walls 2,5,11,14 1:30

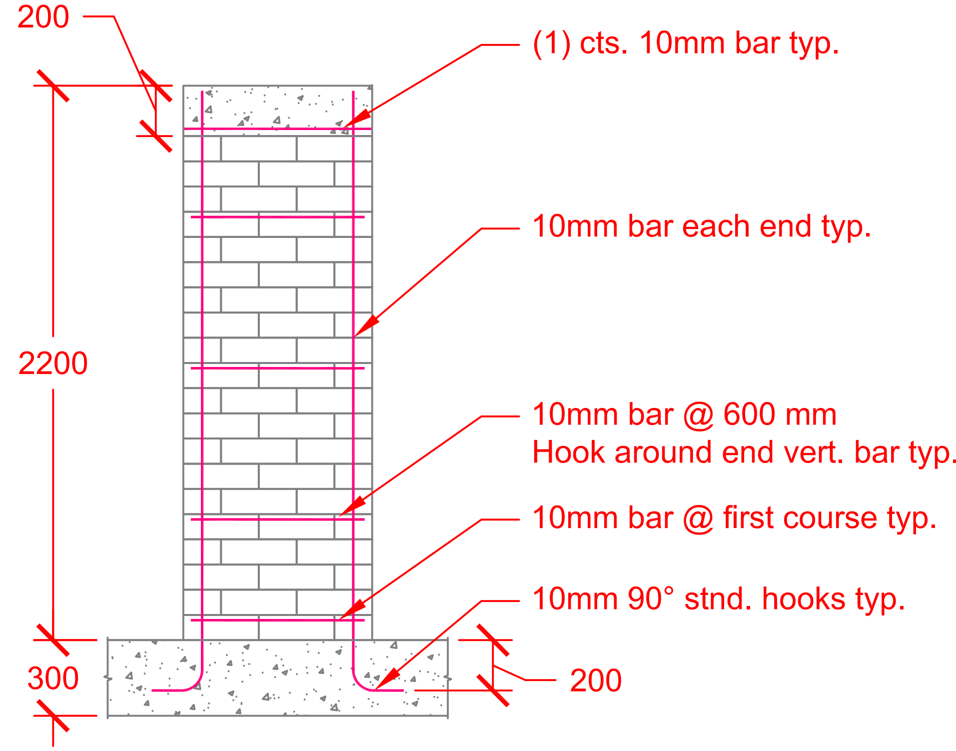


Plan Section AA Wall End

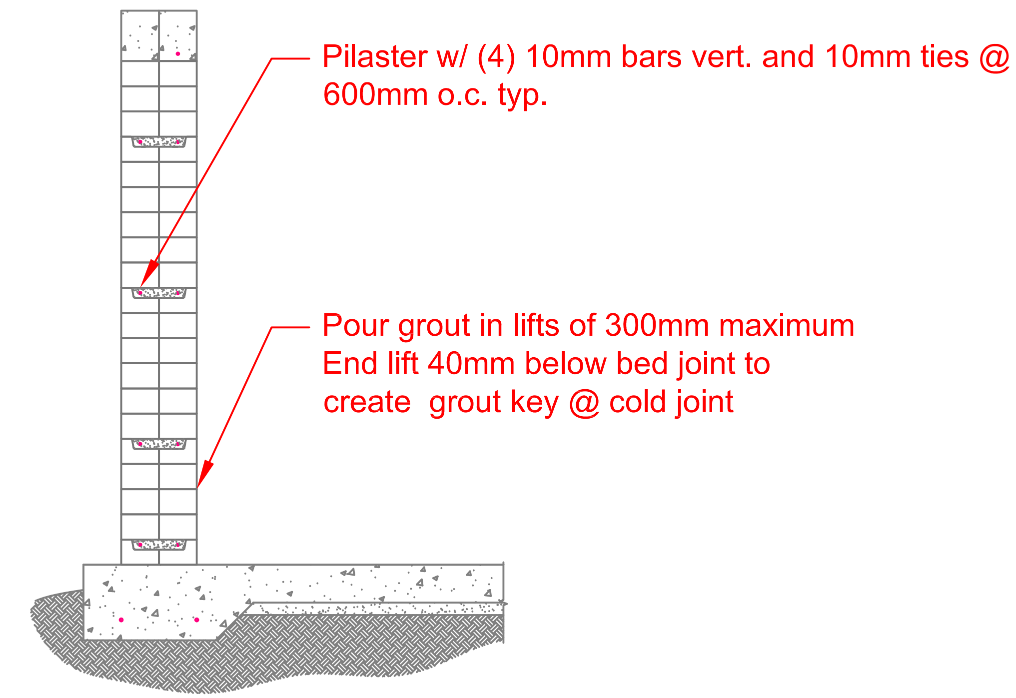
Walls 3,4,12,13 1:30



Plan

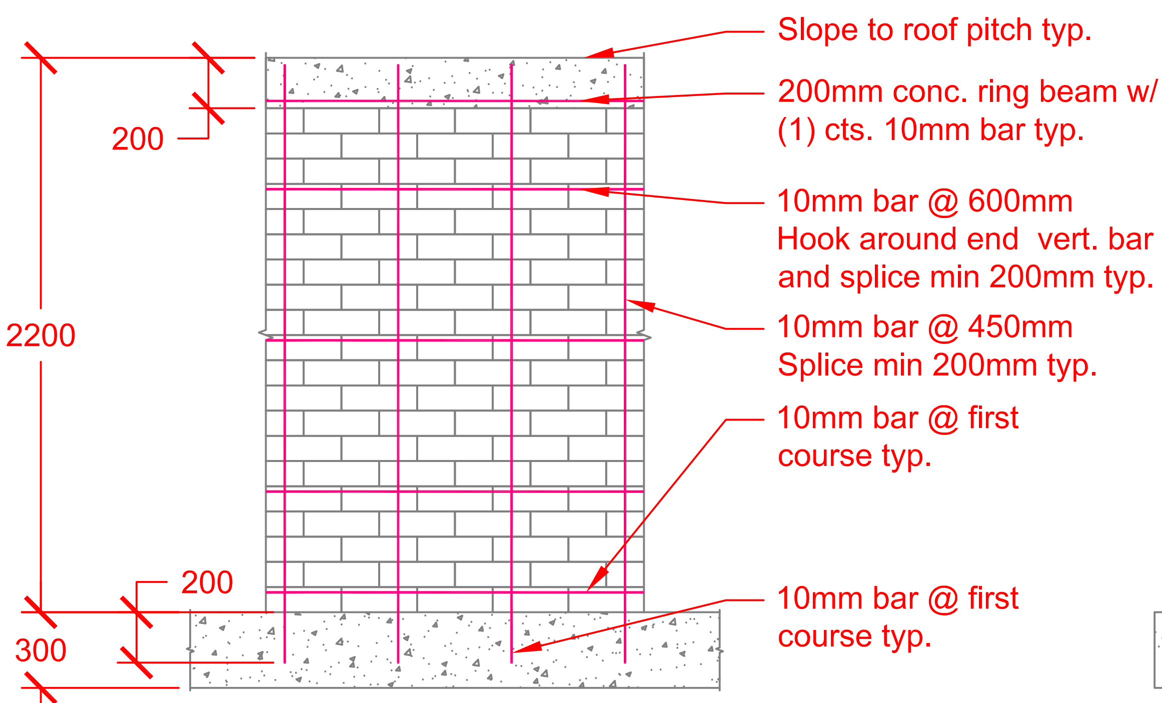


Section AA

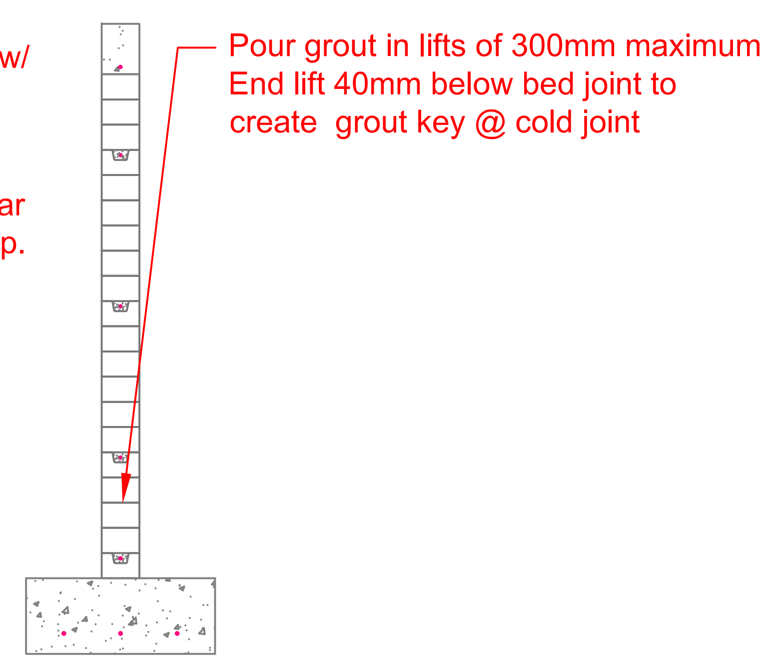


Wall End

Walls 8,C,E 1:30

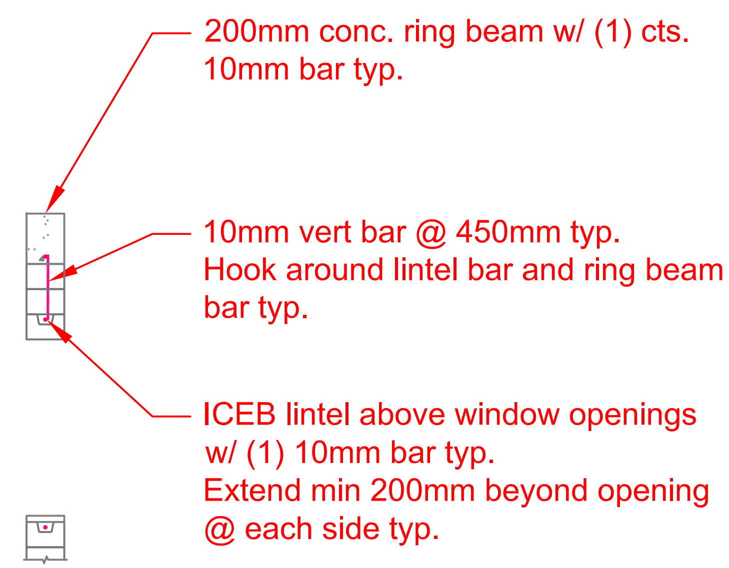


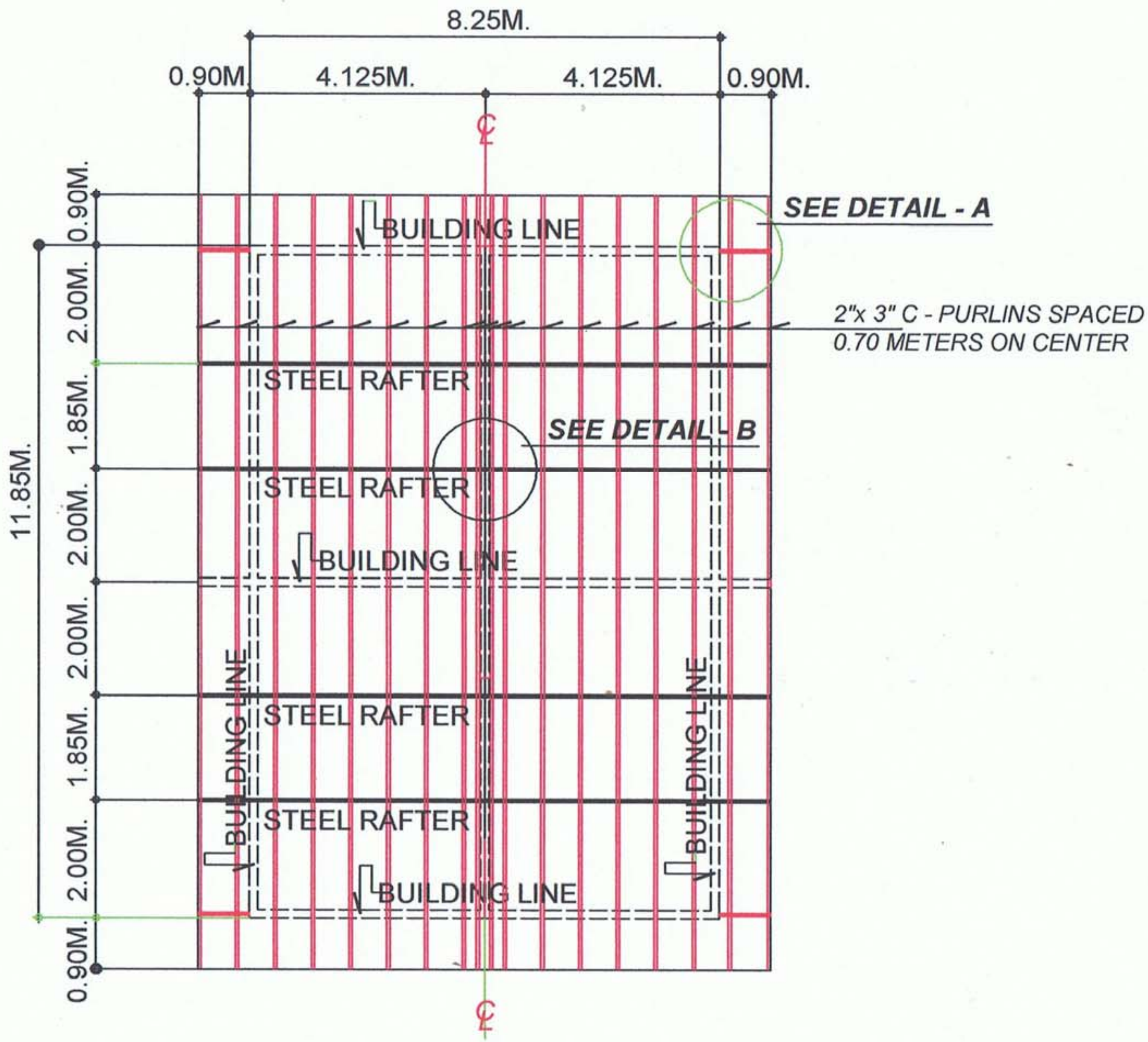
Section



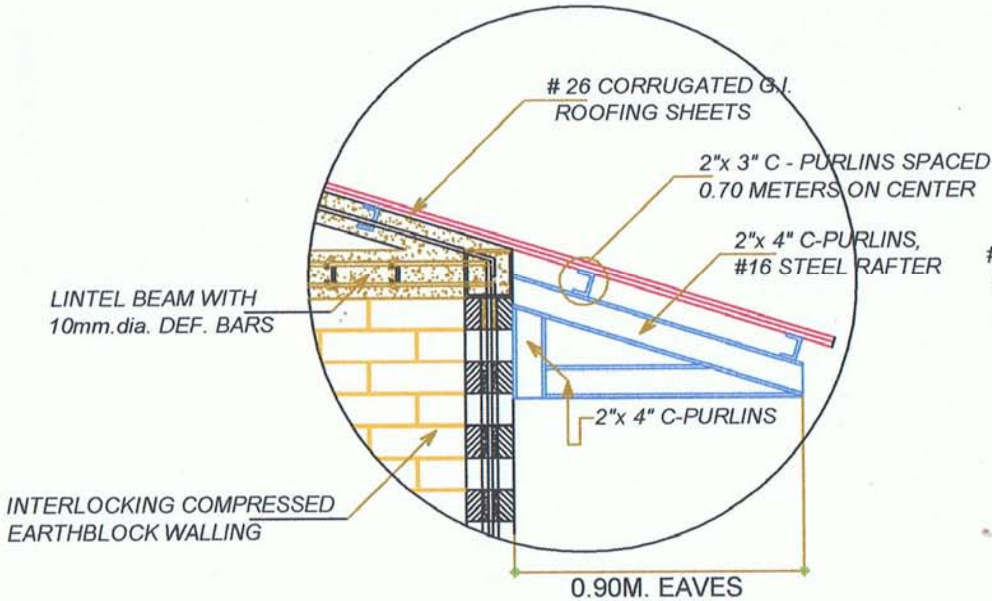
Wall End

Typical Window Opening Section 1:30

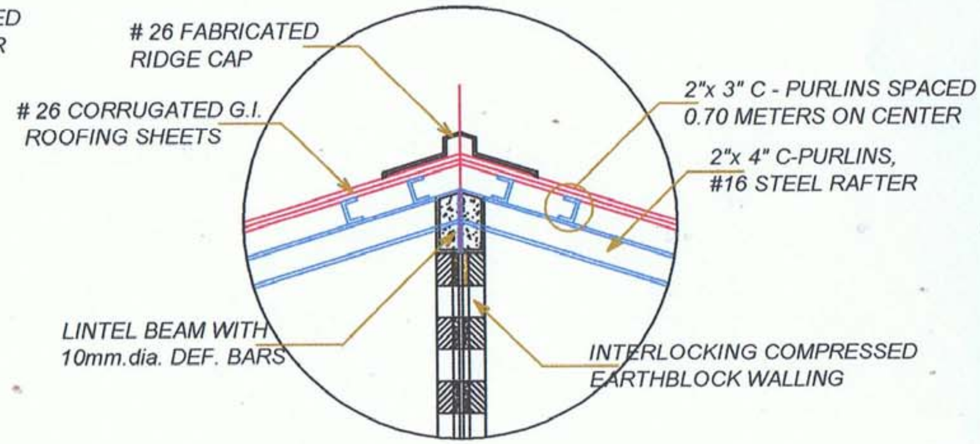




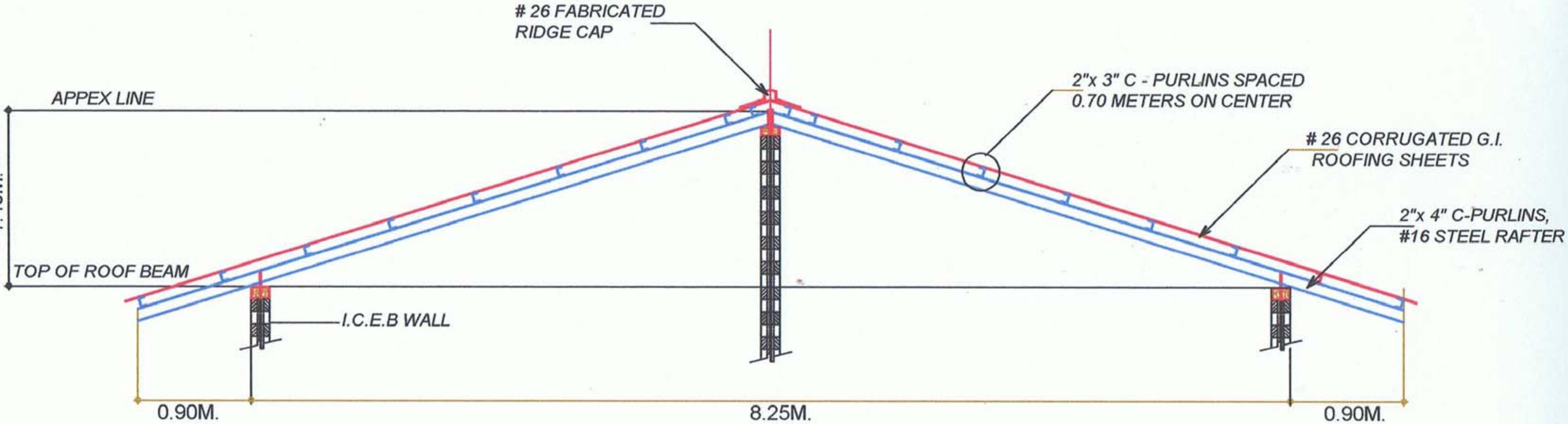
ROOF FRAMING PLAN



BLOW - UP DETAIL - A



BLOW - UP DETAIL - B



STEEL RAFTER DETAIL

Appendix D: Calculation of Minimum Vertical Steel Ratios based on MSJC 3.3.3.5.1

Source Code:

```
clc
clear
fy=30; % grade of steel in ksi
As=78.5; %cross section area of 1 bar in mm^2
alpha=1.5; %tension strain ductility factor (3 for intermediate walls, 
4 for special, 1.5 others)
Es=200000; % modulus of elasticity of steel in MPa
ey=fy/29000; %yield strain of steel
em=0.0025; %ultimate code compressive strain of masonry
dp=75; %distance in mm from comp fiber to comp steel
fm=3; %compressive stress of masonry in MPa
hmin=2000; %minimum height of wall in mm
hmax=4000; %maximum height of wall in mm
h=[hmin:100:hmax]';
if fy==30
    fy=206;
elseif fy==40
        fy=276;
elseif fy==60
    fy=414;
end

for i=1:size(h)
    d(i)={300:150:h(i)};
end
g=cell2mat(d(i));
largestd =size(g,2);
rho=zeros(largestd,size(h,1));
for i=1:size(d,2)
    dv=cell2mat(d(i));

    for j=1:size(dv,2)
        c(j)=dv(1,j)/((ey*alpha/em)+1); %1.5 factor on ey comes from 
3.3.3.5.1
        a(j)=.8*c(j);
        Cm(j)=0.8*fm*a(j)*150;
        eyp(j)=(c(j)-dp)*em/c(j);
        Ts=fy*As*((alpha*ey)/(em+alpha*ey))*((alpha*ey-
ey)/(alpha*ey)+.5/alpha);
        Cs=fy*As*(em/(em+alpha*ey))*((em-ey)/em+.5*(ey/em));
%         if eyp(j)>ey
%             Cs(j)=fy;
%         else
%             Cs(j)=eyp(j)*200000;
%         end
       P(j)=Cm(j)+Cs-Ts; % P is in Newtons
       %P(j)=200000;%h(i)*dv(1,j)*.001*2.62;
       M(j)=fy*As*(dv(1,j)-a(j)/2)+Cs*(a(j)/2-dp); % M is in N-mm
       Pbd=P(j)/(150*dv(1,j));
```

```
       rho(j,i)=(0.64*fm*(em/(em+alpha*ey))-P(j)/(150*dv(1,j)))/(fy-
min(em-(dp/dv(1,j))*(em+alpha*ey),ey)*Es); %see MSJC 2008 CC 3.3.3.5
    end
end

fid=fopen('rho','w+');
fprintf(fid,'Maximum Vertical Reinforcing Ratios Rho\n     ');
for i=1:size(rho,2)
    fprintf(fid,'%6.0f ',h(i));
end
 fprintf(fid,'\n');


for i=1:size(rho,1)
   fprintf(fid,'%4.0f ',dv(1,i));

    for j=1:size(rho,2)
        fprintf(fid,'%6.4f ',rho(i,j));
    end

fprintf(fid,'\n');

end
```

Maximum Vertical Steel Reinforcing Ratios ρ

Wall Heights (mm)

Wall Widths (mm)	2000	2100	2200	2300	2400	2500	2600	2700	2800	2900	3000	3100	3200	3300	3400	3500	3600	3700	3800	3900	4000
300	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938	0.0938
450	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625	0.0625
600	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469	0.0469
750	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375	0.0375
900	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313	0.0313
1050	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268	0.0268
1200	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235	0.0235
1350	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208	0.0208
1500	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188	0.0188
1650	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171	0.0171
1800	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156	0.0156
1950	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144	0.0144
2100	0	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134	0.0134
2250	0	0	0	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125	0.0125
2400	0	0	0	0	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117	0.0117
2550	0	0	0	0	0	0	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011
2700	0	0	0	0	0	0	0	0.0104	0.0104	0.0104	0.0104	0.0104	0.0104	0.0104	0.0104	0.0104	0.0104	0.0104	0.0104	0.0104	0.0104
2850	0	0	0	0	0	0	0	0	0	0.0099	0.0099	0.0099	0.0099	0.0099	0.0099	0.0099	0.0099	0.0099	0.0099	0.0099	0.0099
3000	0	0	0	0	0	0	0	0	0	0	0.0094	0.0094	0.0094	0.0094	0.0094	0.0094	0.0094	0.0094	0.0094	0.0094	0.0094
3150	0	0	0	0	0	0	0	0	0	0	0	0	0.0089	0.0089	0.0089	0.0089	0.0089	0.0089	0.0089	0.0089	0.0089
3300	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0085	0.0085	0.0085	0.0085	0.0085	0.0085	0.0085	0.0085
3450	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0082	0.0082	0.0082	0.0082	0.0082	0.0082
3600	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0078	0.0078	0.0078	0.0078	0.0078
3750	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0075	0.0075	0.0075
3900	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.0072	0.0072

Appendix E: Construction Estimate for a Habitat for Humanity ICEB Home in Udon Thani, Thailand

Estimate courtesy of Geoffrey Wheeler and the Center for Vocational Building Technology.

Construction Estimate

ลำดับ	Construction Materials		Price (Baht)	Number	Total
1	Ready mix concrete (180 ksc)	สีแดงผสม คอนกรีตหัก	1,450	3.5CuM	5,075
2	Stone Dust	หินฝุ่น	460	0.14CuM	66
3	Sand	ทราย	167	1.0CuM	167
4	Mixed cement	ปูนซีเมนต์ TPI green	95	14	1,330
5	Portland cement	ปูนซีเมนต์ CEMEX แดง	116	4	464
6	plastic anti-capillary sheet	ผ้าพลาสติกสีดำ กันน้ำ หนา .30 กวาง 6 ม	254	1	254
	Cement and Aggregate				7,102
7	Building Block	ก้อนก่อ	8.57	1528	13,095
8	Corner Block	ก้อนมุม	8.61	198	1,705
9	Pillar Block	ก้อนเสา	8.64	144	1,244
10	Channel Block	ก้อนเอ็น	8.56	159	1,361
11	Channel Corner Block	ก้อนเอ็นมุม	8.63	8	69
12	Half Block	ครึ่งก้อนก่อ	5.99	137	821
13	Half Corner Block	ครึ่งก้อนมุม	6.03	12	72
14	Half Channel Block	ครึ่งก้อนเอ็น	6.07	3	18
15	Half Channel Corner Block	ครึ่งก้อนเอ็นมุม	6.1	0	-
16	shipping vehicle	ขนส่ง รถ			650
17	shipping labor	ขนส่ง แรงงาน			554
	Blocks				18,385

ลำดับ	Construction Materials		Price (Baht)	Number	Total
18	Bamboo	ไม้ไผ	-	20 lengths	-
19	9 mm rebar	เหล็กเส้นกลม 9 ม.ม..	90	19 lengths	1,665
20	tying wire	ลวดมัด ลวดดำ 3.7	110	1kg	110
21	C channel	เหล็ก "C" 1.5" x 3" x 2.3 mm (rafter)	290	26 lengths	7,540
22	box section	เหล็ก กล้อง 1.5" x 3" x 2.3 mm	290	5 lengths	1,450
23	angle iron	เหล็กฉาก 1", หนา 3/16"	230	32 lengths	7,360
24	rust inhibiting paint	สีกันสนิม	150	3 gallon	450
25	turpentine	น้ำมันสน	95	3 gallon	285
26	welding rod	ลวดเชื่อม	95	1.5 box	143
27	Galvanized Wire #16	ลวดชุปสังกะสี เบอ 16	40	1kg	51
28	Nails, 2"	ตะปู 2"	35	0.5	18
29	Nails, 1.5"	ตะปู 1.5"	35	0.5	18
30	abraisive cutting wheel	ไฟเบอร์	35	2	70
	Steel				19,071

ลำดับ	Construction Materials		Price (Baht)	Number	Total
31	concrete roof tiles	กระเบื้องคอนกรีตมุงหลังคา ศิเพชร 1 หู	5.32	704	-
32		กระเบื้องคอนกรีตมุงหลังคา ศิเพชร 2 หู	5.58	256	-
33		กระเบื้องคอนกรีตมุงหลังคา ศิชัย 2 หู	16.22	160	2,595
34		กระเบื้องคอนกรีตมุงหลังคา ศิชัย 4 หู	17.81	128	2,280
35		กระเบื้องคอนกรีตมุงหลังคา ปีกศิลาเพชร	2.41	80	-
36		กระเบื้องคอนกรีตมุงหลังคา ปีกศิลาชัย ซาย	2.66	32	85
37		กระเบื้องคอนกรีตมุงหลังคา ปีกศิลาเพชร คว	2.66	32	85
38		กระเบื้องคอนกรีตมุงหลังคา ส ครอบยาว	6.4	16	102
39		กระเบื้องคอนกรีตมุงหลังคา ส ครอบสัน	4.49	15	67
40		กระเบื้องคอนกรีตมุงหลังคา ส ครอบปิดมุม	14.13	2	28
41	shipping vehicle	ขนส่ง รถ	750	1	750
42	shipping labor	ขนส่ง แรงลง	1	338	169
43	smooth fibre cement board	Smart Board 60 x 240 cm x 3.5mm	77	24pcs	1,848
	Roof				8,010

ลำดับ	Construction Materials		Price (Baht)	Number	Total
44	Door Jamb	วงกบประตู "80*200"	500	2pcs	1,000
45	Door Jamb	ประตูไม้รวม "80*200"	700	2pcs	1,400
46	Door Knob	ลูกปิดประตู	80	2pcs	160
47	PVC Door	ประตู พีวีซี 70 x 180 ครบชุด	585	1pcs	585
48	Dead bolt	กลอนประตูห้องน้ำ	15	1pcs	15
49	Hinges	บานพับ	34	9pcs	306
50	Window Frame	วงกบหน้าต่าง 90 x 120	350	5pcs	1,750
51	Venitian Window Set	ชุดบานเกร็ดพร้อมอุปกรณ์ (ขาวมัว) 90 x 120	490	5pcs	2,450
52	Screws	สกรูติดตั้งวงกบประตู/หน้าตาง	0.25	100pcs	25
53	Plastic Inserts	ฝุก	0.25	100pcs	25
	Doors & Windows				7,716

ลำดับ	Construction Materials		Price (Baht)	Number	Total
54	Toilet Bowl	ส้วมนั่งยอง white	180	1pcs	180
55	Septic Ring	ท่อบ่อ 80*40	80	6pcs	480
56	Cover	ฝาปิด พีวีซี	90	1pcs	90
57	Cover	ฝารูเล็ก 8	90	1pcs	90
58	PVC Pipe	ท่อ PVC blue 4" x 100 cm	80	4 lengths	320
59		ขอท่อ PVC 4" 3-ทาง	60	1pcs	60
60		ท่อ PVC blue 2" (4 m)	111	1 lengths	111
61		ขอท่อ PVC 2" 3-ทาง	15	1pcs	15
62		ขอท่อ PVC 4"-2" 3-ทาง	60	1pcs	60
63		ของ่อ PVC 2"	10	1pcs	10
64		รางผึง	10	1pcs	10
	Septic	สว่ม			1,426
65		ท่อ PVC blue 1/2"	26	2 lengths	52
66		ขอท่อ PVC 1/2" 3-ทาง	7	1pcs	7
67		ของ่อ PVC 1/2"	5	4pcs	20
68		ขอเกลียวตรง 1/2"	10	2pcs	20
69		ก๊อกน้ำ 1/2	25	2pcs	50
70		กาว PVC	30	1 cans	30
	Water Piping	ประปา			99

ลำดับ	Construction Materials		Price (Baht)	Number	Total
71		ชุดลอดไฟ 20 วัต	80	5	400
72		สายไฟ 2 x 1.5	7	20	140
73		สายไฟ 2 x 2.5	8	10	80
74		กล้องฝัง	9	1	9
75		หน้ากาก 1 ซอง	9	1	9
76		หน้ากาก 2 ซอง	9	2	18
77		หน้ากาก 3 ซอง	9	1	9
78		สวิทช์ National (Matsushitu)	9	5	45
79		ปลัก National (Matsushitu)	9	3	27
80		Cut Out 30 Amp	9	1	9
	Electrical	ไฟฟ้า			540
	Material				64,181

ลำดับ	Construction Materials		Price (Baht)	Number	Total
81	oof Weldin	ช่างเหล็กสร้างโครงหลังคาและมุง			4,500
82	Concrete labor	ช่างเทพื้น 3ค	600	2	1,200
83	Block labor	ช่างก่อิฐ	200	3	600
84	Plaster labor	ช่างฉาบห้องน้ำ	200	1	200
85	Window & smooth board labor	ช่างติดตั้งกระจก smart board อุดรองประตุ	200	1	200
	Total Skilled Labor				6,700
	Material + Labor	รวมวัสดุและแรงงาน			70,881

ลำดับ	Construction Materials		Price (Baht)	Number	Total
	Admin	บริหาร จากค่าก่อสร้างรวม		10%	7,088
	total	รวม			77,969
	Down payment				16,000
	Loan	เงินกู			61,969
	Inflation	เงินเฟ้อ จากส่วนกู ต่อปี		4.50%	8946.42
	Total House Cost	รวมค่าบ้าน			86,915
					$2,897.17