Senior Project:
Same Polytechnic College
Campus Main Core

For: ARCE 453 – Spring 2019
Instructor: Kevin Dong, SE

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Background Information

Mbesese Initiative

The Mbesese Initiative is “a multidisciplinary collaborative of industry professionals, students, academics and humanitarians pioneering a broader, integrated approach to end poverty.” Many people are trying to fix specific problems, like death, famine, instability, and corruption, but the Mbesese Initiative believes that they are all aspects of the broader problem of poverty. They are bringing in many different experts to understand the many aspects of poverty and how they can can work to reduce many of these problems.

The Mbesese Initiative’s four “Drivers of Change” are building human capital, encouraging economic growth, improving the built environment, and fostering environmental stewardship. Human capital is the assets of knowledge and skills that allow a population to produce economic value. This can be greatly improved by providing more and better education services to a larger portion of the population. To encourage economic growth, the Mbesese Initiative says “economic growth can alleviate poverty by increasing both employment opportunities and labor productivity.” Much of these opportunities in Tanzania come out of the agriculture sector since many people live in rural areas. These isn’t as much labor currently being used on this industry, so it has a lot of promising room to grow. The built environment needs vast improvement in Tanzania. Many of the homes are overcrowded and poses many safety risks. The Initiative is working to make houses that are adequate in size, with better construction materials, and that is replicable. Lastly, they also recognize the importance of the environment and how much the people of Tanzania cherish it. They want to figure out ways for the local people, especially those in poverty, can be stewards of the environment and allow it to be sustained while also being a resource for growth.

Same Polytechnic College

The Same Polytechnic College is planned to be a new vocational school that can provide many practical educational opportunities to over 1000 students and to support community service to the surrounding area. The plan is to have seven different schools: Agriculture, Food, and Environmental Sciences; Building Science and Construction Technology; Education; Tourism and Hospitality; Automotive and Mechanical Technology; Social Studies; and Business Management. These subjects of study are very relevant to the current needs of Tanzania.

Current Design

The current design, which was developed by Amir Mahmoodi, is to include a campus main core, market, colleges, farms, and housing. The campus main core is planned to have administration buildings, a library, a cafeteria, and an auditorium. The housing will be both for students and for faculty and staff. It should be running on mostly solar energy and not having many lights, so it only operates when the sun is out to save on electrical needs.

For full architectural plans, see Appendix A. The Revit model and renderings were done by the Cal Poly Architecture Masters student, Amir Mahmoodi.

Project

Overview

My senior project will be to continue moving forward the campus core of the Same Polytechnic College. We started with the main campus core because it can act as a small school where some classes can be taught while the rest of the construction will be completed. With the land boundaries almost finalized and a solid design for the campus, we can now start a structural design and cost estimation. These will be the main steps to begin construction of the campus.

Structural Design

There are three main structural systems to design for this project: the steel frame shade structure, block walls supporting a floor, and a wood frame roof structure. For all structural calculations, see Appendix B.

Material Assumptions

Since the materials are not as good of quality as the structural materials that are standard in the U.S., there were conservative assumptions made about the material properties:

- Steel Framing Members Yield Strength: F_y = 36 ksi
- Block Wall Strength: f'_m = 500 psi
- Concrete Strength: f'_c = 2000 psi
- Reinforcing Steel Strength: f_y = 40 ksi
- Allowable Soil Pressure: 2,000 psf

Steel Roof Structure 1

Steel Roof Structure 2 is to have a steel deck roof supported by steel beams and columns. The beams will be simply supported by girders and columns, and then the girders will also be simply supported by the columns. The columns will be welded to a base plate, which is connected to a spread footing for gravity and grade beams for potential moment induced from lateral forces.

Steel Roof Structure 2

Steel Roof Structure 2 is made with wood joists for shade which are supported by steel beams and columns. It makes up the framing for the center corridor through the Campus Core. The steel beams are simply supported by the girders and the columns, but the girders must be a full weld since they must cantilever out past each column. Similar to the first roof structure, the columns will be welded to a base plate, which is connected to a spread footing for gravity and grade beams for potential moment induced from lateral forces.

Block Walls

The Block Walls will be made of blocks approximately the size of 6”x9”x18”. The Block Walls will be supporting a concrete filled metal deck floor as a second floor. The metal deck will be supported by steel beams which will transfer the load into the Block Walls. The walls will also be resisting out of plane lateral loads, which controlled much of the design. The walls will be on a continuous footing to transfer the forces into the ground.

Final Drawings

The main goal of the final structural drawings is to be easy to communicate with builders in Tanzania. They don’t have lots of text and rely on repetition of structural elements. The plans should allow us to see if this design is possible to build. The finals plans can be found in Appendix D.

These simple plans will be used over the summer when a group of Cal Poly students, including myself, will be going to Tanzania to work with the Mbesese Initiative. While there, we will be able to directly communicate with the local builders to see if these plans are feasible, but also to see if our estimates about cost are close to what it would actually be.

Cost Evaluation

Probably the most vital information from my project is a cost evaluation of the current design to see if it is within the means of the College. All of the data on the price of materials and labor were from the *2019 Building Construction Cost with RSMeans Data*.

I used that data along with the final plans to put together a spreadsheet that would calculate the labor cost, material cost, and total cost of the entire Campus Main Core. The spreadsheet can be found in Appendix C.

Labor Totals

- Total Labor Hours = 21,670 Hours
- Total Labor Cost = $1,142,000 = 2,626,494,000 Shillings

Material Amounts and Costs

- Block Walls Amount = 35,840.00 ft^2
- Block Walls Cost = $92,108.80 = 211,850,240.00 Shillings
- Concrete Amount = 2,603.40 Cubic Yards
- Concrete Cost = $19,942.07 = 45,866,769.06 Shillings
- Steel Decking Amount = 178,560.00 ft^2
- Steel Decking Cost = $283,910.40 = 652,993,920.00 Shillings
- Steel Framing Amount = 490,939.20 lb. = 245.5 Tons
- Steel Framing Cost = $490,939.20 = 1,129,160,160.00 Shillings
- Steel Reinforcement Amount = 623.11 Tons
- Steel Reinforcement Cost = $638,690.37 = 1,468,987,860.20 Shillings

Total Cost

These are the total costs calculated from the spreadsheet, but they are not true final costs.

- Total Cost without Overhead and Profit = $3,289,000 = 7,562,536,000 Shillings
- Total Cost with Overhead and Profit = $4,086,000 = 9,397,722,000 Shillings

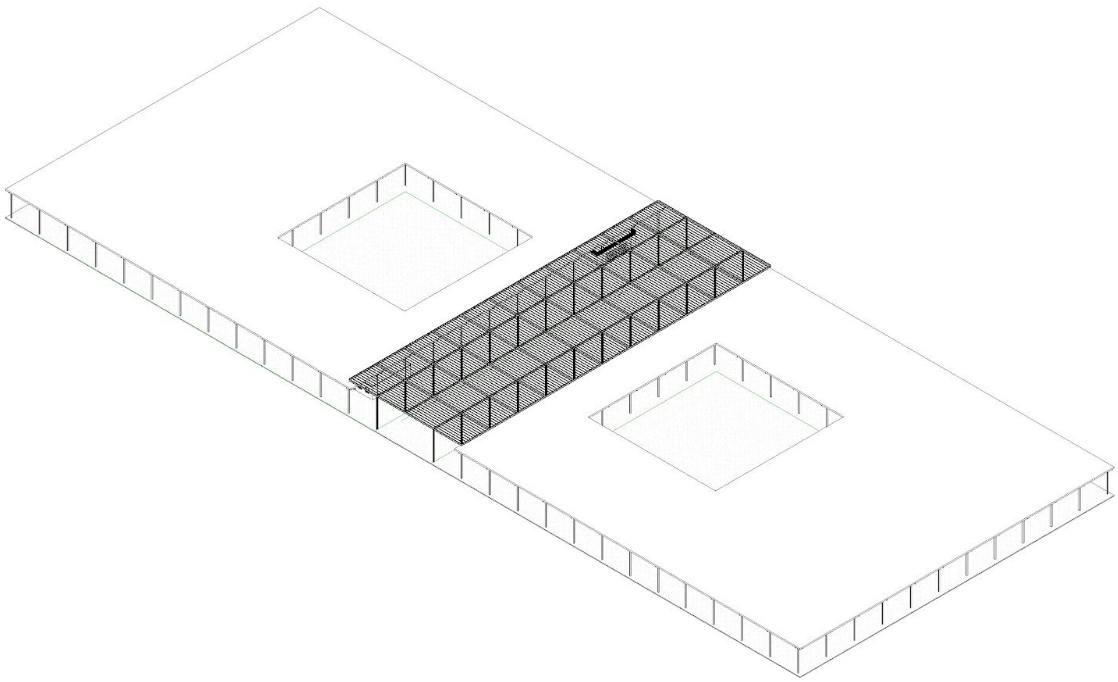
Takeaways

The cost of the Campus Core may seem high, but we haven't yet taken into account the cost of labor and materials in Tanzania as opposed to the United States. Though we have converted the values from U.S. Dollars to Tanzanian Shillings, they are not accurate due to differences in the cost of labor and the quality of materials. In the United States, we are lucky to have high quality building materials, but the building materials in Tanzania have much less quality control and are not as strong.

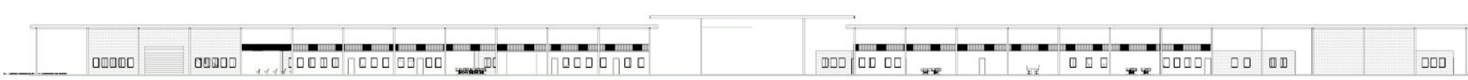
The labor in Tanzania is also very cheap in comparison to the United States. According to Kevin Dong, a full day of labor for one person in Tanzania is about 30,000 Shillings and the laborers are working about 10 hours a day. That equates to approximately $1.50 per hour. This is much lower than the current national minimum wage rate of $7.25 per hour. The current total labor cost of about $1,142,000 could be reduced to about $240,000. These two discrepancies in cost estimation must be taken into account when considering the total cost of this structure. It could be as much as a quarter of the current cost that is estimated in this report, which would take the total cost from about $4,000,000 to about $1,000,000.

Appendix A: Architectural Design

3D View of Campus Core:



Long Elevation of Campus Core:



Short Elevation of Campus Core:

Renderings:

Appendix B: Structural Design Calculations

MATERIAL ASSUMPTIONS:

STEEL FRAMING MEMBERS:

- $F_y = 36$ ksi

BLOCK WALL:

- BLOCK SIZE: 6" x 9" x 18"
- $f'_m = 500$ psi

CONCRETE:

- $f'_c = 2000$ psi

REINFORCING STEEL:

- $f_y = 40$ ksi

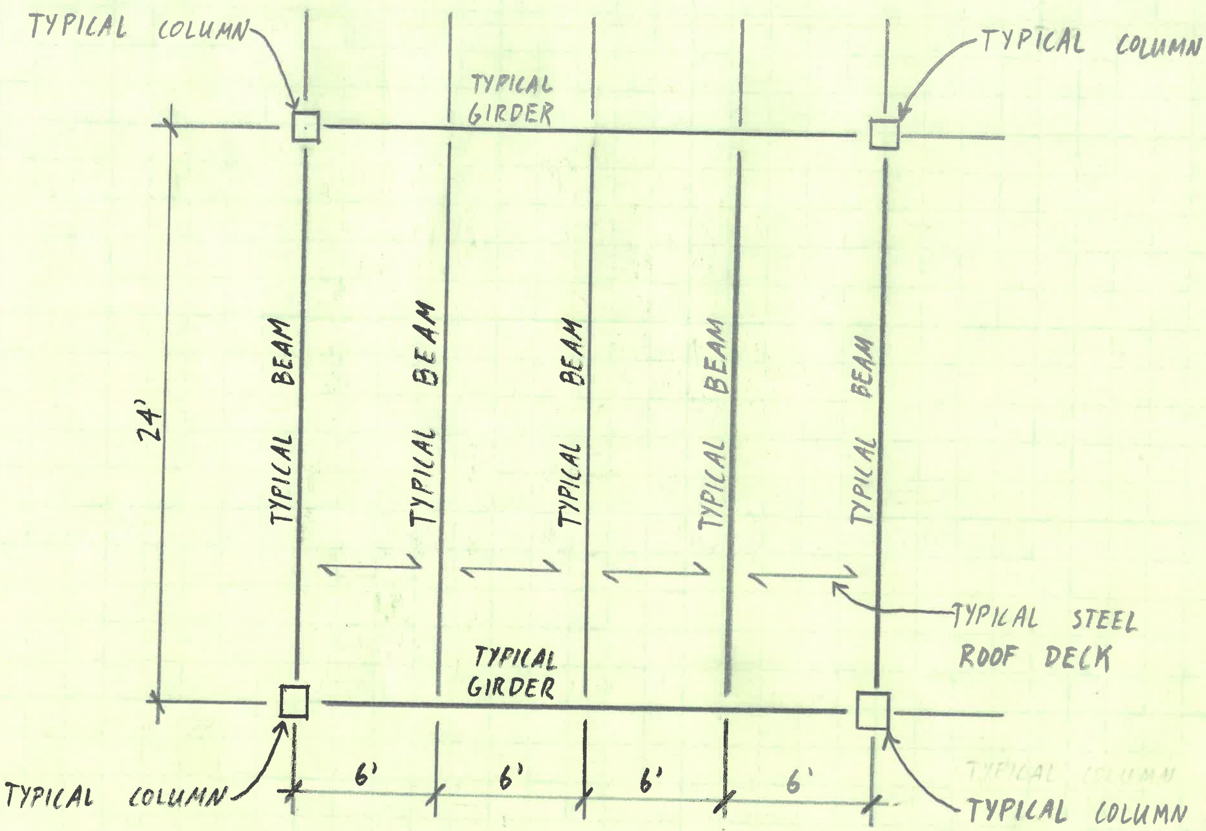
SOIL:

- ALLOWABLE SOIL PRESSURE = 2,000 PSF

ROOF STRUCTURE 1 CALCULATIONS

ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL BAY:



TYPICAL STEEL ROOF DECK:

- $L_r = 20$ PSF, $D = 0$ PSF ⟶ TOTAL LOAD $= D + L_r = 20$ PSF
- SPAN = 6'
- TRY 26 GAUGE DEEP VERCOR: STEEL ROOF DECK CATALOG VR4

  ALLOWABLE UNIFORM LOAD = 66 PSF > 20 PSF ✓

⟶ TYPICAL STEEL ROOF DECK:

26 GAUGE DEEP VERCOR (DEPTH = 1-5/16") OR SIMILAR

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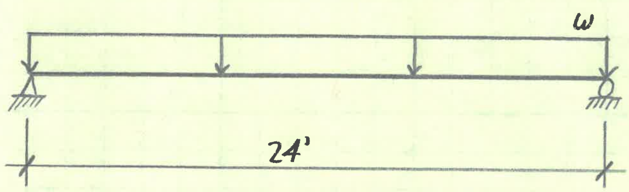
ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL BEAM:

- $D = 2.1$ PSF (CONSERVATIVE ESTIMATE OF 20 GAUGE DEEP VERCOR)
  STEEL ROOF DECK CATALOG VR4
- $L_r = 20$ PSF



- 6' SPACING
- $\omega = (2.1 \text{ PSF} + 20 \text{ PSF})(6') = 132.6 \text{ PLF} = 0.133 \text{ klf}$
- TOTAL LOAD $= (0.133 \text{ klf})(24') = 3.2$ k
- $L_b = 24'$
- TRY W12x14: AISC MANUEL OF STEEL CONSTRUCTION Pg. 2-68
  9TH EDITION

  ALLOWABLE UNIFORM LOADS = 10 k > 3.2 k ✓

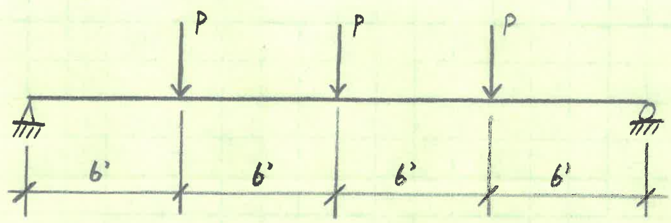
→ TYPICAL BEAM:

W12 x 14 BEAM OR SIMILAR
WITH $L = 24'$

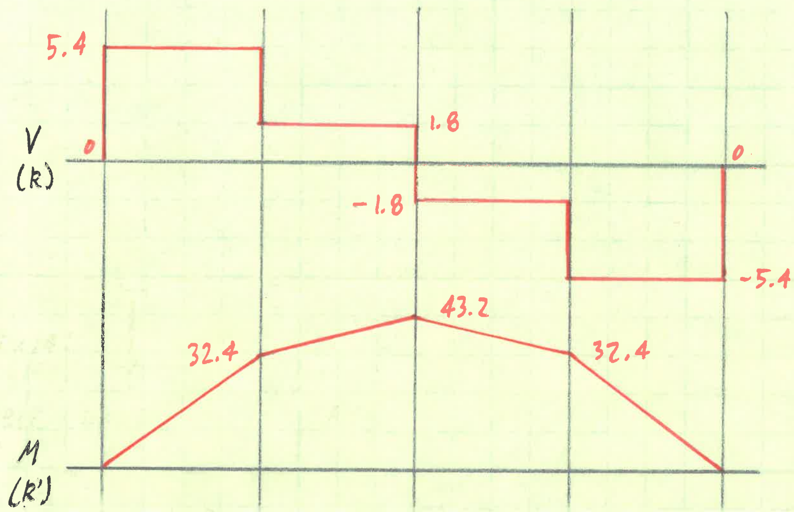
ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL GIRDER:

- LOADING:



- $P = 3.2k + (0.014\ klf)(24') \approx 3.6\ k$



- $L_b = 24'$
- $M_{MAX} = 43.2\ k'$
- TRY W12x40: AISC MANUEL OF STEEL CONSTRUCTION 9TH EDITION Pg. 2-173

ALLOWABLE MOMENT = 62.5 k' > 43.2 k' ✓

→ TYPICAL GIRDER:

W12x40 GIRDER OR SIMILAR WITH L = 24'

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ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL COLUMN:

- AXIAL LOAD:
  - $P_{GIRDER} = 3 \cdot 3.6^{k} + 24'(0.04\ k/f) \approx 11.8\ k$
  - $P_{BEAM} = 3.6\ k$
  - $P_{TOTAL} = P_{GIRDER} + P_{BEAM} = 11.8^{k} + 3.6^{k} = 15.4^{k}$
- LATERAL LOAD ASCE 7-16 12.8
  - $S_{DS} = 0.862$, S_{D1} : NULL
  - $T \approx T_a = C_t h_n^{x}$: $C_t = 0.028$

    $x = 0.8$

    $h_n = 20'$

    $\longrightarrow T \approx T_a = (0.028)(20')^{0.8} = 0.308\ s < T_L = 8\ s$
  - $C_s = \dfrac{S_{DS}}{(R/I_e)}$: $R = 3.5$ (STEEL ORDINARY MOMENT FRAME)

    $I_e = 1.0$

    $\longrightarrow C_s = \dfrac{0.862}{(3.5/1.0)} = 0.246$
  - $W = P_{TOTAL} = 15.4\ k$
  - $V = C_s W = (0.246)(15.4\ k) \approx 3.8^{k}$
  - $M = V \cdot h = (3.8^{k}) \cdot (20') = 76\ k'$

ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL COLUMN:

- TRY HSS 8x8x 5/16
- CHECK AXIAL ONLY: AISC STEEL CONSTRUCTION MANUEL 9TH EDITION
  - $K = 1$
  - $L = 20'$
  - $r = 3.12''$
  - $KL/r = (1)(20' \cdot 12''/1')/(3.12'') = 76.923 \longrightarrow KL/r = 77$

    $\longrightarrow F_a = 15.69$ ksi TABLE C-36
  - $P_{ALLOW} = A \cdot F_a$: $A = 9.36\ in^2$, $Z_x = 26.7\ in^3$

    $\longrightarrow P_{ALLOW} = (9.36\ in^2)(15.69\ ksi) = 146.9\ k > P = 15.4\ k$ ✓
- CHECK COMBINED AXIAL AND BENDING MOMENT: N4
  - $$\frac{P}{P_{cr}} + \frac{C_m M}{(1 - P/P_e)M_m} = \frac{15.4\ k}{249.7\ k} + \frac{0.85\,(76\ k')}{(1 - 15.4\ k/452.75\ k)(74.0\ k')} = 0.965 < 1.0 ✓$$
    - $P_{cr} = 1.7\, F_a A$: $C_c = \sqrt{\dfrac{2\pi^2 E}{F_y}} = \sqrt{\dfrac{2\pi^2 (29{,}000\ ksi)}{36\ ksi}} = 126.1 > \dfrac{KL}{r} = 76.923$

      $$\longrightarrow F_a = \frac{[1 - (KL/r)^2/(2C_c^2)]\, F_y}{(5/3) + 3(KL/r)/(8C_c) - (KL/r)^3/(8C_c^3)}$$

      $$= \frac{[1 - (76.923)^2/(2\cdot(126.1)^2)]\,(36\ ksi)}{(5/3) + 3(76.923)/(8\cdot 126.1) - (76.923)^3/(8\cdot(126.1)^3)}$$

      $= 15.69$ ksi

      $\longrightarrow P_{cr} = 1.7(15.69\ ksi)(9.36\ in^2) = 249.7\ k$
  - $C_m = 0.85$
  - $P_e = (23/12)F_e' A$: $F_e' = \dfrac{12\pi^2 E}{23(KL/r)^2} = \dfrac{12\pi^2(29{,}000\ ksi)}{23(76.923)^2} = 25.237\ ksi$

    $\longrightarrow P_e = (23/12)(25.237\ ksi)(9.36\ in^2) = 452.75\ k$
  - $M_m = [1.07 - (L/r)\sqrt{F_y}/3160]\, M_{px}$: $M_{px} = F_y Z_x = (36\ ksi)(26.7\ in^3)/12 = 80.1\ k'$

    $= [1.07 - (20' \cdot 12''/1'/3.12'')\sqrt{36}/3160](80.1\ k') = 74.0\ k' < 80.1\ k'$ ✓

ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL COLUMN:

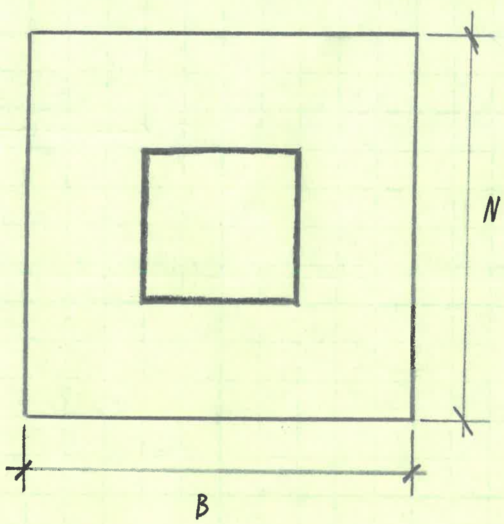
- CHECK COMBINED AXIAL AND BENDING MOMENT:
  - $\frac{P}{P_y} + \frac{M}{1.18M_p} = \frac{15.4\ k}{336.96\ k} + \frac{76\ k'}{1.18(80.1\ k')} = 0.850 < 1.0$ ✓
    - $P_y = F_y A = (36\ ksi)(9.36\ in^2) = 336.96\ k$
    - $M_p = M_{px} = 80.1\ k'$
  - $M = 76\ k' < M_p = 80.1\ k'$ ✓

$\boxed{\longrightarrow \text{TYPICAL COLUMN: HSS } 8 \times 8 \times 5/16 \text{ OR SIMILAR WITH } L = 20'}$

ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL COLUMN BASE PLATE:

- COLUMN: HSS $8 \times 8 \times \frac{5}{16}$
- $P = 15.4^k$
- PLATE STRENGTH: $F_y = 36$ ksi
- FOOTING SIZE: $3' \times 3'$, $f'_c = 2$ ksi
- TRY $N = B = 20"$



- CHECK BEARING:
  - $P_p = 0.8 f'_c A_1 \sqrt{A_2/A_1} \leq 1.7 f'_c A_1$
    - $A_1 = 20" \cdot 20" = 400 \ in^2$
    - $A_2 = (3' \cdot 3')(12"/1')^2 = 1296 \ in^2$

  $\longrightarrow P_p = 0.8(2 \text{ ksi})(400 \ in^2)\sqrt{\frac{1296 in^2}{400 in^2}} = 1152^k \leq 1.7 f'_c A_1 = 1.7(2 \text{ ksi})(400 in^2)$
  $= 1360 k$

  - ALLOWABLE BEARING $= 0.65(1152 k) = 748.8^k >> P = 15.4 k$

ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL COLUMN BASE PLATE:

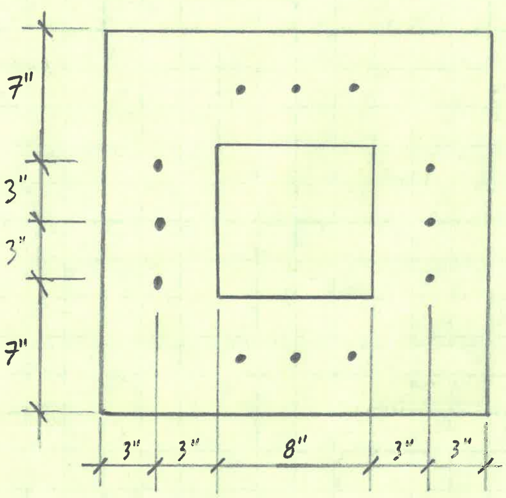
- $m = \frac{N - 0.95d}{2} = \frac{20'' - 0.95(8'')}{2} = 6.2''$
- $n = m = 6.2''$
- $n' = \frac{\sqrt{(8'')(8'')}}{4} = 2$
- $\lambda = 1$ (CONSERVATIVE)

$\longrightarrow l = 6.2''$

- $t_{MIN} = l\sqrt{\frac{2P}{0.9F_y BN}} = 6.2''\sqrt{\frac{2(15.4^k)}{0.9(36\text{ ksi})(20'')(20'')}} = 0.302''$

$\longrightarrow$ USE $5/16'' = 0.3125''$ PLATE

- BOLTS:
  - $M = 76\ k'$
  - LAYOUT:



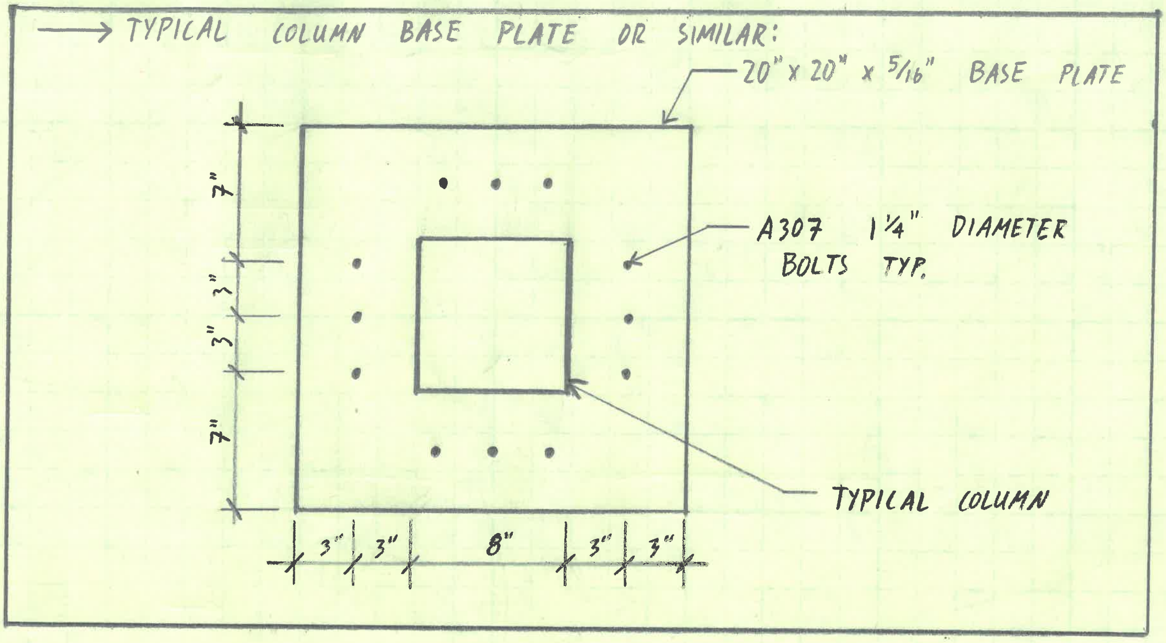
- SAME IN BOTH DIRECTIONS

- $d = 3'' + 8'' + 3'' = 14'' \longrightarrow T = M/d = (76\ k')/(14'' \cdot 1'/12'') = 65.14^k$
- $T_{EACH\ BOLT} = 65.14^k/3 = 21.72^k$
- ASSUMING A307 BOLTS:

$\longrightarrow$ USE $1\frac{1}{4}''$ BOLTS, $T_{ALLOW} = 24.5^k > T = 21.72^k$

ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL COLUMN BASE PLATE:



ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL ISOLATED FOOTING:

- $q_{ALLOW} = 2$ ksf
- $P = 15.4$ k
- $P_L = (20 \text{ PSF})(24')(24') = 11520\ \# = 11.52$ k $\longrightarrow P_D = P - P_L = 15.4^k - 11.52^k = 3.88^k$
- $P_u = 1.2P_D + 1.6P_L = 1.2(3.88\text{ k}) + 1.6(11.52\text{ k}) \approx 23.1^k$
- $b_{BASE\ PLATE} = 20'' = 1.67' = b_{COL}$
- SOIL BEARING:
  - $A_{REQ} = P/q_{ALLOW} = (15.4^k)/(2\text{ ksf}) = 7.7\text{ FT}^2$
  - $b = \sqrt{A_{REQ}} = \sqrt{7.7\text{ FT}^2} = 2.775' \longrightarrow$ USE $b = 3' = 36''$

  $\longrightarrow$ SIZE IN PLAN: $3' \times 3'$
- PUNCHING/TWO-WAY SHEAR: ACI 318-14
  - $q_u = P_u/A = (23.1\text{ k})/(3')^2 = 2.57$ ksf
  - TRY $h = 12''$ WITH #8 BARS $\longrightarrow d = h - \text{COVER} - d_b - \frac{1}{2}d_b$

    $= 12'' - 3'' - 1'' - \frac{1}{2}(1'') = 7.5'' = 0.625'$
  - $V_u = q_u \cdot [b^2 - (b_{col} + d)^2] = (2.57\text{ ksf})[(3')^2 - (1.67' + 0.625')^2]$

    $= 9.633$ k
  - $V_c = c(\lambda \sqrt{f'_c}\, b_o d)$: c = SMALLEST $\begin{cases} 4 \text{ — CONTROLLS} \\ 2 + 4/\beta = 2 + 4/1 = 6 \\ 2 + \alpha_s d/b_o = 2 + (40)(7.5'')/110'' = 4.73 \end{cases}$
    - $\beta = 3'/3' = 1$
    - $\alpha_s = 40$ (INTERIOR COLUMN)
    - $b_o = 4(b_{col} + d) = 4(1.67' + 0.625')$ $\longrightarrow c = 4$

      $= 9.17' = 110''$

  $\longrightarrow V_c = 4(1)\sqrt{2{,}000\text{ psi}}\,(110'')(7.5'') = 147580\ \# = 147.58^k$
- $\phi V_n = \phi(V_c + V_s) = 0.75(147.58^k + 0^k) = 110.685^k > V_u = 9.633$ k ✓

ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL ISOLATED FOOTING:

- BEAM/ONE-WAY SHEAR:
  - $V_u = q_u \cdot [(b - b_{col})/2 - d] \cdot b$

    $= (2.57 \text{ ksf}) \cdot [(36" - 20")/2 - 7.5"] \cdot (36") \cdot (1'/12")^2 = 0.3213 \text{ k}$

    $= 321.3 \#$
  - $V_c = 2\lambda\sqrt{f'_c}\, bd = 2(1)\sqrt{2{,}000 \text{ psi}}\ (36")(7.5") = 24149 \#$
  - $\phi V_n = \phi(V_c + V_s) = (0.75)(24149\# + 0\#) = 18111.75\# > V_u = 321.3\#$
- BENDING:
  - $M_u = q_u \cdot b \cdot \frac{(b - b_{col})}{2} \cdot \frac{(b - b_{col})}{2} / 2$

    $= (2.57 \text{ ksf}) \cdot (3') \cdot \frac{(3' - 1.67')}{2} \cdot \frac{(3' - 1.67')}{2} / 2 = 1.705 \text{ k}'$
  - $A_{s,min} = \text{GREATER} \begin{cases} \frac{0.0018 \times 60{,}000}{f_y} bh = \frac{0.0018 \times 60{,}000}{40{,}000} bh = \underset{\text{CONTROLS}}{\underline{0.0027\, bh}} \\ 0.0014bh \end{cases}$

    $\longrightarrow A_{s,min} = 0.0027\, bh = 0.0027(36")(12") = 1.17 \text{ in}^2$
  - $S_{MAX} = \text{LESSER} \begin{cases} \underset{\text{CONTROLS}}{\underline{18"}} \\ 2h = 2(12") = 24" \end{cases} \longrightarrow S_{MAX} = 18"$
  - DETERMINE A_s: ASSUMING TENSION CONTROLLED

    $a = d - \sqrt{\frac{-2M_u}{\phi(0.85 f'_c b)} + d^2}$

    $= 7.5" - \sqrt{\frac{-2(1.705 \text{ k}')(12"/1')}{(0.9)(0.85)(2 \text{ ksi})(36")} + (7.5")^2} = 0.050"$

    $A_s = \frac{0.85 f'_c a b}{f_y} = \frac{0.85(2 \text{ ksi})(0.050")(36")}{40 \text{ ksi}} = 0.077" < 1.17 \text{ in}^2$

    $\longrightarrow$ TRY (3) #6 BARS $A_s = 3(0.440 \text{ in}^2) = 1.32 \text{ in}^2 > 1.17 \text{ in}^2 \checkmark$

    $s = [36" - 2(3") - 3(6/8")]/2 = 13.875" < S_{MAX} = 18" \checkmark$

    * BARS MUST BE HOOKED FOR DEVELOPMENT LENGTH

ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL ISOLATED FOOTING:

- BENDING:

  - ϕM_n: $a = \dfrac{A_s f_y}{0.85 f'_c b} = \dfrac{(1.32 \text{ in}^2)(40 \text{ ksi})}{0.85(2 \text{ ksi})(36")} = 0.863"$

    $d = 12" - 3" - (6/8") - \frac{1}{2}(6/8") = 7.875"$

    $\phi M_n = \phi A_s f_y (d - a/2)$

    $= 0.9(1.32 \text{ in}^2)(40 \text{ ksi})(7.875" - 0.863"/2) = 353.72 \text{ k"}$

    $= 29.48 \text{ k'} > M_u = 1.705 \text{ k'}$ ✓

  - CHECK IF TENSION CONTROLLED:

    $f'_c = 2{,}000 \text{ psi} \longrightarrow \beta_1 = 0.85$ (CONSERVATIVE)

    $c = \dfrac{a}{\beta_1} = \dfrac{0.863"}{0.85} = 1.015"$

    $\epsilon_s = \dfrac{\epsilon_c (d - c)}{c} = \dfrac{(0.003)(7.875" - 1.015")}{1.015"} = 0.0203 > 0.005$ ✓

    $\longrightarrow$ FOOTING IS TENSION CONTROLLED

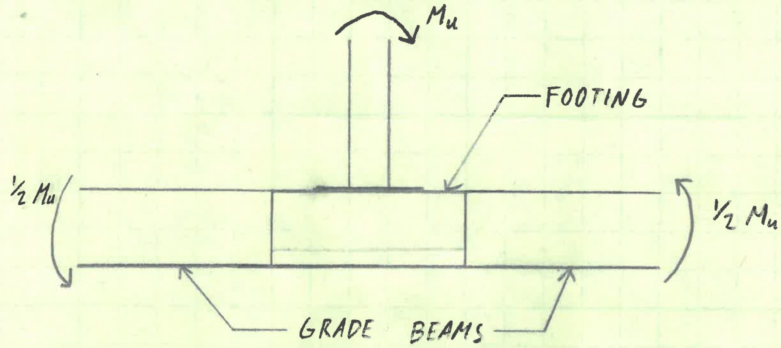
* SINCE d IS NOW LARGER THAN ORIGINALLY ASSUMED, WE KNOW THAT OUR CAPACITIES WILL ONLY INCREASE, SO NO NEED TO REDO THEM.

$\longrightarrow$ TYPICAL ISOLATED FOOTING:

3' x 3' WITH $h = 12"$ WITH (3) #6 BARS EACH WAY OR SIMILAR

ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL GRADE BEAM: ACI 318-14



- $M_u = V \cdot L = 76$ k'
- M_u (FOR EACH BEAM) $= \frac{1}{2}(76$ k'$) = 38$ k'
- ASSUME: $b = 24"$, $h = 12"$
- MINIMUM PRIMARY REINFORCING:
    - GREATER OF $\begin{cases} \dfrac{3\sqrt{f'_c}}{f_y} bd = \dfrac{3\sqrt{2{,}000 \text{ psi}}}{40{,}000 \text{ psi}} bd = 0.0034\, bd \\[2ex] \dfrac{200}{f_y} bd = \dfrac{200}{40{,}000 \text{ psi}} bd = \underset{\text{CONTROLS}}{\underline{0.005\, bd}} \end{cases}$

$\longrightarrow A_{s,MIN} = 0.005\, bd = 0.005(24")(12") = 1.44 \text{ in}^2$

- PRIMARY REINFORCING SPACING REQUIREMENTS:
    - S_{MAX} = SMALLEST $\begin{cases} 3h = 3(12") = 36" \\ 18" \\ 15\left(\dfrac{40{,}000}{\frac{2}{3} f_y}\right) - 2.5 C_c = 15\left(\dfrac{40{,}000}{\frac{2}{3}(40{,}000)}\right) - 2.5(3") = \underset{\text{CONTROLS}}{\underline{15"}} \\ 12\left(\dfrac{40{,}000}{\frac{2}{3} f_y}\right) = 12\left(\dfrac{40{,}000}{\frac{2}{3}(40{,}000)}\right) = 18" \end{cases}$
    - S_{MIN} = GREATEST $\begin{cases} \underset{\text{CONTROLS}}{\underline{1"}} \\ d_b = 1" \quad \text{(\#8 BAR TO BE CONSERVATIVE)} \\ (4/3)\, d_{agg} = (4/3)(3/4") = 1" \end{cases}$
- $S_{MAX} = 15"$, $S_{MIN} = 1"$

ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL GRADE BEAM:

- DETERMINE A_s: ASSUMING TENSION CONTROLLED

$d = 12" - 3" - 3/8" - \frac{1}{2}(1") = 8.125"$ (ASSUMING #3 TIES + #8 LONG. BARS)

$$a = d - \sqrt{\frac{-2M_u}{\phi(0.85 f'_c b)} + d^2}$$

$$= 8.125" - \sqrt{\frac{-2(38.0\ k')(12"/1')}{(0.9)(0.85)(2\ ksi)(24")} + (8.125")^2} = 1.708"$$

$$A_s = \frac{0.85 f'_c a b}{f_y} = \frac{0.85(2\ ksi)(1.708")(24")}{40\ ksi} = 1.742\ in^2 > 1.44\ in^2$$

⟶ TRY (3) #7 BARS $A_s = 3(0.60\ in^2) = 1.80\ in^2 > 1.742\ in^2$ ✓

SPACING $= [24" - 2(3") - 3(1")]/2 = 7.5"$ $> S_{MIN} = 1"$ ✓
$< S_{MAX} = 15"$ ✓

- ϕM_n: $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(1.80\ in^2)(40\ ksi)}{0.85(2\ ksi)(24")} = 1.765"$

$d = 8.125"$

$\phi M_n = \phi A_s f_y (d - a/2)$

$= (0.9)(1.80\ in^2)(40\ ksi)(8.125" - 1.765"/2) = 469.314\ k"$

$= 39.1\ k' > M_u = 38\ k'$ ✓

- CHECK IF TENSION CONTROLLED:

$c = \frac{a}{\beta_1} = \frac{1.765"}{0.85} = 2.077"$

$\varepsilon_s = \frac{\varepsilon_c (d-c)}{c} = \frac{(0.003)(8.125" - 2.077")}{2.077"} = 0.0087 > 0.005$ ✓

⟶ GRADE BEAM IS TENSION CONTROLLED

ROOF STRUCTURE 1 CALCULATIONS:

TYPICAL GRADE BEAM:

- SHEAR DESIGN:
  - SPACING: s_{MAX} = LESSER OF $\begin{cases} d/2 = (8.125")/2 = 4.063" \rightarrow \underline{4"}_{\text{CONTROLS}} \\ 24" \end{cases}$
  - MINIMUM SHEAR REINFORCING:

$$A_{v,MIN}/s = \text{GREATER OF} \begin{cases} 0.75\sqrt{f'_c}\,\dfrac{b}{f_{yt}} = 0.75\sqrt{2{,}000\,psi}\,\dfrac{(24")}{40{,}000\,psi} = 0.02\ in^2/in \\[2ex] 50\,\dfrac{b}{f_{yt}} = 50\,\dfrac{24"}{40{,}000\,psi} = \underline{0.03\ in^2/in}_{\text{CONTROLS}} \end{cases}$$

ASSUME #3 TIES: $A_v = 2(0.11\,in^2) = 0.22\,in^2$

$$s = A_v/(A_{v,min}/s) = (0.22\,in^2)/(0.03\,in^2/in) = 7.33" > s_{MAX} = 4"$$

$\longrightarrow$ MINIMUM SHEAR REINFORCEMEMT: #3 ⊔ @ 4" O.C.

- $\phi V_{n,MIN} = \phi(V_c + V_{s,MIN})$
  - $V_c = 2\lambda\sqrt{f'_c}\,bd = 2(1)\sqrt{2{,}000\,psi}\,(24")(8.125") = 17441\,\# = 17.4\,k$
  - $V_{s,MIN} = \dfrac{A_v f_{yt} d}{s} = \dfrac{(0.22\,in^2)(40\,ksi)(8.125")}{4"} = 17.9\,k$
  - MAXIMUM ALLOWABLE SHEAR CAPACITY $= \phi 10\sqrt{f'_c}\,bd$

$$= (0.75)(10)\sqrt{2{,}000\,psi}\,(24")(14.125")$$

$$= 113704\,\# = 113.7\,k$$

  - $\phi V_{n,MIN} = 0.75(17.4\,k + 17.9\,k) = 26.5\,k < \phi V_{n,MAX} = 113.7\,k$ ✓
- $\phi V_{n,MIN} = 26.5\,k > V_u = M_u/L_{BEAM} = 56.9\,k'/24' = 2.4\,k$ ✓

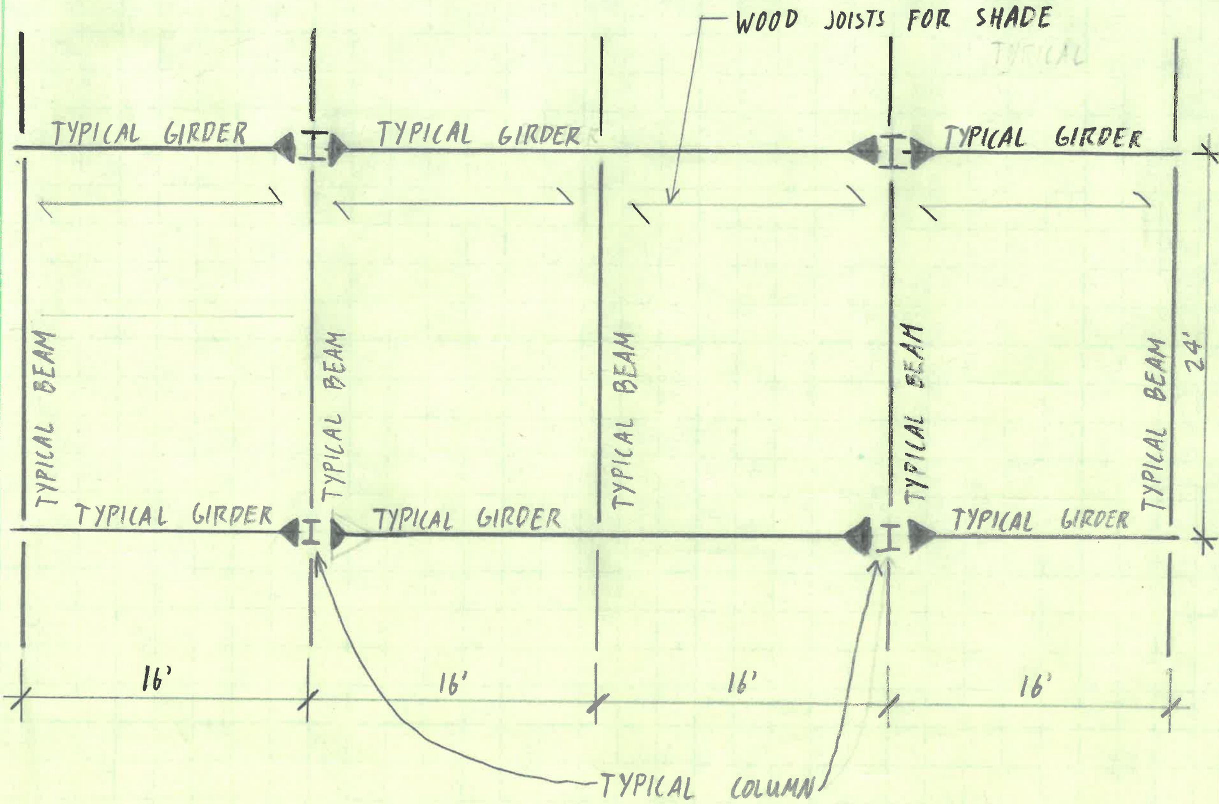
$\longrightarrow$ TYPICAL GRADE BEAM:

$h = 12"$, $b = 24"$ GRADE BEAM WITH
(3) #7 BARS FOR LONGITUDINAL REINF.
AND #3 TIES @ 4" O.C. FOR SHEAR REINF.
OR SIMILAR

ROOF STRUCTURE 2 CALCULATIONS

ROOF STRUCTURE 2 CALCULATIONS:

TYPICAL BAY:



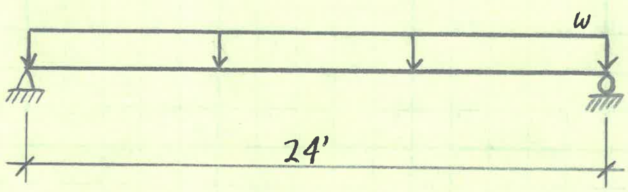
LOADS:

- L = 0 PSF (NO ROOF)
- D = 6 PSF (ASSUMPTION BASED ON 2x12 JOISTS)

ROOF STRUCTURE 2 CALCULATIONS:

TYPICAL BEAM:

- D= 6 PSF, L = 0 PSF
- LOADING:



  - w= (6 PSF) (16') = 96 PLF = 0.096 klf
- TOTAL LOAD = (0.096 klf) (24') = 2.304 k
- L_b = 24'
- TRY W12 x 14: AISC STEEL CONST. MANUEL 9TH EDITION Pg. 2-68

  ALLOWABLE UNIFORM LOADS = 10 k > 2.304 k ✓

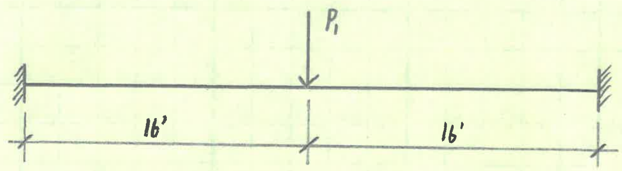
→ TYPICAL BEAM:

W12 x 14 BEAM OR SIMILAR
WITH L = 24'

ROOF STRUCTURE 2 CALCULATIONS:

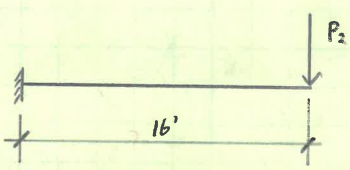
TYPICAL GIRDER:

- LOADING 1:



- $P_1 = (0.096\ klf + 0.014\ klf)(24') = 2.64\ k$
- $M_1 = P_1 L / 8 = (2.64\ k)(32')/8 = 10.56\ k'$
- $L_b = 16'$

- LOADING 2:



- $P_2 = [(0.096\ klf/2) + 0.014\ klf](24') = 1.49\ k$
- $M_2 = P_2 L = (1.49\ k)(16') = 23.84\ k'$
- $L_b = 16'$

- TRY W12 x 26: AISC STEEL CONST. MANUEL 9TH EDITION, Pg. 2-174

ALLOWABLE MOMENT = 37.0 k' > M_{MAX} = 23.84 k' ✓

→ TYPICAL GIRDER:

W12 x 26 GIRDER OR SIMILAR FOR BOTH FIXED-FIXED SECTION WITH L = 32' AND CANTILEVER SECTION WITH L = 16'

ROOF STRUCTURE 2 CALCULATIONS:

TYPICAL COLUMN:

- AXIAL LOAD:
  - $P_{GIRDER} = \frac{1}{2}(2.64\ k) + (1.49\ k) + (0.026\ klf)(32') = 3.642\ k$
  - $P_{BEAM} = 2.64\ k$
  - $P_{TOTAL} = P_{GIRDER} + P_{BEAM} = 3.642^{k} + 2.64^{k} \approx 6.3\ k$
- LATERAL LOAD:
  - $S_{DS} = 0.862,\ S_{D1} : NULL$
  - $T \approx T_a = C_t h_n^{x}$: $C_t = 0.028$

    $x = 0.8$

    $h_n = 25'$

    $\longrightarrow T \approx T_a = (0.028)(25')^{0.8} = 0.368\ s < T_L = 8s$
  - $C_s = \dfrac{S_{DS}}{(R/I_e)}$: $R = 3.5$ (STEEL ORDINARY MOMENT FRAME)

    $\longrightarrow C_s = \dfrac{0.862}{(3.5/1.0)} = 0.246$
  - $W = P_{TOTAL} = 6.3^{k}$
  - $V = C_s W = (0.246)(6.3\ k) \approx 1.6^{k}$
  - $M = V \cdot h = (1.6\ k)(25') = 40\ k'$

SAME POLYTECHNIC BRYCE GAGNER

ROOF STRUCTURE 2 CALCULATIONS:

TYPICAL COLUMN:

- TRY W10 x 33

- CHECK AXIAL ONLY: AISC STEEL CONSTRUCTION MANUEL 9TH EDITION, Pg. 3-30
  - $K = 1$, $L = 25' \longrightarrow KL = (1)(25') = 25'$, $r_y = 1.94"$, $A = 9.71\ in^2$, $Z_x = 38.8\ in^3$
  - $P_{ALLOW} = 56\ k > P = 6.3\ k$ ✓

- CHECK COMBINED AXIAL AND BENDING MOMENT: N4
  - $\frac{P}{P_{cr}} + \frac{C_m M}{(1-P/P_e)M_m} = \frac{6.3^k}{103.1\ k} + \frac{(0.85)(40\ k')}{(1-6.3^k/116.2\ k)(90.4\ k')} = 0.459 < 1.0$ ✓
    - $P_{cr} = 1.7 F_a A$: $C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = \sqrt{\frac{2\pi^2 (29{,}000\ ksi)}{36\ ksi}} = 126.1$

      $KL/r = (1)(25')(12"/1')/(1.94") = 154.64 > C_c = 126.1$

      $\longrightarrow F_a = \frac{12\pi^2 E}{23(KL/r)^2} = \frac{12\pi^2 (29{,}000\ ksi)}{23(154.64)^2} = 6.245\ ksi$

      $\longrightarrow P_{cr} = 1.7(6.245\ ksi)(9.71\ in^2) = 103.1\ k$
    - $C_m = 0.85$
    - $P_e = (23/12)F_e' A$: $F_e' = \frac{12\pi^2 E}{23(KL/r)^2} = F_a = 6.245\ ksi$

      $\longrightarrow P_e = (23/12)(6.245\ ksi)(9.71\ in^2) = 116.2\ k$
    - $M_m = \left[1.07 - (L/r)\sqrt{F_y}/3160\right] M_{px} \leq M_{px}$:
      - $M_{px} = F_y Z_x = (36\ ksi)(38.8\ in^3)(1'/12") = 116.4\ k'$

      $\longrightarrow M_m = \left[1.07 - (25' \cdot 12"/1'/1.94")\sqrt{36}/3160\right](116.4\ k') = 90.4\ k' < 116.4\ k'$ ✓

ROOF STRUCTURE 2 CALCULATIONS:

TYPICAL COLUMN:

- CHECK COMBINED AXIAL AND BENDING MOMENT:
  - $\frac{P}{P_y} + \frac{M}{1.18 M_p} = \frac{6.3\ k}{349.56\ k} + \frac{40\ k'}{1.18(116.4\ k')} = 0.309 < 1.0$ ✓
    - $P_y = F_y A = (36\ ksi)(9.71\ in^2) = 349.56\ k$
    - $M_p = M_{px} = 116.4\ k'$
  - $M = 40\ k' < M_p = 116.4\ k'$ ✓

→ TYPICAL COLUMN:

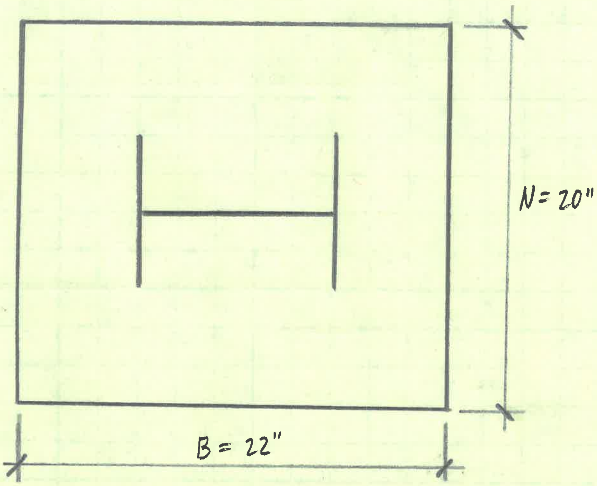
W10 x 33 OR SIMILAR
WITH L = 25'

SAME POLYTECHNIC BRYCE GAGNER

ROOF STRUCTURE 2 CALCULATIONS:

TYPICAL COLUMN BASE PLATE:

- COLUMN: W10 x 33 $d = 9.73''$ $b_f = 7.96''$
- $P = 6.3\ k$
- PLATE STRENGTH: $F_y = 36\ ksi$
- FOOTING SIZE: 3' x 3', $f'_c = 2\ ksi$
- TRY $N = 20''$, $B = 22''$



- CHECK BEARING:
  - $P_p = 0.8 f'_c A_1 \sqrt{A_2/A_1} \leq 1.7 f'_c A_1$
    - $A_1 = N \cdot B = 20'' \cdot 22'' = 440\ in^2$
    - $A_2 = (3' \cdot 3')(12''/1')^2 = 1296\ in^2$
    - $1.7 f'_c A_1 = 1.7(2\ ksi)(440\ in^2) = 1496\ k$

  $\longrightarrow P_p = 0.8(2\ ksi)(440\ in^2)\sqrt{1296\ in^2/440\ in^2} = 1208\ k < 1.7 f'_c A_1 = 1496\ k$

  - ALLOWABLE BEARING $= 0.65(1208\ k) = 785.2\ k >> P = 6.3\ k$

ROOF STRUCTURE 2 CALCULATIONS:

TYPICAL COLUMN BASE PLATE:

- $m = \frac{N - 0.95d}{2} = \frac{20'' - 0.95(9.73'')}{2} = 5.38$

- $n = \frac{B - 0.8b_f}{2} = \frac{22'' - 0.8(7.96'')}{2} = 7.82$

- $n' = \frac{\sqrt{d b_f}}{4} = \frac{\sqrt{(9.73'')(7.96'')}}{4} = 2.20$

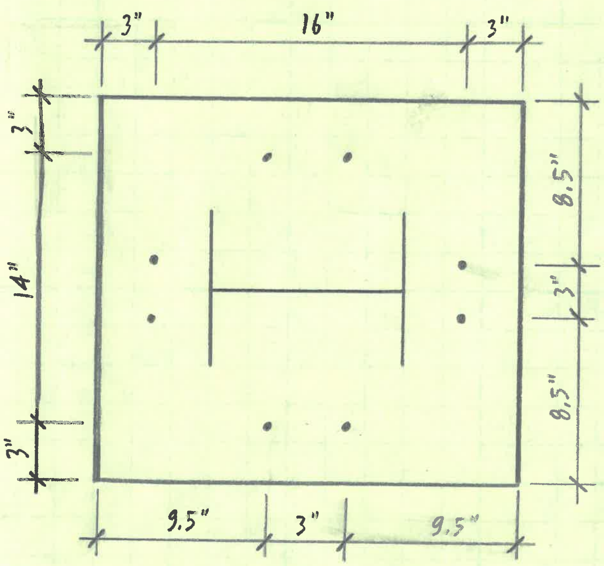
- $\lambda = 1$ (CONSERVATIVE

$\longrightarrow l = 7.82$

- $t_{MIN} = l\sqrt{\frac{2P}{0.9 F_y BN}} = (7.82)\sqrt{\frac{2(6.3\ k)}{0.9(36\ ksi)(22'')(20'')}} = 0.232''$

$\longrightarrow$ USE $t_{MIN} = 0.25'' = 1/4''$ PLATE

- BOLTS:



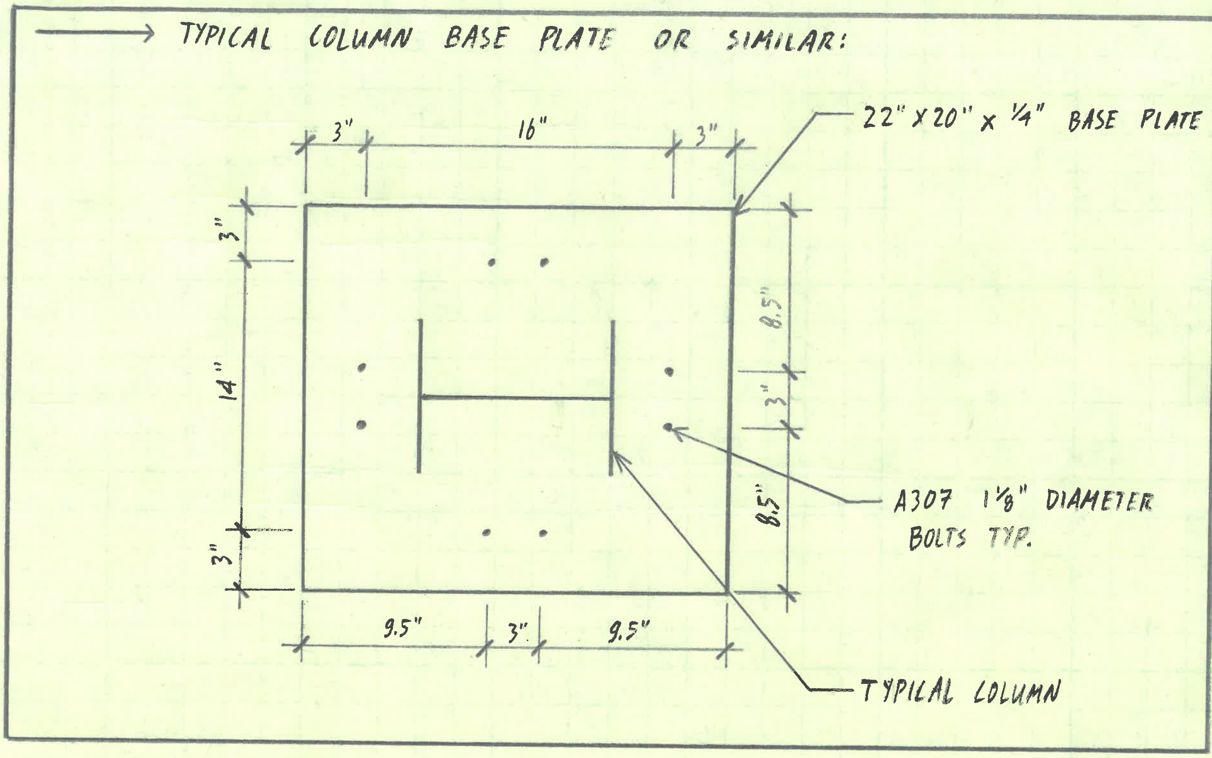
- $M = 40\ k'$
- $d_{MIN} = 14''$
- $T = M/d = (40\ k')/(14'' \cdot 1'/12'') = 34.3\ k$
- $T_{EACH\ BOLT} = 34.3\ k/2 = 17.2\ k$

- ASSUMING A307 BOLTS:

$\longrightarrow$ USE 1 1/8" BOLTS, $T_{ALLOW} = 19.9\ k > T = 17.2\ k$ ✓

ROOF STRUCTURE 2 CALCULATIONS:

TYPICAL COLUMN BASE PLATE:



ROOF STRUCTURE 2 CALCULATIONS:

TYPICAL ISOLATED FOOTING:

- $q_{ALLOW} = 2$ ksf
- $P_D = 6.3^k + (25')(0.03 \text{ klf}) \simeq 7.1$ k
- $P_u = 1.4 P_D = 1.4(7.1 \text{ k}) = 9.94$ k
- BASE PLATE: 22" x 20"

* SINCE THE BASE PLATE IS LARGER AND THE AXIAL LOAD IS SMALLER, WE ARE SAFE TO USE THE SAME DESIGN AS FOR ROOF STRUCTURE 1.

→ TYPICAL ISOLATED FOOTING

3' x 3' WITH h = 12" WITH
(3) #6 BARS EACH WAY
OR SIMILAR

TYPICAL GRADE BEAM:

- $M_u = V \cdot L = 40$ k'
- M_u (FOR EACH BEAM) = $\frac{1}{2}$(40 k') = 20 k'

* SINCE THE ISOLATED FOOTING IS THE SAME SIZE AND THE MOMENT DEMAND IS LESS, WE ARE SAFE TO USE THE SAME DESIGN AS FOR ROOF STRUCTURE 1.

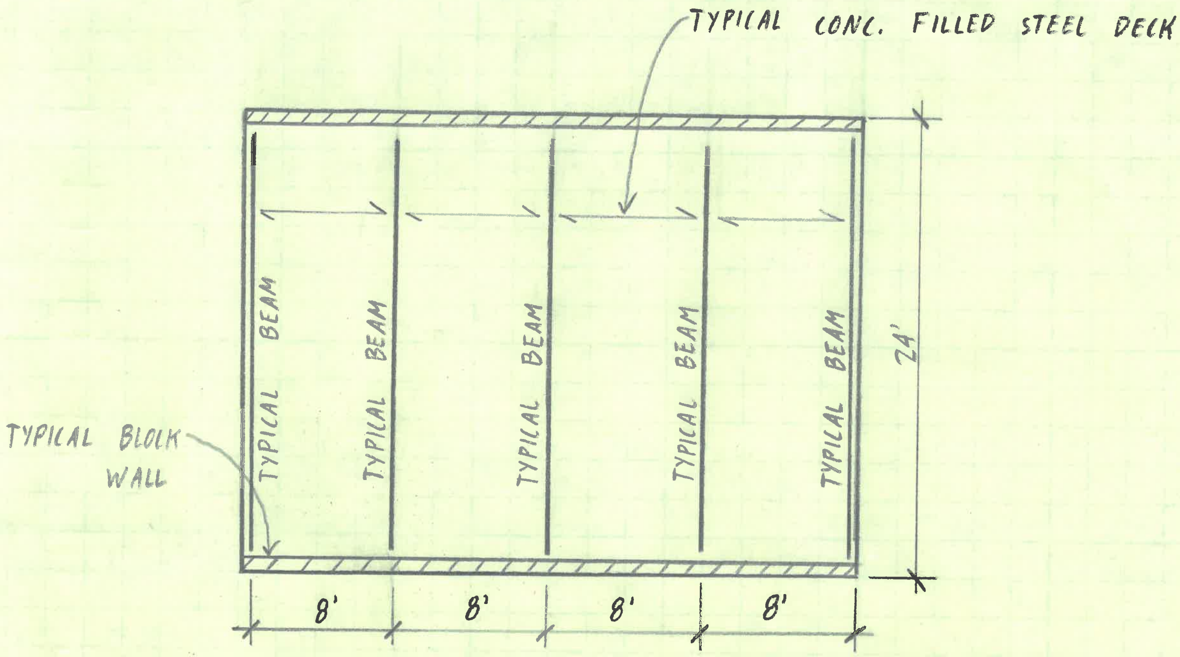
→ TYPICAL GRADE BEAM:

h = 12", b = 24" GRADE BEAM WITH
(3) #7 BARS FOR LONGITUDINAL REINF.
AND #3 TIES @ 4" O.C. FOR SHEAR REINF.
OR SIMILAR

BLOCK WALL CALCULATIONS

BLOCK WALL CALCULATIONS:

TYPICAL BAY:



TYPICAL CONC. FILLED STEEL DECK:

- L = 50 PSF (OFFICE)
- SPAN = 8'-0"
- NUMBER OF DECK SPANS = 4 > 3
- TRY 22 GAUGE W2 FLOOR DECK WITH 2" CONC. FILL
  - ALLOWABLE LOAD = 209 PSF > 50 PSF ✓

STEEL FLOOR DECK CATALOG VF5

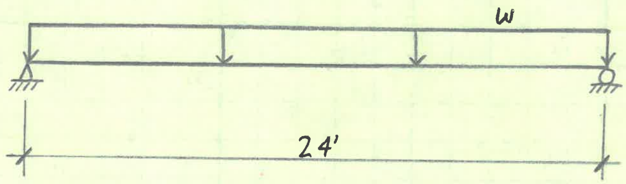
→ TYPICAL CONC. FILLED STEEL DECK:

22 GAUGE W2 (2" DEEP) FLOOR DECK WITH 2" CONC. FILL OR SIMILAR

BLOCK WALL CALCULATIONS:

TYPICAL BEAM:

- $D = 36.3$ PSF
- $L = 50$ PSF
- LOADING:



- $w = (8')(36.3 \text{ PSF} + 50 \text{ PSF}) = 690.4 \text{ PLF} = 0.6904$ klf

- TOTAL LOAD $= (0.6904 \text{ klf})(24') = 16.6$ k
- $L_b = 24'$
- TRY W12x26: AISC STEEL CONSTRUCTION MANUEL 9TH EDITION, Pg. 2-68

  ALLOWABLE UNIFORM LOADS $= 22 \text{ k} > 16.6 \text{ k}$ ✓

→ TYPICAL BEAM:

W12x26 BEAM OR SIMILAR WITH $L = 24'$

BLOCK WALL CALCULATIONS:

TYPICAL BLOCK WALL:

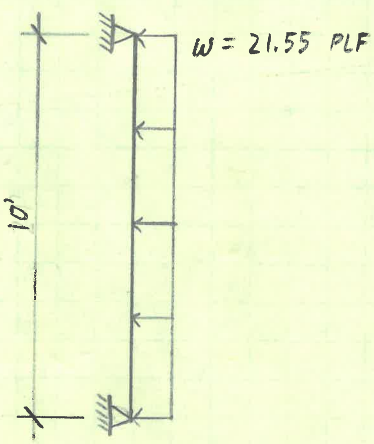
- OUT-OF-PLANE SEISMIC FORCE:

  $F_p = 0.4 S_{DS} I_e W_p$ ASCE 7-16 12.11

  - $S_{DS} = 0.862$
  - $I_e = 1.0$
  - $W = (125\ PCF)(6'' \cdot 1'/12'') = 62.5\ PSF$

  $\longrightarrow F_p = 0.4(0.862)(1.0)(62.5\ PSF) = 21.55\ PSF = 21.55\ PLF/FT$

- MOMENT DEMAND PER FOOT OF WALL:



  - $M = wL^2/8 = (21.55\ PLF)(10')^2/8 =$

    $= 269.4\ \#' = 0.27\ k' = 3.24\ k''$

- WALL CAPACITY:

  - ASSUME STEEL AT CENTER OF WALL
  - CONSERVATIVELY DON'T INCLUDE AXIAL LOAD
  - TRY #3 BARS @ 36" O.C.
    - $A_s = (0.110\ in^2)/(36'' \cdot 1'/12'') = 0.037\ in^2/FT$
    - $a = \dfrac{A_s f_y}{0.8 f'_m b} = \dfrac{(0.037\ in^2/FT)(40{,}000\ psi)}{0.8(500\ psi)(1' \times 12''/1')} = 0.308''$
    - $\phi M_n = \phi A_s f_y (d - a/2)$

      $= (0.9)(0.037 in^2/FT)(40\ ksi)(6''/2 - 0.308''/2) = 3.79\ k'' > M = 3.24\ k''$ ✓

BLOCK WALL CALCULATIONS:

TYPICAL BLOCK WALL:

→ TYPICAL BLOCK WALL:

6" BLOCK WALL WITH #3 BARS @ 36" O.C. EACH WAY OR SIMILAR

BLOCK WALL CALCULATIONS:

TYPICAL CONTINUOUS FOOTING:

- LOADS:
  - $D = 36.3\ PSF + 5(24')(26.0\ PLF)/(24' \cdot 32') = 40.4\ PSF$
  - $L = 50\ PSF$
  - $W_u = 1.2D + 1.6L = [1.2(40.4\ PSF) + 1.6(50\ PSF)](24') = 3083.5\ PLF \approx 3.1\ klf$
  - $w = [40.4\ PSF + 50\ PSF](24') = 2169.6\ PLF \approx 2.2\ klf$
- $q_{ALLOW} = 2\ ksf$
- SOIL BEARING:
  - $b_{REQ} = w/q_{ALLOW} = (2.2\ klf)/(2\ ksf) = 1.1\ FT$

  $\longrightarrow$ USE WIDTH = 2 FT
- BEAM / ONE-WAY SHEAR:
  - $q_u = w_u / b = (3.1\ klf)/(2FT) = 1.55\ ksf$
  - V_u (@ FACE OF WALL) $= q_u \cdot \frac{1}{2}(2' - 0.5') = (1.55 ksf) \cdot \frac{1}{2}(2' - 0.5') = 1.163\ k/FT$
  - TRY $t = 12"$
    - $d = 12" - 3" - \frac{1}{2}(4/8") = 8.75"$ (ASSUMING #4 BARS)
    - $V_c = 2 \cdot \lambda \sqrt{f'_c}\, bd = 2(1)\sqrt{2{,}000\ psi}\,(24")(8.75") = 18783\ \# = 18.78\ k/FT$
    - $\phi V_n = (0.75)(18.78\ k/FT) = 14.1\ k/FT > V_u = 1.163\ k/FT$ ✓
- BENDING:
  - $M_u = q_u \cdot \left(\frac{(b - b_{WALL})}{2}\right)^2 / 2 = (1.55\ ksf) \cdot \left(\frac{(2' - 0.5')}{2}\right)^2 / 2 = 0.44\ k'/FT$
  - $A_{s,MIN} = 0.0027\ bh = 0.0027(12")(12") = 0.389\ in^2 / FT$
  - $S_{MAX} = 18"$

BLOCK WALL CALCULATIONS:

TYPICAL CONTINUOUS FOOTING:

- BENDING:

  - DETERMINE A_s: ASSUMING TENSION CONTROLLED

$$a = d - \sqrt{\frac{-2M_u}{\phi(0.85 f'_c b)} + d^2}$$

$$= 8.75" - \sqrt{\frac{-2(0.44 k'/FT)(12"/1')}{(0.9)(0.85)(2 ksi)(12")} + (8.75")^2} = 0.033"$$

$$A_s = \frac{0.85 f'_c a b}{f_y} = \frac{0.85(2 ksi)(0.033")(12")}{40 ksi} = 0.017 in^2/FT < 0.389 in^2/FT$$

$\longrightarrow$ TRY #4 @ 6" O.C. $A_s = 0.200 in^2 / (6" \cdot 1'/12") = 0.4 in^2/FT > 0.389 in^2/FT$ ✓

*BARS MUST BE HOOKED FOR DEVELOPMENT LENGTH

  - ϕM_n: $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.4 in^2/FT)(40 ksi)}{0.85(2 ksi)(12")} = 0.784"$

$$\phi M_n = \phi A_s f_y (d - a/2)$$

$$= (0.9)(0.4 in^2/FT)(40 ksi)(8.75" - 0.784"/2) = 120.36 k"/FT$$

$$= 10.03 k'/FT > M_u = 0.44 k'/FT$$ ✓

  - CHECK IF TENSION CONTROLLED:

$$c = a/\beta_1 = 0.784" / 0.85 = 0.922"$$

$$\varepsilon_s = \frac{\varepsilon_c(d-c)}{c} = \frac{(0.003)(8.75" - 0.922")}{0.922"} = 0.0255 > 0.005$$ ✓

$\longrightarrow$ FOOTING IS TENSION CONTROLLED

- MINIMUM STEEL IN LONG DIRECTION:

  - $A_{s,MIN} = 0.0020bh = 0.0020(12")(24") = 0.576 in^2$

$\longrightarrow$ USE (3) #4 BARS $A_s = 3(0.2 in^2) = 0.6 in^2$

$\longrightarrow$ TYPICAL CONTINUOUS FOUNDATION:

2' WIDE, 12" DEEP CONTINUOUS WALL FOOTING
WITH (3) #4 BARS IN LONG DIRECTION AND
#4 BARS @ 6" O.C. IN SHORT DIRECTION OR SIMILAR

Appendix C: Cost Analysis Calculations

Total Costs and Material Amounts

Section	Labor Hours	Labor Cost ($)	Labor Cost (Shillings)	Cost ($)	Cost (Shillings)	Cost w/ O&P ($)	Cost w/ O&P (Shillings)
Foundations	13,434.02	718,954.08	1,653,594,393.54	1,364,664.89	3,138,729,256.71	1,787,863.65	4,112,086,397.30
Block Walls	3,747.08	176,859.43	406,776,696.36	280,835.95	645,922,687.76	384,047.84	883,310,032.00
Steel Framing	1,397.90	77,400.96	178,022,208.00	988,160.64	2,272,769,472.00	1,119,540.00	2,574,942,000.00
Steel Decking	1,071.36	62,496.00	143,740,800.00	349,977.60	804,948,480.00	416,044.80	956,903,040.00
Other	47.01	2,672.00	6,145,600.00	6,056.00	13,928,800.00	7,720.00	17,756,000.00
Misc. (10%)	1,965.04	103,571.05	238,213,409.79	298,363.91	686,236,989.65	370,749.63	852,724,146.93
Total	**21,662.41**	**1,141,953.53**	**2,626,493,107.69**	**3,288,058.99**	**7,562,535,686.12**	**4,085,965.92**	**9,397,721,616.23**
Rounded Total	**21670**	**1142000**	**2626494000**	**3289000**	**7562536000**	**4086000**	**9397722000**

Material	Material Amount		Material Cost (US Dollars)	Material Cost (Shillings)
Block Wall	35840.00	ft^2	92108.80	211850240
Concrete	2603.40	CY	19942.07	45866769
Steel Decking	178560.00	ft^2	283910.40	652993920
Steel Framing	490939.20	lb.	490939.20	1129160160
Steel Reinforcing	623.11	Tons	638690.37	1468987860

Current Currency Exhange Rate				
1	US Dollar	=	2300	Tanzanian Shillings
				Date: 6-21-2019

Foundations

Grade Beams

Size	Length (ft)	Width (ft)	Depth (ft)	Number of Long. Reinforcement	Trans. Reinforcement Spacing (in.)	Long. Reinforcing Weight (lb/ft)	Trans. Reinforcing Weight (lb/ft)	Volume of Concrete (CY)	Weight of Reinforcing (Tons)		Labor Hours/CY	Concrete Mixing Labor Hours/CY	Placing Reinforcing Labor Hours/Ton		Labor Cost/CY	Concrete Labor Cost/CY	Placing Reinforcing Labor Cost/Ton	Concrete Cost/CY	Concrete Cost w/ O&P/CY	Reinforcing Cost/Ton	Reinforcing Cost w/ O&P/Ton	Total Cost/CY	Total Cost w/ O&P/CY	Number of Elements	Labor Hours	Labor Cost	Cost	Cost w/ O&P
2'x1' Grade Beam with a length of 32' with (3) #7 Bars of longitudinal reinforcement and #3 Stirrups @ 4" O.C.	32	2	1	3	4	2.044	0.376	2.37037037	1.65672	ft	0.32	0.059	20	hrs	13.65	2.43	1100	7.66	9.45	2125	2775	14.01	21	12	408.39	22,326.09	42,862.75	56,034.91
2'x1' Grade Beam with a length of 24' with (3) #7 Bars of longitudinal reinforcement and #3 Stirrups @ 4" O.C.	24	2	1	3	4	2.044	0.376	1.777777778	0.97182	ft	0.32	0.059	20	hrs	13.65	2.43	1100	7.66	9.45	2125	2775	14.01	21	540	10,859.50	592,697.88	1,135,966.65	1,485,504.27
								0		ft				hrs											0.00	0.00	0.00	0.00
								0		ft				hrs											0.00	0.00	0.00	0.00
								0		ft				hrs											0.00	0.00	0.00	0.00
								0		ft				hrs											0.00	0.00	0.00	0.00

Wall Footings

Size	Length (ft)	Width (ft)	Depth (ft)	Long. Reinforcement Spacing (in.)	Trans. Reinforcement Spacing (in.)	Reinforcing Weight (lb/ft)	Volume of Concrete (CY)	Weight of Reinforcing (Tons)			Labor Hours/CY	Concrete Mixing Labor Hours/CY	Placing Reinforcing Labor Hours/Ton		Labor Cost/CY	Concrete Labor Cost/CY	Placing Reinforcing Labor Cost/Ton	Concrete Cost/CY	Concrete Cost w/ O&P/CY	Reinforcing Cost/Ton	Reinforcing Cost w/ O&P/Ton	Total Cost/CY	Total Cost w/ O&P/CY	Number of Elements	Labor Hours	Labor Cost	Cost	Cost w/ O&P
2' Wide Continuous Footing for 10' Block Wall with #4 Bars @ 6" O.C. and (3) #4 along the length of the wall	2368	2	1	6	9	0.668	175.4074074	6.92048		ft	0.4	0.059	15.238	hrs	17.1	2.43	835	7.66	9.45	1860	2400	17.54	26	1	185.97	9,204.31	17,292.36	22,827.34
2' Wide Continuous Footing for 20' Block Wall with #4 Bars @ 6" O.C. and (3) #4 along the length of the wall	608	2	1	6	9	0.668	45.03703704	1.77688		ft	0.4	0.059	15.238	hrs	17.1	2.43	835	7.66	9.45	1860	2400	17.54	26	1	47.75	2,363.27	4,439.93	5,861.07
							0			ft				hrs											0.00	0.00	0.00	0.00
							0			ft				hrs											0.00	0.00	0.00	0.00
							0			ft				hrs											0.00	0.00	0.00	0.00
							0			ft				hrs											0.00	0.00	0.00	0.00

Isolated Footings

Size	Length (ft)	Width (ft)	Depth (ft)	Long. Reinforcement Spacing (in.)	Trans. Reinforcement Spacing (in.)	Reinforcing Weight (lb/ft)	Volume of Concrete (CY)	Weight of Reinforcing (Tons)			Labor Hours/CY	Concrete Mixing Labor Hours/CY	Placing Reinforcing Labor Hours/Ton		Labor Cost/CY	Concrete Labor Cost/CY	Placing Reinforcing Labor Cost/Ton	Concrete Cost/CY	Concrete Cost w/ O&P/CY	Reinforcing Cost/Ton	Reinforcing Cost w/ O&P/Ton	Total Cost/CY	Total Cost w/ O&P/CY	Number of Elements	Labor Hours	Labor Cost	Cost	Cost w/ O&P
3' x 3' Footing underneath Steel Frame Columns with (3) #6 Bars Each Way	3	3	1	13	13	1.502	0.3333333333	0.0168975		ft	0.873	0.059	15.238	hrs	37.5	2.43	835	7.66	9.45	1860	2400	38.47	57	24	13.64	658.07	1,123.34	1,504.90
3' x 3' Footing underneath Steel Frame Columns with (3) #6 Bars Each Way	3	3	1	13	13	1.502	0.3333333333	0.0168975		ft	0.873	0.059	15.238	hrs	37.5	2.43	835	7.66	9.45	1860	2400	38.47	57	280	159.08	7,677.44	13,105.68	17,557.12
							0			ft				hrs											0.00	0.00	0.00	0.00
							0			ft				hrs											0.00	0.00	0.00	0.00
							0			ft				hrs											0.00	0.00	0.00	0.00
							0			ft				hrs											0.00	0.00	0.00	0.00

Slab on Grade

Size	Length (ft)	Width (ft)	Depth (ft)	Long. Reinforcement Spacing (in.)	Trans. Reinforcement Spacing (in.)	Reinforcing Weight (lb/ft)	Volume of Concrete (CY)	Weight of Reinforcing (Tons)			Labor Hours/CY	Concrete Mixing Labor Hours/CY	Placing Reinforcing Labor Hours/Ton		Labor Cost/CY	Concrete Labor Cost/CY	Placing Reinforcing Labor Cost/Ton	Concrete Cost/CY	Concrete Cost w/ O&P/CY	Reinforcing Cost/Ton	Reinforcing Cost w/ O&P/Ton	Total Cost/CY	Total Cost w/ O&P/CY	Number of Elements	Labor Hours	Labor Cost	Cost	Cost w/ O&P
600'x280' Slab on Grade witha thickness of 4" and #3 Bars @ 18" O.C. Each Way	600	280	0.3333333333	18	18	0.376	2074.074074	52.687		ft	0.436	0.059	13.913	hrs	18.65	2.43	765	7.66	9.45	1790	2275	19.13	28.5	1	1,759.70	84,027.04	149,874.17	198,574.04
							0			ft				hrs											0.00	0.00	0.00	0.00
							0			ft				hrs											0.00	0.00	0.00	0.00
							0			ft				hrs											0.00	0.00	0.00	0.00
							0			ft				hrs											0.00	0.00	0.00	0.00
							0			ft				hrs											0.00	0.00	0.00	0.00

Total Labor Hours =	13,434.02	hrs
Total Labor Cost =	718,954.08	US Dollars
Total Cost =	1,364,664.89	US Dollars
Total Cost w/ O&P =	1,787,863.65	US Dollars

Material Amounts and Costs:		
Concrete Volume =	2603.40	CY
Concrete Cost =	19942.07	US Dollars
Reinforcing Weight =	611.18	Tons
Reinforcing Cost Per Ton =	1025	US Dollars/Ton
Reinforcing Cost =	626464.26	US Dollars

Block Walls

Steel Framing

Wall Thickness	Height (ft)	Length (ft)	Area (ft^2)	Long. Reinforcement Spacing (in.)	Trans. Reinforcement Spacing (in.)	Reinforcing Weight (lb/ft)	Weight of Reinforcing (Tons)		Labor Hours/SF	Placing Reinforcing Labor Hours/Ton		Labor Cost/SF	Placing Reinforcing Labor Cost/Ton	Reinforcing Cost/Ton	Total Cost/SF	Reinforcing Cost w/ O&P/Ton	Total Cost w/ O&P/SF	Labor Hours	Labor Cost	Cost	Cost w/ O&P
6" Thick Wall with #3 Bars at 32" O.C. Each Way	10	2368	23680	32	32	0.376	4.31272	ft	0.101	10.667	hrs	4.74	585	1610	7.3	2000	10.05	2,437.68	114,766.14	179,807.48	246,609.44
6" Thick Wall with #4 Bars at 16" O.C. Each Way	20	608	12160	16	16	0.668	7.6152	ft	0.101	10.667	hrs	4.74	585	1610	7.3	2000	10.05	1,309.39	62,093.29	101,028.47	137,438.40
			0					ft			hrs							0.00	0.00	0.00	0.00
			0					ft			hrs							0.00	0.00	0.00	0.00
			0															0.00	0.00	0.00	0.00
			0															0.00	0.00	0.00	0.00

Total Labor Hours =	3,747.08	hrs
Total Labor Cost =	176,859.43	US Dollars
Total Cost =	280,835.95	US Dollars
Total Cost w/ O&P =	384,047.84	US Dollars

Material Amounts and Costs:		
6" Block Wall Area =	35840.00	ft^2
6" Block Wall Cost/Area =	2.57	US Dollars/ft^2
6" Block Wall Cost =	92108.80	US Dollars
Reinforcing Weight =	11.93	Tons
Reinforcing Cost Per Ton =	1025	US Dollars/Ton
Reinforcing Cost =	12226.12	US Dollars

Steel Framing

Steel Framing

Size	Length (ft)	Weight (PLF)	Labor Hours/LF		Labor Cost/LF	Total Cost/LF	Total Cost w/ O&P/LF	Number of Elements	Labor Hours	Labor Cost	Cost	Cost w/ O&P
W10x33 Column in the Roof Structure 2	25	33	0.054	hrs	3.01	70.16	78.5	24	32.40	1,806.00	42,096.00	47,100.00
W12x14 Beam in the Roof Structure 2	24	14	0.064	hrs	3.53	28.96	33	48	73.73	4,066.56	33,361.92	38,016.00
W12x26 Girder in the center of Roof Structure 2	32	26	0.089	hrs	4.97	131.96	148	12	34.18	1,908.48	50,672.64	56,832.00
W12x26 Girder of the Cantilever in Roof Structure 2	16	26	0.089	hrs	4.97	131.96	148	24	34.18	1,908.48	50,672.64	56,832.00
W12x14 Beam in Roof Strcture 1	24	14	0.064	hrs	3.53	28.96	33	495	760.32	41,936.40	344,044.80	392,040.00
W12x40 Girder in Roof Structure 1	24	40	0.069	hrs	3.84	68.44	77.5	264	437.18	24,330.24	433,635.84	491,040.00
HSS 8x8x5/16 in Roof Structure 1	20	31.84	0.054	hrs	3.01	70.16	78.5	24	25.92	1,444.80	33,676.80	37,680.00
				hrs								
				hrs								
				hrs					0.00	0.00	0.00	0.00

Total Labor Hours =	1,397.90	hrs
Total Labor Cost =	77,400.96	US Dollars
Total Cost =	988,160.64	US Dollars
Total Cost w/ O&P =	1,119,540.00	US Dollars

Material Amounts and Costs:		
Steel Framing Material Amount =	490939.20	lb.
Steel Framing Material Cost =	910759.68	US Dollars

Steel Decking

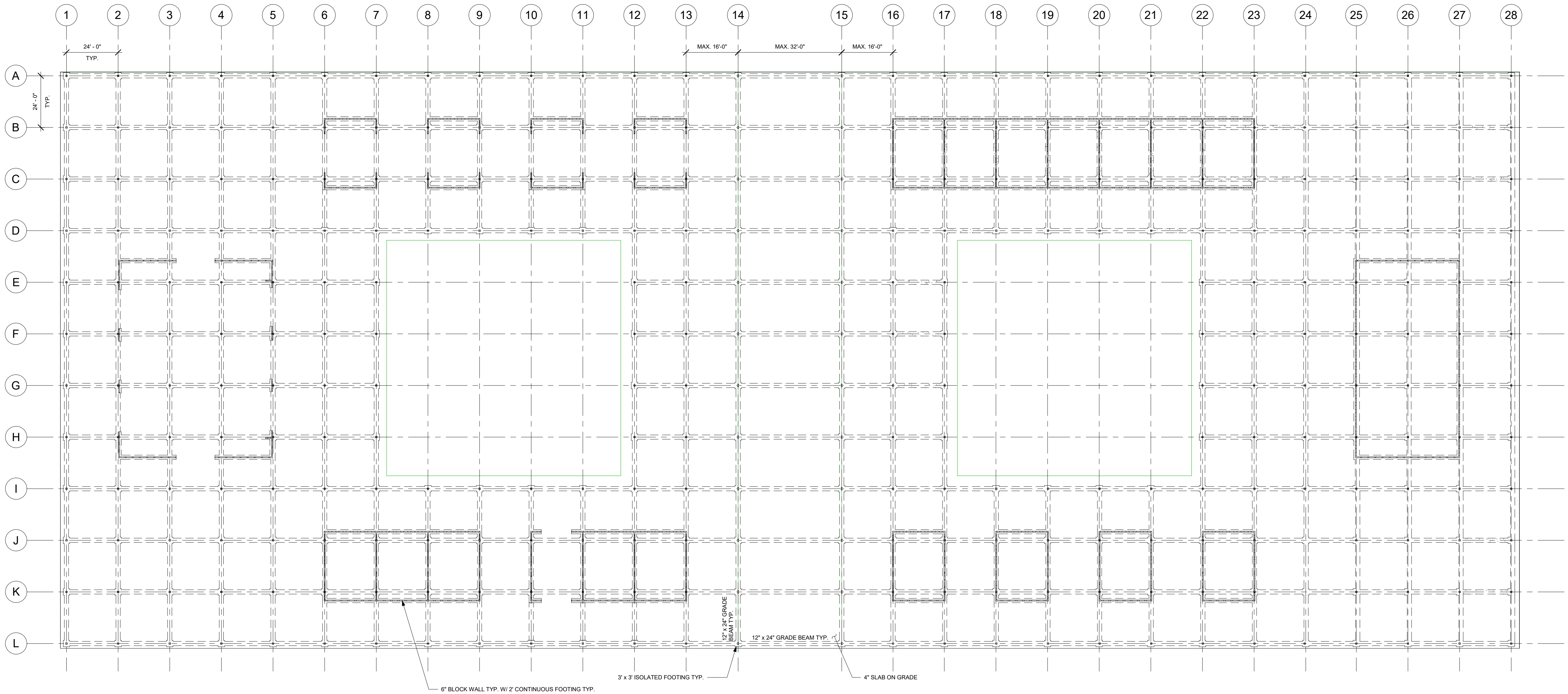
Steel Decking

Size	Length (ft)	Width (ft)	Area (ft^2)		Labor Hours/MBF		Labor Cost/MBF	Total Cost/MBF	Total Cost w/ O&P/MBF	Number of Elements	Labor Hours	Labor Cost	Cost	Cost w/ O&P
22 Gauge, 1-1/2" Steel Roof Deck for Floor on top of 10' Block Wall	170	32	5440	ft	0.006	hrs	0.35	1.96	2.33	4	130.56	7,616.00	42,649.60	50,700.80
22 Gauge, 1-1/2" Steel Roof Deck for Floor on top of Steel Framing in Roof Structure 1	280	280	78400	ft	0.006	hrs	0.35	1.96	2.33	2	940.80	54,880.00	307,328.0	365,344.00
			0	ft		hrs					0.00	0.00	0.00	0.00
			0	ft		hrs					0.00	0.00	0.00	0.00
			0	ft		hrs					0.00	0.00	0.00	0.00
			0	ft		hrs					0.00	0.00	0.00	0.00

Total Labor Hours =	1,071.36	hrs
Total Labor Cost =	62,496.00	US Dollars
Total Cost =	349,977.60	US Dollars
Total Cost w/ O&P =	416,044.80	US Dollars

Material Amounts and Costs:		
Steel Decking Material Area =	178560.00	ft^2
Steel Decking Material Cost/Area =	1.59	US Dollars/ft^2
Steel Decking Material Cost =	283910.40	US Dollars

Appendix D: Final Structural Plans

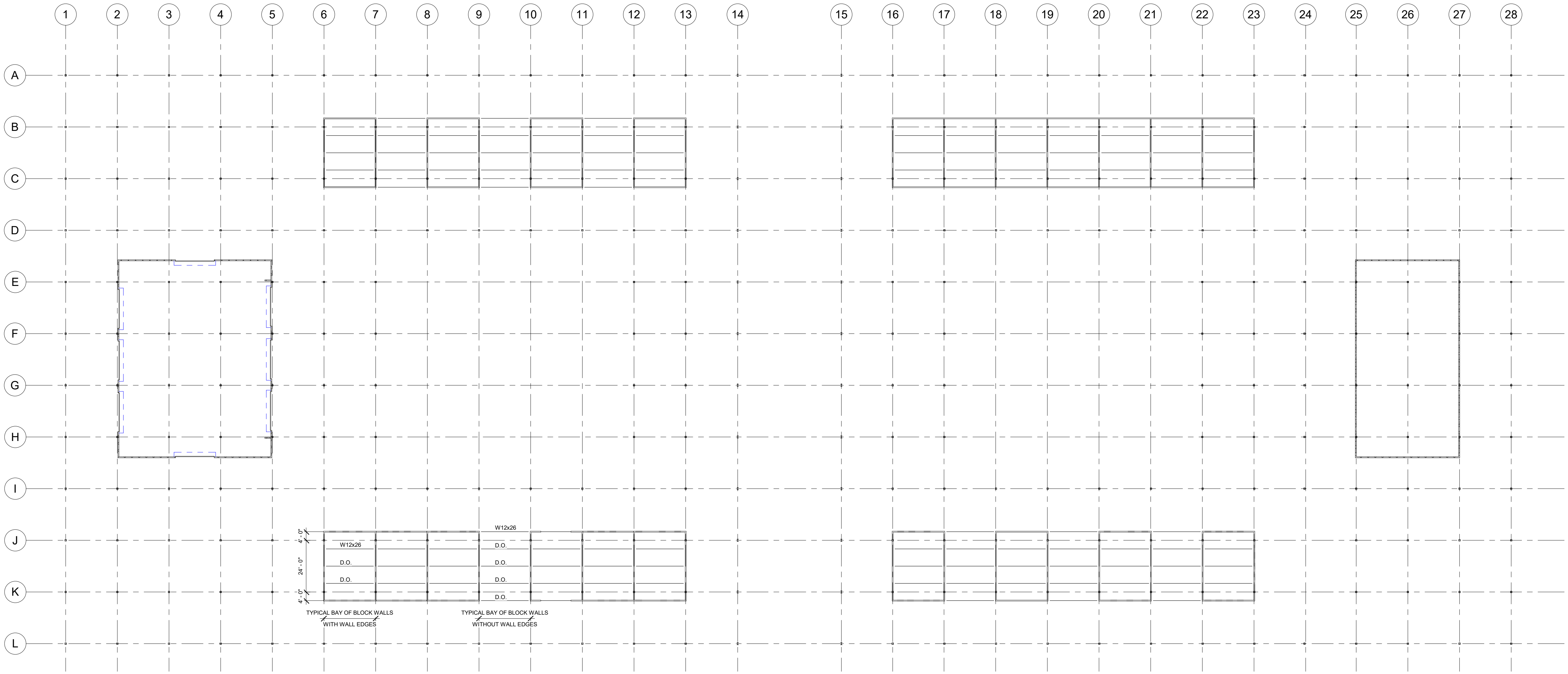


1 FOUNDATION PLAN

SCALE: 1" = 20'-0"

6/21/2019 10:57:30 AM

Same Polytechnic College	Drawn By: Bryce Gagner	Advisor: Kevin Dong	Foundation Plan	S1.1



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2ND FLOOR FRAMING PLAN

SCALE: 1" = 20'-0"

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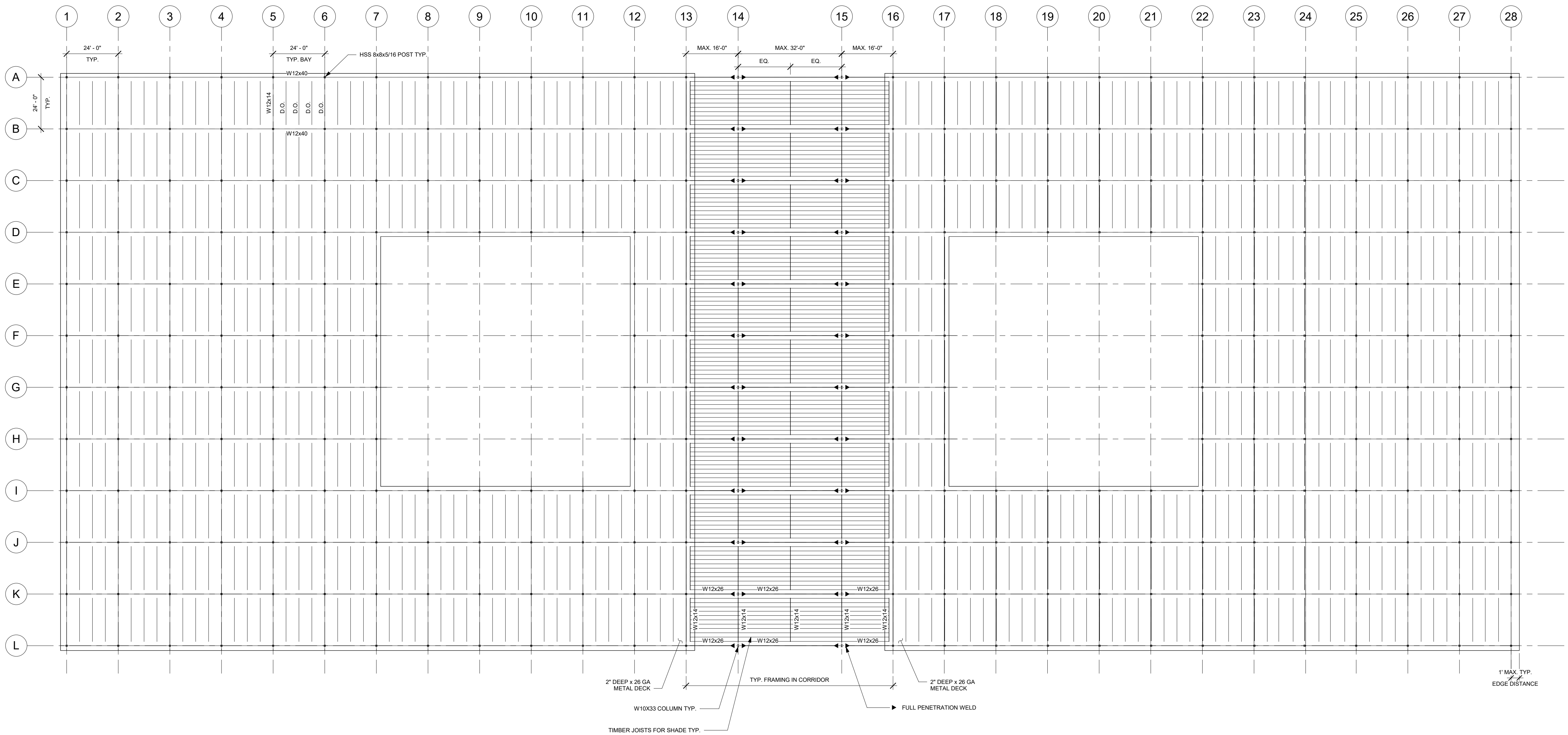

3

ROOF PLAN

SCALE: 1" = 20'-0"

6/21/2019 10:57:30 AM

Appendix E: References

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