PROJECT REPORT
FOR
VIRUNGA COFFEE.COCOA CO-OP
NORTHERN PROVINCE, RWANDA

01 JULY 2018

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NOTE: NOT FOR CONSTRUCTION. TO BE REVIEWED AND APPROVED BY IN-COUNTRY ENGINEER.
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1.0 Abstract

Through the help of Journeyman International, local cooperatives, and the Empowering Villages Organization, a large part of Rwanda is being redesigned and rebuilt. Changes hope to enable access to electricity, and will empower communities with leading socio-economic development solutions in energy, environment, entrepreneurship, and education. These efforts have led to the vision of the Virunga Coffee/Cocoa Co-op along the Mukungwa River in the Northern Province of Rwanda. This project consists of a team of two Architectural Engineering students, an Architecture student, and a Construction Management student from California Polytechnic State University in San Luis Obispo, CA, working with a local representative in Rwanda. The purpose of this project is to create a community center and warehouse for the advancement of coffee and cocoa farming that locals can access for both learning and economic empowerment in order to increase education standards and employment in the community. This project report consists of a personal reflection of travel experiences in Rwanda, preliminary research of the history of Rwanda, drawings and calculations of the structure, challenges faced during the project, and reflections of personal encounters throughout the project.
2.0 Introduction

2.1 What is Journeyman International/Brief Overview

Journeyman International is a nonprofit humanitarian design organization that pairs university students and future designers with organizations and countries that are in need of good architecture and design. Students studying architecture, engineering, construction management and/or landscape architecture are given the opportunity to work on these projects to fulfill their senior thesis requirement. Journeyman International strives to inspire and excite students by doing humanitarian work all over the world. In fact, many of these Journeyman International projects are found outside of the country, thus giving students a chance to travel abroad and get first hand experience of the culture, land, and people. Through its efforts, Journeyman International aims to change the lives of both the students and the people of these communities.

2.2 Situation/Background

Rwanda is on the forefront for building locally and sustainably in developing countries. This can be seen through recent Journeyman International projects such as the Women’s Aquaculture Cooperative and the Sunzu Village. Increased education has led to advancements in construction and engineering that promote environmentally conscious strategies to solve problems in the country. Rwaza, Musanze, in the Northwest province, faces problems concerning energy and socioeconomic growth. The Virunga Coffee-Cocoa Co-Op is working in conjunction with the Empowering Villages organization and DC Hydropower to create more energy and jobs.
in the area and to continue Rwanda’s mission on sustainability. Empowering Villages is a nonprofit organization working with the Global Giving Organization, which is the largest global crowdfunding community connecting nonprofits, donors, and companies in nearly every country. Materials for the community center building are found directly at the site and use building practices as old as time, such as rammed earth walls.

Empowering Villages’ ultimate mission is to enable access to electricity and empower communities with leading socio-economic development solutions in energy, environment, entrepreneurship, and education. Specifically, Empowering Villages is also engaged with the Rwaza Hydropower Plant to fulfill its socio-economic mission to better the lives of the surrounding community. This joint effort has led to the creation of a project that will provide a community center/learning space and a warehouse next to the hydropower plant on the Mukungwa River in the North West of Rwanda. This report will focus specifically on the design of the community center. The design team for this project includes architecture student Dayna Lake, construction management student Tanner Frkovich, and two architectural engineering students Anugrah Gupta and Caleb Azevedo. All four members are graduating seniors at California Polytechnic State University in San Luis Obispo, working to
create more job opportunities and education for the community, and to perpetuate coffee farming in Rwanda.

**Project Site Map - Virunga Coffee/Cocoa Co-op**

### 2.3 Story of Travel

Anugrah is grateful for the opportunity to travel to Rwanda in December 2017. He was able to see the current state of his project site, as well as meet and interact with many local Rwandans who are the main motivation for Anugrah’s dedication to humanitarian work.

In addition to meeting influential people and seeing his project site, Anugrah had many other heartwarming experiences. In 1994, Rwanda went through a horrible genocide, which resulted in the death of over one million people. While in Kigali, Anugrah visited the Rwandan
Genocide Memorial, where some 250,000 people are interred. Having this experience at the beginning of the trip was a major factor in the emotional attachment that Anugrah was to have with the people of Rwanda. In addition to visiting the memorial, Anugrah noticed a great sense of pride among the people of Rwanda; their focus was more to overcome their losses rather than grieve them. Furthermore, given the 1994 tragedy, the people were surprisingly welcoming to tourists and other outsiders. This further inspired Anugrah to dedicate his time and passion to this project.

Of the many villages visited, Sunzu Village in Northern Province, Rwanda was the most memorable for Anugrah. Sunzu is the site of a previously designed (and constructed) Journeyman International project, as well as the current construction site for another Journeyman project. The completed project is a multi-use building designed to provide a place for children and women to meet and learn, in an effort to promote education in one of the poorest regions of Rwanda. When Anugrah arrived, children were either playing outside, in the library reading stories aloud in English, or learning how to use various computer software. The faces of these kids lit up with immense joy as Anugrah and the rest of the Journeyman team arrived. Many pictures were taken, games played,
stories read, and memories made. To this day, Anugrah is pen pals with a thirteen-year-old student at Sunzu named Didier Niyomukiza.

Each and every day in Rwanda was filled with unforgettable adventures whether they were visits to beautiful and vast Lake Kivu, or getting chased by elephants in Rwanda’s Akagera National Park. Anugrah feels a great sense of appreciation for the culture and people of Rwanda and is extremely passionate about helping the country by means of yet another Journeyman International project.

2.4 Preliminary Research

Although Caleb did not travel to Rwanda with others in December of 2017, he still performed preliminary research of the history of Rwanda and also watched the movie “Hotel Rwanda” as recommended by Journeyman International president Daniel Wiens. Rwanda is a
country located in the northern province of Africa with a population of approximately 11.2 million people. The capital is Kigali, which can be found in the center of the country.

Prior to 1994, two groups represented much of the country: the Tutsis and the Hutus. Both groups had similar physical characteristics and religious beliefs. After World War I, Rwanda came under the League of Nations mandates of Belgium, and during the Belgian colonization the Belgians favored the Tutsi minority over the Hutus (Rwandan Genocide, History.com). The Tutsis were perceived to have greater wealth and were granted higher social status than the Hutus by the Belgians (Lichfield, Independent). This created ethnic strife as the Hutus believed that the Tutsis were privileged and could not be trusted. Rebellious Hutus revolted, forcing many Tutsis to leave the country and declaring the country as a republic. Violence continued throughout the years. The military then declared a moderate Hutu, Juvenal Habyarimana, in power of Rwanda. He then became the president of the political party, the National Revolutionary Movement for Development (MRND). In 1990, negotiations between the Rwandese Patriotic Front (consisting mainly of Tutsis) and Habyarimana led to a reconstruction of the Rwandan government that would include the Tutsis which angered the Hutu extremists and caused tensions to rise even higher (Rwandan Genocide).

On April 6, 1994, Rwandan President Juvenal Habyarimana and Burundi President Cyprien Ntaryamira were murdered as their plane was shot down over Kigali (Rwandan Genocide). This culminated in the Rwandan genocide as Hutu militia known as the Interhamwe and Rwandan armed forces began murdering innocent Tutsis and moderate Hutus. Approximately 800,000 Tutsis and moderate Hutus were killed by these forces in a span of 100 days (Rwanda Country Profile, BBC News).
During this time, the international community stayed on the sidelines; the only international assistance the country received came from the French troops who provided safe zones for the Tutsis and helped some escape. The people of Rwanda were helpless, because the United Nations failed to intervene as they saw Rwanda as less important than other countries. Innocent lives were unrightfully taken; the United Nations could have helped more than they did. Though portrayed by actors, “Hotel Rwanda” showed the emotions and the challenges the country faced.

Today, Rwanda still remembers those whose lives had been taken, but also looks to the future of the country. A farmer in Rwanda, Ezekial Shinga, states “Everything in this country has changed. People own businesses, and the majority are tea farmers. There’s peace, and neighbors now love each other (Onnyulo, The Washington Times).” Tea and coffee have been the main reasons for the recent economic growth, and this growth has helped provide currency to the country for new construction of schools, hotels, and other infrastructure (Rwanda Country Profile). The goal is to continue the economic growth and strive for better educational opportunities for the future of Rwanda. Caleb and the rest of the Journeyman International team look to fulfill this goal through the efforts of the Virunga Coffee/Cocoa Co-op.
3.0 Project Description

3.1 Coordination Approach/Assumptions

Coordination throughout the course of this project was very crucial, as there are many limitations when designing in Rwanda as opposed to the U.S. In addition, a lot of information was unavailable to the team that was project-specific. For example, there were no soil reports available for the exact location of the proposed building. Therefore, the engineering team used soil reports from previous projects within 50 km of the Virunga site. With the use of these reports, the team classified the soil and extracted bearing pressures and other coefficients necessary for the design of the foundation.

All communications for this project were conducted between the architecture student, Dayna, and the on-site representatives. The on-site representatives included Carly Althoff and Daniel Klinck, both of whom are permanently situated in Rwanda. To keep communication precise and limited, the engineering team, Anugrah and Caleb, gathered information they needed from the representatives and relayed it to Dayna. As a result, all proposed questions (including those from Dayna) were asked in a single phone call/email approximately once every two weeks, or as otherwise needed.

Another major aspect of the design that needed to be approved by and communicated with the on-site representatives was material use. The initial phase of the project consisted of communication amongst Dayna and the representatives in coordinating what building materials were available for the use of the design. The second phase involved the engineering team communicating ultimately with the representatives in determining material strengths and
properties. The J.I. team concluded that many of the material properties for this project would be similar to previous projects. However, one difference was the compressive strength of rammed earth walls: 1,500 psi as opposed to a more commonly used strength of 4-5,000 psi for concrete. Although some information was found on the internet, the team knew it was more accurate to obtain this information first hand. Therefore, Anugrah was able to incorporate in the design materials that he saw when he traveled to Rwanda. The project therefore consists mainly of the following materials: corrugated metal roofing, timber roof joists, steel roof beams and columns, reinforced rammed earth walls, and reinforced concrete foundations.

Preliminary Materials List

3.2 Drawings and Calculations

The calculations can be found in Appendix A, and the drawings in Appendix B. For the first section of this calculation package, the engineering team determined an overall framing layout and performed a roof load take-off for the structure. In addition, they designed the slab-on-grade, roof joists, and beams. As reflected in the calculations, there are many materials
being used simultaneously. This initially created conflict with member sizes and connections of members in terms of constructability. In addition, loads and member spans governed the design, making the layout increasingly inefficient. As a result, the engineering team used a roofing system including dimensional lumber joists in conjunction with steel wide-flange beams, as opposed to exclusively timber framing. Preliminary designs were also taken into account in order to determine seismic weight and wall layout.

The slab-on-grade consists of a typical design that is commonly used for 1-2 story buildings in California. Due to minimal design loads, this design is applicable. The joists were designed using dimensional lumber, specifically Douglas Fir-Larch No. 3 grade. Regarding available materials in Rwanda, the team felt this grade was the best comparison. A distributed load of 68 pounds per linear foot (plf) spans across the entire joist. Temperature and moisture effects were negligible, and therefore factors were accordingly set to 1.0. The joist design includes checks of bending, shear, and deflection. The final design of the joists is 2x10s at 24 in. on-center.

The roof beams were designed using A992 steel, with a yield stress of 50 ksi and an ultimate stress of 65 ksi. Due to large spans, steel is a more efficient material than timber for the roof beams. A uniform distributed load of 488 plf acts along the entire span of both designed beams. Bending, shear, and deflection were checked for each beam. Beam B1 is designed to be a W18x40 and Beam B2 is designed to be a W14x22.

The second section of this calculation page includes the design of wide flange columns and column foundations. Much of the gravity load is transferred to the rammed earth walls through bearing except for the south-most section of the building. Dayna wanted to have rammed
earth walls that would disconnect from the roof to provide for openings and natural light throughout the community center. In response, the engineering team added columns along that grid-line to support the W14x22 roof beams.

The column selected for design was the “worst-case” column (the tallest column) which has an unbraced length of 17 feet. The column was also designed using A992 steel. The loads used for this design were a dead load of 20 pounds per square foot (psf) and live load of 14 psf. The live load could not be reduced because the tributary area of the column was lower than the minimum, as was the slope of the roof. The tributary area affecting the column was 10 feet by 16 feet, or 160 square feet. The demand gravity load using the tributary area and load combination for LRFD was 7.84 kips.

The capacity values for the column design came from the AISC 14th Edition Steel Manual. “Pin-pin” connections were assumed as the base for the column, which resulted in a K value of 1.0. The column was designed to be a W10x33 with an axial load capacity of 195 kips.

The column footing was designed with reference to the ACI 318-14 code. The footing was designed using concrete and steel reinforcing; the compressive strength of the concrete was 3,000 psi and the yield strength of the reinforcing was 60 ksi. The engineering team was not able to find sufficient soil information for the site, but used previous projects’ soil reports; therefore, the assumed soil classification used for this design was expansive clay soil with an allowable soil pressure of 1.5 kips per square foot (ksf). This pressure is also the worst-case scenario per the International Building Code (IBC) 2015. Under service loads, the area required for this footing was 4 ft². For conservative purposes, the team designed a 4’x4’ square footing with an area of 16
ft². The depth of the footing is 2 feet which was acceptable for two-way action shear and wide-beam action shear.

The footing reinforcement design was similar to that of a concrete slab. The demand moment from the soil pressure on the footing was 2.25 kip-ft. The required area of the steel was .03 in²; therefore, the footing required minimum reinforcement per ACI standards which was 2.07 in². The design team tried (5) #6 reinforcing bars with a total area of 2.20 in². Spacing requirements were checked for both shrinkage and temperature reinforcement, and flexural reinforcement under ACI standards. The provided spacing of the footing reinforcement is approximately 9.5 inches. To ensure ductile behavior, steel should yield before concrete. This assumption was checked as a part of this calculation as the steel strain was 0.076 in/in which is greater than the yielding steel strain of 0.0021 in/in. Using the ACI 318-14, the moment capacity of the column footing was 195 kip-ft. To assume adequate development length within the concrete footing, 90-degree standard hooks shall be used.

The third section of this calculation package includes the lateral design criteria and design of two specific shear walls and their respective foundations. With the final architectural designs of the walls completed by Dayna, the engineering team computed the seismic weight of the building which included the roof dead load and shear wall self-weights. The engineering team used half of each shear wall self weight because the other half would transfer directly to the ground. Seismic criteria is documented in the load take-off section of the design package. The engineering team concluded that the seismic base shear of the building is 14 kips in both directions.
The engineering team verified that wind loading did not govern the lateral design. Using the ASCE 7-10 code, they performed a wind analysis to find the maximum pressures on the building. The engineering team used the lowest wind pressures in the United States (California, at 110 mph) due to inadequate data regarding wind pressures in the project’s region; previous J.I. teams have used the same pressures. After all of the parameters were checked, the engineering team concluded that the base shear for wind is 12 kips in the North-South direction and 9 kips in the East-West direction. As a result, seismic loading governs in the lateral design at 14 kips.

Using the seismic lateral loads, the engineering team designed the shear walls. Based on the layout of the building, they designed one shear wall in each direction of loading (East-West and North-South). The shear walls will be constructed of rammed earth, and therefore a conservative estimate for the design strength of this material was used. As a result of conducting research and looking at past projects, a lower bound compressive strength of 1,500 psi was used and the walls were designed using the same process as reinforced concrete walls.

The shear wall designs include sizing and spacing of shear reinforcement, flexural reinforcement, and boundary element confinement. Due to extremely low seismic loads in comparison with California, the shear walls were designed using minimum code-enforced reinforcing. In addition, no special boundary reinforcement was required, as the lateral loads were relatively small in the East-West and North-South directions (3 kips and 7 kips, respectively). In order to eliminate the iterative process of designing flexural reinforcement, *spColumn* was used to check the flexural capacities of the wall sections.

After the design of the shear walls, the engineering team designed their respective foundations. Again, in order to eliminate the iterative process of designing reinforcement, the
team used a self-created excel spreadsheet that expedited the process. The spreadsheet roughly goes through the design process and makes intermediate checks along the way which helps the designer understand the general foundation dimensions. The design can then be started using these dimensions, and the need for iteration is eliminated.

The shear wall foundation designs consist of soil bearing pressure checks, flexural reinforcement design, and transverse reinforcement design. Due to past experience with shear wall foundation designs, the team did not check/design shear reinforcement; shear reinforcement is extremely uncommon in shear wall foundations unless loads are unusually large. Therefore, the relatively small seismic loads allowed for the use of minimum reinforcing in both directions of the shear wall foundations.

For the soil bearing pressure checks, two load cases were considered: one with solely dead and live loads, and another including seismic effects in addition to dead and live loads. These load combinations were used in conjunction with an Allowable Stress Design (ASD) approach. The reinforcement, however, was designed using Load and Resistance Factor Design (LRFD).

### 3.3 Challenges

One of the main challenges the engineering team faced was coordination with Dayna. Initially, the team wasn’t given much information from her due to the lack of general knowledge on the location. In their lab courses, the engineering team was accustomed to general project information such as materials and site information (seismic, wind, and soil criteria). The
beginning stages of the project had many changes in material use and footprint, which prevented the engineering team from conducting accurate load take-offs and preliminary calculations for the structure.

Dayna and engineering team had trouble deciding what materials to use for the project; for example, Dayna wanted to design the roof framing with timber for aesthetic appeal, while the engineering team believed that steel framing would be structurally efficient. The project initially consisted of a range of materials that the engineering team had not encountered being used simultaneously. Although the design could have been carried out using these materials, the engineering team and Dayna discussed the inefficiencies involved. As a result, the materials were re-chosen to solve the structural issues while still allowing for aesthetic freedom. A draft design was created using plywood sheathing, timber framing, rammed earth walls, and steel columns. However, during the calculation phase, the engineering team determined that timber framing would be difficult to use due to large spans. Therefore, steel wide flange beams are used in the gravity system. This change allowed Dayna to have more vertical clearance with smaller beam depths, while also reducing complications in the framing system for the engineering team.

As stated earlier, there were little to no reports available regarding site conditions of Rwanda. In order to overcome this challenge, the engineering team used similar U.S. wind conditions for this project. The seismic criteria, on the other hand, was taken from a report analyzing seismic loads in Bujumbura and Kigali (Ndihokubwayo, Jiang, and Chen, 2). The site design criteria taken were conservative in comparison with the actual seismic and wind conditions in Rwanda. The soil criteria were taken from minimum requirements in the 2015 edition of the IBC.
Another challenge was the unfamiliarity with the design of connections between members of varying materials. Additionally, the general procedure of designing rammed earth shear walls was unknown. To overcome these challenges, research was conducted which included reading a 2003 article on the construction of rammed earth walls (Maniatidis & Walker, 2003). Daniel Wiens also recommended to the engineering team using the same design approach as with concrete shear walls; the only difference being a reduced compressive strength of 1,500 psi.

### 3.4 Status/Future of Project

The architectural, structural, and construction packages will be submitted and reviewed by Journeyman International, and the calculations will be thoroughly reviewed by (an) in-country engineer(s). Land has already been allocated for the project and construction will hopefully begin after fundraising is complete. In upcoming months, the Journeyman International team, especially those who did not have the opportunity to travel in December of 2017, will hopefully travel back to Rwanda to gain further understanding on the current status of the project.
4.0 Conclusion

4.1 Personal Reflection - Anugrah Gupta

After ten weeks of working on the design of Virunga Coffee/Cocoa Co-op, I can proudly say that I was able to incorporate much of what I have learned at Cal Poly over the years. With the completion of three structural design labs and the combination of various materials in the building concept, I was well-trained to complete the design. Many challenges arose along the way, but I was able to overcome these challenges using the help of my teammates, as well as my engineering judgement. The Virunga Coffee/Cocoa Co-op was a very interesting building to work on due to the large range of materials used, especially because some were new to me (i.e. rammed earth). However, this served solely as additional motivation, and it gave me the opportunity to learn and incorporate new ideas into the project.

For me, this project was more than a graduation requirement, but rather a comprehensive examination of what I have been taught over the last four years. Most importantly, this was an incredible opportunity and privilege for me, as the community center could possibly be constructed one day. Additionally, the opportunity I was given to travel to Rwanda will remain with me for a lifetime. Not only was I able to gain firsthand knowledge of my site, but was also able to get a great sense of appreciation for the culture and people of Rwanda. Journeyman International is an incredible organization that I hope to continue working with after graduation; the opportunities are endless, and there is no other organization that gives students the confidence to design their own structure with the possibility of construction.
4.2 Personal Reflection - Caleb Azevedo

Working with Journeyman International was a great opportunity for me because it introduced me to what working on actual structural projects feels like. From scheduling weekly meetings to coordinating and adapting to changes in design, I understand that when it comes to working on an actual project, it is more than calculations and drawings. I encountered challenges with new materials such as rammed earth and also with designing connections between different materials that I was not accustomed to. These challenges were difficult, but with the help of research, assistance from professors, and consultation with my engineering partner Anugrah, the design challenges were resolved.

I realize that not travelling to Rwanda this past winter made it difficult to understand what this project actually means. My preliminary research taught me about the unfortunate events that occurred in Rwanda. With no assistance from any other nations, Rwanda was helpless. But through all the pain and suffering in 1994, Rwanda has grown as a community today. I am proud to be a part of this process. With coffee being one of the primary sources for economy in Rwanda, the Virunga Coffee/Cocoa Co-op project will increase jobs and economy.

I feel that this senior project has prepared me for the future. I will be starting work at Strandberg Engineering and am excited to utilize everything that I have learned throughout my four years at Cal Poly. I will also still be interested in working with Journeyman International after graduation because it allows me to combine design with humanitarian work that will benefit those that need it. I hope that I will be able to go visit the Rwandan project site and continue to be a part of the design of the Virunga Coffee/Cocoa Co-op.
Appendix A - Calculations
STRUCTURAL CALCULATIONS

FOR

VIRUNGA COFFEE CO-OP
NORTHERN PROVINCE, RWANDA

01 JULY 2018

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  SLAB ON GRADE N/S SLIDING CHECK ..................................... L26
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PROJECT DESCRIPTION:

THE PROJECT CONSISTS OF A SINGLE STORY, MULTI-MATERIAL COMMUNITY CENTER FOR COFFEE AND COCOA FARMING LOCATED IN THE NORTHERN PROVINCE OF RWANDA, AFRICA. THE ROOF IS MADE UP OF STAND SEAM METAL ROOFING.

THE GRAVITY SYSTEM CONSISTS OF DIMENSIONAL LUMBER JOISTS THAT REST ON STEEL WIDE FLANGE BEAMS. THE FRAMING SITS ON RAMMED EARTH BEARING WALLS AND STEEL WIDE FLANGE COLUMNS. THE COLUMN FOUNDATIONS ARE REINFORCED CONCRETE SPREAD FOUNDATIONS. THE INTERIOR OF THE BUILDING IS SLAB-ON-GRADE.

THE LATERAL SYSTEM CONSISTS OF 3/8” PLYWOOD SHEATHING SPANNING TO RAMMED EARTH SHEAR WALLS IN BOTH DIRECTIONS. THE SHEAR WALLS ARE SUPPORTED BY REINFORCED CONCRETE SPREAD FOUNDATIONS.

DESIGN CRITERIA:

DESIGN CODE: 2015 IBC
ASCE 7-10
ACI 318-14
AISC STEEL MANUAL 14TH EDITION
NDS/SDPWS 2015

BUILDING TYPE: TYPE II - OCCUPANCY CATEGORY B

WIND CRITERIA*: 110 MPH, EXPOSURE B

SEISMIC CRITERIA**: SDS = 0.190g
R = 5.0 (ORDINARY CONCRETE SHEAR WALLS)
SEISMIC DESIGN CATEGORY B
IMPORTANCE FACTOR (Ie) = 1.0

SOIL CRITERIA***: ASSUMED EXPANSIVE CLAY SOIL
SITE CLASS D ASSUMED
ALL EXCAVATIONS, FILLING, BACKFILLING, AND SOIL COMPACTION PER GEOTECHNICAL REPORT: NOT AVAILABLE
ALLOWABLE PASSIVE SOIL PRESSURE: 100 PSF/FT
ALLOWABLE VERTICAL FOUNDATION PRESSURE: 1500 PSF
COEFFICIENT OF FRICTION: 0.3

*WIND CRITERIA BASED ON WORST CASE UNITED STATES CONDITIONS PER ASCE 7-10. SEE APPENDIX C.

**SEISMIC CRITERIA BASED ON SCHOLARLY ARTICLE. SEE APPENDIX C.

***NO SOILS REPORT GIVEN. WORST CASE CONDITION USED PER IBC SECTION 1602.4. SEE APPENDIX C.
LOAD TAKE-OFF

**GRAVITY**

<table>
<thead>
<tr>
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<tr>
<td>Weatherproofing</td>
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<tr>
<td>3/8&quot; Plywood Sheathing</td>
<td>2.0</td>
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<tr>
<td>Misc.</td>
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<tr>
<td><strong>Total Gravity</strong></td>
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**Framing**

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**SEISMIC WEIGHT**

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<tr>
<td>Total Shear Wall</td>
<td>284.0</td>
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<tr>
<td><strong>Total</strong></td>
<td><strong>362.0</strong></td>
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</tbody>
</table>

**LIVE LOAD**

<table>
<thead>
<tr>
<th>Item</th>
<th>Load (PSF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Live Load (Reducible)</td>
<td><strong>20.0</strong></td>
</tr>
</tbody>
</table>
SEISMIC WEIGHT KEY PLAN
# Seismic Weight

Refer to T3

<table>
<thead>
<tr>
<th>Description</th>
<th>Calculation</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>14.0 psf × 77' × 72.25' /1000</td>
<td>78.0 kips</td>
</tr>
<tr>
<td>SH 1</td>
<td>125 pcf × 12'' / 12 × 12'/2 × 77'/1000</td>
<td>58.0 kips</td>
</tr>
<tr>
<td>SH 2</td>
<td>125 pcf × 12'' / 12 × 12'/4 × 13'/1000</td>
<td>11.0 kips</td>
</tr>
<tr>
<td>SH 3</td>
<td>125 pcf × 12'' / 12 × 12'/2 × 86.5'/1000</td>
<td>68.0 kips</td>
</tr>
<tr>
<td>SH 4</td>
<td>125 pcf × 12'' / 12 × 14'/2 × 35.5'/1000</td>
<td>31.0 kips</td>
</tr>
<tr>
<td>SH 5</td>
<td>Same as SH 3</td>
<td>11.0 kips</td>
</tr>
<tr>
<td>SH 6</td>
<td>125 pcf × 12'' / 12 × 12'/4 × 40'/1000</td>
<td>39.0 kips</td>
</tr>
<tr>
<td>SH 7</td>
<td>Same as SH 2</td>
<td>39.0 kips</td>
</tr>
<tr>
<td>SH 8</td>
<td>125 pcf × 12'' / 12 × 12'/2 × 19.5'/1000</td>
<td>15.0 kips</td>
</tr>
<tr>
<td>SH 9</td>
<td>Same as SH 8</td>
<td>15.0 kips</td>
</tr>
</tbody>
</table>

\[ \text{SLI} = 362 \text{ kips} \]
STEEL MID RISE
CONSTRUCTION
ARTICLE

ASCE 7-10
SECTION 12

<table>
<thead>
<tr>
<th>REFERENCE</th>
<th>CALCULATIONS</th>
</tr>
</thead>
</table>

**SEISMIC CRITERIA**

$I = 1.0$

$R = 6.0$ (ORDINARY CONCRETE SHEAR WALL)

SITE CLASS = B

SEISMIC DESIGN CATEGORY = B

$S_g = 0.28 \text{g}$ \hspace{1cm} $F_a = 1.0$

$S_t = 0.11 \text{g}$ \hspace{1cm} $F_t = 1.0$

$S_{ms} = F_a S_g = 0.28 \text{g}$

$S_{mt} = F_t S_t = 0.11 \text{g}$

$S_{ds} = \frac{2}{3} S_{ms} = 0.19 \text{g}$

$S_{dt} = \frac{2}{3} S_{mt} = 0.07 \text{g}$

$C_s = \frac{S_{ds}}{(R/4)} = \frac{0.19 \text{g}}{(5/4)} = 0.038 \text{g}$

$C_t = 0.02$ \hspace{1cm} $x = 0.75$ \hspace{1cm} $h = 17.4 \text{ft}$

$T_a = C_t h x = 0.17 \text{ sec.}$

$C_{s_{max}} = \frac{S_{ds}}{(T_a (\frac{2}{T}))} = 0.08 \text{g}$

$C_{s_{min}} = 0.044 S_{ds} L = 0.008 \text{g}$

$C_s = 0.038 \text{g}$

**BASE SHEAR**

$V_s = C_s L$

$V_s = (0.038 \text{g})(362 \text{ KIPS}) = 14.0 \text{ KIPS}$

$V_s = 14.0 \text{ KIPS}$
WIND LOADING - N/S DIRECTION

STEP 0: \( h_{\text{max}} = 19' \text{ s 100'} \checkmark \)

1. ENCLOSED BUILDING
2. 19' ROOF HEIGHT \( \leq 60' \)
3. \( L = 86.5' \)
   \( B = 95.5' \)
   \( \frac{L}{B} = 0.93 \quad (0.2 \leq 0.93 \leq 5.0) \)
4. \( k_{24} = 1.0 \)

STEP 1: RISK CATEGORY II

STEP 2: MINIMUM US \( V = 110 \text{ MPH} \) (CALIFORNIA COMPARISON)

STEP 3: WIND LOAD PARAMETERS

- EXPOSURE CATEGORY B
- \( k_{3} = 1.0 \)
- ENCLOSED BUILDING

STEP 4: \( P_{0} = \frac{17.7 + 16.7}{2} = 17.1 \text{ PSF} \)

STEP 5: \( P_{0} = \frac{17.7 + 16.7}{2} = 17.0 \text{ PSF} \)

STEP 5: SLOPE = 2:13 or 7:40

1.84:12 2.1:12

ADJUSTMENT FACTOR = 0.605

<table>
<thead>
<tr>
<th>FLAT &lt; 2:12</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>( h = 17' )</td>
<td>LC1</td>
<td>NA</td>
<td>NA</td>
<td>-24.6</td>
<td>-21.8</td>
</tr>
<tr>
<td></td>
<td>LC2</td>
<td>NA</td>
<td>NA</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

WITH ADJ:

ZONE 3 = -16.8 PSF
ZONE 4 = -14.9 PSF
ZONE 5 = -12.3 PSF

\( V_{4\text{max N/S}} = 17.1 \text{ PSF} \times \frac{19'}{2} \times 72.5' / 1000 = 120 \text{ KIPS} \)

\( V_{4\text{max N/S}} < V_{s} \)

120 KIPS < 14.0 KIPS

SEISMIC GOVERNS
WIND LOADING - E/W DIRECTION

**ASCE 7-10 TABLE 21.5-1**

**STEP 0:** Building is an Enclosed, Simple Diaphragm Building

- Building roof height = 19'-0" (4100 ft)
- L/B = 94' / 187' = 1.08 (>0.2 & <5.0)

**STEP 1:** Risk Category II

**STEP 2:** V = 110 MPM (Based on California Comparison)

**STEP 3:** Exposure Category - B

Kzt = 1.0
Enclosed Building

**STEP 4:**

\[
P_k = \frac{16.7 + 17.5}{2} = 17.1 \text{ PSF}
\]

\[
P_0 = \frac{16.7 + 17.2}{2} = 16.95 = 17.0 \text{ PSF}
\]

**STEP 5:**

20 ft Height

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Zone 3</th>
<th>Zone 4</th>
<th>Zone 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pz = -29.5</td>
<td>Pz = -21.6</td>
<td>Pz = -17.9</td>
<td></td>
</tr>
<tr>
<td>Pb = 0.00</td>
<td>Pb = 0.00</td>
<td>Pb = 0.00</td>
<td></td>
</tr>
</tbody>
</table>

Adjustment Factor: 0.6845 → Pz = -16.8 | Pz = -14.9 | Pz = -12.3 |
| Pb = 0.00 | Pb = 0.00 | Pb = 0.00 |

**STEP 6:** Not Applicable (Kzt = 1.0)

**STEP 7:** Assume \( P_k \) & \( P_0 \) = 17.1 PSF

Wall Dimensions: 86.5' x 12'

Wind Base Shear = 17.1 PSF (86.5' x 12') = 8875 # = 9k

:: Seismic Loads Govern (14k > 9k)

\[ \Rightarrow \text{USE } V_s = 14k \]
SLAB-ON-GRADE

DUE TO MINIMAL DESIGN LOADS AND A ONE-STORY BUILDING, USE A TYPICAL SLAB-ON-GRADE DESIGN AS SPECIFIED BELOW. IN ADDITION, SOIL (SUBGRADE) MUST BE COMPACTED PRIOR TO CONSTRUCTION OF SLAB.

SLAB DESIGN:

5" NET CONCRETE SLAB
#3 @ 18" O.C. EACH WAY
ON COMPACTED SUBGRADE.

(MINIMUM COVER: 1" EACH FACE)

CONCRETE: f'c = 4000 PSI
REBAR: fy = 60 KSI
JOIST DESIGN - J1

ASSUMPTIONS: SPAN = 13 FT
DOUGLAS FIR LARCH NO.5
ALLOWABLE STRESS DESIGN

FBD:

$w_{D+W_L} = (2')(14.0\text{ psf} + 20.0\text{ psf}) = 68\text{ plf}$

BENDING

$$M = \frac{wL^2}{8} = \frac{(68\text{ psf})(13')^2}{8} = 1437\text{ lb}\cdot\text{ft}$$

$$f_b = \frac{M}{S} = \frac{1437\text{ lb}\cdot\text{ft} \times 12''}{S_{allow}} = \frac{17,241\text{ lb}\cdot\text{in}}{S_{allow}}$$

$$f_b' = \frac{525\text{ psi} \times C_D \times C_A \times C_D \times C_L \times C_C \times C_{fu} \times C_T \times C_Y}{1.25 \times 1.0 \times 1.0 \times 1.0 \times 1.1 \times 1.0 \times 1.0 \times 1.0} = 830\text{ psi}$$

$$S_{allow} = \frac{17,241\text{ lb}\cdot\text{in}}{830\text{ psi}} = 20.78\text{ in}^3$$

$$f_b = \frac{17,241\text{ lb}\cdot\text{in}}{21.39\text{ in}^3} = 807\text{ psi}$$

DEFLECTION

$$\Delta_{D+L,\text{allow}} = \frac{L}{240} = \frac{13'\times12''}{240} = 0.65\text{ in}$$

$$\Delta_{D+L,\text{demand}} = \frac{5wL^4}{384EI} = \frac{5(68\text{ plf})(13')^4(12'')^3}{384(1,400,000\text{ psi})(93,933\text{ in}^4)} = 0.32\text{ in} < 0.65\text{ in}$$
SHEAR:

\[ V = \frac{Wl}{2} = \frac{6.9 \times 12 \times 13^2}{2} = 442 \text{ lbs} \]

\[ f_v = \frac{V}{A} = \frac{442 \text{ lbs}}{13.88 \text{ in}^2} = 32 \text{ psi} \]

\[ F_v' = 180 \text{ psi} \times \frac{0.6 \times 1.0 \times 1.0 \times 1.0}{1.25} = 225 \text{ psi} \]

\[ F_v \leq F_v' \checkmark \]

\[ 32 \text{ psi} \leq 225 \text{ psi} \checkmark \]

\[ \frac{d}{C} = 0.142 \checkmark \]

SELF WEIGHT CHECK

\[ w = \frac{(\text{specific gravity})(\text{unit weight of water})(\text{volume of member})}{\text{trib area}} \]

\[ w = \frac{(0.5)(62.4 \text{pcf})(\frac{1.5}{12} \times \frac{9.25}{12} \times 13')}{(2' \times 13')} \]

\[ w = 1.5 \text{ psf} \leq 5.0 \text{ psf} \text{ assumption} \checkmark \]

**USE 2\times10 @ 24\text{°} O.C.**

**DOUGLAS FIR-LARCH NO.3**
BEAM DESIGN - B1

ASSUMPTIONS:
- Span = 45'-0"
- A992 Steel (Fy = 50 ksi, Fu = 65 ksi)
- LRFD Design

FBD:

\[ 12w_b + 1.6w_e = \left[ 1.2(14) + 1.6(20) \right] \times 10' = 188 \text{ PLF} \]

BENDING:

\[ M = \frac{wL^2}{8} = \frac{(188)(45)^2}{8} = 123.5 \text{ k-Ft} \]

\[ L_b = 0' \quad (JOISTS \ ARE \ BRACING \ BEAM \ LATERALLY) \]

TRY:
- W14 x 22
- W12 x 26
- W19 x 26

\[ \phi M_p = \phi M_n = 140 \text{ k-Ft} \]

\[ M_u = 123.5 + 1.2 \left( \frac{0.026 \text{ kLF}(45)^2}{8} \right) = 131.9 \text{ k-Ft} \]

\[ M_u = 131.9 \text{ k-Ft} < \phi M_n = 140 \text{ k-Ft} \quad \checkmark \]

\[ D/c = 0.94 \quad \checkmark \]

SHEAR:

\[ V_u = \frac{wL}{2} = \left( 188 + 1.2 \times 0.026 \right)(45) \times 12 = 11.7 \text{ k} < \phi M_n = 84.2 \text{ k} \quad \checkmark \]

DEFLECTION:

\[ \Delta_{\text{max}} = \frac{9wL^4}{8E_1} = \frac{5 \times (0.031(10) + 0.026)(45)^4(12)^3}{389(25,000)(209 \text{ in}^4)} = 5.7'' \quad (>2.25'') \]
DETERMINE $I_{eq}$

$$2.25 = \frac{5 \left( 0.034(10) + 0.050 \right) (45)^4 (12)^3}{384 (29,000)} \Rightarrow I_{eq} = 480.8 \text{ in}^4$$

TRY W16 x 40

BENDING:

$$M_u = 123.5 + 1.2 \left( \frac{0.040 (45)}{8} \right) = 135.7 \text{ k-ft} \quad \left( < \Phi M_n = 274 \text{ k-ft} \right) \checkmark$$

SHEAR:

$$V_u = \frac{W L}{2} = (0.488 + 1.2(0.040)(45))/2 = 12.06 \text{ k} \quad \left( < \Phi V_n = 146 \text{ k} \right) \checkmark$$

DEFLECTION:

$$\Delta_{PL, U, N} = 2.25^\prime$$

$$\Delta_{L, U, N} = \frac{P L}{360} = \frac{45(12)}{360} = 1.5^\prime$$

$$A_{pl} = \frac{5 \left( 0.034(10) + 0.040 \right) (45)^4 (12)^3}{384 (29,000)} = 2.33^\prime \quad (> 2.25)$$

TRY W18 x 40

BENDING:

$$M_u = 135.7 \text{ k-ft} \quad \left( < \Phi M_n = 274 \text{ k-ft} \right) \checkmark$$

SHEAR:

$$V_u = 12.06 \text{ k} \quad \left( < \Phi V_n = 146 \text{ k} \right) \checkmark$$

DEFLECTION:

$$\Delta_{PL, U, N} = \frac{5(0.034(10) + 0.040)(45)^4 (12)^3}{384 (29,000) (612)} = 1.98^\prime \quad (\leq 2.25^\prime) \checkmark$$

$$\Delta_{L} = \frac{5 (0.020)(45)^4 (12)^3}{384 (29,000) (612)} = 0.104^\prime \quad (\leq 1.5^\prime) \checkmark$$

USE W18 x 40
**BEAM DESIGN - B2**

**ASSUMPTIONS:**
- B2' SPAN
- A992 STEEL (F_y = 60 ksi, F_u = 65 ksi)
- LOAD RESISTANCE FORCE DESIGN (LRFD)

FBD:

![FBD Diagram](image)

**BENDING:**

\[
M_u = \frac{W_{plf} \cdot (32')^2}{3} \times \frac{1 \text{kip}}{1,000 \text{lbs}} = 63 \text{ kip-ft}
\]

L_b = 0 (JOISTS BRACE LATERALLY)

**TRY:**
- H114 x 22
- H10 x 18

\[
\phi M_p = \phi M_n = 1.2 \text{ kip-ft}
\]

\[
M_u = 63 \text{ kip-ft} + 1.2 \left( \frac{0.022 \text{ kip/ft}}{\text{8}} \right) (32')^2 = 66.4 \text{ kip-ft}
\]

\[
M_u \leq \phi M_n \checkmark
\]

\[
66.4 \text{ kip-ft} \leq 1.25 \text{ kip-ft} \checkmark
\]

\[
\frac{P}{L} = 0.63 \checkmark
\]

**SHEAR:**

\[
V_u = \frac{W_{plf}}{2} = 9 \text{ kips}
\]

\[
\phi V_n = 94.5 \text{ kips}
\]

\[
V_u \leq \phi V_n \checkmark
\]

\[
9 \text{kips} \leq 94.5 \text{kips} \checkmark
\]

\[
\frac{P}{L} = 0.10 \checkmark
\]

**DEFLECTION:**

\[
\Delta_{plf, demand} = \frac{5 \cdot W_{plf} \cdot (10' \times 34 \text{ psf}) + 0.022 \text{ kip/ft}}{384 \cdot E \cdot I} = \frac{180}{160} \text{ in} \leq \text{1.60 in} \checkmark
\]

\[
\Delta_{plf, demand} = 0.02 \text{ m} \leq 1.07 \text{ m} \checkmark
\]

**USE H114 x 22.**
COLUMN DESIGN - C1

ASSUMPTIONS: LID FLANGE COLUMN
HEIG = 17 FT
\( A_{trib} = 15' \times 16' = 240 \text{ SF} \)
\( L_o = 20 \text{ PSF} \)
\( D = 14 \text{ PSF} \)

REDUCED LIVE LOAD:
\( R_1 = 1.0 \)
\( R_2 = 1.0 \)

\[ L_s = L_o R_1 R_2 = 20 \text{ PSF} \times (1.0)(1.0) = 20.0 \text{ PSF} \]

LOAD COMBINATION
\[ 1.2 D + 1.6 L = 1.2(14 \text{ PSF}) + 1.6(20.0 \text{ PSF}) \]
\( = 49 \text{ PSF} \)

LOAD \( P_u = 49 \text{ PSF} \times 160 \text{ SF} = 7,840 \text{ lbs} = 7,840 \text{ KIPS} \)

\( P_u = 7,840 \text{ KIPS} \)

CAPACITY
\( K = 1.0 \) (PIN-PIN CONNECTION)

TRY L110 × 33
\( f_p = 195 \text{ kips} > 7,840 \text{ kips} \checkmark \)
\( f_s = 4.19 \text{ in} \quad , \quad f_y = 1.94 \text{ in} \)

STRONG AXIS
\[ \frac{K_L}{f_s} = \frac{(1)(17' \times 12'')}{4.19''} = 49 \]

WEAK AXIS
\[ \frac{K_L}{f_y} = \frac{(1)(17' \times 12'')}{1.94} = 105 \quad \text{WEAK AXIS CONTROLS} \]

USE L110 × 33 COLUMN
COLUMN FOOTING DESIGN - F1

ASSUMPTIONS: EXPANSIVE CLAY SOIL
ALLOWABLE SOIL PRESSURE = 1.5 KSF

SERVICE LOADS: (14 PSF + 20 PSF) x 160 SF = 5440 lbs = 6 kips

AREA REQUIRED = \frac{6 \text{kips}}{1.5 \text{KSF}} = 4 \text{ SF} \quad L_{\text{min}} = 2 \text{ FT}

TRY 4' x 4' SQUARE FOOTING (A = 16 SF)

FACTORED LOADS: \left[ 1.2(14 \text{PSF}) + 1.6(20 \text{ PSF}) \right] \times 160 \text{ SF} = 8 \text{kips}

\[ q_s = \frac{8 \text{kips}}{16 \text{ FT}^2} = 0.5 \text{ KSF} \]

DEPTH OF FOOTING

TRY 2' DEEP (+2', d = 20") \quad b_w = \sqrt{4q_s \text{trib area}}

\text{trib area} = \left[ \left(4''\right)^2 - \frac{(12'' + 20'')^2}{144} \right] = 9 \text{ FT}^2

\[ V_u = 0.9 \text{KSF} \times 9 \text{ FT}^2 = 4.6 \text{kips} \]

\[ V_c = 4\sqrt{f_c} \]

\[ \phi V_n = 0.75 V_c \left(\frac{d}{b}\right) \]

\[ = 0.75 \left( 4\left(\frac{3000 \text{ PSI}}{144}\right)\left(12'' + 20''\right)\left(20''\right) \right) = 436 \text{kips} \]

\[ \phi V_n \geq V_u \checkmark \]

TWO WAY ACTION

\[ V_u = 0.5 \text{KSF} \times 4' \left( 2' - \frac{20''}{12} \right) = 6.70 \text{kips} \]

\[ \phi V_n = \phi \left(2\sqrt{f_c} bw d\right) \]

\[ = 0.75 \left( \frac{2 \sqrt{5000} \left(4' \times 12''\right) \left(20''\right)}{144} \right) = 78 \text{kips} \]

\[ \phi V_n > V_u \checkmark \]

\[ \phi V_n \geq 78 \text{kips} > 6.7 \text{kips} \checkmark \]
FOOTING REINFORCEMENT

\( f'c = 3,000 \text{ psi} \)
\( f_t = 60 \text{ ksi} \)
\( P_d = 8 \text{ kips} \)
\( q_s = 0.5 \text{ ksf} \)

\[
M_u = 0.5 \text{ ksf} \times 4' \times \left( \frac{1.5'}{2} \right) = 2.15 \text{ kip-ft}
\]

\[
\text{Boreq} = \frac{M_u}{f_t f_y j d} = \frac{2.15 \text{ kip-ft} \times 12''}{0.9(60 \text{ ksi})(0.95)(20'')} = 0.03 \text{ in}^2
\]

USE MINIMUM REINFORCEMENT

\[
A_{\text{min}} = \max \left\{ \frac{0.018 \times 140,000}{45,000} \times (48'') \times (24'') = 2.07 \text{ in}^2 \right\}
\]

\[
\text{TRY (5) #6 (} A_s = 2.20 \text{ in}^2 \text{)}
\]

\[
\text{smax} = \min \left\{ \frac{18''}{5h} = 120'' \right\}
\]

\[
\text{smax} = \min \left\{ \frac{2h}{18''} = 72'' \right\}
\]

\[
S_{\text{prov}} = (4' \times 12'') - (2 \times 3'') - 5(0.75 \text{ in}) - 4s_{\text{prov}}
\]

\[
S_{\text{prov}} = 9.5''
\]

USE 5 #6 BARS EACH WAY

\( A_s = 2.20 \text{ in}^2 \)
**SOLVE FOR MOMENT CAPACITY**

\[
C = \frac{A_s f_y}{0.85 \gamma_m} = \frac{5(0.44 \text{ in}^2)(60 \text{ ksi})}{0.85(0.85)(0.85)(48 \text{ in})} = 0.76 \text{ in}
\]

\[
a = 0.36(0.76) = 0.27 \text{ m}
\]

\[
\epsilon_+= \frac{0.003(20" - 0.76")}{0.76"} = 0.076 \text{ in/in} > 0.0021 \text{ in/in} \quad \text{STEEL YIELDS}\checkmark
\]

\[
\phi M_n = 0.9 A_s f_y (d - \frac{a}{2})
\]

\[
= 0.9 (5)(0.44 \text{ in}^2)(60 \text{ ksi})(20" - \frac{0.27"}{2})
\]

\[
= 195 \text{ kip-ft}
\]

\[
I_d(\phi c) = \frac{48"}{2} - 10" - 3" = 12"^3
\]

\[
I_d(\phi c) = \frac{60,000}{25 \times 3000}(0.75) = 32.9"^3 > 12"^3 \quad \text{N.G}
\]

\[
\phi M_n \geq M_n \checkmark
\]

\[
195 \text{ kip-ft} \geq 225 \text{ kip-ft} \checkmark
\]

**USE NO STANDARD HOOKS**
SHEAR WALL DESIGN - N/S SU1

\[ V_s = 14.0 \text{kips} \]

\[ \uparrow \text{17 kips} \]

\[ \uparrow \text{7.0 kips} \]

\[ \uparrow \text{5.3 kips} \]

68' - 0"

53' - 0"

20' - 0"
**Shear Wall Design - SL1**

**Assumptions:**
- Rammed Earth Wall \( f'c = 1500 \text{ psi} \)
- Height = 12'-0"  
- Span = 20'-0"

**Load Resistance Factor Design (LRFD)**
- \( S_{D5} = 0.190 \)

**Loads:**
- \( 0.18 + 0.2S_{D5} \) D + 0.5L + E  
- \( 0.09 - 0.2S_{D5} \) D - E
- \( 0.12 + 0.2(0.19) \) D + 0.5L + E  
  \( 1.24D + 0.5L + E \)
- \( 0.09 - 0.2(0.19) \) D - E  
  \( 0.862 (D - E) \)

**Tributary Width of Wall = 5'**

\( w_{wall} = 1.24 (14.0 \text{ psf}) (5') + 0.5 (20 \text{ psf}) (5') = 0.14 \text{ klf} \)

\( p_{wall} = 1.24 (125 \text{ psf}) (\frac{5'}{12}) (20') (12') = 37.2 \text{ kips} \)

\( V_u = 5.8 \text{ kips} \)

**Shear Design**

\( \frac{h_u}{b_w} = \frac{12'}{20'} = 0.60 \leq 1.5 \checkmark \)

\( \frac{h_u}{b_w} = 2.0 \quad P_{sh} \geq P_h \quad \text{TWO CURTAINS OF REINFORCING IS NOT REQUIRED} \)

**Assume **\( \phi_v = 0.6 \)

\( \phi v_c = \phi A_{c} (0.6 \sqrt{f''c}) \)

\( = 0.6 (20' \times 12') (1.0) (1500 \text{ psi}) \)

\( = 16.75 \text{ kips} > V_u \)

**Use Minimum Reinforcing**

\( p_f = 0.025 (12' \times 12') = 0.36 \text{ in}^2/ft \)

**Use #4 E.F.**  
\( A_v = 2 (0.2 \text{ in}^2) = 0.4 \text{ in}^2 \)

\( V = \frac{(0.2 \text{ in}^2) (60 \text{ kips}) (20' \times 12')} {12} = 480 \text{ kips} \)

\( \phi V_h = 304 \text{ kips} \)

\( V_v = 8 A_{c_v} \sqrt{f''c} = 8 (12') (20' \times 12') (1500 \text{ psi}) = 892 \text{ kips} > 304 \text{ kips} \checkmark \)
FLEXURAL DESIGN

ASSUME T = C = \( \frac{Mu}{0.9lw} = \frac{(53 \text{kips})(12')}{0.9(10')} = 4 \text{kips} \)

\( A_{req} = \frac{4 \text{kips}}{0.9(60 \text{ ksi})} = 0.07 \text{ in}^2 \)

TRY #4

\( \phi M_n = 4,136 \text{ kip-ft} \)

DEPTH = 31'

CHECK IF \( \phi = 0.6 \)

\( V_n = \frac{304 \text{kips}}{0.6} = 507 \text{kips} \)

\( M_{n_{\text{max}}} = \frac{4,136 \text{ kip-ft}}{0.9} = 4,600 \text{ kip-ft} \)

\( V_n \times h_{\text{eff}} = 507 \text{kips} \times 12' = 6,084 \text{ kip-ft} > M_n \)

\( \therefore \phi = 0.75, \text{ wall will yield in flexure before shear} \)

\( \phi V_n = 507 \times 0.75 = 380 \text{kips} \)
BOUNDARY CONFINEMENT DESIGN

\[ \frac{f_w}{f_{uw}} = \frac{12''}{20'} = 0.60 < 2.0 \therefore \text{USE } 18.10.0.5 \]

\[ f_c = \frac{P_u}{A_{cv}} = \frac{M_u}{b_w l_w} \]

\[ f_c' = \frac{40kip}{(12'')(240')} + \frac{6.14\text{kip} \times 12''}{(12'')(240'')^2} = 0.021 = 21 \text{ psi} \]

0.2f'_c = 0.2(1500 \text{ psi}) = 300 \text{ psi} > f_c \therefore \text{SPECIAL CONFINEMENT IS NOT REQUIRED}

0.15f'_c = 228 \text{ psi} > f_c

(a) Transverse ties shall be hoops
(b) Bends shall engage longitudinal bars
(c) No.3 ties enclosing No.10 or smaller bars
(d) See above
(e) \( h_a \leq 14'' \)

\[ \frac{400}{f_y} = 0.0067 \]

\[ p_{\text{boundary}} = \frac{A_s}{bd} = \frac{0.2in^2}{12'' \times 12''} = 0.0014 < 0.0067 \quad \text{TRANSVERSE NOT REG'D} \]

VERTICAL SPACING

\[ s_{\text{max}} = \text{min} \left\{ \begin{array}{c} 6'' = 6'' \\ 6d_b = 6(0.5'') = 3'' \end{array} \right\} \]

90° HOOKS SHALL BE USED

CONFINEMENT STEEL SHALL EXTEND 12'' INTO FOOTING UNTIL \( f_c \leq 1.5f'_c \)
Code: ACI 318-14
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 05/22/18
Time: 10:42:12

File: c:\users\agupta\appdata\local\temp\sw #1.col
Project: Virunga SW #1

Column:
\[ f_c = 2 \text{ ksi} \quad f_y = 60 \text{ ksi} \]
\[ E_c = 2208 \text{ ksi} \quad E_s = 29000 \text{ ksi} \]
\[ f_c = 1.275 \text{ ksi} \quad e_{yt} = 0.00206897 \text{ in/ln} \]
\[ e_u = 0.003 \text{ in/in} \]
\[ \beta_1 = 0.85 \]

Engineer: CA
\[ A_g = 2880 \text{ in}^2 \]
\[ A_s = 8.00 \text{ in}^2 \]
\[ \rho = 0.28\% \]
\[ l_x = 1.3824e+007 \text{ in}^4 \]
\[ l_y = 34560 \text{ in}^4 \]
\[ \text{Min clear spacing} = 7.23 \text{ in} \quad \text{Clear cover} = 0.75 \text{ in} \]

\[ \phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65 \]
General Information:
=======================
File Name: c:\users\agupta\appdata\local\temp\sw #1.col
Project: Virunga SW #1
Column: ACI 318-14
Engineer: CA
Units: English
Run Option: Investigation
Slenderness: Not considered
Run Axis: X-axis
Column Type: Structural

Material Properties:
=====================
Concrete: User-defined
f'c = 2 ksi
Ec = 2207.6 ksi
fc = 1.275 ksi
Eps_u = 0.003 in/in
Beta = 0.85

Steel: Standard
f_y = 60 ksi
f_a = 29000 ksi
Eps_yt = 0.00206897 in/in

Section:
========
Rectangular: Width = 12 in
Depth = 240 in

Gross section area, A_g = 2880 in^2
I_x = 1.3824x10^4 in^4
r_x = 69.282 in
X_o = 0 in
Y_o = 0 in

Reinforcement:
===============
Bar Set: ASTM A615
Size Diam (in) Area (in^2) Size Diam (in) Area (in^2) Size Diam (in) Area (in^2)
--- ----------------- --- ----------------- --- ----------------- --- -----------------
# 3 0.35 0.11 # 4 0.50 0.20 # 5 0.63 0.31
# 6 0.75 0.44 # 7 0.88 0.60 # 8 1.00 0.79
# 9 1.13 1.00 # 10 1.27 1.27 # 11 1.41 1.56
# 14 1.69 2.25 # 18 2.26 4.00

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Pattern: Irregular
Total steel area: A_s = 8.00 in^2 at rho = 0.28% (Note: rho < 0.50%)
Minimum clear spacing = 7.23 in

Factored Loads and Moments with Corresponding Capacities:
========================================================

<table>
<thead>
<tr>
<th>No.</th>
<th>F_n</th>
<th>M</th>
<th>PhiMax</th>
<th>PhiMin/Mu</th>
<th>NA depth</th>
<th>D_t depth</th>
<th>eps_t</th>
<th>Phi</th>
</tr>
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<tr>
<td>1</td>
<td>39.00</td>
<td>84.00</td>
<td>4135.51</td>
<td>49.232</td>
<td>31.02</td>
<td>233.00</td>
<td>0.01953</td>
<td>0.900</td>
</tr>
</tbody>
</table>

*** End of output ***
**Shear Wall Design - E/W - SW2**

**Distribution of Base Shear**

\[ V = 14k \]

\[ k_1 = 32'-0" \]
\[ k_2 = 45'-0" \]
\[ k_3 = 77'-0" \]

\[ k_3 = 14k \left( \frac{45 - 38.5}{45} \right) = 2.02k \]
\[ k_2 = 14 - 2.02k = 11.98k = 12k \]
\[ k_1 = \frac{14k}{77'} = 0.18 \text{ k/ft} \left( \frac{32/12}{3} \right) = 2.91k = 3k \]

→ Design support \( k_1 \) as shear wall SW2.

**Shear Wall SW2 Design**

\[ L = 13'-0" \quad H = 13'-0" \quad \text{(Mean)} \]

\[ h > 10" : \text{Use 2 layers of reinf.} \]

Min. Wall Thickness \( \geq 4" \)

\[ \frac{135(13'x12')}{6.24"} \quad \text{(Governs)} \]

\[ h = 12" \quad (> h_{min} = 6.24" ) \checkmark \]

\[ h_w = 13' \]
LOAD COMBINATIONS

\((1.2 + 0.2 \text{ psf}) D + 0.5L + E = [1.2 + 0.2 (0.19)] D + 0.56 + E = 1.290 + 0.5L + E\)

\((0.9 - 0.2 \text{ psf}) D - E = (0.9 - 0.2 (19)) D - E = 0.862 D - E\)

LOADS

\(W_{\text{wall}} = 1.24 (19.0 \text{ psf}) (1') + 0.5 (20 \text{ psf}) (1') = 27.4 \text{ plf}\)

\(P_{\text{wall}} = 1.24 (125 \text{ kcf}) (12''/12') (13') (13') = 26.2 \text{ kips}\)

\(P_{\text{beam}} = 1.24 (19.0 \text{ psf}) (16' \times 11.5') = 3.2 \text{ kips}\)

Assume beam is distributed along wall length: 3.2k/13' = 0.25 klf

\(\Rightarrow NWALL \text{ total} = 27.4 \text{ plf} + 250 \text{ plf} = 277.4 \text{ plf} = 277 \text{ plf}\)

SHEAR DESIGN

\(W_{\text{w}} / W_{\text{n}} = 13'/13' = 1 (\pm 15\%) \checkmark\)

Assume \(\phi_{v} = 0.6\)

\(\phi Vc = 0.6 (462 / \sqrt{150})\)

\(= 0.6 (13' \times 13') (3)(1.0) (1/1500)\)

\(= 11.8k \ (> Vc = 3k) \checkmark\)
USE MINIMUM REINFORCEMENT

\[ f_c = 0.0025 \ (12'')(12'') = 0.36 \text{ in}^2/\text{ft} \]

⇒ USE #4 E.F. \( A_v = 2(0.2 \text{ in}^2) = 0.4 \text{ in}^2 \)

\[ V_s = \frac{(0.2 \text{ in}^2)(2)(60 \text{ ksi})(13'\times 12'')}{12} = 312 \text{ kips} \]

\[ \phi V_s = 0.6(312 \text{ k}) = 187.2 \text{ k} \]

\[ \phi V_n = \phi V_c + \phi V_s = 11.8k + 187.2k = 199k = 200k \]

\[ V_n \max = 8ACU\sqrt{f_c} = 8(12')(13'\times 12'\%) \sqrt{1500\text{ psi}} = 580k \Rightarrow \phi V_n \max = 348k \]

\[ \phi V_n \max = 348 \text{ k} > \phi V_n = 200 \text{ k} \checkmark \]

:: USE #4 @ 12'' E.F. (HORIZONTAL)

#4 @ 12'' E.F. (VERTICAL)

FLEXURAL DESIGN

ASSUME \( T = C = Mu / 0.91w = (8k)(13') / 0.9(13') = 3.33k \)

\[ A_{S \text{ req}} = \frac{3.33k}{0.3(60 \text{ ksi})} = 0.062 \text{ in}^2 \Rightarrow 7\text{ bars} (A_S = 0.2 \text{ in}^2) \]

(#3 could be acceptable, but try #4 to be conservative)

\[ \phi M_n = 1820 \text{ k-ft} \]

VERIFY \( \phi = 0.6 \)

\[ V_n = \frac{200k}{0.6} = 333.3k \]

\[ M_{\max} = 1820 \text{ k-ft}/0.9 = 2022.2 \text{ k-ft} \]

\[ V_n \times h_{eff} = 333.3k(13') = 4333 \text{ k-ft} (> M_{\max}) \]

⇒ \( \phi = 0.75 \Rightarrow \text{wall will yield in flexure before shear} \)

⇒ \( \phi V_n = 0.75(333.3k) = 250k \)
**BOUNDARY CONFINEMENT DESIGN**

\[ \frac{h_w}{b_w} = \frac{13'}{13'} = 1.0 (\leq 2.0) \quad \therefore \text{USE ACI 18.10.6.3} \]

\[ f_c = \frac{Mu}{A_{Ov}} \pm \frac{\frac{Mu}{b_{w}I_{w}^{2}}}{6} = \frac{30k}{(12\times156)} \pm \frac{(3k)(13') \times 12''}{(12'')(156')^2} = 0.026 = 26 \text{ psi} \]

\[ 0.2f_c' = 0.2(1500 \text{ psi}) = 300 \text{ psi} \quad (> f_c) \quad \therefore \text{SPECIAL CONFINEMENT NOT REQ'D.} \]

\[ 0.15f_c' = 225 \text{ psi} \quad (> f_c) \]

18.7.5.2

(a) **TRANVERSE TIES SHALL BE HOOPS**

(b) **BENDS SHALL ENGAGE LONGITUDINAL BARS**

(c) **NO.3 TIES ENCLOSING NO.10 OR SMALLER BARS**

(d) **SEE ABOVE**

(e) \[ f_y = 60,000 \quad \therefore \frac{400}{f_y} = \frac{400}{60,000} = 0.0067 \]

\[ \rho_{BOUNDARY} = \frac{A_s}{bd} = \frac{0.2in^2}{(12'')(12'')} = 0.0014 \quad (< 0.0067) \quad \rightarrow \text{TRANSVERSE REINF. NOT REQ'D} \]

**VERTICAL SPACING**

\[ \rho_{MAX} = \min \left\{ \frac{6''}{6db} = \frac{6''}{6(0.5'')} = 3'' \quad \therefore \text{GOVERNS} \right\} \]

* **STANDARD 90° HOOKS SHALL BE USED**

* **CONFINEMENT STEEL SHALL EXTEND 12'' INTO FOZING UNTIL f_c \leq 1.5 f_c'"
Code: ACI 318-14
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 05/22/18
Time: 10:42:39

File: u:\senior project\shear wall sw2.col
Project:
Column:
$ f_c = 2 \text{ ksi} $   $ f_y = 60 \text{ ksi} $
$ E_c = 2549 \text{ ksi} $   $ E_s = 29000 \text{ ksi} $
$ e_{yt} = 0.00206897 \text{ in/in} $
$ e_u = 0.003 \text{ in/in} $
$ \beta_1 = 0.85 $
Confinement: Tied
$ \phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65 $

Engineer:
$ A_g = 1872 \text{ in}^2 $   $ 28 \#4 \text{ bars} $
$ A_s = 5.60 \text{ in}^2 $   $ \rho = 0.30\% $  
$ x = 3.79642e+006 \text{ in}^4 $  
$ y = 22464 \text{ in}^4 $  
Min clear spacing = 7.25 in  
Clear cover = 1.87 in
General Information:

===

File Name: u:\senior project\shear wall sw2.col
Project:
Column: ACI 318-14
Engineer:
Run Option: Investigation
Slenderness: Not considered
Run Axis: X-axis
Column Type: Structural

Material Properties:

==

Concrete: Standard
f'c = 2 ksi
fc = 1.7 ksi
Ec = 2549.12 ksi
Eps_u = 0.003 in/in
Beta1 = 0.85

Steel: Standard
fy = 60 ksi
Es = 29000 ksi
Eps_yt = 0.00206897 in/in

Section:

===

Rectangular: Width = 12 in
Depth = 156 in

Gross section area, Ag = 1872 in^2
Ix = 3.79642e+006 in^4
rx = 45.0333 in
Xo = 0 in

Reinforcement:

=====

Bar Set: ASTM A615
Size Diam (in) Area (in^2) Size Diam (in) Area (in^2)
--- ---- --- --- ---- ---- --- --- ---- ----
# 3 0.38 0.11 # 4 0.50 0.20 # 5 0.63 0.31
# 6 0.75 0.44 # 7 0.88 0.60 # 8 1.00 0.79
# 9 1.13 1.00 # 10 1.27 1.27 # 11 1.41 1.56
# 14 1.69 2.25 # 18 2.26 4.00

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: Equal Bar Spacing (Cover to transverse reinforcement)
Total steel area: As = 5.60 in^2 at rho = 0.30% *(Note: rho < 0.50%)
Minimum clear spacing = 7.25 in

28 #4 Cover = 1.5 in

Factored Loads and Moments with Corresponding Capacities:

=================================

Pu Mux PhiMnx PhiMn/Mu NA depth Dt depth eps_t Phi
No. kip k-ft k-ft in in
--- ---- ---- ---- ---- ---- ---- ---- ---- 
1 3.00 0.00 1819.95 999.999 15.23 153.88 0.02731 0.900

*** End of output ***
**Shear Wall Foundation - N/S F2**

\[ V_e = 5.3 \text{ kips} \]

\[ P_{DL} = 39 \text{ kips} \]

\[ P_{LU} = 1 \text{ kip} \]

**Service Load Combinations**

5) \( (1.0 + 0.14 \text{ SPS})D + 0.7E \)

6) \( (1.0 + 0.103 \text{ SPS})D + 0.75L + 0.525E \)

8) \( 0.6D + 0.7E \Rightarrow \text{SPS not req'd} \)

Combine 5) & 6): \( (1.0 + 0.14 \text{ SPS})D + 0.75L + 0.7E \)

**Overturning Moment**

\[ M_{ot} = (0.75)(0.7)(5.5 \text{ kips})(12') = 29 \text{ kip-ft} \]

Try \( L = 26', H = 6', t = 3' \)

**Load Case I: 0.6D + 0.7E**

\[ P_{DL} = 39 \text{ kips} \]

\[ P_{FG} = (0.18 \text{ ksf})(26')(6')(3') = 71 \text{ kips} \]

\[ E_{P} = 39 \text{ kips} + 71 \text{ kips} = 110 \text{ kips} \times 0.6 = 66 \text{ kips} \]

\[ M_{Resist} = (66 \text{ kips})(26'/2) = 888 \text{ kip-ft} \geq 29 \text{ kip-ft} \checkmark \]

\[ x = \frac{M_{Resist} - M_{ot}}{2P} = \frac{888 \text{ kip-ft} - 29 \text{ kip-ft}}{66 \text{ kips}} = 12.48 \text{ ft} \]

\[ I = 3x = 37 \text{ ft} \]

\[ f_{bearing} = \frac{66 \text{ kips}}{(\frac{1}{2})(37')(6')} = 0.6 \text{ ksf} \]

\[ f_{allow} = 1.5 \text{ ksf} \times 1.82 = 2.0 \text{ ksf} \]

\[ f_{allow} \geq f_{bearing} \checkmark \]

\[ 2.0 \text{ ksf} > 0.6 \text{ ksf} \checkmark \]
LOAD CASE II: \((1.0 \times 0.14 \text{SPS}) D + 0.76L + 0.7E\)

\[ P_{DL} = 59 \text{ KIPS} \]
\[ P_{L} = 1 \text{ KIP} \]
\[ P_{ETC} = 71 \text{ KIPS} \]

\[ S = (1.0 + 0.14(0.19))(39 \text{ KIPS} + 71 \text{ KIPS}) + 0.75(1 \text{ KIP}) = 114 \text{ KIPS} \]

\[ M_{RES} = 114 \text{ KIPS} \times (26.2') = 1,482 \text{ KIP}.\text{FT} \geq 264 \text{ KIP}.\text{FT} \checkmark \]

\[ x = \frac{1,482 \text{ KIP}.\text{FT} - 264 \text{ KIP}.\text{FT}}{114 \text{ KIPS}} = 12.7 \text{ FT} \]

\[ f_{\text{fallow}} = 2.0 \text{ KSF} \]

\[ f_{\text{fallow}} = 2.0 \text{ KSF} \]

\[ f_{\text{fallow}} = 1.02 \text{ KSF} \]

FACTORED LOAD DESIGN

5) \((1.2 + 0.2 \text{ SPS})D + 0.5L + E\)

7) \(0.9D + E\)

USE LC # 5

\[ P = (1.2 + 0.2(0.19))(110 \text{ KIPS}) + 0.5(1 \text{ KIP}) = 137 \text{ KIPS} \]

ASSUME \( d = 2.67' \)

\[ V_u = \left( \frac{0.9}{0.6} \right) \left[ \frac{1}{2}(\frac{1}{2})(1.24 - 0.4) + 0.4 \text{ KSF} \times \frac{3}{2} \right] (6') = 2.96 \text{ KIPS} \]

\[ V_u = 0.75 \left( \frac{2}{1500 \text{ PSI}} \right) (6' \times 12')(2.67')(2') = 134 \text{ KIPS} \]

\( f_{Vc} \times V_u \checkmark \)

\( 134 \text{ KIPS} \geq 2.46 \text{ KIPS} \checkmark \)
LONGITUDINAL BTM REINFORCEMENT

\[ q = 1.24 \text{ ksf} \]

\[ V_1 = \frac{0.4}{0.8} (0.5) (1.24 \text{ ksf}) (0.4 \text{ ksf}) (3') (6') = 11.34 \text{ kips} \]

\[ V_2 = \frac{0.4}{0.6} (0.4 \text{ ksf}) (3') (6') = 10.8 \text{ kips} \]

\[ \beta_1 = \frac{3}{5} (3') = 2' \]

\[ \beta_2 = 1.5' \]

\[ \sum M_u = 11.34 \text{ kips} (2') + 10.8 \text{ kips} (1.5') \]

\[ = 39 \text{ KIP-FT} \]

\[ A_{\text{min}} = 0.00181 \bar{f}_t = 4.67 \text{ in}^2 \]

TRY \((\beta) = 5 \text{ (B) LONG} \)

\[ \bar{f} = 8(0.81 \text{ ksi})(60 \text{ ksi}) = 149 \text{ kips} \]

\[ \alpha = \frac{149 \text{ kips}}{0.85 (1.5 \text{ ksi}) (6' \times 12''')} = 1.62'' \]

\[ c = 1.91'' \]

\[ \varepsilon_s = \frac{0.003 (2.67'' \times 12'' - 1.91'')}{1.91''} = 0.097''/\text{in} > 0.0021''/\text{in} \text{ R. STEEL YIELDS} \]

\[ \phi = 0.9 \]

\[ \phi M_n = 0.9 (3)(0.81 \text{ ksi})(60 \text{ ksi}) (31'' - \frac{1.02''}{2}) / 12 = 837 \text{ KIP-FT} \]

\[ \phi M_n = M_u \checkmark \]

\[ 337 \text{ KIP-FT} < 39 \text{ KIP-FT} \checkmark \]

\((\beta) = 5 \text{ LONG. (B)} \)
TRANSVERSE FLEXURAL REINFORCEMENT

\[ W_u = 0.9 \times 0.6 \times 1.24 \text{ ksf} = 1.86 \text{ ksf} \]

\[ M_u = 1.86 \text{ ksf} \times (2.5')^2 / 2 = 6 \text{ kip-ft/ft} \]

\[ A_{\text{min}} = \frac{0.001 \times (3' \times 12'')(26' \times 12'')}{2} = 10.1 \text{ in}^2 \]

TRY #7 @ 16" 0.6 (A_s = 11.4 in²)

\[ a = \frac{(11.4 \text{ in}^2) \times (60 \text{ ksf})}{0.85(1.6 \text{ ksf}) (26' \times 12'')} = 1.72'' \quad c = 2.02'' \]

\[ E_s = 0.044'' / \text{in} \quad \phi = 0.9 \]

\[ \phi M_n = 0.9 \times 0.6 \text{ in}^2 \times (60 \text{ ksf}) (36'' - 3'' - 2'' - 1.72'' / 2) / 16 = 61 \text{ kip-ft/ft} \]

\#7 @ 16" TRANSV (E)

LONGITUDINAL TOP REINFORCEMENT

\[ W_u = 1.4 (3') \times 0.15 \text{ ksf} = 0.63 \text{ ksf} \]

\[ M_u = 0.63 \text{ ksf} \times (3')^2 / 2 \times 6' = 17 \text{ kip-ft} \]

\[ A_{\text{min}} = 4.67 \text{ in}^2 \]

\[ A_{\text{min}} (\text{top}) = \frac{4.67 \text{ in}^2}{2} = 2.34 \text{ in}^2 \Rightarrow \#5 \text{ MIN} \quad A_s = 2.48 \text{ in}^2 \]

\[ a = \frac{8(0.51 \text{ in}^2) \times (60 \text{ ksf})}{0.85 (1.5 \text{ ksf}) (4' \times 12'')} = 1.62'' \quad c = 1.91'' \]

\[ E_s = 0.044'' / \text{in} \quad \phi = 0.9 \]

\[ \phi M_n = 0.9 (0.31 \text{ in}^2) \times (60 \text{ ksf}) (36'' - 3'' - 2'' - 1.62'' / 2) / 12 = 557 \text{ kip-ft} \]

\[ \#7 \text{ @ 16" TRANSV (B)} \]

TRANSVERSE TOP REINF.

\[ W_u = 0.63 \text{ ksf} \]

\[ M_u = 0.63 \text{ ksf} \times (2.5')^2 / 2 = 1.97 \text{ kip-ft/ft} \]

USE #7 @ 16" 0.6 (A_s = 11.4 in²)

\[ \phi M_n = 61 \text{ kip-ft/ft} \]

\[ \#7 \text{ @ 16" TRANSV (F)} \]
**SHEAR WALL FOUNDATION - E/W F3**

\[ V_e = 3 \text{kips} \quad (V) \]
\[ P_{DL} = 29 \text{kips} \quad (P) \]
\[ P_{LL} = 1 \text{kip} \quad (P) \]

---

**ASCE 7-10**
12.2.4.3

**SERVICE LOAD COMBINATIONS**

5) \((1.0 + 0.14 \times 50) \times 0 + 0.7E\)

6) \((1.0 + 0.10 \times 50) \times 0.75E + 0.525E\)

8) \(0.60 + 0.7E \rightarrow 50\% \text{ Nat Rebid}\)

\[ \rightarrow \text{COMBINE 5} + 6: (1.0 + 0.14 \times 50) \times 0 + 0.75E + 0.7E \]

**OVERTURNING MOMENT**

\[ M_{OT} = 0.75(0.7)(3k)(13') = 21 \text{kifft} \]

\[ \rightarrow \text{TRY } L = 16', W = 4', t = 3' \]

**LOAD CASE 1:** \(0.60 + 0.7E\)

\[ P_{DL} = 29 \text{kips} \]
\[ P_{FIG} = (0.150 \text{kcf})(16')(4')(3') = 28.8k \]
\[ \Delta P = 29K + 29K = 58K \times 0.6 = 35K \]
\[ M_{RESIST} = 35K \times 16'/2 = 280 \text{kft} \geq 21 \text{kft} \checkmark \]
\[ X = \frac{M_{RESIST} - M_{OT}}{2P} = \frac{280 - 21}{35} = 7.9' \]
\[ L = 3X = 3(7.9') = 23.7' \]
\[ f_{BEARING} = \frac{35K}{(4')(22.2')(4')} = 0.79 \text{kps} \]
\[ f_{ALLOW} = 1.5 \text{kps} \times 1.33 = 2.0 \text{kps} \checkmark \text{(33\%, increase allowed)} \]
**Load Case II**: 

\[ (1.0 + 0.145D5)D + 0.75L + 0.76 \]

- \( P_o = 2.9 \text{ kips} \)
- \( P_L = 1 \text{ kip} \)
- \( P_{FG} = 2.9 \text{ kips} \)

\[ 80 = (1 + 0.14(0.19))(29 + 29) + 0.75(10) = 60.3 \text{ k} \]

**MRESIST**: 60.3 k (16/12) = 482.4 k-ft (> 211 k-ft) ✓

\[ \chi = \frac{482.4 \text{ k-ft}}{60.3} = 7.65' = 7.7' \]

\[ I = 3x = 3(7.7') = 23.1' \]

**fBEARING**: 60.3 / (1/2 x 23.1' x 1') = 1.30 kSF

**fALLOW**: 2.0 kSF ✓

**Factored Load Design**

- 5) \((1.2 + 0.2SDS)D + 0.5L + E\)
- 7) \(0.9D + E\)

**LC #5**: \[ P = (1.2 + 0.2(0.19))(58k) + 0.5(1k) = 72.3k \]

* Assume \( d = 2.67' \)

\[ 1.30 \text{ kSF} = \frac{\chi}{23.1'} \]

\[ \Rightarrow \chi = 1.27 \text{ kSF} \]

\[ V_o = \left( \frac{0.9}{0.6} \right) \left[ \frac{1}{2} (0.33')(1.30 - 1.27) + 1.27(0.33') \right] 4' = 2.57 \text{ k} \]

\[ \phi V_o = 0.75(2) \frac{1}{1500} (4'x12')(2.67')(12') \]

\[ \Rightarrow \phi V_o = 89.3 \text{ k} (> V_o = 2.57 \text{ k}) ✓ \]
LONGITUDINAL BOTTOM REINFORCEMENT

\[ Q = 1.27 \text{ ksf} \]

\[ V_1 = (0.9) (0.5) (1.30 - 1.27) (3') (4') = 11.43 \text{ k} \]

\[ V_2 = (0.9) (1.27 \text{ ksf}) (3') (4') = 10.91 \text{ k} \]

\[ \beta_1 = 2/3 (3') = 2' \]

\[ \beta_2 = 1.5' \]

\[ \\]

\[ E_{MU} = 11.43 \text{ k} (2') + 10.91 \text{ k} (1.5') = 32.2 \text{ k} \cdot \text{ft} \]

\[ \text{ASMIN} = 0.0018 \text{AG} = 0.0018 (4'x12')(3'x12') = 3.11 \text{ in}^2 \quad \text{(SPLIT TOP & BOTTOM)} \]

\[ \text{TRY} \#6 \quad \text{(BOTTOM LONG.)} \]

\[ 7 = 6 (0.31 \text{ in}^2) (60 \text{ ksi}) = 112 \text{ k} \]

\[ a = \frac{112 \text{ k}}{0.85 (1.5 \text{ ksi}) (4'x12')} = 1.83'' \]

\[ c = 2.15'' \quad (a/\beta_1) \rightarrow \beta_1 = 0.85 \]

\[ E_S = 0.003 \frac{2.67'x12'' - 2.15''}{2.15''} = 0.0417 \text{ in/min} \quad (>0.0421) \quad \text{STEEL YIELDS (\phi=0.9)} \]

\[ \phi_{MU} = \left[ 0.9 (6) (0.31 \text{ in}^2) (60 \text{ ksi}) \left( 32'' - \frac{1.83}{2} \right) \right] / 12 = 200.2 \text{ k-ft} \quad (>\mu=39.2 \text{ k-ft}) \]

\[ \therefore \text{USE} \#6 \quad \text{#5 BARS LONG. (B)} \]

TRANSVERSE FLEXURAL REINFORCEMENT (BTM)

\[ W_u = (0.9/0.6) (1.27 \text{ ksf}) = 1.9 \text{ ksf} \]

\[ \mu_u = 1.9 \text{ ksf} \left( 2.67' \right)^2/2 = 6.7 \text{ k-ft/ft} \]

\[ \text{ASMIN} = 0.0018 (3'x12'')(16x12'') / 2 = 6.22 \text{ in}^2 \]

\[ \text{TRY} \#6 @ 12'' \cdot c \quad (A5 = 7.04 \text{ in}^2) \]

\[ a = \frac{7.04 (60 \text{ ksi})}{0.85 (1.5 \text{ ksi}) (16x12'')} = 1.72'' \quad \rightarrow \quad c = 2.03'' \]
**CALCULATIONS**

\[ E_s = 0.041 \text{ in/ln} \quad (> 0.021) \quad \checkmark \quad \phi = 0.9 \quad \checkmark \]

\[ \phi M_n = 0.9 (0.44 \text{ in}^2) (60 \text{ ksi}) (36^\circ - 3^\circ - 2^\circ - \frac{1.72^\circ}{2}) / 12 = 59.7 \text{ k-ft/ft} \]

\( (> 6.7 \text{ k-ft/ft}) \quad \checkmark \)

\[ \text{USE #6 @ 12^\circ o.c. TRANS. BOTTOM REINF.} \]

**LONGITUDINAL TOP REINFORCEMENT**

\[ W_u = 1.1 (3') (1') (0.15 \text{ ksf}) = 6.88 \text{ ksf} \]

\[ M_{u} = 0.68 \text{ ksf} (3')^2 / 2 \times 4' = 11.34 \text{ k-ft} \]

\[ A_{\text{min}} = 3.11 \text{ in}^2 \]

\[ A_{\text{min}} \,(\text{top}) = 3.11 \text{ in}^2 / 2 = 1.56 \text{ in}^2 \rightarrow \text{TRY (6) #5 (As = 1.86 in}^2) \]

\[ a = \frac{6 (0.31) (60 \text{ ksi})}{0.85 (1.5 \text{ ksf}) (4 \times 2)} = 1.82^\circ \rightarrow C = 2.15^\circ \]

\[ E_s = 0.0401 \quad (> 0.021) \quad \checkmark \quad \phi = 0.9 \quad \checkmark \]

\[ \phi M_n = 0.9 (6) (0.31) (60) (36^\circ - 3^\circ - 2^\circ - \frac{1.82^\circ}{2}) / 12 = 251.8 \text{ k-ft} \quad (> 11.34 \text{ k-ft}) \quad \checkmark \]

\[ \text{USE (6) #5 LONG. (7) REINFORCEMENT} \]

**TRANSVERSE TOP REINFORCEMENT**

\[ W_u = 0.68 \text{ ksf} \]

\[ M_{u} = 0.68 \text{ ksf} (2.67')^2 / 2 = 2.25 \text{ k-ft/ft} \]

\[ \text{TRY #6 @ 12^\circ o.c. (As = 7.04 in}^2) \]

\[ \phi M_n = 59.7 \text{ k-ft/ft} \quad (> M_u = 2.25 \text{ k-ft/ft}) \quad \checkmark \]

\[ \text{USE #6 @ 12^\circ o.c. TRANS. TOP REINFORCEMENT} \]
DIAPHRAGM DESIGN - N/S

\[ \omega = \frac{V}{h} = \frac{16 k k}{53'} \]
\[ \Rightarrow \omega = 0.264 \text{ klf} \]

**Aspect Ratio:**

\[ 77' / 53' = 1.45 \]
\[ \Rightarrow 1.45 \neq 3:1 \]

---

\[ V_{(H)} \begin{array}{c}
0 \begin{array}{c}
1.7 \\
22
\end{array}
\end{array} \\
V_{(Kl)} \begin{array}{c}
0 \begin{array}{c}
-1.7 \\
-22
\end{array}
\end{array} \\
M_{(kft)} \begin{array}{c}
0 \begin{array}{c}
5.3 \\
53
\end{array}
\end{array} \\
T_{(G)_{(k)}} \begin{array}{c}
0 \begin{array}{c}
72 \\
688
\end{array}
\end{array} \]

---

\[ \text{REFERENCE} \]

\[ \text{CALCULATIONS} \]
**SHEAR DESIGN**

3/8" SHEATHING - STRUC 1 (UNBLOCKED)
ASSUME 8d NAILS

\[ V = 0.7 Vu = 0.7 \times 91 \text{ plf} = 63.7 \text{ plf} \]

TRY 8d @ 6", 6", 12", NO BLKG. CASE 3

\[ V_{\text{allow}} = \frac{360}{2} = 180 \text{ plf} \]

\[ V_{\text{allow}} = 180 \text{ plf} > 63.7 \text{ plf} \checkmark \]

\[ \therefore \text{ 8d @ 6", 6", 12", NO BLOCKING} \]

(3/8" SHEATHING - STRUC 1)

**CHORD FORCES**

\[ T/C \text{ CHORD} = 688 \# \times 0.7 = 482\# \]

**COLLECTOR FORCES**

**INTERIOR COLLECTOR**

\[ T/C \text{ MAX} = 2912 \# \]
EXTERIOR COLLECTOR:

$V = 69 \text{ PLF}$

$V = 205 \text{ PLF}$

$20'$

$57'$

$T/C \text{ MAX} = 3920 \#$

* COLLECTOR FORCES GOVERN
DIAPHRAGM DESIGN - E/W

$W_e = 14 \text{ kips/ft}, \quad 0.18 \text{ kip/ft}$

$V(kips)$

$M(\text{kip-ft})$

$\gamma(\text{plf})$

$\gamma_c(\text{lbs})$

$\gamma_{max} = 77 \text{ plf}$

Calculated By: CA, Checked By: NG
DEFLECTION - ASPECT RATIO

\[
55' / 77' = 0.69 : 1.0 < 3 : 1 \quad \checkmark
\]

SHEAR DESIGN

TRY 3/8" SHEATHING, STRUCTURAL 1 (UNBLOCKED), 8d NAILS

\[V_u = 0.7 V_{max} = 0.7 (131 \text{ PLF}) = 92.0 \text{ PLF}\]

TRY 8d @ 6", 6", 12" NO BLOCKING, CASE 1

\[V_{ABD} = 480 \text{ PLF}\]

\[V_{allow} = \frac{480 \text{ PLF}}{2} = 240 \text{ PLF}\]

\[V_{allow} = \frac{240 \text{ PLF}}{54 \text{ PLF} \geq 240 \text{ PLF}} \quad \checkmark\]

CHORD FORCE = 0.7 (868 lbs) = 608 lbs

COLLECTOR FORCES

LINE E:

\[V = 131 \text{ PLF}\]

\[V = 174 \text{ PLF}\]

\[V = 1720 \text{ PLF}\]

USE 5/8" STRUC 1, 8d @ 6", 6", 12" NO BLOCKING REQUIRED
**Line G:**

\[ 54 \text{ PLF} = Y \]

\[ Y = 220 \text{ PLF} \]

\[ 40' '0'' \]

\[ 13' '0'' \]

**Line B:**

\[ Y = 77 \text{ PLF} \]

\[ < 146 \text{ PLF} \]

\[ 19' '0'' \]

\[ 28' '0'' \]

\[ 6' '0'' \]

**Collector Force Governs**

\[ P = 2160* \]
CONCRETE BOND BEAM DESIGN

ASSUMED SECTION

** Section will be placed horizontally.**

\[ f'c = 1,000 \text{ psi} \quad f_y = 60 \text{ ksi} \quad A_s = 2 \text{ in}^2 \quad M_u = 53 \text{ k-ft} \]

\[ a = d - \frac{2M_u}{\sqrt{f'c A_s + d^2}} \]

\[ d = 10" - 1.5" - 0.375" = 10" \]

COVER STIRRUP

\[ a = 10" - \sqrt{\frac{2 \times 65 \times 12}{(0.75)0.85(4)(8) + 10^2}} \]

ASSUME \( d = 0.75 \) (CONSERVATIVE)

\[ a = 3.86" \implies A_s = \frac{0.85 f'c b a}{f_y} = \frac{0.85(4)(8)(3.86)}{60} = 1.75 \text{ in}^2 \]

\[ \therefore \text{ USE (2) #9 REBAR T/L} \implies 2(1.00 \text{ in}^2) = 2 \text{ in}^2 \ (> 1.75 \text{ in}^2) \]

FLEXURAL DESIGN

(\text{ASSUME } f_s = f_y)

\[ T = A_s f_y = (2)(60) = 120 \text{ k} = 0.85 f'c b a = 0.85(4)(8)(a) \implies a = 4.41" \]

\[ C = \frac{a}{f} = 4.41" = 5.19" \]

\[ e_s = \frac{0.003}{5.19"} = 0.0028 \] (\text{FY = 0.0028}) \checkmark \text{ STEEL YIELDS}
\[ M_n = T (d - 9\%e) = 120k \left( 10^5 - 4.41^{1/2} \right) = 935.4 \text{ k} = 78 \text{ k}' \]

\[ \phi M_n = 0.75 (78 \text{ k}') = 59 \text{ k}' \ (\geq 53 \text{ k-Fr}) \checkmark \text{ O.K.} \]
Code: ACI 318-14
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/03/18
Time: 19:59:13

f'c = 4 ksi  fy = 60 ksi  Ag = 96 in^2  4 #9 bars
Ec = 3605 ksi  Es = 29000 ksi  As = 4.00 in^2  rho = 4.17%
fc = 3.4 ksi  e_yt = 0.00206897 in/in  Xo = 0.00 in  lx = 1152 in^4
e_u = 0.003 in/in  Yo = 0.00 in  ly = 512 in^4
Beta1 = 0.85  Min clear spacing = 3.49 in  Clear cover = 1.12 in

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65
General Information:

File Name: U:\SENIOR PROJECT\Bond Beam.col

Project:
Column: ACI 318-14
Run Option: Investigation
Run Axis: X-axis

Material Properties:

Concrete: Standard
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Eps_u = 0.003 in/in
Beta1 = 0.85

Steel: Standard
fy = 60 ksi
Ere = 29000 ksi
Eps_yt = 0.00206697 in/in

Section:
Rectangular: Width = 8 in
Gross section area, Ag = 96 in^2
Ix = 1152 in^4
rx = 3.4641 in
Xo = 0 in

Reinforcement:
Bar Set: ASTM A615
Size Diam (in) Area (in^2)
3 0.38
6 0.75
9 1.13
14 1.69

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)

Top Bottom Left Right
Bars 2 #9 2 #9 0 #3 0 #3
Cover(in) 0.75 0.75 0.75 0.75

Factored Loads and Moments with Corresponding Capacities:

No. Pu Mu/k-ft PhiM/k-ft Mux PhiM/Mu k-ft NA Depth Dt depth in in eps_t Phi
1 3.92 53.00 81.48 1.537 2.76 10.31 0.00822 0.900

*** End of output ***
**SLAB ON GRADE - N/S SLIDING CHECK**

\[ V_E (service) = 5.3 \text{kips} \times 0.7 = 4.0 \text{kips} \]

\[ \Sigma P_{DL} = 66 \text{kips} \]

\[ \text{FRICTION} = \mu P_{DL} = 0.3 (66 \text{kips}) = 20 \text{kips} \]

\[ \text{PASSIVE} = \frac{0.100 \text{kcf} (1 \times 3')(3')^2}{2} = 4 \text{kips} \]

\[ \Sigma \text{FRICTION + PASSIVE} = 24 \text{kips} > 4.0 \text{kips} \]

**NO DOUBLES REQUIRED**

*Also greater than max \( V_E \) in direction 0.7(7 kips) = 5.0 kips*
SLAB ON GIRDER - E/W SLIDING CHECK

\[ V_A \text{ (SERVICE)} = 2.1 \text{ KIPS} \]

\[ E_{PDL} = 35 \text{ KIPS} \]

\[ \text{FRICTION} = 0.3(35 \text{ KIPS}) = 10.5 \text{ KIPS} \]

\[ \text{PASSIVE} = \frac{(0.100 \text{ KCF})(1.38)(3')(4')}{2} = 2.4 \text{ KIPS} \]

\[ \text{FRICTION} + \text{PASSIVE} = 12.9 \text{ KIPS} > 2.1 \text{ KIPS} \checkmark \]

* ALSO GREATER THAN MAX \( V_A \) IN DIRECTION \( 0.7(12 \text{ KIPS}) = 8.4 \text{ KIPS} \checkmark *

NO DOUBLES REQUIRED
Appendix B - Drawings
SENIOR PROJECT THESIS

VIRUNGA COFFEE//COCOA CO-OP
RWANDA, AFRICA

JOURNEYMAN INTERNATIONAL ENGINEERING TEAM
CALEB AZEVEDO | ANUGRAH GUPTA

DR. ALLEN ESTES

SPRING QUARTER 2018
CALIFORNIA POLYTECHNIC STATE UNIVERSITY
SAN LUIS OBISPO, CA

NOTE: NOT FOR CONSTRUCTION, TO BE REVIEWED AND APPROVED BY IN-COUNTRY ENGINEER.

SHEET INDEX

S0.1 - TITLE SHEET & SHEET INDEX
S1.1 - GENERAL NOTES
S2.1 - TYPICAL DETAILS
S2.2 - TYPICAL DETAILS
S2.3 - TYPICAL DETAILS
S2.4 - NORTH-SOUTH SHEAR WALL TYPICAL DETAILS
S2.5 - EAST-WEST SHEAR WALL TYPICAL DETAILS
S3.1 - FOUNDATION PLAN
S3.2 - ROOF FRAMING PLAN
1. Applicable Standard: ACI 301

2. Portland Cement: ASTM C150, Type II.

3. Normal Weight Concrete (145 psi): ASTM C33 for aggregates of natural sand and rock. Concrete to contain the following 28-day minimum compressive strengths (psi), unless shown on contract documents or pre-approved by Structural Engineer.

- 4,000 psi
- 4,000 psi
- 4,000 psi
- 4,000 psi
- 5,000 psi

4. Maximum Aggregate Sizes: 1-1/2 inches at foundations and slabs on grade and 1 inch elsewhere.

5. Lean Concrete: Where specifically indicated, containing 2 sacks of cement per cubic yard of concrete.

6. Maximum Slump: 5 inches in 4-inch test; 6 inches in 8-inch test.

7. Shrinkage: ASTM C157, limit to 0.55 percent.

8. Use of Chlorines: Not permitted.

9. Concrete Mix Design Data: Submit for each type and compressive strength of concrete required and signed by a registered civil engineer in state to Structural Engineer. Basis of design on field experience or trial mixtures as stipulated in ACI.

10. Shop Drawings: Submit to Structural Engineer indicating location of concrete construction prior to placing concrete. Locate points at locations to minimize effects of shrinkage as well as being placed at points of low stress.

11. Conduits, Pipes, and Sleeves: Do not embed other than electrical conduits 1 inch outside diameter and smaller in structural concrete. Locate electrical conduit 4 inches apart minimum and within middle third of member height.

12. Chamfered Corners: Provide ½ inch chamfer at exposed corners of columns, beams and walls unless detailed otherwise.


14. Concrete Abutting Structural Masonry Walls: Roughen concrete surface to full wall thickness. Use 2-inch straight edge.

15. Curing: Maintain concrete above 50 degrees Fahrenheit and in a moist condition for a minimum of 7 days after placement unless otherwise approved by Structural Engineer.

16. Rammed Earth

1. Applicable Standards: ACI 301

2. Rammed Earth: Rammed Earth to attain a minimum compressive strength (f’c) of 1500 psi.

3. Minimum distance between rammed earth structures should be free from organic material and any other harmful substances. Check and marked in conformance with the standard grading of the Rwandan Engineer.

4. All framing lumber shall be kiln dried of MC 0.3%.

5. Fire-Gravel and Sand: 50% to 70% gravel and 30% to 50% sand.

6. Soil: 5 to 15% clay.

7. Soil should be tested using the "roll" method with the break off being between 60 mm and 120 mm.

8. Water: Water should come from a clean source free from organic material and any other harmful substances.

9. Rammed Earth Mix Design Data: Soils shall be well mixed prior to ramming. Mixing by hand or mechanical mixer should continue until there is uniform distribution of materials with uniform color and consistency.

10. Stabilization: Stabilized materials may be added to rammed earth structures to improve strength, improve resistance against water, or achieve less shrinkage. Approved materials are:

   a. (ordinary Portland cement
   b. lime or hydrated lime
   c. lime combined with pozzolanas such as pulverized fuel ash and

11. Structural Steel


2. Specification for Structural Framing: Load & Resistance Factor Design

3. Joists: Specification for structural joists using ASTM A525 or A900 bolts

4. Structural Steel: All structural steel shall conform to the following:

   a. Structural Framing

   b. ASTM A920, F = 50 ksi Plates and Rolled Shapes
   c. ASTM A572 Type E or F, 65 ksi Structural Tubing
   d. ASTM A500 Grade B, F = 46 ksi and Connections

5. Connection Bolts: All A325 connection bolts shall be installed to the snug-tight condition per AISC specifications, in strict accordance with the manufacturer's published recommendations.

6. Welds: All welding shall be in conformance with AISC and AWS standards and shall be performed by licensed and certified welders using specified electrodes. Only prequalified welds (as defined by AWS) shall be used.

7. Reinforcing Steel

1. Reinforcing Steel:

   a. All bars unless indicated otherwise
   b. ASTM A615, Grade 60
   c. ASTM A416, Grade 75
   d. Bars, Excepting Ties, in Direct Driving Rebar: 60 ksi
   e. Deformed wire stirrups (D4 and larger only) ASTM A497

2. Use of Chlorides: Except for use in Grade 75 and larger stirrups, use of chlorides is not permitted.

3. Wire Reinforcing:

   a. Smooth wire: 4 diameter electrodes
   b. ASTM A618
   c. Deformed wire: 4 diameter electrodes (D and larger only)
   d. ASTM A416

4. Shop Drawings: ACI 315, Part B: Place plating such that splice locations and lengths and submit to Structural Engineer. Promptly notify Structural Engineer prior to proceeding with work if design requirement further clarifies.

5. Connections:

   a. Connection Bolts: All A325 connection bolts shall be installed to the snug-tight condition per AISC specifications, in strict accordance with the manufacturer's published recommendations.
   b. Welds: All welding shall be in conformance with AISC and AWS standards and shall be performed by licensed and certified welders using specified electrodes. Only prequalified welds (as defined by AWS) shall be used.

6. Minimum Footing Depths: 4 inches below grade adjacent or finish floor, whichever is lower.

7. Backfill of Retaining Walls: Place after completion and inspection of waterproofing. Adequately shore retaining walls during backfill operation. Unloading details do not place backfill behind building structure retaining walls (excluding site retaining walls) until concrete at all floor elevations adjacent to walls are completely poured (in area) and have cured for at least 7 days.

8. Minimum Clearances Between Parallel Reinforcing Steel Including Anchorage:


9. Minimum Footing Depths: 4 inches below grade adjacent or finish floor, whichever is lower.

10. Foundation Design Values:

   a. Bearing Capacity: 1,500 psi
   b. Lateral Bearing Resistance: 200 psi for isolated pile type footing.

11. Excavations, Backfill, and Compaction of Backfill:

   a. Excavations: AAI and Backfill:
   b. With looseness: 3 to 4, otherwise: 4 to 5
   c. Compaction: 95% for top 3 ft, 90% for deeper zones

12. Minimum Clearance Between Parallel Reinforcing Steel Including Anchorage:

   a. Distance Between Bars: 1-1/2 inch for 1 bar diameter, 3-inches for larger diameters

13. Soil Reports Unavailable for Project:

   a. Default soil conditions
   b. Shall be approved by Structural Engineer.

14. Reinforcing Steel:

   a. Structural Steel
   b. ASTM A615, Grade 60
   c. ASTM A416, Grade 75
   d. Bars, Excepting Ties, in Direct Driving Rebar: 60 ksi
   e. Deformed wire stirrups (D4 and larger only) ASTM A497

15. Soil Reports Unavailable for Project:

   a. Default soil conditions
   b. Shall be approved by Structural Engineer.

16. Minimum Clearance Between Parallel Reinforcing Steel Including Anchorage:

   a. Distance Between Bars: 1-1/2 inch for 1 bar diameter, 3-inches for larger diameters

17. Minimum Clearances Between Parallel Reinforcing Steel Including Anchorage:

   a. Distance Between Bars: 1-1/2 inch for 1 bar diameter, 3-inches for larger diameters

18. Minimum Clearances Between Parallel Reinforcing Steel Including Anchorage:

   a. Distance Between Bars: 1-1/2 inch for 1 bar diameter, 3-inches for larger diameters

19. Drainage System:

   a. As required by future geotechnical report. Retaining walls were not designed to resist future geotechnical report.

20. Dimensional Lumber:

   a. Framing:
   b. Joists:
   c. Structural Steel

   a. Structural Steel
   b. ASTM A615, Grade 60
   c. ASTM A416, Grade 75
   d. Bars, Excepting Ties, in Direct Driving Rebar: 60 ksi
   e. Deformed wire stirrups (D4 and larger only) ASTM A497

21. Construction Observation:

   a. Structural Engineer will perform a report for each significant stage of construction observed. Original of observation report will be sent to governing code authority and will be signed and sealed (seal stamped) by responsible Structural Engineer. One copy of observation report and all supporting data will be kept by the Structural Engineer (other than Structural Observer).

   b. Final Observation Report: Structural Observer will submit a report that shows that all observed deficiencies were resolved and structural system is complete as per construction documents.

   c. Earthfound and Works:

      a. Geotechnical Engineer: Retained by Owner and satisfactory to Structural Engineer and Governing Code Authority to perform required observations.

      b. Geotechnical Design:

         a. Selection for Geotechnical Design:
         b. Selection for Geotechnical Design:

     1. Geotechnical Engineer:
     2. Geotechnical Design:

     a. Geotechnical Design:
     b. Geotechnical Design:

   2. Foundation Design Values:

      a. Bearing Capacity: 1,500 psi
      b. Lateral Bearing Resistance:

   3. Foundation Design Values:

      a. Bearing Capacity: 1,500 psi
      b. Lateral Bearing Resistance:

   4. Excavations, Backfill, and Compaction of Backfill:

      a. Excavations: AAI and Backfill:
      b. With looseness: 3 to 4, otherwise: 4 to 5
      c. Compaction: 95% for top 3 ft, 90% for deeper zones

   5. Minimum Clearance Between Parallel Reinforcing Steel Including Anchorage:

      a. Distance Between Bars: 1-1/2 inch for 1 bar diameter, 3-inches for larger diameters

   6. Soil Reports Unavailable for Project:

      a. Default soil conditions
      b. Shall be approved by Structural Engineer.

   7. Reinforcing Steel:

      a. Structural Steel
      b. ASTM A615, Grade 60
      c. ASTM A416, Grade 75
      d. Bars, Excepting Ties, in Direct Driving Rebar: 60 ksi
      e. Deformed wire stirrups (D4 and larger only) ASTM A497

   8. Construction Observation:

      a. Structural Engineer will perform a report for each significant stage of construction observed. Original of observation report will be sent to governing code authority and will be signed and sealed (seal stamped) by responsible Structural Engineer. One copy of observation report and all supporting data will be kept by the Structural Engineer (other than Structural Observer).

      b. Final Observation Report: Structural Observer will submit a report that shows that all observed deficiencies were resolved and structural system is complete as per construction documents.
### DEVELOPMENT LENGTH AND SPLICE DETAILS

#### Typical Non-Seismic Straight Bar Development Length Schedule

<table>
<thead>
<tr>
<th>Size</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
<th>#8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size Diameter (in)</td>
<td>0.12</td>
<td>0.16</td>
<td>0.21</td>
<td>0.24</td>
<td>0.28</td>
<td>0.31</td>
<td>0.35</td>
</tr>
<tr>
<td>Length (in)</td>
<td>12</td>
<td>16</td>
<td>21</td>
<td>24</td>
<td>28</td>
<td>31</td>
<td>35</td>
</tr>
</tbody>
</table>

#### Typical Seismic Straight Bar Development Length Schedule

<table>
<thead>
<tr>
<th>Size</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>#6</th>
<th>#7</th>
<th>#8</th>
</tr>
</thead>
<tbody>
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<td>Size Diameter (in)</td>
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<td>0.21</td>
<td>0.24</td>
<td>0.28</td>
<td>0.31</td>
</tr>
<tr>
<td>Length (in)</td>
<td>0.5</td>
<td>0.6</td>
<td>0.7</td>
<td>0.75</td>
<td>0.8</td>
<td>0.9</td>
</tr>
</tbody>
</table>

#### Notes

1. All seismic bars shall be sized as shown to provide 1.5 times the non-seismic detail as required in Schedule C.2.
2. Use Schedule C2 shown in Schedule C.1 for bars in seismic environments only.
3. BARS TO BE DEFORMED AT BEND OR SPlice LOCATION.

### Typical Reinforcing Splice Details

#### Notes

1. Bars near bend or splice include bar to be faced for bend or splice location. Splice or bend bars are not included.
2. Bars included for bend or splice location are to be bent or spliced as shown.
3. Bars near bend or splice may be included for bend or splice location.
4. Bars near bend or splice may be included for bend or splice location.
SHEAR WALL REINFORCING PER ELEVATION

TOP OF WALL ELEVATION PER PLANS AND SHEAR WALL ELEVATION

DOWELS TO MATCH SIZE and SPACING OF VERTICAL REINFORCING

SHEAR DOWELS - PER PLAN LOCATE AT MID-DEPTH OF SLAB ON GRADE

2x4 CONTINUOUS SHAPED KEY

TOP OF FOOTING ELEVATION PER PLANS AND SHEAR WALL ELEVATION

#4 @ 12" E.F. (TYP.)

TOP OF WALL 12'-0"

#7 @ 16" (T) TRANSVERSE

(8) #5 (T) LONGITUDINAL

#7 @ 16" (B) TRANSVERSE

(8) #5 (B) LONGITUDINAL

3' - 0"

26' - 0" X 6' - 0"

20' - 0"

6' - 0"

3' - 0"

SHEAR DOWELS - PER PLAN
LOCATE AT MID-DEPTH OF SLAB ON GRADE
DOWELS TO MATCH SIZE and SPACING OF VERTICAL REINFORCING

TOP OF FOOTING ELEVATION PER PLANS AND SHEAR WALL ELEVATION

#4 @ 12" E.F. (TYP.)

#4 @ 12" o.c. (T) TRANSVERSE

(6) #5 (T) LONGITUDINAL

S.O.G.

TOP OF WALL 13'-0"

TOP OF WALL 13'-0"

3 FT.

3 FT.

S2.5

SHEAR WALL REINFORCING PER ELEVATION

16 FT. x 4 FT.

13 FT.

13 FT.

13' - 0"

13' - 0"

1' - 0"

2x4 CONTINUOUS SHAPED KEY

#6 @ 12" o.c. (B) TRANSVERSE

(6) #5 (B) LONGITUDINAL

#6 @ 12" o.c. (T) TRANSVERSE

#6 @ 12" E.F. (TYP.)

RAMMED EARTH WALL ELEVATION - GRID G
FOUNDATION NOTES
1. SEE GENERAL NOTES ON S1.1 FOR SPECIFICATIONS
2. REFER TO FOOTING SCHEDULE FOR SITE, THICKNESS, AND REINFORCING.
3. CENTER COLUMNS ON GRIDLINES UNLESS NOTED OTHERWISE.
4. SEE ARCHITECTURAL DRAWINGS FOR TOP OF CONCRETE SLAB ON GRADE
   ELEVATIONS, DEPRESSIONS, SLOPES, OPENINGS, CURBS, DRAINS, TRENCHES
   SLAB EDGE LOCATIONS, WALL OVERAL DIMENSIONS, AND LOCATIONS OF
   OPENINGS NOT INDICATED ON STRUCTURAL DRAWINGS.
5. PROVIDE CONSTRUCTION JOINTS & WEAKENED PLANE JOINTS IN SLABS ON GRADE.
   REFER TO S2.3 FOR APPROPRIATE CONTROL JOINT (C.J.) DETAIL.
6. INDICATES RAMMED EARTH SHEAR WALL
   REFER TO S2.4 & S2.5 FOR TYPICAL DETAILS
    INDICATES NONSTRUCTURAL WALLS
    INDICATES CONCRETE SPREAD FOOTING
    REFER TO X/S2.3 FOR TYPICAL DETAIL
    REFER TO S2.4 & S2.5 FOR SHEAR WALL FOUNDATIONS F2 & F3

FOOTING SCHEDULE

<table>
<thead>
<tr>
<th>MARK</th>
<th>SIZE</th>
<th>THK.</th>
<th>REINFORCING</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>4' X 0&quot;</td>
<td>2'</td>
<td>(5) #6 EACH WAY (B)</td>
<td>SEE DETAIL S2.4</td>
</tr>
<tr>
<td>F2</td>
<td>16' X 0&quot;</td>
<td>3'</td>
<td>SEE DETAIL S2.5</td>
<td></td>
</tr>
<tr>
<td>F3</td>
<td>3' X 0&quot;</td>
<td>3'</td>
<td>SEE DETAIL S2.5</td>
<td></td>
</tr>
</tbody>
</table>

NOTES:
1. USE 12" MIN SPACING FOR ALL REINFORCING IN ALL FOOTINGS.
2. REFER TO X/S2.3 FOR TYPICAL DETAIL.
3. REFER TO S2.4 & S2.5 FOR SHEAR WALL FOOTINGS F2 & F3
ROOF FRAMING PLAN NOTES:
1. SEE GENERAL NOTES ON S1.1 FOR SPECIFICATIONS
2. SEE ARCHITECTURAL DRAWINGS FOR TOP OF ROOF ELEVATIONS, WALL OVERALL
DIMENSIONS, AND LOCATIONS OF OPENINGS NOT INDICATED ON STRUCTURAL DRAWINGS.
3. CENTER COLUMNS ON GRIDLINES UNLESS NOTED OTHERWISE.
4. INDICATES RAMMED EARTH SHEAR WALL
REFER TO S2.4 & S2.5 FOR TYPICAL DETAILS
5. INDICATES NONSTRUCTURAL WALLS
Appendix C - Project Presentation Slides
Virunga: A Coffee/Cocoa Co-op

Journeyman International Engineering Team
Caleb Azevedo || Anugrah Gupta

Introduction

- Journeyman International Team
- Background Information
- Travel Experience
- Structural Design
- Challenges
- Conclusion

Journeyman International Team

- Humanitarian Partner: Empowering Villages
- Sponsor: Domum Architects
- Architecture Student: Dayna Lake
- Construction Management Student: Tanner Frkovich
- Architectural Engineering Students: Caleb Azevedo & Anugrah Gupta

Background Information
Rwandan Genocide of 1994

- Two groups: Tutsis & Hutus
- Ethnic strife based on social status
- Approximately 800,000 killed in a span of 100 days

Rwanda Today

Anugrah’s Travels

Anugrah’s Travels: Virunga Site-Visit
Project Description

- Community Center
- Library
- Education Center
- Warehouse
- Market
- Washing Station

Communication

- On-site representatives: Carly Althoff & Daniel Klinck
- Advisor: Al Estes
- Weekly meetings with architect

Design Criteria

- Materials:
  - Steel
  - Timber
  - Concrete
  - Rammed Earth
Rammed Earth Construction

Rammed Earth Advantages

- Temperature and noise control
- Durable and weather resistant
- Low maintenance
- Environmentally Friendly

Gravity System

Lateral System

- Plywood Diaphragm
- Reinforced Concrete Bond Beams
- Rammed Earth Shear Walls
- Reinforced Concrete Footings
Challenges

- Coordination with Architect
  - Lack of site information
  - Changes in footprint & material
- Aesthetics vs. Structural Efficiency
- Unfamiliar design in rammed earth shear walls

Future of Project

- Allocation of land
- Future site visits
- Possibility of construction

Conclusion

Questions?
Appendix D - Bibliography
Bibliography

Lichfield, John. “Guide to the Zaire crisis: The difference between a Hutu and Tutsi.”


