SENIOR PROJECT REPORT

FOR ARCE 453

Karambo Micro-Industrial Complex
Mahoko Village, Rwanda - Karambo River Masterplan

Prepared by:
Sophia Abshire & Stella Bates

Presented to the faculty of the Architectural Engineering Department at
California Polytechnic State University, San Luis Obispo

In fulfillment of the requirements for the degree of BS Architectural Engineering

June 2018
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1.0 Introduction

As architectural engineering students, we were first introduced to Journeyman International (J.I.) as freshmen and have been eager to be a part of J.I. since then. Journeyman International is a humanitarian non-profit organization in San Luis Obispo, California. J.I. gives students the opportunity to build a project start to finish with an interdisciplinary team of students for their senior thesis project. These projects range from orphanages to hospitals and are built all over the world for those in need. Each project includes at least one architecture student, construction management student and an architectural engineering student. Each team of students is paired with a client who is funding a project and expects a specific outcome.

Every year, J.I. takes applications from students to be a part of a team. After hearing about the successes of Journeyman International projects for the last three years, we were very motivated to complete a J.I. project for our senior project. Not only are J.I. projects great learning experiences and good design practice, they also help those in need. As architectural engineering students at California Polytechnic State University, San Luis Obispo we have gained a skillset that engineers in other countries are not fortunate enough to have and we both believe that this knowledge should be used for the benefit of those in need. Therefore, one of our goals for our senior project was to use that knowledge to benefit those who are not fortunate to have it. After applying this fall and explaining our motives to J.I., we were accepted and put on a project immediately. We were put on a team with Shea Menzel, a 5th year architecture student and Nick Somera, a 4th year construction management student. We were then assigned to a client, Empowering Villages, who is funding the development of the Karambo Micro-Industrial Complex.
2.0 Project Description

The Karambo Micro-Industrial Complex will be placed along the Karambo River in the Rubavu District of Western Rwanda which is 86.7 km from the capital, Kigali. The Karambo Micro-Industrial Complex consists of 3 one-story buildings which are all at different elevations and are located in a rural area along the Karambo River. For the sake of organization we have named each of these buildings “Building A” “Building B”, and “Building C” throughout the project.

2.1 Architectural Design

For the design of this project the architect considered some of the demographics in Rwanda. First that 42% of the people in Rwanda are below the age of 15, therefore their population consists of a lot of young children. Second, that 41% of the people in our projects area are illiterate. And third, that 62% of the population live below the poverty line and 37% of the population lives in extreme poverty. Based on these demographics, Journeyman International’s mission, and the site location, the goal was to gather the community around its industries. In doing so the architect wanted to help develop and build this area as well as empower the citizens who live there. Therefore the architect developed a design that would be dedicated towards their industries and another towards learning.
Building A will be used as an industrial work space for the adults of this village. It will consist of an open area covered by steel decking in addition to an indoor work area. Buildings B and C are mainly for the young children of this village to learn. Building B will be used as a library and will have covered outdoor stairs to the right of it to be used as a shaded outdoor sitting area and as access to Building C, which will be used as a classroom.

The architect also took into consideration that the weather in this area is typically very hot and therefore lifted the trusses above the walls allow for airflow as seen below.
2.1 Gravity System

Steel decking acts as the roof that connects a system of steel trusses together. The truss are supported by steel columns. For the outdoor open areas the columns are wide flanges and are extended down to a reinforced concrete retaining wall or to the foundation. For the steel trusses above the confined masonry walls, the trusses are supported by square HSS columns. The HSS columns are supported by the concrete tie-columns which transfer the load into the load-bearing masonry walls through a “toothing” connection. The reasoning behind the selection of column sections in each location was due to the architect’s preference and because both HSS and Wide Flange sections can easily connect to.

![Diagram of gravity system]

2.2 Lateral System

The steel decking acts as a flexible diaphragm. The loads from the diaphragm are transferred through braces to the foundation or to the confined masonry walls which is supported by a continuous strip footing. Due to the flexible diaphragm, concrete bond beams are designed in each direction to resist out of plane loads.

![Diagram of lateral system]

2.3 Foundation System

For the foundation below the confined masonry walls a continuous footing is used to take both the gravity and lateral loads. For the design of the footing below the walls, a plinth band is needed to confine the bottom of the confined masonry wall and connect it seamlessly to the footing. For the foundation below the steel columns, a reinforced concrete pad footing is used to take both the lateral and gravity loads.

![Diagram of foundation system]
3.0 Design Criteria

3.1 Codes

For the majority of this project we decided to follow 2015 IBC as well as ASCE 7-10, ACI 318-14, AISC 370-10 and TMS 402-13 & TMS 602-13. However, for the design of the confined masonry walls, the Seismic Design Guide for Low-Rise Confined Masonry Buildings which was developed by A Project of the World Housing Encyclopedia, EERI, & IAEE was used.

3.2 Material Properties

For this project we used a variety of materials; steel, concrete, and masonry. The table below demonstrated the design properties we used and where we got these values from. We also made sure that materials with these properties are available in Rwanda.

<table>
<thead>
<tr>
<th>Material/Shape</th>
<th>Design Property</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>$f'_c = 3$ ksi</td>
<td>Seismic Design Guide for Low-Rise Confined Masonry Buildings</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>$f_y = 60$ ksi</td>
<td>AISC 370-10</td>
</tr>
<tr>
<td>Masonry</td>
<td>$f'_c = 0.58$ ksi</td>
<td>Seismic Design Guide for Low-Rise Confined Masonry Buildings</td>
</tr>
<tr>
<td>Steel Square HSS</td>
<td>$f_y = 46$ ksi</td>
<td>AISC 370-10</td>
</tr>
<tr>
<td>Steel Round HSS</td>
<td>$f_y = 42$ ksi</td>
<td>AISC 370-10</td>
</tr>
<tr>
<td>Steel Wide Flange</td>
<td>$f_y = 50$ ksi</td>
<td>AISC 370-10</td>
</tr>
<tr>
<td>Steel Angle/Double Angle</td>
<td>$f_y = 36$ ksi</td>
<td>AISC 370-10</td>
</tr>
</tbody>
</table>
3.3 Soil Information

A soils report was produced from Empowering Villages for a nearby project and it was recommended for us to use the same soil samples for design (see Appendix A.3). From that report the following information was found:

0.0-0.2m : Loam Soil

0.2-0.4m: Compacted Clay Soil

0.4-1.2m: Sandstone

![Table 1806.2](image)

Based on the above information, the values below were found:

- Allowable Vertical Foundation Pressure = 2,000 psf
- Allowable Lateral Bearing Pressure = 150 psf/ft below natural grade
- Coefficient of Friction = 0.25
3.4 Gravity Loads

The dead loads used were derived from numerous sources including Rwandan steel manufacturers, AISC 370-10, ACI 381-14, and VERCO decking catalog.

The live loads used came from ASCE 7-10 Chapter 4. Because each building has a different purpose the live loads used on each building was different. For Building A, a live load of 120 psf was used and for buildings B&C a live load of 100 was used. Additionally, a roof live load of 20 psf was used on all buildings.

3.5 Seismic Analysis

The seismic design of the confined masonry walls is based off of the Seismic Design Guide for Low-Rise Confined Masonry Buildings which was developed by A Project of the World Housing Encyclopedia, EERI, & IAEE. Per the definitions of this guide, our site was considered to be in a moderate seismic zone and required certain design checks. This guide gave requirement for the size of members, amount of reinforcement, wall lengths, and out of plane resistance. The only requirement that was difficult to meet was out of plane resistance for the confined masonry. However, one bond beam was designed at the top of each wall to take the out of plane loads.

The seismic analysis of each building were based off of ASCE 7-10 section 12 and the following values were used:

- Risk Category: II
- Importance Factor: 1.0
- Spectral Response Accelerations
  (see Appendix A.2 for the article we obtained these values from))
  \[ S_S = 0.28 \quad S_1 = 0.11 \]
  \[ S_{DS} = 0.19 \quad S_{D1} = 0.07 \]
- Site Classification: B
- Seismic Design Category: B
- R= 4
- Cs= 0.0475
- Max Base Shear, V= 8.1 kips
3.6 Wind Analysis

The wind analysis was based off of ASCE 7-10 sections 26-30. Because each building had slightly different properties, they each had different parameters for the wind analysis. Below is an example of the wind analysis done on building A.

<table>
<thead>
<tr>
<th>WIND ANALYSIS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Building A</strong></td>
</tr>
<tr>
<td>Risk Category</td>
</tr>
<tr>
<td>Wind Speed, V</td>
</tr>
</tbody>
</table>

**Wind Load Parameters:**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Kd</td>
<td>0.85</td>
</tr>
<tr>
<td>Exposure Category</td>
<td>B</td>
</tr>
<tr>
<td>Kzt</td>
<td>1.00</td>
</tr>
<tr>
<td>G</td>
<td>0.85</td>
</tr>
<tr>
<td>Enclosure Classification</td>
<td>Partially Enclosed Building</td>
</tr>
<tr>
<td>GCpi</td>
<td>-0.55</td>
</tr>
<tr>
<td>Kz (h=4.00 m)</td>
<td>0.57</td>
</tr>
<tr>
<td>Kz (h=4.00 m)</td>
<td>0.57</td>
</tr>
<tr>
<td>qz</td>
<td>15.01</td>
</tr>
<tr>
<td>qh</td>
<td>15.01</td>
</tr>
<tr>
<td>E/W - L/B</td>
<td>1.00</td>
</tr>
<tr>
<td>Windward, Cp</td>
<td>0.8 (use with qz)</td>
</tr>
<tr>
<td>Leeward, Cp</td>
<td>-0.50 (use with qh)</td>
</tr>
<tr>
<td>N/S - L/B</td>
<td>1.00</td>
</tr>
<tr>
<td>Windward, Cp</td>
<td>0.8 (use with qz)</td>
</tr>
<tr>
<td>Leeward, Cp</td>
<td>-0.5 (use with qh)</td>
</tr>
</tbody>
</table>

*p= 18.46 psf

**Wall Elevation SF (E/W)=** 358.8 sf

**Wall Elevation SF (N/S)=** 323.8 sf

| Force from wind (E/W) | 6.62 k |
| Force from wind (N/S) | 5.98 k |

**NOTES:**

**Value taken as the minimum wind speed in the USA**

**Worst case wind load due to windward side and full building height**

Although each building had different parameters, because we took the worst case wind load on the windward side, each building had a maximum wind pressure was 18.46 psf.
4.0 Structural Design

Throughout the design of this project there were many aspects of design we have never been exposed to. First the design of a truss. For all three buildings two different truss designs were required. After doing research it was found that double angle steel trusses are available in Rwanda. To start the design the truss was modeled in ETABS. Once the demands of the truss were found, members were selected that met the demand. At first it was difficult to find a top chord because the unbraced length was too long for the desired member. In order to solve this issue, a steel beam was added at the mid-span of the top chord. An example of the ETABS output and truss design is below.

Axial:

Shear:

Moment:

Design:

Chords: 2L2½ x 2½ x ¾

Web Members: 2L2½ x 2½ x ¼
Once the trusses were designed, the square HSS columns were designed and the compressive strength of the concrete tie columns and masonry were checked in order to take the load from the columns (Load Path 1). Along with the HSS columns, wide flange columns were designed to extend from the foundation up to the truss where the truss was not over the confined masonry walls (Load Path 2).

For the lateral design of the building the lateral demand on each building varied between wind and seismic depending on the overall weight, wall length, and direction of loading. Once it was determined what load governed in each direction, the chords and collectors were designed. For each building the beams between each of the trusses and the trusses themselves act as the chords and collectors.

The next step in the design was to design the braces. After doing research on available materials in Rwanda we decided to use small round HSS sections. Our design transferred the lateral load to the ground in two different paths. Load Path 1 transferred the load from diaphragm to the trusses and then directly to the braces. Load Path 2 transferred the load in from the collectors to the braces and then to the confined masonry walls.
For the design of the confined masonry walls, little knowledge was known on how the design worked. After researching the Seismic Design Guide for Low-Rise Confined Masonry Buildings heavily, we felt comfortable with the design of the walls. However, because the design includes a flexible diaphragm, out of plane effects had to be taken into account.

Due to the seismic zone our site is located in, the design guide required the walls to be braced or to make the walls shorter, which became an issue. In order to address this issue, a concrete bond beam was designed at the top of each masonry wall as well as lintel and sill bands around all of the openings to resist out of plane bending.

After both the lateral and gravity designs, the foundation was designed. A continuous footing with a plinth band was used to support the confined masonry walls for both gravity and lateral loading. Additionally, pad footings were designed to support the wide flange steel columns.
5.0 Conclusion

Overall, this project was a great learning experience. Throughout our classes at Cal Poly we have learned to design masonry, steel, timber, and concrete buildings. However, we have never had to design buildings with a wide variety of materials. The Karambo Micro-Industrial Complex is a project that included masonry, concrete, and steel. Additionally, confined masonry is not something that is taught at Cal Poly because it is not built in the US so we had to research and teach ourselves how to design confined masonry. In our classes at Cal Poly we have analyzed trusses but we have never had to design one and after this project we feel confident in our ability to do so.

Though we learned a lot though the design aspect of this project we also had to put more effort into the analysis. Due to the location of this project, we were required to work with our team to do research on available materials and their properties since it is so different from the US where just about anything is available. Additionally, we were required to research the seismic, wind, and soil properties of this site because there is no equivalent of ASCE 7-10 or USGS type sources for Rwanda.

Not only did we benefit from this project because of how much we learned but we are also extremely excited about our contribution to this community. We are so grateful that we have had the opportunity to work with Journeyman International and provide a safe and usable building for the Rubavu District of Rwanda.
A.0 Appendix
A.1 Appendix 1

Excerpt From VERCO Decking Catalog
Deep VERCOR™ Deck

- 1\(\frac{5}{16}\)" Deep Deck
- Galvanized

Dimensions

- Standard Overlapping Sidelap

Deck Weight and Section Properties

<table>
<thead>
<tr>
<th>Gage</th>
<th>Weight (psf)</th>
<th>(I_d) for Deflection</th>
<th>Moment</th>
<th>Allowable Reactions per ft of Width (lb)</th>
<th>One Flange Loading</th>
<th>Two Flange Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Galv</td>
<td>Single Span (in.(^3)/ft)</td>
<td>Multi Span (in.(^3)/ft)</td>
<td>(+S_{eff})</td>
<td>(-S_{eff})</td>
<td>End Bearing Length</td>
</tr>
<tr>
<td>26</td>
<td>1.1</td>
<td>0.075</td>
<td>0.075</td>
<td>0.099</td>
<td>0.103</td>
<td>492</td>
</tr>
<tr>
<td>24</td>
<td>1.4</td>
<td>0.097</td>
<td>0.097</td>
<td>0.137</td>
<td>0.138</td>
<td>802</td>
</tr>
<tr>
<td>22</td>
<td>1.7</td>
<td>0.120</td>
<td>0.120</td>
<td>0.172</td>
<td>0.171</td>
<td>1184</td>
</tr>
<tr>
<td>20</td>
<td>2.1</td>
<td>0.143</td>
<td>0.143</td>
<td>0.204</td>
<td>0.204</td>
<td>1628</td>
</tr>
</tbody>
</table>

Notes:
1. Section properties are based on \(F_y = 60,000\) psi (specified minimum \(F_y = 80,000\) psi).
2. \(I_d\) is for deflection due to uniform loads.
3. \(S_{eff}\) (+ or -) is the effective section modulus.
4. Allowable (ASD) reactions are based on web crippling, per AISI S100 Section C3.4, where \(\Omega_w = 1.70\) for end bearing and 1.75 for interior bearing. Nominal reactions may be determined by multiplying the table values by \(\Phi_w\) LRFD reactions may be determined by multiplying nominal reactions by \(\Phi_w = 0.90\) for end reactions and 0.85 for interior reactions.

Attachment Patterns to Supports

- 36/4 Inverted Position
- 36/5 Normal Position
- 36/8 Inverted Position
- 36/9 Normal Position
Footnotes for Allowable Uniform Load Tables

2. L/360, L/240 or L/180 = Uniform load which produces selected deflection in deck.
3. The symbol * indicates allowable uniform load based on deflection exceeds allowable uniform load based on stress.
4. Nominal uniform loads governed by stress may be determined by multiplying the allowable values in the table by $\Omega_b = 1.67$.
   LRFD loads may be determined by multiplying nominal loads by $\Phi_b = 0.95$.

Footnotes for Diaphragm Shear Strength and Flexibility Factor Tables

**General Notes**

1. #10 = #10 Generic Screw. Sidetap connections are not required at support locations.
2. The dimension from the first and last sidetap connection within each span is to be no more than one-half of specified spacing.
3. $R$ is the ratio of vertical span ($L_v$) of the deck to the length ($L_s$) of the deck sheet: $R = L_v / L_s$.
4. Interpolation of diaphragm shear strength between adjacent spans or sidetap spacings is permissible. For interpolation of the diaphragm flexibility factor between adjacent spans, use the flexibility factor for the closest adjacent span length.
5. Diaphragm shear values for side seam fasteners placed at spacings other than those in the table should be determined based on the number of fasteners in each span.
6. The allowable diaphragm shear values in the tables utilize a factor of safety, $\Omega = 2.5$ (limited by connections) with the exception of the gray shaded table values, which utilize a factor of safety of $\Omega = 2.0$ (limited by panel buckling).
7. Deck is attached with minimum #12 Screws (self drilling, self tapping) to supports. Select appropriate screw based on actual substrate thickness. This table is provided as a guide, proper selection should be verified based on the specific fasteners used.

<table>
<thead>
<tr>
<th>Support Thickness</th>
<th>Fastener Designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>33 mil (0.0346&quot;) to 3/16&quot;</td>
<td>#3 Drill Point</td>
</tr>
<tr>
<td>1/8&quot; to 1/4&quot;</td>
<td>#4 Drill Point</td>
</tr>
<tr>
<td>1/8&quot; to 1/2&quot;</td>
<td>#5 Drill Point</td>
</tr>
</tbody>
</table>

8. All tabulated diaphragm values shown in this section are for a minimum 0.0385 in. thick support with SDI recognized screws produced by Buildex, Elco, Hilli or Simpson Strong-Tie. If the minimum support thickness can not be met or a screw that is not recognized by SDI is used, modify tabulated $q$ and $F$ values based on actual substrate and thickness using Adjustment Factors listed in the following tables.

For 9/16" (Shallow) VERCOR:

<table>
<thead>
<tr>
<th>Deck Gage</th>
<th>Factors</th>
<th>20 ga</th>
<th>18 ga</th>
<th>16 ga</th>
<th>14 ga</th>
<th>≥ 12 ga</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>33 mil (0.0345 in)</td>
<td>43 mil (0.0451 in)</td>
<td>54 mil (0.0566 in)</td>
<td>68 mil (0.0713 in)</td>
<td>≥ 79 mil (0.1017 in)</td>
</tr>
<tr>
<td></td>
<td>$q$</td>
<td>$R_q$</td>
<td>$R_F$</td>
<td>$q$</td>
<td>$R_q$</td>
<td>$R_F$</td>
</tr>
<tr>
<td>26</td>
<td>$R_q$</td>
<td>0.66</td>
<td>0.69</td>
<td>0.69</td>
<td>0.69</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td>$R_F$</td>
<td>1.26</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>24</td>
<td>$R_q$</td>
<td>0.52</td>
<td>0.66</td>
<td>0.68</td>
<td>0.69</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td>$R_F$</td>
<td>1.51</td>
<td>1.38</td>
<td>1.37</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>22</td>
<td>$R_q$</td>
<td>0.38</td>
<td>0.54</td>
<td>0.59</td>
<td>0.69</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td>$R_F$</td>
<td>1.69</td>
<td>1.58</td>
<td>1.36</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

For 1-5/16" (Deep) VERCOR:

<table>
<thead>
<tr>
<th>Deck Gage</th>
<th>Factors</th>
<th>20 ga</th>
<th>18 ga</th>
<th>16 ga</th>
<th>14 ga</th>
<th>≥ 12 ga</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>33 mil (0.0345 in)</td>
<td>43 mil (0.0451 in)</td>
<td>54 mil (0.0566 in)</td>
<td>68 mil (0.0713 in)</td>
<td>≥ 79 mil (0.1017 in)</td>
</tr>
<tr>
<td></td>
<td>$q$</td>
<td>$R_q$</td>
<td>$R_F$</td>
<td>$q$</td>
<td>$R_q$</td>
<td>$R_F$</td>
</tr>
<tr>
<td>26</td>
<td>$R_q$</td>
<td>0.69</td>
<td>0.74</td>
<td>0.74</td>
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<td>0.74</td>
</tr>
<tr>
<td></td>
<td>$R_F$</td>
<td>1.13</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>24</td>
<td>$R_q$</td>
<td>0.58</td>
<td>0.70</td>
<td>0.73</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>$R_F$</td>
<td>1.21</td>
<td>1.17</td>
<td>1.13</td>
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</tr>
<tr>
<td>22</td>
<td>$R_q$</td>
<td>0.48</td>
<td>0.61</td>
<td>0.65</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>$R_F$</td>
<td>1.27</td>
<td>1.24</td>
<td>1.24</td>
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<td>1.00</td>
</tr>
<tr>
<td>20</td>
<td>$R_q$</td>
<td>0.39</td>
<td>0.53</td>
<td>0.57</td>
<td>0.71</td>
<td>0.71</td>
</tr>
<tr>
<td></td>
<td>$R_F$</td>
<td>1.32</td>
<td>1.33</td>
<td>1.25</td>
<td>1.21</td>
<td>1.16</td>
</tr>
</tbody>
</table>

9. Adjustment factors are based on connection strengths determined in accordance with Section E4 of AISI S100. These self drilling, self tapping screws must be compliant with ASTM C1315.

10. Allowable Diaphragm Strength = $q \cdot R_q$. Flexibility Factor = $F \cdot R_F$.

11. These adjustment factors are based on the maximum adjustment for the tabulated span lengths and fastener patterns.

To calculate a specific condition, use design equations listed at the end of Evaluation Report ER-0217.
A.2 Appendix 2

Article: Comparative Analysis of Seismic Loading on High-Rise Steel Building Structures in Bujumbura and Kigali Cities
COMPARATIVE ANALYSIS OF SEISMIC LOADING ON HIGH-RISE STEEL BUILDING STRUCTURES IN BUJUMBURA AND KIGALI CITIES

NDIHOKUBWAYO Athanase ¹, JIANG Cangru ² and CHEN Zhihua ³

¹ Doctorate Student, ² Professor, ³ Asso.Professor,
Department of Civil Engineering and Architecture,
Wuhan University of Technology, Wuhan, China
E-Mail: athanand@yahoo.com

ABSTRACT:
The seismic loading analysis is very important for designing high-rise buildings in the most seismic ground motions zone. The objective of this study was to compare some seismic loading forces on two similar high-rise building structures located in Bujumbura city (Capital of Burundi) and Kigali city (Capital of Rwanda); using ETABS software. These two cities are located in the Western side of the East African rift system; known as the African most seismic ground motions zone. In the present study, all the seismic design data were the same for the two buildings except their seismic site spectral response acceleration Sₛ and S₁. The result of the study showed that the average of the seismic base shear forces and the seismic stories lateral forces on the building located in Bujumbura city is ranged between 2.3 and 2.4 times greater than the average of these seismic loading forces on the building located in Kigali city.

KEYWORDS: East African rift system, Sₛ and S₁, high-rise steel building, seismic base shear forces, seismic stories lateral forces.

1. INTRODUCTION

The East African region is often shaken by the earthquake principally because of the presence of the rift valley. The earthquake in that region which covers about 5.5 million km² and holds more than 120 million people has been identified as the major threat.¹ Thus, with major population growth and urbanization increasing, the vulnerability to the earthquake hazards has greatly increased.² For facing simultaneously to the urbanization target and the earthquake threat, it is very important to construct the high-rise buildings with high seismic design consideration. The vulnerability of the East African populations to seismic events has been underscored by a study which advised that the region’s capacity in earthquake preparedness and hazards mitigation need to be improved significantly.³ In the present study the seismic base shear forces and the seismic stories lateral forces acting on each building structures (20 stories) were calculated and a comparison analysis of these forces was done for the two buildings.
As the seismic spectral response acceleration (Ss and S1) are known for the two building construction sites, the International Building Code [4] and other design references was helpful to determine the input data for the seismic loading design.

The Extended Three Dimensional Analysis of Building Systems (ETABS) software [5] was used to perform automatically the seismic loading design process. Bujumbura city which has high values of seismic spectral response acceleration than Kigali city is the sixth African city with high seismic spectral response acceleration Ss and S1 after Alger (Algeria), Tunis (Tunisia), Djibouti (Djibouti), Bukavu (Congo Democratic) and Cairo (Egypt) [6]. These factors are very decisive for seismic building design process on any given construction site. The result of the seismic loading comparison showed that the average of the seismic loading base shear forces and seismic stories lateral forces on building located in Bujumbura city is ranged between 2.3 and 2.4 times greater than the average of the seismic loading base shear forces and seismic stories lateral forces on building located in Kigali city.

Table 1  Seismic loading design data

<table>
<thead>
<tr>
<th>Seismic design characteristics</th>
<th>Bujumbura Building</th>
<th>Kigali Building</th>
</tr>
</thead>
<tbody>
<tr>
<td>Occupancy Category and Seismic Use Group, SUG [7]</td>
<td>I</td>
<td>I</td>
</tr>
<tr>
<td>Seismic Importance Factor, I [8]</td>
<td>1</td>
<td>1</td>
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<tr>
<td>Seismic Site Class [9]</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>Mapped 0.2 sec. Period Spectral Acceleration, Ss [10]</td>
<td>0.66</td>
<td>0.28</td>
</tr>
<tr>
<td>Mapped 1.0 sec. Period Spectral Acceleration, S1 [11]</td>
<td>0.26</td>
<td>0.11</td>
</tr>
<tr>
<td>Acceleration-based Site Coefficient, Fa [12]</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Velocity-based Site Coefficient, F1 [13]</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Maximum Spectral Response Acceleration, SMS [14]</td>
<td>SMS = Fa x Ss = 0.66</td>
<td>SMS = Fa x Ss = 0.28</td>
</tr>
<tr>
<td>Maximum Spectral Response Acceleration, SM1 [15]</td>
<td>SM1 = F1 x S1 = 0.26</td>
<td>SM1 = F1 x S1 = 0.11</td>
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<tr>
<td>Design Spectral Response Acceleration, SDS [16]</td>
<td>SDS = 2/3 SMS = 0.44</td>
<td>SDS = 2/3 SMS = 0.19</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration, SD1 [17]</td>
<td>SD1 = 2/3 SM1 = 0.17</td>
<td>SD1 = 2/3 SMS = 0.07</td>
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<tr>
<td>Seismic Design Category, SDC [18]</td>
<td>SDC = C (SUG) = I</td>
<td>SDC = B (SUG = I)</td>
</tr>
<tr>
<td>Seismic Response Modifier, R [19]</td>
<td>R = 8</td>
<td>R = 8</td>
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</table>

2. DESIGN SPECTRAL RESPONSE ACCELERATION FOR THE TWO CITIES

On the Fig.1 below, the response times T0 and Ts needed for constructing response spectra curves for the two cities are calculated as follows:

\[
T_0 = 0.2 \frac{SD1}{SDS}; \quad T_s = \frac{SD1}{SDS} \quad (2.1)
\]

where T0 is the short period ground motion range and Ts is the characteristic period of ground motion.
For the building located in Bujumbura city: $T_0 = 0.08$ ; $T_{s-B} = 0.39$ ; $SD-B = 0.17$ ; $SDS-B = 0.44$
For building located in Kigali city: $T_0 = 0.08$ ; $T_{s-K} = 0.37$ ; $SD1-K = 0.07$ ; $SDS-K = 0.19$

3. SEISMIC BASE SHEAR FORCES (V)

The seismic base shear forces is given by the following equation:

$$V = Cs \times W$$ \hspace{1cm} (3.1)

where $Cs$ is the seismic design coefficient and $W$ is the building reactive weight including cladding.

$$Cs = \frac{SDS}{(R/I)} \leq \frac{SD1}{(TR/I)} \geq 0.044 \times SDS \times I$$ \hspace{1cm} (3.2)

The fundamental period $T$ and the Seismic Response Modifier $R$ values are respectively equal to 2 and 8.

For building located in Bujumbura city, $Cs = 0.044 \times 0.44 \times 1 = 0.01936$.
For building located in Kigali city, $Cs = 0.044 \times 0.19 \times 1 = 0.00836$.

Fig. 2 Seismic Base Shear Forces Result
4. SEISMIC STORIES LATERAL FORCES (Fx)

The equation for the seismic stories lateral forces calculation is:

\[ F_x = C_{vx} \times V \quad \text{[23]} \]

where: \( C_{vx} \) is the vertical distribution factor and \( V \) is the shear forces

\[ C_{vx} = \left[ \frac{W_x (h_x)^k}{\sum_{i=1}^{n} W_i (h_i)^k} \right] \quad \text{[24]} \]

where: \( W_i \) and \( W_x \) are respectively portion of the total gravity load \( W \) assigned to the level \( i \) and \( x \); \( h_i \) and \( h_x \) are respectively the height from the base to level \( i \) and \( x \); \( k \) is the exponent related to the building period \( T \) and takes into account the whiplash effects in tall slender buildings.

\[ T = C_1 (h_x)^{3/4} \quad \text{[25]} \]

where: \( h_x \) is the total height (in feet) of the building and \( C_1 \) is a period coefficient.

\[ k = (2-1) \times (T-0.5) + 1 \quad \text{(for 0.5 sec < k < 2.5 sec)} \]
\[ \quad \text{[2.5 – 0.5]} \]

The value of \( k \) is equal to 1.75 for the two buildings.

5. CONCLUSION

After analysis and calculation of the seismic base shear and the seismic stories lateral forces acting on the two buildings as illustrated on the figures 2 and 3, we realized that the average of these loading forces on building located in Bujumbura city is ranged between 2.3 and 2.4 times greater than the average of the same loading forces acting on the building located in Kigali city. The highest seismic loading forces for building located in Bujumbura city is given especially by the highest values of its site seismic spectral response acceleration \( S_s \) and \( S_1 \) comparatively to \( S_s \) and \( S_1 \) for building site located in Kigali city.
The earthquake shakes in the region had already caused humans and animals life losses and a lot of infrastructure injuries. Thus, the decision makers in matters of political policies, engineering and planning professionals need to understand the nature of the hazardous phenomena and take all preventive measures against the earthquake threat. The decision can be taken at three levels of commitment to implement mitigation and preparedness. These three levels of commitment are the development knowledge, public awareness raising and education, preparedness investments. High-rise building construction can be a sustainable solution for development in the countries that are over-populated and the system can also create sufficient space in urban area for other development infrastructures. More researches are needed to enrich this study in order to contribute to the urban development without earthquake threat in East African rift system.

REFERENCES

[6] UFC3-310-01. (2005). Earthquake loading data at additional locations outside of the United States, its territories and possessions (Tab. E-1), USA
[10] International Building Code. (2003). Mapped 0.2 Sec Period Spectral Acceleration, Ss (Fig. 16151), Berkeley, USA.
[20] ASCE7, Section 9.5.5.3. (2002). Fundamental period of structural. New York, USA.
[21] ASCE7, Section 9.5.5.2 (2002). Seismic Base Shear. New York, USA.
[23] ASCE7, Section 9.5.5.4 (2002). Lateral Force at any level. New York, USA.
[25] ASCE7, Section 9.5.5.3.2. (2002). Fundamental Period. New York, USA.
A.3 Appendix 3

Soils Report on for Neighboring Project
FEASIBILITY STUDY
Vol III Geological Report
Karambo II Hydropower Plant (HPP)

Publication & Revision Tables
Date of Final Publication: April, 2015

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<td>Adina Paraschivescu Enterprise</td>
<td>+250 783 420 423</td>
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SHPP Feasibility Study Report Karambo II HPP / Vol III Geological Report
AE-002-25032015-v1 1
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1. INTRODUCTION

In order to assess the relevant geological conditions prevailing at the sites of all major structures of Karambo II HPP, the consultant has carried out field investigations that involve mapping of the project area and logging of test pits.

The findings have a bearing on the feasibility of this proposed hydro power project with regard to structural aspects, competence of rock and soil formations as well as availability of materials for construction.

This project is located a few kilometers west of the proposed Bihongora hydro power project also in Kanama sector, Rubavu district. The landscape at the two project sites is somewhat similar with undulating hills and valleys that are a perfect setting for creation of sufficient water head for power generation.

The consultant has conducted field investigations that involve mapping of the project area and logging of test pits so as to discern the geological conditions prevailing at the project site area.

2. REGIONAL GEOLOGY

The geology of Rwanda has formations of mainly sedimentary and metamorphic rocks that consist of granite, migmatites, gneisses and mica schists of the paleoproterozoic Ruzizian basement overlain by the Mesoproterozoic Kibaran belt formations of the Burundian sub-group. These rock formations can be classified as either politic comprising of mainly shales, schists and para-gneisses; or Arenaceous comprising of conglomerates, sandstones and quartzites.

Magmatic rocks which are mainly granites and gneisses include mafic plutonic rocks and basalts. In terms of abundance, granites and gneisses are the first followed by mafic plutonic rocks and basalts are the last. Granites are mainly gneissic and are observed in the south, central and partly towards the west and are occasionally foliated with some meta-sediments. Mafic plutonic rocks of significant extent are confined in the central part while small granitic bodies are observed near these plutonic mafic rocks and are outcropping throughout the country.

Generally, in the east the country is predominated by older granite gneisses while neogene volcanics are found in the north western and south western parts. Young alluvial deposits and lake sediments are a common occurrence along rivers. Basaltic flows of significant thicknesses are observed in the south-west towards Lake Kivu and at the DR Congo and Burundi borders. They are considered to originate from south Kivu volcanic province in DR Congo. Also, thin inter layers of mafic and acidic lavas are observed within the typical Burundian meta-sediments.

Faulting sequences observed in the field and as per the main features of the broad landscape in terms of morphology of Rwanda has led to formations of conglomerates, sandstones, shales in the east while in the west the rift valley formation has led to the creation of successive host and graben systems where Lake Kivu is and southwards.
3. GEOLOGY OF KARAMBO II HPP AREA

3.1 Topography

*Landscape*

The landscape around the proposed hydro power project is hilly and undulating with the river flowing in the valley section. This reflects past tectonic earth movements related to rift valley formation. The river has cut a gorge in the valley about 1.5m deep. The left and right banks of the river stand at about the same height at the proposed intake point but the right bank stands progressively higher in the desander section (from intake to forebay).

Pitting findings at the proposed power house zone indicate presence of an intermittent flood plain where the river breaks its banks and pours out silt, sand and pebbles onto the surrounding land during heavy rain season.

![River landscape](image)

*Figure 1 Land around the proposed intake dam*
3.2 Vegetation Cover

Green vegetation occurs on the river banks mostly short to medium size grass and shrubs. Beyond the river banks is forest cover, mostly eucalyptus trees, on the right hand side bank and short to medium size grass and scattered trees on the left hand side bank. This vegetation cover limits erosion of soil into the river. However, around the proposed power house zone there is scattered vegetation cover on the left hand side river bank exposing soil that keeps eroding into the river. That’s why water at this river point is not clear but brown in colour, unlike the water at the intake which is clear.

![Image of vegetation cover on river banks]

*Figure 2 Vegetation cover on the river banks around the proposed intake dam*
3.3 Rocks and Soils

Large boulders of basalt rock and granite metamorphosing into gneisses lie at the river base. Occurrence of these igneous rocks is indicative of past volcanic activity in this zone. Quartzites and mica schist are present but on a small scale. Pebbles, silt and sandy material line the base of the river and migmatites predominantly form the sides and bottom of the river.

At the proposed intake point, water is clear possibly due to filtration by these materials lining the river base. However, the water is of brown colour at the proposed power house point because of the exposed reddish brown soil (limonite) on the left hand side river bank and gray to brown clay soils on the right hand side river bank. At the proposed intake dam, there is a fault structure in the rock formations on the right hand side bank that allows water from the hill to flow underground into the river (see test pit no. 2 log).

![Image of rock boulders at power house zone](image)

*Figure 3 Rock boulders at power house zone*
3.4 Site Test Pits
Six pits of mostly 1.2m depth were dug to reveal the soil and rock formations underground. Logging of these holes was done and snap shots taken.
Findings are presented in the appendices.

3.5 Construction materials
Near the project site on the road side are scattered micro quarries producing stone hard core and hand-crushed stone aggregate of mainly quartzite material. These stones can be used in the construction of intake dam and associated desilting plant, water canal, forebay for buffer water stock and power house.
Also accessible, not far away from project site, are volcanic rocks that are perfect for lining the canal.
Sand is available in and around the project area in the river.

4. CONCLUSION
The water supply to the proposed intake dam is sustainable as it originates from two independent sources i.e Karambo River itself and the underground stream joining this river from the right hand side bank. So, faulting in the rock structure that created this stream is of advantage to this proposed project.
The water at the intake point is clear requiring no filtration but obviously a desilting plant is still a necessity.
The geomorphology at the right hand side bank offers better potential for realizing sufficient water head for the hydro power project.
Construction materials, sand and stone aggregate, are available close to the project site. This project site is easier to access than the proposed Bihongora HPP site.

5. RECOMMENDATION
The consultant recommends setting up the hydro power plant on the right hand side of the river flow. This includes the intake dam, desilting plant, water conveyance system, forebay and power house.
When constructing the foundation of the power house, it must be taken into account that it is positioned in a seasonal flood plain.
APPENDIX 1: Log results of test pits at proposed Intake Dam

<table>
<thead>
<tr>
<th>Test Pit No. 1:</th>
<th>Intake Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location:</td>
<td>RHS US 4m from river</td>
</tr>
<tr>
<td>Log:</td>
<td>Loam soil</td>
</tr>
<tr>
<td>0.00 – 0.20m</td>
<td>Decomposed rock</td>
</tr>
<tr>
<td>0.20 – 0.80m</td>
<td>Decomposed rock embedded in clay soil</td>
</tr>
<tr>
<td>Note:</td>
<td>RHS = Right Hand Side, US = Up Stream</td>
</tr>
<tr>
<td>Pic 3:</td>
<td>Test Pit No. 1</td>
</tr>
<tr>
<td>Test Pit No. 2:</td>
<td>Intake Dam</td>
</tr>
<tr>
<td>------------------------------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td>Location:</td>
<td>RHS  DS  4m from river</td>
</tr>
<tr>
<td>Log:</td>
<td></td>
</tr>
<tr>
<td>0.00 – 0.10m</td>
<td>Loam soil</td>
</tr>
<tr>
<td>0.10 – 0.30m</td>
<td>Sandy material in clay groundmass</td>
</tr>
<tr>
<td>At 0.30m (Bottom)</td>
<td>Foliated granitic rock boulders (small) in clay soil under water. *Water percolates through this pit into the river. This could be due to occurrence of geophysical faulting in the rock structure that allows dissemination of water underground.</td>
</tr>
<tr>
<td>Note:</td>
<td>DS = Down Stream</td>
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<tr>
<td>Pic 4:</td>
<td>Test Pit No. 2</td>
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<td>Test Pit No. 3:</td>
<td>Intake Dam</td>
</tr>
<tr>
<td>----------------</td>
<td>------------</td>
</tr>
<tr>
<td>Location:</td>
<td>LHS US 4.4m from river</td>
</tr>
<tr>
<td>Log:</td>
<td></td>
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<tr>
<td>0.00 – 0.80m</td>
<td>Clay soil</td>
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<tr>
<td>0.80 – 1.10m</td>
<td>Decomposed rock and small boulders of granite rock embedded in clay groundmass.</td>
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<tr>
<td>Note:</td>
<td>LHS = Left Hand Side</td>
</tr>
<tr>
<td>Pic. 5:</td>
<td>Test Pit No. 3</td>
</tr>
<tr>
<td>Test Pit No. 4</td>
<td>Intake Dam</td>
</tr>
<tr>
<td>-----------------</td>
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</tr>
<tr>
<td>Location:</td>
<td>LHS  DS</td>
</tr>
<tr>
<td></td>
<td>3.4m from river</td>
</tr>
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<td></td>
</tr>
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<td>0.00 – 0.50m</td>
<td>Clay soil</td>
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<tr>
<td>0.50 – 1.20m</td>
<td>Decomposed loose rock material in clay soil</td>
</tr>
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</table>

Pic. 6: Test Pit No. 4
APPENDIX 2: Log Results of Test pits at proposed power house

<table>
<thead>
<tr>
<th>Test Pit No. 5</th>
<th>Power House</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Location:</strong></td>
<td>RHS DS 12m from river</td>
</tr>
</tbody>
</table>

**Log:**

- **0.00 – 0.20m** Loam soil
- **0.20 – 0.40m** Pebbles embedded in clay soil, compacted.
- **0.40 – 1.20m** Sandstone

**Pic. 7:** Test Pit No. 5
A.4 Appendix 4

Structural Calculation Package – *Journeyman International Deliverable*
STRUCTURAL CALCULATIONS

FOR

Karambo Micro-Indsutrial Complex

Rubavu District, Rwanda - Karambo River Masterplan

CLIENT: Empowering Villages

Prepared by:

Sophia Abshire & Stella Bates

June 2018

NOTE: THESE CALCULATIONS ARE NOT FOR CONSTRUCTION AND MUST BE REVIEWED BY AN IN COUNTRY ENGINEER
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A.1 Appendix 1: Excerpt From VERO Decking Catalog

A.2 Appendix 2: Comparative Analysis of Seismic Loading on High-Rise Steel Building Structures in Bujumbura and Kigali Cities

A.3 Appendix 3: Soils Report for Neighboring Project
Project Description/Data

Project: Karambo Micro-Industrial Complex
Location: Karambo River – Rubavu District, Rwanda
Architect: Shae Menzel
Owner: Dan Klinck – Empowering Villages

Building Codes: 2015 International Building Code (IBC)

Selected IBC References:
- Loads: ASCE 7-10
- Concrete: ACI 381-14
- Masonry: TMS 402-13 & TMS 602-13
- Steel: AISC 370-10
- Timber: NDS 2015

Project Description:
The Karambo Micro-Industrial Complex consists of 3 buildings located approximately 3.5 m above and 50 m away from the Karambo River in the Rubavu District of Western Rwanda. Each building will be constructed of confined masonry. Each building will have a steel truss canopy roof system that will frame into or around the building. One building will consist of a classroom, one of a library, and another building will have both an indoor and outdoor work area.
Structural Systems:

Gravity:

**Building A** – Steel decking acts as the roof that connects a system of steel trusses together. For the steel trusses that frame the outdoor space, the back of the truss extends down frames into a concrete retaining wall. The front of the truss cantilevers over a continuous steel column. The weight of the trusses is then transferred into the concrete tie-columns which is then transferred into the load-bearing masonry wall through a “toothing” connection. The weight of the walls, tie-columns and tie-beams are transferred down into the slab and into the reinforced concrete foundations.

**Building B & C** – Steel decking acts as the roof that connects a system of steel trusses together. Both the back and front sides of the steel trusses frame into the concrete tie-columns. The weight of the trusses and deck is then transferred into the concrete tie-columns which is then transferred into the load-bearing masonry wall through a “toothing” connection. The weight of the walls, tie-columns, and tie-beams are transferred down into the slab and into the reinforced concrete foundations.

Lateral:

**Building A** – The steel decking acts as the diaphragm. The loads from the diaphragm are transferred down into the confined masonry shear walls through bracing. Which is then transferred down into the foundation. For the outdoor areas we will need to add bracing between the retaining wall and steel trusses as well as in the openings with the steel columns.

**Building B & C** – The steel decking acts as the diaphragm. The loads from the diaphragm are transferred down into the confined masonry shear walls through bracing. The loads are then transferred down into the foundation.

**Foundation:** Continuous footing for all walls, Spread footing for columns
Design Criteria

**Classification of Building:** Category II

**Soils Information:**
- 0.0-0.2m: Loam Soil
- 0.2-0.4m: Compacted Clay Soil
- 0.4-1.2m: Sandstone

*Per IBC 2015 Table 1806.2:*
- Allowable Vertical Foundation Pressure = 2,000 psf
- Allowable Lateral Bearing Pressure = 150 psf/ft below natural grade
- Coefficient of Friction = 0.25

**Seismic Information:**
- Seismic Design Category: B
- Importance Factor: 1.0
- Spectral Response Accelerations:
  - $S_S = 0.28$  $S_1 = 0.11$
  - $S_{DS} = 0.19$  $S_{D1} = 0.07$

**Site Classification:** B

**Response Modification Coefficient (R):**

**Seismic Response Coefficient (Cs):**

**Seismic Base Shear:**

**Material Specifications:** (Typical unless noted otherwise in calculations)

- **Concrete:**  $f'_c = 3$ ksi
- **Masonry:**  $f'_c = 0.58$ ksi
- **Reinforcement:**  $f_y = 60$ ksi
- **Steel:**  $f_y = 50$ ksi (Wide Flange Sections)
  - $f_y = 46$ ksi (Square HSS Sections)
  - $f_y = 42$ ksi (Round HSS Sections)
  - $f_y = 36$ ksi (Angle and Double Angle Sections)
# DEAD LOAD TAKEOFF

## BUILDING A

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</tr>
<tr>
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<td>4</td>
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<tr>
<td><strong>Dead Load to Trusses</strong></td>
<td><strong>8</strong></td>
<td><strong>5.4</strong></td>
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<tr>
<td>Steel Trusses</td>
<td>-</td>
<td>8</td>
</tr>
<tr>
<td><strong>Sloped Dead Load to Trusses</strong></td>
<td><em><em>8 + 5.4</em>(37.1'/36.9')</em>*</td>
<td><strong>13.4</strong></td>
</tr>
<tr>
<td>Steel Beam Selfweight</td>
<td>-</td>
<td>2.5</td>
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<tr>
<td>Stee Brace Selfweight</td>
<td>-</td>
<td>2.5</td>
</tr>
<tr>
<td>Concrete Beam Selfweight (E/W)</td>
<td>150pcf x (36 ft^3)</td>
<td>2.1</td>
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<tr>
<td>In Wall</td>
<td>2583.3 sf</td>
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<tr>
<td>Concrete Beam Selfweight (N/S)</td>
<td>150pcf x (39.9 ft^3)</td>
<td>2.32</td>
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<tr>
<td>in wall</td>
<td>2583.3 sf</td>
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<tr>
<td>Steel HSS Column Selfweight</td>
<td>(8) 15.6 plf x 1.64'</td>
<td>0.08</td>
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<td></td>
<td>2583 sf</td>
<td></td>
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<tr>
<td>Steel Short WF Column Selfweight</td>
<td>(6) 31 plf x 6.56</td>
<td>0.47</td>
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<td></td>
<td>2583 sf</td>
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<td>Steel Short WF Column Selfweight</td>
<td>(7) x 31 plf x 13.12'</td>
<td>1.1</td>
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<td>(23) x 150pcf x (6.31cf)</td>
<td>0.43</td>
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<td><strong>Total Weight</strong></td>
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<td><strong>56.43</strong></td>
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<td><strong>Seismic Weight</strong></td>
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<td><strong>145.78</strong></td>
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## DEAD LOAD TAKEOFF

### BUILDING B

<table>
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<tr>
<th>Load Type/Material</th>
<th>Calculation</th>
<th>Unit Weight (psf)</th>
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<tr>
<td>Deck</td>
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</tr>
<tr>
<td>MEP &amp; MISC</td>
<td>-</td>
<td>4</td>
</tr>
<tr>
<td>Dead Load to Trusses</td>
<td>-</td>
<td>5.4</td>
</tr>
<tr>
<td>Steel Trusses</td>
<td>-</td>
<td>8</td>
</tr>
<tr>
<td><strong>Sloped Dead Load to Trusses</strong></td>
<td>$8 + 5.4 \times (29.6'/29.5')$</td>
<td>13.4</td>
</tr>
<tr>
<td>Steel Beam Selfweight</td>
<td>-</td>
<td>2.5</td>
</tr>
<tr>
<td>Steel Brace Selfweight</td>
<td>-</td>
<td>2.5</td>
</tr>
<tr>
<td>Concrete Beam Selfweight (E/W)</td>
<td>$150\text{pcf} \times (59.4\text{ ft}^3)$ (\frac{2325\text{ sf}}{2325\text{ sf}})</td>
<td>3.83</td>
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<tr>
<td>Concrete Beam Selfweight (N/S)</td>
<td>$150\text{pcf} \times (36\text{ ft}^3)$ (\frac{2325\text{ sf}}{2325\text{ sf}})</td>
<td>2.32</td>
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<tr>
<td>Steel HSS Column Selfweight</td>
<td>$(12) \times 15.6\text{ plf} \times 1.64'$ (\frac{2325\text{ sf}}{2325\text{ sf}})</td>
<td>0.13</td>
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<tr>
<td>Steel Short WF Column Selfweight</td>
<td>$(3) \times 31\text{ plf} \times 8.04'$ (\frac{2325\text{ sf}}{2325\text{ sf}})</td>
<td>0.32</td>
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<tr>
<td>Steel Long WF Column Selfweight</td>
<td>$(3) \times 31\text{ plf} \times 13.06'$ (\frac{2325\text{ sf}}{2325\text{ sf}})</td>
<td>0.52</td>
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<tr>
<td>Concrete Column Selfweight</td>
<td>$150\text{pcf} \times (6.31\text{cf}) \times (21)$ (\frac{2325\text{ sf}}{2325\text{ sf}})</td>
<td>8.55</td>
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<tr>
<td>Masonry Wall Weight</td>
<td>$94.5\text{pcf} \times (9.84' \times 771' \times 126.6)$ (\frac{2325\text{ sf}}{2325\text{ sf}})</td>
<td>39.04</td>
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<td><strong>Total Weight</strong></td>
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<td>73.11</td>
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<tr>
<td><strong>SQ FT of building</strong></td>
<td></td>
<td>2325</td>
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<td><strong>Seismic Weight</strong></td>
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<td>169.98075 kips</td>
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# Structural Design Calculations

## Dead Load Takeoff - Building C

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<tr>
<th>Load Type/Material</th>
<th>Calculation</th>
<th>Unit Weight (psf)</th>
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<tbody>
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<td>Deck</td>
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<td>1.4</td>
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<td>MEP &amp; MISC</td>
<td>-</td>
<td>4</td>
</tr>
<tr>
<td>Dead Load to Trusses</td>
<td>-</td>
<td>5.4</td>
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<tr>
<td>Steel Trusses</td>
<td>-</td>
<td>8</td>
</tr>
<tr>
<td>Sloped Dead Load to Trusses</td>
<td>$8 + 5.4^*{(29.6'/29.5')}$</td>
<td>13.4</td>
</tr>
<tr>
<td>Steel Beam Selfweight</td>
<td>-</td>
<td>2.5</td>
</tr>
<tr>
<td>Steel Brace Selfweight</td>
<td>-</td>
<td>2.5</td>
</tr>
<tr>
<td>Concrete Beam Selfweight (E/W)</td>
<td>$\frac{150 \text{pcf} \times (47.7 \text{ ft}^3)}{1166.4 \text{ sf}}$</td>
<td>6.13</td>
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<tr>
<td>Concrete Beam Selfweight (N/S)</td>
<td>$\frac{150 \text{pcf} \times (36 \text{ ft}^3)}{1166.4 \text{ sf}}$</td>
<td>4.63</td>
</tr>
<tr>
<td>Steel HSS Column Selfweight</td>
<td>$\frac{(10) 15.6 \text{ plf} \times 1.64'}{1166.4 \text{ sf}}$</td>
<td>0.22</td>
</tr>
<tr>
<td>Concrete Column Selfweight</td>
<td>$\frac{150 \text{pcf} \times (6.31 \text{cf}) \times (24)}{1166.4 \text{ sf}}$</td>
<td>19.5</td>
</tr>
<tr>
<td>Masonry Wall Weight</td>
<td>$\frac{94.5 \text{ pcf} \times (9.84' \times .771' \times 90.42')}{1166.4 \text{ sf}}$</td>
<td>55.57</td>
</tr>
<tr>
<td><strong>Total Weight</strong></td>
<td></td>
<td><strong>112.45</strong></td>
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<td><strong>SQ FT of building</strong></td>
<td></td>
<td><strong>1166.4</strong></td>
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<tr>
<td><strong>Seismic Weight</strong></td>
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<td><strong>131.16168</strong> kips</td>
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### LIVE LOAD

<table>
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<td>125</td>
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<tr>
<td>Libraries</td>
<td>-</td>
<td>60-150</td>
</tr>
<tr>
<td>Retail</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>Assembly Area</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>Classrooms</td>
<td>-</td>
<td>40</td>
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<tr>
<td><strong>Live Load for Building A</strong></td>
<td></td>
<td><strong>120</strong></td>
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<tr>
<td><strong>Live Load for Building B &amp; C</strong></td>
<td></td>
<td><strong>100</strong></td>
</tr>
<tr>
<td><strong>Roof Live Load</strong></td>
<td></td>
<td><strong>20</strong></td>
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</tbody>
</table>
Building A - Key Plan
Building C - Key Plan
Foundation - Key Plan
Gravity Design
STEEL DECK DESIGN

DL = 0 psf
LL = 20 psf

SPAN = 3 m = 9.84 ft  \approx 9' 10"  
ASSUME, DOUBLE SPAN

USE 10'-0"

- DEFORMATION

FOR A ROOF WITH ONLY LL, NOT SUPPORTING CEILING

\[ \Delta \leq \frac{L}{180} \]

USE 24 GAUGE DEEP VERCOR OR EQUIVALENT

FOR \( \frac{L}{180} \) 20 psf \( \geq 20 \text{ psf} \)

\( \checkmark \)
Building a Truss Design

TRIP AREA = \(3\text{m} \times 11.25\text{m}\)
= \(33.75 \times 36.9\text{ft}^2\)
= \(361.6\text{SF}\)

LIVE LOAD = 20 PSF = \(L_0\)

LIVE LOAD REDUCTION = \(L_0R_1R_2\)

\[R_1 = \frac{AT}{A} = \frac{361.6\text{SF}}{1.2 - 0.001(361.6\text{SF})} = 0.84\]

\[R_2 = \text{SLOPE, } F \leq 4 = 1\]

\[L_R = 20 \text{ PSF}(0.84)(1) = 16.8 \text{ PSF}\]

BACK POINT:
\[L = 16.8 \text{ PSF}(20.6 \text{ SF}) = 0.3436 \text{ k} = 1.54 \text{ kN}\]
\[D = 14.5 \text{ PSF}(20.6 \text{ SF}) = 0.299 \text{ k} = 1.33 \text{ kN}\]

MIDDLE POINT:
\[L = 14.8 \text{ PSF}(40.2 \text{ SF}) = 0.675 \text{ k} = 3.00 \text{ kN}\]
\[D = 14.5 \text{ PSF}(40.2 \text{ SF}) = 0.583 \text{ k} = 2.69 \text{ kN}\]

CANTILEVER POINT:
\[L = 16.8 \text{ PSF}(60.8 \text{ SF}) = 1.021 \text{ k} = 4.54 \text{ kN}\]
\[D = 14.5 \text{ PSF}(60.8 \text{ SF}) = 0.882 \text{ k} = 3.92 \text{ kN}\]
Truss A ETABS Analysis

Axial Diagram

Shear Diagram

Moment Diagram
<table>
<thead>
<tr>
<th>Member</th>
<th>Type of Force</th>
<th>Force (metric)</th>
<th>Force (imperial)</th>
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<tr>
<td>Reactions at Support 1</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>Fx=</td>
<td>0.00 kN</td>
<td>0.00 k</td>
</tr>
<tr>
<td></td>
<td>Fy=</td>
<td>31.97 kN</td>
<td>7.19 k</td>
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<td>Reactions at Support 2</td>
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<td></td>
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<tr>
<td></td>
<td>Fx=</td>
<td>0.00 kN</td>
<td>0.00 k</td>
</tr>
<tr>
<td></td>
<td>Fy=</td>
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<td>9.24 k</td>
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<td>Top Chord</td>
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<tr>
<td>P (T) =</td>
<td>13.07 kN</td>
<td>2.94 k</td>
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<tr>
<td>P (C) =</td>
<td>97.23 kN</td>
<td>21.86 k</td>
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<tr>
<td>V =</td>
<td>0.40 kN</td>
<td>0.09 k</td>
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<tr>
<td>M =</td>
<td>0.17 kN-m</td>
<td>0.13 k-ft</td>
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<tr>
<td>Bottom Chord</td>
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<tr>
<td>P (T) =</td>
<td>103.88 kN</td>
<td>23.35 k</td>
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<tr>
<td>P (C) =</td>
<td>0.00 kN</td>
<td>0.00 k</td>
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<tr>
<td>V =</td>
<td>0.41 kN</td>
<td>0.09 k</td>
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<td>M =</td>
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<td>A</td>
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<td>0.00 k</td>
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<td>P (C) =</td>
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<td>V =</td>
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<td>M =</td>
<td>0.00 kN-m</td>
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<td>V =</td>
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<td>0.00 k</td>
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<td>M =</td>
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<td></td>
<td>M =</td>
<td>0.00 kN-m</td>
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<td>D</td>
<td>P (T) =</td>
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<td>P (C) =</td>
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<td>0.00 k</td>
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<td>V =</td>
<td>0.00 kN</td>
<td>0.00 k</td>
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<td>M =</td>
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<td>E</td>
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<td>P (C) =</td>
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<td>V =</td>
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<td>M =</td>
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<td>F</td>
<td>P (T) =</td>
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<td>0.00 k</td>
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<td>P (C) =</td>
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<td>6.36 k</td>
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<td>V =</td>
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<td>0.00 k</td>
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<td>M =</td>
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<td>V =</td>
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<td>M =</td>
<td>0.00 kN-m</td>
<td>0.00 k-ft</td>
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<tr>
<td>H</td>
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<tr>
<td></td>
<td>P (T) = 0.00 kN</td>
<td>0.00 k</td>
<td></td>
</tr>
<tr>
<td>-----</td>
<td>----------------</td>
<td>--------</td>
<td></td>
</tr>
<tr>
<td></td>
<td>P (C) = 0.21 kN</td>
<td>0.05 k</td>
<td></td>
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<td></td>
<td>V = 0.00 kN</td>
<td>0.00 k</td>
<td></td>
</tr>
<tr>
<td></td>
<td>M = 0.00 kN-m</td>
<td>0.00 k-ft</td>
<td></td>
</tr>
</tbody>
</table>
Building A Truss Design (cont.)

FBD

\[ D_1 = 0.30K \]
\[ D_2 = 0.60K \]
\[ L_1 = 0.35K \]
\[ L_2 = 0.70K \]

- All members modeled as (2) L 2\( \frac{1}{2} \times 2\frac{3}{4} \times \frac{3}{4} \) in ETABS
- Governing load combo: L2D + 1.6L
- All member forces found using ETABS (max values)

Assumptions:
- \( f_y = 36 \) ksi, \( f_u = 58 \) ksi
- Pin-pin connection
- Top & bottom chords are continuous

Check Axial

Top Chord - 2L2\( \frac{1}{2} \times 2\frac{3}{4} \times \frac{3}{8} \)

\[ P_u = 2.93k \text{ kips} \]
\[ 21.86 \text{ kips} \]

Tension:
\[ \Phi_t P_n = 112 \text{ k} \]
\[ \Phi_t P_n > P_u (T) \checkmark \]

Compression:
\[ L = 5.02 \text{ m} = 16.47 \text{ ft} \]
\[ K = 1.0 \]
\[ K_L = 16.47 \text{ ft} \rightarrow \text{ USE 17} \]

\[ \Phi_c P_n = 24.9 \]
**Building A Truss Design (cont.)**

**Bottom Chord:** ZL 2\(\frac{3}{8}\) × 2\(\frac{3}{8}\) × 3\(\frac{3}{8}\)

\(P_u = 23.35 \text{kN (T)}\)
\(\Delta K \text{ (C)}\)

**Tension:**
\(\phi_e P_n = 112 K\)
\(\phi_e P_n > P_u \text{ (T)}\)

**Compression:**
**No Capacity Needed**

```
USE DOUBLE ANGLE ZL 2\(\frac{3}{8}\) × 2\(\frac{3}{8}\) × 3\(\frac{3}{8}\)
FOR TRUSS TOP & BOTTOM CHORDS
```

**Web (Interior) Members:** ZL 2\(\frac{3}{8}\) × 2\(\frac{3}{8}\) × 1/4

\(P_{u_{\text{max}}} = 8.27 \text{kN (T)}\)
\(18.51 \text{kN (C)}\)

**Tension:**
\(\phi_e P_n = 77 K\)
\(\phi_e P_n > P_u \text{ (T)}\)

**Compression:**
\(L = 1.6m = 5.25 \text{ ft}, k = 1.0\)
\(kL = 5.25 \rightarrow \text{USE 6}\)

\(\phi_e P_n = 60.2 K\)
\(\phi_e P_n > P_u \text{ (C)}\)

```
USE DOUBLE ANGLE
ZL 2\(\frac{3}{8}\) × 2\(\frac{3}{8}\) × 3\(\frac{3}{8}\) FOR WEB MEMBERS
```
**Karambo Micro-Industrial Complex**  
Journeyman International  
Structural Design Calculations  

**BUILDING B & C TRUSS DESIGN**

- **FBD**

- **LOAD**
  - **TRIB WIDTH**
  - **TRIB AREA = 3m x 9m**
    - $= 9.0\times 29.5\text{'}$
    - $= 289.15\text{SF}$

- **LIVE LOAD = 20 PSF = L_0**
- **LIVE LOAD REDUCTION = L_0 \times r_1 \times r_2**
  - $r_1 = A_t = 289.15\text{SF} = 1.2 - 0.001(289.15\text{SF})$
    - $= 0.91$
  - $r_2 = \text{SLOPE} = \frac{0.163\text{m} - 0.167\text{m}}{0.9\text{m} - 0.0\text{m}} = 0.07 \times 0.12 = 0.008$
  - $F \leq 4$
  - $r_2 = 1$

- **L_1 = 20\times 0.91 \times 1.0 = 18.2 \text{PSF}**

- **END POINTS:**
  - $L = 18.2 \text{PSF} (24.9 \text{SF}) \times 0.446 = 8.99 \text{KN}$
  - $D = 14.9 \text{PSF} (24.0 \text{SF}) = 0.355 K = 1.58 \text{KN}$

- **MIDDLE POINTS:**
  - $L = 18.2 \text{PSF} (48.0 \text{SF}) = 0.874 K = 3.89 \text{KN}$
  - $D = 14.9 \text{PSF} (24.5 \text{SF}) = 0.696 K = 3.10 \text{KN}$
Truss B & C ETABS Analysis

Axial Diagram

Shear Diagram

Moment Diagram
<table>
<thead>
<tr>
<th>Member</th>
<th>Type of Force</th>
<th>Force (metric)</th>
<th>Force (imperial)</th>
</tr>
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<tbody>
<tr>
<td><strong>Support 1</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Fx</strong></td>
<td>0.00 kN</td>
<td>0.00 k</td>
</tr>
<tr>
<td></td>
<td><strong>Fy</strong></td>
<td>29.92 kN</td>
<td>6.73 k</td>
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<tr>
<td><strong>Support 2</strong></td>
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<tr>
<td></td>
<td><strong>Fx</strong></td>
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<td>0.00 k</td>
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<td></td>
<td><strong>Fy</strong></td>
<td>29.92 kN</td>
<td>6.73 k</td>
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<tr>
<td><strong>Top Chord</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>P (T)</strong></td>
<td>0.00 kN</td>
<td>0.00 k</td>
</tr>
<tr>
<td></td>
<td><strong>P (C)</strong></td>
<td>-139.2 kN</td>
<td>-31.29 k</td>
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<tr>
<td></td>
<td><strong>V</strong></td>
<td>-0.449 kN</td>
<td>-0.10 k</td>
</tr>
<tr>
<td></td>
<td><strong>M</strong></td>
<td>0.61 kN-m</td>
<td>0.45 k-ft</td>
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<td><strong>Bottom Chord</strong></td>
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<tr>
<td></td>
<td><strong>P (T)</strong></td>
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<td>27.20 k</td>
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<td>0.00 k</td>
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<td><strong>V</strong></td>
<td>0.451 kN</td>
<td>0.10 k</td>
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<td></td>
<td><strong>M</strong></td>
<td>0.61 kN-m</td>
<td>0.45 k-ft</td>
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<tr>
<td><strong>A</strong></td>
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<td><strong>P (T)</strong></td>
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<td><strong>M</strong></td>
<td>0 kN-m</td>
<td>0.00 k-ft</td>
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<td><strong>B</strong></td>
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<td><strong>M</strong></td>
<td>0 kN-m</td>
<td>0.00 k-ft</td>
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<td><strong>M</strong></td>
<td>0 kN-m</td>
<td>0.00 k-ft</td>
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<td><strong>D</strong></td>
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<td><strong>P (T)</strong></td>
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<td><strong>V</strong></td>
<td>0 kN</td>
<td>0.00 k</td>
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<td><strong>M</strong></td>
<td>0 kN-m</td>
<td>0.00 k-ft</td>
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<td></td>
<td><strong>M</strong></td>
<td>0 kN-m</td>
<td>0.00 k-ft</td>
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<tr>
<td><strong>F</strong></td>
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<tr>
<td></td>
<td><strong>P (T)</strong></td>
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<tr>
<td></td>
<td><strong>V</strong></td>
<td>0 kN</td>
<td>0.00 k</td>
</tr>
</tbody>
</table>
**Karambo Micro-Industrial Complex**
Journeyman International
Structural Design Calculations

**BUILDING B Truss Design Cont'd**

- Top & Bottom Chord Modeled as (2) 2 1/2 x 2 1/2 x 3/8, Web Members
- Governing Load Combo: L 2D + 1.6 L
- All Member Forces Found Using ETABS

**Assumptions**

- $f_y = 30 \text{ ksi}$, $f_u = 58 \text{ ksi}$
- Pin-Pin Connections
- Top & Bottom Chords are Continuous
- Flexural Forces in Chords are so small they can be ignored

**Check Axial**

- **Top Chord**: (2) L 2 1/2 x 2 1/2 x 3/8
  - $P_u = 31.29 \text{k}$ (C)
  - $L = 9.03 \text{m} = 29.53 \text{ft}/2 = 14.8 \text{ft}$, $k = 1.0$
  - $KL = 19.8 \text{ft}^2$ → Use 1 1/2
  - $\phi P_n = 31.9 \text{k} > 31.3 \text{k}$ →

- **Bottom Chord**: (2) L 2 1/2 x 2 1/2 x 3/8
  - $P_u = 27 \text{k}$ (T)
  - $\phi P_n = 112 \text{k} > 27 \text{k}$ →

  **Use (2) L 2 1/2 x 2 1/2 x 3/8 for Top & Bottom Chord**

- Web (Interior) Members: (2) L 2 1/2 x 2 1/2 x 1/4
  - $P_u = 27.2 \text{k}$ (C)
  - $\text{max} = 10.5 \text{k}$ (T)
BUILDING B C TRUSS DESIGN CONT'D

TENSION:
\[ \phi P_n = 77.1 k > 10.5 k \sqrt{ }\]

COMPRESSION:
\[ L = 1.53 m = 5.02 ft, \ k = 1.0 \]
\[ W_L = 5.02 ft \Rightarrow USE \ 5 P \]
\[ \phi P_n = 63.6 k > 27.2 k \sqrt{ }\]

USE (2) L 2' - 1/2" x 2' - 1/2" x 1/4" FOR ALL INTERIOR MEMBERS

CHECK DEFORMATION

FOR ROOF MEMBERS SUPPORTING NON-PLASTER CEILING

(2) \[ \Delta \leq \frac{f}{180} \]
\[ 
\Delta_L = 5.29 mm = 0.21 in \leq 1.97 in = \frac{359.4 in}{180} \sqrt{ }\]

(DL) \[ \Delta \leq \frac{f}{120} \]
\[ 
\Delta_{DL} = 0.21 in + 0.17 in = 0.38 \in \leq \frac{359.4 in}{120} \sqrt{ }\]
COLUMN C1:

9.24 k

1.64 ft

ASSUMPTIONS
- CANTILEVER
- $F_y = 40,000$ psi

COMPRESSION
$L = 1.64$ ft
$k = 2.0
KL = 3.28
USE $\phi = 0$

TRY HSS 5 x 5 x Y4

(T4-11)

\( \phi P_n = 171 \) k  > 9.28 k \(
\)

USE HSS 5 x 5 x Y4

CHECK SELF WEIGHT

\( P_y = 9.24 k + \frac{15.02 \# / ft \times (1.64 \text{ ft} + 1.64 \text{ ft})}{1000 \# / k} = 9.27 k < 171 k \) \(
\)

T1-12
AISC 14
COLUMN C2:

9.24k

6.56'

ASSUMPTIONS
- CANTILEVER
- Fy = 50KSI

COMPRESSION

L = 6.56', k = 2.0, KL = 13.12 \rightarrow \text{USE 14}

TRY W8 x 31

(T4-1) \( \Phi_c P_n = 248 \text{ k} > 9.24 \text{ k} \) \checkmark

USE W8 x 31

CHECK SELF WEIGHT

\( P_y = 9.24k + \frac{31PLF(6.56\text{ ft})}{1000 \#/\text{k}} = 9.44k < 248 \text{ k} \) \checkmark
COLUMN C3

\[ 9.24 \text{k} \]

**Assumptions**
- CANTILEVER
- \( F_y = 50 \text{ ksi} \)

**Compression**
\[ L = 13.12 \text{ ft}, \quad K = 2.0, \quad K_L = 26.24 \quad \rightarrow \text{USE 28} \]
\[ \text{TRY} \quad W8\times31 \]
\[ (T4-1) \quad \Phi_cP_n = 74.5k > 9.24k \quad \checkmark \]

**Use W8\times31**

**Check Self Weight**
\[ P_y = 9.24k + \frac{31 \text{ PLF} (13.12k)}{1000 \text{ ft/}k} = 9.45k < 74.5k \quad \checkmark \]
COLUMN CA DESIGN

ASSUMPTIONS

- $f_y = 46$ ksi
- CANTILEVER (CONSERVATIVE)
- HSS

LOAD

$P_u = 6.73k (C)$

CHECK AXIAL

$L = 0.9m = 2.95ft$, $k = 2.0$

$K_l = 1.64 + (2) = 3.28ft \Rightarrow USE A4A$

TRY HSS 5x5 x 1/4

$P_h = 17.1k > 6.73k$

CHECK SLENDERNESS

$\frac{K_l}{r} = \frac{3.28ft}{1.93in} = 20.4 < 200 \checkmark$

USE HSS 5x5 x 1/4 FOR COLUMN CA

CHECK SELF WEIGHT

$P_w = 0.73k + \frac{15.62 \text{ PLF (1.64 ft)}}{1000 \# / k} = 0.76k < 171k \checkmark$
COLUMN C5 DESIGN

FBD

\[ \frac{1}{2} \cdot \frac{73}{k} \]

2.45 m

L = 2.45 m = 8.04 ft, K = 2.0

\[ kL = 8.04 \text{ ft} \times 2 = 16.08 \text{ ft} \Rightarrow \text{USE 16 ft} \]

LOAD

\[ P_u = 6.73k \]

CHECK AXIAL:

\[ L = 2.45 \text{ m} = 8.04 \text{ ft}; K = 2.0 \]

\[ kL = 8.04 \text{ ft} \times 2 = 16.08 \text{ ft} \Rightarrow \text{USE 16 ft} \]

TRY W8 x 31

\[ \phi P_n = 212k > 6.73k \checkmark \]

USE W8 x 31 FOR COLUMN C5

CHECK SELF WEIGHT

\[ P_u = 6.73k + 31 \text{ PLF} \left( 8.04 \text{ ft} \right) = 6.98k < 212k \checkmark \]

ASSUMPTIONS

- \( f_y = 50 \text{ ksi} \)
- CANTILEVER (CONSERVATIVE)
- WF
COLUMN C6 DESIGN

FBD

\[ 0.73k \]

ASSUMPTIONS
- \( F_y = 50 \text{ ksi} \)
- CANTILEVER (CONSERVATIVE)
- WF

LOAD
\[ P_u = 0.73k \]

CHECK AXIAL
\[ L = 13.09\text{ ft}, K = 2.0 \]
\[ K_L = 13.09\text{ ft}(2) = 26.18\text{ ft} \rightarrow USE 26\text{ ft} \]

TRY W8 x31
\[ f_{ph} = 86.5k \geq 0.73k \]

USE W8 x31 FOR COLUMN C6

CHECK SELF WEIGHT
\[ P_u = 0.73k + \frac{31 \text{ PLF}(13.06\text{ ft})}{1000 \times 1k} = 7.13k \leq 86.5k \]
COMPRESSION STRENGTH OF CONFINED MASONRY

MASONRY (TMS 402-13)

\[ h = 9.843' \times 12''/1' = 118.12 \text{ in} \]
\[ r = \frac{t}{\sqrt{12}} = \frac{9.25''}{\sqrt{12}} = 2.67 \]
\[ h = 118.12 = 44.24 \leq 99 \]
\[ r = 2.67 \]

\[ P_n = 0.8 \left[ 0.8 \frac{f_m}{f_y} (A_n - A_{s+}) + f_y A_s \right] \left[ 1 - \left( \frac{h}{180r} \right)^2 \right] \]

\( A_{s+} = 0 \) b/c it is unreinforced masonry

\[ P_n = 0.8 \left[ 0.8 \left( \frac{58 \text{ksi}}{} \right) (9.25'' \times 8.47'') \right] \left[ 1 - \left( \frac{118.12}{180(2.67)} \right)^2 \right] \]

\[ P_n = 26.18 \]

\[ \Phi P_n = 0.6 P_n = 15.71 \text{ k} \]

CONCRETE (ACI 318-14 T22.4.2.1)

\[ P_n = 0.8 \left[ 0.85 \frac{f_c}{f_y} (A_g - A_{s+}) + f_y A_s \right] \]

\( A_{s+} = 0 \) (FOR NOW - IF NEEDED - WILL ADD)

\[ P_n = 0.8 \left[ 0.85 \left( 3 \text{ksi} \right) (9.25 \text{ in}^2) \right] = 174.55 \text{ k} \]

\[ \Phi P_n = 0.6 \left( 174.55 \text{ k} \right) = 104.73 \text{ k} \]

\[ P_{u \text{max}} = 9.25 \text{ k (weight to col)} + 0.40 \text{ k (weight of col)} = 9.65 \text{ k} < 104.73 \text{ k} \]
BUILDING A, B, & C CONFINED MASONRY DESIGN

TIE COLUMNS

3.1.2.2

\[ A = 235 \text{mm} \times 235 \text{mm} \]

MINIMUM DEPTH = 150 mm < 235 mm \( \checkmark \)

MINIMUM WIDTH = \( t = 235 \text{mm} \leq 235 \text{mm} \)

3.1.2.3

MINIMUM OF 4 LONGITUDINAL BARS #3 OR GREATER

\[ \text{MINIMUM } \#3 \text{ TIES W/135\' HOOKED ENDS} \]

W/ MINIMUM 200 mm SPACING; IT IS NOT REQUIRED TO REDUCE TIE SPACING AT TIE-COLUMN ENDS. (7.87 in)

MINIMUM COVER OF 20 mm (0.79 in)

USE 235 mm x 235 mm TIE-COLUMNS W/ 4 #4 LONGITUDINAL BARS & #3 TIES @ 200 mm (≈ 7 in)

DESIGN SUMMARY

\[ \#3 \text{TIES W/135\' HOOK C Tin 0.6} \]

\[ (A) \#45 \]

\[ 1.0 \text{in (38.10 mm)} \]

9.25 in (235 mm)

9.25 in (235 mm)
BUILDING A, B, & C CONFINED MASONRY DESIGN CONT'D

TIE BEAMS

3.1.2.2

A = 235 mm x 235 mm
MINIMUM DEPTH = 150 mm < 235 mm
MINIMUM THICKNESS = t = 235 mm ≥ 235 mm

3.1.2.3

MINIMUM 4 LONGITUDINAL BARS #3 OR GREATER

MINIMUM #3 STIRRUPS W/ 135° HOOKED ENDS
W/ MINIMUM 200 mm SPACING (7.87 in).

MINIMUM COVER OF 20 mm (0.79 in)

USE 235 mm x 235 mm TIE BEAMS W/ 4 #4s & #3 STIRRUPS
c @ 200 mm (≈ 7 in)

DESIGN SUMMARY

#3 TIES W/ 135° HOOK @ 7 in O.C.

(4) #4s

1.5 in (30.8 mm)
**Beam by Design**

**FBD**

\[
D = 1.4 \text{psf}, \ L = 20 \text{psf}
\]

\[
\begin{array}{c}
\downarrow \\
\downarrow \\
\downarrow \\
\downarrow \\
\downarrow \\
\downarrow \\
\downarrow \\
3m \\
(9.84 \text{ft})
\end{array}
\]

**Load**

\[
W_u = 1.2D + 1.6L = 1.2(1.4 \text{psf}) + 1.6(20 \text{psf}) = 33.68 \text{psf}
\]

**UB Width**

\[W_u = 33.68 \text{psf} (18.4 \text{ft}) = 0.919 \times \text{PLF} \times \frac{1}{1000} = 0.62 \text{kip}
\]

**Assumptions**

- L Section
- \( f_y = 36 \text{ksi} \)
- Conservative by Nut Reducing Line Load

**Design for Flexure**

\[M_u \leq \phi M_n\]

**Try**

\[L \times 0.12 \times 0.25\]

\[M_n = F_y z_y = (36 \text{ksi})(12.08 \text{in}) = 433.7 \text{kip}-\text{in} = 8.04 \text{k-ft}\]

\[\phi M_n = 0.9(8.04 \text{k-ft}) = 7.24 \text{k-ft} > 7.5 \text{k-ft} \checkmark\]

**Check Shear**

\[V_u \leq \phi V_n\]

\[V_n = 0.6 F_y A_w C_y\]

\[\frac{h}{t_w} = \frac{b}{t} = \frac{6 \text{in}}{0.5 \text{in}} = 12 < 260\]

\[k_v = 1.2\]

\[A_w = b t = 6 \text{in} (0.5 \text{in}) = 3 \text{in}^2\]
BEAM B1 DESIGN CONT'D

\[ 1.10 \sqrt{\frac{W}{E}} = 1.10 \sqrt{\frac{1.2(29,000 \text{ ksi})}{36 \text{ ksi}}} = 34.2 \]

\[ 12 \leq 34.2 \therefore CV = 1.0 \]

\[ V_n = 0.6(36 \text{ ksi})(3 \text{ in}^2)(1.0) = 64.8 \text{k} \]

\[ \phi V_n = 0.9(64.8 \text{k}) = 58.3 \text{k} > 3.0 \text{ k} \checkmark \]

CHECK DEFORMATION

\[ \Delta_{\text{max}} = \frac{5 WL^4}{384EI} = \frac{5(0.594 \text{ kLF} \times \frac{1}{12})(9.84 \text{ ft} \times 12)^4}{384(29,000 \text{ ksi})(16.6 \text{ in}^4)} = 0.26 \text{ in} \]

\[ \Delta_{\text{max}} = \frac{5 WL^4}{384EI} = \frac{5(0.041 \text{ kLF} \times \frac{1}{12})(9.84 \text{ ft} \times 12)^4}{384(29,000 \text{ ksi})(16.6 \text{ in}^4)} = 0.018 \text{ in} \]

\[ 0.26 \text{ in} \leq 0.06 \text{ in} = \frac{9.84 \text{ ft}(12 \text{ in}+)}{180} \checkmark \]

\[ 0.28 \text{ in} \leq 0.98 \text{ in} = \frac{9.84 \text{ ft}(12 \text{ in}+)}{120} \checkmark \]

USE L (6 x 3\frac{1}{2} x \frac{1}{2}) FOR B1
Lateral Design
SEISMIC BASE SHEAR

\[ S_{ds} = 0.19g \quad S_{d1} = 0.07g \]

\[ I_c = 1.0 \quad R = 4 \text{ (SOLID MASONRY UNITS)} \]

\[ C_s = \frac{S_{ds}}{\frac{R}{I_c}} = \frac{0.19}{\frac{4}{1}} = 0.0475 \]

\[ C_{s,\text{max}} = \frac{S_{d1}}{T(\frac{R}{I_c})} \]

\[ T_a = 0.1N = 0.1(1) = 0.15 \]

\[ C_{s,\text{max}} = \frac{0.07}{0.1(4)} = 0.175 \]

\[ C_{s,\text{min}} = 0.044S_{ds}I_c \geq 0.01 \]

\[ C_{s,\text{min}} = 0.044(0.19)(1) = 0.01 \]

\[ 0.01 \leq 0.0475 \leq 0.175 \checkmark \]

\[ C_s = 0.0475 \]

\[ V = C_sW \]

BUILDING A

\[ W = 145.78k \]

\[ V = (0.0475)(145.78k) = 6.9k \]

BUILDING B

\[ W = 169.98k \]

\[ V = (0.0475)(169.98k) = 8.1k \]

BUILDING C

\[ W = 131.16k \]

\[ V = (0.0475)(131.16k) = 6.2k \]
# Wind Analysis

<table>
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<th>Reference (ASCE)</th>
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<td>Risk Category= II</td>
<td>T 1.5-1</td>
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<td>Wind Speed, V= 110 mph</td>
<td>**</td>
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<tr>
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<tr>
<td>Kd= 0.85</td>
<td>T 26.6-1</td>
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<tr>
<td>Exposure Category= B</td>
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<tr>
<td>Kzt= 1.00</td>
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<tr>
<td>G= 0.85</td>
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<td>Enclosure Classification= Partially Enclosed Building</td>
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<tr>
<td>GCpi= -0.55</td>
<td>T 26.11-1</td>
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<tr>
<td>Kz= 0.57 (h=4.00 m)</td>
<td>T 27.3-1</td>
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<tr>
<td>Kh= 0.57 (h=4.00 m)</td>
<td>T 27.3-1</td>
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<tr>
<td>qz= 15.01</td>
<td>27.3-1</td>
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<tr>
<td>qh= 15.01</td>
<td>27.3-1</td>
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<td>E/W - L/B= 1.00</td>
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<td>27.4-1</td>
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<tr>
<td>Leeward, Cp= -0.50 (use with qh)</td>
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</tr>
<tr>
<td>N/S - L/B= 1.00</td>
<td>-</td>
</tr>
<tr>
<td>Windward, Cp= 0.8 (use with qz)</td>
<td>27.4-1</td>
</tr>
<tr>
<td>Leeward, Cp= -0.5 (use with qh)</td>
<td>27.4-1</td>
</tr>
<tr>
<td>*p= 18.46 psf</td>
<td>27.4-2</td>
</tr>
<tr>
<td>Wall Elevation SF (E/W)= 358.8 sf</td>
<td></td>
</tr>
<tr>
<td>Wall Elevation SF (N/S)= 323.8 sf</td>
<td></td>
</tr>
<tr>
<td>Force from wind (E/W) = 6.62 k</td>
<td></td>
</tr>
<tr>
<td>Force from wind (N/S) = 5.98 k</td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**

**Value taken as the minimum wind speed in the USA**

*Worst case wind load due to windward side and full building height*
### WIND ANALYSIS

<table>
<thead>
<tr>
<th>Building B</th>
<th>Reference (ASCE)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Risk Category=</td>
<td>II</td>
</tr>
<tr>
<td>Wind Speed, V=</td>
<td>110 mph</td>
</tr>
</tbody>
</table>

#### Wind Load Parameters:

- **Kd=** 0.85  
  - Reference: T 20.6-1

- **Exposure Category=** B  
  - Reference: 26.7

- **Kzt=** 1.00  
  - Reference: 26.8-1

- **G=** 0.85  
  - Reference: 26.9

- **Enclosure Classification=** Partially Enclosed Building  
  - Reference: 26.10

- **GCpi=** -0.55  
  - Reference: T 26.11-1

- **Kz=** 0.57 (h=4.00 m)  
  - Reference: T 27.3-1

- **Kh=** 0.57 (h=4.00 m)  
  - Reference: T 27.3-1

- **qz=** 15.01  
  - Reference: 27.3-1

- **qh=** 15.01  
  - Reference: 27.3-1

- **E/W - L/B=** 1.79  
  - Reference: -

- **Windward, Cp=** 0.8 (use with qz)  
  - Reference: 27.4-1

- **Leeward, Cp=** -0.34 (use with qh)  
  - Reference: 27.4-1

- **N/S - L/B=** 0.56  
  - Reference: -

- **Windward, Cp=** 0.8 (use with qz)  
  - Reference: 27.4-1

- **Leeward, Cp=** -0.5 (use with qh)  
  - Reference: 27.4-1

- ***p=** 18.46 psf  
  - Reference: 27.4-2

- **Wall Elevation SF (E/W)=** 323.8 sf

- **Wall Elevation SF (N/S)=** 533.8 sf

- **Force from wind (E/W) =** 5.98 k

- **Force from wind (N/S) =** 9.85 k

**NOTES:**

- **Value taken as the minimum wind speed in the USA**

- **Worst case wind load due to windward side and full building height**
## WIND ANALYSIS

<table>
<thead>
<tr>
<th>Building C</th>
<th>Reference (ASCE)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Risk Category= II</td>
<td>T 1.5-1</td>
</tr>
<tr>
<td>Wind Speed, V= 110 mph</td>
<td>**</td>
</tr>
<tr>
<td><strong>Wind Load Parameters:</strong></td>
<td></td>
</tr>
<tr>
<td>$K_d$= 0.85</td>
<td>T 20.6-1</td>
</tr>
<tr>
<td>Exposure Category= B</td>
<td>26.7</td>
</tr>
<tr>
<td>$K_z=$ 1.00</td>
<td>26.8-1</td>
</tr>
<tr>
<td>$G$= 0.85</td>
<td>26.9</td>
</tr>
<tr>
<td>Enclosure Classification= Partially Enclosed Building</td>
<td>26.10</td>
</tr>
<tr>
<td>$G_{Cpi}$= -0.55</td>
<td>T 26.11-1</td>
</tr>
<tr>
<td>$K_z$= 0.57 (h=4.00 m)</td>
<td>T 27.3-1</td>
</tr>
<tr>
<td>$K_h$= 0.57 (h=4.00 m)</td>
<td>T 27.3-1</td>
</tr>
<tr>
<td>$q_z$= 15.01</td>
<td>27.3-1</td>
</tr>
<tr>
<td>$q_h$= 15.01</td>
<td>27.3-1</td>
</tr>
<tr>
<td>$E/W - L/B$= 1.32</td>
<td>-</td>
</tr>
<tr>
<td>Windward, $C_p$= 0.8 (use with $q_z$)</td>
<td>27.4-1</td>
</tr>
<tr>
<td>Leeward, $C_p$= -0.44 (use with $q_h$)</td>
<td>27.4-1</td>
</tr>
<tr>
<td>$N/S - L/B$= 0.75</td>
<td>-</td>
</tr>
<tr>
<td>Windward, $C_p$= 0.8 (use with $q_z$)</td>
<td>27.4-1</td>
</tr>
<tr>
<td>Leeward, $C_p$= -0.5 (use with $q_h$)</td>
<td>27.4-1</td>
</tr>
</tbody>
</table>

*p= 18.46 psf 27.4-2

Wall Elevation SF (E/W)= 323.8 sf 27.4-2
Wall Elevation SF (N/S)= 428.8 sf 27.4-2

Force from wind (E/W) = 5.98 k 27.4-2
Force from wind (N/S) = 7.92 k 27.4-2

**NOTES:**

**Value taken as the minimum wind speed in the USA**

*Worst case wind load due to windward side and full building height*
BUILDING A DIAPHRAGM ANALYSIS

LOAD

- E/W DIRECTION
  \[ E = 6.9 \text{k} \quad \text{< GOVERNS} \]
  \[ W = 0.62 \text{t} \]

- N/S DIRECTION
  \[ E = 6.9 \text{k} \quad \text{< GOVERNS} \]
  \[ W = 5.98 \text{k} \]

EAST/WEST DIRECTION

\[ w = 0.210 \text{kLF} \]

\[ f_4 = f_5 = \frac{0.210 \text{kLF} (32.8 \text{ft})}{2} = 3.45 \text{k} \]
BUILDING A DIAPHRAGM ANALYSIS CONT'D

NORTH/SOUTH DIRECTION

\[ w = \frac{6.94 \text{k}}{79.83 \text{ft}} = 0.086 \text{kLF} \]

\[ f_1 = \frac{0.086 \text{kLF}(50.3 \text{ft})}{2} = 2.16 \text{k} \]

\[ f_3 = \frac{0.086 \text{kLF}(29.53 \text{ft})}{2} = 1.27 \text{k} \]

\[ f_2 = 2.16 \text{k} + 1.27 \text{k} = 3.43 \text{k} \]

\[ f_1 = 2.16 \text{k}, \ f_2 = 3.43 \text{k}, \ f_3 = 1.27 \text{k}, \ f_4 = 3.45 \text{k}, \ f_5 = 3.45 \text{k} \]
**BUILDING B DIAPHRAGM ANALYSIS**

**LOAD**

- **E/W DIRECTION**
  - E = 8.1 k < GOVERNS
  - W = 5.98 k

- **N/S DIRECTION**
  - E = 8.1 k
  - W = 9.86 k < GOVERNS

**EAST/WEST DIRECTION**

\[ f_4 = f_5 = \frac{0.274 k \cdot e d (29.03 ft)}{2} = 4.05 k \]
BUILDING B DIAPHGRAM ANALYSIS CONT'D

NORTH/SOUTH DIRECTION

\[ W = 18.46 \text{ psf (10.67 ft)} = 0.197 \text{ klf} \]

\[ f_1 = \frac{0.197 \text{ klf} \times 50.3 \text{ ft}^2}{2} = 4.93k \]

\[ f_3 = 0k \Rightarrow f_2 = 4.93k \]

**Because there is no confined masonry wall in the section between f_2 and f_3, there is no applied wind load, so the lateral force on f_3 must be found using the earthquake load.**

\[ W = 8.1k \times 79.83 \text{ ft} = 0.101 \text{ klf} \]

\[ f_1 = \frac{0.101 \text{ klf} \times 50.3 \text{ ft}^2}{2} = 2.54k < 4.93k \]

\[ f_3 = \frac{0.101 \text{ klf} \times 29.53 \text{ ft}^2}{2} = 1.50k \]

\[ f_2 = 2.54k + 1.50k = 4.04k < 4.93k \]

\[ f_1 = 4.93k, f_2 = 4.93k, f_3 = 1.50k, f_4 = 4.04k, f_5 = 4.05k \]
BUILDING C DIAPHRAGM ANALYSIS

LOAD

- E/N DIRECTION
  \( E = 6.2k \) \( \leq \) GOVERNS
  \( W = 5.98k \)

- N/S DIRECTION
  \( E = 6.2k \)
  \( W = 7.92k \) \( \leq \) GOVERNS

FACTOR/WEST DIRECTION

\[ w = \frac{0.2k \times 29.53 \text{ ft}}{2} = 0.21 \text{ klf} \]

\[ f_3 = f_4 = \frac{0.21 \text{ klf}(29.53 \text{ ft})}{2} = 3.1k \]

NORTH/SOUTH DIRECTION

\[ W = 18.46 \text{ psf} \times \frac{10.07 \text{ ft}^2}{1000} = 0.197 \text{ klf} \]

\[ f_2 = f_1 = \frac{0.197 \text{ klf}(39.37 \text{ ft})}{2} = 3.87k \]

\[ f_1 = 3.87k, f_2 = 3.87k, f_3 = 3.1k, f_4 = 3.1k \]
CHORD DESIGN - E/W

MAX CONDITION

210 plf (MAX LOAD BY ALL BLDGS)

37'

3.9 k

35.9 k'

T/C FORCE = \frac{35.9 k'}{40'} = 0.9 K

COMPRESSION = \frac{2L2\frac{1}{2} \times 2\frac{1}{2} \times 3/8}{P_n = F_{cr} A_{y}}

\frac{Kl}{r} = \frac{11(36.9)}{49.27} = 49.27

4.71 \sqrt{\frac{29,000}{36}} = 133.7 > 49.27

\therefore F_{cr} = [0.658 \frac{F_y}{F_{cr}}] F_y

F_e = \frac{\pi^2 29,000}{(49.27)^2} = 117.9 \text{ ksi}

F_{cr} = [0.658 \frac{30}{117.9}] (30 \text{ ksi}) = 31.68 \text{ ksi}

P_n = 31.68 \text{ ksi} \times (3.4 \text{ in}^2) = 109.6 \text{ k}

\phi P_n = 0.9 \times 109.6 = 98.64 \text{ k} \checkmark
TENSION - 2L 2\frac{1}{2} x 2\frac{1}{2} x 3\frac{1}{8}

P_n = F_y A_g = 30 \text{kSi} \times (3.44 \text{ in}^2) = 124.6 \text{k}

\phi P_n = .9 P_n = 112.1 \text{k} ✓

2L 2\frac{1}{2} x 2\frac{1}{2} x 3\frac{1}{8} IS ADEQUATE FOR CHORD
CHORD DESIGN - N15

MAX CONDITION

\[ 107 \text{ plf} = 18.6 \text{ psf} (5.74') \]

\[ (\text{MAX LOAD}) \]

\[ \frac{33.5 k'}{250'} \]

\[ T/C \text{ FORCE} = \frac{33.5 k'}{250'} = 1.12 \text{ k} \]

COMPRESSION - [2 \frac{1}{2} \times 2 \frac{1}{2}] \times \frac{1}{4}

\[ \frac{KL}{r} = \frac{1 (10)}{0.764} = 13.1 \left< 4.71 \sqrt{29,000} \div 36 \right. \]

\[ F_{cr} = \left[ 0.658 \frac{F_y}{F_e} \right] F_y = 31.68 \text{ ksi} \]

\[ P_n = 31.68 \text{ ksi} (1.19 \text{ in}^2) = 37.7 \text{ K} \]

\[ \phi P_n = 0.9 P_n = 33.9 \text{ K} \] ✔

TENSION - [2 \frac{1}{2} \times 2 \frac{1}{2}] \times \frac{1}{4}

\[ P_n = F_y A_g = 36 \text{ ksi} (1.19 \text{ in}^2) = 42.84 \text{ K} \]

\[ \phi P_n = 0.9 P_n = 38.56 \text{ K} \] ✔

L 2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{1}{4} \text{ IS ADEQUATE FOR CHORD DESIGN}
COLLECTOR DESIGN - N/S

Pu = 2.68 k (SEE CHORD DESIGN)

\[ \phi P_{nc} = 9.8.66 k \]

\[ \phi P_{nt} = 11.1 k \]

\{ SEE CHORD DESIGN \}

2 L 2 1/2 x 2 1/2 x 3/8 IS ADEQUATE FOR COLLECTOR

---

COLLECTOR DESIGN - E/W

Pu = 3.9 k (SEE CHORD DESIGN)

\[ \phi P_{nc} = 33.9 k \]

\[ \phi P_{nt} = 38.56 k \]

\{ SEE CHORD DESIGN \}

L 2 1/2 x 2 1/2 x 1/4 IS ADEQUATE FOR COLLECTOR
BRACE FRAME DESIGN

For brace frames that transfer the lateral load from the truss to the confined masonry, brace frame B1 & brace frame B2, the maximum forces from all 3 buildings were used, thus the design was for the worst case scenario & will work for all buildings.

For brace frames that transfer the lateral load from the truss to the ground, brace frame B3 & brace frame B4, the maximum forces from all 3 buildings were used, thus the design was for the worst case scenario & will work for all 3 buildings.
**BRACE FRAME B1 DESIGN**

**FBD**

\[ L_r = 0.447 \text{k} \]

\[ p = 4.05 \text{k} \]

\[ d = 2.57 \text{k} \]

\[ \theta = 3.12' \]

\[ 3 \text{m} \]

\[ 9.84' \]

**EQUIVALENT LOAD COMBOS**:  

\[ (1.2 + 0.2 \cdot S_{pd}) \cdot D + \frac{Q_E}{10} + 0.5 \cdot L \]  

\[ (0.9 - 0.2 \cdot S_{pd}) \cdot D + \frac{Q_E}{10} \]

\[ S_{pd} = 0.19 \text{ g} \]

\[ p = 1.0 (\text{SEISMIC DESIGN CATEGORY B}) \]

**DESIGN FORCES**

\[ 1.89 \text{k} (c) \]

\[ 1.93 \text{k} (c) \]

\[ 2.27 \text{k} (c) \]

\[ 0.39 \text{k} (c) \]

\[ 1.87 \text{k} (c) \]

\[ 1.93 \text{k} (c) \]

\[ 2.27 \text{k} (c) \]

\[ 0.904 \text{k} (c) \]

**DESIGN OF BRACES**

\[ P_n = 1.93 \text{k} \]

\[ P_n = 2.27 \text{k} \]

\[ \Phi \cdot P_n = 28.4 \text{k} \checkmark \]

\[ HSS 2.500 \times 1.125 \text{ ROUND (T6-G)} \]

\[ F_c = \frac{\pi^2 (29,000)}{144^2} = 13.25 \]

\[ F_{cr} = \left[ \frac{0.658 \cdot 13.25}{42} \right] = 11.14 \text{ ksi} \]

\[ \Phi \cdot P_n = 0.9 (11.14 \text{ ksi}) (0.869 \text{ in}^2) = 8.72 \text{k} \checkmark \]

\[ HSS 2.500 \times 1.125 \text{ ROUND USE} \]
BEAM DESIGN

FBD:  $E = 40.05 \text{k}$

$L = 164.04 \text{ plf}$

$P = 11.48 \text{ plf}$

(beam was already checked for flexure alone)

COMPRESSION CHECK

$k \ell = 1.0(9.84) = 10$

$P_{n0} = 40.3 \text{k} \checkmark$

COMBINED LOADING

$M_u = \frac{(11.48)(9.84)^2}{2} (1.238) + 0.5 \left[ \frac{164.04(9.84)^2}{2} \right]$

$M_u = 4.66 \text{ k-ft}$

$\Phi M_n = 7.78 \text{ k-ft} \quad \text{(previous calc)}$

$\frac{P_c}{P_c} = 4.05 < 1.00 < 0.2$

\[
2P_c + \left( \frac{M_{ex}}{M_{ex}} + \frac{M_{ey}}{M_{ey}} \right) \leq 1.0 \quad \text{(H1-1b)}
\]

$4.05 + 4.66 = 0.65 \leq 1.0 \checkmark$

USE $L_{c0} \times 2 \frac{1}{2} \times \frac{1}{2}$ FOR BEAM

COL DESIGN

$P_{ult} = 0.39 \text{k}$

$P_n (c) = 1.26 \text{k}$

$\Phi_c P_n = 63.6 \text{k} \checkmark$

$\Phi_t P_n = 77.1 \text{k} \checkmark$

SEE TRUSS B DESIGN

USE $(2)L_{2} \frac{1}{2} \times \frac{1}{2} \times \frac{1}{4}$ FOR COLUMN
BRACE FRAME B2 DESIGN

GOVERNING LOAD COMBO: 1.2D + 1.0W + 0.5L

DESIGN FORCES

DESIGN OF BRACES

Pu(c)= 2.24k  
Pu(c)= 2.7k

HSS 2.500 x 1.25  ROUND:

ΦcPn= 8.72 k  ✓

ΦPn= 28.4 k  ✓

USE HSS 2.500 x 1.25 ROUND

CHECK BOT CHORD OF TRUSS

Pu= 2.24k (c)

ΦPn= 31.9 k ✓

(2) L 2 1/2 x 2 1/2 x 3/8  ✓
**CHECK HSS COLUMNS**

\[ P_u(c) = 3.15k \]

\[ \phi P_n = 171k \checkmark \quad \text{HSS 5x6x1/4 \( \square \)} \]

---

**BEAM DESIGN FOR OTHER DIRECTION**

\[ P_u = 4.93k \]

\[ \phi P_n = 14.1k \]

\[ KL = 10 \]

\[ 3 \text{m} \]

\[ 9.84 \text{ ft} \]

**USE**

\[ L3\frac{1}{2} \times 3 \times 3/8 \quad \text{FOR COLLECTOR} \]
SPACE FRAME B3 DESIGN

ETABS

BUILDING B
DIAPHRAGM ANALYSIS

LOAD

E = 1.50 k
SEE FBDO ABOVE FOR PD & PL LOADS.
CHECKED LOAD COMBOS:

(1.2 + 0.25 ps) + Pe + 0.5 L
(0.9 - 0.25 ps) + Pe

SDS = 0.19 g

SEISMIC DESIGN CATEGORY B :: Pe = 1.0

DESIGN FORCES

* FORCES FOUND FROM MAXIMUM VALUES BETWEEN BOTH LOAD COMBINATIONS
SPACE FRAME B3 DESIGN CONT'D

**DESIGN OF B3**

\[ P_u = 1.03k(c), 1.04k(T) \]

**TRY HSS3.00 x 0.25**

\[ L_e = 5m = 14.4\text{ ft} \]

- **COMPRESSION**

  \[ k = 1.0, \quad \frac{kL}{V} = \frac{1.0(16.44\text{ ft} \times 12)}{0.982} = 200.4 \]

  \[ 4.71 \sqrt{\frac{E}{F_y}} = 4.71 \sqrt{\frac{29,000\text{ ksi}}{42\text{ ksi}}} = 123.6 \]

  \[ 200.4 > 123.6 \]

  \[ F_{cr} = 0.877F_0 \]

  \[ F_c = \frac{n^2E}{(kL)^2} = \frac{11^2(29,000\text{ ksi})}{(200.4)^2} = 7.13\text{ ksi} \]

  \[ F_{cr} = 0.877(7.13\text{ ksi}) = 6.25\text{ ksi} \]

  \[ P_{nc} = 0.25(2.03\text{ in}^2) = 12.69k \]

  \[ \phi P_n = 0.9(12.69k) = 11.43k > 7.13k \checkmark \]

- **TENSION**

  \[ P_{nt} = F_y A_y = 42\text{ ksi}(2.03\text{ in}^2) = 86.26k \]

  \[ \phi P_n = 0.9(86.26k) = 77.6k > 1.04k \checkmark \]

**USE HSS3.00 x 0.25 FOR B3**

**CHECK BOTTOM CHORD OF TRUSS**

\[ P_u = 0.62k(c) \]

\[ P_n \text{ OF A (2) L}2\frac{1}{2} \times 2\frac{1}{2} \times 3\frac{1}{8} \text{ FOUND IN TRUSS BAC DESIGN} \]

\[ \phi P_n = 31.9k > 0.62k \checkmark \]

**CHECK COLUMN C6**

FROM COLUMN C6 CALCULATIONS

\[ P_u = 6.75k + 2.61k = 9.36k \]

\[ \phi P_n = 86.1k > 9.36k \checkmark \]
BRACE FRAME B4 DESIGN

FBD

D = 2.35k
L = 2.73k

BOTTOM CHORD OF TRUSS

D = 0.001k
L = 0.001k

20m
(6.6ft)

2.5m
(8.2ft)

LOAD

E = 2.16k

SEE ABOVE FOR Pd & Pl LOADS.

CHECKED LOAD COMBOS:
(1.2 + 0.2Sd5)D + 0.5E
(0.9 - 0.2Sd5)D + 0.5E

Sd5 = 0.19g

SEISMIC DESIGN CATEGORY B: E = 1.0

DESIGN FORCES

TOOK MAX FORCES FROM BOTH LOAD COMBINATIONS

1.86k(c)
1.27k(d)
0.918k(c)
0.918k(c)
0.918k(c)
0.918k(c)

12.4.2.3
12.4.1
ASCE 7-10

ETABS
SPACEFRAME B4 DESIGN

DESIGN OF B4
Pu = 1.80k (C), 1.29 k (T)

TRY HSS 3.00 x 0.25
Lc = 3.56 m = 11.65 ft

- COMPRESSION

k = 1.0, \( \frac{kL}{r} = \frac{1.0 \times (11.65 ft \times 12)}{0.942} = 142.4 \)

\[ \frac{F_t}{f_y} = 4.71 \times \frac{29,000 ksi}{42 ksi} = 123.8 \]

142.4 > 123.8

Fcr = 0.877 Fe

Ft = \( \frac{\pi^2 E}{(kL)^2} = \frac{\pi^2 (29,000 ksi)}{(142.4)^2} = 14.11 \text{ ksi} \)

Fcr = 0.877 (14.11 ksi) = 12.38 ksi

PnC = 12.38 ksi (2.03 in\(^2\)) = 25.13 k

\[ \phi Pn = 0.9 (25.13 k) = 22.6 k > 1.86 k \]

- TENSION

Pnt = Fy Ag = 42 ksi (2.03 in\(^2\)) = 85.26 k

\[ \phi Pn = 0.9 (85.26 k) = 76.7 k > 1.29 k \]

USE HSS 3.00 x 0.25 FOR B4

CHECK BOTTOM OF TRUSS

Pu = 0.918k (C)

Pn of A (22.12 x 2.12 x 2/8 FOUND IN TRUSS A DESIGN

\[ \phi Pn = 24.9 k > 0.918 k \]

CHECK COLUMN C2

Pu = 2.96 k

FROM COLUMN C2 DESIGN

Pu = 2.96 k + 9.28 k = 12.24 k < 248 k
BUILDING A CONFINED MASONRY DESIGN

ALL OF THE FOLLOWING CALCS IN THIS SECTION ARE BASED OFF OF:

SEISMIC DESIGN GUIDE FOR LOW-RISE CONFINED MASONRY BUILDINGS

CREATED BY THE CONFINED MASONRY NETWORK FROM EERI & IAEE. USED PARTS OF THIS GUIDE CAN BE FOUND IN APPENDIX.

MINIMUM WALL DENSITY INDEX, \( d \)

(TABLE 6)
- \( SDS = 0.19 \)  
- \( S_{di} = 0.7 \)  
- \( N = 1 \) STORY
- Soil Type: CLAY \( \rightarrow C \)
- Brick Type: Solid Clay Bricks (Mortar Type 1,11, \( \frac{3}{11} \))

\[ d_{min} = 1/0 \]

WHERE

\[ d = \frac{A_w}{A_p} \]

\[ A_p = 9m \times 10m = 90m^2 \]

\[ 0.01 = \frac{A_w}{90m^2} \]

\[ A_{w_{min}} = 0.9m^2 \]

\[ A_w = L_{wall} \times t \]

\[ t = 0.235m \]

\[ L_{min} = 3.83m \]
Karambo Micro-Industrial Complex  
Journeyman International  
Structural Design Calculations

Building A – East/West

**EAST**

\[
A_{\text{window}} = 1.25 \text{ m}^2
\]

\[
A_{\text{door}} = 2.15 \text{ m}^2
\]

**WEST**

**AREA OF OPENINGS**

**EAST:**

\[
A_{\text{window}} \times 3 = 3.75 \text{ m}^2 = A_{\text{ openings}}
\]

\[
A_{\text{wall}} = 10 \text{ m} \times 3 \text{ m} = 30 \text{ m}^2
\]

\[
\frac{3.75 \text{ m}^2}{30 \text{ m}^2} = 12.5\% \quad \text{**CONSIDEERED LARGE**}
\]

**WEST:**

\[
A_{\text{openings}} = (A_{\text{window}})(2) + A_{\text{door}} = 4.65 \text{ m}^2
\]

\[
\frac{4.65 \text{ m}^2}{30 \text{ m}^2} = 15.5\% \quad \text{**CONSIDEERED LARGE**}
\]

**LARGE OPENINGS → MIN WALL LENGTH L TO BE CONSIDERED IN WALL DENSITY**

\[
L > h/1.5 = \frac{3 \text{ m}}{1.5} = 2 \text{ m}
\]

**TOTAL WALL LENGTH CONSIDERED IN DIRECTION:**

\[
L = 2.47 + 2.86 = 5.33 \text{ m} > 3.88 \text{ m} (L_{\text{min}})
\]

**WALL DENSITY IS ADEQUATE IN E/W DIRECTION**
Building A – North/South

**NORTH**

**SOUTH**

**AREA of OPENINGS**

**NORTH:** NO OPENINGS

**SOUTH:**

Area of openings = 2.15 m²

\[
\frac{A_{\text{wall}} = 9 \text{m} \times 3 \text{m} = 27 \text{ m}^2}{2.15 \text{ m}^2} = 7.96 \%
\]

**CONSIDERED SMALL**

**IT DOES NOT NEED TO BE CONFINED**

**TOTAL WALL LENGTH CONSIDERED IN DIRECTION:**

L = 9.25 m + 7.10 m = 16.35 m > 3.83 m (Lmin)

**WALL DENSITY IS ADEQUATE IN N/S DIRECTION**
BUILDING B CONFINED MASONRY DESIGN

ALL OF THE FOLLOWING CALCULATIONS IN THIS SECTION ARE BASED OFF OF:

SEISMIC DESIGN GUIDE FOR LOW-RISE CONFINED MASONRY BUILDINGS

CREATED BY THE CONFINED MASONRY NETWORK FROM EERI \\ IABSE. USED PARTS OF THIS GUIDE CAN BE FOUND IN APPENDIX — —

MINIMUM WALL DESIGN INDEX, d

3.1.1.1, TABLE 6

• N = 1 STOREY

• SOIL TYPE C

• SOLID CLAY BRICKS (MORTAR TYPE I, II, III)

• PGA = 0.19 : MODERATE SEISMIC HAZARD

\[ d \geq 1.10 \times 0.01 \]

\[ d = \frac{A_w}{A_p} \quad \text{WHERE,} \quad A_w = \text{CROSS-SECTIONAL AREA OF ALL WALLS IN ONE DIRECTION} \]

\[ A_p = (9 \text{m})(15 \text{m}) = 135 \text{m}^2 \]

\[ 0.01 \geq \frac{A_w}{135 \text{m}^2} \quad \therefore A_w \quad \text{min} = 1.35 \text{m}^2 \]

\[ A_w = L_{\text{wall}} \times t_{\text{wall}} \quad \therefore L_{\text{wall}} \quad \text{min} = \frac{1.35 \text{m}^2}{0.235 \text{m}} = 5.74 \text{m} \]

\[ N-S \text{ DIRECTION} \]

\[ L_w = 9\text{m} + 9\text{m} = 18\text{m} > 5.74 \text{m} \checkmark \]

\[ d \geq 1.10 \checkmark \]
Building B Confined Masonry Design Cont'd

E-W Direction

\[ LW = 9 \text{ m} \]

\[ LW = 9 \text{ m} > 5.74 \text{ mV} \]
\[ d \geq 1\% \checkmark \]

Check Simple Building Requirements

1. a) Uniform Building Plan
   b) Symmetric Wall Layout
   c) Exterior Walls Extend Over at Least 50% of Length
   d) 75% of Building Weight Supported by CMW

2. a) \( H = 5.03 \text{ m} \leq 6 \text{ m} \)
   b) \( H/W = 5.03/15 = 0.33 < 1.5 \)
   c) \( L/W = 15/9 = 1.67 < 2.0 \)

3. Rigid Roof and Floor Diaphragms

4. Confined Masonry Walls
   a) Minimum Masonry Properties
   b) Solid Wall Panels Confined with Tie-Columns & Tie-Beams on All Four Sides
   c) Continuous Walls
   d) All Walls Built Using Same Properties

3.1.1.4

- \( t_{min} = 110 \text{ mm} < 235 \text{ mmV} \)
- \( H/t = 5.03 \text{ m} / 0.235 \text{ m} = 21.4 < 25 \checkmark \)
- Wall Panel 3m/3m = 1.0 > 0.5
Building B – East/West

**EAST**

**WEST**

\[ L = 9.25m \times 2 = 18.5m > 5.74m \ (L_{min}) \]

**TOTAL WALL LENGTH**

**CONSIDERED IN DIRECTION**: 

WALL DENSITY IS ADEQUATE IN E/W DIRECTION
Karambo Micro-Industrial Complex
Journeyman International
Structural Design Calculations

Building B – North/South

**NORTH**

**SOUTH**

AREA OF OPENINGS

**NORTH:** NO OPENINGS

**SOUTH:**
\[ \text{AOPENINGS} = \text{Awindow} + \text{Adoor} = 1.5 \text{m}^2 + 2.15 \text{m}^2 = 6.65 \text{m}^2 \]
\[ \text{Awall} = 5.90 \text{m} \times 3 \text{m} = 17.7 \text{m}^2 \]
\[ \frac{6.65 \text{m}^2}{17.7 \text{m}^2} = 0.37 \% ightarrow \text{LARGE OPENING} \]

LARGE OPENINGS \( \rightarrow \) MIN WALL LENGTH CONSIDERED IN WALL DENSITY:
\[ L > \frac{h}{l} \times 5 = \frac{1.5}{1.5} \times 5 = 2 \text{m} \] ✓

TOTAL WALL LENGTH CONSIDERED IN DIRECTION:
\[ L = 5.90 \text{m} + 15.20 \text{m} = 21.10 \text{m} > 6.74 \text{m} \] ✓

WALL DENSITY IS ADEQUATE IN N/S DIRECTION
BUILDING C CONFINED MASONRY DESIGN

ALL OF THE FOLLOWING CALCS IN THIS SECTION ARE BASED OFF OF:

SEISMIC DESIGN GUIDE FOR LOW-RISE CONFINED MASONRY BUILDINGS

CREATED BY THE CONFINED MASONRY NETWORK FROM EERI & IAEE. USED PARTS OF THIS GUIDE CAN BE FOUND IN APPENDIX.

MINIMUM WALL DENSITY INDEX C

(TABLE C)

- $S_{tr} = 0.19$ & $S_{br} = 0.7$ & MODERATE SEISMIC HAZARD
- $n = 1$ STORY
- SOIL TYPE: CLAY -> C
- BRICK TYPE: SOLID CLAY BRICKS (MORTAR TYPE I, II, III)

: $d_{min} = 1\%$

WHERE $d = \frac{A_w}{A_p}$ = AREA OF WALL IN CONSIDERED DIRECTION

$A_p = 12 \times 9 m = 108 m^2$

$: A_w = \frac{A_w}{108 m^2}$

$: A_{w_{min}} = 1.08 m^2$

$A_w = L_{WALL} \times t$

$\therefore t = 23.5 m$

$: L_{min} = 4.60 m$
Building C – East/West

**EAST**

\[ A_{\text{door}} = 2.16 \text{m}^2 \]

\[ 6.46 \text{m} \]

**WEST**

\[ A_{\text{window1}} = 1.8 \text{m}^2 \]
\[ A_{\text{window2}} = 1.13 \text{m}^2 \]
\[ A_{\text{window3}} = 2.25 \text{m}^2 \]
\[ 2.5 \text{m} \]

**AREA OF OPENINGS**

**EAST**

\[ A_{\text{opening}} = 2.16 \text{m}^2 \]
\[ A_{\text{wall}} = 9 \text{m} \times 3 \text{m} = 27 \text{m}^2 \]
\[ \frac{2.16 \text{m}^2}{27 \text{m}^2} = 7.96 \% \rightarrow \text{CONSIDERED SMALL,} \]
\[ \text{NO CONFINEMENT NEEDED} \]

**WEST**

\[ A_{\text{openings}} = (1.08 \text{m}^2)^2 + 1.13 \text{m}^2 + 2.25 \text{m}^2 = 7.14 \text{m}^2 \]
\[ \frac{7.14 \text{m}^2}{27 \text{m}^2} = 26.4 \% \rightarrow \text{CONSIDERED LARGE OPENING} \]

**LARGE OPENING → MIN WALL LENGTH, L, TO BE CONSIDERED IN WALL DENSITY \( \Rightarrow L > \frac{h}{1.6} = \frac{3}{1.6} = 1.9 \text{m} \)**

**TOTAL WALL LENGTH TO BE CONSIDERED IN DIRECTION**
\[ L = 2.5 \text{m} + 6.45 \text{m} = 8.95 \text{m} \]
\[ > 4.16 \text{m} \]

**WALL DENSITY IS ADEQUATE IN E/W DIRECTION**
Building C – North/South

**NORTH**

- Window = 1.6 m²

**SOUTH**

- Door = 2.15 m²

---

**Area of Openings**

**NORTH**: A_openings = 1.5 m² (3) = 4.5 m²

\[
\text{AWall} = 12 \text{ m} \times 9 \text{ m} = 36 \text{ m}²
\]

\[
\frac{4.5 \text{ m}²}{36 \text{ m}²} = 12.5\% \rightarrow \text{LARGE OPENING}
\]

**SOUTH**: A_openings = 2.15 m²

\[
\frac{2.15 \text{ m}²}{36 \text{ m}²} = 5.97\% \rightarrow \text{SMALL OPENING}
\]

NO CONFINEMENT NEEDED

**LARGE OPENING → MIN WALL LENGTH, L, TO BE CONSIDERED IN WALL DENSITY ⇒ L \frac{h}{k} = \frac{3}{1.5} = 2 \text{ m} √**

**TOTAL WALL LENGTH TO BE CONSIDERED IN DIRECTION:**

\[
L = 10\text{ m} + 5\times 1\text{ m} = 15\text{ m} > 4.50\text{ m} √
\]

**WALL DENSITY IS ADEQUATE IN N/S DIRECTION**
BOND BEAM DESIGN

Since the diaphragm for all buildings (A, B, & C) is flexible, some additional requirements must be met for the confined masonry, according to the:

Seismic Design Guide for Low-Rise Confined Masonry Buildings

By: EERI & IAEE

3.1.3

The resistance of confined masonry walls to out-of-plane seismic vibrations can be enhanced in one of the following ways:

a) Providing a rigid RC tie-beam at top of the wall
b) Providing an intermediate RC tie-beam at unperf/sill
c) Connecting the walls to the RC tie-columns through horizontal dowels

We will be doing a) through designing bond beams at the top of each wall & by doing b) throughout all three buildings.

Assumptions

• The bond beam in building B in the north/south direction will be subject to the highest lateral loading, thus the worst case.
BOND BEAM B1 DESIGN CONT'D

LOAD

WIND GOVERNS IN N/S DIRECTION, SO W = 18.46 PSF

\[ W_u = 18.46 \text{ PSF} \times \text{HEIGHT OF WALL} = \frac{10.6 \text{ ft} \times 18.46 \text{ PSF}}{1000 \text{ PSF/ft}} = 0.196 \text{ klf} \]

\[ V_u = 4.93k \quad M_u = 162k \text{ ft} \]

DESIGN ASSUMPTIONS

- \( f_y = 60 \text{ ksi}, \ f_{tc} = 3 \text{ ksi} \)
- SIMPLY SUPPORTED
- TENSION CONTROLLED

DESIGN FOR FLEXURE

TRY A 12 in x 12 in BEAM

\[ A_{s, \min} = \frac{2 + f_{tc}}{f_y} \text{ bw d} \quad \text{or} \quad \frac{200}{f_y} \text{ bw d} = \frac{200}{60,000} \times 12 \times 12 = 0.48 \text{ in}^2 \text{ - GOVERNS} \]

\[ = \frac{3}{12,000} \times 12 \times 12 = 0.39 \text{ in}^2 \]
BOND BEATN BY DESIGN CONT'D

\[ a = \frac{d - \sqrt{\frac{-2 \cdot 42}{0.85 + \frac{6\text{in}}{2} + d^2}}}{0.85 + \frac{6\text{in}}{2}} \]
\[ d = 12\text{in} - 1.5\text{in} - 0.375\text{in} = 10.125\text{in} \]

USE \( \phi = 0.9 \Rightarrow \) ASSUME TENSION CONTROLLED

\[ a = 10.125 - \frac{-2(62 \times 12)}{0.9(0.85)(3)(12)(12)} + 10.125^2 = 3.16\text{in} \]

\[ A_s = \frac{0.85 + \frac{6\text{in}}{2}}{60} \]
\[ = 0.85(3)(12)(3.61) \]
\[ = 1.61\text{in}^2 \geq 0.48\text{in}^2 \]

USE (3) \& 75 = 1.80\text{in}^2 \geq 1.61\text{in}^2

CHECK TO SEE IF TENSION CONTROLLED

\[ a = \frac{A_s \cdot f_y}{0.85 + \frac{6\text{in}}{2}} = \frac{1.20\text{in}^2(60\text{ksi})}{0.85(3)(12)\text{in}} = 3.53\text{in} \]

\[ c = \frac{a}{0.85} = \frac{3.53\text{in}}{0.85} = 4.15\text{in} \]
\[ \phi = \frac{0.65 + 0.25(0.009 - 0.0021)}{0.009 - 0.0021} \]
\[ = 0.81 \]

\[ a = 10.125 - \frac{-2(62 \times 12)}{0.85(0.85)(3)(12) + 10.125^2} = 3.61\text{in} \]

\[ A_s = \frac{0.85(3)(12)(3.61)}{60} \]
\[ = 1.80\text{in}^2 \leq 1.80\text{in}^2 \checkmark \]

DESIGN FOR SHEAR

\[ V_s = 4.93k \Rightarrow \text{ASSUME } V_s \leq A_f f_c bwd \]

\[ S_{max} = \frac{d}{2} = \frac{10.125}{2} = 5.06\text{in} \]

USE \#3 TIES ✔

\[ A_v = \frac{50\text{bns}}{f_y t} = \frac{50(12\text{in})(4\text{in})}{60,000} \]
\[ = 0.04\text{in}^2 \]

\[ A_v = 0.11\text{in}^2 \geq 0.04\text{in}^2 \checkmark \]
Bond Beam B1 Design Cont'd

\[ V_c = 2 \left( \frac{f_{pc}}{b_d} \right) = \frac{2 \cdot 3,000 \times (12)(10.125)}{1,000} = 13.31 k \]

\[ V_S = \frac{A_f y_{cd}}{s} = \frac{0.11 \times 12 (60)(10.125)}{1} = 10.71 k \]

\[ V_{S, \text{max}} = 6 \left( \frac{f_{pc}}{b_d} \right) = \frac{6 \cdot 3,000 \times (12)(10.125)}{1,000} = 53.24 k > 16.71 k \checkmark \]

\[ f_{Vn} = 0.75 \left( 13.31 k + 10.71 k \right) = 22.5 k > 4.93 k \checkmark \]

Use #3 C 4" O.C. Typ

Design Summary

- Use (3) #7s top & bottom
- Use #3 C 4 in. O.C.

\[ S_{\text{min}} = d_b = 0.875 in. < 3.62 in. \checkmark \]

\[ S_{\text{max}} = 15 \left( \frac{\ell}{8} \right) - 2.5 (1.5) = 11.25 in. \]

or

\[ 12 \left( \frac{30}{40} \right) = 12 in. > 3.62 in. \checkmark \]

Make sure this beam design still complies w/ tie-beam minimums

3.1.2.2

- \( A = 305 \text{mm} \times 305 \text{mm} \)
- Minimum Depth = 150mm ≤ 305mm \checkmark
- Minimum Width = \( t = 235 \text{mm} \leq 305 \text{mm} \checkmark \)

3.1.2.3

- (6) #7s > Min. Alongitudinal Bars #3 or Greater \checkmark
- #3 C 4 in. O.C. > Min #3 C 7.87 in. O.C. \checkmark
- 1.5 in. Cover > Min 0.80 in. Cover \checkmark
SILL & LINTEL BAND DESIGN

3.1.3.

OUT-OF-PLANE RESISTANCE OF CONFINED MASONRY WALL PANELS CAN ALSO BE ENHANCED BY PROVIDING INTERMEDIATE RC TIE-BEAMS ➔ SILL & LINTEL BANDS

MAX DISTANCE BETWEEN BANDS 120 cm ✓

[Diagram with dimensions: 255 mm, 27 mm, 76 mm, #3 TIE @ 7 in O.C., (2) #3 BARS]
Foundation Design
FOUNDATION F1 DESIGN

ASSUMPTIONS
- NEGLECT WEIGHT OF SOIL
- ALLOWABLE VERTICAL FOUNDATION PRESSURE = 2,000 SF

LOAD

FROM COLUMN C3 B/C THAT WILL YIELD THE LARGEST LOAD, THUS BEING THE WORST CASE A FOUNDATION DESIGNED WILL APPLY FOR ALL COLUMNS

\[ P_u = 9.24k + 31\#/ft (13.12\text{ft}) \times 1k/1000\# = 9.05k \]

DETERMINE FOOTING SIZE

SERVICE LOAD = 13.47k + 16.59k = 29.06k = 29,060\#  
\[ A_{req} = \frac{29,060\#}{2,000\text{PSF}} = 14.53\text{SF} \sqrt{14.53\text{SF} - 3.8\text{ft}} \rightarrow L = 4\text{ft} \]

USE 4ft x 4ft SQ. FOOTING

\[ q_u = \frac{9.05k}{165k} = 0.553 \text{kSF} \]
FOUNDATION F1 DESIGN CONT'D

**SHEAR**

\[
\text{TRY } t = 15\text{ in} \quad d = 15\text{ in} - 3\text{ in} - 0.5\text{ in} = 11.5\text{ in}
\]

\[
\text{TRIB} = 4\text{ ft} \left[ \left( 24\text{ in} - 9.13\text{ in}/2 \right) - 11.5\text{ in} \right]/12\text{ in}/\text{ft} = 2.65\text{ kSF}
\]

\[
\text{VU} = 0.603 \text{ kSF (2.65 SF)} = 1.60 \text{ k}
\]

\[
\phi\text{VH} = \phi(V_s + V_c) \quad V_s = 0 \quad \therefore \phi\text{VH} = \phi V_c
\]

\[
V_c = 2 \sqrt{\frac{F_c b d}{2}} = 2 \sqrt{13,000\text{ psi} \times 4\text{ ft} \times 12\text{ in}/\text{ft}} \times 11.5\text{ in} \times \frac{1\text{ k}}{1000\text{ ft}}
\]

\[
V_c = 60.5 \text{ k}
\]

\[
\phi\text{VH} = 0.75 \times 60.5\text{ k} = 45.4\text{ k} > 1.60\text{ k} \checkmark
\]

**PUNCHING SHEAR**

\[
\phi b_0 = 8.252 \text{ in}
\]

\[
\text{TRIB} = \left[ \left( 4\text{ ft} \times 4\text{ ft} \right) - \left( 9.13 + 11.5\text{ in} \right)^2 \right]/144 = 13\text{ SF}
\]

\[
\text{VU} = 0.603 \text{ kSF (13 SF)} = 7.8\text{ k}
\]

\[
V_c = 4\sqrt{F_c} - 4 \sqrt{3,000\text{ psi}} = 219\text{ psi} < \text{GOVERNS}
\]

\[
(2 + \frac{4}{4})\sqrt{F_c} = 6 \sqrt{3,000\text{ psi}} = 329\text{ psi}
\]

\[
(2 + \frac{\phi d}{b_0})\sqrt{F_c} = (2 + \frac{20(11.5)}{82.52}) \sqrt{3,000\text{ psi}} = 339\text{ psi}
\]

\[
V_c = 219\text{ psi} \times (82.52\text{ in})(11.5\text{ in}) \times \frac{1\text{ k}}{1000\text{ ft}} = 208\text{ k}
\]

\[
\phi\text{VH} = 0.75 \times 208\text{ k} = 156\text{ k} > 7.8\text{ k} \checkmark
\]

\[
t = 15\text{ in} \text{ WORKS}
\]

**FLEXURE**

\[
M_u = 0.603 \text{ kSF (4 ft)} \left( \frac{1.62\text{ ft}^2}{2} \right) = 3.17 \text{ k-ft}
\]

\[
a = d - \frac{-2M_u}{\phi 0.65 F_c b} + d^2 = 11.5 - \sqrt{\frac{-2(3.17 \times 12)}{0.9 \times 0.85 \times 3 \times 48} + 11.5^2}
\]

\[
a = 0.030 \quad \therefore \quad c = \frac{0.030}{0.85} = 0.036\text{ in}
\]
FOUNDATION #1 DESIGN CONT'D

\[ A_s = \frac{0.80 \gamma f'c b a}{f_y} = \frac{0.85(3)(48)(0.036)}{60} = 0.07 \text{ in}^2 \]

\[ A_s, \text{min} = 0.00 \text{ (Edw.d) } = 0.0018(48\text{in})(11.5\text{in}) = 0.99 \text{ in}^2 < \text{GOVERNS} \]

TRY (A) #5S \[ A_s = 1.24 \text{ in}^2 > 0.99 \text{ in}^2 \]

CHECK IF TENSION CONTROLLED

\[ \varepsilon_t = \frac{0.0033}{0.036} (11.5 - 0.036) = 0.9 > 0.005 \text{ V} \]

\[ S_{\text{max}} = 3h = 3(15\text{in}) = 45\text{in} < 18\text{in} < \text{GOVERNS} \]

USE #6S @ 12in 0.0. EACHWAY

CHECK SELF WEIGHT

SERVICE LOAD = 29,060# + 150 PCF (165F x 1.25F) = 32,060# \[ \frac{32,060\#}{100\text{SF}} = 2000 \text{ PSF} \geq 2000 \text{ PSF} \text{ V} \]

CHECK DEVELOPMENT LENGTH

\[ l_d = \left[ \frac{3}{40} f_y \right] \left[ \frac{\psi_t \psi_e \psi_s \lambda}{C_0 + \kappa_{\text{er}}} \right] d_b \]

CLEAR COVER = 3in

CENTER to CENTER BAR SPACING = \[ \frac{48\text{in} - 2(3\text{in})}{2(0.3125)} = 3.45\text{in} \]

\[ C_0 = 3\text{in} + 0.313\text{in} = 3.313\text{in} < \text{GOVERNS} \]

\[ \frac{3.45\text{in}}{2} = 1.73\text{in} \]

\[ \frac{C_0 + \kappa_{\text{er}}}{d_b} = \frac{3.13 + 1.0}{0.625} = 5 \]

\[ \psi_t = 1.0, \psi_e = 1.0, \psi_s = 0.8, \lambda = 1.0 \]

\[ l_d = \left[ \frac{3}{40} \right] \left[ \frac{60,000}{13,000} \right] \left[ \frac{1\times1\times1}{15} \right] 0.625 = 24.65\text{in} > 12\text{in} \text{ OK} \]

\[ \left( \frac{48}{2} - \frac{9.12}{2} - 3 \right) = 16.44\text{in} < 24.65\text{in} \text{ : ADD 90' Hook} \]
FOUNDATION F1 DESIGN CONT'D

DESIGN SUMMARY

- #9 C 12in O.C. TAB w/ STD HOOK
- #9s C 12in O.C. TAB w/ STD HOOK
FOUNDATION F2 DESIGN

THIS IS THE STRIP FOOTING FOR THE CONFINED MASONRY WALLE

ASSUMPTIONS

- $f'c = 3,000$ ksi
- $f_y = 60,000$ ksi
- PICK LARGEST LATERAL LOAD * LONGEST WALL TO PRODUCE MAXIMUM LOADS. THUS, THE DESIGN WILL BE FOR THE WORSE CASE SCENARIO * FOUNDATION WILL WORK FOR EVERY WALL * BUILDING
- NEGLECT WEIGHT OF SOIL

LOAD

$D_{max} = 0.945 \text{ KCF} \left[ \frac{9.29}{12} \times 45.3 + 9.84 \right] = 32.47 \text{ k}$

$D_{col} = 1.50 \text{ KCF} \left[ \frac{9.29}{12} \times 10.61 \times 6 \right] = 5.67 \text{ k}$

$D_{beam} = 1.50 \text{ KCF} \left[ 1.4 \times 50 \right] = 7.5 \text{ k}$

$D_{flo} = 1.50 \text{ KCF} \left[ (12 \times 9.29) \times 50 \right] + \left[ 1.4 \times 3.9 \times 50 \right] = 28.28 \text{ k}$

$L = 100 \text{ KSF}$

$E = 0.25 \text{ QF} + 0.2S_{ds}D \text{ WHERE } E=1.0, S_{ds} = 0.199, \text{ and } \text{ QF}=8.1 \text{ k}$

GOVERNING LOAD COMBOS:

$P_{max} = D + 0.7E = (1.0 + 0.2(0.19))(73.9) + 0.7(8.1) = 82.4 \text{ k}$

$P_{min} = 0.6D + 0.7E = (0.6 - 0.2(0.19))(73.9) + 0.7(8.1) = 47.2 \text{ k}$
FOUNDATION F2 DESIGN CONT'D

\( V = 8.1k \)
\( M_{ot} = 8.1k(1.5\text{ ft}) + 4.93k(10.66\text{ ft}) = 64.7\text{ k-ft} \)

SOIL VERIFICATION

\( q_{allow} = \frac{2000\text{ psf} \times 1.33}{1000\text{ ft}^2/\text{ k}} = 2.66\text{ ksf} \)

\( M_{ot} = 0.75(67.7\text{ k-ft}) = 48.5\text{ k-ft} \)

\[ P_{\text{min}} \]
\[ e = \frac{48.5\text{ k-ft}}{47.2\text{ k}} = 1.03\text{ ft} \]

\[ P \leq \frac{50\text{ ft}}{6} < 8.33\text{ ft} > 1.03\text{ ft} \]

ELASTIC, NO UPLIFT

\[ P_{\text{max}} \]
\[ e = \frac{48.5\text{ k-ft}}{82.4\text{ k}} = 0.586\text{ ft} \]

\[ P \leq \frac{50\text{ ft}}{6} < 8.33\text{ ft} > 0.586\text{ ft} \]

ELASTIC, NO UPLIFT

SHEAR

\[ \Phi V_n = \Phi V_c = 0.75(27 + 1.0) \text{ b} \times d = 0.75(2 + 3.000)(3\text{ in})(8.5\text{ in}) = 25.1\text{ k} \]

\[ d = 12\text{ in} - 3\text{ in} - 0.5\text{ in} = 8.5\text{ in} \]

\[ P_{\text{max}} \]

\[ q_{\text{max}} = \frac{P}{BL} \left(1 + 6 \frac{L}{L} \right) \]

\[ = \frac{47.2k}{(3\text{ ft})(50\text{ ft})} \left[1 + 6 \left( \frac{10.3\text{ ft}}{50\text{ ft}} \right) \right] \]

\[ = 0.354\text{ ksf} < 2.66\text{ ksf} \]

\[ q_{\text{max}} = \frac{P}{BL} \left(1 + 6 \frac{L}{L} \right) \]

\[ = \frac{82.4k}{(3\text{ ft})(50\text{ ft})} \left[1 + 6 \left( \frac{0.586\text{ ft}}{50\text{ ft}} \right) \right] \]

\[ = 0.586\text{ ksf} < 2.66\text{ ksf} \]

\[ q_{\text{max}} = \frac{0.586\text{ ksf}}{0.7} = 0.84\text{ ksf} \]

\[ V_n = \frac{1}{2} \left[ 0.84 + (0.84 - 0.91 \left( \frac{0.84}{2.59} \right) \right] (3)(0.4) \]

\[ = 0.95k < 25.1k \]
FOUNDATION P2 DESIGN CONT'D

\[ P_{\text{min}} = \frac{0.354 \text{ ksf}}{0.7} = 0.51 \text{ ksf} \]

\[ V_u = \frac{1}{2} \left[ 0.51 + (0.51 - 0.41) \left( \frac{0.51}{2.59} \right) \right] (3)(0.41) \]
\[ = 0.81 \text{ k} < 25.1 \text{ k} \checkmark \]

\[ \therefore \text{ NO SHEAR REINFORCEMENT REQUIRED} \]

FLEXURE

\[ P_{\text{max}} \]
\[ M_u = (3\text{ft})(1.11)^2 \frac{1}{2} \left[ 0.84 + (0.84 - 1.11) \left( \frac{0.84}{2.59} \right) \right] \]
\[ = 1.22 \text{ k-ft} \leq \text{GOVERNS} \]

\[ P_{\text{min}} \]
\[ M_u = (3\text{ft})(1.11)^2 \frac{1}{2} \left[ 0.51 + (0.51 - 1.11) \left( \frac{0.51}{2.59} \right) \right] \]
\[ = 0.74 \text{ k-ft} \]

\[ a = \frac{d - \frac{2M_u}{12.85f_{c}\phi}}{+d^2} = 8.5 - \frac{-2(1.22 \times 12)}{0.9 \times 0.85 \times 3 \times 36} + 8.5^2 \]

\[ a = 0.021 \text{ in} \]

\[ A_2 = \frac{0.85f_{c}\phi_{\text{ab}}}{f_y} = \frac{0.85(3)(0.021)}{60} = 0.032 \text{ in}^2 \]

\[ A_{\text{min}} = 0.0018 \text{ in} \times \text{in} = 0.0018(3\text{ ft})(0.5) = 0.55 \text{ in}^2 \leq \text{GOVERNS} \]

USE (A) #4s \[ A_5 = 0.80 \text{ in}^2 > 0.55 \text{ in}^2 \]

\[ s = \frac{3w - 0.5in}{2} = 7.38 \text{ in} \]

\[ S_{\text{max}} = \frac{18 \text{ in}}{15 \left( \frac{40,000}{2/3 (60,000)} \right) - 2.5(3) = 7.5 \text{ in}} \]
\[ = 12 \left( \frac{40,000}{2/3 (60,000)} \right) = 12 \text{ in} \]
**FOUNDATION F2 DESIGN CONT'D**

- **CHECK IF TENSION CONTROLLED**

\[ q = \frac{(0.80 \text{in}^2)(60)}{0.85(3)(36)} = 0.523 \text{in} \]

\[ c = \frac{0.523 \text{in}}{0.85} = 0.62 \text{in} \]

\[ \varepsilon_t = \frac{0.003}{0.62} (8.5 - 0.62) = 0.03870.005 \checkmark \]

USE #4 S. 9 in O.C. IN REST OF FOOTING.

**DESIGN SUMMARY**

**BASED ON CALCULATIONS & SEISMIC DESIGN GUIDE FOR LOW-RISE CONFINED MASONRY BUILDINGS**

- #3 TIE w/135° HOOK
- 61cm > 40cm
- 91.4cm > 50cm
- 15.24cm > 10cm
- 15.24cm
- 30.5cm
- (2) #4S
- (A) #4S
- #4S C 9 in O.C.
RETAINING WALLS

FOR BUILDING A:

RET WALL HEIGHT = 1.5m = 4.9 ft

→ 6" THICK REINFORCED CONCRETE WALL

USE 9.25" TO MATCH TIE COLUMNS

FOR BUILDING B:

RET WALL HEIGHT = 3.25m = 10.67 ft

→ USE 13" THICK

ALL RETAINING WALLS TO BE DESIGNED BY IN COUNTRY ENGINEER
Detail Design
BRACE TO TRUSS CONNECTION

WELD OR BOLT
CLEVIS CONNECTION
HSS ROUND BRACE
DOUBLE ANGLE

To be designed by in country engineer

BRACE TO HSS COL. CONNECTION

STEEL TRUSS
CAP PLATE
CLEVIS CONNECTION
HSS ROUND BRACE
WELD OR BOLT TO COL.
HSS SQUARE COL.

To be designed by in country engineer

BRACE TO WF COL. CONNECTION

WF COL.
WELD OR BOLT TO COL.
ROUND HSS BRACE
CLEVIS CONNECTION

To be designed by in country engineer
HSS Col. to Conc. Col. Connection

- HSS Column
- Anchor Bolts
- Steel Plate
- Grout
- Conc. Col.

To be designed by in Country Engineer

Truss to HSS Col. Connection

- Steel Gusset Plate
- Double Angle Bottom Chord
- Anchor Bolts
- Steel Cap Plate
- HSS Column

To be designed by in Country Engineer

Truss to WF Col. Connection

- Steel Gusset Plate
- Double Angle Bottom Chord
- Anchor Bolt
- Steel Cap Plate
- WF Col.

To be designed by in Country Engineer
A.0 Appendix
A.1 Appendix 1

Excerpt From VERCO Decking Catalog
Deep VERCOR™ Deck

- 1\(\frac{1}{16}\)" Deep Deck
- Galvanized

Dimensions

![Diagram showing dimensions of Deep VERCOR Deck](image)

**Deck Weight and Section Properties**

<table>
<thead>
<tr>
<th>Gage</th>
<th>Weight (psf)</th>
<th>Weight (lb/ft^2)</th>
<th>Single Span (in.(^2)/ft)</th>
<th>Multi Span (in.(^2)/ft)</th>
<th>+S(_{\text{eff}})</th>
<th>-S(_{\text{eff}})</th>
<th>End Bearing Length</th>
<th>Interior Bearing Length</th>
<th>End Bearing Length</th>
<th>Interior Bearing Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>26</td>
<td>1.1</td>
<td>0.075</td>
<td>0.075</td>
<td>0.099</td>
<td>0.103</td>
<td></td>
<td>492</td>
<td>572</td>
<td>639</td>
<td>829</td>
</tr>
<tr>
<td>24</td>
<td>1.4</td>
<td>0.097</td>
<td>0.097</td>
<td>0.137</td>
<td>0.138</td>
<td></td>
<td>802</td>
<td>927</td>
<td>1032</td>
<td>1366</td>
</tr>
<tr>
<td>22</td>
<td>1.7</td>
<td>0.120</td>
<td>0.120</td>
<td>0.172</td>
<td>0.171</td>
<td></td>
<td>1184</td>
<td>1361</td>
<td>1510</td>
<td>2029</td>
</tr>
<tr>
<td>20</td>
<td>2.1</td>
<td>0.143</td>
<td>0.143</td>
<td>0.204</td>
<td>0.204</td>
<td></td>
<td>1628</td>
<td>1864</td>
<td>2064</td>
<td>2807</td>
</tr>
</tbody>
</table>

**Notes:**
1. Section properties are based on F\(_y\) = 60,000 psi (specified minimum F\(_y\) = 80,000 psi).
2. I\(_d\) is for deflection due to uniform loads.
3. S\(_{\text{eff}}\) (+ or -) is the effective section modulus.
4. Allowable (ASD) reactions are based on web crippling, per AISI S100 Section C3.4, where \(\Omega_{\text{w}} = 1.70\) for end bearing and 1.75 for interior bearing. Nominal reactions may be determined by multiplying the table values by \(\Omega_{\text{w}}\). LRFD reactions may be determined by multiplying nominal reactions by \(\Phi_{\text{w}} = 0.90\) for end reactions and 0.85 for interior reactions.

**Attachment Patterns to Supports**

- 36/4 Inverted Position
- 36/5 Normal Position
- 36/8 Inverted Position
- 36/9 Normal Position

www.vercodeck.com VERCO DECKING, INC. VR4 145
Footnotes for Allowable Uniform Load Tables

2. L360, L240 or L180 = Uniform load which produces selected deflection in deck.
3. The symbol *** indicates allowable uniform load based on deflection exceeds allowable uniform load based on stress.
4. Nominal uniform loads governed by stress may be determined by multiplying the allowable values in the table by \( \Omega_b = 1.67 \).
   LRFD loads may be determined by multiplying nominal loads by \( \Phi_b = 0.95 \).

Footnotes for Diaphragm Shear Strength and Flexibility Factor Tables

General Notes
1. #10 = #10 Generic Screw. Sidelap connections are not required at support locations.
2. The dimension from the first and last sidetap connection within each span is to be no more than one-half of specified spacing.
3. R is the ratio of vertical span \( (L_v) \) of the deck to the length \( (L_s) \) of the deck sheet: \( R = L_v / L_s \).
4. Interpolation of diaphragm shear strength between adjacent spans or sidetap spacings is permissible. For interpolation of the diaphragm flexibility factor between adjacent spans, use the flexibility factor for the closest adjacent span length.
5. Diaphragm shear values for side seam fasteners placed at spacings other than those in the table should be determined based on the number of fasteners in each span.
6. The allowable diaphragm shear values in the tables utilize a factor of safety, \( \Omega = 2.5 \) (limited by connections) with the exception of the gray shaded table values, which utilize a factor of safety of \( \Omega = 2.0 \) (limited by panel buckling).
7. Deck is attached with minimum #12 Screws (self drilling, self tapping) to supports. Select appropriate screw based on actual substrate thickness. This table is provided as a guide, proper selection should be verified based on the specific fasteners used.

<table>
<thead>
<tr>
<th>Support Thickness</th>
<th>Fastener Designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>33 mil (0.0346&quot;) to 3/16&quot;</td>
<td>#3 Drill Point</td>
</tr>
<tr>
<td>1/8&quot; to 1/4&quot;</td>
<td>#4 Drill Point</td>
</tr>
<tr>
<td>1/8&quot; to 1/2&quot;</td>
<td>#5 Drill Point</td>
</tr>
</tbody>
</table>

8. All tabulated diaphragm values shown in this section are for a minimum 0.0385 in thick support with SDI recognized screws produced by Buildex, Elco, Hill or Simpson Strong-Tie. If the minimum support thickness can not be met or a screw that is not recognized by SDI is used, modify tabulated q and F values based on actual substrate and thickness using Adjustment Factors listed in the following tables.

For 9/16" (Shallow) VERCOR:

<table>
<thead>
<tr>
<th>Deck Gage</th>
<th>Factors</th>
<th>Substrate Thickness and Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>26</td>
<td>R_q</td>
<td>0.66 0.69 0.69 0.69 0.69 0.69 0.69 0.69 0.69 0.69 0.69 0.69</td>
</tr>
<tr>
<td></td>
<td>R_F</td>
<td>1.26 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00</td>
</tr>
<tr>
<td>24</td>
<td>R_q</td>
<td>0.52 0.66 0.68 0.69 0.69 0.69 0.69 0.69 0.69 0.69 0.69 0.69</td>
</tr>
<tr>
<td></td>
<td>R_F</td>
<td>1.51 1.38 1.27 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00</td>
</tr>
<tr>
<td>22</td>
<td>R_q</td>
<td>0.38 0.54 0.59 0.69 0.69 0.69 0.69 0.69 0.69 0.69 0.69 0.69</td>
</tr>
<tr>
<td></td>
<td>R_F</td>
<td>1.69 1.58 1.36 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00</td>
</tr>
</tbody>
</table>

For 1-5/16" (Deep) VERCOR:

<table>
<thead>
<tr>
<th>Deck Gage</th>
<th>Factors</th>
<th>Substrate Thickness and Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>26</td>
<td>R_q</td>
<td>0.69 0.74 0.74 0.74 0.74 0.74 0.74 0.74 0.74 0.74 0.74 0.74</td>
</tr>
<tr>
<td></td>
<td>R_F</td>
<td>1.13 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00</td>
</tr>
<tr>
<td>24</td>
<td>R_q</td>
<td>0.58 0.70 0.73 0.75 0.75 0.75 0.75 0.75 0.75 0.75 0.75 0.75</td>
</tr>
<tr>
<td></td>
<td>R_F</td>
<td>1.21 1.17 1.13 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00</td>
</tr>
<tr>
<td>22</td>
<td>R_q</td>
<td>0.48 0.61 0.65 0.75 0.75 0.75 0.75 0.75 0.75 0.75 0.75 0.75</td>
</tr>
<tr>
<td></td>
<td>R_F</td>
<td>1.27 1.24 1.16 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00 1.00</td>
</tr>
<tr>
<td>20</td>
<td>R_q</td>
<td>0.39 0.53 0.57 0.71 0.71 0.76 0.76 0.76 0.76 0.76 0.76 0.76</td>
</tr>
<tr>
<td></td>
<td>R_F</td>
<td>1.32 1.33 1.25 1.21 1.16 1.00 1.00 1.00 1.00 1.00 1.00 1.00</td>
</tr>
</tbody>
</table>

9. Adjustment factors are based on connection strengths determined in accordance with Section E4 of AISI S100. These self drilling, self tapping screws must be compliant with ASTM C1315.
10. Allowable Diaphragm Strength = q • R_q, Flexibility Factor = F • R_F.
11. These adjustment factors are based on the maximum adjustment for the tabulated span lengths and fastener patterns.

To calculate a specific condition, use design equations listed at the end of Evaluation Report ER-0217.
A.2 Appendix 2

Article: *Comparative Analysis of Seismic Loading on High-Rise Steel Building Structures in Bujumbura and Kigali Cities*
COMPARATIVE ANALYSIS OF SEISMIC LOADING ON HIGH-RISE STEEL BUILDING STRUCTURES IN BUKUMBURA AND KIGALI CITIES

NDIHOKUBWAYO Athanase ¹, JIANG Cangru ² and CHEN Zhihua ³

¹ Doctorate Student, ² Professor, ³ Asso.Professor,
Department of Civil Engineering and Architecture,
Wuhan University of Technology, Wuhan, China
E-Mail: athanand@yahoo.com

ABSTRACT:

The seismic loading analysis is very important for designing high-rise buildings in the most seismic ground motions zone. The objective of this study was to compare some seismic loading forces on two similar high-rise building structures located in Bujumbura city (Capital of Burundi) and Kigali city (Capital of Rwanda); using ETABS software. These two cities are located in the Western side of the East African rift system; known as the African most seismic ground motions zone. In the present study, all the seismic design data were the same for the two buildings except their seismic site spectral response acceleration $S_s$ and $S_1$. The result of the study showed that the average of the seismic base shear forces and the seismic stories lateral forces on the building located in Bujumbura city is ranged between 2.3 and 2.4 times greater than the average of these seismic loading forces on the building located in Kigali city.

KEYWORDS: East African rift system, $S_s$ and $S_1$, high-rise steel building, seismic base shear forces, seismic stories lateral forces.

1. INTRODUCTION

The East African region is often shaken by the earthquake principally because of the presence of the rift valley. The earthquake in that region which covers about 5.5 million km² and holds more than 120 million people has been identified as the major threat.¹ Thus, with major population growth and urbanization increasing, the vulnerability to the earthquake hazards has greatly increased.² For facing simultaneously to the urbanization target and the earthquake threat, it is very important to construct the high-rise buildings with high seismic design consideration. The vulnerability of the East African populations to seismic events has been underscored by a study which advised that the region’s capacity in earthquake preparedness and hazards mitigation need to be improved significantly.³ In the present study the seismic base shear forces and the seismic stories lateral forces acting on each building structures (20 stories) were calculated and a comparison analysis of these forces was done for the two buildings.
As the seismic spectral response acceleration (Ss and S1) are known for the two building construction sites, the International Building Code \(^4\) and other design references was helpful to determine the input data for the seismic loading design. The Extended Three Dimensional Analysis of Building Systems (ETABS) software \(^5\) was used to perform automatically the seismic loading design process. Bujumbura city which has high values of seismic spectral response acceleration than Kigali city is the sixth African city with high seismic spectral response acceleration Ss and S1 after Algier (Algeria), Tunis (Tunisia), Djibouti (Djibouti), Bukavu (Congo Democratic) and Cairo (Egypt)\(^6\). These factors are very decisive for seismic building design process on any given construction site. The result of the seismic loading comparison showed that the average of the seismic loading base shear forces and seismic stories lateral forces on building located in Bujumbura city is ranged between 2.3 and 2.4 times greater than the average of the seismic loading base shear forces and seismic stories lateral forces on building located in Kigali city.

### Table 1  Seismic loading design data

<table>
<thead>
<tr>
<th>Seismic design characteristics</th>
<th>Seismic design values</th>
<th>Bujumbura Building</th>
<th>Kigali Building</th>
</tr>
</thead>
<tbody>
<tr>
<td>Occupancy Category and Seismic Use Group, SUG(^7)</td>
<td>I</td>
<td>I</td>
<td></td>
</tr>
<tr>
<td>Seismic Importance Factor, I (^8)</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Seismic Site Class (^9)</td>
<td>B</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>Mapped 0.2 sec. Period Spectral Acceleration, Ss (^10)</td>
<td>0.66</td>
<td>0.28</td>
<td></td>
</tr>
<tr>
<td>Mapped 1.0 sec. Period Spectral Acceleration, S1 (^11)</td>
<td>0.26</td>
<td>0.11</td>
<td></td>
</tr>
<tr>
<td>Acceleration-based Site Coefficient, Fa (^12)</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Velocity-based Site Coefficient, F1 (^13)</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Maximum Spectral Response Acceleration, SMS (^14)</td>
<td>SMS = Fa x Ss = 0.66</td>
<td>SMS = Fa x Ss = 0.28</td>
<td></td>
</tr>
<tr>
<td>Maximum Spectral Response Acceleration, SM1 (^15)</td>
<td>SM1 = F1 x S1 = 0.26</td>
<td>SM1 = F1 x S1 = 0.11</td>
<td></td>
</tr>
<tr>
<td>Design Spectral Response Acceleration, SDS (^16)</td>
<td>SDS = 2/3 SMS = 0.44</td>
<td>SDS = 2/3 SMS = 0.19</td>
<td></td>
</tr>
<tr>
<td>Design Spectral Response Acceleration, SD1 (^17)</td>
<td>SD1 = 2/3 SM1 = 0.17</td>
<td>SD1 = 2/3 SMS = 0.07</td>
<td></td>
</tr>
<tr>
<td>Seismic Design Category, SDC (^18)</td>
<td>SDC = C (SUG) = I</td>
<td>SDC = B (SUG = I)</td>
<td></td>
</tr>
<tr>
<td>Seismic Response Modifier, R (^19)</td>
<td>R = 8</td>
<td>R = 8</td>
<td></td>
</tr>
</tbody>
</table>

2. DESIGN SPECTRAL RESPONSE ACCELERATION FOR THE TWO CITIES

On the Fig.1 below, the response times To and Ts needed for constructing response spectra curves for the two cities are calculated as follows:

\[
To = 0.2 \frac{SD1}{SDS}; \quad Ts = \frac{SD1}{SDS} \quad (2.1)
\]

where To is the short period ground motion range and Ts is the characteristic period of ground motion.
For the building located in Bujumbura city: \( T_0 = 0.08 \); \( T_s-B = 0.39 \); \( SD-B = 0.17 \); \( SDS-B = 0.44 \)
For building located in Kigali city: \( T_0 = 0.08 \); \( T_s-K = 0.37 \); \( SD1-K = 0.07 \); \( SDS-K = 0.19 \)

3. SEISMIC BASE SHEAR FORCES (V)

The seismic base shear forces is given by the following equation:

\[
V = Cs \times W \tag{3.1}
\]

where \( Cs \) is the seismic design coefficient and \( W \) is the building reactive weight including cladding.

\[
Cs = \frac{SDS}{(R/I)} \geq \frac{SD1}{(TR/I)} \geq 0.044 SDS \times 1 \tag{3.2}
\]

The fundamental period \( T \) and the Seismic Response Modifier \( R \) values are respectively equal to 2 and 8.
For building located in Bujumbura city, \( Cs = 0.044 \times 0.44 \times 1 = 0.01936 \).
For building located in Kigali city, \( Cs = 0.044 \times 0.19 \times 1 = 0.00836 \).

Fig. 2 Seismic Base Shear Forces Result
4. SEISMIC STORIES LATERAL FORCES (Fx)

The equation for the seismic stories lateral forces calculation is:

\[ F_x = C_{vx} \times V \]  \hspace{1cm} (4.1)

where: \( C_{vx} \) is the vertical distribution factor and \( V \) is the shear forces.

\[ C_{vx} = \frac{W_x (h_x^k)}{\sum_{i=1}^{n} W_i (h_i^k)} \]  \hspace{1cm} (4.2)

where: \( W_i \) and \( W_x \) are respectively portion of the total gravity load \( W \) assigned to the level \( i \) and \( x \); \( h_i \) and \( h_x \) are respectively the height from the base to level \( i \) and \( x \); \( k \) is the exponent related to the building period \( T \) and takes into account the whiplash effects in tall slender buildings.

\[ T = C_t (h_x)^{3/4} \]  \hspace{1cm} (4.3)

where: \( h_x \) is the total height (in feet) of the building and \( C_t \) is a period coefficient.

\[ k = \begin{cases} (2-1) \times (T-0.5) + 1 & \text{for} \ 0.5 \text{ sec} < k < 2.5 \text{ sec} \\ (2.5-0.5) & \text{for} \ 2.5 \text{ sec} \end{cases} \]  \hspace{1cm} (4.4)

The value of \( k \) is equal to 1.75 for the two buildings.

![Fig. 3 Seismic Stories Lateral Forces Result](image)

5. CONCLUSION

After analysis and calculation of the seismic base shear and the seismic stories lateral forces acting on the two buildings as illustrated on the figures 2 and 3, we realized that the average of these loading forces on building located in Bujumbura city is ranged between 2.3 and 2.4 times greater than the average of the same loading forces acting on the building located in Kigali city. The highest seismic loading forces for building located in Bujumbura city is given especially by the highest values of its site seismic spectral response acceleration \( S_s \) and \( S_1 \) comparatively to \( S_s \) and \( S_1 \) for building site located in Kigali city.
The earthquake shakes in the region had already caused humans and animals life losses and a lot of infrastructure injuries. Thus, the decision makers in matters of political policies, engineering and planning professionals need to understand the nature of the hazardous phenomena and take all preventive measures against the earthquake threat. The decision can be taken at three levels of commitment to implement mitigation and preparedness. These three levels of commitment are the development knowledge, public awareness raising and education, preparedness investments. High-rise building construction can be a sustainable solution for development in the countries that are over-populated and the system can also create sufficient space in urban area for other development infrastructures. More researches are needed to enrich this study in order to contribute to the urban development without earthquake threat in East African rift system.

REFERENCES

[6] UFC3-310-01. (2005). Earthquake loading data at additional locations outside of the United States, its territories and possessions (Tab E-1), USA
[10] International Building Code. (2003). Mapped 0.2 Sec Period Spectral Acceleration, Ss (Fig.16151). Berkeley, USA.


[20] ASCE7, Section 9.5.5.3 (2002). Fundamental period of structural. New York, USA.

[21] ASCE7, Section 9.5.5.2 (2002). Seismic Base Shear. New York, USA.


[23] ASCE7, Section 9.5.5.4 (2002). Lateral Force at any level. New York, USA.


[25] ASCE7, Section 9.5.5.3.2 (2002). Fundamental Period. New York, USA.
A.3 Appendix 3

Soils Report on for Neighboring Project
FEASIBILITY STUDY
Vol III Geological Report
Karambo II Hydropower Plant (HPP)

Publication & Revision Tables
Date of Final Publication: April, 2015

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<th>Made by</th>
<th>Contact Details</th>
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</thead>
<tbody>
<tr>
<td>Adina Paraschivescu Enterprise</td>
<td>+250 783 420 423</td>
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<th>Rev. Date</th>
<th>Details</th>
<th>Revised By:</th>
</tr>
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</table>
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1. INTRODUCTION

In order to assess the relevant geological conditions prevailing at the sites of all major structures of Karambo II HPP, the consultant has carried out field investigations that involve mapping of the project area and logging of test pits.

The findings have a bearing on the feasibility of this proposed hydro power project with regard to structural aspects, competence of rock and soil formations as well as availability of materials for construction.

This project is located a few kilometers west of the proposed Bihongora hydro power project also in Kanama sector, Rubavu district. The landscape at the two project sites is somewhat similar with undulating hills and valleys that are a perfect setting for creation of sufficient water head for power generation.

The consultant has conducted field investigations that involve mapping of the project area and logging of test pits so as to discern the geological conditions prevailing at the project site area.

2. REGIONAL GEOLOGY

The geology of Rwanda has formations of mainly sedimentary and metamorphic rocks that consist of granite, migmatites, gneisses and mica schists of the paleoproterozoic Ruzizian basement overlain by the Mesoproterozoic Kibaran belt formations of the Burundian sub-group. These rock formations can be classified as either politic comprising of mainly shales, schists and para-gneisses; or Arenaceous comprising of conglomerates, sandstones and quartzites.

Magmatic rocks which are mainly granites and gneisses include mafic plutonic rocks and basalts. In terms of abundance, granites and gneisses are the first followed by mafic plutonic rocks and basalts are the last. Granites are mainly gneissic and are observed in the south, central and partly towards the west and are occasionally foliated with some meta-sediments. Mafic plutonic rocks of significant extent are confined in the central part while small granitic bodies are observed near these plutonic mafic rocks and are outcropping throughout the country.

Generally, in the east the country is predominated by older granite gneisses while neogene volcanics are found in the north western and south western parts. Young alluvial deposits and lake sediments are a common occurrence along rivers. Basaltic flows of significant thicknesses are observed in the south-west towards Lake Kivu and at the DR Congo and Burundi borders. They are considered to originate from south Kivu volcanic province in DR Congo. Also, thin inter layers of mafic and acidic lavas are observed within the typical Burundian meta-sediments.

Faulting sequences observed in the field and as per the main features of the broad landscape in terms of morphology of Rwanda has led to formations of conglomerates, sandstones, shales in the east while in the west the rift valley formation has led to the creation of successive host and graben systems where Lake Kivu is and southwards.
3. GEOLOGY OF KARAMBO II HPP AREA

3.1 Topography

*Landscape*

The landscape around the proposed hydro power project is hilly and undulating with the river flowing in the valley section. This reflects past tectonic earth movements related to rift valley formation. The river has cut a gorge in the valley about 1.5m deep. The left and right banks of the river stand at about the same height at the proposed intake point but the right bank stands progressively higher in the desander section (from intake to forebay).

Pitting findings at the proposed power house zone indicate presence of an intermittent flood plain where the river breaks its banks and pours out silt, sand and pebbles onto the surrounding land during heavy rain season.

![Figure 1 Land around the proposed intake dam](image)

*Figure 1 Land around the proposed intake dam*
3.2 Vegetation Cover

Green vegetation occurs on the river banks mostly short to medium size grass and shrubs. Beyond the river banks is forest cover, mostly eucalyptus trees, on the right hand side bank and short to medium size grass and scattered trees on the left hand side bank. This vegetation cover limits erosion of soil into the river. However, around the proposed power house zone there is scattered vegetation cover on the left hand side river bank exposing soil that keeps eroding into the river. That’s why water at this river point is not clear but brown in colour, unlike the water at the intake which is clear.

Figure 2 Vegetation cover on the river banks around the proposed intake dam
3.3 Rocks and Soils

Large boulders of basalt rock and granite metamorphosing into gneisses lie at the river base. Occurrence of these igneous rocks is indicative of past volcanic activity in this zone. Quartzites and mica schist are present but on a small scale. Pebbles, silt and sandy material line the base of the river and migmatites predominantly form the sides and bottom of the river.

At the proposed intake point, water is clear possibly due to filtration by these materials lining the river base. However, the water is of brown colour at the proposed power house point because of the exposed reddish brown soil (limonite) on the left hand side river bank and gray to brown clay soils on the right hand side river bank. At the proposed intake dam, there is a fault structure in the rock formations on the right hand side bank that allows water from the hill to flow underground into the river (see test pit no. 2 log).

![Figure 3 Rock boulders at power house zone](image)
3.4 Site Test Pits

Six pits of mostly 1.2m depth were dug to reveal the soil and rock formations underground. Logging of these holes was done and snap shots taken.

Findings are presented in the appendices.

3.5 Construction materials

Near the project site on the road side are scattered micro quarries producing stone hard core and hand-crushed stone aggregate of mainly quartzite material. These stones can be used in the construction of intake dam and associated desilting plant, water canal, forebay for buffer water stock and power house.

Also accessible, not far away from project site, are volcanic rocks that are perfect for lining the canal.

Sand is available in and around the project area in the river.

4. CONCLUSION

The water supply to the proposed intake dam is sustainable as it originates from two independent sources i.e Karambo River itself and the underground stream joining this river from the right hand side bank. So, faulting in the rock structure that created this stream is of advantage to this proposed project.

The water at the intake point is clear requiring no filtration but obviously a desilting plant is still a necessity.

The geomorphology at the right hand side bank offers better potential for realizing sufficient water head for the hydro power project.

Construction materials, sand and stone aggregate, are available close to the project site. This project site is easier to access than the proposed Bihongora HPP site.

5. RECOMMENDATION

The consultant recommends setting up the hydro power plant on the right hand side of the river flow. This includes the intake dam, desilting plant, water conveyance system, forebay and power house.

When constructing the foundation of the power house, it must be taken into account that it is positioned in a seasonal flood plain.
APPENDIX 1: Log results of test pits at proposed Intake Dam

<table>
<thead>
<tr>
<th>Test Pit No. 1:</th>
<th>Intake Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location:</td>
<td>RHS US 4m from river</td>
</tr>
<tr>
<td>Log:</td>
<td>Loam soil</td>
</tr>
<tr>
<td>0.00 – 0.20m</td>
<td>Decomposed rock</td>
</tr>
<tr>
<td>0.20 – 0.80m</td>
<td>Decomposed rock embedded in clay soil</td>
</tr>
<tr>
<td>Note:</td>
<td>RHS = Right Hand Side, US = Up Stream</td>
</tr>
<tr>
<td>Pic 3:</td>
<td>Test Pit No. 1</td>
</tr>
</tbody>
</table>
**Test Pit No. 2:** Intake Dam

**Location:** RHS  DS  4m from river

**Log:**

**0.00 – 0.10m**  Loam soil

**0.10 – 0.30m**  Sandy material in clay groundmass.

**At 0.30m (Bottom)**  Foliated granitic rock boulders (small) in clay soil under water.

*Water percolates through this pit into the river. This could be due to occurrence of geophysical faulting in the rock structure that allows dissemination of water underground.*

**Note:**  DS = Down Stream

**Pic 4:**  Test Pit No. 2
<table>
<thead>
<tr>
<th>Test Pit No. 3:</th>
<th>Intake Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location:</td>
<td>LHS US 4.4m from river</td>
</tr>
<tr>
<td>Log:</td>
<td>0.00 – 0.80m Clay soil</td>
</tr>
<tr>
<td></td>
<td>0.80 – 1.10m Decomposed rock and small boulders of granite rock embedded in clay groundmass.</td>
</tr>
<tr>
<td>Note:</td>
<td>LHS = Left Hand Side</td>
</tr>
<tr>
<td>Pic. 5:</td>
<td>Test Pit No. 3</td>
</tr>
<tr>
<td>Test Pit No. 4</td>
<td>Intake Dam</td>
</tr>
<tr>
<td>---------------</td>
<td>------------</td>
</tr>
<tr>
<td><strong>Location:</strong></td>
<td>LHS  DS  3.4m from river</td>
</tr>
<tr>
<td><strong>Log:</strong></td>
<td></td>
</tr>
<tr>
<td><strong>0.00 – 0.50m</strong></td>
<td>Clay soil</td>
</tr>
<tr>
<td><strong>0.50 – 1.20m</strong></td>
<td>Decomposed loose rock material in clay soil</td>
</tr>
<tr>
<td><strong>Pic. 6:</strong></td>
<td>Test Pit No. 4</td>
</tr>
</tbody>
</table>

![Image of Test Pit No. 4]
APPENDIX 2: Log Results of Test pits at proposed power house

<table>
<thead>
<tr>
<th>Test Pit No.</th>
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<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>RHS</td>
<td>Power House</td>
</tr>
<tr>
<td></td>
<td>DS</td>
<td>12m from river</td>
</tr>
</tbody>
</table>

**Log:**

- **0.00 – 0.20m** Loam soil
- **0.20 – 0.40m** Pebbles embedded in clay soil, compacted.
- **0.40 – 1.20m** Sandstone

**Pic. 7:** Test Pit No. 5
A.5 Appendix 5

Structural Construction Documents – Journeyman International Deliverable
KARAMBO MICRO-INFUSTRUSTRIAL COMPLEX
STRUCTURAL DRAWINGS

SITE:
MAHOKO VILLAGE (KANAMA SECTOR), RUBAVU DISTRICT, RWANDA
37 25.818' N
122 05.36' W

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<tbody>
<tr>
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<td>GENERAL NOTES</td>
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<tr>
<td>S1.10</td>
<td>TYPICAL DETAILS</td>
</tr>
<tr>
<td>S2.10</td>
<td>BUILDING A FOUNDATION PLAN</td>
</tr>
<tr>
<td>S2.11</td>
<td>BUILDING A FRAMING PLAN</td>
</tr>
<tr>
<td>S2.12</td>
<td>BUILDING A ROOF FRAMING PLAN</td>
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<tr>
<td>S2.20</td>
<td>BUILDING B FOUNDATION PLAN</td>
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<td>S2.21</td>
<td>BUILDING B FRAMING PLAN</td>
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<td>S2.22</td>
<td>BUILDING B ROOF FRAMING PLAN</td>
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<td>S2.30</td>
<td>BUILDING C FOUNDATION PLAN</td>
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<td>S2.31</td>
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<td>S2.32</td>
<td>BUILDING C ROOF FRAMING PLAN</td>
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<tr>
<td>S3.00</td>
<td>ELEVATIONS</td>
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<tr>
<td>S3.10</td>
<td>ELEVATIONS</td>
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<tr>
<td>S4.00</td>
<td>SCHEDULE AND DETAILS</td>
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<td>S5.00</td>
<td>CONFINED MASONRY DETAILS</td>
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Author: Stella Bates
Checked By: Sophia Abshire
Plot Date: 06/15/2018
Client: Empowering Villages

Journeyman International
Project Title: Karambo Micro-Industrial Complex
Site: Mahoko Village, Rubavu District, Rwanda

S0.10
GENERAL NOTES

1. All reinforcing bar bends shall be made cold.
2. All splices are to be Class B unless specifically noted otherwise.
3. Where not specifically detailed, the minimum concrete cover on reinforcing steel shall be
   A. Concrete against earth fault earth pipe: 
      6.0 cm
   B. Concrete against earth pipe: 
      4.0 cm
   C. Beams, girders & columns, not exposed to earth or water: 
      3.0 cm
   D. Reinforcing bars larger than #8 are not permitted unless specifically detailed or noted otherwise.

GROUT

1. Masonry units shall have a minimum compressive strength (f'/m) of 4 MPa.

MASONRY

1. Mortar shall be a minimum of M200 and shall be mixed in a mechanical mixer or a portable cement mixer and shall be spread by float. Where possible, a mechanical mixer shall be used.

PLACING CONCRETE

1. All concrete shall be placed in the order shown on the drawings, but the sequence may be changed. All workmen shall spread the concrete as uniformly as possible. All workmen shall be provided with tools and equipment to ensure a uniform spread.
2. All concrete shall be placed in one lift.
3. All concrete shall be placed in a single phase, and all workmen shall be provided with tools and equipment to ensure a uniform spread.

10. Vibrate all concrete (including slabs on grade) as it is placed. The vibrator shall be used to consolidate the concrete, not transport it. Re-inforcing bars and forms shall not be vibrated.

ELEVATED BEAMS AND SLABS:

1. The Contractor may use concrete admixtures or a concrete mix and methods to achieve specified concrete strengths. Use of admixtures is subject to the Engineer's approval.

CONCRETE

1. All structural steel shall be painted one shop coat and field touched up.

10. All soils report was produced from Empowering Villages for a nearby project and it was determined that the soils will be used.

11. Based on the above information and IBC 2015 Table 1906.2, the following values were used:

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Vertical Foundation Pressure</th>
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</thead>
<tbody>
<tr>
<td>Loam Soil</td>
<td>95.76 kN/m²</td>
</tr>
</tbody>
</table>

12. Allowable Vertical Foundation Pressure = 95.76 kN/m²

13. All reinforcing bars are to be Class B unless specifically noted otherwise.

14. Steel decking shall be cold rolled steel conforming to ASTM A611, Grade C structural steel, and have a minimum yield strength (Fy) of 344 MPa (50 ksi)

15. Steel decking shall be placed on the supporting framework in a minimum end lap of 500 mm (20") from the support.

16. Steel roof deck shall be welded to the supports and lap of adjacent units welded. Steel roof deck shall be welded to furring (size & type as manufacturer requirements).

17. These structural "Contract or Construction Documents" shall not be modified without the prior written approval of the Engineer of Record.
38 mm CLR TYP

REBAR FOR SUB BASE UNDER SLAB
SEE GEOTECHNICAL REPORT AND SPECS TYP

3.18mm WIDE x T/4 DEEP
PRE FORMED STRIP OR SAW CUT THE SAME DAY OF THE POUR

DEPRESS REINFORCEMENT UNDER JOINT AS REQUIRED TO CLEAR STRIP OR SAW CUT

CUT 1/2 REINF. ACROSS CONTROL JOINT
SEE PLAN FOR SLAB REINF. TYP, UNO

1. IF SAW-CUT CONTROL JOINT TO BE USED, SAW-CUT WITHIN 24 HOURS OF POUR.
2. SEE PLAN FOR T.

NOTE:
STANDARD HOOK DETAILS FOR PRINCIPAL REINFORCEMENT
STANDARD HOOK DETAILS FOR STIRRUPS & TIES

OFFSET & SPLICES
6db FOR #3 THRU #5
12db FOR #6 THRU #8
"D" 4" MIN OR "db" 12db MIN. BEND DIA.

BAR       D
#4      2"
#3      1 1/2"
#6      4 1/2"
#5      2 1/2"

4db or 63.5mm min MIN BEND DIA. (PRINCIPAL REINF.)
D = 10db FOR #14 & #18
D = 8db FOR #9 THRU #11
D = 6db FOR #3 THRU #8

"a" IS THE CLEAR COVER
"b" IS THE CLEAR SPACING
"db" IS THE BAR DIA.

1. ALL SPLICES SHALL BE TENSION LAP SPLICES UNO.
2. LENGTHS SHOWN ARE FOR GRADE 60 UNCOATED BARS.
3. LENGTHS SHOWN ARE IN INCHES.
4. INCREASE LENGTHS 30% FOR LIGHT WEIGHT CONCRETE AND AT FOUR BAR BUNDLES (WHERE TWO BARS LAP WITH TWO OTHER BARS). INDIVIDUAL BARS WITHIN A BUNDLE SHALL NOT OVERLAP.
5. TOP BARS - HORIZONTAL BARS PLACED WITH MORE THAN 38.10 mm OF FRESH CONCRETE CAST BELOW THEM.
6. INCREASE LENGTHS 50% WHERE a < db OR WHERE b < db FOR BEAMS AND COLUMNS OR WHERE b < 2db FOR OTHER ELEMENTS.
7. FOR #14 AND #18 BARS, USE MECHANICAL SPLICE IN ACCORDANCE WITH UBC REQUIREMENTS.

NOTE:
1. ALL SPLICE SHALL BE TENSION LAP SPLICES UNO.
2. LENGTHS SHOWN ARE FOR GRADE 60 UNCOATED BARS.
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7. FOR #14 AND #18 BARS, USE MECHANICAL SPLICE IN ACCORDANCE WITH UBC REQUIREMENTS.
**NOTES:**

1. Structural general notes per S1.0
2. Verify all dimensions and elevations with the architectural drawings
3. Typical details per:
   - TYPICAL LAP SPICE SCHEDULE
   - STANDARD HOOKS AND BAR BENDS
   - FOUNDATION DETAILS
   - LAB ON GRADE

---

**FOOTING SCHEDULE**

<table>
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<th>SIZE</th>
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<tbody>
<tr>
<td>F1</td>
<td>SPREAD</td>
<td>122 cm x 122 cm</td>
<td>38.1 cm</td>
<td>#5 @ 305 mm O.C. W/ STD. HOOK EW</td>
</tr>
<tr>
<td>F2</td>
<td>STRIP</td>
<td>91.4 cm</td>
<td>61 cm</td>
<td>SEE DETAIL 155.0 FOR CONFINED MASONRY FOOTING</td>
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Project Title: Karambo Micro-Industrial Complex
Site: Mahoko Village, Rubavu District, Rwanda
Client: Empowering Villages
Revisions
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Author: Stella Bates
Checked By: Sophia Abshire
Plot Date: 06/15/2018
Sheet Name: BUILDING A FOUNDATION PLAN
Scale: 1:50
Sheet No: S2.10
NOTES:
1. STRUCTURAL GENERAL NOTES PER S1.0
2. VERIFY ALL DIMENSIONS AND ELEVATIONS WITH THE ARCHITECTURAL DRAWINGS
3. COLUMN SCHEDULE PER S4.0
4. BEAM SCHEDULE PER S4.0
5. BRACE FRAME DETAILS PER S4.0
6. CONFINED MASONRY DETAILS PER S5.0
7. BOND BEAM DETAILS S5.0
8. TRUSS DETAILS PER S3.1
NOTES:

1. STRUCTURAL GENERAL NOTES PER S1.0
2. VERIFY ALL DIMENSIONS AND ELEVATIONS WITH THE ARCHITECTURAL DRAWINGS
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4. BEAM SCHEDULE PER S4.0
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6. CONFined MASONRY DETAILS PER S5.0
7. BOND BEAM DETAILS S5.0
8. TRUSS DETAILS PER S3.1
NOTES:

1. STRUCTURAL GENERAL NOTES PER S1.0
2. VERIFY ALL DIMENSIONS AND ELEVATIONS WITH THE ARCHITECTURAL DRAWINGS
3. TYPICAL DETAILS PER:
   - 4/S1.1 SHEET SCHEDULE
   - TYPICAL LAP SPLICE SCHEDULE
   - 3/S1.1 STANDARD HOOKS AND BAR BENDS
   - 9/S1.1 & 1/S5.0 FOUNDATION DETAILS
   - S1.1 LAB ON GRADE

FOOTING SCHEDULE

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<td>SPREAD</td>
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<td>38.1 cm</td>
<td>#5 @ 205 mm O.C. W/ STD HOOK EW</td>
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<tr>
<td>F2</td>
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<td>9.4 cm</td>
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<td>SEE DETAIL S1.5 FOR CONFINED MASONRY FOOTING</td>
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3. COLUMN SCHEDULE PER S4.0
4. BEAM SCHEDULE PER S4.0
5. BRACE FRAME DETAILS PER S4.0
6. CONFINED MASONRY DETAILS PER S5.0
7. BOND BEAM DETAILS S5.0
8. TRUSS DETAILS S2.1
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2. VERIFY ALL DIMENSIONS AND ELEVATIONS WITH THE ARCHITECTURAL DRAWINGS
3. COLUMN SCHEDULE PER S4.0
4. BEAM SCHEDULE PER S4.0
5. BRACE FRAME DETAILS PER S4.0
6. CONFINED MASONRY DETAILS PER S5.0
7. BOND BEAM DETAILS S3.0
8. TRUSS DETAILS PER S3.1
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2. VERIFY ALL DIMENSIONS AND ELEVATIONS WITH THE ARCHITECTURAL DRAWINGS

3. TYPICAL DETAILS PER:

- TYPICAL LAP SPlice SCHEDULE
- STANDARD HOOKS AND BAR BENDS
- FOUNDATION DETAILS

FOOTING SCHEDULE

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<tr>
<td>F1</td>
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<tr>
<td>F2</td>
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<td>91.4 cm</td>
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<td>SEE DETAIL 1S5.0 FOR CONFINED MASONRY FOOTING</td>
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CLIENT:
Mahoko Village, Rubavu District, Rwanda

Empowering Villages

Author:
Stella Bates

Checked By:
Sophia Abshire

Plot Date:
06/15/2018

Sheet Name:
BUILDING C FOUNDATION PLAN

Scale:
1 : 50

Sheet No:
S2.30
NOTES:

1. STRUCTURAL GENERAL NOTES PER S1.0
2. VERIFY ALL DIMENSIONS AND ELEVATIONS WITH THE ARCHITECTURAL DRAWINGS
3. COLUMN SCHEDULE PER S4.0
4. BEAM SCHEDULE PER S4.0
5. BRACE FRAME DETAILS PER S4.0
6. CONFINED MASONRY DETAILS PER S5.0
7. BOND BEAM DETAILS S5.0
8. TRUSS DETAILS PER S3.1
NOTES:
1. STRUCTURAL GENERAL NOTES PER S1.0
2. VERIFY ALL DIMENSIONS AND ELEVATIONS WITH THE ARCHITECTURAL DRAWINGS
3. COLUMN SCHEDULE PER S4.0
4. BEAM SCHEDULE PER S4.0
5. BRACE FRAME DETAILS PER S4.0
6. CONFINED MASONRY DETAILS PER S5.0
7. BOND BEAM DETAILS S5.0
8. TRUSS DETAILS PER S3.1
NOTES:

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5. BRACE FRAME DETAILS PER S4.0
6. CONFINED MASONRY DETAILS PER S5.0
7. BOND BEAM DETAILS S5.0
STEEL COLUMN SCHEDULE

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<th>COLUMN MARK</th>
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</table>

NOTE: 1. ALL COLUMN SIZES REFLECT STEEL SECTIONS AVAILABLE IN THE UNITED STATES AND IN THE IMPERIAL SYSTEM. ANY SECTION FOUND TO BE EQUIVALENT IN THE METRIC SYSTEM CAN BE USED.

NOTE: 1. ALL COLUMN AND BRACE SIZES REFLECT STEEL SECTIONS AVAILABLE IN THE UNITED STATES AND IN THE IMPERIAL SYSTEM. ANY SECTION FOUND TO BE EQUIVALENT IN THE METRIC SYSTEM CAN BE USED.

STEEL BEAM SCHEDULE

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<tr>
<td>B2</td>
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<td>L 3 1/2 x 3 x 3/8</td>
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</table>

NOTE: 1. ALL BEAM SIZES REFLECT STEEL SECTIONS AVAILABLE IN THE UNITED STATES AND IN THE IMPERIAL SYSTEM. ANY SECTION FOUND TO BE EQUIVALENT IN THE METRIC SYSTEM CAN BE USED.

CLEVIS CONNECTION

TRUSS BOTTOM CHORD, SEE PLAN
HSS ROUND BRACE, SEE PLAN
HSS COLUMN, SEE PLAN
WELD OR BOLT PLATE TO HSS COL.
SLAB ON GRADE

(2) #4
(4) #4
#4 @ 23 cm O.C.

#3 TIE w/ 135 DEGREE HOOK @ 23 cm O.C.

CONFINED MASONRY WALL

91 4 30 5
23 5 15 2
30 5 23 8

(6) #4
(6) #4
#3 STIRRUP w/ 135 DEGREE HOOK @ 200 mm O.C.

NOTE:
1. CONFINED MASONRY WALL TO BE CONSTRUCTED OUT OF LOCAL ADOBE BRICKS

<table>
<thead>
<tr>
<th>No.</th>
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Author: Stella Bates
Checked By: Sophia Abshire
Plot Date: 06/15/2018

Client: Empowering Villages

Project Title: Karambo Micro-Industrial Complex
Site: Mahoko Village, Rubavu District, Rwanda