STRUCTURAL CALCULATIONS

FOR

Mbandazi Village

Mbandazi Village, Rwanda

FOR

Journeyman International

SUBMITTED BY
Joshua Shockey
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Project Description</td>
<td>3</td>
</tr>
<tr>
<td>Project Information</td>
<td>5</td>
</tr>
<tr>
<td>Structural Materials</td>
<td>6</td>
</tr>
<tr>
<td>Seismic Research</td>
<td>7</td>
</tr>
<tr>
<td>Loads</td>
<td>UL1</td>
</tr>
<tr>
<td>Slab Design</td>
<td>SL1</td>
</tr>
<tr>
<td>Roof Framing Design</td>
<td>RF1</td>
</tr>
<tr>
<td>Floor Framing Design</td>
<td>FF1</td>
</tr>
<tr>
<td>Column Design</td>
<td>CO1</td>
</tr>
<tr>
<td>Lateral Analysis/Design</td>
<td>LA1</td>
</tr>
<tr>
<td>Foundation Design</td>
<td>FO1</td>
</tr>
<tr>
<td>Irregularities</td>
<td>IR1</td>
</tr>
<tr>
<td>Appendix</td>
<td>A1</td>
</tr>
</tbody>
</table>
Project Description

The Mbandazi Village project consists of two buildings: the wellness center and the shopping center/housing. The building covered in this design was the shopping center/housing.

Global considerations for this project include the improvement of life in this area of Rwanda. Though my scope of this project was a residential development and an economic center, the whole project also includes a wellness center. Altogether this is an important step to bring a developing nation up to speed as well as bring people together.

Culturally, this project does not seem to add anything. Moreover, I felt like this culturally impacted me. In the U.S., we have nearly everything available to us, using the biggest, most efficient materials and tools. When designing this part of the project, I had to keep in mind what was available in that region of the world. This made me realize the privilege that I have in America where I do not have to worry about much.

The social impact of this project as a whole is a place to come together to shop but also to relax. How often do you hear about a wellness center in developing countries? This project is an opportunity to give the people of Rwanda the experience of what we live in everyday. The ability to relax is something that all people should get to enjoy.

For this project, the environmental and economical impact seemed to be hand in hand. This building was designed probably a little more in depth in the gravity system than many other people would have done it. I designed more beams and girders into similar spans and loadings. This allowed me to adjust the rebar and sizes of the framing members to be more accurate to the demands placed on them. Economically, this project pushes a little further. The bottom floor was designed to be a shopping center to help stimulate the economy.

Working on a project alone is very difficult, especially when all I had was schooling and no professional practice. This project specifically had very many aspects that were never covered in our design studios. A senior project should be something challenging, pushing you to learn something new. However, nearly every step of the way was a new design problem I needed to fix. From curved framing members to curved diaphragm to different roof heights, everything seemed to be one huge problem to recognize and fix after another. I learned that asking for help is important in order to continue with a project when you are stuck and to talk with colleagues to pool together ideas on how to solve a problem. I learned to find the seismic design values without using an online program that gives you everything with a simple push of a button. I learned that ETABS is the best and simultaneously the most confusing design program to use. I jumped into the famous “trash in, trash out” saying with my first ETABS model after putting all my eggs in one basket. This project taught me that when I am on my own, I need to exhaust every possible effort, scour through every code book, use every note from class, and to ask questions when absolutely stuck. I also learned that students can not just jump into designing by themselves after graduating. We need to be guided through the problems we encounter, to use our colleagues in order to do our best. At the end of the day, the more I worked with others, the more confident I felt about what I was doing. The second I decided to try to solve something I had no idea about, it would take me a day at least to find a solution.

Reflection
This project pushed me far past what I thought I could do. I felt like I was relearning everything from my design labs. This project was very rewarding, knowing that I am able to make a difference so far from where this is planning to be built. A lot of useful information was gained from the questions I asked and the problems I solved. It made me very excited for my next chapter in life and what I will learn at my job. I hope to one day look back at this project and laugh at what I currently think is very difficult.

Design Assumptions for the lateral load resisting system

1) Pinned columns at base, for conservative design.

2) Fixity only at connection from beams to shear walls, with shear wall stiffness significantly higher than beam stiffness.
**Project Information**

**Project:** Mbandazi Village  
**Location:** Mbandazi Village, Rwanda  
**Architect:** Journeyman International  
**Owner:**  
**Jurisdiction:**  
**Building Code:** 2018 International Building Code (IBC)  
ASCE 7-16  
ACI 318-19  
AISC 15th Edition  

**Structural Systems:**  
**Vertical/Lateral**  
Concrete floor framing  
Steel roof framing  
Concrete slab floors  
Steel decking roof  
Concrete slab - on - grade  
Concrete Shear Walls  
Concrete foundation  

**Building Description:**  
**Construction Type:** 1b  
**Risk Category:** II  
**Sprinkler System:** No  

**Other:**  
**Soils Engineer:**  
**Soils Report No.:**  
**Soils Report Date:** 25th March, 2021  
**Soil Bearing:** D+L: 2000 psf
**Structural Materials**

**Material Specifications** Typical unless noted otherwise in calculations

**Concrete:** NWC S-O-G w/ Spread FTGs (3000 psf)

**Reinforcing:** ASTM A615 (Fy = 60 ksi)

**Wall Reinforcing:** ASTM A615 (Fy = 60 ksi)

**Steel:** ASTM A992 (Fy = 50 ksi), UNO
Rwanda sits very close to a divergent plate.

Baja California also has a divergent plate. I used the PGA to get Ss and S1 values.

https://www.geoengineer.org/news/is-africa-gradually-splitting-into-two-sections

**FLOOR**

**SUPERIMPOSED UNIT LOADS**

<table>
<thead>
<tr>
<th></th>
<th>JOISTS</th>
<th>GIRDERS</th>
<th>COLUMNS</th>
<th>SEISMIC</th>
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<tr>
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<td>16&quot; SQ COLUM</td>
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**TOTAL DEAD LOAD**

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<td>PARTITIONS</td>
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<td>146</td>
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<tr>
<td>DESIGN LOAD</td>
<td>120</td>
<td>150</td>
<td>160</td>
<td>180</td>
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**LIVE LOAD** (REDUCIBLE) = 40.0 PSF

ASCE 7-16 T4.3-1 (15)
## SUPERIMPOSED UNIT LOADS

<table>
<thead>
<tr>
<th></th>
<th>JOISTS</th>
<th>GIRDER</th>
<th>COLUMNS</th>
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<tr>
<td>MEP</td>
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<tr>
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<td><strong>Girders</strong></td>
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<tr>
<td>16&quot; SQ Col</td>
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<td>7.0</td>
<td></td>
<td></td>
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<tr>
<td>12&quot; Walls</td>
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<td><strong>Total Dead Load</strong></td>
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<td>24</td>
<td>29</td>
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<td><strong>Partitions</strong></td>
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**Live Load (reducible) = 20.0 PSF**

ASCE 7-16 T4.3.1 (15)
ESTIMATING A SLAB THICKNESS

UNIFORM SLAB THICKNESS : WORST SPAN WILL GOVERN

$L_{\text{min}} = 9' - 7\frac{1}{2}'' = 116.5''$

$\Delta = \frac{L}{20}$ to $\frac{L}{24}$ ACI 318-19 T 7.3.1.1

5.8'' to 4.3''

ASSUME 5.5'' SLAB

= 68.75 $\text{sf}$

$P$

$DL = 68.75 + 16.5 = 86 \text{ psf}$
SLAB DESIGN

No fire rating for Rwanda. Following a 2-hr rating. Min. slab thickness is 5" with 3/4" siliceous aggregate per IBC T 721.1(3), 722.2.1.1, 722.2.2.1, & 722.2.3(1).

\[
DL = 86 \text{ PSF} \\
LL = 40 \text{ PSF} \\
TL = 126 \text{ PSF}
\]

\[
LC : 1.2D + 1.6L = 1.2(86) + 1.6(40) = 167 \text{ PSF}
\]

LOOKING @ 12" PIECE OF SLAB

\[
\omega_L = 40 \text{ PSF} \times 1' = 0.04 \text{ klf}
\]

\[
\omega_b = 86 \text{ PSF} \times 1' = 0.086 \text{ klf}
\]
SLAB DEFLECTION

\[ f'_c = 3 \text{ksi} \]
\[ E_c = 31,220 \text{ ksi} \]
\[ f_r = 7.5 \left( \frac{f'_c}{E_c} \right) = 0.411 \text{ ksi} \]
\[ L = 10' - \frac{1}{2} " = 10.04' \]

\[ M_{MID} = 0.04(0.126 kif)(1004 ft^2) - \frac{(0.126 kif)(0.4 \times 10.04 ft)(10.04 ft - 0.4(10.04 ft))}{2} \]
\[ = 1.02 \text{ k-ft} \]

Concrete Remains Theoretically Uncracked

\[ \frac{M_{MID}}{f_r} = \frac{5w_{eq}}{3} = \frac{12(t)^2}{6} \Rightarrow t = \sqrt{6 \left( \frac{5w_{eq}}{12} \right)} = \sqrt{\frac{6(1.02 \text{ k-ft} \times 12)}{12 \times (0.411 \text{ ksi})}} = 3.9 \text{ in} \]

\[ I = \frac{bt^3}{12} = \frac{12n(3.9 \text{ in})^3}{12} = 59 \text{ in}^4 \]

\[ \Delta = \frac{5wl^4}{384EI} = \frac{5(126 \text{ kif})(10.04 \text{ ft})^4 \times 12^3}{384(3122 \text{ ksi})(59 \text{ in}^4)} = 0.16 \text{ in} = \frac{L}{710} < \frac{L}{240} \]
SLAB SHEAR

\[ V_{u,\text{max}} = \frac{3}{4} \omega_{u} L = \frac{3}{4} (0.167 \text{kips})(10.04 \text{ ft}) = 1.04 \text{ kips/ft} \text{ (upper)} \]

\[ V_u = \frac{1.15}{2} \omega_{u} L = \frac{23}{40} (0.167 \text{kips})(10.04 \text{ ft}) = 0.96 \text{ kips/ft} \]

\[ \phi V_c = \phi 2 \sqrt{f_c} bd \Rightarrow d = \frac{V_u \times 1000}{\phi 2 \sqrt{f_c} b} = \frac{(1.04 \text{kips}) \times 1000}{0.75(2) \times 3000 (12 \text{ in})} \]

\[ d = 1.05'' \]

\[ t_{\text{slab}} = d + d = 2(1.09\text{ in}) = 2.1 \text{ in} \]

AS STATED ABOVE, THE MIN. THICKNESS OF THE SLAB IS 5'', SO THAT WILL BE THE DESIGNED THICKNESS

5'' THICK SLAB
SLAB FLEXURAL

FIXED LOADS

\[ DL = 79 \text{ PSF} \]
\[ LL = 40 \text{ PSF} \]
\[ TL = 119 \text{ PSF} \]

\[ LC = 1.2D + 1.6L = 1.2(79) + 1.6(40) = 159 \text{ psf} \]

USING \[ + = 5 \text{ in} \quad \text{ (FROM } \bullet \text{ )} \]

\[ d = \frac{1}{2}(5 \text{ in}) = 2.5 \text{ in} \]

\[ w_u = 0.159 \text{ kif} \]

\[ Mu^+ = \frac{w_uL^2}{16} = \left( \frac{0.159 \text{ kif}}{11} \right) (9.625 \text{ ft})^2 = 1.34 \text{ k-ft} \]

\[ Mu^- = \frac{w_uL^2}{10} = \left( \frac{0.159 \text{ kif}}{11} \right) (9.625 \text{ ft})^2 = 1.47 \text{ k-ft} \]

\[ A_s = \frac{Mu^-}{0.9(60 \text{ ksi})(0.95 \times 2.5 \text{ in})} = 0.14 \text{ in}^2/\text{ft} \]

USE #4 @ 12" O.C. (0.2 in²/ft)

\[ S = \frac{A_s}{bh} = \frac{0.2 \text{ in}^2}{12 \text{ in} \times 5 \text{ in}} = 0.3 \% \]

\[ A_str = 0.0018bh = 0.0018(12\text{ in})(5\text{ in}) = 0.108 \text{ in}^2 \]

\[ #4@12" \text{ O.C. 5 ft} \]

\[ \alpha = \frac{A_s f_y}{0.85 f_c b} = \frac{(0.2 \text{ in}^2)(60 \text{ ksi})}{0.85(3 \text{ ksi})(12\text{ in})} = 0.392 \text{ in} \]

\[ \phi M_n = \phi A_s f_y \left( \frac{h - d}{2} \right) = 0.9 \left[ \left( \frac{0.2 \text{ in}^2}(60 \text{ ksi}) \left( \frac{5\text{ in} - 0.392 \text{ in}}{2} \right) \right) \right] / 12 = 2.52 \text{ k-ft} \]

\[ \phi M_n = 2.52 \text{ k-ft} > Mu = 1.47 \text{ k-ft} \checkmark \]
1. TOP OF CONCRETE ELEVATION = VARIES. SEE ELEVATIONS FOR HEIGHTS
2. SEE S.407, S.408, S.409, S.411, & S.412 FOR JOIST AND GIRDER DETAILS
3. SEE SHEET NOTES FOR ADDITIONAL INFORMATION

ROOF FRAMING PLAN
1 : 200
\[
\begin{align*}
\theta &= \tan^{-1} \left( \frac{6.56}{43.64} \right) = 8.55^\circ \\
LL &= \frac{20}{x} = \cos(85.5^\circ) = 20.22 \text{ psf}
\end{align*}
\]

**B1**

\[
\begin{align*}
DL &= 15 \text{ psf} \\
LL &= 20.2 \text{ psf} \\
TW &= 7.14' \\
L &= 14.98'
\end{align*}
\]

\[
\omega = 1.2(15) + 1.6(20.2) = 50.3 \text{ psf} \times 7.14' = 0.40 \text{ klf}
\]

\[
M_u = \frac{\omega L^2}{8} = \frac{0.40(14.98)^2}{8} = 11.22 \text{ k-ft}
\]

\[
\varphi = \frac{M_u}{\omega D L} = 0.9
\]

\[
Z = \frac{M_u}{\omega D L} = 2.99 \text{ m}^3
\]

**START W/ W/8 x 13**

\[
Z = 11.4 > 2.99 \text{ m}^3 \checkmark
\]

**CAPACITY**

\[
\begin{align*}
Vu &= \frac{\omega L}{2} = \frac{0.4(14.98)}{2} = 3.0 \text{ k} \\
Mu &= 11.22 \text{ k-ft}
\end{align*}
\]

\[
\varphi V_n = 55.1 \text{ k} \checkmark < \varphi V_n = 55.1 \text{ k} < \varphi_b M_n = 42.8 \text{ k-ft} \checkmark
\]

\[
L_b = 17' \varphi M_n = 115
\]
DEFLECTION

LL => L/360

DL => 4L/40

\[
\frac{LL}{DL} = 0.67
\]

\[
\frac{LL}{DL} = \frac{20.2}{15} = 1.3 > 0.67
\]

LIVE LOAD GOVERNS △

\[
\Delta_{act} = \frac{5LL^2}{384EI} = \frac{5(20000kNm \times 7.14')(14.98')^3 \times 12^3}{384 \times (29000kN/m)(39.6m^3)} = 0.14''
\]

\[
\Delta_{all} = 14.98''/360 = 0.50'' > \Delta_{act} = 0.14'' \checkmark
\]

B1: W8 x 13
1. TOP OF CONCRETE ELEVATION = VARIES. SEE ELEVATIONS FOR HEIGHTS
2. SEE S.407, S.408, S.409, S.411, & S.412 FOR JOIST AND GIRDER DETAILS
3. SEE SHEET NOTES FOR ADDITIONAL INFORMATION
B2

DB = 19 PSF

HL = 20.2 PSF

TW = 10.04'

L = 34.67'

\[ \omega = 1.2(19) + 1.6(20.2) = 50.3 \text{ PSF} \times 10.04' = 0.51 \text{ kft} \]

\[ M_u = \frac{\omega L^2}{8} = \frac{0.51(34.67)^2}{8} = 76.63 \text{ k-ft} \]

\[ Z_{k = 0.9} = \frac{M_u}{\phi_b F_y} \quad \phi_b = 0.9 \]

\[ = \frac{76.63 \text{ k-ft} \times 12}{0.9 \times 50 \text{ k}^3} = 20.43 \text{ m}^3 \]

**START W/ 8 x 24**

\[ Z = 23.1 \text{ m}^3 > 20.43 \text{ m}^3 \checkmark \]

\[ V_u = \frac{\omega H}{2} = \frac{0.51(34.67)}{2} = 8.84k \]

\[ M_u = 76.63 \text{ k-ft} \]

\[ < \quad \phi_b M_u = 86.6 \text{ k-ft} \checkmark \]

\[ \text{CAPACITY} \quad AISC T5-2 (s-108) \]

\[ \phi V V_n = 58.3k \checkmark \]

\[ l_0 = 10' \phi M_n = 76.3 \]
DEFORMATION

\[ LL \Rightarrow L/360 \quad DL \Rightarrow L/240 \]

\[ \frac{LL}{DL} = 0.67 \]

\[ \frac{LL}{DL} = \frac{20.2}{15} = 1.3 > 0.67 \]

LIVE LOAD GOVERNS \( \Delta \)

\[ \Delta_{act} = \frac{56UL^2}{384EI} = \frac{1(0.0205 \times 100)(34.67)^9 \times 12}{384 \ (29000 \text{ psi})(82.7 \text{ in}^3)} = " \]

\[ \Delta_{net} = \frac{34.67 \times L^2}{360} = 1.16 " > \Delta_{act} = 0.27 " \]

B2: W8 \times 24
1. TOP OF CONCRETE ELEVATION = VARIES. SEE ELEVATIONS FOR HEIGHTS
2. SEE S.407, S.408, S.409, S.411, & S.412 FOR JOIST AND GIRDER DETAILS
3. SEE SHEET NOTES FOR ADDITIONAL INFORMATION
B3

DL = 19 PSF
HL = 20.2 PSF
TW = 9.00'
L = 23.54'

\[ \omega = 1.2(19)+1.6(20.2) = 0.3 \text{ ksf} \times 9.00' = 0.45 \text{ klf} \]

\[ M_u = \frac{\omega L^2}{2} = \frac{0.45(23.80)^2}{2} = 31.86 \text{ k-ft} \]

\[ Z_{x, n=0.9} = \frac{M_u}{\phi_b h^2} \quad \phi_b = 0.9 \]

\[ = \frac{31.86 \text{ k-ft} \times 12}{0.9 \times 50^{0.5}} = 8.5 \text{ m}^3 \]

**START W/ W/B x 18**

\[ Z = 17.0 \text{ m}^3 > 8.5 \text{ m}^3 \checkmark \]

\[ V_u = \frac{\omega L}{2} = \frac{0.39(23.80)}{2} = 4.09k \]

\[ M_u = 31.86 \text{ k-ft} \]

\[ 4.09k < \phi_u V_u = 56.2k \checkmark \]

\[ \phi_b M_u = 63.8 \text{ k-ft} \checkmark \]

**CAPACITY**

HSC T8-2 ( )

\[ L_b = 16', \phi M_a = 31.5 \]
DEFLECTION

\[ LL \Rightarrow \frac{L}{360} \quad DL \Rightarrow \frac{4L}{360} \]

\[ \frac{LL}{DL} = 0.67 \]

\[ \frac{LL}{DL} = \frac{20.2}{15} = 1.3 > 0.67 \]

LIVE LOAD GOVERNS \( \Delta \)

\[ \Delta_{rel} = \frac{5wL^4}{384EI} = \frac{5 \left(0.020 \times 9.00 \text{ in} \right)^4 \times 12^3}{384 \times \left(29000 \text{ psi} \right) \left(61.0 \text{ in}^4 \right)} = 0.69'' \]

\[ \Delta_{ull} = 23.54 \times \frac{L^2}{360} = 0.78'' > \Delta_{rel} = 0.69'', \sqrt{\} \]

B2: \[ W8 \times 18 \]
SHEET NOTES

1. TOP OF CONCRETE ELEVATION = VARIES. SEE ELEVATIONS FOR HEIGHTS
2. SEE S.407, S.408, S.409, S.411, & S.412 FOR JOIST AND GIRDER DETAILS
3. SEE SHEET NOTES FOR ADDITIONAL INFORMATION
B4

$D_L = 15$ PSF

$U_L = 20.2$ PSF

$T_W = 8.37'$

$L = 18.54'$

$w = 1.2(15) + 1.6(20.2) = 50.3$ PSF $\times 8.37' = 0.42$ klf

$M_u = \frac{wL^2}{8} = \frac{0.42(18.54)^2}{8} = 18.05$ k-ft

$Z_{x, 	ext{net}} = \frac{M_u}{\phi b L} = 0.9$

$= \frac{18.05\text{ k-ft} \times 12}{0.9 \times 50\text{ in}} = 4.81\text{ m}^3$

\[\text{START W/ W/8x13}\]

$Z_w = 11.4\text{ m}^3 > 4.81\text{ m}^3 \checkmark$

$V_u = \frac{\phi L}{2} = \frac{0.42(18.54)}{2} = 3.89k$

$M_u = 18.05\text{ k-ft}$

$\phi V_u V_n = 56.1k$

$\phi b M_n = 42.8 \text{ k-ft}$

$L_b = 12, \phi M_n = 18$

$\text{CAPACITY}$

$AISC TB-2 (6.101)$
**DEFLECTION**

\[ LL \Rightarrow L/360 \quad DL \Rightarrow L/240 \]

\[ \frac{LL}{DL} = 0.67 \]

\[ \frac{LL}{DL} = \frac{20.2}{15} = 1.3 > 0.67 \]

LIVE LOAD GOVERNS \( \Delta \)

\[ \Delta_{nt} = \frac{5wL^4}{384EI} = \frac{5(2000 \text{ lbs}) \times 837 \text{ in}^4}{384 (29000 \text{ kips})(39.6 \text{ in}^3)} = 0.39'' \]

\[ \Delta_{ml} = 18.64 \times L/360 = 0.62'' \quad > \Delta_{nt} = 0.39'' \quad \checkmark \]

B2: W 8 x 13
ROOF FRAMING PLAN
1 : 200

SHEET NOTES
1. TOP OF CONCRETE ELEVATION = VARIES. SEE ELEVATIONS FOR HEIGHTS
2. SEE S.407, S.408, S.409, S.411, & S.412 FOR JOIST AND GIRDER DETAILS
3. SEE SHEET NOTES FOR ADDITIONAL INFORMATION
G1

DL = 20 PSF
LL = 20.2 PSF

TRIB AREA = (6.82' x 11.75') + (8.13' x 8.65') = 150.46 ft²

P₀ = 1.2(3.01) + 1.6(3.09) = 8.48 k

V₀ = 1.5(8.48) = 12.72 k

M₀ = 0.5(8.48)(27.28) = 115.7 k-ft

Zₙ₀ = Mu / Øμ = 115.7 k-ft / 0.75 = 30.9 in³

START W/ W₁₂ x 40

Z = 67 in³ > 30.9 in³

V₀ = 12.72 k < Øµ V₀ = 105 k

M₀ = 115.7 k-ft < Øb M₀ = 214 k-ft

CAPACITY

A: SL T.C - 2 (8.95)

MAX Lt = 24, φmₙ = 114 k < 115.7 k

DEFLECTION

LL => L₁/360

DL => L₁/240

\[
\frac{LL}{DL} = 0.67
\]

\[
\frac{LL}{DL} = \frac{20.2}{20} = 1.01 > 0.67
\]

LIVE LOAD COVERS Δ

SEE P FOR DEFLECTION

\[
Δₚ = 27.28' × 12 / 360 = 0.91" > Δₚₑₓ = 0.59" \]

G1: W₁₂ x 40
1. TOP OF CONCRETE ELEVATION = VARIES. SEE ELEVATIONS FOR HEIGHTS
2. SEE S.407, S.408, S.409, S.411, & S.412 FOR JOIST AND GIRDER DETAILS
3. SEE SHEET NOTES FOR ADDITIONAL INFORMATION
**G2**

\[ P_{u1} \downarrow \quad P_{u2} \downarrow \quad P_{u3} \downarrow \]

**DL = 20 PSF**
**LL = 20.2 PSF**

**TRIB AREA\_1 = (5.70' \times 6.91') = 21.47 ft\^2**

**TRIB AREA\_2 = (5.70' \times 6.91') = 39.97**

\[ P_{u1} = 1.2(4.23)\times 1.6(4.27) = 11.91 k \]

\[ P_{u2} = 1.2(0.79)\times 1.6(0.90) = 2.23 k \]

\[ V_{\max} = 0.5(11.91)\times 2.23 = 8.19 k \]

\[ M_{\max} = (11.91 + 2.23)(20.08)/4 = 70.98 k ft \]

\[ Z_{x \leq 0} = M_{x} / \phi \gamma = 70.98 k ft \times 60 \times 105 = 16.9 m^3 \]

**START W/ W10 x 26**

\[ 2.53 \times 31.3 m^3 > 18.9 m^3 \]

**CAPACITY**

\[ V_{u} = 8.19 k < \phi_{v} V_{n} = 50.3 k \] ✓

\[ M_{u} = 70.98 k ft < \phi_{h} M_{n} = 117 k ft \] ✓

\[ AISC TC-2 (1-09) \]

**DEFLECTION**

\[ LL \Rightarrow L/360 \quad DL \Rightarrow L/240 \]

\[ LL / DL = 0.67 \]

\[ LL / DL = 20.2 / 20 = 1.01 > 0.67 \]

**LIVE LOAD GOVERNS**

\[ \Delta_{u} = 20.29 \times 12 / 360 = 0.69 " > \Delta_{max} = 0.37 " \] ✓

\[ G2 : W10 x 26 \]
1. TOP OF CONCRETE ELEVATION = VARIES. SEE ELEVATIONS FOR HEIGHTS
2. SEE S.407, S.408, S.409, S.411, & S.412 FOR JOIST AND GIRDER DETAILS
3. SEE SHEET NOTES FOR ADDITIONAL INFORMATION
G3

$P_u$  $P_u$

$V_i$

$M_i$

$Z \times \text{NEG} = \frac{M_u}{19} = \frac{39.27 k \times \text{yd}^2}{0.1 \times 50 \text{ kN}} = 9.4 \text{ m}^3$

$\text{CAPACITY}$

$V_u = 5.38 k \leq \varnothing_v V_n = 76.5 k \checkmark$

$M_u = 35.3 k \text{ ft}^2 \leq \varnothing_b M_n = 81 k \text{ ft}^2 \checkmark$

$\text{DEFLECTION}$

$LL \Rightarrow L/360 \quad DL \Rightarrow L/40$

$\frac{LL}{DL} = 0.67$

$\frac{LL}{DL} = \frac{20.2}{20} = 1.01 > 0.67$

LIVE LOAD GOVERNS $\Delta$

SEE P FOR DEFLECTION

$\Delta_{\text{ill}} = 19.67 \times 12 / 360 = 0.66 " > \Delta_{\text{act}} = 0.32 " \checkmark$

G2 : W/10 x 19
ESTIMATE JOIST SIZE

\[ L_{\text{joist}} = 34' \Rightarrow h_{\text{beam}} = \frac{L_{\text{joist}}}{16} \text{ to } \frac{L_{\text{joist}}}{18.5} \]

\[ 26'' \text{ to } 23'' \]

\[ h_{\text{beam}} = 24'' \]

\[ b = \frac{L_{\text{joist}}}{16} \]

\[ b = 16'' \]

\[ 16'' \times 24'' \text{ JOISTS} \]

ESTIMATE A GIRDER SIZE

ASSUMING BUILDING DL = 160

\[ U = 40 \Rightarrow 1.2(160) + 1.6(40) = 256 \text{ psf} \times 10' = 2.56 \text{ ksf} \]

\[ P = 2.56 \times \frac{23.25'' + 18.25''}{2} = 53.12 \text{ k} \]

\[ M_u = 0.465 \times \text{PL} \]

\[ M_u = 67.3 \text{ k'-ftp} \]

\[ d = 1.5b \]

\[ 20M_u = bd^2 \Rightarrow b = \sqrt{\frac{20M_u}{1.5^2}} \]

\[ b = 18'' \]

\[ d = 27'' \]

\[ h = 30'' \]

\[ 18'' \times 30'' \text{ GIRDER} \]
FIRST FLOOR FRAMING PLAN
1 : 200

SHEET NOTES
1. TOP OF CONCRETE ELEVATION = +3m U.N.O.
2. SEE SHEET S.405 FOR TYPICAL SUSPENDED SLAB DETAILS
3. SEE S.406, S.408, & S.409 FOR SPAN JOIST DETAILS
4. SEE S.410, S.411, & S.412 FOR GIRDER DETAILS
5. SEE SHEET NOTES FOR ADDITIONAL INFORMATION
BL
DL = 120 \text{ psf} \times 7.14' = 856.8 \text{ klf}
LL = 40 \text{ psf} \times 7.14' = 0.2856 \text{ klf}

TW = 7.14'
1.2D + 1.6L = 1.49 \text{ klf}

16'' x 24'' BM

SEE P FOR ETABS RESULTS

\begin{align*}
M_u^- &= 32.32 \text{ k-ft} \\
M_u^+ &= 26.00 \text{ k-ft} \\
V_u &= 12.83 \text{ k}
\end{align*}

ESTIMATE STEEL REINFORCEMENT

\begin{align*}
A_5^- \approx \frac{M_u^-}{\phi_y d} &= \frac{32.32 \text{ k-ft} \times 12}{0.9(60^\circ)(0.9 \times 21')} = 0.38 \text{ in}^2 \\
\text{USE: (2) - # 4} \quad (A_5 = 2\times 0.2 = 0.4 \text{ in}^2) \\
\phi &= 0.4 \\
\frac{f_p}{16 \times 21} &= 0.12\%
\end{align*}

\begin{align*}
A_5^+ \approx \frac{M_u^+}{\phi_y d} &= \frac{26.00 \text{ k-ft} \times 12}{0.9(60^\circ)(0.93 \times 21')} = 0.30 \text{ in}^2 \\
\text{USE: (2) - # 4} \quad (A_5 = 2\times 0.2 = 0.4 \text{ in}^2) \\
\phi &= 0.4 \\
\frac{f_p}{16 \times 21} &= 0.12\%
\end{align*}
VERIFY FLEXURAL CAPACITY, $\phi M_n$

BM DIMENSIONS:  
- $b = 16''$
- $d = 21''$
- $h = 24''$

\[ b_{eff} = 2\left[8\frac{h}{2}\right] + b_w = 2\left[8\frac{21}{2}\right] + 16 = 96'' = 6' 6'' = 78'' \]

\[ = 2\left[\frac{491}{8}\right] + 16 = 60'' \leq \]

@ SUPPORT (-M)

USING (2) - #4 $= 0.4in^2$

\[ \alpha = \frac{A_s - f_y}{0.85f'c' b} = \frac{(0.4in^2)(60ksi)}{0.85(3ksi)(16'')} = 0.59'' \]

ASSUME $\phi = 0.9$

\[ \phi M_n = \phi \left[ A_s - \frac{f_y}{d - 0.25}\right] / 12 = 0.9 \left[ 0.4in^2 \times 60ksi \left( 21'' - \frac{0.59''}{2} \right) \right] / 12 = 37.27 \text{k-ft} \]

\[ \phi M_n = 37.27 \text{k-ft} > M_u = 31.21 \text{k-ft} \quad \checkmark \]

@ MIDSPLAYN (+M)

USING (2) - #4 $= 0.4in^2$

\[ \alpha = \frac{A_s + f_y}{0.85f'c' b} = \frac{(0.4in^2)(60ksi)}{0.85(3ksi)(16'')} = 0.16'' \]

ASSUME $\phi = 0.9$

\[ \phi M_n = \phi \left[ A_s - \frac{f_y}{d - 0.25}\right] / 12 = 0.9 \left[ 0.4in^2 \times 60ksi \left( 21'' - \frac{0.16''}{2} \right) \right] / 12 = 37.66 \text{k-ft} \]

\[ \phi M_n = 37.66 \text{k-ft} > M_u = 24.13 \text{k-ft} \quad \checkmark \]
**SHEAR CHECK**

\[ V_u = 12.83^k \]

**2-LEGGED #3 STIRRUPS**

\[ \Phi V_U = \Phi (2 \sqrt{F_c bd}) = 0.75 \left( 2 \times 1.0 \sqrt{3000} \right) (16 \times 21) = 27.6^k \]

SPACING \( \frac{d}{2} = 2 \frac{1}{2} = 10.5'' \Rightarrow 10'' \)

\[ \Phi V_s = \Phi A_{\text{net}} \frac{f_d}{\bar{f}} = 0.75 \left( 2 \times 0.11 \times 10^{\text{kn}} \right) (21'') = 20.8^k \]

\[ \Phi V_c + \Phi V_s = 27.6^k + 20.8^k = 48.4 > V_u = 12.83^k \checkmark \]

**USE #3 STIRRUPS @ 10'' %C**
See pgs A1-A4 for code

In [55]: runfile('/Users/joshuahockey/Documents/ARCE/JI/sxnprops.py',
               wdir='/Users/joshuahockey/Documents/ARCE/JI')

b effective =  60 in

flange thickness =  5 in

h =  24 in

b =  16 in

Neutral Axis =  15.46 in

Ixx =  31513.38 in^4

fc =  3000 psi

Mcr+ =  69.778 k-ft

Mcr- =  126.326 k-ft

I cracked midspan =  1489.035 in^4

I cracked support =  1444.343 in^4

dead deflection =  0.005 in

live deflection =  0.002 in

Ma midspan [D,D+L,D+0.1L] =  [13.05 17.4  13.49] k-ft

I effective midspan [D,D+L,D+0.1L] =  [31513.38 31513.38 31513.38] in^4

Ma support [D,D+L,D+0.1L] =  [18.  24.  18.6] k-ft

I effective support [D,D+L,D+0.1L] =  [31513.38 31513.38 31513.38] in^4

deflection due to creep and live =  0.012 in  <1"  <L/360 = 0.49"

deflection due to creep, live, and dead =  0.017 in  <L/240 = 0.74"
1. TOP OF CONCRETE ELEVATION = +3m U.N.O.
2. SEE SHEET S.405 FOR TYPICAL SUSPENDED SLAB DETAILS
3. SEE S.406, S.408, & S.409 FOR SPAN JOIST DETAILS
4. SEE S.410, S.411, & S.412 FOR GIRDER DETAILS
5. SEE SHEET NOTES FOR ADDITIONAL INFORMATION
B2

LONGER SPAN
DL = 120 PSF x 10' = 1.2 kif
LL = 40 PSF x 10' = 0.4 kif
TW = 10'
1.2D+1.6L = 2.08 kif

SHORER SPAN
DL = 120 PSF x 5' = 0.6 kif
LL = 40 PSF x 5' = 0.2 kif
TW = 5'
1.2D+1.6L = 1.04 kif

16" x 30" BM

SEE P FOR ETABS RESULTS

\[ M_{u} = 219.22 \text{ k-ft} \]
\[ M_{u}^{*} = 199 \text{ k-ft} \]
\[ V_{u} = 41.51 \text{ k} \]

ESTIMATE STEEL REINFORCEMENT

\[ A_{5}^{-} \approx \frac{M_{u}}{\phi f_{y} J d} = \frac{219.22 \text{ k-ft} \times 12}{0.9(60'')(0.9\times27'')} = 1.97 \text{ in}^{2} \]

USE: (6) - + S (\( A_{5} = 6 \times 0.31 = 1.86 \text{ in}^{2} \))

\[ f = \frac{1.86}{18 \times 27} = 0.38 \% \]

\[ A_{5}^{+} \approx \frac{M_{u}^{*}}{\phi f_{y} J d} = \frac{199 \text{ k-ft} \times 12}{0.9(60'')(0.9\times27'')} = 1.76 \text{ in}^{2} \]

USE: (6) - + S (\( A_{5} = 6 \times 0.31 = 1.86 \text{ in}^{2} \))

\[ f = \frac{1.86}{18 \times 27} = 0.38 \% \]
VERIFY FLEXURAL CAPACITY, $\phi M_n$

**BM DIMENSIONS:**

- $b = 18''$
- $d = 25''$
- $h = 28''$
- $b_{eff} = 2\left[\frac{8''}{2}\right] + 18' = 98''$
- $d_{eff} = 5' = 60''$
- $w = 2\left[\frac{19.08' \times 12''}{8}\right] + 18' = 60''$

**@ SUPPORT (MIN)**

**USING (6)**

- $A_s = 5'' = 0.86''^2$
- $\alpha = \frac{A_s - f_y}{0.95 f'c b} = \frac{(0.86''^2)(60^{ksi})}{0.95(3^{ksi})(18'')} = 2.43''$
- $\phi = 0.9$

- $\phi M_n = \phi \left[ A_s - f_y \left( d - \frac{9}{2} \right) \right]/12 = 0.9 \left[ 1.96 ''^2 \times 60^{ksi} \left( 27'' - \frac{2.43''}{2} \right) \right]/12 = 219.82^{k-ft}$

- $\phi M_n = 219.82^{k-ft} > M_u = 215.72^{k-ft}$ ✓

**@ MIDSPAN (MAX)**

**USING (6)**

- $A_s = 5'' = 0.86''^2$
- $\alpha = \frac{A_s + f_y}{0.95 f'c b} = \frac{(0.86''^2)(60^{ksi})}{0.95(3^{ksi})(18'')} = 0.73''$
- $\phi = 0.9$

- $\phi M_n = \phi \left[ A_s - f_y \left( d - \frac{9}{2} \right) \right]/12 = 0.9 \left[ 1.96 ''^2 \times 60^{ksi} \left( 27'' - \frac{0.73''}{2} \right) \right]/12 = 222.9^{k-ft}$

- $\phi M_n = 222.9^{k-ft} > M_u = 199^{k-ft}$ ✓
SHEAR CHECK

\( V_u = 41.51 \text{k} \)

**2-LEGGED #3 STIRRUPS**

\[ \Phi_f = 0.75(2 \times 1.0 - \sqrt{3000})(18 \times 25) = 37.0 \text{k} \]

**SPACING**

\[ d/2 = 27/2 = 13.5" \Rightarrow 12" \]

\[ \Phi \sqrt{V_2} = 0.75(2 \times 0.114)(60^{\text{ksi}})(25\text{"}) = 20.63 \text{k} \]

\[ \Phi \sqrt{V_c} + \Phi \sqrt{V_2} = 37.0 \text{k} + 20.63 = 57.6 \text{k} > V_u = 41.51 \text{k} \]

**USE #3 STIRRUPS @ 12" OC**
In [6]: runfile('/Users/joshuashockey/Documents/ARCE/JI/sxnprops.py', wdir='/Users/joshuashockey/Documents/ARCE/JI')

b effective = 60 in

flange thickness = 5 in

h = 30 in

b = 18 in

Neutral Axis = 18.5 in

Ixx = 64562.5 in^4

fc = 3000 psi

Mcr+ = 119.467 k-ft

Mcr- = 192.187 k-ft

I cracked midspan = 9604.292 in^4

I cracked support = 8909.583 in^4

dead deflection = 0.1 in

live deflection = 0.033 in

Ma midspan [D,D+L,D+0.1L] = [108.59 144.79 112.21] k-ft

I effective midspan [D,D+L,D+0.1L] = [17717.444 12936.249 16815.699] in^4

Ma support [D,D+L,D+0.1L] = [124.17 165.56 128.31] k-ft

I effective support [D,D+L,D+0.1L] = [64562.5 18417.764 63417.396] in^4

deflection due to creep and live = 0.89 in < 1" < L/360 = 1.14"
deflection due to creep, live, and dead = 1.15 in < L/240 = 1.71"
1. TOP OF CONCRETE ELEVATION = +3m U.N.O.
2. SEE SHEET S.405 FOR TYPICAL SUSPENDED SLAB DETAILS
3. SEE S.406, S.408, & S.409 FOR SPAN JOIST DETAILS
4. SEE S.410, S.411, & S.412 FOR GIRDER DETAILS
5. SEE SHEET NOTES FOR ADDITIONAL INFORMATION
B3
DL = 120 psf \times 7.05 = 0.85 klf
LL = 40 psf \times 7.05' = 0.28 klf
TW = 7.05'
1.2D + 1.6L = 1.47k If

16'' x 24'' BM

- SEE P FOR ETABS RESULTS

M_u^- = 35.61 k-ft
M_u^+ = 52.75 k-ft
V_u = 12.52 k

ESTIMATE STEEL REINFORCEMENT

\[ A_{5^-} \approx \frac{M_u^-}{f_y \cdot J} = \frac{35.61 \text{ k-ft} \times 12}{0.9 (60^{\text{ksi}}) (0.95 \times 24')} = 0.42 \text{ in}^2 \]

USE: (2) - #4 \hspace{1cm} (A_{5^-} = 2 \times 0.2 = 0.4 \text{ in}^2)

\[ f_p = \frac{0.4}{16 \times 21} = 0.12\% \]

\[ A_{5^+} \approx \frac{M_u^+}{f_y \cdot J} = \frac{52.75 \text{ k-ft} \times 12}{0.9 (60^{\text{ksi}}) (0.95 \times 24')} = 0.60 \text{ in}^2 \]

USE: (3) - #4 \hspace{1cm} (A_{5^+} = 3 \times 0.2 = 0.6 \text{ in}^2)

\[ f_p = \frac{0.6}{16 \times 21} = 0.18\% \]
VERIFY FLEXURAL CAPACITY, $\Phi M_n$

**BM DIMENSIONS:**
\[
\begin{align*}
  b &= 16'' \\
  d &= 21'' \\
  h &= 24''
\end{align*}
\]
\[
\begin{align*}
  b_{eff} &= 2\left[8\frac{5}{8}\right] + 2\left[8\frac{5}{8}\right] + 16 = 96'' \\
  &= 7.05' = 84.6'' \\
  &= 2\left[\frac{[12.06' \times 12]}{8}\right] + 16 = 67''
\end{align*}
\]

@ SUPPORT (-M)

**USING (2) - #4 = 0.4 in²**
\[
\phi = \frac{A_s - f_y}{0.85 f'c b} = \frac{(0.4 \text{ in}^2)(60 \text{ ksi})}{0.85(3 \text{ ksi})(16''')} = 0.59''
\]

**ASSUME $\phi = 0.9**

\[
\phi M_n = \phi \left[ A_s - f_y \left( d - \frac{9}{2}\right) \right] / 12 = 0.9 \left[ 0.4 \text{ in}^2 \times 60 \text{ ksi} \left( 21'' - \frac{0.59''}{2} \right) \right] / 12 = 37.27 \text{ k-ft}
\]

$\phi M_n = 37.27 \text{ k-ft} > M_u = 39.61 \text{ k-ft} \checkmark$

@ MIDSPAN (+M)

**USING (3) - #4 = 0.6 in²**
\[
\phi = \frac{A_s + f_y}{0.85 f'c b} = \frac{(0.6 \text{ in}^2)(60 \text{ ksi})}{0.85(3 \text{ ksi})(67''')} = 0.21''
\]

**ASSUME $\phi = 0.9**

\[
\phi M_n = \phi \left[ A_s - f_y \left( d - \frac{9}{2}\right) \right] / 12 = 0.9 \left[ 0.6 \text{ in}^2 \times 60 \text{ ksi} \left( 21'' - \frac{0.21''}{2} \right) \right] / 12 = 56.42 \text{ k-ft}
\]

$\phi M_n = 56.42 \text{ k-ft} > M_u = 52.79 \text{ k-ft} \checkmark$
**SHEAR CHECK**

\[ V_u = 12.52^k \]

**2-LEGGED #3 STIRRUPS**

\[ \phi V_c = \phi (2 \pi F' c \cdot b d) = 0.75 \left( 2 \times 1.0 - \sqrt{3000} \right) (16 \times 21) = 27.6^k \]

**SPACING**

\[ d_{1/2} = 21/2 = 10.5'' \Rightarrow 10'' \]

\[ \phi V_b = \frac{\phi A_v \cdot f_d}{\phi} = 0.75 \left( 2 \times 0.114 \right) (80^k) (21'') = 20.8^k \]

\[ \phi V_c + \phi V_b = 27.6^k + 20.8^k = 48.4^k \Rightarrow V_u = 12.52^k \]

**USE #3 STIRRUPS @ 10'' 9/6**
In [74]: runfile('/Users/joshuashockey/Documents/ARCE/JI/sxnprops.py', 
       wdir='/Users/joshuashockey/Documents/ARCE/JI')

b effective =  67 in

flange thickness =  5 in

h =  24 in

b =  16 in

Neutral Axis =  15.791 in

Ixx =  32793.109 in^4

fc =  3000 psi

Mcr+ =  71.09 k-ft

Mcr- =  136.753 k-ft

I cracked midspan =  2190.97 in^4

I cracked support =  1452.099 in^4

dead deflection =  0.02 in

live deflection =  0.007 in

Ma midspan [D,D+L,D+0.1L] =  [30.54 40.6  31.55] k-ft

I effective midspan [D,D+L,D+0.1L] =  [32793.109 32793.109 32793.109] in^4

Ma support [D,D+L,D+0.1L] =  [20.62 27.41 21.3 ] k-ft

I effective support [D,D+L,D+0.1L] =  [32793.109 32793.109 32793.109] in^4

deflection due to creep and live =  0.05 in  \( < \frac{1}{L/360} = 0.60" \)
deflection due to creep, live, and dead =  0.07 in  \( < \frac{L/240}{0.90"} \)
SHEET NOTES
1. TOP OF CONCRETE ELEVATION = +3m U.N.O.
2. SEE SHEET S.405 FOR TYPICAL SUSPENDED SLAB DETAILS
3. SEE S.406, S.408, & S.409 FOR SPAN JOIST DETAILS
4. SEE S.410, S.411, & S.412 FOR GIRDER DETAILS
5. SEE SHEET NOTES FOR ADDITIONAL INFORMATION

FF17

1. TOP OF CONCRETE ELEVATION = +3m U.N.O.
2. SEE SHEET S.405 FOR TYPICAL SUSPENDED SLAB DETAILS
3. SEE S.406, S.408, & S.409 FOR SPAN JOIST DETAILS
4. SEE S.410, S.411, & S.412 FOR GIRDER DETAILS
5. SEE SHEET NOTES FOR ADDITIONAL INFORMATION

FIRST FLOOR FRAMING PLAN
1:200
**MBANDAZI VILLAGE**

**PROJECT NO.**
**DATE**

**CLIENT** JI  **BY** JSS  **SHEET NO.** FF18

---

**G1**

**DL = 150 PSF**

**LL = 40 PSF**

---

**TW = 7.14'**

**L = 14.36'/2**

**TW = 6.24'**

**L = 14.36'/2**

**TW = 6.56'**

**L = 14.36'/2**

**TW = 6.56'**

**L = 14.36'/2**

---

**D = 12.88k**

**L = 2.05'**

**D = 10.97k**

**L = 2.95'**

**D = 9.39k**

**L = 3.93'**

---

**TW = 3.93'**

**L = 14.81'/2**

**TW = 1.15'**

**L = 14.81'/2**

**TW = 6.56'**

**L = 14.81'/2**

**TW = 6.56'**

**L = 14.81'/2**

---

**16" x 30"**

---

**SEE P**

**FOR ETABS RESULTS**

\[ M_{u}^- = 127.41 \text{ k-ft} \]

\[ M_{u}^+ = 110.58 \text{ k-ft} \]

\[ V_u = 29.81 \text{ k} \]

---

**ESTIMATE STEEL REINFORCEMENT**

\[ A_s^- \approx \frac{M_u^-}{f_{y} \cdot J \cdot d} = \frac{127.41 \text{ k-ft} \times 12}{0.9(60'')(0.9 	imes 27')} = 1.17 \text{ m}^2 \]

\[ \text{USE: (4) - # 5} \quad (A_s = 4 \times 0.31 = 1.24 \text{ m}^2) \]

\[ f = \frac{124}{18 \times 27} = 0.26\% \]

\[ A_s^+ \approx \frac{M_u^+}{f_{y} \cdot J \cdot d} = \frac{110.58 \text{ k-ft} \times 12}{0.9(60'')(0.9 	imes 27')} = 0.98 \text{ m}^2 \]

\[ \text{USE: (3) - # 5} \quad (A_s = 3 \times 0.31 = 0.93 \text{ m}^2) \]

\[ f = \frac{93}{18 \times 27} = 0.19\% \]
VERIFY FLEXURAL CAPACITY, $\phi M_n$

**BM DIMENSIONS:**
- $b = 18''$
- $d = 27''$
- $h = 30''$
- $b_{eff} = 2[8 + h] + b_w = 2[8(5)] + 18 = 96''$
  - $= 14.58'' = 175''$
  - $= 2[\frac{19.69'' \times 12}{8}] + 18 = 77''$

@ SUPPORT (-M)

**USING (4)** - #5 = 1.24 in²

$$
\alpha = \frac{A_s - f_{y}}{0.85 f'c b} = \frac{(1.24 \text{ in}^2)(60 \text{ ksi})}{0.85(3 \text{ ksi})(18''')} = 1.62''
$$

**ASSUME $\phi = 0.9$**

$$
\phi M_n = \phi \left[A_s - f_y \left(d - \frac{h}{2}\right)\right] / 12 = 0.9 \left[1.24 \text{ in}^2 \times 60 \text{ ksi} \left(27'' - \frac{162'''}{2}\right)\right] / 12 = 146.14 \text{ k-ft}
$$

**$\phi M_n = 146.14 \text{ k-ft}$ > $M_u = 127.41 \text{ k-ft}$ √

@ MIDSPAN (+M)

**USING (3)** - #5 = 0.93 in²

$$
\alpha = \frac{A_s + f_{y}}{0.85 f'c b} = \frac{(0.93 \text{ in}^2)(60 \text{ ksi})}{0.85(3 \text{ ksi})(77''')} = 0.28''
$$

**ASSUME $\phi = 0.9$**

$$
\phi M_n = \phi \left[A_s - f_y \left(d - \frac{h}{2}\right)\right] / 12 = 0.9 \left[0.93 \text{ in}^2 \times 60 \text{ ksi} \left(27'' - \frac{162'''}{2}\right)\right] / 12 = 112.41 \text{ k-ft}
$$

**$\phi M_n = 112.41 \text{ k-ft}$ > $M_u = 110.98 \text{ k-ft}$ √
SHEAR CHECK

\[ V_u = 29.81 \, k \]

**2-LEGGED #3 STIRRUPS**

\[ \Phi V_{lc} = \Phi \left( 2 \times \sqrt{F'c \cdot bd} \right) = 0.75 \left( 2 \times 1.0 \times \sqrt{3000} \right) (16 \times 27) = 39.9 \, k \]

SPACING \[ \phi/2 = 27/2 = 13.5'' \Rightarrow 12'' \]

\[ \Phi V_s = \frac{\Phi A_v f_d}{S} = 0.75 \left( 2 \times 0.11 \right) (1600) (24) = 22.3 \, k \]

\[ \Phi V_c + \Phi V_s = 39.9 \, k + 22.3 \, k = 62.2 \, k \]

\[ V_u = 29.81 \, k \geq 62.2 \, k \]

**USE #3 STIRRUPS @ 12'' %c**
In [78]: runfile('/Users/joshuashockey/Documents/ARCE/JI/sxnprops.py', wdir='/Users/joshuashockey/Documents/ARCE/JI')

b effective = 77 in

flange thickness = 5 in

h = 30 in

b = 18 in

Neutral Axis = 19.416 in

Ixx = 70923.715 in^4

fc = 3000 psi

Mcr+ = 125.046 k-ft

Mcr- = 229.398 k-ft

I cracked midspan = 5584.82 in^4

I cracked support = 6927.097 in^4

dead deflection = 0.014 in

live deflection = 0.004 in

Ma midspan [D,D+L,D+0.1L] = [54.9 69.6 56.4] k-ft

I effective midspan [D,D+L,D+0.1L] = [70923.715 70923.715 70923.715] in^4

Ma support [D,D+L,D+0.1L] = [78.3 99.2 80.4] k-ft

I effective support [D,D+L,D+0.1L] = [70923.715 70923.715 70923.715] in^4

deflection due to creep and live = 0.03 in \( < L/360 = 0.065" \)
deflection due to creep, live, and dead = 0.04 in \( < L/240 = 0.08" \)
FIRST FLOOR FRAMING PLAN

1:200

SHEET NOTES
1. TOP OF CONCRETE ELEVATION = +3m U.N.O.
2. SEE SHEET S.405 FOR TYPICAL SUSPENDED SLAB DETAILS
3. SEE S.406, S.408, & S.409 FOR SPAN JOIST DETAILS
4. SEE S.410, S.411, & S.412 FOR GIRDER DETAILS
5. SEE SHEET NOTES FOR ADDITIONAL INFORMATION
\[ G_2 \]

DL = 150 PSF  
LL = 40 PSF

\[ D = 6.12 \times 10\,^2 \times 1.63^2 \]

\[ L = 1.63 \times \]

\[ D = 8.56 \times 10\,^2 \times 2.92^2 \]

\[ L = 2.92 \]

\[ T_W = 10', \quad L = 34.25/2 \]

\[ T_W = 6.70', \quad L = 17.03/2 \]

\[ 18'' \times 30'' \]

SEE P FOR ETABS RESULTS

\[ M_u^- = 172.68 \, k\cdot ft \]

\[ M_u^+ = 224.04 \, k\cdot ft \]

\[ V_u = 44.4 \, k \]

**ESTIMATE STEEL REINFORCEMENT**

\[ A_{S^-} \approx \frac{M_u^-}{\Omega_{fy} J_d} = \frac{172.68 \, k\cdot ft \times 12}{0.9(60\,^{2})(0.9\times 27)} = 1.98 \, m^2 \]

**USE:** \( \text{(5)} \) - #5 \( (A_s = 5 \times 0.31 = 1.55 \, m^2) \)

\[ f_p = \frac{1.98}{18 \times 27} = 0.32\% \]

\[ A_{S^+} \approx \frac{M_u^+}{\Omega_{fy} J_d} = \frac{224.04 \, k\cdot ft \times 12}{0.9(60\,^{2})(0.93\times 27)} = 1.98 \, m^2 \]

**USE:** \( \text{(6)} \) - #5 \( (A_s = 6 \times 0.31 = 1.86 \, m^2) \)

\[ f_p = \frac{1.98}{18 \times 27} = 0.36\% \]
VERIFY FLEXURAL CAPACITY, $\phi M_n$

**BM DIMENSIONS:**
- $b = 18''$
- $d = 27''$
- $h = 30''$

$b_{eff} = 2[8''] + b_w = 2[8(5)] + 18 = 96''$

$b_{eff} = 96'' = 8\frac{12}{8}'' + 18 = 48'' \leftarrow$

@ SUPPORT (-M)

USING (5) - #5 = 1.95 in<sup>2</sup>

$$A_s = \frac{\phi a \cdot f_y}{0.85 f'c} = \frac{(155 \text{ in}^2)(60 \text{ksi})}{0.85(3 \text{ksi})(18''}) = 203''$$

ASSUME $\phi = 0.9$

$$\phi M_n = \phi \left[ A_s - f_y (d - \frac{d}{2}) \right] / 12 = 0.9 \left[ 155 \text{ in}^2 \times 60 \text{ksi} \left( 27'' - \frac{203''}{2} \right) \right] / 12 = 181.25 \text{ k-ft}$$

$$\phi M_n = 181.25 \text{ k-ft} > M_u = 172.68 \text{ k-ft} \checkmark$$

@ MIDSPAN (+M)

USING (6) - #5 = 1.86 in<sup>2</sup>

$$a = \frac{A_s + f_y}{0.85 f'c} = \frac{(186 \text{ in}^2)(60 \text{ksi})}{0.85(3 \text{ksi})(48''}) = 0.91''$$

ASSUME $\phi = 0.9$

$$\phi M_n = \phi \left[ A_s - f_y (d - \frac{d}{2}) \right] / 12 = 0.9 \left[ 186 \text{ in}^2 \times 60 \text{ksi} \left( 27'' - \frac{0.91''}{2} \right) \right] / 12 = 222.18 \text{ k-ft}$$

$$\phi M_n = 222.18 \text{ k-ft} \approx M_u = 229.04 \text{ k-ft} \checkmark$$

---

SXN@ SUPPORT

SXN@ MIDSPAN
SHEAR CHECK

$V_u = 44.4 \text{k}$

2-LEGGED #3 STIRRUPS

$\phi_{V_k} = \phi(2\times\sqrt{f_{\text{c}} \times bd}) = 0.75 \times (2 \times \sqrt{1.0 \times 3000}) \times (15 \times 27) = 39.9 \text{ k}$

SPACING $d/2 = 27/2 = 13.5'' \Rightarrow 12''$

$\phi_{V_6} = \phi(A \times f_{\text{d}} / \phi_c \times (10000) / 12'' = 22.3 \text{ k}$

$\phi_{V_c} + \phi_{V_6} = 39.9 \text{ k} + 22.3 \text{ k} = 62.2 \text{ k} \geq V_u = 44.4 \text{ k}$

USE #3 STIRRUPS @ 12'' OC
See pgs A1-A4 for code

```
In [81]: runfile('/Users/joshuashockey/Documents/ARCE/JI/sxnprops.py',
               wdir='/Users/joshuashockey/Documents/ARCE/JI')

b effective =  48 in

flange thickness =  5 in

h =  30 in

b =  18 in

Neutral Axis =  17.717 in

Ixx =  59154.891 in^4

fc =  3000 psi

Mcr+ =  114.296 k-ft

Mcr- =  164.87 k-ft

I cracked midspan =  10170.35 in^4

I cracked support =  8100.081 in^4

dead deflection =  0.039 in

live deflection =  0.01 in

Ma midspan [D,D+L,D+0.1L] =  [138.61 174.68 142.22] k-ft

I effective midspan [D,D+L,D+0.1L] =  [13564.839 12072.578 13341.647] in^4

Ma support [D,D+L,D+0.1L] =  [104.43 134.04 107.4 ] k-ft

I effective support [D,D+L,D+0.1L] =  [59154.891 19301.134 59154.891] in^4

deflection due to creep and live =  0.33 in <1" <L/360 = 0.67"

deflection due to creep, live, and dead =  0.44 in <L/240 = 1.00"
FIRST FLOOR FRAMING PLAN

1. TOP OF CONCRETE ELEVATION = +3m U.N.O.
2. SEE SHEET S.405 FOR TYPICAL SUSPENDED SLAB DETAILS
3. SEE S.406, S.408, & S.409 FOR SPAN JOIST DETAILS
4. SEE S.410, S.411, & S.412 FOR GIRDER DETAILS
5. SEE SHEET NOTES FOR ADDITIONAL INFORMATION

57mm CONC. SLAB w/ #16 @ 305mm O.C. IN SPAN DIRECTION AND #13 @ 305mm O.C. THE OTHER WAY
$G_3$

$DL = 150 \text{ PSF}$

$LL = 40 \text{ PSF}$

$D = 6.12^k$

$L = 1.63^k$

$TV = 5.75^\circ$

$L = 14.19/2$

$18'' \times 30''$

- SEE P FOR ETABS RESULTS

$M_u^- = 71.42 \text{ k-ft}$

$M_u^+ = 114.28 \text{ k-ft}$

$V_u = 14.93 \text{ k}$

**ESTIMATE STEEL REINFORCEMENT**

$A_s^- \approx \frac{M_u^-}{\phi f_y \cdot d} = \frac{71.42 \text{ k-ft} \cdot 12}{0.9(60^\circ)(0.9\times27^\circ)} = 0.69 \text{ in}^2$

USE: $(2) - 4.5 \left(A_s = 2 \times 0.31 = 0.62 \text{ in}^2\right)$

$f_s = \frac{0.62}{18 \times 27} = 0.13\%$

$A_s^+ \approx \frac{M_u^+}{\phi f_y \cdot d} = \frac{114.28 \text{ k-ft} \cdot 12}{0.9(60^\circ)(0.9\times27^\circ)} = 1.01 \text{ in}^2$

USE: $(3) - 4.5 \left(A_s = 3 \times 0.31 = 0.93 \text{ in}^2\right)$

$f_s = \frac{0.93}{18 \times 27} = 0.19\%$
VERIFY FLEXURAL CAPACITY, $\phi M_n$

BM DIMENSIONS: $b=18''$, $d=27''$, $h=30''$

$b_{eff} = 2\left[\frac{b}{2}\right] + b_w = 2\left[\frac{9}{2}\right] + 18 = 96''$

$= 7.1' = 85''$

$= \left[\frac{23 \times 12}{8}\right] + 18 = 52''$

@ SUPPORT (-M)

USING (2) - #5 = 0.62 in²

$\alpha = \frac{A_s - f_y}{0.85f'_c b} = \frac{0.62\text{ in}^2 \times 60\text{ ksi}}{0.85 \times 3\text{ ksi} \times (18'') \times 12} = 0.81''$

ASSUME $\phi = 0.9$

$\phi M_n = \phi \left[ \frac{A_s - f_y (d - \frac{9}{2})}{6} \right] / 12 = 0.9 \left[ \frac{0.62\text{ in}^2 \times 60\text{ ksi} \left(27'' - \frac{0.81}{2}''\right)}{12} \right] / 12 = 74.20 \text{ k-ft}$

$\phi M_n = 74.20 \text{ k-ft} > M_u = 71.42 \text{ k-ft} \quad \checkmark$

@ MIDSPAN (+M)

USING (3) - #5 = 0.93 in²

$\alpha = \frac{A_s + f_y}{0.85f'_c b} = \frac{(0.93\text{ in}^2 \times 60\text{ ksi}}{0.85 \times 3\text{ ksi} \times (52'') \times 12} = 0.92''$

ASSUME $\phi = 0.9$

$\phi M_n = \phi \left[ \frac{A_s - f_y (d - \frac{9}{2})}{6} \right] / 12 = 0.9 \left[ \frac{0.93\text{ in}^2 \times 60\text{ ksi} \left(27'' - \frac{0.92}{2}''\right)}{12} \right] / 12 = 112.12 \text{ k-ft}$

$\phi M_n = 112.12 \text{ k-ft} \approx M_u = 114.28 \text{ k-ft} \quad \checkmark$

Diagram:

SXP @ SUPPORT

SXP @ MIDSPAN
SHEAR CHECK

\( V_u = 14.93 \, k \)

2-LEGGED #3 STIRRUPS

\[ \phi V_{le} = \phi(2\phi \sqrt{f'c} \, bd) = 0.75 \left( 2 \times 1.0 - \sqrt{3000} \right) (16 \times 27) = 39.9 \, k \]

SPACING \( \frac{r}{2} = 27.5 = 13.5'' \Rightarrow 12'' \)

\[ \phi V_{s} = \frac{\phi A_v 6d}{s} = 0.75 \left( 2 \times 0.06m \right)(60k')(27) = 22.3 \, k \]

\[ \phi V_c + \phi V_s = 39.9 + 22.3 = 62.2 \, k > V_u = 14.93 \, k \rightleftharpoons \sqrt{ } \]

USE #3 STIRRUPS @ 12'' %
In [83]: runfile('/Users/joshuashockey/Documents/ARCE/JI/sxnprops.py', wdir='/Users/joshuashockey/Documents/ARCE/JI')
b effective =  52 in

flange thickness =  5 in

h =  30 in

b =  18 in

Neutral Axis =  17.993 in

Ixx =  61056.631 in^4

fc =  3000 psi

Mcr+ =  116.164 k-ft

Mcr- =  174.075 k-ft

I cracked midspan =  5440.652 in^4

I cracked support =  3602.809 in^4

dead deflection =  0.033 in

live deflection =  0.009 in

Ma midspan [D,D+L,D+0.1L] =  [70.28 88.99 72.15] k-ft

I effective midspan [D,D+L,D+0.1L] =  [61056.631 17541.015 61056.631] in^4

Ma support [D,D+L,D+0.1L] =  [43.92 55.62 45.09] k-ft

I effective support [D,D+L,D+0.1L] =  [61056.631 61056.631 61056.631] in^4

deflection due to creep and live =  0.14 in  <1"  
< L/360 = 0.75"

deflection due to creep, live, and dead =  0.17 in  <L/240 = 1.13"

See pgs A1-A4 for code
1. TOP OF CONCRETE ELEVATION = +3m U.N.O.
2. SEE SHEET S.405 FOR TYPICAL SUSPENDED SLAB DETAILS
3. SEE S.406, S.408, & S.409 FOR SPAN JOIST DETAILS
4. SEE S.410, S.411, & S.412 FOR GIRDER DETAILS
5. SEE SHEET NOTES FOR ADDITIONAL INFORMATION
G4
DL = 150 PSF
LL = 40 PSF

\[ D = \frac{8.02 \times 10^3}{L} = 2.14 \]
\[ D = \frac{8.02 \times 10^3}{L} = 2.14 \]
\[ D = \frac{22.56 \times 10^3}{L} = 6.02 \]

TV = 9.0'
TW = 6.0'
TW = 8.13'

18" x 30"

- SEE P FOR ETABS RESULTS

\[ M_u^- = 333.86 \text{k-ft} \]
\[ M_u^+ = 340.03 \text{k-ft} \]
\[ V_u = 61.89 \text{k} \]

ESTIMATE STEEL REINFORCEMENT

\[ A_s^- \approx \frac{M_u^-}{\phi f_y d} = \frac{333.86 \text{k-ft} \times 12}{0.9(60\text{ksi})(0.927)} = 3.05 \text{in}^2 \]

\[ \text{USE: (10) - #5} \quad (A_s = 10 \times 0.31 = 3.1 \text{ in}^2) \]

\[ f_p = \frac{3.1}{18 \times 27} = 0.64\% \]

\[ A_s^+ \approx \frac{M_u^+}{\phi f_y d} = \frac{340.03 \text{k-ft} \times 12}{0.9(60\text{ksi})(0.927)} = 3.01 \text{in}^2 \]

\[ \text{USE: (10) - #5} \quad (A_s = 10 \times 0.31 = 3.1 \text{ in}^2) \]

\[ f_p = \frac{3.1}{18 \times 27} = 0.69\% \]
VERIFY FLEXURAL CAPACITY, $\phi M_n$

BM DIMENSIONS: 
- $b = 18''$
- $d = 27''$
- $h = 30''$

$\Delta b_{eff} = 2[b + 2(b - d) + b_w = 2[b(5)] + 18 = 9.6'' \leftarrow$
- $= 20.4' = 244.8''$
- $= 2[\frac{27.25 \times 12}{8}] + 18 = 100''$

@ SUPPORT (-M)

USING (10) - # 5 = 3.1 in²

$\alpha = \frac{A_s - f_y}{0.85 f'_c b} = \frac{(3.1 \text{ in}^2)(60 \text{kpsi})}{0.85(3 \text{kpsi})(18''')} = 4.09''$

ASSUME $\phi = 0.9$

$\phi M_n = \phi \left[ A_s - f_y (d - 9.2) \right] / 12 = 0.9 \left[ 3.1 \text{ in}^2 \times 60 \text{kpsi} \left(27'' - \frac{4.09''}{2} \right) \right] / 12 = 348.40 \text{k-ft}$

$\Box M_n = 348.40 \text{k-ft} > M_u = 327.00 \text{k-ft} \checkmark$

@ MIDSSPAN (+M)

USING (10) - # 5 = 3.1 in²

$\alpha = \frac{A_s + f_y}{0.85 f'_c b} = \frac{(3.1 \text{ in}^2)(60 \text{kpsi})}{0.85(3 \text{kpsi})(9.6''')} = 0.76''$

ASSUME $\phi = 0.9$

$\phi M_n = \phi \left[ A_s + f_y (d - 9.2) \right] / 12 = 0.9 \left[ 3.1 \text{ in}^2 \times 60 \text{kpsi} \left(27'' - \frac{0.76''}{2} \right) \right] / 12 = 371.35 \text{k-ft}$

$\Box M_n = 371.35 \text{k-ft} > M_u = 335.23 \text{k-ft} \checkmark$

SXR @ SUPPORT

SXR @ MIDSSPAN
SHEAR CHECK

\[ V_u = 60.85 \text{ k} \]

2-LEGGED #3 STIRRUPS

\[ \phi V_{lc} = 0.75 \left( 2 \times 1.0 \times \sqrt{13000} \right) \left( 16 \times 27 \right) = 39.9 \text{ k} \]

SPACING \[ \phi /2 = 27/2 = 13.5'' \Rightarrow 12'' \]

\[ \phi V_s = \frac{2 \times A_{fs}d}{s} = 0.75 \left( 2 \times 0.11 \times 12 \right) \left( 27 \right) = 22.3 \text{ k} \]

\[ \phi V_c + \phi V_s = 39.9 + 22.3 \text{ k} = 62.2 \text{ k} > V_u = 60.85 \text{ k} \checkmark \]

USE #3 STIRRUPS @ 12'' %C
See pgs A1-A4 for code

In [89]: runfile('/Users/joshuashockey/Documents/ARCE/JI/sxnprops.py',
     wdir='/Users/joshuashockey/Documents/ARCE/JI')
b effective =  96 in

flange thickness =  5 in
h =  30 in
b =  18 in
Neutral Axis =  20.242 in
Ixx =  76695.565 in^4
fc =  3000 psi
Mcr+ =  129.706 k-ft
Mcr- =  269.059 k-ft
I cracked midspan =  15960.359 in^4
I cracked support =  14430.807 in^4
dead deflection =  0.09 in
live deflection =  0.023 in

Ma midspan [D,D+L,D+0.1L] =  [175.92 221.72 180.  50.  ] k-ft
I effective midspan [D,D+L,D+0.1L] =  [19736.468 18145.988 19529.378
76695.565] in^4

Ma support [D,D+L,D+0.1L] =  [206.18 260.22 211.58] k-ft
I effective support [D,D+L,D+0.1L] =  [37429.741 23493.265 34647.111]
in^4
deflection due to creep and live =  0.72 in <1" <L/360 = 0.91"
deflection due to creep, live, and dead =  1.03 in <L/240 = 1.36"
TORSION GIRDERS

\[ D_L = 150 \]
\[ L_L = 40 \]

\[ A_{CP} = 18'' \times 30'' = 540 \text{ in}^2 \]
\[ P_{CP} = 2 \left( 18'' + 30'' \right) = 96 \text{ in} \]

\[ T_{	ext{required}} = \lambda \sqrt{P C \left[ \frac{A_{WP}}{P_{CP}} \right]} \]
\[ = 1.0 \sqrt{3000 \left[ \frac{540 \text{ in}^2}{96} \right] / (1000 \times 12)} = 13.9 \text{ k-ft} \]

GIADER 4 3-4

\[ T \]

\[ 6.75' \]

\[ T/2 \]

\[ T \]

\[ 6.75' \]

\[ T/2 \]

\[ TV = 6.75' \]
\[ L = 34.25/2 \]

\[ T \]

\[ D_{iso} = \frac{(6.75)(17.14)}{1000 \times 29} = 12.39 \]
\[ T = 1.2D + 1.66 = 20.16 \]

\[ T_{w} = 20.16/2 = 10.08 \text{ k-ft} < 17.9 \text{ k-ft} \]

\[ \text{TORSIONAL REINFORCEMENT NOT NECESSARY} \]
GIRDERS J 3-4

\[
T = \frac{D \cdot 150}{L} \cdot \frac{(5.67)(6.75)^2}{1000 \times 24} = 1.61 \quad 0.93 \quad 1.2D \cdot 1.6L = 2.63
\]

\[T_w = 1.5(2.62) = 3.94 \leq 13.9\]

\[T_w = 5.67' \quad L = 13.49\]

TORSIONAL REINFORCEMENT NOT NECESSARY
GIRDER G 5-6

\[ T \quad T \]

\[ \frac{T}{D} \quad \frac{150}{40} \quad \frac{(7.19)(9)}{1000 \times 29} = \frac{3.61}{0.96} \]

1.2T + 1.6L = 5.87

\[ T_u = 1.0(5.87) = 5.87 \text{ kips} < 13.90 \text{ kips} \]

**TORSIONAL REINFORCEMENT NOT NECESSARY**
GIRDER G 6-7

\[ T = \frac{D \times 0.92 \times (9.17)^2}{1000 \times 24} = \frac{3.89}{0.96} \]

\[ Tu = 1.5(5.83) = 8.75 \text{ kN} < 13.9 \text{ kN} \]

TORSIONAL REINFORCEMENT NOT NECESSARY
**GIRDER A 6-7**

\[ T \quad T \quad T \]

\[ \frac{D_{150}}{L_{40}} \quad \frac{(9.09)(11.77)^2}{1000 \times 24} = 7.87 \quad \frac{1.20 \times 1.6L}{12.80} = 12.80 \]

\[ T_u = 1.0(12.80) = 12.80 < 13.9 \]

**TORSIONAL REINFORCEMENT NOT NECESSARY**
ESTIMATE COL SIZE

\[ \frac{KL}{r} \leq 34 - 12\left(\frac{1.11}{1.1}\right)^{0.625} \]

\[ r = \frac{KL}{34} = \frac{1(11.11 \times 12)}{34} = 4.98 \]

\[ h_{col} = \frac{r}{0.3} = \frac{4.98}{0.3} = 16.6" \]

**AREA OF FLOOR**

\[ F_u = 0.65 f_{tk} \left[ 1.2(160+30)+1.6(40+20) \right] = 183 \text{ k} \]

\[ \Phi_{Pu} = 0 \left[ 0.85 f_{ck} A_y + 0.8 f_y A_g \right] \quad \Phi = 1\% \]

\[ = 0.65 \left[ 0.85 (0.85 \times 3 \text{ ksi} \times A_g + 0.01 \times 60 \text{ ksi} \times A_y) \right] \]

\[ 1.64 A_g = 183 \text{ k} \]

\[ A_y = 112 \text{ in}^2 \]

\[ \Rightarrow h_{col} = 11" \]

USE 16" x 16" COL
SHEET NOTES
1. TOP OF CONCRETE ELEVATION = +0m U.N.O.
2. TOP OF FOOTING TO BE (0.61m) BELOW TOP OF SLAB U.N.O.
3. SEE SHEET S.001 TO S.006 FOR GENERAL NOTES
4. SEE 21/S.414 FOR CONCRETE COLUMN SCHEDULE
5. SEE S.403 & S.404 FOR FOOTING DETAILS
6. SEE 1/S.401 FOR TYPICAL SLAB-ON-GRADE DETAILS
7. SEE S.415 & S.417 FOR W.WALL ELEVATIONS

1. TOP OF CONCRETE ELEVATION = +0m U.N.O.
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7. SEE S.415 & S.417 FOR W.WALL ELEVATIONS

FOUNDATION PLAN
1 : 200
EXT COL

\[ DL_F = 160 \text{ psi} \quad LL_F = 40 \text{ psi} \quad DL_A = 25 \text{ psi} \quad LL_A = 20 \text{ psi} \]

ALONG GIADRA LINE

\[
\begin{align*}
DL_{A1} &= 25 \left( \frac{6.82' \times 11.77'}{100} \right) = 2.01' \\
DL_{A2} &= 25 \left( \frac{6.99' \times 11.77'}{100} \right) = 2.65' \\
DL_{A3} &= 25 \left( \frac{4.50' \times 3.41'}{100} \right) = 2.38' \\
DL_1 &= 160 \left( \frac{6.82' \times 11.77'}{100} \right) = 6.99' \\
DL_2 &= 160 \left( \frac{6.99' \times 11.77'}{100} \right) = 16.99' \\
DL_3 &= 160 \left( \frac{4.50' \times 3.41'}{100} \right) = 7.84' \\
LL_{A1} &= 20 \left( \frac{6.82' \times 11.77'}{100} \right) = 1.61' \\
LL_{A2} &= 20 \left( \frac{6.99' \times 11.77'}{100} \right) = 1.92' \\
LL_{A3} &= 20 \left( \frac{4.50' \times 3.41'}{100} \right) = 1.84' \\
LL_1 &= 40 \left( \frac{6.82' \times 11.77'}{100} \right) = 3.21' \\
LL_2 &= 40 \left( \frac{4.50' \times 3.41'}{100} \right) = 4.23' \\
LL_3 &= 40 \left( \frac{4.50' \times 3.41'}{100} \right) = 3.97'
\end{align*}
\]

AXIAL

\[
\begin{align*}
P_{ud} &= 6 = 1.5DL_{A1} + 1.5DL_1 + DL_{A3} + DL_3 + 0.5DL_{A2} + 0.5DL_2 = 49.30' \\
P_{ul} &= 6 = 1.5LL_{A1} + 1.5LL_1 + LL_{A3} + LL_3 + 0.5LL_{A2} + 0.5LL_2 = 16.24'
\end{align*}
\]

\[ P_u = 1.2 \left( 49.30' \times 1.6(16.24') \right) = 93.14' \]

MOMENT

SHEAR LEFT OF 6

\[
\begin{align*}
V_{ULR} &= 1.2 \left( 1.5DL_{A1} \right) + 1.6 \left( 1.5LL_{A1} \right) = 7.48' \\
V_{ULF} &= 1.2 \left( 1.5DL_1 \right) + 1.6 \left( 1.5LL_1 \right) = 30.82'
\end{align*}
\]

\[ M_{ULR} = 30.59' \quad M_{ULF} = 126.09' \]

SHEAR RIGHT OF 6

\[
\begin{align*}
V_{URR} &= 1.2 \left( 0.5DL_{A2} \right) = 1.59' \\
V_{URF} &= 1.2 \left( 0.5DL_2 \right) = 10.16'
\end{align*}
\]

\[ M_{URR} = 4.28' \quad M_{URF} = 27.33' \]

\[ M_{UR} = 26.31' \quad \text{MAX} \]

\[ M = 98.72' \quad P = 93.14' \]
ALONG JOIST LINE

\[
\begin{align*}
DL_a &= 4.34^k \\
DL &= 27.73^k \\
W_{DR} &= 25(3.91' + 3.37') = 0.17\text{kip} \\
W_{LR} &= 20(3.91' + 3.37') = 0.14\text{kip} \\
W_D &= 160(3.91' + 3.37') = 1.08\text{kip} \\
W_{L} &= 40(3.91' + 3.37') = 0.27\text{kip}
\end{align*}
\]

AXIAL

SAME AS ABOVE: \( P_u = 85.14^k \)

MOMENT

\[
\begin{align*}
V_{UR} &= 1.2 \left[ W_o \left( \frac{0.7 \times 23.54}{2} \right) \left( 0.1 \times 23.54 \right) \right] + 1.6 \left[ W_o \left( \frac{0.7 \times 23.54}{2} \right) \left( 0.1 \times 23.54 \right) \right] = 8.3^k \\
M_{UR} &= 19.51^k-ft \\
M_{UR} &= 78.75^k-ft
\end{align*}
\]

MAX

\[
\begin{align*}
M &= 98.26^k-ft \\
P &= 95.19^k
\end{align*}
\]
SHEET NOTES
1. TOP OF CONCRETE ELEVATION = +0m U.N.O.
2. TOP OF FOOTING TO BE (0.61m) BELOW TOP OF SLAB U.N.O.
3. SEE SHEET S.001 TO S.006 FOR GENERAL NOTES
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7. SEE S.415 & S.417 FOR WWALL ELEVATIONS

FOUNDATION PLAN
1 : 200
INT COL

ALONG GIRDER LINE

\[ DL_{n1} = 25 \times (7.14' \times 9.17') = 1.64^k \]
\[ LL_{n1} = 20 \times (7.14' \times 9.17') = 1.31^k \]
\[ DL_{n2} = 25 \times (8.99' \times 11.77') = 2.59^k \]
\[ LL_{n2} = 20 \times (8.99' \times 11.77') = 2.12^k \]
\[ DL_{n3} = 25 \times (6.92' \times 11.77' \times 8.13' \times 9.17') = 3.87^k \]
\[ LL_{n3} = 20 \times (6.92' \times 11.77' \times 8.13' \times 9.17') = 3.10^k \]
\[ DL_{n4} = 25 \times (7.91' \times 11.77' \times 7.64' \times 9.17') = 4.08^k \]
\[ LL_{n4} = 20 \times (7.91' \times 11.77' \times 7.64' \times 9.17') = 3.26^k \]

AXIAL

\[ P_{ud} @ 6 = 0.5(DL_{n2} + DL_{n3}) + (DL_{n1} + DL_{n1} + DL_{n1}) + 1.5(DL_{n3} + DL_{n3}) = 95.06^k \]
\[ P_{ul} @ 6 = 0.5(LL_{n2} + LL_{n3}) + (LL_{n1} + LL_{n1} + LL_{n1}) + 1.5(LL_{n3} + LL_{n3}) = 30.83^k \]
\[ P_u = 1.2(95.06) + 1.6(30.83) = 163.4^k \]

MOMENT

\[ VM_{n1} = 1.2(0.5DL_{n2} + DL_{n1}) = 5.96^k \]
\[ VM_{n2} = 1.2(0.5DL_{n2} + DL_{n1}) = 22.73^k \]
\[ VM_{n3} = 1.2(1.5DL_{n3}) = 14.41^k \]
\[ VM_{n4} = 1.2(1.5DL_{n3}) = 59.44^k \]

\[ M_{da} = 65.96^k \]
\[ M_{df} = 242.71^k \]

\[ MAX \]
\[ P = 163.4^k \]
\[ M = 242.71^k \]
ALONG JOIST LINE

\[ W_{D1} = 25(7.91') = 0.20 \text{ klf} \]
\[ W_{D2} = 25(7.64') = 0.19 \text{ klf} \]
\[ W_{L1} = 20(7.91') = 0.16 \text{ klf} \]
\[ W_{L2} = 20(7.64') = 0.15 \text{ klf} \]
\[ W_{D1} = 160(7.91') = 1.27 \text{ klf} \]
\[ W_{L1} = 40(7.91') = 0.32 \text{ klf} \]
\[ W_{D2} = 160(7.64') = 1.22 \text{ klf} \]
\[ W_{L2} = 40(7.64') = 0.31 \text{ klf} \]

AXIAL

SAME AS ABOVE: 163.9 k

MOMENT

SHEAR @ O2L LEFT

\[ V_{ULR} = 1.2 \left[ \frac{W_{D1}(0.7 \times 23.54')}{2} \right] 0.2 \times 23.54' \]
\[ = 15.33^k \]

M_{ULR} = 72.20 k-ft

SHEAR @ O2L RIGHT

\[ V_{URR} = 1.2 \left[ \frac{W_{D2}(0.7 \times 23.54')}{2} \right] 0.2 \times 23.54' \]
\[ = 8.64^k \]

M_{URR} = 32.44 k-ft

MAX

\[ P = 163.9^k \]
\[ M = 126.70^k-ft \]
### 6. Diagrams

#### 6.1. PM at $\theta=0$ [deg]

<table>
<thead>
<tr>
<th>No.</th>
<th>$P_c$</th>
<th>$M_{ex}$</th>
<th>$\phi P_c$</th>
<th>$\phi M_{ex}$</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>85.1</td>
<td>98.7</td>
<td>66.58</td>
<td>102.57</td>
<td>0.94</td>
</tr>
</tbody>
</table>

Max. Capacity Ratio: 0.94

**General Information**
- Project: exterior column
- Column: --
- Engineer: --
- Code: ACI 318-19
- Bar Set: ASTM A615
- Units: English
- Run Option: Investigation
- Run Axis: X - axis
- Slenderness: Not Considered
- Column Type: Structural
- Capacity Method: Critical capacity

**Materials**
- $f_c$: 3 ksi
- $f_y$: 60 ksi
- $f_y$: 290 ksi

**Section**
- Type: Rectangular
- Width: 16 in
- Depth: 18 in
- $A_s$: 256 in$^2$
- $I_x$: 5461.33 in$^4$
- $I_y$: 5461.33 in$^4$

**Reinforcement**
- Pattern: All sides equal
- Bar layout: Rectangular
- Cover to: Transverse bars
- Clear cover: 1.5 in
- Bars: 8 #5
- Confinement type: Tied

**Total steel area, $A_s$:** 2.48 in$^2$
**Rho:** 0.97 %
**Min. clear spacing:** 5.19 in
### 6. Diagrams

#### 6.1. PM at $\theta=0$ [deg]

<table>
<thead>
<tr>
<th>No.</th>
<th>$P_s$</th>
<th>$M_{\text{max}}$</th>
<th>$\Phi_P_s$</th>
<th>$\Phi M_{\text{max}}$</th>
<th>Capacity Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>163.4</td>
<td>242.7</td>
<td>140.39</td>
<td>251.41</td>
<td>0.95</td>
</tr>
</tbody>
</table>

Max. Capacity Ratio: 0.95

---

**General Information**
- Project: Interior column
- Column: ---
- Engineer: ---
- Code: ACI 318-19
- Bar Set: ASTM A615
- Units: English
- Run Option: Investigation
- Run Axis: X - axis
- Stenderness: Not Considered
- Column Type: Structural
- Capacity Method: Critical capacity

**Materials**
- $f_y$: 3 ksi
- $f_y'$: 3122.02 ksi
- $f_y''$: 60 ksi
- $f_y''': 29000$ ksi

**Section**
- Type: Rectangular
- Width: 20 in
- Depth: 20 in
- $A_0$: 400 in$^2$
- $I_x$: 133333.3 in$^4$
- $I_y$: 133333.3 in$^4$

**Reinforcement**
- Pattern: Equal spacing
- Bar layout: Rectangular
- Cover to: Transverse bars
- Clear cover: 1.5 in
- Bars: 16 #5
- Confinement type: Tied
- Total steel area, $A_s$: 4.96 in$^2$
- Rho: 1.24%
- Min. clear spacing: 3.28 in
TRANSVERSE BEARER

\( l_{u1} = 9.64' - 2.5' = 7.34' \)

\( l_{u2} = 7.55' - 2.5' = 5.05' \)

\( l_{o1} = \frac{b = 16''}{72''} \left( \frac{1}{3} l_{u1} = 14.7'' \right) \)

\( l_{o2} = \frac{b = 20''}{72''} \left( \frac{1}{3} l_{u2} = 10.1'' \right) \)

**EXT**
- \( b/4 = 4.5'' \)
- \( 6d_h = 3.75'' \)
- \( c_h = 4 + \left( \frac{3d_h}{2} \right) = 7.2'' \div 6'' \)
- \( h_c = 4.125'' \)

**INT**
- \( b/4 = 5'' \)
- \( 6d_h = 3.75'' \)
- \( c_h = 4 + \left( \frac{3d_h}{2} \right) = 7.07'' \div 6'' \)
- \( h_c = 4.71'' \)

**1ST STORY**
- \( 15.38'' \)
- \( A_{11} = \left( 15'' \times 2 \times 15'' \times 0.375'' \right)^2 = 234.4 \text{ in}^2 \)
- \( k_r = 1.0 \)
- \( k_n = 0.65 \times 1.0 = 1.39 \)
- \( A_n = 3.24 \text{ in}^2 \)

\( h_{1/2} = 0.0046 \times \frac{0.375''}{0.2 \times 1.0} = 0.00046 \times 1.536 = 0.0035'' \)

\( h_{1/2} = 0.0045 \times \frac{0.375''}{0.2 \times 1.39} = 0.00045 \times 1.536 = 0.0031'' \)

**USE 3 STIRRUPS @ 3 1/2''**
MBANDAZI VILLAGE

PROJECT NO. ___________________________ DATE: ___________________________

CLIENT: JI BY: JSS SHEET NO.: C011

2nd STORY

\[ A_{ch} = (18° - 2(15° - 0.375)) \times 2 = 23.9 \text{ m}^2 \]

\[ A_x = 3.29 \text{ m}^3 \]

\[ A_{sh} = \text{GREATEST} \]

\[ \frac{0.3(46 - 1)}{0.09 + 0.17} = 0.0066 \]

\[ \frac{0.3}{0.09 + 0.17} = 0.0045 \]

\[ S = \frac{A_{sh}}{0.0045} = 0.0006 + 15.36 = 3.5'' \]

USE #3 STIRRUPS @ 3.5° %

BEYOND 60

\[ S = \text{LEAST} \leq 6'' \]

\[ \leq 60 = 3.75'' \]

#3 STIRRUPS WHOLE WAY @ 3.5° %
Roof Floor

Dead 35 psf Dead 180 psf

Area 4493.71 ft²

Weight = 966147.65 Site Class D Risk Cat II

Height = 22.97 Ct = 0.02 x = 0.75

Ss = 1.58 S1 = 0.645 Eq. 11.4-2

Sms = 1.58 Sm1 = 1.097 Fa = 1

Sds = 1.05333333 Sd1 = 0.731 Fv = 1.7

Ts = 0.694

Table 12.2- R = 5

I = 1

Eq 12.8-7 T = 0.210 1.5Ts = 1.041

Actual 12.8-2 Cs = 0.211
Max 12.8-3 Cs = 0.697
Min 12.8-5/6 Cs = 0.065

V = 203.535

11.6 Design Cat D
### E/W

<table>
<thead>
<tr>
<th>Level</th>
<th>Height (ft)</th>
<th>Weight (k)</th>
<th>Wh^k (k=1)</th>
<th>Cvx</th>
<th>Fx</th>
<th>a = Fx/W</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>22.97</td>
<td>157.280</td>
<td>3612.718</td>
<td>0.312</td>
<td>63.532</td>
<td>0.404</td>
</tr>
<tr>
<td>2</td>
<td>9.8425</td>
<td>808.868</td>
<td>7961.281</td>
<td>0.688</td>
<td>140.003</td>
<td>0.173</td>
</tr>
<tr>
<td>Σ</td>
<td>-</td>
<td>966.14765</td>
<td>11573.999</td>
<td>1</td>
<td>203.535</td>
<td></td>
</tr>
</tbody>
</table>

### N/S

<table>
<thead>
<tr>
<th>Level</th>
<th>Height (ft)</th>
<th>Weight (k)</th>
<th>Wh^k (k=1)</th>
<th>Cvx</th>
<th>Fx</th>
<th>a = Fx/W</th>
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<td>7961.281</td>
<td>0.688</td>
<td>140.003</td>
<td>0.17309</td>
</tr>
<tr>
<td>Σ</td>
<td>-</td>
<td>966.148</td>
<td>203.535</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
PGA ≈ 60% = 0.6g × 1.1 = 0.66g

\[ \frac{S_s}{PGA} = 0.3986PGA + 2.1696 \Rightarrow S_s = 1.579 \]

\[ \frac{S_i}{PGA} = 0.5776PGA + 0.5967 \Rightarrow S_i = 0.645 \]
**Wall 1**

- **Height:** 19.35', 19.81', 13.69'
- **Thickness:** 19.91', 9.81'

**Calculations:**

\[ TA_1 = \left(14.35 \times 9.81\right) + \left(13.69 \times 9.81\right) = 279.07 \text{ ft}^2 \]

- **Weight:** 35 PSF
- **Coefficient:** 0.404
- **Force:** 3.89 k

\[ F_1 = 3.89 \text{ k} \]

\[ TA_2 = 279.07 + \left(9.91 \times 1.50 \times 19.81\right) = 398.14 \text{ ft}^2 \]

- **Force:** 5.63 k

**Wall 2**

- **Height:** 18.88', 42.85'
- **Thickness:** 47.78'

**Calculations:**

\[ TA_2 = 42.85 \times 18.88' = 809.01 \text{ ft}^2 \]

- **Weight:** 35 PSF
- **Coefficient:** 0.404
- **Force:** 11.44 k

\[ F_2 = 11.44 \text{ k} \]

**Note:** The calculations for **Wall 3** are not shown in the image provided.
**MBANDAZI VILLAGE**

### WALL 3

\[ TA_3 = \left(16.77' + 20.09'\right) \frac{17.27'}{2} = 318.29 \text{ ft}^2 \]

\[ \text{WEIGHT} = 35 \text{ psf} \]

\[ \alpha = 0.404 \]

\[ F_3 = 4.50 \text{ k} \]

\[ TA_4 = \left(13.47' + 21.93'\right) \frac{91.54'}{2} = 724.87 \text{ ft}^2 \]

\[ F_4 = 10.25 \text{ k} \]

### WALL 4 & 5

\[ TA_4 = TA_5 = \left(10.22' + 16.26'\right) \frac{41.54'}{2} = 550.00 \text{ ft}^2 \]

\[ \text{WEIGHT} = 35 \text{ psf} \]

\[ \alpha = 0.404 \]

\[ F_4 = F_5 = 7.78 \text{ k} \]

\[ \Sigma F_1 = 3.99 \]

\[ \Sigma F_2 = 17.07 \]

\[ \Sigma F_3 = 12.19 \]

\[ \Sigma F_4 = 18.03 \]

\[ \Sigma F_5 = 7.78 \]

\[ 63.44x \cong 63.53 \text{ k} \]
TOTAL AREA ≈ 2560 SF
WEIGHT = 35 psf
\( q = 0.904 \)
\( F_1 = 17.38k \)
\( F_2 = 18.82k \)

I'M ASSUMING THIS WILL TAKE VERY LITTLE LOAD

TOTAL AREA ≈ 1963 SF
WEIGHT = 35 psf
\( q = 0.904 \)
\( F_3 = F_4 = 13.88k \)
N/S WALLS

**P4:**

\[
\begin{align*}
\text{FROM P} & : D_1 = 99.08^k \\
L_1 &= 30.83^k \\
D_2 &= 220.83^k(160\text{sf} + 25\text{sf}) = 40.85^k \\
L_2 &= 220.83^k(40\text{sf} - 20\text{sf}) = 13.25^k \\
D_{nu} &= 150(5^* \times 18^* \times 20.67^*) = 37.21^k \\
\end{align*}
\]

\[
P_u = [1.2(2.10)] D + 0.5L + fQ_e
\]

\[
P_u = 266.13^k
\]

\[
M_u = 12.57(20.67^*) + 47.1(9.89^*) = 729.35^k ft
\]

**GOVERNING WALL**

**P6:**

\[
\begin{align*}
\text{FROM P} & : D_1 = 240.18^k(160\text{sf} + 25\text{sf}) = 99.13^k \\
L_1 & = 240.18^k(40\text{sf} - 20\text{sf}) = 19.41^k \\
D_2 &= 71.68^k(160\text{sf} + 25\text{sf}) = 13.26^k \\
L_2 &= 71.68^k(40\text{sf} - 20\text{sf}) = 4.31^k \\
D_{nu} &= 150(5^* \times 17.90^* \times 20.67^*) = 27.80^k \\
\end{align*}
\]

\[
P_u = [1.2(2.10)] D + 0.5L + fQ_e
\]

\[
P_u = 130.09^k
\]

\[
M_u = 19.21(20.67^*) + 39.38(9.89^*) = 681.02^k ft
\]
\[ P_1 = [1.2 \times 0.2(1.05)] \times 0.5 \times \phi \times Q_e \]

\[ P_u = 129.31 \, k \]

\[ M_u = 13.85\,(20.67\,') + 47.16\,(9.89\,') = 790.63 \, k \, ft \]
E/W WALLS

PS:

\[ P_u = \left[ 1.2 \times 0.2^{\frac{50}{50}} \right] D \times 0.5L \times f_{Q_e} \]

\[ P_u = 137.89^k \]

\[ M_u = 14.95(20.67') + 9.76(9.89') = 649.76 \text{ k-ft} \]

GOVERNING WALL

PB:

\[ P_u = \left[ 1.2 \times 0.2^{\frac{50}{50}} \right] D \times 0.5L \times f_{Q_e} \]

\[ P_u = 115.83^k \]

\[ M_u = 15.79(20.67') + 39.42(9.89') = 664.49 \text{ k-ft} \]
Walls 2, 10 & 12 were deleted after analysis. Considering the small amounts of shear added and the already way stronger walls, it was not rechecked.
DESIGN FORCES

\[ \frac{h}{L_{eq}} = 20.67/13 = 1.57, \quad \frac{h}{L_{ps}} = 20.67/21.4 = 0.97 \]

\[ \leq 1.5 \quad \Rightarrow \Omega = 1.0 \quad \& \quad \alpha = 3.0 \]
\[ \leq 2.0 \quad \Rightarrow \omega_c = 1.0 \]

\[ V_c = V_u \]

REINFORCEMENT

\[ P_r = 0.0025 \]
\[ A_r = 0.0025 (6'')(12''/ft) = 0.24 \text{ in}^2/ft \]

\[ \frac{A_{cv,pa}}{s} = \frac{8'' \times (18'' \times 12)}{12''} = 1728 \text{ in}^2 \]
\[ A_{cv,ps} = 8'' \times (21.4'' \times 12) = 2054.4 \text{ in}^2 \]

\[ \phi V_n = \phi A_{cv} (f'c + s_f f_y) \]

\[ \phi V_{n,pa} = 0.6 \left[ 1728 \left( 3 - \sqrt{300/1000} + 0.0032 \times 60 \right) \right] = 368.42 \Rightarrow V_u = 61.1\text{ k} \checkmark \]

\[ \phi V_{n,ps} = 0.6 \left[ 2054.4 \left( 3 - \sqrt{300/1000} + 0.0032 \times 60 \right) \right] = 437.7 \Rightarrow V_u = 69.71\text{ k} \checkmark \]
SHEET NOTES
1. TOP OF CONCRETE ELEVATION = +0m U.N.O.
2. TOP OF FOOTING TO BE (0.61m) BELOW TOP OF SLAB U.N.O.
3. SEE SHEET S.001 TO S.006 FOR GENERAL NOTES
4. SEE 21/S.414 FOR CONCRETE COLUMN SCHEDULE
5. SEE S.403 & S.404 FOR FOOTING DETAILS
6. SEE 1/S.401 FOR TYPICAL SLAB-ON-GRADE DETAILS
7. SEE S.415 & S.417 FOR WWALL ELEVATIONS

FOUNDATION PLAN
1 : 200
BOUNDARY ELEMENTS

P4

FROM ETABS P

Δ_p = 0.0074''

\[ \Delta_u = 5(0.0074) = 0.037 \]

\[ \frac{\Delta_u}{h_{wc}} = 0.037 \times \frac{20.67}{600} = 0.0021 < 0.005 \text{ USE THIS} \]

\[ C = \frac{L}{600} \times \frac{1}{\frac{15}{0.005}} = \frac{19.4 \times 12}{600 \times 15 \times 0.005} = 48'' \]

FROM spColumn: C = 20.26'' ≤ 48'' :: SPECIAL BE NOT REG'D

\[ V_{limit} = \sqrt{\frac{F_c A_c}{1000}} = \sqrt{\frac{1728}{1000}} = 94.69 > 66.63 \text{ IN WALL} \]

:: NO BE NEEDED

P5

FROM ETABS P

Δ_p = 0.0077''

\[ \Delta_u = 5(0.0077) = 0.0385 \]

\[ \frac{\Delta_u}{h_{wc}} = 0.0385 \times \frac{20.67}{600} = 0.0019 < 0.005 \text{ USE THIS} \]

\[ C = \frac{L}{600} \times \frac{1}{\frac{15}{0.005}} = \frac{21.9 \times 12}{600 \times 15 \times 0.005} = 57.07'' \]

FROM spColumn: C = 12.21'' < 57.07'' :: SPECIAL BE NOT REG'D

\[ V_{limit} = \sqrt{\frac{F_c A_c}{1000}} = \sqrt{\frac{12054}{1000}} = 112.50 \text{ > 70.47 IN WALL} \]

:: NO BE NEEDED
6. Diagrams

6.1. PM at θ=0 [deg]

General Information
Project: P4
Column: —
Engineer: —
Code: ACI318-19
Bar Set: ASTM A615
Units: English
Run Option: Investigation
Run Axis: X - axis
Stiffness: Not Considered
Column Type: Structural
Capacity Method: Critical capacity

Materials
$T_u$: 3 ksi
$E$: 3122.02 ksi
$f_y$: 60 ksi
$E_s$: 29000 ksi

Section
Type: Irregular
$A_s$: 2300 in$^2$
$I_x$: 1.35402e+007 in$^4$
$I_y$: 30486.7 in$^4$

Reinforcement
Pattern: Irregular
Bar layout: —
Cover to: —
Clear cover: —
Bars: —
Confinement type: Tied

Total steel area, $A_s$: 17.98 in$^2$
$\rho$: 0.78 %
Min. clear spacing: 0.87 in

<table>
<thead>
<tr>
<th>No.</th>
<th>$P_s$</th>
<th>$M_{ax}$</th>
<th>$\phi P_s$</th>
<th>$\phi M_{ax}$</th>
<th>Capacity</th>
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<tbody>
<tr>
<td>1</td>
<td>268.1</td>
<td>725.3</td>
<td>0.00</td>
<td>9911.86</td>
<td>0.51</td>
</tr>
</tbody>
</table>

Max. Capacity Ratio: 0.51
6. Diagrams

6.1. PM at θ=0 (deg)

General Information
Project: PS
Engineer: —
Code: ACI 318-19
Bar Set: ASTM A615
Units: English
Run Option: Investigation
Run Axis: X-axis
Slenderness: Not Considered
Column Type: Structural
Capacity Method: Critical capacity

Materials

- $f_c$: 3 kksi
- $E_c$: 3122.02 kksi
- $f_y$: 60 kksi
- $E_y$: 29000 ksi

Section

- Type: Irregular
- $A_t$: 2412.4 in$^2$
- $I_x$: 1.640/66.1007 in$^4$
- $I_y$: 28826.1 in$^4$

Reinforcement

- Pattern: Irregular
- Bar layout: ---
- Cover to: ---
- Clear cover: ---
- Bars: ---
- Confinement type: Tied

Total steel area, $A_s$: 11.16 in$^2$
Rho: 0.46 %
Min. clear spacing: 2.37 in

<table>
<thead>
<tr>
<th>No.</th>
<th>$P_s$ (kip)</th>
<th>$M_{ax}$ (k-ft)</th>
<th>$\phi P_s$</th>
<th>$\phi M_{ax}$</th>
<th>Capacity Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>137.9</td>
<td>847.9</td>
<td>0.00</td>
<td>6176.00</td>
<td>0.71</td>
</tr>
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Max. Capacity Ratio: 0.71
### AXIAL ON FRAMING

<table>
<thead>
<tr>
<th>ALONG WALL</th>
<th>FLOOR</th>
<th>ROOF</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>29.49</td>
<td>4.41</td>
</tr>
<tr>
<td>P3</td>
<td>21.75</td>
<td>13.36</td>
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<td>P4</td>
<td>67.61</td>
<td>12.98</td>
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<td>P5</td>
<td>70.70</td>
<td>14.91</td>
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<tr>
<td>P6</td>
<td>56.76</td>
<td>19.22</td>
</tr>
<tr>
<td>P7</td>
<td>93.00</td>
<td>1.219</td>
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<tr>
<td>P8</td>
<td>93.37</td>
<td>15.78</td>
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<tr>
<td>P9</td>
<td>23.58</td>
<td>4.14</td>
</tr>
<tr>
<td>P11</td>
<td>50.60</td>
<td>14.74</td>
</tr>
<tr>
<td>P13</td>
<td>70.72</td>
<td>14.50</td>
</tr>
</tbody>
</table>

**P1**

\[ L_{tot} = 41.54' \]

**FLOOR**

\[ 4.41/41.54 = 0.11 \]

\[ 4.41/18 = 0.25 \]

\[ 2.52 \times \]

**ROOF**

\[ 13.36/32.95 = 0.395 \]

\[ 13.36/20.4 = 0.655 \]

\[ 5.31 \times \]

**P3**

\[ L_{tot} = 33.85' \]

**FLOOR**

\[ 21.75/33.85 = 0.63 \]

\[ 21.75/20.4 = 1.066 \]

**ROOF**

\[ 13.36/32.95 = 0.395 \]

\[ 13.36/20.4 = 0.655 \]

\[ 8.64 \times \]
P4
L_{tot} = 91.54'

\[
\begin{align*}
12.58/41.54 &= 0.303 \\
12.58/14 &= 0.899 \\
\text{7.13} \\
\end{align*}
\]

\[
\begin{align*}
6.76/14.54 &= 1.628 \\
6.76/14 &= 3.756 \\
\text{26.31} \\
\end{align*}
\]

P5
L_{tot} = 59.8'

\[
\begin{align*}
14.91/59.8 &= 0.277 \\
14.91/21.9 &= 0.697 \\
\text{8.98} \\
\end{align*}
\]

\[
\begin{align*}
70.70/59.8 &= 1.319 \\
70.70/21.9 &= 3.304 \\
\text{42.98} \\
\end{align*}
\]

P6
L_{tot} = 47.78'

\[
\begin{align*}
14.22/47.78 &= 0.298 \\
14.22/13.90 &= 1.053 \\
\text{10.20} \\
\end{align*}
\]

\[
\begin{align*}
56.76/47.78 &= 1.188 \\
56.76/13.90 &= 4.206 \\
\text{98.79} \\
\end{align*}
\]

P8
L_{tot} = 39.29'

\[
\begin{align*}
15.78/39.29 &= 0.402 \\
15.78/19.62 &= 0.804 \\
\text{7.9} \\
\end{align*}
\]

\[
\begin{align*}
53.39/39.29 &= 1.357 \\
53.39/19.62 &= 2.721 \\
\text{26.73} \\
\end{align*}
\]
A W8×13 has a OPn of 173 kN, worst case 218 kN @ Lc = 19

All steel members will work
This was my worst case axial load on the smallest concrete framing member. It is adequate and as are all other concrete framing members.
SHEET NOTES
1. TOP OF CONCRETE ELEVATION = +0m U.N.O.
2. TOP OF FOOTING TO BE (0.61m) BELOW TOP OF SLAB U.N.O.
3. SEE SHEET S.001 TO S.006 FOR GENERAL NOTES
4. SEE 21/S.414 FOR CONCRETE COLUMN SCHEDULE
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7. SEE S.415 & S.417 FOR WWALL ELEVATIONS

FOUNDATION PLAN
1 : 200
EXT COL FOUNDATION

18' x 18' COL

ALLOWABLE BEARING PRESSURE $f_b = 2$ ksf

$P_u = P_{ed} = 49.30 + 1624 \times 65.54$ ksf

Allowable $P_u = 65.54 \times \frac{1}{2} = 33$ ksf $=> 6$' SQ

$c = \frac{1}{2} (6' - \frac{18}{12}) = 2.25'$

d = $\sqrt{\frac{P_{ed} \times c}{f_b}} = 6.7'' \Rightarrow 8''$

$\therefore h = d + 4 = 12''$

$A_s = 0.0018bh = 0.0018(6' \times 12')(12'') = 1.86$ in$^2$

USE (5) - 5

1.55 in$^2$ OR 0.26 in$^2$ ft$^2$

FROM $P$

$f_{bu} = 59.19 \times \frac{1}{2} = 2.4$ ksf

$\uparrow$

FACTORED $P$

BEAM SHEAR

$V_u = f_{bu} \times c = 2.4 \times 2.25' = 5.4$ ksf

$\phi V_u = 0.75 \times 2.4 \times 6 \times 8'' = 7.89$ ksf $> V_u \checkmark$
**PUNCHING SHEAR**

**BASE VALUE:**

\[ 4 - \sqrt[3]{f_c} = 4 \cdot \sqrt[3]{3000} = 219.09 \text{ ksi} \]

**COL SHAPE:**

\[ \left[ (2 - \frac{b}{2}) + \frac{1}{2} \right] \cdot \sqrt{f_c} = \left[ (2 + 4) \cdot \sqrt{3000} \right] = 328.63 \text{ ksi} \]

\[ B_c = \frac{b}{h_{col}} = 1 \]

\[ A_{CI} = 22.053 \text{ ksi} \]

**COL LOCATION:**

\[ \left( \alpha \cdot \frac{d}{b_0} + 2 \right) \cdot \sqrt{f_c} = \left( \frac{40 \times 0.5}{104} + 2 \right) \cdot \sqrt{3000} = 278.1 \text{ ksi} \]

\[ \Rightarrow 2 \left[ (b_{col} \cdot d) + (h_{col} \cdot d) \right] = 104'' \]

\[ 0.255 = 0.75 \left[ 4 \cdot 1.0 \cdot \sqrt{3000} \left( 104 \times 8 \right) \right] / 1000 = 136.7 \text{ ksi} \]

\[ L = \frac{1}{\alpha \cdot h_0} = 1.05 \leq 1 \times \]

**V_u = P_u - (f_{cu} \cdot A_{b0}) = 88.14 - (2.9 \times \left( \frac{26.26}{149} \right)) = 74 < \Phi V_h \]

**FLEXURE**

\[ M_u = \left( 2.4 \text{ ksf} \right) \left( 2.25 \text{ ft} \right)^2 = 6.08 \text{ k-ft-ft} \]

\[ \alpha = \left( \frac{0.26 \times 60 \times 1.5}{0.85 \times 3 \times 12} \right) = 0.91 \]

\[ \Phi M_h = 0.9 \left( 0.26 \times 60 \left( C - 0.5 \times 2 \right) \right) / 12 = 9.06 \text{ k-ft-ft} > M_u \]

**DEVELOPMENT LENGTH**

\[ f_y = 60 \text{ ksi} \]

\[ f'c = 3 \text{ ksi} \]

\[ \lambda = 1.0 \text{ NWC} \]

\[ \psi_t = 1.0 \text{ OTHER} \]

\[ \psi_c = 1.0 \text{ UNCOATED/ZINC-COATED (GALVANIZED)} \]

\[ \psi_g = 1.0 \text{ GRADE 60} \]

\[ \lambda d = 27.39'' < c = 2.25 \times 12 = 27'' \]

**EXT COL FTGS:** 6' SQ x 12" (S)-#5 E.W.E.F.
1. TOP OF CONCRETE ELEVATION = +0m U.N.O.
2. TOP OF FOOTING TO BE (0.61m) BELOW TOP OF SLAB U.N.O.
3. SEE SHEET S.001 TO S.006 FOR GENERAL NOTES
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FOUNDATION PLAN
1 : 200
INT COL FOUNDATION

20x20 COL

ALLOWABLE BEARING PRESSURE  \( f_b = 2 \text{ ksf} \)

\[ P_u = 95.06 + 30.89 = 125.95 \text{ k} \]

\[ A_{geo} = \frac{125.95}{2} \text{ ksf} = 63.97 \text{ sq} \]

\[ C = \frac{(12)(8 - 20)}{12} = 3.17' \]

\[ d = 2.2 \sqrt{\frac{P_u}{f_b} \cdot C^2} = 9.8'' \geq 16'' \]

\[ h = d + 4 = 20'' \]

\[ A_s = 0.0016bh = 0.0016(8'x12')(20'') = 3.46'' \]

**USE (12) - 5**

3.72 in on 0.47 in.  

FROM P

\[ f_{bu} = 163.9 \text{ ksf} \]

FACTORED PU

BEAM SHEAR

\[ V_u = f_{bu} \cdot C = 4.54 \text{ ksf} \cdot 3.17' = 14.9 \text{ ksf} \]

\[ 0.75 \left[ 2 \sqrt{3000(12'' \times 16'')} \right] / 1000 = 15.77 \text{ ksf} > V_u \checkmark \]
PUNCHING SHEAR

BASE VALUE: \[ 4 \sqrt{f_{c}} = 4 \sqrt{3000} = 219.09 \text{ kips} \]

COL SHAPE: \[ \left[(2+\frac{9}{16}) \sqrt{f_{c}} \right] = \left[(2+4) \sqrt{3000} \right] = 328.63 \]

\[ C = \frac{b_{w}}{h_{w}} > 1 \]

\[ ACI \quad 22.6.6.3 \]

COL LOCATION: \[ \left[(\alpha - \frac{3}{4} + 2) \sqrt{f_{c}} \right] = \left[(\frac{40 \times 16}{144} + 2) \sqrt{3000} \right] = 353 \]

\[ l = 2 \left[ \left(b_{w} \times d\right) + \left(h_{w} \times d\right) \right] = 144 \]

\[ 0.75 \]

\[ \phi V_{n} = 0.4 \lambda \alpha \sqrt{f_{c}} \left(b_{w} \times d\right) = 0.75 \left[4 \times 0.28 \times \sqrt{3000} \left(144 \times 16\right) \right] / 1000 = 333.16 \text{ kips} \]

\[ \lambda = \frac{3}{1.37} = 0.98 \leq 1 \]

\[ V_{u} = R_{u} - (f_{w} \times A_{h_{w}}) = 163.4 - (4.54 \times \left(\frac{36+26}{144}\right)) = 123 \text{ kips} < \phi V_{n} \checkmark \]

FLEXURE

\[ M_{u} = \left(\frac{1}{2}\right) \left(4.54 \times 3.17^2 \right) = 22.9 \text{ kips} / \text{ft} \]

\[ C = (0.47 \times 16 \times 60) / (0.95 \times 3 \times 12) = 0.92 \]

\[ \phi M_{n} = 0.9 \left[0.47 \times 60 \left(16 - 0.92^2\right) / 12 = 22.9 \text{ kips} / \text{ft} \right] > M_{u} \checkmark \]

DEVELOPMENT LENGTH

\[ f_{y} = 60 \text{ ksi}; \quad f'_{c} = 3 \text{ ksi}; \quad \lambda = 1.0 \text{ NWC} \]

\[ a_{e} = 1.0 \text{ OTHER} \]

\[ a_{c} = 1.0 \text{ UNCOATED/ZINC-COATED (GALVANIZED)} \]

\[ a_{g} = 1.0 \text{ GRADE 60} \]

\[ l_{d} = 27.39'' < C = 3.17 \times 12 = 38'' \checkmark \]

INT COL FTGS: 8" SQ X 20" (12) - #5 E.W.E.F.
SHEET NOTES
1. TOP OF CONCRETE ELEVATION = +0m U.N.O.
2. TOP OF FOOTING TO BE (0.61m) BELOW TOP OF SLAB U.N.O.
3. SEE SHEET S.001 TO S.006 FOR GENERAL NOTES
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7. SEE S.415 & S.417 FOR WWALL ELEVATIONS
FINDING AXIAL ON COLS W/ WANS

<table>
<thead>
<tr>
<th>WALL</th>
<th>COL 1 (k)</th>
<th>COL 2 (k)</th>
<th>V2 WALL</th>
<th>T/C (WORST CASE)</th>
<th>WORST COMP (k)</th>
<th>WORSE TENS (k)</th>
<th>FACTORED</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>50.16</td>
<td>24.35</td>
<td>18.60</td>
<td>20.60</td>
<td>99.36</td>
<td>5.13</td>
<td>133.82</td>
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<td>P3</td>
<td>43.63</td>
<td>21.66</td>
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<td>4.72</td>
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<td>30.93</td>
<td>18.61</td>
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<td>176.29</td>
<td>9.02</td>
<td>21.69</td>
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<td>40.95</td>
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<td>22.12</td>
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<td>44.43</td>
<td>14.41</td>
<td>13.95</td>
<td>63.10</td>
<td>111.13</td>
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<td>23.29</td>
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<td>24.89</td>
<td>4.61</td>
<td>20.28</td>
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<td>90.41</td>
<td>-12.99</td>
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<td>36.46</td>
<td>8.07</td>
<td>14.83</td>
<td>69.33</td>
<td>107.36</td>
<td>-30.64</td>
<td>110.06</td>
</tr>
</tbody>
</table>

COMP: \((1 \times 0.145\%) D \times 0.75 f_{Qe}\)

\((1 \times 0.105\%) D \times 0.75 L \times 0.525 f_{Qe}\)

TENS: \((0.6 \times 0.145\%) D \times 0.75 f_{Qe}\)

FACTORED: \[(1.2 \times 0.25\%) D \times L \times f_{Qe}\]

DESIGN FTG FOR WALLS USING \(A = 176.29^k\)

\(u = -31.84^c\)

\(A_{req, D} = 176.29^k \times 9.4^f = 9.4^d \Rightarrow 9.5^e\)

\(C = \frac{1}{2} \times (9.5^f - \frac{13}{2}) = 4^g\)

\(d = 2.21 \times \sqrt{\frac{14^h \times 9^i}{4^j \times 4^k}} \approx 12.3^l \Rightarrow 13^m\)

\(h > 13\times 4 = 17^n\)

\(h = \frac{31.84}{150 \times 9.5 \times 9.5 \times 1000} = 2.4^p \Rightarrow 2.5^q\)

\(h = 2.5^r = 30^s\)

\(d = 26^t\)
\[ A_s = 0.00186bh = 0.00186(35 \times 12)(30) = 6.16 \text{ m}^2 \]

**USE (20)**

USE (20) x 5

6.2 m² or 0.65 m²

\[ f_{bu} = 211.69 \times 40.25^{\circ} = 2.35 \text{ ksf} \]

\[ f_{bc} = \text{FACTORED F} \]

**BEAM SHEAR**

\[ V_u = f_{bu} \times c = 2.35 \text{ ksf} \times 4" = 9.4 \text{ ksf} \]

\[ \phi V_u = \phi (25FC) \text{ bd} \]

\[ = 0.75 \left( \frac{2.13000(2" \times 26")}{1000} \right) = 25.63 \text{ ksf} > V_u \checkmark \]

**PUNCHING SHEAR**

**BASE VALUE:**

\[ 4\sqrt{f'c} = 4 \sqrt{3000} = 219.07 \leftarrow \]

**COL SHAPE:**

\[ \{(2 + 0.8)\sqrt{f'c}\} = \{ (2 + 4)\sqrt{3000}\} = 328.63 \]

\[ \phi c = \frac{b_{col}}{d_{col}} = 1 \]

\[ b_{col} = 22.635 \text{ in} \]

\[ 40 \text{ ACI 22.635} \]

**COL LOCATION:**

\[ \{(1.76^{\circ} + 2)\sqrt{f'c}\} = \left( \frac{40 \times 26\text{ in}}{176} \right) = 4.33 \text{ ksf} \]

\[ 2\left[ (b_{col} \times d) + (d_{col} \times d) \right] = 176 \]

\[ 2 \left[ (b_{col} \times d) + (d_{col} \times d) \right] = 176 \]

\[ \frac{0.75}{2(0.44)} = 0.75 \leq 1 \]

\[ \phi V_u = \phi (4A_s \times f'c) \times (b \times d) = 0.75 \left[ 4 \times 0.75 \times \sqrt{3000} (176 \times 26) \right] / 1000 = 569 \text{ ksf} \]

\[ \phi V_u \checkmark \]

\[ V_u = P_u - (f_{bu} \times A_{bo}) = 211.69 - \left( 2.35 \times \frac{44 \times 44}{144} \right) = 188 \text{ ksf} < \phi V_u \checkmark \]
FLEXURE

\[ M_u = \left( \frac{1}{2} \right) (2.35 \text{ ksf}) (4 \text{ ft})^2 = 19.8 \text{ k-ft/ft} \]

\[ \alpha = \left( \frac{0.65}{\text{ksi}} \times 60 \text{ ksi} \right) \left( \frac{5}{\text{in}} \times \frac{12}{\text{in}} \right) = 1.27 \]

\[ \hat{M}_u = 0.9 \left( 0.65 \times 60 \text{ ksi} - \frac{1}{2} \right) / 12 = 74.19 \text{ k-ft/ft} \]

\[ \lambda > M_u \checkmark \]

DEVELOPMENT LENGTH

\[ f_y = 60 \text{ ksi} \quad f_c = 3 \text{ ksi} \quad \lambda = 1.0 \quad \text{NWC} \]

\[ \psi_t = 1.0 \quad \text{OTHER} \quad db = 0.62 \]

\[ \psi_c = 1.0 \quad \text{UNCOATED/ZINC-COATED (GALVANIZED)} \]

\[ \psi_g = 1.0 \quad \text{GRADE 60} \]

\[ l_d = 27.39'' < c = 4' \times 12 = 48'' \checkmark \]

**WALL**

FTGS: 9.5' SQ \times 30'' (20) - #5 E.W.E.F.
IRREGULARITIES

E/W

TORSONAL/EXTREME TORSIONAL

$\delta_1 = 0.013$

$1.2(\delta_1 + \delta_2)/2 = 0.0081 > \delta_2$

$\delta_2 = 0.0005$

$1.9(\delta_1 + \delta_2)/2 = 0.0095 > \delta_2$

EXTREME TORSIONAL IRREGULARITY

N/S

$\delta_1 = 0.0023$

$1.2(\delta_1 + \delta_2)/2 = 0.0050 < \delta_2$

$\delta_2 = 0.0061$

$1.9(\delta_1 + \delta_2)/2 = - > \delta_2$

NO TORSIONAL IRREGULARITY

RE-ENTRANT CORNERS IRREGULARITY

NO DIAPHRAGM IRREGULARITY

NO OUT OF PLANE OFFSET IRREGULARITY

NON PARALLEL IRREGULARITY

NO STIFFNESS IRREGULARITY - SOFT STORY

NO WEIGHT IRREGULARITY

NO VERTICAL GEOMETRIC IRREGULARITY

NO IN-PLANE DISCONTINUITY IN VERT. CAT. FORCE RESISTING ELEMENT

NO DISCONTINUITY IN LATERAL WEAK STORY
```python
import numpy as np

# Beam Section Properties
beff = 60  # in
f_flange = 5  # in
h = 24  # in
b = 16  # in
h_pri = h - f_flange  # in

print('b effective = ', beff, 'in 
')
print('flange thickness = ', f_flange, 'in 
')
print('h = ', h, 'in 
')
print('b = ', b, 'in 
')

# Steel Properties
d = h - 3  # in
d_pri = 3  # in
As_pos_bot = 0.4  # in^2 ; + moment rebar at bottom
As_pos_top = 0.4  # in^2 ; + moment rebar at top
As_neg_bot = 0.4  # in^2 ; - moment rebar at bottom
As_neg_top = 0.4  # in^2 ; - moment rebar at top

# Finding Neutral Axis

# Flange
A_flange = f_flange * beff  # in^2
y_flange = h - f_flange / 2  # in
Ay_flange = A_flange * y_flange  # in^3

# Web
A_web = b * h_pri  # in^2
y_web = (h - f_flange) / 2  # in
Ay_web = A_web * y_web  # in^3

A_total = A_flange + A_web
Ay_tot = Ay_flange + Ay_web

y_NA = Ay_tot / A_total
print('Neutral Axis = ', np.round(y_NA, 3), 'in 
')
```

```
\[
d_{\text{flange}} = y_{\text{flange}} - y_{\text{NA}} \\
A_{\text{d,flange}} = A_{\text{flange}} \times (h_{\text{pri}} + t_{\text{flange}}/2 - y_{\text{NA}})^2 \\
I_{\text{o,flange}} = b_{\text{eff}} \times (t_{\text{flange}}^3)/12
\]

\[
d_{\text{web}} = y_{\text{NA}} - y_{\text{web}} \\
A_{\text{d,web}} = A_{\text{web}} \times (y_{\text{NA}} - h_{\text{pri}}/2)^2 \\
I_{\text{o,web}} = b \times (h_{\text{pri}}^3)/12
\]

\[
I_{\text{xx, tot}} = I_{\text{o,flange}} + A_{\text{d,flange}} + A_{\text{d,web}} + I_{\text{o,web}} \quad \text{#in}^4
\]

\[
\text{print('Ixx = ',np.round(I_{\text{xx, tot}},3),'} \quad \text{in}^4 \text{ \n')}
\]

\[
# \text{Cracked Moment} \\
# \text{Modulus of Rupture} \\
f_c = 3000 \quad \text{#psi} \\
f_r = 7.5*1*\text{np.sqrt}(f_c)/1000 \quad \text{ksi} \; \text{; 1 is lambda for NWC} \\
M_{\text{cr, pos}} = f_r \times I_{\text{xx, tot}}/(y_{\text{NA}} \times 12) \quad \text{#k-ft} \\
M_{\text{cr, neg}} = f_r \times I_{\text{xx, tot}}/((h_{\text{NA}} - y_{\text{NA}}) \times 12) \quad \text{#k-ft} \\
\text{print('f_c = ',f_c,' psi \n')} \\
\text{print('M_{cr, pos} = ',np.round(M_{\text{cr, pos}},3),' k-ft \n')} \\
\text{print('M_{cr, neg} = ',np.round(M_{\text{cr, neg}},3),' k-ft \n')}
\]

\[
# \text{Cracked Section} \\
E_c = 57000*\text{np.sqrt}(f_c)/1000 \quad \text{ksi} \\
E_s = 29000 \quad \text{ksi} \\
n = E_s/E_c
\]

\[
# \text{Midspan} \\
# PNA & I cracked \\
a_{\text{mid}} = b_{\text{eff}}/2 \\
b_{\text{mid}} = n \times A_{\text{pos bot}} + (n-1) \times A_{\text{pos top}} \\
c_{\text{mid}} = -(n \times A_{\text{pos bot}} \times d) - ((n-1) \times A_{\text{pos top}} \times d_{\text{pri}}) \\
discriminant_{\text{mid}} = b_{\text{mid}}^2 - 4 \times a_{\text{mid}} \times c_{\text{mid}} \\
y_{\text{mid}} = (-b+\text{np.sqrt(discriminant_{\text{mid}})))/(2 \times a_{\text{mid}}) \\
I_{\text{cr, mid}} = b_{\text{eff}} \times (y_{\text{mid}}^3)/3 + (n \times A_{\text{pos bot}} \times (d-y_{\text{mid}})^2) + ((n-1) \times A_{\text{pos top}} \times (y_{\text{mid}}-d_{\text{pri}})^2) \\
\text{print('I cracked midspan = ',np.round(I_{\text{cr, mid}},3),' in}^4 \text{ \n')}
\]

\[
# \text{Support} \\
# PNA & I cracked \\
a_{\text{sup}} = b/2 \\
b_{\text{sup}} = n \times A_{\text{neg bot}} + (n-1) \times A_{\text{neg top}} \\
c_{\text{mid}} = -(n \times A_{\text{neg bot}} \times d) - ((n-1) \times A_{\text{neg top}} \times d_{\text{pri}}) \\
discriminant_{\text{mid}} = b_{\text{mid}}^2 - 4 \times a_{\text{mid}} \times c_{\text{mid}} \\
y_{\text{sup}} = (-b+\text{np.sqrt(discriminant_{\text{mid}})))/(2 \times a_{\text{mid}}) \\
I_{\text{cr, sup}} = b_{\text{eff}} \times (y_{\text{sup}}^3)/3 + (n \times A_{\text{neg bot}} \times (d-y_{\text{sup}})^2) + ((n-1) \times A_{\text{neg top}} \times (y_{\text{sup}}-d_{\text{pri}})^2) \\
\text{print('I cracked support = ',np.round(I_{\text{cr, sup}},3),' in}^4 \text{ \n')}
\]
#Deflection

defl_dead = 0.005  #etabs
defl_live = 0.002  #etabs
print('dead deflection = ',defl_dead,'in \n')
print('live deflection = ',defl_live,'in \n')

#Midspan
Ma_mid = np.array([13.05,17.40,13.49])  #Ma for D,D+L,D+0.1L @ midspan
Ma_Mcr_mid = ((2/3)*(Mcr_pos/Ma_mid))**2
Icr_Ig_mid = 1-(Icr_mid/Ixx_tot)
Ieff_mid = Icr_mid/(1-(Ma_Mcr_mid*Icr_Ig_mid))
for i in range(0,len(Ma_mid)):
    if Ma_mid[i]<=2*Mcr_pos/3:
        Ieff_mid[i] = Ixx_tot
print('Ma midspan [D,D+L,D+0.1L] = ',Ma_mid,'k-ft \n')
print('I effective midspan [D,D+L,D+0.1L] = ',np.round(Ieff_mid,3),'in^4 \n')

#Support
Ma_sup = np.array([18.00,24.00,18.60])  #Ma for D,D+L,D+0.1L @ support
Ma_Mcr_sup = ((2/3)*(Mcr_neg/Ma_sup))**2
Icr_Ig_sup = 1-(Icr_sup/Ixx_tot)
Ieff_sup = Icr_sup/(1-(Ma_Mcr_sup*Icr_Ig_sup))
for i in range(0,len(Ma_sup)):
    if Ma_sup[i]<=2*Mcr_neg/3:
        Ieff_sup[i] = Ixx_tot
print('Ma support [D,D+L,D+0.1L] = ',Ma_sup,'k-ft \n')
print('I effective support [D,D+L,D+0.1L] = ',np.round(Ieff_sup,3),'in^4 \n')

#Immediate D Deflection
I_d = 0.85*Ieff_mid[0]+0.15*Ieff_sup[0]
delta_d = defl_dead*Ixx_tot/I_d

#Immediate Live Deflection
I_dl = 0.85*Ieff_mid[1]+0.15*Ieff_sup[1]
delta_l = (defl_dead+defl_live)*(Ixx_tot/I_dl)-delta_d

#Longterm Deflection
I_d1l = 0.85*Ieff_mid[2]+0.15*Ieff_sup[2]
delta_d1l_def = defl_dead+0.1*defl_live
delta_d1l_I = Ixx_tot/I_d1l
delta_d1l = delta_d1l_def*delta_d1l_I

#Creep
epsilon = 2
rho_prime = As_pos_top/(b*d)
lam = epsilon/(1+50*rho_prime)
delta_creep = lam*delta_d1l

delta_l_creep = np.round(delta_l+delta_creep,3)
delta_d_l_creep = np.round(delta_l_creep+delta_d,3)
print('deflection due to creep and live = ',delta_l_creep,'in \n')
print('deflection due to creep, live, and dead = ',delta_d_l_creep,'in \n')
Beam B1

Fig 1: Max Positive Moment Diagram

Fig 1.1: Max Negative Moment Diagram

Fig 2: Shear Diagram

Figure 3: Dead Deflection

Figure 4: Live Deflection
Figure 5: $M_d$ @ midspan

Figure 6: $M_d$ @ support

Figure 7: $M_d + l$ @ midspan

Figure 8: $M_d + l$ @ support

Figure 9: $M_d + 0.1l$ @ midspan

Figure 10: $M_d + 0.1l$ @ support
This can follow B3 design, but an extra bar is needed at the top.
Beam B2

Fig 11: Moment Diagram

Fig 12: Shear Diagram

Figure 13: Dead Deflection

Figure 14: Live Deflection
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Moment M3 Diagram</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>Md @ midspan</td>
<td><img src="image" alt="Figure 15 Md @ midspan" /></td>
</tr>
<tr>
<td>16</td>
<td>Md @ support</td>
<td><img src="image" alt="Figure 16 Md @ support" /></td>
</tr>
<tr>
<td>17</td>
<td>Md+1 @ midspan</td>
<td><img src="image" alt="Figure 17 Md+1 @ midspan" /></td>
</tr>
<tr>
<td>18</td>
<td>Md+1 @ support</td>
<td><img src="image" alt="Figure 18 Md+1 @ support" /></td>
</tr>
<tr>
<td>19</td>
<td>Md+0.1l @ midspan</td>
<td><img src="image" alt="Figure 19 Md+0.1l @ midspan" /></td>
</tr>
<tr>
<td>20</td>
<td>Md+0.1l @ support</td>
<td><img src="image" alt="Figure 20 Md+0.1l @ support" /></td>
</tr>
</tbody>
</table>
Beam B3

Fig 21: Moment Diagram Pin-Pin

Fig 22: Moment Diagram Pin-Pin

Fig 23: Shear Diagram

Figure 24: Dead Deflection

Figure 25: Live Deflection

Figure 26: Md @ midspan

Figure 27: Md @ support
Figure 28: Md+1 @ midspan

Figure 29: Md+1 @ support

Figure 30: Md+0.1l @ midspan

Figure 31: Md+0.1l @ support
Fig 32: Moment Diagram

Fig 33: Shear Diagram

Figure 34: Dead Deflection

Figure 35: Live Deflection

Figure 36: Md @ midspan

Figure 37: Md @ support
Figure 38: Md+l @ midspan

Figure 39: Md+l @ support

Figure 40: Md+0.1l @ midspan

Figure 41: Md+0.1l @ support

Figure 41.1: Torsion
Fig 42: Moment Diagram

Fig 43: Shear Diagram

Figure 44: Dead Deflection

Figure 45: Live Deflection

Figure 46: Md @ midspan

Figure 47: Md @ support
Figure 48: $M_d + l$ @ midspan

Figure 49: $M_d + l$ @ support

Figure 50: $M_d + 0.1l$ @ midspan

Figure 51: $M_d + 0.1l$ @ support

Figure 51.1: Torsion
Figure 59: \( M_{d+1} @ \) midspan

Figure 60: \( M_{d+1} @ \) support

Figure 61: \( M_{d+0.1} @ \) midspan

Figure 62: \( M_{d+0.1} @ \) support

Figure 62.1: Torsion
Girder G4

Fig 63: Moment Diagram Pinned Rxn

Fig 64: Shear Diagram

Figure 65: Dead Deflection

Figure 66: Live Deflection

Figure 67: Md @ midspan

Figure 68: Md @ support
Figure 69: $M_d + l$ @ midspan

Figure 70: $M_d + l$ @ support

Figure 71: $M_d + 0.1l$ @ midspan

Figure 72: $M_d + 0.1l$ @ support
ROOF ETABS OUTPUT

Girder G1

Deflection (Down +)

I End Jt: 1
J End Jt: 2
0.582702 in at 13.6400 ft

Figure 73: Live Deflection

Girder G2

Deflection (Down +)

I End Jt: 1
J End Jt: 2
0.374897 in at 10.0466 ft

Figure 74: Live Deflection

Girder G3

Deflection (Down +)

I End Jt: 1
J End Jt: 2
0.322534 in at 9.0350 ft

Figure 75: Live Deflection

Beam B2

Deflection (Down +)

I End Jt: 1
J End Jt: 2
0.270427 in at 17.3350 ft

Figure 76: Live Deflection
**LATERAL ETABS OUTPUT**

**Table 1: N/S EQ wall forces**

<table>
<thead>
<tr>
<th>Story</th>
<th>Pier</th>
<th>Output Case</th>
<th>Step Type</th>
<th>Location</th>
<th>VZ (kip)</th>
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<tbody>
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<td>Story2</td>
<td>P1</td>
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<td>Max</td>
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**Table 2: E/W EQ wall forces**

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<th>Step Type</th>
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<th>VZ (kip)</th>
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</table>

**Figure 77: Wall Callouts**
Figure 78: P4 Forces

Figure 79: P4 Deflection

Figure 80: P5 Forces

Figure 81: P5 Deflection

*Based off of before deleted walls
Figure 82: Delta u 1 E/W

Figure 83: Delta u 2 E/W
Figure 84: Delta u 1 N/S

Figure 85: Delta u 2 N/S
GENERAL CRITERIA

1. ALL WORK SHALL BE CARRIED OUT IN ACCORDANCE WITH THE PROJECT SPECIFICATIONS. IN THE EVENT OF CONFLICT BETWEEN THE SPECIFICATION AND THE DRAWINGS CONSULT WITH THE ARCHITECT.

2. THE STRUCTURAL DRAWINGS SHOW THE STRUCTURAL CONSTRUCTION REQUIREMENTS FOR THE BUILDING. ITEMS REQUIRING COORDINATION WITH THE ARCHITECTURAL DRAWING INCLUDE, BUT ARE NOT LIMITED TO THE FOLLOWING:
   A. SIZE AND LOCATION OF CONCRETE SLAB DEPRESSIONS, STEPS, AND CURBS
   B. ELEVATIONS, SLOPES, AND DETAILS OF TOPPING SLABS OVER STRUCTURAL DECKS.
   C. EXPOSED CONCRETE FLOOR SLAB FINISHED, SLOPES, AND SCORED JOINT LOCATIONS.

3. ITEMS REQUIRING COORDINATION WITH MECHANICAL, PLUMBING AND ELECTRICAL DRAWINGS INCLUDE, BUT ARE NOT LIMITED TO THE FOLLOWING:
   A. WALL AND SLAB OPENINGS FOR PIPE RUNS, SLEEVES, HANGARS, TRENCHES ETC.
   B. WALL AND SLAB OPENING FOR ELECTRICAL CONDUIT, BOXES OR OUTLETS.
   C. CONCRETE INSERTS FOR ELECTRICAL, MECHANICAL OR PLUMBING FIXTURES.
   D. SIZE AND LOCATION OF MACHINE OR EQUIPMENT BASES, PADS, AND ANCHORS BOLTS.

4. TYPICAL DETAILS ARE INTENDED TO APPLY TO APPLICABLE SITUATIONS UNLESS NOTED OTHERWISE. IN GENERAL, TYPICAL DETAILS ARE NOT SPECIFICALLY REFERENCED.

5. WHERE MEMBER LOCATIONS ARE NOT SPECIFICALLY DIMENSIONED MEMBERS ARE LOCATED EITHER ON COLUMN LINES OR EQUALLY SPACED BETWEEN COLUMN LINES OR BETWEEN MEMBERS OTHERWISE LOCATED.

6. ELEVATOR SUPPORT FRAMING IS NOT SHOWN ON THE DRAWINGS. CONTRACTOR IS RESPONSIBLE FOR COORDINATING SUPPORT FRAMING WITH MANUFACTURER'S SPECIFIED REQUIREMENTS. THIS COORDINATION SHALL INCLUDE THE ENTIRE RUN OF THE ELEVATORS INCLUDING, BUT NOT LIMITED TO, PITS AND MACHINE ROOMS.
DESIGN CRITERIA

1. APPLICABLE CODE:
   A. INTERNATIONAL BUILDING CODE, (IBC), 2018

2. DESIGN LOADS:
   A. DEAD LOADS - ACTUAL IN PLACE WEIGHTS OF ALL MATERIALS SHOWN ON THE CONTRACT DOCUMENTS
   B. LIVE LOAD - UNIFORM AS FOLLOWS:
      (1) ROOF TYPICAL 1.197 psf
      (2) TYPICAL FLOOR 1.915 psf
   C. WIND LOAD - BASED ON ASCE 7-16 CHAPTER 27 WITH EXPOSURE C CONDITION AND BASIC WIND SPEED OF 92 MPH
   D. MINIMUM SEISMIC LOAD - DESIGN BASE SHEAR SHALL BE BASED ON IBC CHAPTER 12, EARTHQUAKE DESIGN AS
      FOLLOWS:

      $$ V = \text{TOTAL DESIGN BASE SHEAR} $$
      $$ V = [SDS*ie/R]*W \leq [SD1*ie/(R^2T)]*W \text{ for } T \leq TL $$
      $$ \leq [SD1*ie*TL/(R*T)]*W \text{ for } T > TL $$

      WHERE:
      $$ SDS = 1.05 $$
      $$ SD1 = 0.731 $$
      $$ T = 0.210 $$
      $$ W = \text{SEISMIC DEAD LOAD} $$
      $$ ie = 1 $$
      $$ R = 5 \text{ (CONCRETE SHEAR WALL)} $$

      E. SEISMIC LOAD FOR STRUCTURAL ELEMENTS, NONSTRUCTURAL COMPONENTS, AND EQUIPMENT SUPPORTED BY THE
      PRIMARY STRUCTURAL FRAME:

      $$ Fp = (0.2*SDS*ie)*W, \text{ BUT NO MORE THAN} $$
      $$ Fp = (0.4*SDS*ie)*W $$

      WHERE:
      $$ SDS = 1.05 $$
      $$ ie = 1 $$

      F. FOUNDATION DESIGN: BASED ON THE GEOTECHNICAL INVESTIGATION REPORT AS PREPARED BY STEPPING ON
      PEBBLES, INC., REPORT #1009-8436-C234 DATED JANUARY 2021.

      ALLOWABLE BEARING PressURES
      $$ DL + LL = 2000 \text{ psf} $$
MATERIAL CRITERIA

1. STRUCTURAL STEEL:
   A. SHAPES, BARS, PLATES:
      (1) ASTM A36: PLATES
      (2) ASTM A992 GRADE 50: W SHAPES
      B. ASTM A992 GRADE 65: WHERE DENOTED BY "**"
   C. HIGH STRENGTH BOLTS:
      (1) ASTM A325: TYPICAL
   D. MACHINE BOLTS: ASTM A307 TYPICAL
   E. ANCHOR BOLTS:
      (1) ASTM F1554: TYPICAL
   F. HEADED STUDS: ASTM A108
   G. WELDING ELECTRODES: E70XX

2. METAL DECK:
   A. METAL DECKING & ACCESSORIES

   B. WELDING ELECTRODES: E60XX

3. REINFORCED CONCRETE:
   A. CONCRETE FOR FOUNDATION, SLAB-ON-GRADE AND CURBS: NORMAL WEIGHT CONCRETE WITH MINIMUM COMPRESSIVE STRENGTH OF 3000 PSI AT 28 DAYS
   B. CONCRETE FOR WALLS, CONCRETE SHEAR WALLS SHOWN ON S. THROUGH S.
      NORMAL WEIGHT CONCRETE WITH MINIMUM COMRESSIVE STRENGTH OF 3000 PSI AT 28 DAYS.
   C. REINFORCEMENT:
      ASTM A615, GRADE 60, DEFORMED BARS, TYPICAL.
      ASTM A706 FOR REINFORCEMENT REQUIRING WELDING OR TRIM/Boundary REINFORCEMENT NOTED ON CONCRETE WALL ELEVATIONS (DWGS S. THROUGH S.).
TESTING & INSPECTION

1. TESTING AND INSPECTION SHALL INCLUDE, BUT IS NOT LIMITED TO THE FOLLOWING ITEMS:
   A. SHOP AND FIELD WELDING.
   B. BOLTS.
   C. STRUCTURAL STEEL.
   D. CONCRETE.
   E. CONCRETE REINFORCEMENT.
   F. EMBEDDED ANCHOR BOLTS.
   G. EXPANSION ANCHORS.

2. SEE PROJECT SPECIFICATIONS AND THE CALIFORNIA BUILDING CODE FOR ADDITIONAL INFORMATION / REQUIREMENTS.

<table>
<thead>
<tr>
<th>ABBREVIATIONS</th>
<th>MEANING</th>
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</thead>
<tbody>
<tr>
<td>ABT</td>
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<td>C.</td>
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<td>FACE OF STEEL OR STUDS</td>
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<tr>
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<td>INSIDE DIAMETER</td>
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<td>O.C.</td>
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<td>O.D</td>
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<td>OPPOSITE</td>
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<td>TOP OF WALL</td>
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<td>T.W</td>
<td>WEB THICKNESS</td>
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<td>U.N.O.</td>
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<td>W/</td>
<td>WITH</td>
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<td>WF</td>
<td>WIDE FLANGE</td>
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<tr>
<td>WP</td>
<td>WORK POINT</td>
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<tr>
<td>W.W.F</td>
<td>WELDED WIRE FABRIC</td>
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</tbody>
</table>
SHEET NOTES
1. TOP OF CONCRETE ELEVATION = +0m U.N.O.
2. TOP OF FOOTING TO BE (0.61m) BELOW TOP OF SLAB U.N.O.
3. SEE SHEET S.001 TO S.006 FOR GENERAL NOTES
4. SEE 21/S.414 FOR CONCRETE COLUMN SCHEDULE
5. SEE S.403 & S.404 FOR FOOTING DETAILS
6. SEE 1/S.401 FOR TYPICAL SLAB-ON-GRADE DETAILS
7. SEE S.415 & S.417 FOR WALL ELEVATIONS
1. TOP OF CONCRETE ELEVATION = +3m U.N.O.
2. SEE SHEET S.405 FOR TYPICAL SUSPENDED SLAB DETAILS
3. SEE S.406, S.408, & S.409 FOR SPAN JOIST DETAILS
4. SEE S.410, S.411, & S.412 FOR GIRDER DETAILS
5. SEE SHEET NOTES FOR ADDITIONAL INFORMATION
SHEET NOTES
1. TOP OF CONCRETE ELEVATION = VARIES. SEE ELEVATIONS FOR HEIGHTS
2. SEE S.407, S.408, S.409, S.411, & S.412 FOR JOIST AND GIRDER DETAILS
3. SEE SHEET NOTES FOR ADDITIONAL INFORMATION

ROOF FRAMING PLAN
1 : 200
SECTION 1

1 : 100
SECTION 2

1 : 100
SECTION 3
1: 50

TOP EDGE OF HIGHER ROOF 7300

FIRST FLOOR FRAMING PLAN 3000

TOP EDGE OF LOWER ROOF 6200

FOUNDATION PLAN 0

PROJECT: MBANDAZI VILLAGE
SITE: MBANDAZI VILLAGE

SCALE: 1:50
NUMBER: S.302

DRAWN BY: JS
CHECKED BY:
PLOT DATE: 5/31/2021 12:40:54 AM

ELEVATIONS
SEAL:

PROJECT:
MBANDAZI VILLAGE

SITE:
MBANDAZI VILLAGE

REVISIONS:

DRAWN BY: JH
CHECKED BY: 

PLOT DATE: 5/31/2021 12:40:56 AM

SHEET NAME: ELEVATIONS

SCALE: 1:50

NUMBER: S.303
SECTION 5
1 : 50
1. **TYPICAL SLAB ON GRADE EDGE**

1: 20

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<th>CLEAR COVERAGE mm</th>
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<td>38</td>
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<td>EXPOSED TO GROUND</td>
<td>51</td>
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<tr>
<td>COLUMN BARS</td>
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<tr>
<td>INTERIOR FACES</td>
<td>38</td>
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<tr>
<td>SLAB BARS</td>
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<td>CAST AGAINST GROUND</td>
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<td>GRADE BEAM BARS</td>
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<td>51</td>
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<td>SLAB ON GRADE BARS</td>
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<td>51</td>
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2. **CONCRETE COVER SCHEDULE**
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<th>BAR SIZE</th>
<th>HOOK DEVELOPMENT LENGTH (LH)</th>
<th>DEVELOPMENT LENGTH (LD)</th>
<th>LAP SPLICE LENGTH (LS)</th>
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<tr>
<td></td>
<td>f'c = 3000 psi</td>
<td>f'c = 3000 psi</td>
<td>f'c = 3000 psi</td>
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<tr>
<td>#10</td>
<td>186 mm</td>
<td>417 mm</td>
<td>542 mm</td>
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<tr>
<td>#13</td>
<td>286 mm</td>
<td>557 mm</td>
<td>724 mm</td>
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<tr>
<td>#16</td>
<td>375 mm</td>
<td>696 mm</td>
<td>905 mm</td>
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**NOTES**

1. USE THE EMBEDMENT AND LAP SPLICE LENGTHS SHOWN IN THE SCHEDULE MULTIPLIED BY ALL APPLICABLE FACTORS AS DEFINED IN NOTES BELOW, UNO ON DRAWINGS.

2. USE THIS TABLE UNLESS NOTED OTHERWISE ON DRAWINGS.

**REINFORCEMENT SCHEDULE**
FOOTING DETAILS

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<td>1.83m</td>
<td>1.83m</td>
<td>0.31m</td>
<td>(5) - #16</td>
<td>(5) - #16</td>
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<tr>
<td>F2</td>
<td>2.44m</td>
<td>2.44m</td>
<td>0.51m</td>
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<td>(12) - #16</td>
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<td>F3</td>
<td>2.90m</td>
<td>2.90m</td>
<td>0.77m</td>
<td>(20) - #16</td>
<td>(20) - #16</td>
<td>SEE DET.</td>
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SPREAD FOOTING SCHEDULE

FOOTING REINF. AT BOTTOM EA WAY

COLUMN REINF. - SEE SCHEDULE

CONCRETE COLUMN - SEE SCHEDULE

(8) - #13 EQUALLY SPACED AROUND COL.

SPREAD FOOTING SECTION

1: 30
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**FOOTING DETAILS**

**SPREAD FOOTING PLAN VIEW**

5

1 : 30

**PROJECT:** Mبان達茲威村

**SITE:** Mبان達茦威村

**SCALE:** 1 : 25

**NUMBER:** S.404
SLAB ELEVATION
1:20

#16 @ 12" O.C.

#13 @ 12" O.C.

LAP SPlice WHERE OCCURS
SEE SCHEDULE 3/S.402 FOR LENGTH

152mm MIN
356mm MAX

JS

DRAWN BY: JS
CHECKED BY:
PLOT DATE: 5/31/2021 12:41:03 AM
SHEET NAME: SLAB ELEVATION
SCALE: 1:20
NUMBER: S.405

JOYNERMAN INTERNATIONAL
3471 N. MAIN ST.
PRINEVILLE, OR

PROJECT: MBANDAZI VILLAGE
SITE: MBANDAZI VILLAGE
REVISIONS:
NO. DESC. DATE

5/31/2021 12:41:03 AM

5:31/2021 12:41:03 AM

SEAL:
## Concrete Beam Schedule

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<th>NAME</th>
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<td>406mm</td>
<td>610mm</td>
<td>(2) #13</td>
<td>#10 @ 254mm O.C.</td>
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<td>B1-1</td>
<td>406mm</td>
<td>610mm</td>
<td>(2) #13</td>
<td>#10 @ 254mm O.C.</td>
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<tr>
<td>B1-2</td>
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<td>610mm</td>
<td>(2) #13</td>
<td>#10 @ 254mm O.C.</td>
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<td>B2</td>
<td>457mm</td>
<td>762mm</td>
<td>(4) #16 x 3.0m (4) #16 x 5.64m</td>
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<td>762mm</td>
<td>(4) #16 x 1.0m (4) #16 x 1.0m</td>
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</tr>
<tr>
<td>B2-2</td>
<td>457mm</td>
<td>762mm</td>
<td>(4) #16 x 2.0m (4) #16 x 2.0m (4) #16 x 2.5m</td>
<td>#10 @ 305mm O.C.</td>
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<tr>
<td>B3</td>
<td>406mm</td>
<td>610mm</td>
<td>(1) #13 x WHOLE LENGTH</td>
<td>#10 @ 254mm O.C.</td>
</tr>
<tr>
<td>B4</td>
<td>406mm</td>
<td>610mm</td>
<td>(1) #13 x 1.5m (1) #13 x 3m (1) #13 x 8.69m</td>
<td>#10 @ 254mm O.C.</td>
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<tr>
<td>B4-1</td>
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<td>(1) #13 x 1.5m (1) #13 x 2.0m CANT SPlice HERE</td>
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* IF NO REBAR IS SHOWN IN THE SCHEDULE, THE ONLY NEEDED REBAR ARE THE CONT. BARS. FOLLOW MIN. DISTANCES FROM ELEVATION
** SPANS ARE NOT DRAWN TO SCALE
*** DISTANCES SHOWN BETWEEN BOTTOM REBAR AND COL. CENTER LINE IS DISTANCE FROM COL. FACE
**** DISTANCES SHOWN BETWEEN COL. CENTER LINE AND END OF TOP REBAR IS DISTANCE FROM COL. CENTER LINE

SEE S.410 FOR SECTIONS
CONCRETE ROOF BEAM SCHEDULE

<table>
<thead>
<tr>
<th>NAME</th>
<th>WIDTH</th>
<th>HEIGHT</th>
<th>CONT. BARS</th>
<th>STIRRUPS</th>
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<tbody>
<tr>
<td>CB1</td>
<td>406mm</td>
<td>610mm</td>
<td>(2)-#13</td>
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<tr>
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<td></td>
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<td>TOP: (2)</td>
<td>#10 @</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>254mm O.C.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>BOT: (2)</td>
<td>#10 @</td>
</tr>
<tr>
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<td></td>
<td>305mm O.C.</td>
</tr>
<tr>
<td>CB2</td>
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<td>762mm</td>
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</tr>
<tr>
<td></td>
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<td></td>
<td>TOP: (2)</td>
<td>#10 @</td>
</tr>
<tr>
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<td></td>
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<td></td>
<td>254mm O.C.</td>
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<td>#10 @</td>
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<td>TOP: (2)</td>
<td>#10 @</td>
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<td>254mm O.C.</td>
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<td>BOT: (2)</td>
<td>#10 @</td>
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<td>305mm O.C.</td>
</tr>
<tr>
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<td>254mm O.C.</td>
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<td>254mm O.C.</td>
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<tr>
<td></td>
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<td></td>
<td>305mm O.C.</td>
</tr>
</tbody>
</table>

* IF NO REBAR IS SHOWN IN THE SCHEDULE, THE ONLY NEEDED REBAR ARE THE CONT. BARS. FOLLOW MIN. DISTANCES FROM ELEVATION
** SPANS ARE NOT DRAWN TO SCALE
*** DISTANCES SHOWN BETWEEN BOTTOM REBAR AND COL. CENTER LINE IS DISTANCE FROM COL. FACE
**** DISTANCES SHOWN BETWEEN COL. CENTER LINE AND END OF TOP REBAR IS DISTANCE FROM COL. CENTER LINE

SEE S.410 FOR SECTIONS
SCHEDULE 8/S.407 ALSO APPLIES WHERE 7/S.406 DOES

CONT. BAR
LAP SPLICE WHERE OCCURS
SEE SCHEDULE 3/S.402 FOR LS

COLUMN FACE, TYP.

LAP SPLICE WHERE OCCURS
SEE SCHEDULE 3/S.402 FOR LENGTH

SPLICE & BAY
152mm MIN. 203mm MAX
TIE REINF. ALONG WHOLE BEAM - SEE SCHEDULE 7/S.406

152mm MIN. 203mm MAX, TYP.
SEE SCHEDULE 7/S.406
SEE SCHEDULE 7/S.406 FOR NUMBER, SIZE, LENGTH

BEAM ELEVATION
9
1:30

SEE SCHEDULE 7/S.406 FOR NUMBER, SIZE, LENGTH
SEE SCHEDULE 3/S.402 FOR LS
SEE SCHEDULE 3/S.402 FOR LENGTH

51 mm CLR

TIE REINF. ALONG WHOLE BEAM - SEE SCHEDULE 7/S.406

SEE SCHEDULE 7/S.406 FOR NUMBER, SIZE, LENGTH
SLAB REINF. - SEE 6/S.405

SEE SCHEDULE 7/S.406 FOR REBAR

CROSS TIE MATCH STIRRUP

SEE SCHEDULE 7/S.406 FOR BEAM SIZE

SEE SCHEDULE 7/S.406 FOR REBAR

JOIST SECTION FOR BOTTOM REBAR

JOIST SECTION FOR TOP REBAR

10 1 : 20

11 1 : 20

SCHEDULE 8/S.407 ALSO APPLIES WHERE 7/S.406 DOES

S.409
## CONCRETE GIRDER SCHEDULE

<table>
<thead>
<tr>
<th>NAME</th>
<th>WIDTH</th>
<th>HEIGHT</th>
<th>CONT. BARS</th>
<th>STIRRUPS</th>
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<td>G1</td>
<td>457mm</td>
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<td>(2)-#16 x 1.0m</td>
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* If no rebar is shown in the schedule, the only needed rebar are the cont. bars. Follow min. distances from elevation.

** Spans are not drawn to scale.

*** Distances shown between bottom rebar and col. center line is distance from col. face.

**** Distances shown between col. center line and end of top rebar is distance from col. center line.
LAP SPLICE WHERE OCCURS
SEE SCHEDULE 3/S.402 FOR LS

COLUMN FACE, TYP.

SEE SCHEDULE 12/S.410 FOR NUMBER, SIZE, LENGTH

CONT. BARS

152mm MIN, 203mm MAX, TYP.

TIE REINF. ALONG WHOLE BEAM - SEE SCHEDULE 12/S.410

152mm MIN, 203mm MAX

SEE SCHEDULE 12/S.410
FOR NUMBER, SIZE, LENGTH

SEE SCHEDULE 3/S.402 FOR LS

51mm CLR

SCHEDULE 8/S.407 ALSO APPLIES WHERE 12/S.410 DOES
SCHEDULE 8/S.407 ALSO APPLIES WHERE 12/S.410 DOES

TYP. GIRDER SECTION FOR TOP REBAR
1:30

TYP. GIRDER SECTION FOR BOTTOM REBAR
1:30

GIRDER SECTION FOR TOP REBAR G4
1:30

GIRDER SECTION FOR BOTTOM REBAR G4
1:30
TIES, SPACE @ 152mm O.C. MAX

FIRST FLOOR FRAMING PLAN
(+3.0m)

FOUNDATION FLOOR PLAN
(+0.0m)

TIES @ S1 SPACING, TYP.
SEE 21/S.414

TIES @ S2 SPACING TYP.
SEE 21/S.414

51mm CLR TYP.

CROSS TIE TO MATCH DIAMETER SIZE AND SPACING. ALTERNATE CONSECUTIVE TIES END FOR END, TYP.

EXTERIOR COLUMN ELEVATION

COLUMN ELEVATION

INTERIOR COLUMN ELEVATION

COLUMN DETAILS

S.413
22 TYPICAL WALL SECTION
1:30

23 WALL TO ROOF CONNECTION
1:20

24 WALL TO FLOOR CONNECTION
1:20

25 WALL TO FTG CONNECTION
1:20

SEE COL SCHEDULE FOR SIZE AND REBAR

SEE 8/S.407 FOR DIMENSIONS AND REINFORCEMENT

WALL REINF.

WALL REINF. - SEE 22/S.415

SHEAR KEY @ 305mm O.C.

CONC. Beam - SEE PLAN AND SCHEDULE FOR SIZE, REBAR

SLAB REINF

SLAB REINF.

SEE SCHEDULE

SEE ELEV.
343mm x 6.35mm THICK A36 STEEL PLATE

(3)-19mm DIA. A325-N BOLTS @ 76mm O.C.

SEE PLAN FOR BM SIZE

ROOF DECKING

SEE PLAN FOR BM SIZE

STEEL BEAM TO STEEL GIRDER

229mm x 6.35mm THICK A36 STEEL PLATE

(4)-19mm DIA. A325-N BOLTS @ 76mm O.C.

38mm MIN TYP.

1/4" TYP.
SEE PLAN FOR BM SIZE

ADD CONCRETE TO TOP OF BEAM IF CONNECTED TO STEEL. DO NOT TAKE AWAY CONCRETE.

(2) 22mm DIA. BOLTS, EACH SIDE

6.35mm THICK A36 PLATE - SEE ELEVATION FOR MIN SIZE

STEEL BEAM TO CONCRETE BEAM/COL SECTION

STEEL BEAM TO CONCRETE BEAM/COL ELEVATION

1/4"

76mm MIN

25mm MIN. TYP.

76 mm MIN

25mm MIN. TYP.

SEE PLAN FOR CONC. BM/COL SIZE

6.35mm THICK A36 PLATE - SEE ELEVATION FOR MIN SIZE

SEE PLAN FOR CONC. BM/COL SIZE

SEE PLAN FOR BM SIZE

(2) 22mm DIA. BOLTS, EACH SIDE

ADD CONCRETE TO TOP OF BEAM IF CONNECTED TO STEEL. DO NOT TAKE AWAY CONCRETE.

178mm MIN

STEEL BEAM TO CONCRETE BEAM/COL SECTION

1 : 10

1 : 10

1 : 10
STEEL GIRDER TO CONCRETE COL SECTION

1 : 10
EXTEND THE REBAR FROM THE GIRDER INTO SLAB

EXTENDED REBAR IN SLAB

SLAB REINF.

RUN THE CONT. BARS CALLED OUT ON S.411 ALONG GRIDLINE WHERE CALLED OUT

GIRDER - SEE S.411 FOR SIZES AND REINFORCEMENT

EXTENDED REBAR 1:10

35

S.422
305

13mm SPOT WELDS

SEE PLAN FOR BM SIZE

STEEL DECKING
0.81mm GAUGE

STEEL TO DECKING CONNECTION

1:10
100mm HANGOVER TYPICAL

13mm BOLTS @ 610mm O.C.

CONCRETE TO DECKING CONNECTION

1 : 10
PRESENTED BY: JOSHUA SHOCKEY
PROJECT ADVISOR: JAMES MWANGI
MATERIALS
CHALLENGES

CURVED DIAPHRAGM

ETABS

SEISMIC DESIGN VALUES

AVAILABLE MATERIALS

<table>
<thead>
<tr>
<th>U.S. &quot;Imperial&quot; Bar Size</th>
<th>Metric Size</th>
<th>Weight per foot (lb/ft)</th>
<th>Mass per meter (kg/m)</th>
<th>Nominal Diameter (inches)</th>
<th>Nominal Diameter (mm)</th>
<th>Nominal Area (in²)</th>
<th>Nominal Area (mm²)</th>
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Source: www.harrissupplysolutions.com
CONNECTIONS

SEE PLAN FOR GIRDER SIZE

(4)-19mm DIA. A325-N BOLTS @ 76mm O.C.

229mm x 6.35mm THICK A36 STEEL PLATE

SEE PLAN FOR BM SIZE

38mm MIN TYP. 383mm x 6.35mm THICK A36 STEEL PLATE

SEE PLAN FOR BM SIZE

(3)-19mm DIA. A325-N BOLTS @ 76mm O.C.

1/4°

ROOF DECKING
STEEL BEAM TO CONCRETE BEAM/COL SECTION

1:10
THANK YOU

JAMES MWANGI

JOURNEYMAN INTERNATIONAL

EVERYONE ELSE THAT HELPED ME