STRUCTURAL CALCULATIONS

FOR

TAKE HEART SCHOOL IN KENYA

TAKE HEART

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Bookmark Summary

Take Heart School in Kenya

Take heart is a new organization focused on helping widows with small business opportunities, it has continued to grow over the years and now also provides orphans safe living conditions and education opportunities. This is Take Heart's first construction project; Take Heart bought a three-acre piece of land in Migori county in Kenya, where a Missionary base school will be built to provide care for orphans and windows. The overall project is a multistep project that when completed will have a primary school(pre k – 8th grade), dormitories , training center, guest house, and a cafeteria that will have a feeding program. There are currently 100 students that are being helped by take Heart 35 high school age, 30 widows, and 35 elementary age.

I worked on this project alongside Quinn Porter. We did the structural design of the school, which is the first phase and main focus of the project. I Focused on two classroom structures that are similar in design on the northern edge of the property. The school buildings are 80' by 25', each classroom is 20' by 25'. It is a rigid wall flexible diaphragms structure with shear walls between 8' tall by 6' wide door openings. The roof material is meat decking supported by square HSS members. The design has sloping walls in the north and south direction to create a gestalt look to the buildings as if the roof diaphragms slope creates a sense of connection and continuity between buildings.

Based on the location this project, using CMU as the primary building material was the best choice. CMU is a great material that is durable good for sound proofing and fire resistant. Using CMU allows for reduction of material cost since using other materials like timber or concrete would be more costly. CMU construction is more common in Migori County and allows for better labor quality due to material familiarity. Using local materials helps reduce the environmental effects transporting other materials would produce.

The outcome of this project will provide orphans and aged out orphans with education opportunities and safe living conditions. This will help Prevent human trafficking, child marriages, and crime. The microfinance loans, small business opportunities and access to the healthcare and clean water helps widows provide for their families through entrepreneurship opportunities and financial independence.

Design Criteria:

- Risk Category II
- Assumed Site Class D
- Seismic forces based on Sacramento CA
- Soil Load bearing capacity Assumed 3,000 (IBC TBL 1806.2)

Design Codes:

- ICB
- TMS 402/602-13
- ASCE 7-16

LOADS

Roof:

Metal Deck (3W 22gage) …………………………… 1.9psf ~ 2psf

MISC ………………………………………………… 3psf

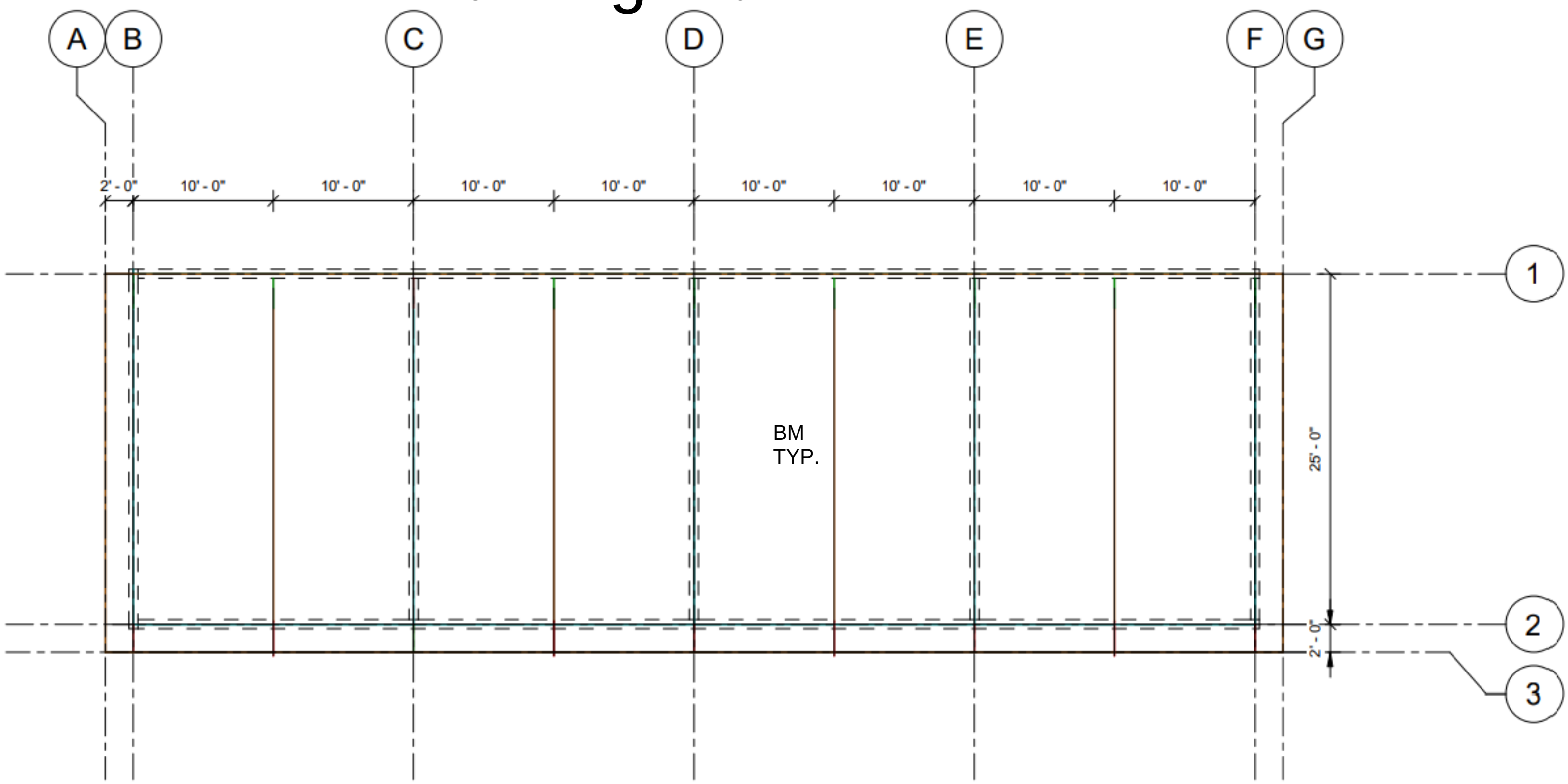
8x8x 1/2 HSS Beam weight ………………………… 48.85 plf

Wall:

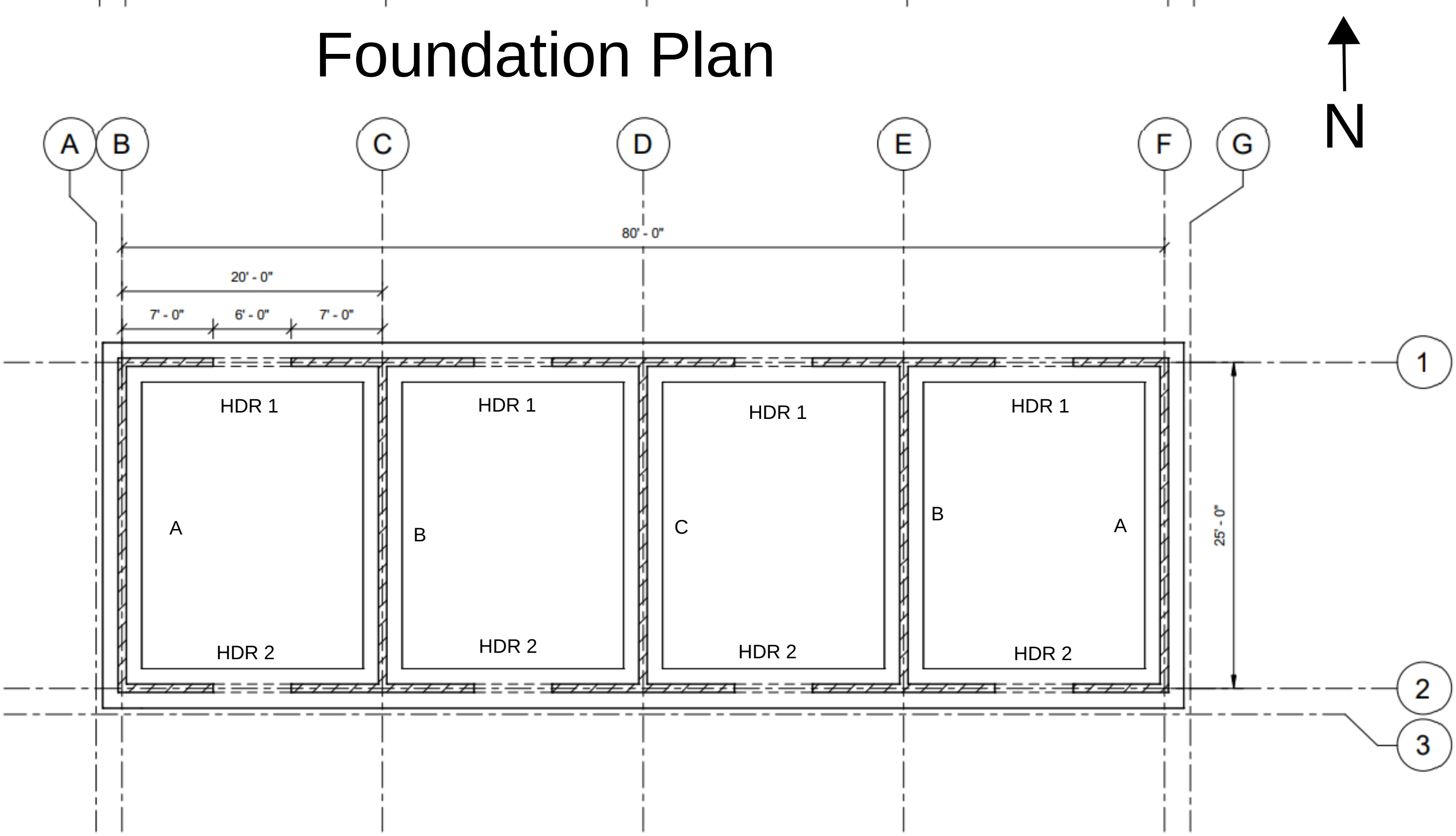
Fully grouted CMU ………………………………… 115 pcf

Key Plan

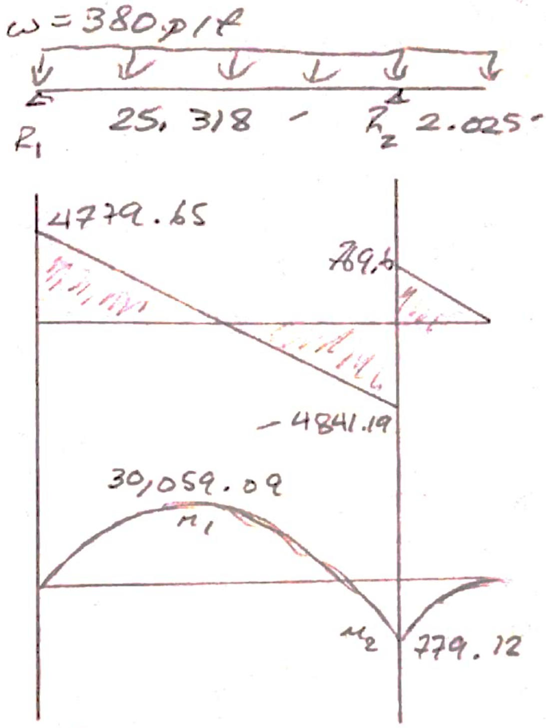
Framing Plan



Foundation Plan



BM :



L between supports:

$\alpha = 9.09°$

$L = \sqrt{25^2 + 4^2} = 25.318'$

cantilever $= \sqrt{2^2 + .32^2} = 2.025$

Load: $1.2D + 1.6L$ $D = 5psf$ $L = 20psf$

$1.2(5 \times 10) + 1.6(20 \times 10) = 380$ plf

$R_1 = [380/(2 \times 25.318)](25.318^2 - 2.025^2)$
$= 4{,}779.65$

$R_2 = [380/(2 \times 25.318)](25.318 + 2.025)^2$
$= 5{,}610.86$

$M_1 = [380/(8 \times 25.318^2)](25.318 + 2.025)^2 + (25.318 - 2.025)^2$
$= 30{,}059.09$ lb-ft

$M_2 = 380(2.025)^2/2 = 779.12$ lb-ft

$Z_{required} = M/\phi f_y$ $f_y = 50{,}000$ ksi $\phi = .9$
$= (30{,}059.09 \times 12)/(.9 \times 50{,}000) = 8.016$

Try HSS 8x8x½ $Z = 37.5 in^3 > 8.016$ ✓

Deflection check:

$\Delta allow = L/180$ (based on Live Load) $(25.318 + 2.025) \times 12/180 = 1.8"$

From Risa; $\Delta = 1.17"$

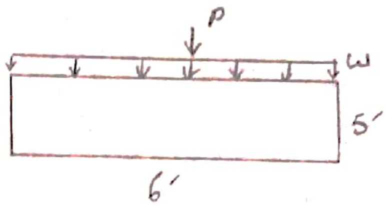
$\phi M_n = 74.3$ k-ft $> M_u$

$\phi V_n = .6 F_y A_w C_{v2}$ $F_y = 50$ ksi $C_{v2} = 14.2$ $A_w = .465 \times 6.603 = 3.07$

$\phi V_n = .6(50)(3.07)14.2$
$= 1307.8$ K

$\phi V_n > V_u$

CMU HDR 1:



$P = 4779.65 + (48.85 / (2\times25.318)\ (25.318^2 - 2.025^2))$
$= 5394.1$

$w = (78\times2)\times1.2 = 187.2$

$V = (187.2\times6)/2 + (5394.1/2) = 3258.65$ lb

$M = (5394.1\times6)/4 + (187.2\times6^2)/8 = 8,933.55$ lb-ft

Max Reinf Ratio:

$\rho_{max} = .64 \frac{f'_m}{f_y} \left(\frac{.0025}{1.5 f_y / E_s + .0025} \right)$ $f'_m = 2000$ $f_y = 60$ $E = 29000$

$\rho_{max} = .0095$

$d = \sqrt{M_u / .8 \phi \rho b f_y}$

$d = \sqrt{(8,933.55\times12)/(.8\times.9\times.0095\times7.625\times60000)} = 5.853$

$h = 60$ $d = 60 - 4 = 56$ use $d = 56"$

$A_s\ req = M_u / \phi f_y (.8d) \rightarrow (8933.55\times12)/(.9(60000)\times.8\times56) = .044$

Try #7 bar $A = .6$

$\rho = .6/(7.625\times56) = .0014 < \rho_{max}$ ✓

$\phi M_n = \phi f_y A_s (d - f_y A_s / 1.6 f'_m b)$

$\phi M_n = .9(60,000).6 \left(56 - \frac{60000\times.6}{1.6\times2000\times7.625}\right) \frac{1}{12}$
$= 147,216.4$

$\phi M_n > M_u$ ✓

$M_{cr} = S_n f_r$
$= 7.625\times60^2/6\ (267)/12 = 101,793.75$

$1.3 M_{cr} = 1.3\times101,793.75 = 132,331.9$

$1.3 M_{cr} < \phi M_n$ ✓

shear check:

$V_u = 3,258.65$ lb

$\phi V_{nm} = .8(2.25) A_n \sqrt{f'_m}$
$= .8(2.25)(7.625 \times 60)\sqrt{2000} = 36,828$

$\phi V_{nm} > V_u$

No reinforcement needed for shear

Deflection check:

$\Delta_{allow} = (6 \times 12)/180 = .4$

$\Delta = (5wl^4/384 E_m I_y) + (Pl^3/48 E_m I_y)$

$E_m = 1,800,000$

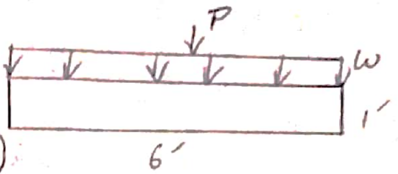
$I_g = 7.625 \times 60^3/12 = 137,250$

$I_y = .24 I_g \rightarrow .24(137,250) = 32940$

$\Delta = 5(187.2)6^4(12)^3/(384 \times 1800000 \times 32940)$
$+ (5394.1 \times (12 \times 6)^3)/(48 \times 1800000 \times 32940)$

$\Delta = .0001 + .0001 = .0002 < .4$ ✓

CMU HDR 2:



$P = 5{,}610.86 + \left(48.85/(25.318\times2) \times (25.318+2.025)^2\right)$
$= 6339.129$

$w: (115\times.67) = 77.05 \approx 78\ psf$
$w = (78\times1)\times1.2 = 93.6$

$V = (93.6\times6)/2 + (6339.129/2) = 3450.3645\ lb$

$M = (6339.129\times6)/4 + (96.3\times6^2)/8 = 9942.0435\ lb\text{-}ft$

max Reinf Ratio: $\rho_{max} = .64\ f'_m/f_y \left(\frac{.0025}{1.5\ f_y/E_s + .0025}\right)$

$f'_m = 2000\ psi$ $f_y = 60\ ksi$ $E = 29{,}000\ ksi$

$\rho_{max} = .64(2/60)\left(\frac{.0025}{1.5\ 60/2900 + .0025}\right)$
$= .0095$

$A_{s\,req} = M_u/\phi f_y(d - \frac{a}{2}) \approx M_u/\phi f_y(.8d)$

$d = \sqrt{M_u/.8\phi\rho b f_y}$

$d = \sqrt{(9942.0435\times12)/(.8\times.9\times.0095\times7.625\times60000)} = 6.17$

$h = 12$ $d = 12-4 = 8''$ use $d = 8''$

$A_{s\,req} = (9942.0435\times12)/(.9\times60000(.8\times8)) = 0.35$

Try #6 bar $A_s = .44$

$\rho = .44/(7.625\times8) = .0072 < \rho_{max}$ ✓

$\phi M_n = \phi f_y A_s \left(d - \frac{f_y A_s}{1.6 f'_m b}\right)$

$\phi M_n = .9(60{,}000).44\left(8 - \frac{60000\times.44}{1.6\times2000\times7.625}\right)\frac{1}{12}$
$= 13{,}697.7$

$\phi M_n > M_u$ ✓

$M_{cr} = S_n f_r$
$= 7.625\times12^2/6\ (267)\ \frac{1}{12} = 4071.75$

$1.3M_{cr} = 1.3\times4071.75 = 5{,}293.3 < \phi M_n$ ✓

shear check:

$V_u = 3450.3645$ lb

$\phi V_{nm} = .8(2.25) A_n \sqrt{f'_m}$

$= .8(2.25)(7.625 \times 12)\sqrt{2000} = 7,365.61$

$\phi V_{nm} > V_u$ $\quad$ $d/c = .47 < 1$ ✓

No reinforcement needed for shear

Deflection check:

$\Delta_{allow} = L/180$

$= (6 \times 12)/180 = 0.4$

$\Delta = (5wl^4/384 E I_y) + (Pl^3/48 E I_y)$

$E_m = 900 \times 2000 = 1,800,000$

$I_g = 7.625 \times 12^3/12 = 1098$

$I_y = .24\, I_g \rightarrow .24(1098) = 263.52$

$\Delta = (5(96.3)6^4(12)^3/(384 \times 1800000 \times 263.52)) + (6339.129(12 \times 6)^3/(48 \times 1.8 \times 10^6 \times 263))$

$\Delta = .0059 + 0.104 = .11 < .4$ ✓

Scanned with CamScanner

Seismic forces in Kenya comparable to Sacramento CA

Risk category II (IBC 1604.5)

Assumed site class D

From Seismicmaps.org: SS = .568 S1 = .253
SMS = .764 SDS = .509 Fa = 1.346

From ASCE 7-16 Tbl 11.4.2: Fv = 2.1

$S_{M1} = F_v S_1 \rightarrow (2.1)\,.253 = .5313$
$S_{D1} = (2/3) S_{M1} \rightarrow (2/3).5313 = .3542$

From ASCE 7-16 Tbl 12.2-1 : R = 5

From ASCE 7-16 Tbl 1.5-2 : $I_e = 1$

$V = C_s W \rho$

$C_s = SDS/(R/I_e) \rightarrow .509/5 = 0.1018$

$C_{s\,max} = S_{D1}/T(R/I)$

$T_a = C_t h_n^x \quad C_t = .02 \quad x = .75 \quad h_n = 13'$
$= (.02)13^{.75} = .137$

$C_{s\,max} = .3542/.137(5) = .52$

$C_{s\,min} = .044(S_{DS}) I_e = .044(.509) = .022$

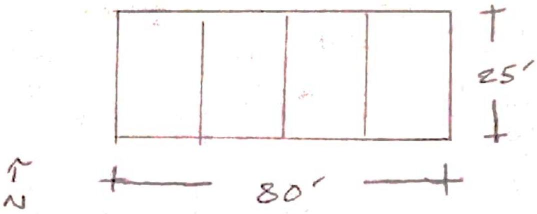
Use $C_s = .1018$

$\rho = 1.3$ (ASCE 7-16 12.3.4.2)

$V = 1.3(.1018)\,W$

$V = .132\,W$

DL (N-S) Direction: Roof (5+49) = 54 psf
Wall (115 × .67) = 78 psf

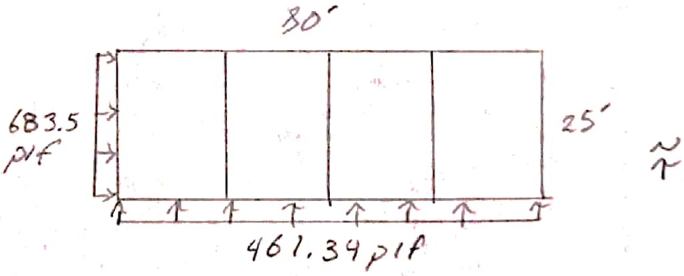


Roof: (54 × 25) = 1350 plf
Wall: (78 × 11/2) 5 = 2145 plf
3495 plf

V = .132(3495) = 461.34 plf

DL (E-W) Direction: Roof: (54 × 80) = 4320 plf
Wall1: (78 × 9/2) = 351 plf
Wall2: (78 × 13/2) = 507 plf
5178 plf

V = .132(5178) = 683.5 plf



Diaph Forces:

T/C = M/d

N/S Direction: $T/C = \frac{461.34 (80)^2}{8 (25)} = 14{,}762.88$ lbs

E/W Direction: $T/C = \frac{683.5 \times 25^2}{8 (80)} = 667.5$ lbs

N/S chords:

$T/C = 14,762.88$ lbs

$Pu < \phi P_n \rightarrow As = Pu/.9fy$ $fy = 60$ ksi

$As = 14,762.88 / .9(60000) = .273 \text{ in}^2$

Try #5 bar $As = .31 \text{ in}^2 > .273$

use #5 bars horizontally fy=60ksi for Tension

Compresion:

$Pu = \phi .8 An f'm$

$f'm = 2000$ psi

$14,762.88 = .9(.8)(An)(2000)$

$An = 10.252 \text{ in}^2$

Wall (N/S) $An = 7.625 \times (11 \times 12) = 1,1865 > 10.25$ ✓

CMU has sufficient comp strength

E/W chords:

$T/C = 667.5$ lbs

$As = Pu/.9fy \rightarrow As = 667.5/.9(60000) = .012$

Try #4 bar $As = .2 > .012$

use #4 bars horizontally fy=60ksi for Tension

Compression:

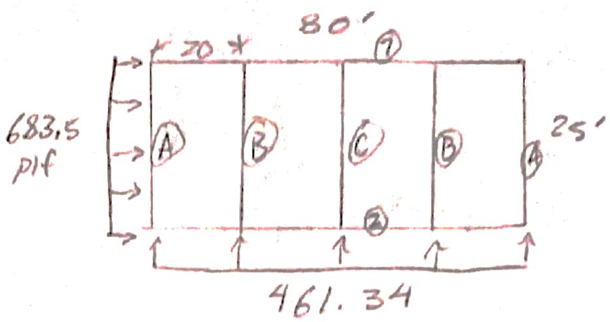
$Pu = \phi .8 f'm An \rightarrow 667.5 = .8(.9)2000(An)$

$An = .46 \text{ in}^2$

smallest BM $An = 7.625 \times 12 = 91.5 \text{ in}^2 > .46$

CMU has sufficient comp strength

Loads to shear wall:



E-W shear wall force:

$(683.5 \times 25)/2 = 8,543.75$ lbs

N-S Reactions:

Exterior walls (A) $R = .393wl$

$R = .393(461.34)20 = 3,626.13$ #s

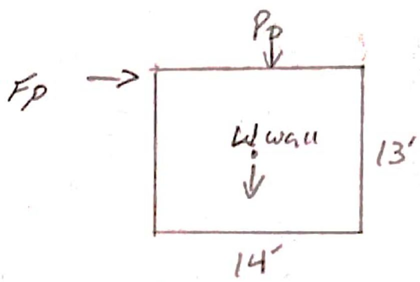
Interior (B) $R = 1.14wl$

$R = 1.14(461.34)20 = 10,518.55$ #s

mid wall (C) $R = .928wl$

$R = .928(60)461.34 = 8,562.47$ #s

Shear Wall Design: (1)



Grid line _ walls:

$F_p = 8,543.75/3 = 2,847.9$ #s

P_D: 614.45 (Bm self weight)

$5 \times 10 = 50$ plf

$P_D = 614.45 + (50/(2 \times 25.318))(25.318^2 - 2.025^2)$

$= 1243.35$ #s

$W_{wall} = 115 \times .67 \times 13 \times 14 = 14,023.1$

LC: #6 $1.2D + E_v + E_h$ $E_v = .2 S_{DS} D$ $E_h = \rho Q_E$

#7 $.9D - E_v + E_h$ $S_{DS} = .509$ $\rho = 1$

$\#6 = 1.2D + (.2 \times .509)D + Q_E$

$= 1.3D + Q_E$

$\#7 = .9D - (.2 \times .509)D + Q_E$

$= .798D + Q_E$ ← governs

$P_u = .798(1243.35) + .798(14,023.1) = 12,182.6$

$\phi P_n = P_u \rightarrow P_n = P_u/\phi$ $\phi = .9$

$P_n = 12,182.6/.9 = 13,536.3$ #s

$M_u = F_b \times h = 2,847.9(13) = 37,022.7$ lb-ft

shear wall Design continued...

$d = l - 8" \rightarrow (14 \times 12) - 8 = 160"$

$t = 13 \times 12 = 156"$ $b = 7.625"$ $f_m' = 2000\,psi$ $f_y = 60\,ksi$

$a = d - \sqrt{d^2 - [2(P_u(d - t/2)) + M_u] / \phi .8 f_m' b}$

$a = 160 - \sqrt{160^2 - [2 \times (12{,}182.6(160 - 156/2) + 37{,}022.7)] / (.9 \times .8 \times 2000 \times 7.625)}$

$a = .8$ in

$A_s = (.8 f_m' b a - P_u/\phi) / f_y$

$A_s = (.8(2000)7.625(.8) - 12{,}182.6/.9) / 60{,}000$
$= .06$

Try #4 bar $A_s = .2$

New $a = P_u + f_y A_s / .8 f_m' b \rightarrow 13536.3 + (60000 \times .2) / (.8 \times 2000 \times 7.625)$

$a = 2.09$

$C = .8 f_m' b a = .8(2000)7.625(2.09)$
$= 25{,}498$

$\phi M_n = \phi [f_y A_s (d - l_w/2) + P(l_w/2 - a/2)]$
$= .9[60(.2)(160 - 168/2) + 25.498(168/2 - 2.09/2)]$
$= 2724.467$ k-in
$= 227{,}038$ lb-ft

$\phi M_n > M_u$ ✓

ϕM_n w/ $P_u = 1.3(1243.35 + 14{,}023.1) = 19{,}846.4$
$P_n = 22{,}051.5$

$a = 22{,}051.5 + (60000 \times .2) / .8(2000)7.625$
$a = 2.8$

$C = .8(2000)(7.625)\,2.8 = 34{,}160$

$\phi M_n = .9[60(.2)(160 - 168/2) + 34.16(168/2 - 2.8/2)]$
$= 3733.6$ k-in
$= 311{,}134.67$ lb-ft $> M_u$ ✓

Moment capacity only (no axial)

C=T → $.8(2)7.625(a) = .2(60)$

$a = .98 \approx 1$ in

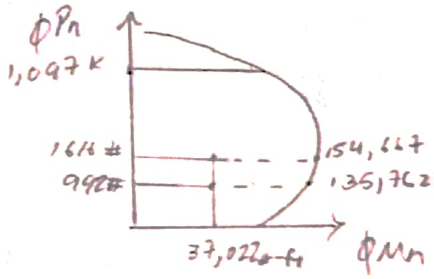
$C = .8(2000)7.625(1) = 12,200$ lbs

Axial strength: $A_n = (7.625 \times 168) = 1281\ in^2$
$A_s = \emptyset$
$r = t/\sqrt{12} = 7.625/\sqrt{12} = 2.2$

$H/r = (13 \times 12)/2.2 = 70.87 < 99$ ✓

$P_n = .8[.8(2000)(1281)][1-(70.87/140)^2]$
$= 1,219.5$ k

$\phi P_n = 1,097.6$ k



Shear Design:

$S_{max} = H/3$ or $L/3$
$= (13 \times 12)/3 = 52$ in

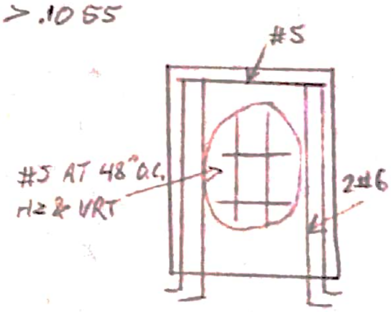
min horiz Reinf $= .0007 \times 7.625 \times 12 = .064\ in^2/ft$

Try #5 AT 48" o.c.
$A = .31/(48/12) = .0775\ in^2/ft > .064$ ✓

Min total Reif:
$.002(7.625)12 = .183\ in^2/ft$
Req Vert Reinf: $.183 - .0775 = .1055$

Try (2) #6 @ each end $4(.44)/14 = .1257 > .1055$

Shear check:
$V_n = V_m + V_s$ V_s (not required)



$V_u = 2.5(2,847.9) = 7,119.75$

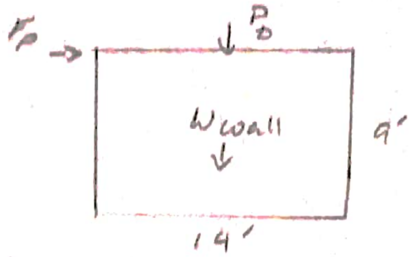
$M_u/V_u d \rightarrow (37,022.7/7119.75(160/12)) = .39$

$V_m = [4 - 1.75(.39)]1281\sqrt{2000} + .25(12,182.6) = 193,098.8$

$.4(A_n)\sqrt{f'_m} > V_m > V_u$ ✓

Ductility: $1.25(227,038) = 283,797.5$
$V_u = 283,797.5/13 = 21830.6 < V_m$ ✓

shear wall Grid line (2) walls:



$F_p = 2{,}847.9$ (previous wall calc)

P_D: 728.3 (Bm weight)
$(80/(2\times25.318))(25.318+2.025)^2 = 738.25$

$P_D = 728.3 + 738.25 = 1{,}466.55$ #

$W_{wall} = 115 \times .67 \times 14 \times 9 = 9708.3$ #

LC: #6 = 1.5D + QE
#7 = .798D + QE

using LC#7 : $P_U = .798\,(1{,}466.55 + 9708.3) = 8{,}917.5$

$\phi P_n = P_u \rightarrow P_n = P_u/\phi \qquad \phi = .9$

$P_n = 8{,}917.5/.9 = 9{,}908.4$ #s

$M_u = F_b \times h = 2{,}847.9 \times 9' = 25{,}631.1$ lb-ft

$d = L - 8" \rightarrow (14\times12) - 8 = 160$

$t = 9\times12 = 108 \qquad b = 7.625 \qquad f'_m = 2000\,psi \qquad f_y = 60\,ksi$

$a = 160 - \sqrt{160^2 - [2\times(8{,}917.5(160 - 108/2)) + 25{,}631.1)/(.9\times.8\times2000\times7.625)}$

$a = .55$

$A_s = (.8 f'_m b a - P_u/\phi)/f_y$
$= (.8(2000)7.625(.56) - 9{,}908.4)/60000$
$= .04$

Try #4 bar $A_s = .2$

New $a = P_n + f_y A_s / .8 f'_m b \rightarrow 9{,}908.4 + (60000\times.2)/(.8(2000)7.625)$

$a = 1.8$

$C = .8 f'_m b a = .8(2000)7.625(1.8)$
$C = 21{,}960$

$\phi M_n = \phi\,[f_y A_s (d - l_w/2) + P(l_w/2 - a/2)]$
$= .9\,[60(.2)(160 - 168/2) + 21.96(168/2 - 1.8/2)]$
$= 2463.2$ k-in
$= 205{,}265.7$ lb-ft $> M_u$

shear walls continued:

w/ Load Combo #6 : $P_U = 1.3(1466.55 + 9708.3) = 14,527.3$

$P_n = 14,527.3 / .9 = 16,141.5$ #s

$a = 16,141.5 + (60000 \times .2) / .8(2000)7.625$
$a = 2.3$

$C = .8(2000)\,7.625(2.3) = 28,060$

$\phi M_n = .9\left[60(.2)(160 - \frac{168}{2}) + (28.06)(168/2 - \frac{2.3}{2})\right]$

$\phi M_n = 2,913$ k-in
$= 242,757.8$ lb-ft $> M_U$ ✓

Moment capacity only (no axial):

$C = T \rightarrow .8(2)\,7.625(a) = .2(60)$

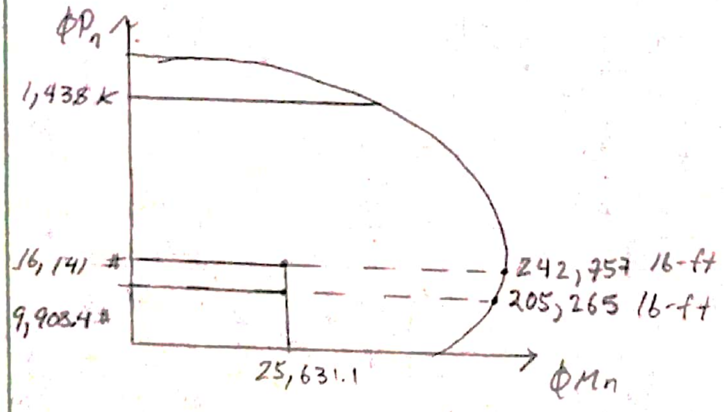
$a = .98 \approx 1$ in

$C = .8(2000)7.625(1) = 12,200$ lbs

Axial Strength : $A_n = (168 \times 7.625) = 1281$ in^2
$A_s = \phi$
$r = t/\sqrt{12} = 7.625/\sqrt{12} = 2.2$

$H/r = (9 \times 12)/2.2 = 49 < 99$ ✓

$P_n = .8[.8(2000)1281][1 - (49/140)^2]$
$= 1,438.8$ k



Shear Design:

$S_{max} = H/3$ or $L/3$ $= (9\times12)/3 = 36$ in

Min horiz Reinf: $.0007 \times 7.625 \times 12 = .064$ in²/ft

Try #4 AT 32" O.C.

$A = .2/(32/12) = .075$ in²/ft $> .064$ in²/ft ✓

min total Reinf:

$.002(7.625)12 = .183$ in²/ft

Req vert Reinf:

$.183 - .075 = .108$

Try (2) #6 @ each end : $4(.44)/14 = .1257$ in²/ft $> .108$ ✓

shear check:

$V_n = V_m + V_s$ (Vs not required)

$V_u = 2.5(F_p) \rightarrow 2.5(2847.9) = 7119.75$

$M_u/V_u d \rightarrow (25,631.1/7119.75(160/12)) = .27$

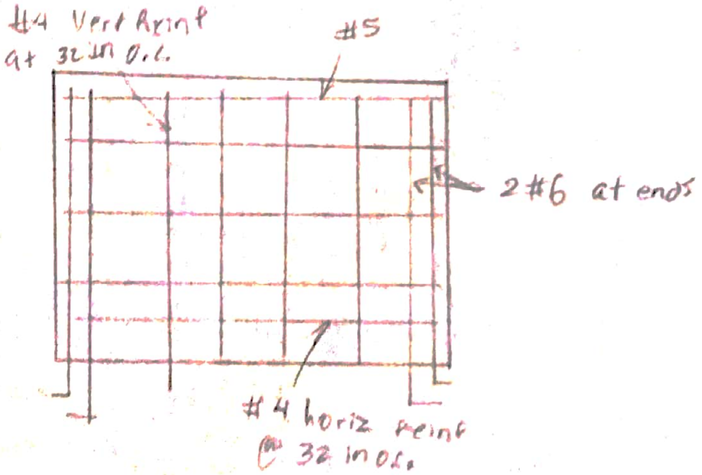
$V_m = [4 - 1.75(.27)]\ 1281\sqrt{2000} + .25(8977.5)$
$= 204,313$

$.4\sqrt{2000}(1281) > V_m > V_u$ ✓

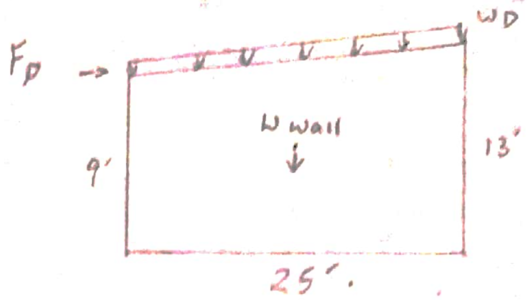
Ductility:

$1.25(205,265.7) = 256,582.125$

$V_u = 256,582.125/9 = 28,509.125 < V_m$ ✓



Shear Wall (A) Design:



$W_D = 50$ plf

$W_{wall} = 115 \times .67 \times 25.318 \times 11 = 21458.3$

$P_D = 50(25.318) + 21458.3 = 22,724.2$

$F_p = 3,626.13$ #s

using LC #7: $P_U = .798(22,724.2) = 18,133.9$

$\phi P_n = P_U \rightarrow P_n = P_U/\phi \qquad \phi = .9$

$P_n = 18,133.9/.9 = 20,148.8$ #

$M_u = 3,626.13 \times (\frac{9+13}{2}) = 39,887.43$ lb-ft

$d = (25 \times 12) - 8 = 292''$

$t = 11 \times 12 = 132'' \qquad b = 7.625 \qquad f'_m = 2000$ psi $\qquad f_y = 60$ ksi

$a = 292 - \sqrt{292^2 - [2 \times (18133.9(292 - \frac{132}{2})) + (39,887.43 \times 12) / (.9(.8)2000(7.625)}$

$a = 1.43$

$A_s = (.8 \times 2000 \times 7.625 \times 1.43 - 20,148.8)/60000$

$A_s = .05$

Try #4 bar $A_s = .2$

New $a = P_n + f_y A_s / .8 f'_m b$

$a = 20,148.8 + (60000 \times .2) / .8(2000)7.625$

$= 2.64$

$C = .8 f'_m b a \rightarrow .8(2000)7.625(2.64)$

$C = 32,208$

$\phi M_n = \phi [f_y A_s (d - \frac{l_w}{2}) + P(\frac{l_w}{2} - \frac{a}{2})]$

$= .9[60(.2)(292 - \frac{300}{2}) + 32.208(\frac{300}{2} - \frac{2.64}{2})]$

$= 6,492.7$ k-in

$= 541,057.12$ lb-ft

shear Wall (A) continued...

using LC #6 $= 1.3(22,724.2) = 29,541.5$

$P_n = 29541.5 / .9 = 32,823.9$

$a = 32,823.9 + (60000 \times .2) / .8(2000) 7.625$

$a = 3.67$

$C = .8(2000) 7.625(3.67) = 44,774$

$\phi M_n = .9 \left[60(.2)(292 - \frac{300}{2}) + 44.774 (\frac{300}{2} - \frac{3.67}{2}) \right]$

$\phi M_n = 8,337.94$ k-in

$= 694,828.3$ lb-ft

Moment Capacity only (no axial):

$C = T \rightarrow .8(2) 7.625 (a) = .2(60)$

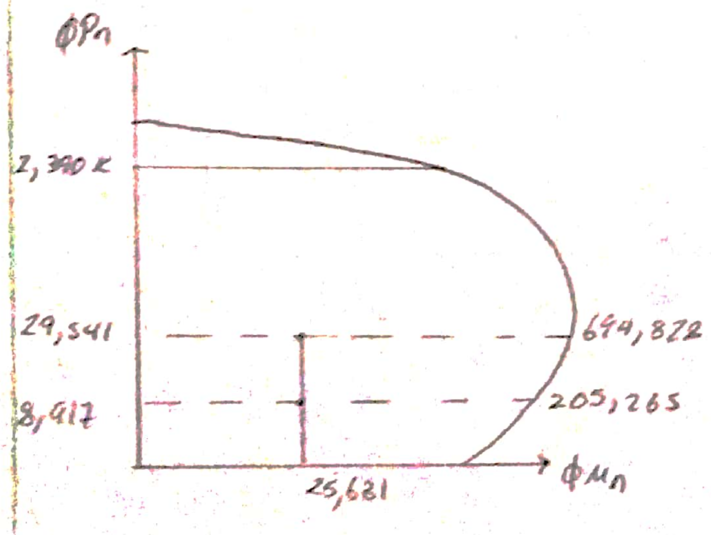
$a = .98 \approx 1$ in

$C = .8(2000) 7.625(1) = 12,200$ lbs

Axial strength: $A_n = (300 \times 7.625) = 2,287.5 \text{ in}^2$
$A_s = \phi$
$r = 7.625/\sqrt{12} = 2.2$

$H/r = (11 \times 12)/2.2 = 60 < 99$ ✓

$P_n = .8[.8(2000)2,287.5][1-(60/140)^2]$
$= 2,390.2$ k



Shear Design:

$S_{max} = H/3$ or $L/3$ → $(11 \times 12)/3 = 44$ in

Min horiz Reinf: $.007 \times 7.625 \times 12 = .064$ in²/ft

Try #5 AT 40" O.C.

$A = .31/(40/12) = .093$ in²/ft

Min total Reinf:

$.002(7.625)12 = .183$

Req Vert Reinf:

$.183 - .093 = .09$

Try (2) #7 AT each end: $(4 \times .6)/25 = .096$ in²/ft $> .09$ ✓

Shear check:

$V_n = V_m + V_s$ (V_s not required)

$V_u = 2.5(F_p) \rightarrow 2.5(3,626.13) = 9,065.33$

$M_u/V_u d \rightarrow 39,887.43 / 9,065.33(300/12) = .18$

$V_m = [4 - 1.75(.18)]\, 2,287.5 \sqrt{2000} + .25(18\,133.9)$

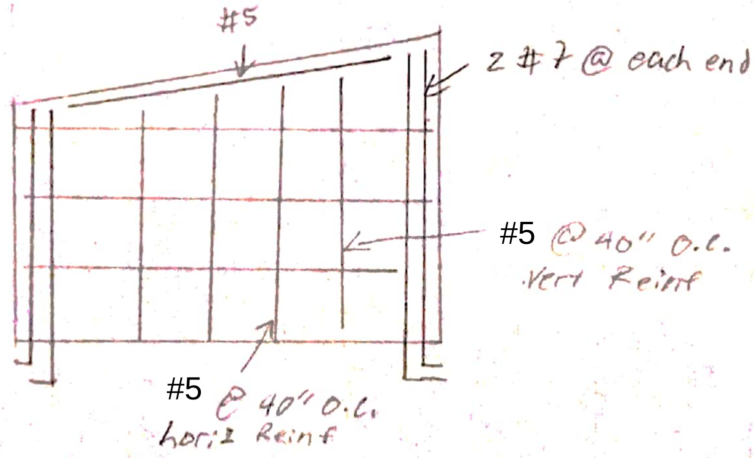
$= 381,509.4$

$4 \times \sqrt{2000} \times 2287.5 > V_m > V_u$ ✓

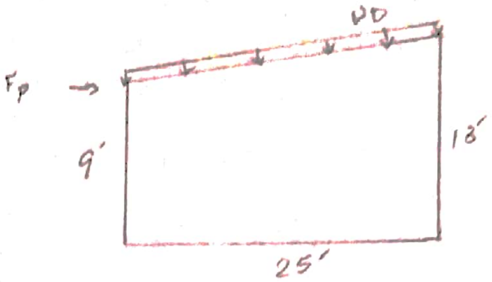
Ductility:

$1.25(541,057.12) = 676,321.4$

$V_u = 676,321.4 / 11 = 61,483.8 < V_m$ ✓



Shear Wall (B) Design:



$W_D = 50 plf$

$P_D = (50 \times 25.318) = 1265.9$ #

$W_{wall} = 115 \times .67 \times 25.318 \times 11 = 21458.3$ #

$F_p = 10,518.55$ #

using LC #7 : $P_u = .798(21458.3 + 1265.9) = 18,133.9$

$P_n = 18133.9 / .9 = 20,148.8$ #

$M_u = 10,518.55 \left(\frac{9+13}{2}\right) = 115,704$ lb-ft

$d = 292''$ $t = 132''$ $b = 7.625''$

$f'_m = 2000 psi$ $f_y = 60 ksi$

$a = 292 - \sqrt{292^2 - [2 \times (18133.9(292 - \frac{132}{2}) + (115,704 \times 12) / .9(.8)2000(7.625)}$

$a = 1.72$

$A_s = (.8 \times 2000 \times 7.625 \times 1.72 - 20,148.8)/60000$

$A_s = .014$

Try #4 bar $A_s = .2$

New $a = P_n + f_y A_s / .8(f'_m) b$

$a = 20,148.8 + (60000 \times .2) / .8(2000) 7.625$

$a = 2.64$

$C = .8 f'_m b a \rightarrow .8(2000) 7.625(2.64)$

$C = 32,208$

$\phi M_n = \phi [f_y A_s (d - \frac{l_w}{2}) + P(\frac{l_w}{2} - \frac{a}{2})]$

$= .9[60(.2)(292 - \frac{300}{2}) + 32.208(\frac{300}{2} - \frac{2.64}{2})]$

$= 541,057.12$ lb-ft

shear wall (B) continued:

Using LC #6 $= 1.3(22,724.2) = 29,541.5$

$P_n = 29541.5/.9 = 32,823.9$

$a = 32823.9 + (60000 \times .2) / .8(2000)7.625$
$= 3.67$

$C = .8(2000)7.625(3.67) = 44,774$

$\phi M_n = .9[60(.2)(292 - \frac{300}{2}) + 44.774(\frac{300}{2} - \frac{3.67}{2})]$

$\phi M_n = 694,828.3$ lb-ft

Moment capacity only (no axial):

$C = T \rightarrow .8(2)7.625(a) = .2(60)$

$a = 1$ in

$C = .8(2000)7.625(1) = 12,200$ lbs

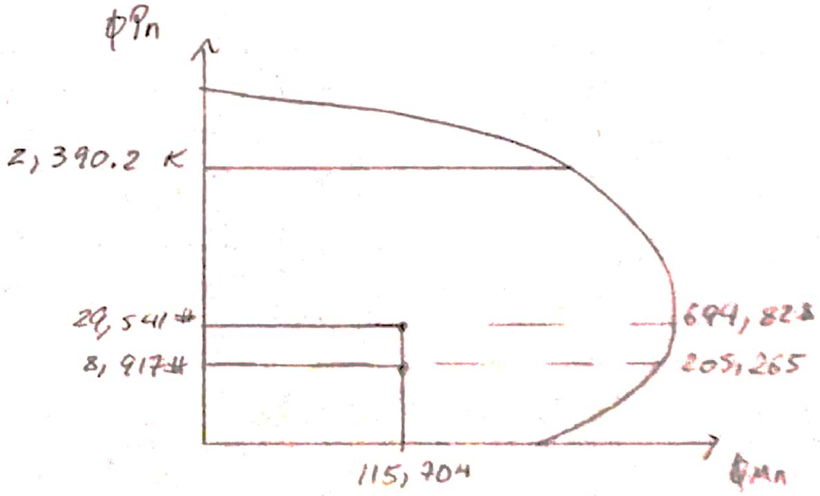
Axial strength: $A_n = 2,287.5$
$A_s = \phi$

$r = 7.625/\sqrt{12} = 2.2$

$H/r = (11 \times 12)/2.2 = 60 < 99$ ✓

$P_n = .8(.8(2000)2287.5)(1-(60/140)^2)$

$P_n = 2,390.2$ k



shear Design:

$S_{max} = H/3$ or $L/3 \rightarrow (11\times12)/3 = 44$ in

min horiz Reinf: $.007(7.625)12 = .064\ in^2/ft$

Try #5 @ 40" O.C.

$A = .31/(40/12) = .093\ in^2/ft$

min total Reinf: $.002(7.625)12 = .183$

Req vert Reinf: $.183 - .093 = .09\ in^2/ft$

Try (2) #7 at each end: $(4\times.6)/25 = .096\ in^2/ft > .09$ ✓

shear check:

$V_n = V_m + V_s$ (V_s not required)

$V_u = 2.5(F_p) \rightarrow 2.5(10,518.55) = 26,296.4$

$M_u/V_u d \rightarrow (115,704)/(26,296.4 \times (300/12)) = .18$

$V_m = [4 - 1.75(.18)]\ 2,287.5\sqrt{2000} + 2.5(18133.9)$

$V_m = 381,509.4$

$4\sqrt{2000}\ 2,287.5 > V_m > V_u$ ✓

Ductility:

$1.25(541,057.12) = 676,321.4$

$V_u = 676,321.4/11 = 61,483.8 < V_m$ ✓

Shear Wall (C) Design

$W_D = 50 plf$
$P_D = 50(25.318) = 1265.9$

$W_{wall} = 115 \times .67 \times 25.318 \times 11 = 21458.3$

$F_p = 8,562.47$ #s

Using 2C #7 : $P_u = .798(22,724.2) = 18,133.9$ #

$P_n = 18133.9 / .9 = 20,148.8$ #

$M_u = 8,562.47\left(\frac{1349}{2}\right) = 94,187.17$

$d = 292"$ $t = 132"$ $b = 7.625"$

$f'_m = 2000$ psi $f_y = 60$ ksi

$a = 292 - \sqrt{292^2 - [2 \times (18133.9(292 - \frac{132}{2}) + (94187.17 \times 12)) / .9(.8)2000(7.625)}$

$a = 1.64$

$A_s = (.8 \times 2000 \times 7.625 \times 1.64 - 20148.8) / 60000$

$A_s = .002$

Try #4 bar $A_s = .2$

New $a = 20,148.8 + (60000 \times .2) / .8(2000) 7.625$
$a = 2.64$

$C = .8(2000) 7.625 (2.64)$

$C = 32,208$

$\phi M_n = .9\left[60(.2)\left(292 - \frac{300}{2}\right) + 32.208\left(\frac{300}{2} - \frac{2.64}{2}\right)\right]$

$\phi M_n = 541,057.12$ lb-ft

Shear wall (C) continued:

using LC #6 = 1.3 (22,724.2) = 29,541.5

P_n = 29,541.5 / .9 = 32,823.9

a = 32823.9 + (60000x.2) / .8 (2000) 7,625

a = 3.67

C = .8 (2000) 7.625 (3.67) = 44,774

$\phi M_n = .9 \left[60(.2)\left(292 - \frac{300}{2}\right) + 44.774\left(\frac{300}{2} - \frac{3.67}{2}\right) \right]$

ϕM_n = 694,828.3 lb-ft

Moment capacity only (NO axial):

C = T -> .8 (2) 7.625 (a) = .2 (60)

a = 1 in

C = .8 (2000) 7.625 (1) = 12,200 lbs

Axial strength: A_n = 2,287.5
A_s = ∅

$r = 7.625 / \sqrt{12} = 2.2$

H/r = (11 x 12) / 2.2 = 60 < 99 ✓

$P_n = .8(.8(2000)\ 2287.5)(1 - (60/140)^2)$

P_n = 2,340.2 K

Shear Design:

S_{max} = H/3 or 2/3 → (11x12)/3 = 44in

Min horiz Reinf: .007(7.625)12 = .064 in²/ft

Try #5 @ 40" o.c.

A = .31/(40/12) = .093 in²/ft

Min total Reinf: .002(7.625)12 = .183

Req vert Reinf: .183 − .093 = .09 in²/ft

Try (2) #7 at each end: (4x.6)/25 = .096 in²/ft > .09 ✓

Shear check:

$V_n = V_m + V_s$ (V_s not req)

V_u = 2.5(8,562.47) = 21,406.2

$M_u/V_u d$ → 94,187.17 / 21,406.2 (300/12) = .18

$V_m = [4 - 1.75(.18)]\, 2287.5 \sqrt{2000} + 2.5(18133.4)$

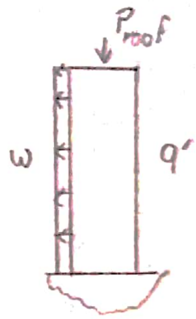
V_m = 381,509.4

$4\sqrt{2000}$ (2,287.5) > V_m > V_u ✓

Ductility: 1.25(541,057.12) = 676,321.4

V_u = 676,321.4 / 11 = 61,483.8 < V_m ✓

Out of Plane Forces on Wall (2)



$t = 7.625"$ $f_y = 60$ ksi
$W_{wall} = 78$ psf $S_{DS} = .509$

load on wall: $1.2D + (1)E$
$W_{wall} = 78(9/2) = 351$
Roof $= 105$ plf

$P_u = 351 + 105 = 456$ plf

Earthquake:

$w_u = .4 S_{DS} I W_p \rightarrow .4(.509)(1)78(1)$
$w_u = 15.88$ plf

Slender wall: $P/A < .05 f'_m$

$456/(7.625 \times 7.625) = 7.84 < .05(2000) = 100$ ✓

1st Order $M_u = M_u(oop) + M_u(ecc)$

No eccentricity $M_u(ecc) = 0$
$M_u(oop) = w_u \times L^2/8$
$= (15.88(9)^2 \times 12)/8 = 1{,}929.42$ lb-in

2nd order M_u:

$P\Delta$ estimate
$\Delta = 5 M_u h^2 / 48 E_m I_{cr}$

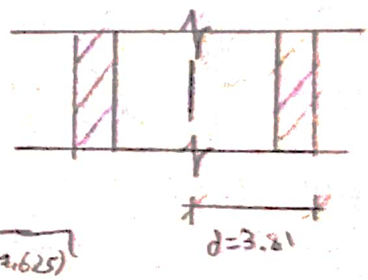
$\Delta = 5(1{,}929.42)(9 \times 12)^2 / 48(1.8 \times 10^6)14.1$
$= .09$

$I_n = bh^3/12$
$= (7.625 \times 7.625^3/12) = 281.7$
$I_{cr} = .05 I_n$
$= .05(281.7) = 14.1$
$E_m = 1{,}800{,}000$ psi

$P\Delta = .09(456) = 42.12$ lb-in

$M_u(est) = 1{,}929.42 + 42.12 = 1971.54$ lb in

Est Reinf As (flexure)



$a = d - \sqrt{d^2 - (2(P_u(d - t/2) + M_u)) / (\phi .8 f'_m b)}$

$a = 3.81 - \sqrt{3.81^2 - (2(456(3.81 - \frac{7.625}{2}) + 1971.54) / (.9(.8)2000(7.625)}$

$a = 1.95$

Estimate Steel Area:

$A_s = [.8 f'_m b a - P_u/\phi] / f_y$
$A_s = [.8(2000)\, 7.625(1.95) - 456/.9] / 60{,}000$

$A_s = .388$ per ft of wall

Try #6 bar $A_s = .44$
Spacing : $.44/.388 = 1.13' = 13.6"$

Try #6 At 12" O.C. $A_s = .44(12)/12 = .44 > .388$ ✓

$M_{cr} = (f_r + P_u/A_n)(I_n/t_2)$
$M_{cr} = (153 + 456/58.14)(281.7/3.81)$
$= 11{,}842.26$

$f_{cr} = 153$ psi
$A_n = 7.625^2 = 58.14$
$I_n = 281.7$

$M_{cr} < M_u$ (section is cracked)

Cracked section properties:

$c = A_s f_y + P_u / .64 f'_m b = (.44(60{,}000) + 456)/.64(2000)\,7.625 = 2.75$

$n = 29{,}000/1800 = 16.11$

$I_{cr} = n[A_s + P_u(t)/f_y(2d)](d-c)^2 + bc^3/3$
$= 16.11[.44 + 456(7.625)/60{,}000(7.625)](3.81 - 2.75)^2 + \frac{7.625(2.75)^3}{3}$

$I_{cr} = 53\ in^2$ compared to 14.1

$$M_u = \frac{\frac{25.88(9\times12)^2}{8} + \frac{5(11{,}892.26)\,9^2(12^2)}{48\,(1800\,000)} \times \left(\frac{1}{281.7} - \frac{1}{53}\right)}{1 - \left((5\times456\times9^2\times12^2)/48(1800000)\,53\right)} \quad 23152.9$$

$M_u = 23{,}288.16$

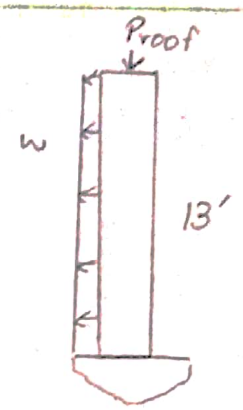
ϕM_n : $a = (.44(60{,}000) + 456/.9)/(.8(2000)\,7.625)$
$a = 2.2$

$M_n = [456/.9 + (.44 \times 60{,}000)](7.625 - 2.2/2) + (.44(60000)(3.81 - \frac{7.625}{2}))$

$M_n = 72{,}984.33$

$\phi M_n = 72{,}984.33 \times .9 = 65{,}685.9$ lb/in

Out of Plane Force on Wall ①



$t = 7.625$ in $\quad f_y = 60$ ksi
$W_{wall} = 78$ psf $\quad S_{DS} = .509$

Load on Wall:
$W_{wall} = 78(13/2) = 507$ plf
Roof $= 88.81$ plf

$P_u = 595.81$ plf

Earthquake: $w_u = .4 S_{DS} I w_p$

$w_u = .4(.509)(1)\,78(1') = 15.88$ plf

Slender Wall: $P/A < .05 f'_m$

$595.81/(7.625 \times 7.625) = 10.25 < .05(2000) = 100$ ✓

1st Order $M_u = M_{u(oop)} + M_{u(ecc)}$

No eccentricity $\quad M_{u(ecc)} = 0$

$M_{u(oop)} = w_u \times L^2/8 = (15.88 \times 13^2 \times 12)/8$
$= 4{,}025.58$ lb-in

2nd Order M_u:

$P\Delta$ estimate
$\Delta = 5 M_u h^2 / 48 E_m I_{cr}$

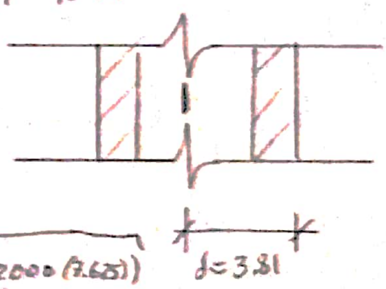
$\Delta = 5(4{,}025.58)(13 \times 12)^2 / 48 \times 1.8 \times 10^6)\,14.1$
$= .4$

$I_n = bh^3/12 = 281.7$
$I_{cr} = .05\, I_n$
$= 14.1$
$E_m = 1{,}800{,}000$ psi

$P\Delta = .4(595.81) = 238.324$ lb-in

$M_{u(est)} = 4{,}025.58 + 238.324 = 4{,}263.9$ lb-in

Est Reinf A_s (flexure)



$a = d - \sqrt{d^2 - \dfrac{2(P_u(d - \frac{t}{2}) + M_u)}{1.8\,\phi f'_m b}}$

$a = 3.81 - \sqrt{3.81^2 - (2(595.81(3.81 - \frac{7.625}{2}) + 4263.9)/(.9(1.8)2000(7.625))}$

$a = 2.07$

Estimate Steel Area:

$A_s = [.8 f'_m b a - P_u/\phi] / f_y$

$A_s = (.8(2000)7.625(2.07) - 595.81/.9) / 60{,}000$

$A_s = .41$

Try #6 bar $A_s = .44$

spacing : $.44/.41 = 1.07' \approx 12.9''$

Try #6 AT 12" O.C. $A_s = .44(12)/12 = .44 > .41$ ✓

$M_{cr} = (f_r + P_n/A_n)(I_n/t_{/2})$ $f_{cr} = 153$ psi

$= (153 + 595.81/58.14)(281.7/3.81)$ $A_n = 58.14$ in²

$= 12{,}070$ $I_n = 281.7$

Cracked Section Properties:

$c = A_s f_y + P_u / .64 f'_m b = (.44 \times 60000) + 595.81) / .64(2000) 7.625$

$c = 2.77$

$n = 29{,}000/1800 = 16.11$

$I_{cr} = n[A_s + P_u(t)/f_y(2d)](d-c)^2 + \frac{bc^3}{3}$

$= 16.11[(.44 + 595.81(7.625))/60000(7.625)](3.81-2.77)^2 + \frac{7.625(2.77)^3}{3}$

$I_{cr} = 54.2$ compared to 14.1

$$M_U = \frac{\frac{15.88(13\times12)^2}{8} + \frac{5(12{,}070)\,595.81\,(13\times12)^2}{48(1800000)} \times \left(\frac{1}{281.7} - \frac{1}{54.2}\right)}{1 - \left(\frac{5(595.81 \times 13^2 \times 12^2)}{48(1800000)54.2}\right)}$$

$M_U = 48{,}913.3$ lb-in

ϕM_n: $a = (.44(60000) + 595.81/.9) / (.8(2000)7.625)$

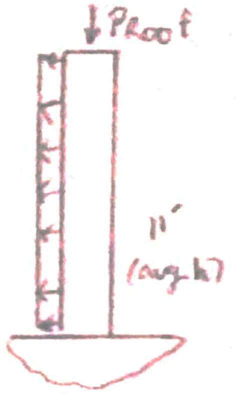
$a = 2.22$

$M_n = [595.81/.9 + (.44 \times 60000)][(7.625 - 2.22)/2]$

$M_n = 73{,}135$

$\phi M_n = 73135 \times .9 = 65{,}821.6$ lb-in

Out of Plane Force on Walls (N-S)



$t = 7.625$ in $\quad f_y = 60$ ksi
$W_{wall} = 78$ psf $\quad S_{DS} = .509$

load on Wall:
$W_{wall} = 78(11/2) = 429$ plf
Roof $= 98.85$ plf

$P_u = 429 + 98.85 = 527.85$

Earthquake: $W_u = .4 S_{DS} I w_p$

$W_u = .4(.509)(1)(78)(1') = 15.88$ plf

slender wall:
$P/A < .05 f'_m$
$527.85 / 7.625 \times 7.625 = 9.08 < 100$ ✓

1st order Mu:

$M_u(oop) = w_u L^2/8$
$= 15.88(11^2) \times 12 / 8 = 2,882.22$

2nd order Mu:

$P\Delta$ estimate
$\Delta = 5 M_u h^2 / 48 (E_m) I_{cr}$

$I_n = bh^3/12 = 281.7$
$I_{cr} = .05 I_n$
$= 14.1$
$E_m = 1,800,000$ psi

$\Delta = 5(2,882.22)(11 \times 12)^2 / 48(1800000) 14.1$
$= .206$

$P\Delta = .206(527.85) = 108.8$

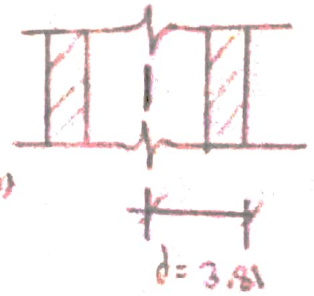
$M_u(est) = 2,882.22 + 108.8 = 2991$ lb-in

Est Reinf As (flexure):

$a = d - \sqrt{d^2 - (2(P_u(d - t/2) + M_u) / (.80 f'_m b))}$

$a = 3.81 - \sqrt{3.81^2 - (2(527.85(3.81 - 3.81) 2991) / (.8(.9) 2000 (7.625))}$

$a = .072$



Estimate steel Area:

$As = [.8 f'_m ba - P_u/\phi]/f_y$

$As = (.8(2000)7.625(1.072) - 527.85/.9)/60000$

$= .085$

Try #6 AT 12" O.C.

$As = .44(12)/12 = .44 > .005$ ✓

$M_{cr} = (f_r + P_n/A_n)(I_n/t_n)$

$= (153 + 527.85/58.14)(281.7/3.81)$

$= 11,983.6$

$f_{cr} = 153$ psi
$A_n = 58.14$
$I_n = 281.7$

Cracked section properties

$c = As f_y + P_u / .64 f'_m b = (.44 \times 60000) + 527.85 / .64(2000)7.625$

$c = 2.76$

$n = 16.11$

$I_{cr} = n(As + P_u(t)/f_y(2d))(d-c)^2 + bc^3/3$

$= 16.11[(.44 + 527.86(7.625))/60000(7.625)](3.81 - 2.76)^2 + \frac{7.625(2.76)^3}{3}$

$I_{cr} = 53.6$

$$M_u = \frac{\frac{15.88(11\times12)^2}{8} + \frac{5(11,983.6)527.85(11\times12)^2}{48(1800000)} \times \left(\frac{1}{281.7} - \frac{1}{53.6}\right)}{1 - (5(527.85 \times 11^2 \times 12^2)/48(1800000)53.6}$$

$M_u = 34,836.22$

ϕM_n : $a = [.44(60000)] + (527.85/.9) / (.8 \times 2000 \times 7.625)$

$a = 2.212$

$M_n = [527.85/.9 + (.44 \times 60000)][(7.625 - 2.212/2]$

$M_n = 73,038.96$

$\phi M_n = 65,735$

shear check:

$V_u = 3450.3645$ lb

$\phi V_{nm} = .8(2.25) A_n \sqrt{f'_m}$
$= .8(2.25)(7.625 \times 12)\sqrt{2000} = 7,365.61$

$\phi V_{nm} > V_u$ $\quad d/c = .47 < 1$ ✓

No reinforcement needed for shear

Deflection check:

$\Delta_{allow} = L/180$
$= (6 \times 12)/180 = 0.4$

$\Delta = (5wl^4/384E_mI_y) + (Pl^3/48E_mI_y)$

$E_m = 900 \times 2000 = 1,800,000$

$I_g = 7.625 \times 12^3/12 = 1098$

$I_y = .24\ I_g \rightarrow .24(1098) = 263.52$

$\Delta = (5(96.3)6^4(12)^3/(384 \times 1800000 \times 263.52)) + (6339.129(12 \times 6)^3/(48 \times 1.8 \times 10^6 \times 263))$

$\Delta = .0059 + 0.104 = .11 < .4$ ✓

Wall Continous Foundation:

• Wall (1): W wall = (115×.67×13) = 1,001.65
Roof loads = (5,394.1 × 9)/80 = 606.8

P(per foot) = 1608.5 plf

W = 1608.5/3000 = .54' ≈ 6.4"

use 9" wide continous foundation

• Wall (2): Wwall = (115×.67×9) = 693.5 plf
Roof loads = (6,339.13×9)/80 = 713.15 plf

P(per foot) = 1406.7 plf

W = 1406.7/3000 = .47' ≈ 5.6"

use 9" wide continous foundation

• Walls N/S : (based on int wall)

Wwall = (115×.67×11) = 847.55 plf
Roof loads = 380+(1.2×48.85) = 438.62 plf

P(per foot) = 1280.17 plf

W = 1280.17/3000 = .43' ≈ 5"

use 9" wide continous foundation

Connections:

• Metal Deck to HSS Bm

T/C = 14,762.88 lbs (Diaph forces)

14,762.88/25 = 590.5 plf

ARC SPOT WELD 3,4 capacity = 783 lbs
(From Verco catalog 2014 pg 14)

USE ARC SPOT WELD 3,4 AT 12" O.C.

Bm To CMU:

(based on highest load) : 5,610 lbs

Plate thickness: $\phi P_n \geq P_u$

Assume A36 PL $f'_c = 3ksi$ $\phi = .65$

$.65(.85(3)A_1) \geq 5.61$

$A_1 = 3.38\ in^2$

PL As large As Bm Dimensions $b \times h = 43.6\ in^2$

Try 7"x7" PL $A = 49\ in^2 > 43.6$

$M = 7 - .95(6.6)/2 = 3.865$
$n = 7 - .8(6.6)/2 = 4.36$
$\lambda = 1$ (assume)
$n' = \frac{1}{4}\sqrt{6.6 \times 6.6} = 1.65$
$\ell = 4.36$

$t = 4.36\sqrt{2(5.61)/.9 \times 7^2 \times 36}$ -> $t = .36" \approx .5"$

use 7"x½"x7" (A36) PL

min weld leg (per AISC M): PL = ½" min leg = 3/16" < 7/16" ✓

max weld leg: ½" − 1/16" = 7/16"

weld capacity: $\phi R_n = 1.392 \times 7 = 9.74\ k/in$

USE 7/16" Fillet welds

Anchor bolts:

USE (2) SSTB16 5/8" diam Anchor bolt (Simpson)
capacity = 2x2865 = 5730 lbs

GENERAL

1. ALL NEW CONSTRUCTION SHALL COMPLY WITH THE CONTRACT DOCUMENTS AND THE CURRENT EDITION OF THE IBC AND ACI.

2. THESE GENERAL NOTES SUPERSEDE THE REQUIREMENTS OF THE PROJECT SPECIFICATIONS. IN CASE OF CONFLICT BETWEEN THE PLANS AND SPECIFICATIONS, CONTACT THE OWNER'S REPRESENTATIVE.

3. REFERENCE TO CODES, RULES, REGULATIONS, STANDARDS, MANUFACTURER'S INSTRUCTIONS OR REQUIREMENTS OF REGULATORY AGENCIES IS TO THE LATEST PRINTED EDITION OF EACH IN EFFECT AT THE DATE OF SUBMISSION OF BID UNLESS THE DOCUMENT DATE IS SHOWN.

4. TYPICAL DETAILS AND GENERAL NOTES APPLY TO ALL PARTS OF THE WORK EXCEPT WHERE SPECIFICALLY DETAILED OR UNLESS NOTED OTHERWISE (U.N.O.)

5. THE STRUCTURAL DRAWINGS ILLUSTRATE THE NEW STRUCTURAL MEMBERS. REFER TO ARCHITECTURAL, MECHANICAL AND ELECTRICAL DRAWINGS FOR NON-STRUCTURAL ITEMS WHICH REQUIRE SPECIAL PROVISIONS DURING THE CONSTRUCTION OF THE STRUCTURAL MEMBERS.

6. REFER TO ARCHITECTURAL DRAWINGS FOR FLOOR DEPRESSIONS, EDGE OF SLAB, OPENINGS, SLOPES, DRAINS, CURBS, PADS, EMBEDDED ITEMS, NON-BEARING PARTITIONS, ETC. REFER TO MECHANICAL AND ELECTRICAL DRAWINGS FOR SLEEVES, OPENINGS, AND HANGERS FOR PIPES, DUCTS AND EQUIPMENT.

7. THE CONTRACTOR SHALL VERIFY AND BE RESPONSIBLE FOR COORDINATING THE WORK OF ALL TRADES AND SHALL VERIFY ALL DIMENSIONS AND CONDITIONS WHICH IMPACT THE WORK. FIELD VERIFY SIZES, ELEVATIONS, HOLE LOCATIONS, ETC. PRIOR TO FABRICATION.

8. DRAWING DIMENSIONS ARE TO FACE OF FINISH, JOINT CENTERLINE OR COLUMN GRID CENTERLINE UNLESS NOTED OTHERWISE. DO NOT SCALE THE DRAWINGS.

9. CONTRACTOR SHALL CAREFULLY REVIEW THE DRAWINGS TO IDENTIFY THE SCOPE OF WORK REQUIRED. VISIT THE SITE TO RELATE THE SCOPE OF WORK TO EXISTING CONDITIONS AND DETERMINE THE EXTENT TO WHICH THOSE CONDITIONS AND PHYSICAL SURROUNDINGS WILL IMPACT THE WORK.

10. EXISTING CONDITIONS AS SHOWN ON THESE PLANS ARE FOR REFERENCE ONLY. CONTRACTOR IS REQUIRED TO FIELD VERIFY ALL EXISTING CONDITIONS PRIOR TO CONSTRUCTION. CONTRACTOR SHALL REPORT CONDITIONS THAT CONFLICT WITH THE CONTRACT DOCUMENTS TO THE OWNER'S REPRESENTATIVE. DO NOT DEVIATE FROM THE CONTRACT DOCUMENTS WITHOUT WRITTEN DIRECION FROM THE OWNER'S REPRESENTATIVE.

11. THE CONTRACTOR SHALL RESOLVE ANY CONFLICTS ON THE DRAWINGS OR IN THE SPECIFICATIONS WITH THE OWNER'S REPRESENTATIVE BEFORE PROCEEDING WITH THE WORK.

12. ANY DEVIATION, MODIFICATION & SUBSTITUTION FROM THE APPROVED SET OF STRUCTURAL DRAWINGS SHALL BE SUBMITTED TO THE OWNER'S REPRESENTATIVE FOR REVIEW/APPROVAL PRIOR TO ITS USE OR INCLUSION ON THE SHOP DRAWINGS & PRIOR TO PROCEEDING WITH THE WORK.

13. THE CONTRACTOR SHALL PROVIDE ALL NECESSARY SHORES, BRACES AND GUYS REQUIRED TO SUPPORT ALL LOADS TO WHICH THE BUILDING STRUCTURE AND COMPONENTS, SOILS, OTHER STRUCTURES AND UTILITIES MAY BE SUBJECTED DURING CONSTRUCTION. SHORING SYSTEMS SHALL BE DESIGNED AND STAMPED BY A CIVIL ENGINEER LICENSED IN THE STATE OF CALIFORNIA. VISITS TO THE SITE BY THE OWNER'S REPRESENTATIVE WILL NOT INCLUDE OBSERVATION OF THE ABOVE NOTED ITEMS.

14. THE CONTRACTOR SHALL PROVIDE MEANS, METHOD, TECHNIQUES, SEQUENCE AND PROCEDURE OF CONSTRUCTION AS REQUIRED. SITE VISITS PERFORMED BY THE OWNER'S REPRESENTATIVE DO NOT INCLUDE INSPECTIONS OF MEANS AND METHODS OF CONSTRUCTION PERFORMED BY CONTRACTOR.

15. THE CONTRACTOR SHALL PROTECT ALL WORK, MATERIALS AND EQUIPMENT FROM DAMAGE AND SHALL PROVIDE PROPER STORAGE FACILITIES FOR MATERIALS AND EQUIPMENT DURING CONSTRUCTION.

16. STRUCTURAL OBSERVATIONS PERFORMED BY ENGINEER DURING CONSTRUCTION ARE NOT THE CONTINUOUS AND SPECIAL INSPECTION SERVICES AND DO NOT WAIVE THE RESPONSIBILITY FOR THE INSPECTIONS REQUIRED OF THE BUILDING INSPECTOR OR THE DEPUTY INSPECTOR. OBSERVATIONS ALSO DO NOT GUARANTEE CONTRACTOR'S PERFORMANCE AND SHALL NOT BE CONSIDERED AS SUPERVISION OF CONSTRUCTION.

17. CONTRACTORS SHALL REVIEW SHOP DRAWINGS FOR COMPLETENESS AND COMPLIANCE WITH CONTRACT DOCUMENTS. CONTRACTOR SHALL STAMP SHOP DRAWINGS PRIOR TO SUBMISSION TO OWNER'S REPRESENTATIVE.

18. REVIEW OF THE SHOP DRAWINGS SHALL NOT BE CONSTRUED AS AN AUTHORIZATION TO DEVIATE FROM CONTRACT DOCUMENTS.

19. SHOP DRAWINGS WILL NOT BE PROCESSED DUE TO INCOMPLETENESS, LACK OF CO-ORDINATION WITH RELEVANT PORTION OF CONTRACT DOCUMENTS, LACK OF CALCULATIONS IF REQUIRED AND WHERE DEVIATIONS, MODIFICATIONS AND SUBSTITUTIONS ARE INDICATED WITHOUT PRIOR WRITTEN APPROVAL FROM OWNER'S REPRESENTATIVE.

20. ALLOW FOURTEEN WORKING DAYS FOR PROCESSING SHOP DRAWINGS AFTER RECEIPT.

CONCRETE

1. CONCRETE IS REINFORCED AND CAST-IN-PLACE UNLESS OTHERWISE NOTED. WHERE REINFORCING IS NOT SPECIFICALLY SHOWN OR WHERE DETAILS ARE NOT GIVEN, PROVIDE REINFORCING SIMILAR TO THAT SHOWN FOR SIMILAR CONDITIONS, SUBJECT TO REVIEW BY THE OWNER'S REPRESENTATIVE.

2. ALL STRUCTURAL CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH AT 28 DAYS AS FOLLOWS:

SLAB ON GRADE	4000 PSI NORMAL WEIGHT
ALL OTHER CONCRETE	4000 PSI NORMAL WEIGHT

3. ALL STRUCTURAL CONCRETE MIXES SHALL BE TYPE II CEMENT AND SHALL BE DESIGNED BY AN APPROVED LABORATORY.

4. NORMAL WEIGHT CONCRETE AGGREGATES SHALL CONFORM TO ASTM C-33. LIGHT WEIGHT CONCRETE AGGREGATES SHALL CONFORM TO ASTM C-330.

5. NO MORE THAN ONE GRADE OF CONCRETE SHALL BE ON THE JOB SITE AT ANY ONE TIME.

6. THOROUGHLY CLEAN AND ROUGHEN ALL HARDENED CONCRETE AND MASONRY SURFACES TO RECEIVE NEW CONCRETE. INTERFACE SHALL BE ROUGHENED TO A FULL AMPLITUDE OF 1/4" UNLESS NOTED OTHERWISE.

7. KEY AND DOWEL POUR JOINTS AS SHOWN ON THE PLANS. ANY DEVIATION FROM POUR JOINTS SHOWN ON THE PLANS MUST BE APPROVED BY THE OWNER'S REPRESENTATIVE.

8. DEFECTIVE CONCRETE (VOIDS, ROCK POCKETS, HONEYCOMBS, CRACKING, ETC.) SHALL BE REMOVED AND REPLACED AS DIRECTED BY THE OWNER'S REPRESENTATIVE.

REINFORCEMENT

1. REINFORCING TO CONFORM TO THE FOLLOWING, UNLESS OTHERWISE NOTED:

LOCATION	TYPE
REINFORCING STEEL U.N.O.	ASTM A615, 60 KSI
REINFORCING STEEL TO BE WELDED AND IN CONCRETE SHEAR WALL BOUNDARY ELEMENTS	ASTM A706, 60 KSI
WELDED STEEL WIRE FABRIC	ASTM A185, 70 KSI
SMOOTH DOWELS IN SLAB ON GRADE	ASTM A36, 36 KSI

2. REINFORCING BARS SHALL HAVE THE FOLLOWING MINIMUM COVERAGE. PLACE BARS AS NEAR TO THE CONCRETE SURFACE AS THESE MINIMUMS PERMIT WHEREVER POSSIBLE UNLESS NOTED OTHERWISE:

	MIN. CONCRETE COVER
CONCRETE POURED AGAINST EARTH	3"
FORMED CONCRETE IN CONTACT WITH EARTH	2"
EXPOSED TO WEATHER (#6 AND LARGER)	2"
EXPOSED TO WEATHER (#5 AND SMALLER)	1.5"
SLABS & WALLS NOT EXPOSED TO WEATHER	3/4"

3. #5 AND LARGER REINFORCING BARS SHALL NOT BE SPLICED EXCEPT AS LOCATED AND DETAILED ON THE DRAWINGS. #4 AND SMALLER BARS WITH LENGTH NOT SHOWN SHALL BE CONTINUOUS, LAPPING 1'-6" MINIMUM IN CONCRETE (SEE TYPICAL DETAILS). HORIZONTAL WALL SPLICES SHALL BE STAGGERED. VERTICAL BARS SHALL NOT BE SPLICED EXCEPT AT HORIZONTAL SUPPORT, SUCH AS FLOOR OR ROOF, UNLESS DETAILED OTHERWISE. ALL BARS ENDING AT THE FACE OF A WALL, COLUMN, OR BEAM SHALL EXTEND TO WITHIN 2" OF THE FAR FACE AND HAVE A 90 DEGREE HOOK UNLESS OTHERWISE SHOWN.

4. BARS SHALL BE FIRMLY SUPPORTED AND ACCURATELY PLACED AS REQUIRED BY THE A.C.I. STANDARDS, USING TIE AND SUPPORT BARS IN ADDITION TO REINFORCEMENT SHOWN WHERE NECESSARY FOR FIRM AND ACCURATE PLACING. ALL DOWELS SHALL BE ACCURATELY SET IN PLACE BEFORE PLACING CONCRETE.

5. DRAWINGS SHOW TYPICAL REINFORCING CONDITIONS. CONTRACTOR SHALL PREPARE DETAILED PLACEMENT DRAWINGS OF ALL CONDITIONS SHOWING QUANTITY, SPACING, SIZE, CLEARANCES, LAPS, INTERSECTIONS AND COVERAGE REQUIRED BY STRUCTURAL DETAILS, APPLICABLE CODE AND TRADE STANDARDS. CONTRACTOR SHALL NOTIFY REINFORCING INSPECTOR OF ANY ADJUSTMENTS FROM TYPICAL CONDITIONS THAT ARE PROPOSED IN PLACEMENT DRAWINGS TO FACILITATE FIELD PLACEMENT OF REINFORCING STEEL AND CONCRETE.

6. NO WELDING OF REINFORCEMENT (INCLUDING TACK WELDING) SHALL BE DONE UNLESS SHOWN ON THE DRAWINGS. WHERE SHOWN ON THE DRAWINGS, WELDING OF REINFORCING STEEL SHALL BE PERFORMED BY WELDERS SPECIFICALLY CERTIFIED FOR REINFORCING STEEL. USE E90XX ELECTRODES.

CONCRETE
- MASONRY

A. CONCRETE MASONRY TO BE SUPPLIED PER 2018 CBC SECTION 2105.22 AND PLACED PER SECTION 2104.
B. ASSEMBLY STRENGTH F'm - 2000 psi AT 28 DAYS.
C. UNITS: MEDIUM WEIGHT 2 CELL BLOCKS CONFORMING TO ASTM C90. SHRINKAGE OF BLOCKS SHALL NOT EXCEED 0.065% WHEN TESTED PER ASTM C426.
D. MORTAR: ASTM C270, TYPE M.
E. GROUT: ASTM C476. COMPRESSIVE STRENGTH AS REQUIRED TO ATTAIN SPECIFIED ASSEMBLY STRENGTH. ALL CELLS SHALL BE FULLY GROUTED.
F. USE LOW LIFT CONSTRUCTION WITH MAXIMUM GROUT POUR HEIGHT OF 4 FT. HIGH LIFT GROUTING IS ACCEPTABLE IF APPROVED IN WRITING BY THE ENGINEER.
G. ALL MASONRY TO BE REINFORCED UNLESS SPECIFICALLY MARKED "NOT REINFORCED".
H. SEE PLAN FOR LOCATIONS OF VERTICAL CONTROL JOINTS. HORIZONTAL BOND BEAM AND LINTEL REINFORCING SHALL BE CONTINUOUS ACROSS VERTICAL CONTROL JOINTS.
I. ALL CELLS, SHALL BE GROUTED SOLID. REINFORCING STEEL SHALL BE SECURED IN PLACE PRIOR TO GROUTING.
J. MASONRY BUILDING WALLS HAVE BEEN DESIGNED TO SPAN VERTICALLY AS SIMPLE SPANS FROM FLOOR TO ROOF AND ARE DEPENDENT UPON THE COMPLETED ROOF STRUCTURE AND COMPLETION OF ALL MASONRY WALLS FOR STABILITY AND FOR RESISTANCE TO WIND AND SEISMIC FORCES. THE CONTRACTOR IS SOLELY RESPONSIBLE FOR PROVIDING ALL NECESSARY TEMPORARY BRACING AS REQUIRED FOR CONSTRUCTION LOADS, FOR STABILITY AND FOR RESISTANCE TO WIND AND SEISMIC FORCES UNTIL THE ENTIRE STRUCTURE IS COMPLETE. THE SHORING SHALL NOT RELY ON ANY MOMENT RESISTANCE CAPACITY OF THE FOOTINGS.

INSPECTION / TESTING

1. CONTINUOUS SPECIAL INSPECTION IS REQUIRED PER IBC. CONTINUOUS SPECIAL INSPECTION IS REQUIRED FOR THE FOLLOWING WORK AS DESCRIBED IN IBC:

1.1 ALL CONCRETE WORK
1.2 BOLTS INSTALLED IN CONCRETE
1.3 EXPANSION & ADHESIVE ANCHORS
1.4 REINFORCING STEEL AND PRESTRESSING STEEL TENDONS
1.5 WELDING OF REINFORCING STEEL
1.6 JUST PRIOR TO PLACING CONCRETE FOUNDATIONS TO ENSURE SUBGRADE IS SUITABLE, FREE FROM LOOSE SOIL, AND FOUNDATIONS ARE OF PROPER DIMENSIONS

SEAL:

DRAWINGS TO BE REVIEWED BY IN-COUNTRY ARCHITECT AND ENGINEER
DRAWINGS NOT FOR CONSTRUCTION

PROJECT: Take Heart School in Kenya

SITE: Migori County, Kenya

REVISIONS

No.	DESC.	DATE

DRAWN BY:

CHECKED BY:

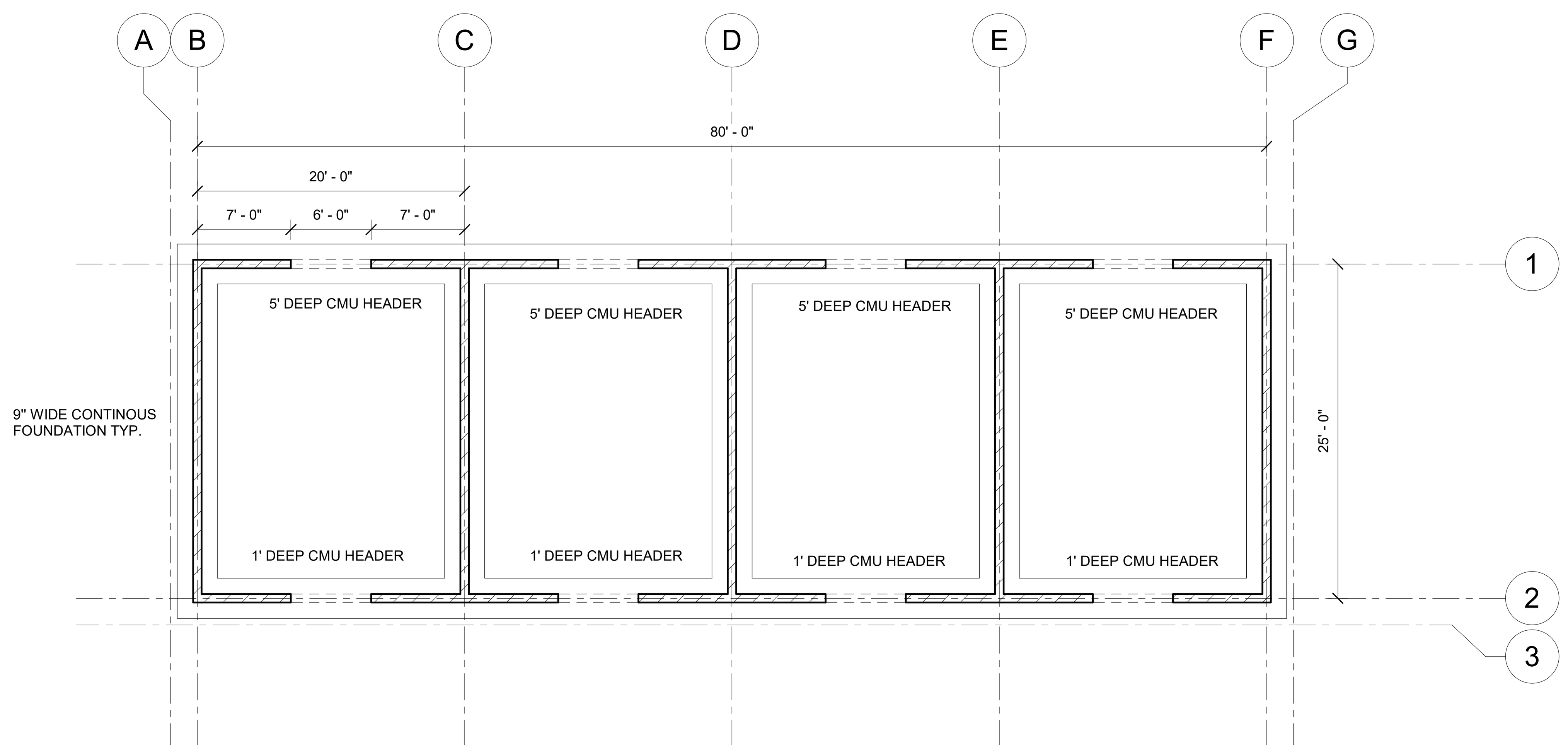
PLOT DATE: 6/10/2021 8:15:27 PM

SHEET NAME:

SCALE:

SHEET No.:

NOTES:
1. SEE GENERAL NOTES
2. SEE SHEETS S.8 FOR WALL AND HEADER DETAIL AND SCHEDULE



1 Level 1
1/8" = 1'-0"

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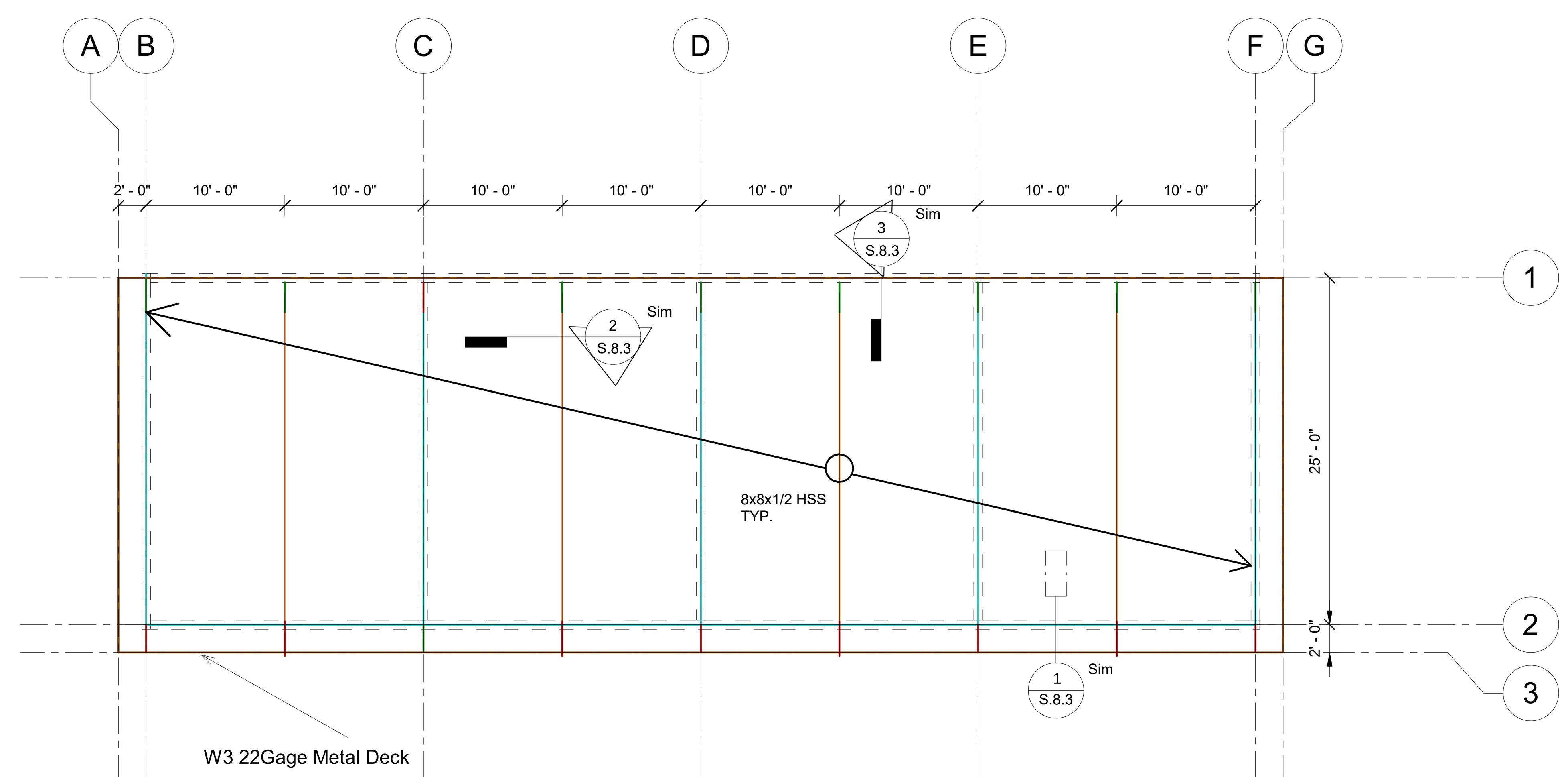

1 Level 2
1/8" = 1'-0"

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SITE:

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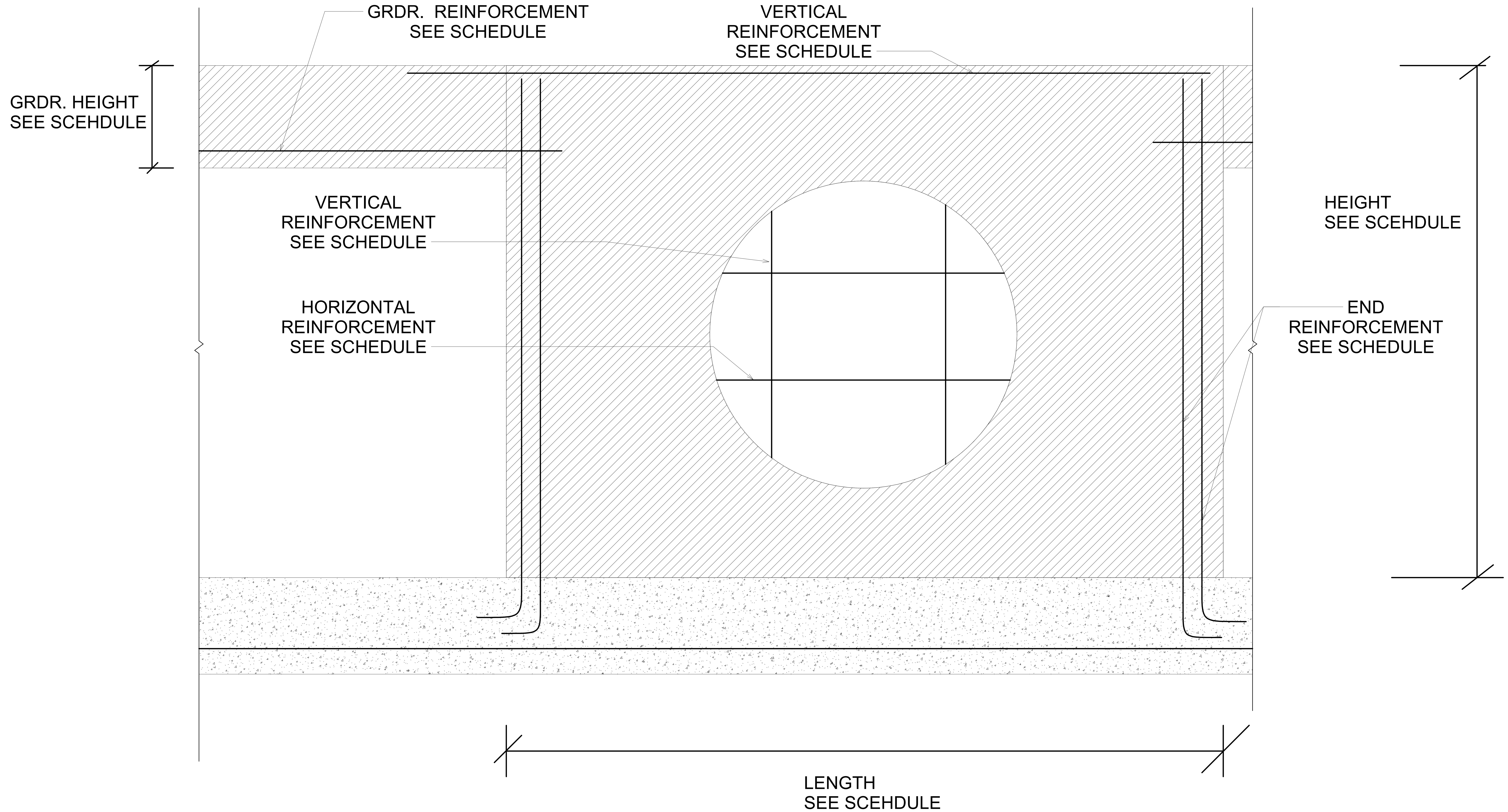

1 E/W Wall
1" = 1'-0"

E/W SHEAR WALL SHEDULE						
Grid Line	Height	Length	Vert. Reinf.	Horiz. Reinf.	Top Reinf.	End Reinf.
1	13'	14'	#5 at 48"O.C.	#5 at 48"O.C.	#5	(2) #6
2	9'	14'	#4 at 32"O.C.	#4 at 32"O.C.	#5	(2) #6

HEADER SHEDULE			
Grid Line	Height	Length	Horiz. Reinf.
1	5'	6'	#7
2	1'	6'	#6

2 E/W Shedule
1" = 1'-0"

JOURNEYMAN INTERNATIONAL

SEAL:

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SITE:

Migori County, Kenya

REVISIONS

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PLOT DATE:

6/10/2021 8:15:28 PM

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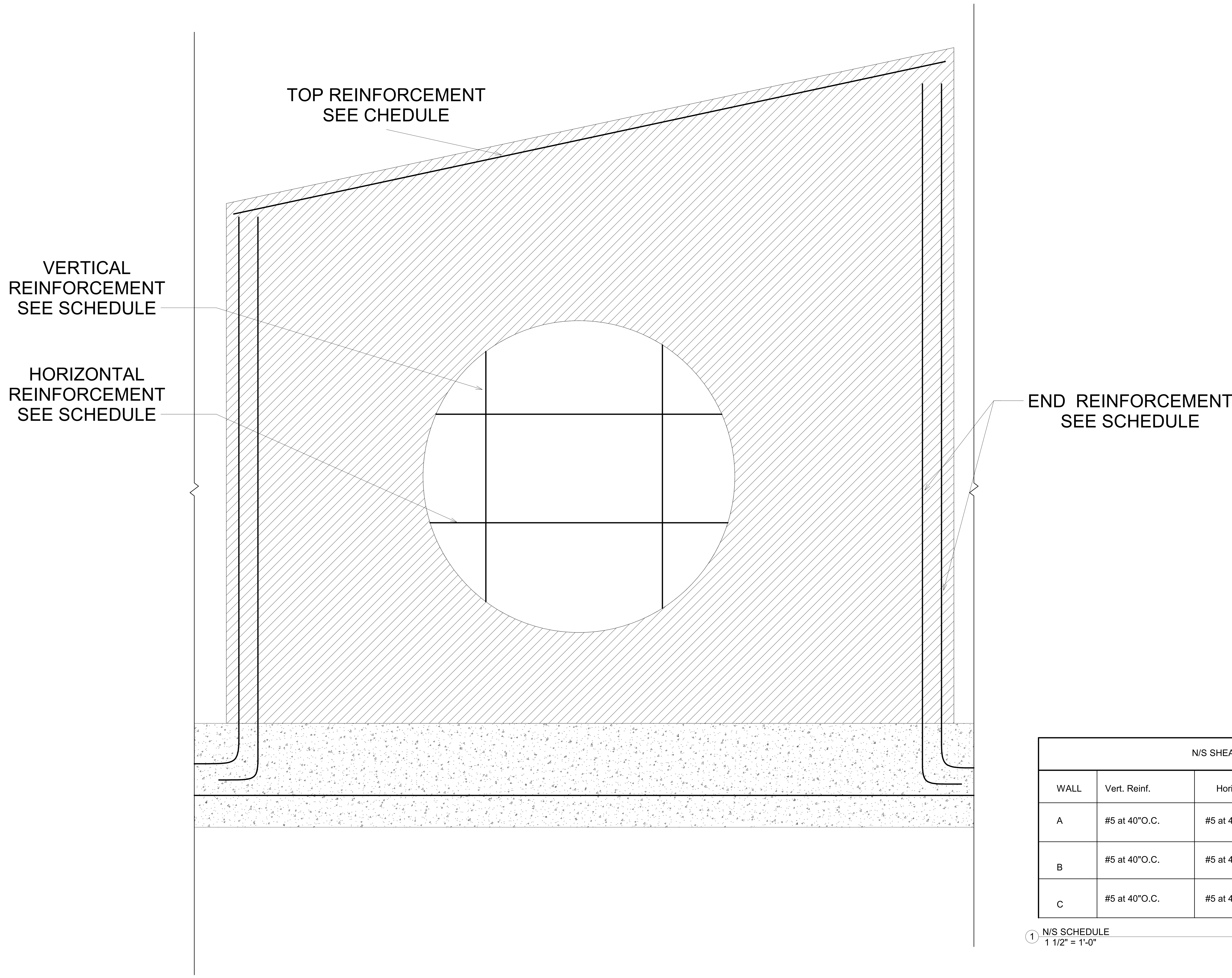
SHEET No.:



2 N/S Wall
1" = 1'-0"

N/S SHEAR WALL SHEDULE				
WALL	Vert. Reinf.	Horiz. Reinf.	Top Reinf.	End Reinf.
A	#5 at 40"O.C.	#5 at 40"O.C.	#5	(2) #7
B	#5 at 40"O.C.	#5 at 40"O.C.	#5	(2) #7
C	#5 at 40"O.C.	#5 at 40"O.C.	#5	(2) #7

1 N/S SCHEDULE
1 1/2" = 1'-0"

SEAL:

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DRAWINGS NOT FOR CONSTRUCTION

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SITE: Migori County, Kenya

REVISIONS

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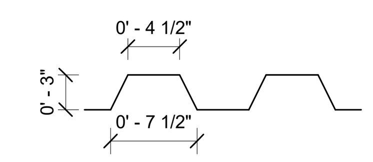
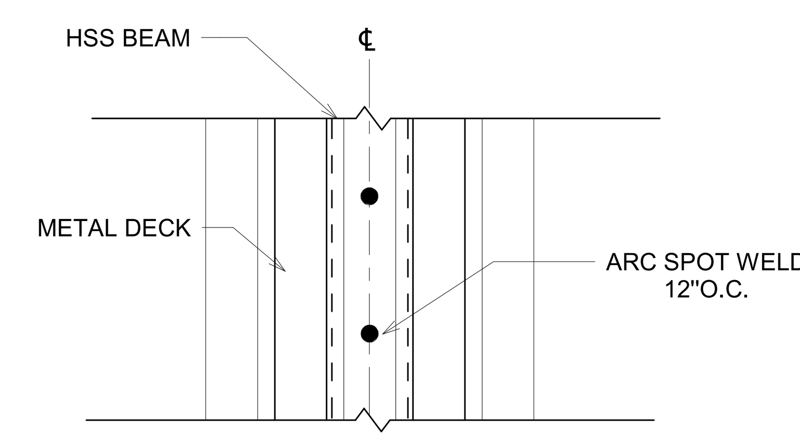
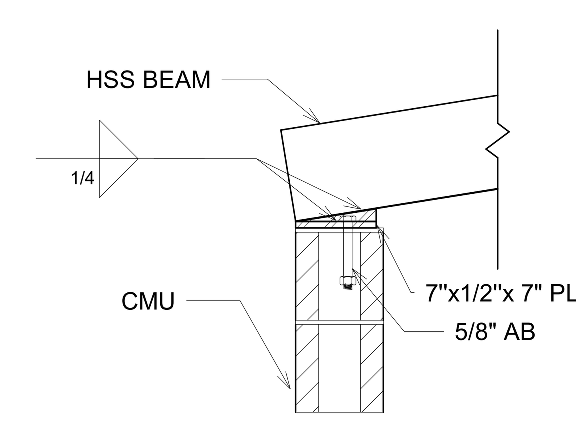


1 TYP. METAL DECK SECTION
1" = 1'-0"



2 METAL DECK TO BEAM
1" = 1'-0"



3 BEAM TO CMU WALL
1" = 1'-0"

SEAL:

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DRAWINGS NOT FOR CONSTRUCTION

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LOADS

Roof:

Metal Deck (3W 22gage) ……………………………… 1.9psf ~ 2psf

MISC ………………………………………………… 3psf

Gravity Members:

8x8x 1/2 HSS Beam weight ………………………… 48.85 plf

6x6x1/2 HSS GRDR ………………………………… 35.24 plf

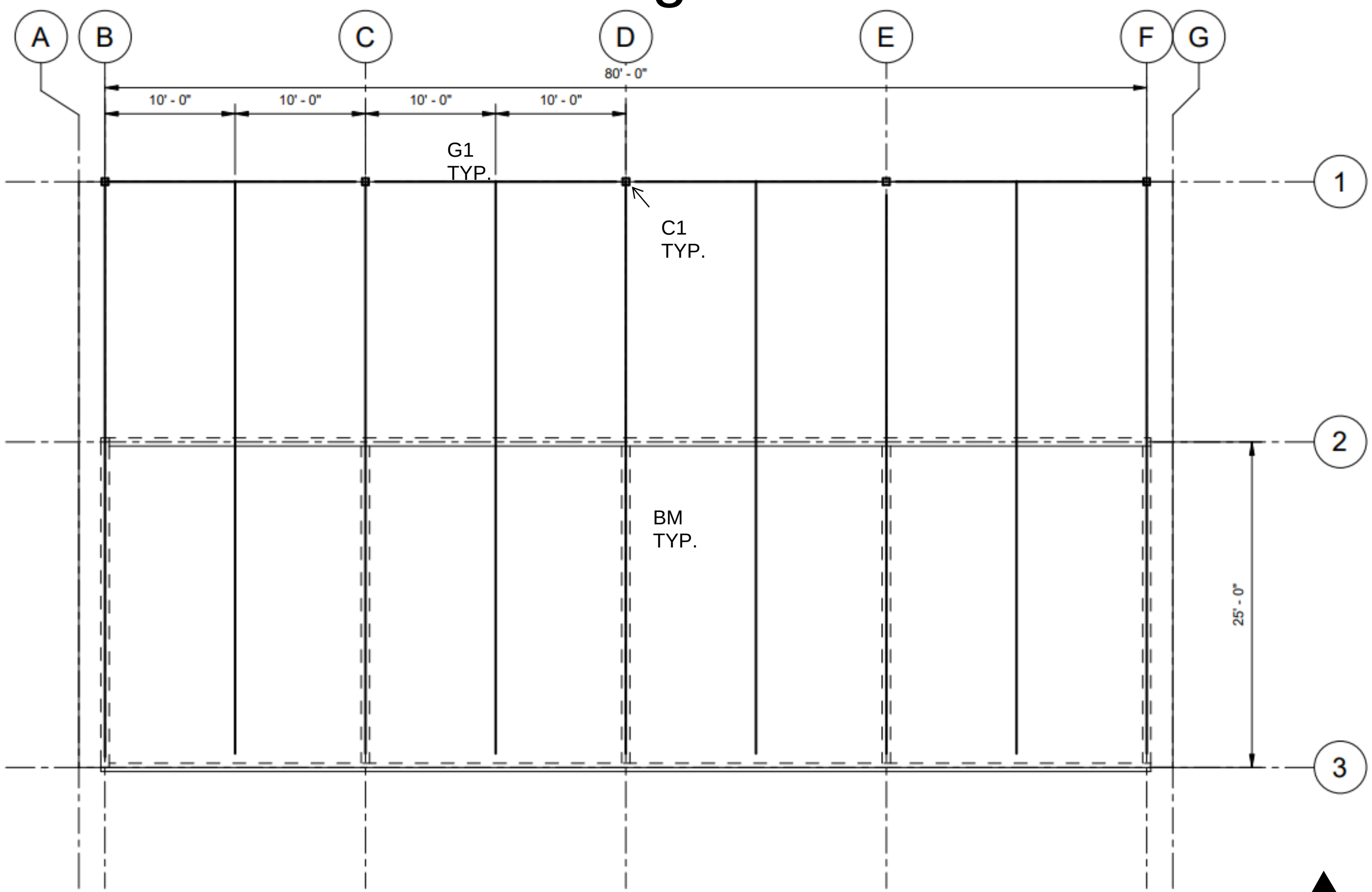
6x6x3/8 HSS Col. …………………………………… 27.48 plf

Wall:

Fully grouted CMU ………………………………… 115 pcf

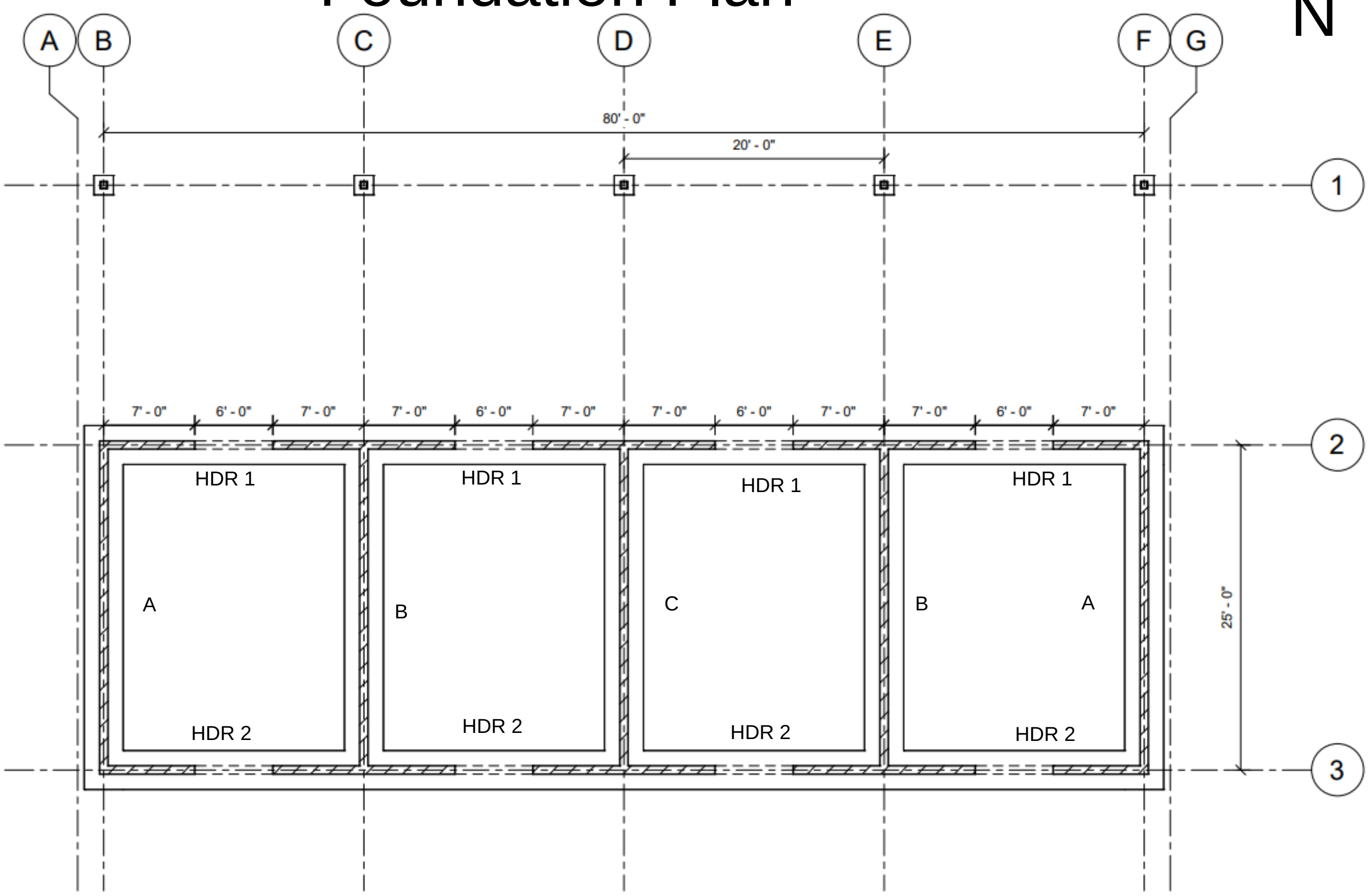
Key Plan

Framing Plan

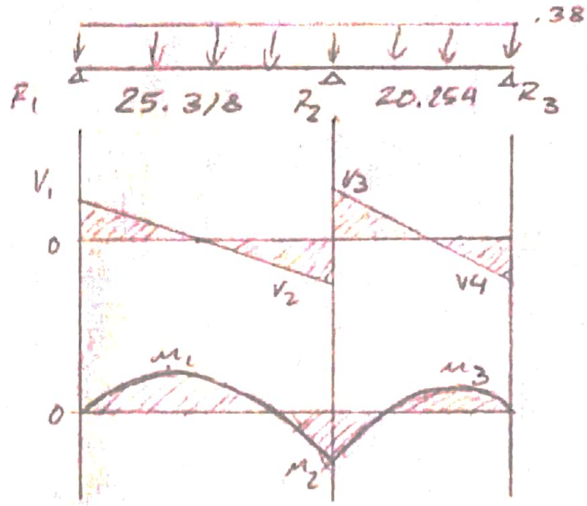


N

Foundation Plan



BM



Length: tan(9.09) × 45 = 7.2'

$L = \sqrt{7.2^2 + 45^2} = 45.57$

$L_1 = 25.318$

$L_2 = 45.57 - 25.318 = 20.254$

$R_1 = .4\,wl = .4(380 \times 25) = 3800$

$R_2 = 1.1\,wl = 1.1(380 \times 25) = 10450$

$R_3 = .4\,wl = .4(380 \times 20) = 3040$

$V_1 = 3.8\,K$

$V_2 = 3800 - (380 \times 25.318) = -5820.84$

$V_3 = 10450 - 5820.84 = 4629.16$

$V_4 = 4629.16 - (20.254 \times 380) = -340$

$M_1 = .08\,wl^2 = 19$ k-ft

$M_2 = .1\,wl^2 = 23.75$ k-ft

$M_3 = .08\,wl^2 = 12.16$ k-ft

Zrequired = (23,750×12)/(.9×50000)
= 6.33

Try 8×8×½ HSS   Z = 37.5 in³

Deflection check: L/180

(25.318 × 12)/180 = 1.69"

From Risa model Δ = .6 < 1.69" ✓   d/c = .36 < 1 ✓

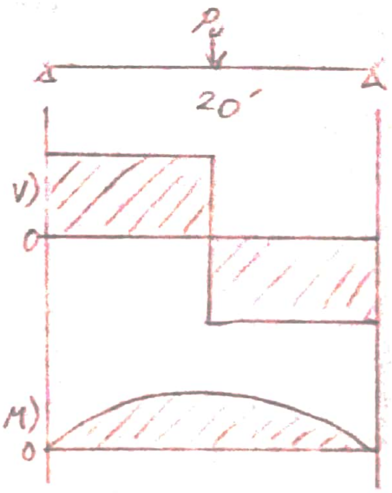
$\phi M_n = 141$ k-ft > M_u (Tbl 3-13)

$\phi V_n = .6\,F_y\,A_w\,C_{v2}$: $F_y = 50$ ksi $C_{v2} = 14.2$

$A_w = (.465 \times 6.603) = 3.07$

$\phi V_n = .6(50)\,3.07(14.2)$
$= 1307.8$ k > V_u ✓

Girder (G1) Design:



P = 3040 lbs + BM (weight)

BM Weight = .4(48.85 x 20.254) = 396 #s

Pu = 3040 + 396 = 3436 #s

Vu = 3436/2 = 1718 #s

Mu = 3436(20)/4 = 17,180 lb-ft

Zreq = M/φfy

φ = .9   fy = 50 ksi

Zreq = (17,180 x 12)/(.9 x 50000) = 4.58

Try HSS 6x6x½   Z = 7.7 > 4.58 ✓

Tbl 3-13

φMn = 74.3 k-ft > 17.18 k-ft ✓

φVn = .6 Fy Aw Cv2

Fy = 50 ksi   Cv2 = 9.9   Aw = 2(.465 x 4.6) = 4.28

φVn = .6(50) 9.9 (4.28) = 1271.16 k > 1.7 k ✓

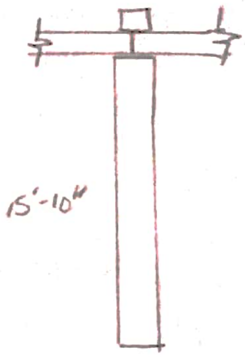
Deflection check:

L/180 -> (20x12)/180 = 1.33"

Δ = Pl³/48EI   E = 29,000 ksi   I = 48.3 in⁴

$$\Delta = \frac{(3436)(20)^3(12)^3}{48(29000)(48.3)} = 0.7" < 1.33" \checkmark$$

Column Design: (based on interior col) (C1)



GRDR Reactions: 2(1718) = 3436#

BM Reaction: 3436#

GRDR (weight) = 2 [(35.24 x 20)/2] = 352.4 #

$P_u = 2(3436) + 1.2(352.4) = 7294.88$ #s

$KL/r < 200$

$L = 15'-10"$ $K = 1$ (Tbl C-A-7.1)

$KL = (1)\,15.833 = 15.833$

using Tbl 4-4:

Try HSS 6x6x3/8 @ 16' $\phi P_n = 203K$

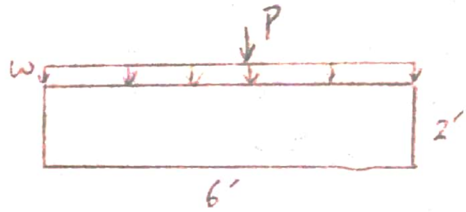
$r = 13.2$

$(15.833 \times 12)/13.2 = 14.4 < 200$ ✓

$\phi P_n > P_u$ & no buckling

use HSS 6x6x3/8

CMU HDR 1



$P = 3800 + (.4 \times 25.318 \times 48.85) = 4294.7$

$w: (115 \times .67) = 77.05 \approx 78\,psf$

$w = (78 \times 2) \times 1.2 = 187.2\,plf$

$V = (4294.7/2) + (187.2 \times 6)/2 = 2708.95$

$M = (4294.7 \times 6)/4 + (187.2 \times 6^2)/8 = 7284.45$

Flexure check:

$\rho_{max} = .64 \frac{f_m'}{f_y} \left(\frac{.0025}{1.5\ f_y/E_s + .0025} \right)$

$f_m' = 2000\,psi \quad f_y = 60\,ksi$
$E = 29000\,ksi$

$\rho_{max} = .64 \left(\frac{2}{60}\right) \left(\frac{.0025}{1.5(60/29000) + .0025} \right) = .0095$

$A_s\ Req = M_U / \phi f_y (.8d)$

$d = \sqrt{M_U / .8 \phi f_y \rho b}$

$d = \sqrt{(7289.45 \times 12) / .8(.9)(60000)\,7.625(.0095)}$

$d = 5.29$

$h = (2 \times 12)$ Try $d = 24 - 4'' = 20''$

$A_s\ Req = (7284.45 \times 12) / (.9 \times 60000 \times .8 \times 20) = .1$

Try # 5 bar $A_s = .31$

$\rho = .31 / (7.625 \times 20) = .002 < .0095$ ✓

$\phi M_n = \phi f_y A_s \left(d - \frac{f_y A_s}{1.6 f_m' b}\right)$

$\phi M_n = .9(60000)\,.31 \left(20 - \frac{60000 \times .31}{1.6(2000)\,7.625}\right) \frac{1}{12}$
$= 26{,}836.6$

$\phi M_n > M_u$ ✓

$M_{cr} = S_n f_r$
$= (7.625 \times 24^2)/6\ (267)\ \frac{1}{12} = 16{,}287$

$1.3 M_{cr} = 1.3 \times 16287 = 21{,}173 < \phi M_n$ ✓

shear check:

$V_u = 2708.98$

$\phi V_{nm} = .8(2.25) A_n \sqrt{f'_m}$
$= .8(2.25)(7.625 \times 24)\sqrt{2000} = 14,731.2$

$\phi V_{nm} > V_u$ $d/c = .18 < 1$ ✓

No reinf needed for shear

Deflection check:

$\Delta_{allow} = L/180$
$(6 \times 12)/180 = 0.4"$

$\Delta = (5wl^4 / 384 E_m I_y) + (Pl^3 / 48 E_m I_y)$

$E_m = 900 f'_m = 1,800,000$

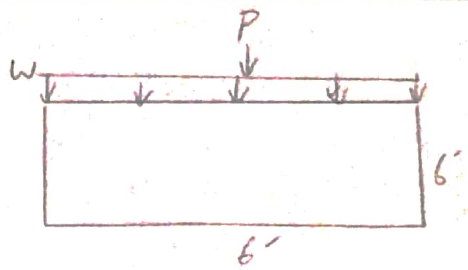
$I_g = (7.625 \times 24^3)/12 = 8784$

$I_y = .24 I_g$
$= .24(8784) = 2108.16$

$\Delta = 5(181.2)(6 \times 12)^4 / 384(1800000)2108.16 + 4294.7(6 \times 12)^3 / 48(1800000)2108$

$\Delta = .0014 + .0088 = .01 < .4$ ✓

CMU HDR 2



$P = 10{,}450 + (1.1 \times 48.85 \times 25.318) = 11810.5$

$w: (115 \times .67) = 77.05 \approx 78\,psf$

$w = (78 \times 6) \times 1.2 = 561.6\,plf$

$V = (11810.5/2) + (561.6 \times 6)/2 = 7590.05$

$M = (11810.5 \times 6)/4 + (561.6 \times 6^2)/8 = 20{,}242.95$

flexure check:

$\rho_{max} = .64 \frac{f'_m}{f_y} \left(\frac{.0025}{1.5(f_y/E_s) + .0025} \right)$ $f'_m = 2000\,psi$ $f_y = 60\,ksi$ $E_s = 29000$

$\rho_{max} = .64 \left(\frac{2}{60}\right) \left(\frac{.0025}{1.5(60/29000) + .0025} \right) = .0095$

$d = \sqrt{Mu/.8\phi f_y \rho b}$

$d = \sqrt{(20242.95 \times 12)/.8(.9)60{,}000(.0095)7.625}$

$d = 8.8''$

$h = (6 \times 12) = 72''$ Try $d = 72'' - 4'' = 68''$

$A_s\,Req = Mu/\phi f_y (.8d)$

$A_s\,Req = (20242.95 \times 12)/.9(60000)(.8)68 = .08$

Try (2) #6 bar $A_s = 2 \times .44 = .88$

$\rho = .88/(7.625 \times 68) = .0017 < \rho_{max}$ ✓

$\phi M_n = \phi f_y A_s \left(d - \frac{f_y A_s}{1.6 f'_m b}\right)$

$\phi M_n = .9(60000)\,.88 \left(68 - \left(\frac{60000 \times .88}{1.6(2000)7.625}\right)\right) \frac{1}{12}$

$= 260{,}710.8$

$\phi M_n > M_u$ ✓

$M_{cr} = S_n f_r$

$= (7.625 \times 72^2)/6\ (267)\left(\frac{1}{12}\right) = 146{,}583$

$1.3 M_{cr} = 1.3 \times 146{,}583 = 190{,}557.9 < \phi M_n$ ✓

Shear check:

$V_u = 7,590.05$

$\phi V_{nm} = .8(2.25) A_n \sqrt{f'_m}$
$= .8(2.25)(7.625 \times 72)\sqrt{2000} = 44,193.65$

$\phi V_{nm} > V_u$

No reinf is needed for shear

Deflection check:

$\Delta_{allow} = L/180$
$(6 \times 12)/180 = .4''$

$\Delta = (5wl^4/384 E_m I_y) + (Pl^3/48 E_m I_y)$

$E_m = 900 f'_m = 1,800,000$

$I_g = (7.625 \times 72^3)/12 = 237,168$

$I_y = .24 I_g$
$= .24(237,168) = 56,920.32$

$\Delta = [5(561.6)6^4(12^3)/384(1800000)56920.32]$
$+ 11810.5(6 \times 12)^3/48(1800000)56920.32$

$\Delta = .00016 + .000896 = .001 < .4$ ✓

Seismic forces in Kenya comparable to Sacramento CA

Risk category II (IBC 1604.5)

Assumed site class D

From Seismicmaps.org: $S_S = .568$ $S_1 = .253$
$S_{MS} = .764$ $S_{DS} = .509$ $F_a = 1.346$

From ASCE 7-16 Tbl 11.4.2: $F_V = 2.1$

$S_{M1} = F_V S_1 \rightarrow (2.1)\,.253 = .5313$
$S_{D1} = (2/3) S_{M1} \rightarrow (2/3).5313 = .3542$

From ASCE 7-16 Tbl 12.2-1 : $R = 5$

From ASCE 7-16 Tbl 1.5-2 : $I_e = 1$

$V = C_s W \rho$

$C_s = S_{DS}/(R/I_e) \rightarrow .509/5 = 0.1018$

$C_{s\,max} = S_{D1}/T(R/I)$

$T_a = C_t h_n^x$ $C_t = .02$ $x = .75$ $h_n = 13'$
$= (.02)13^{.75} = .137$

$C_{s\,max} = .3542/.137(5) = .52$

$C_{s\,min} = .044(S_{DS})I_e = .044(.509) = .022$

Use $C_s = .1018$

$\rho = 1.3$ (ASCE 7-16 12.3.4.2)

$V = 1.3(.1018)W$

$V = .132\ W$

DL (N-S) Direction: Roof (5+49) = 54 psf

Wall (115x.67) = 78 psf

Roof: (54x25) = 1350 plf

Wall: (78x $\frac{12}{2}$)5 = 2340 plf

3690 plf

V = .132 (3690) = 487.1 plf

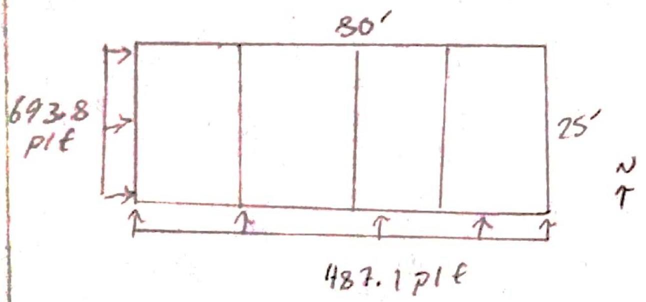
DL (E-W) Direction: Roof (54x80) = 4320 plf

Wall 1 (78x $\frac{10}{2}$) = 390 plf

Wall 2 (78x $\frac{14}{2}$) = 546 plf

5256 plf

V = .132 (5256) = 693.8 plf



Diaph Forces: T/C = M/d

N/S Direction:

$T/C = 487.1(80)^2 / 8(25) = 15,587.2$ lbs

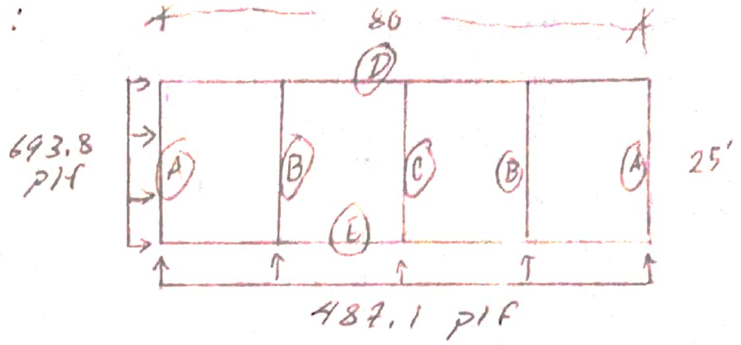
E/W Direction:

$T/C = 693.8 \times 25^2 / 8(80) = 677.6$ lbs

Loads to shear wall:

E/W shear wall force:

(693.8x25)/2 = 8,672.5 #s



N/S Reactions:

Wall (A) R = .393 wl

R = .393(487.1)20 = 3,828.6 #s

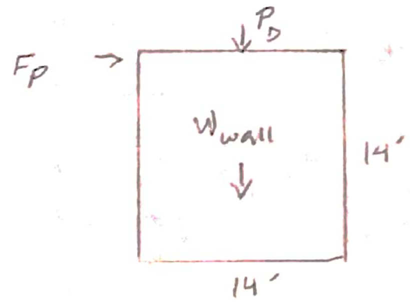
Wall (B) R = 1.14 wl

R = 1.14(487.1)(20) = 11,105.9 #s

Wall (C) R = .928 wl

R = .928(487.1)20 = 9,040.6 #s

Shear wall (D) Design:



F_p = 8,672.5 #s

P_D: 1360.5 (Bm weight)

Dead load = 50 plf → (50 x25)1.1 = 1375 #

P_D = 1375+1360.5 = 2735.5 #

W_{wall} = 115x.67 x 14x14 = 15,101.8 #

LC #6 = 1.2D + Ev + Eh $E_v = .2 S_{DS} D$ $E_h = \rho Q_E$

LC #7 = .9D + Ev + Eh S_{DS} = .509 $\rho = 1$

LC #6 = 1.3D + Q_E

LC #7 = .798 D + Q_E ← governs

P_u = (15,101.8 + 2735.5).798 = 14,234.2

$\phi P_n = P_u$ → $P_n = P_u/\phi$ ϕ = .9

P_n = 14,234.2/.9 = 15,815.7 #s

M_u = Fb x h

M_u = 8,672.5(14) = 121,415 lb-ft

Shear wall (D) Design continued...

$d = L - 8 \rightarrow (14 \times 12) - 8 = 160''$

$L = (14 \times 12) = 168''$ $b = 7.625''$ $f'_m = 2000 psi$ $f_y = 60 ksi$

$a = d - \sqrt{d^2 - (2[P_u(d - \frac{L}{2}) + M_u) / \phi .8 f'_m b}$

$a = 160 - \sqrt{160^2 - (2(14,234.2(160 - \frac{168}{2}) + (121,415 \times 12) / .9(.8)2000(7.625)}$

$a = 1.45$ in

$A_s = (.8) f'_m ba - P_n / f_y$

$A_s = .03$

Try #4 bar $A_s = .2$

New $a = P_n + f_y A_s / .8 f'_m b \rightarrow 15,815.7 + (60000 \times .2) / .8(2000)(7.625)$

$a = 2.3$

$C = .8 f'_m ba \rightarrow .8(2000)\,7.625\,(2.3) = 28,060$

$\phi M_n = \phi[f_y A_s (d - \frac{L_w}{2}) + P(\frac{L_w}{2} - \frac{a}{2})]$

$= .9[60(.2)(160 - \frac{168}{2}) + 28.06(\frac{168}{2} - \frac{2.3}{2})]$

$\phi M_n = 2,913$ k-in $\rightarrow$ 242,757.8 lb-ft $\phi M_n > M_u$ ✓

using LC #6:

$P_u = 1.3(15101.8 + 2735.5) = 23,188.5$

$P_n = 23188.5 / .9 = 25,765$

$a = 25,765 + (60000 \times .2) / .8(2000)\,7.625$

$a = 3$

$C = .8(2000)\,7.625\,(3) = 36,600$

$\phi M_n = .9[60(.2)(160 - \frac{168}{2}) + 36.6\,(168/2 - \frac{3}{2})]$

$= 3,931.5$ k-in

$\phi M_n = 327,625$ lb-ft $> M_u$ ✓

Moment Capacity only (no axial):

$C = T \rightarrow .8(2)7.625(a) = .2(60)$

$a = 1$ in

$C = .8(2000)7.625(1) = 12,200$ #s

Axial strength: $A_n = (7.625 \times 168) = 1281$ in^2
$A_s = \phi$

$r = t/\sqrt{12} = 7.625/\sqrt{12} = 2.2$

$H/r = (14 \times 12)/2.2 = 76.4 < 99$ ✓

$P_n = .8[.8(2000)1281][1-(76.4/140)^2]$

$P_n = 1,151.4$ k

$\phi P_n = 1,036.2$ k

Shear Design:

$S_{max} = H/3$ or $L/3$
$= (14 \times 12)/3 = 56$ in

Min horiz Reinf $= .007 \times 7.625 \times 12 = .064$ in^2/ft

Try #5 AT 56" O.C. $A = .31/(56/12) = .066$ in^2/ft $> .064$ ✓

Min Total Reinf $= .002(7.625)12 = .183$ in^2/ft

Req Vert Reinf $= .183 - .066 = .117$ in^2/ft

try (2) #6 @ each end : $A = 4(.44)/14 = .1257 > .117$ ✓

Shear check: $V_n = V_m + V_s$ (V_s not required)

$V_u = 2.5(8,672.5) = 21,681.25$

$M_u/V_u d \rightarrow 121,415/21,681.25(168/12) = .42$

$V_m = [4-1.75(.42)]1281\sqrt{2000} + .25(14,234.2) = 190,604$

$4\sqrt{2000}(1281) > V_m > V_u$ ✓

Ductility: $1.25(242,757.8) = 303,447$

$V_u = 303,447/14 = 21,674.8 < V_m$

Shear Wall (E) Design:

$F_p = 8,672.5$ #s

P_D: Bm (weight) = 994.7
Dead Load = (50×25).4 = 500

$P_D = 994.7$

$W_{wall} = .67 \times 115 \times 14 \times 10 = 10,787$

Using LC#7: $P_U = .78(10,787 + 994.7) = 9401.8$

$\phi P_n \rightarrow P_U \rightarrow P_n = P_U/\phi \qquad \phi = .9$

$P_n = 9401.8/.9 = 10,446.4$

$M_U = F_p \times h = (8,672.5 \times 10) = 86,725$ lb-ft

$d = L\text{-}8'' = 160'' \qquad t = 120'' \qquad b = 7.625$

$f_y = 60$ ksi $\qquad f'_m = 2000$ psi

$a = 160 - \sqrt{160^2 - [2 \times (9401.8(160 - \frac{120}{2}) + (86,725 \times 12)) / (.8 \times .9 \times 2000 \times 7.625)]}$

$a = 1.13$

$A_s = (.8 f'_m b a) - P_n / f_y$

$= .8(2000)7.625(1.13) - 10,446.4 / 60000$
$= .06$

Try #4 bar $A_s = .2$

new $a = P_n + f_y A_s / .8 f'_m b \rightarrow 10,446.4 + (60000 \times .2)/(.8(2000)7.625)$

$a = 1.8$

$C = .8(2000)7.625(1.8) = 21,960$

$\phi M_n = \phi [f_y A_s (d - \frac{l_w}{2}) + P(\frac{l_w}{2} - \frac{a}{2})]$

$\phi M_n = .9[60(.2)(160 - \frac{168}{2}) + 21.96(\frac{168}{2} - \frac{1.8}{2})]$

$= 205,265.7$ lb-ft > M_U ✓

Shear Wall (E) Design continued:

using LC #6 : $P_u = 1.3(10{,}787 + 999.7) = 15{,}316.21$

$P_n = 15316.21/.9 = 17018$

$a = 17018 + (60000 \times .2)/(.8 \times 2000 \times 7.625)$
$= 2.4$

$C = .8(2000)7.625(2.4) = 29{,}280$

$\phi M_n = .9\left[60(.2)(160 - \frac{168}{2}) + 29.28(\frac{168}{2} - \frac{2.4}{2})\right]$

$\phi M_n = 3{,}002.7$ k-in → 250,228.8 lb-ft

Moment Capacity only (no axial):

$C = T$ → $.8(2)7.625(a) = .2(60)$

$a = 1$ in

$C = .8(2000)7.625(1) = 12{,}200$ lbs

Axial strength : $A_n = (168 \times 7.625) = 1281$ in^2
$A_s = \phi$

$r = t/\sqrt{12}$ → $7.625/\sqrt{12} = 2.2$

$H/r = (10 \times 12)/2.2 = 54.5 < 99$ ✓

$P_n = .8[.8(2000)1281]\left[1 - (54.5/140)^2\right]$

$P_n = 1{,}391.2$ K

Shear Design:

$S_{max} = H/3$ or $L/3 \rightarrow (10 \times 12)/3 = 40$ in

Min horiz Reinf: $.007 \times 7.625 \times 12 = .064$ in²/ft

Try #5 AT 40 in

$A = .31/(40/12) = .093$ in²/ft $> .064$ in²/ft

Min total Reinf: $.002(7.625)12 = .183$ in²/ft

Req Vert Reinf: $.183 - .093 = .09$ in²/ft

Try (2)#6 AT each end:

$A = 4(.44)/14 = .1257 > .09$ ✓

Shear check:

$V_n = V_m + V_s$ (V_s not required)

$V_u = 2.5(F_p) \rightarrow 2.5(8,672.5) = 21,681.25$

$M_u/V_u d \rightarrow 86,725/(21,681.25 \times (160/12)) = .3$

$V_m = [4 - 1.75(.3)]\,1281\sqrt{2000} + .25(9401.8)$
$= 201,426.5$

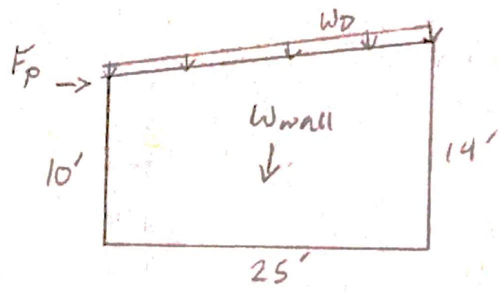
$.4\sqrt{2000}(1281) > V_m > V_u$ ✓

Ductility:

$1.25(205,265.7) = 256,582$

$V_u = 256,282/10 = 25658 < V_m$ ✓

N/S Shear Wall Design:



-Design all based on worst case

$F_P = 11,105.9$

$W_D = 50\,plf + 49 = 99\,plf$

$W_{wall} = 115 \times .67 \times 25.318 \times \left(\frac{10+14}{2}\right) = 23,409$

$P_D = (99 \times 25,318) + 23,409 = 25,915.5$

Using LC #7 : $P_u = .798\,(25,915.5) = 20,680.6$

$\phi P_n = P_u \rightarrow P_n = P_u/\phi \qquad \phi = .9$

$P_n = 20,680.6/.9 = 22,978.4$

$M_u = 11,105.9\left(\frac{10+14}{2}\right) = 133,270.8$ lb-ft

$d = 292'' \qquad t = 144'' \qquad b = 7.625''$

$f'_m = 2000$ psi $\qquad f_y = 60$ ksi

$a = 292 - \sqrt{292^2 - [2 \times (20,680.6\,(292 - \frac{149}{2}) + (133270.8 \times 12) / (.9 \times .8 \times 2000 \times 7.625]}$

$a = 1.92$

$A_s = (.8 \times 2000 \times 7.625 \times 1.92 - 22,978.4)/60,000$

$A_s = .01$

Try #4 bar $A_s = .2$

New $a = P_n + f_y A_s / .8 f'_m b$

$a = 22,978.4 + (60,000 \times .2) / .8(2000)\,7.625$
$= 2.87$

$C = .8(2000)\,7.625\,(2.87) = 35,014$

$\phi M_n = .9\left[60(.2)(292 - \frac{300}{2}) + 35.014\left(\frac{300}{2} - \frac{2.87}{2}\right)\right]$

$= 6,215.3$ K-in

$\phi M_n = 517,939$ lb-ft

N/S shear wall continued:

using LC #6 = 1.3(25,915.5) = 33,690.2

P_n = 33,690.2 / .9 = 37,433.5

a = 37,433.5 + (60000 x .2) / .8(2000) 7.625

a = 4

C = .8(2000) 7.625 (4) = 48,800

ϕM_n = .9 [60(.2)(292 - $\frac{300}{2}$) + 48.8 ($\frac{300}{2}$ - $\frac{4}{2}$)]

ϕM_n = 8,033.76 k-in -> 669,480 lb-ft

Moment capacity only (no axial):

C = T -> .8(2) 7.625 (a) = .2(60)

a = 1 in

C = .8(2000) 7.625(1) = 12,200 lbs

Axial strength: An = (300 x 7.625) = 2,287.5 in²
As = ϕ

r = 7.625 / $\sqrt{12}$ = 2.2

H/r = (12 x 12)/2.2 = 65.5

P_n = .8 [.8(2000) 2287.5] [1 - (65.5/140)^2]

P_n = 2,287 k

Shear Design:

S_{max} = H/3 or L/3 -> (12x12)/3 = 48"

Min horiz Reinforcement: .007(7.625)12 = .064 in²/ft

Try #5 AT 48" o.c.

A = .31/(48/12) = .0775 > .064 in²/ft

Min total Reinf: .002(7.625)12 = .183

Req vert Reinf: .183 - .0775 = .1055

Try (2) #8 AT each end:

A = (4x.79)/25 = .1264 > .1055 ✓

Shear check:

$V_n = V_m + V_s$ (V_s not required)

V_u = 2.5(11,105.9) = 27,764.75

$M_u/V_u d$ -> 133,270.8 / 27,764.75 (300/12) = .192

$V_m = [4 - 1.75(.192)]\, 2{,}287.5\sqrt{2000} + 2.5(20{,}680.6)$

V_m = 426,529.1

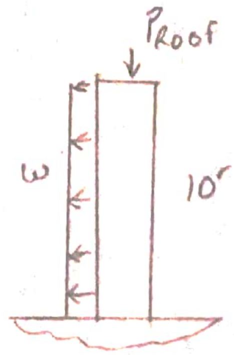
$4 \times \sqrt{2000} \times 2287.5 > V_m > V_u$ ✓

Ductility:

1.25(517,939) = 647,423.75

V_u = 647,423.75 / 12 = 53,951.98 < V_m ✓

Out of Plane Forces Wall (E)



$t = 7.625''$ $f_y = 60\,ksi$
$W_{wall} = 78\,psf$ $S_{DS} = .509$

Load on Wall:
$W_{wall} = 78(10/2) = 390\,plf$
$Roof = 71.05\,plf$

$P_u = 390 + 71.05 = 461.05\,plf$

Earthquake: $w_u = .4\,S_{DS}(I)\,w_p$

$w_u = .4(.509)\,78(1') = 15.88\,plf$

Slender Wall:
$P/A < .05 f'_m$
$461.05 / 7.625^2 = 7.9 < 700$ ✓

1st order M_u:

$M_u(oop) = w_u l^2/8$
$= 15.88\,(10)^2 \times 12 / 8 = 2{,}382\ lb\text{-}in$

2nd order M_u: $P\Delta$

$I_n = bh^3/12 = 281.7$
$I_{cr} = .05\,I_n = 14.1$
$E_m = 1{,}800{,}000\ psi$

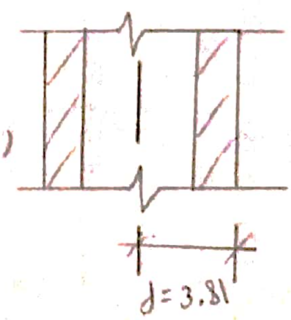
$\Delta = 5 M_u h^2 / 48 E_m I_{cr}$
$= (5(2{,}382)(10)^2 \times 12) / (48 \times 1800000 \times (14.1))$
$\Delta = 0.14$

$P\Delta = .14(461.05) = 64.55$

$M_u(est) = 2{,}382 + 64.55 = 2446.55\ lb\,in$

Est Reinf As (flexure)

$a = d - \sqrt{d^2 - 2(P_u(d - \frac{t}{2}) + M_u) / (.8\,\phi\,(f'_m b))}$
$a = 3.81 - \sqrt{3.81^2 - 2(461.05(3.81 - 3.81) + 2446.55) / (.8(.9)\,2000(7.625))}$
$a = .06$



Estimate steel Area:

$A_s = [.8f'_c ba - P_u/\phi] / f_y$

$A_s = [.8(2000)7.625(.06) - 461.05/.9] / 60000$
$= .004$ per ft

Try #5 AT 12" O.C.

$A_s = .31(12)/12 = .31 \quad > .004$ ✓

$M_{cr} = (f_r + P_n/A_n)(I_n/t/2)$
$= (153 + 461.05/58.14)(281.7/3.81)$
$M_{cr} = 11,898.7$

$f_{cr} = 153$ psi
$A_n = 58.14$
$I_n = 281.7$

Cracked section properties:

$c = A_s f_y + P_u / .64 f'_c b = .31(60000) + 461.05 / (.64 \times 2000 \times 7.625)$
$c = 1.95$

$n = 16.11$

$I_{cr} = n[A_s + P_u(t)/f_y(2d)](d-c)^2 + \frac{bc^3}{3}$
$= 16.11[.31 + 461.05(7.625)/60000(7.625)](3.81-1.95)^2 + \frac{7.625(1.95)^3}{3}$

$I_{cr} = 19.27$

$$M_u = \frac{\frac{15.88(10\times12)^2}{8} + \frac{5(11898.7)\,16^2(12)^2}{48(1800000)} \times \left(\frac{1}{281.7} - \frac{1}{19.27}\right)}{1 - (5 \times 461.05 \times 10^2 \times 12^2 / 48(1800000)\,19.27)}$$

$M_u = 28583.52$

ϕM_n: $a = (.31(60,000) + 461.05/.9) / (.8(2000)7.625)$

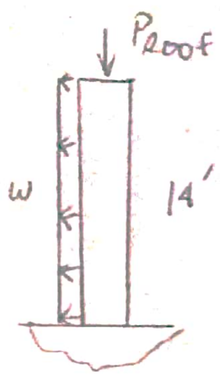
$a = 1.57$

$M_n = [461.05/.9 + (.31 \times 60000)]\left(\frac{7.625 - 1.57}{2}\right)$

$M_n = 57,862.42$

$\phi M_n = 52,076.2$

Out of plane forces Wall D:



$t = 7.625"$ $f_y = 60$ ksi
$W_{wall} = 78$ psf $S_{DS} = .509$

Load on Wall:
$W_{wall} = 78(14/2) = 546$ plf
Roof = 195.4 plf

$P_u = 546 + 195.4 = 741.4$ plf

Earthquake: $w_u = .4 S_{DS}(I) w_p$

$w_u = .4(.509) 78(1') = 15.88$ plf

Slender wall: $P/A < .05 f'_m$
$741.4 / 7.625^2 = 72.75 < 100$ ✓

1^{st} order M_u

$M_u(oop) = w_u l^2/8$
$= 15.88(14)^2 12/8 = 4,668.72$ lb-in

2^{nd} order M_u
$P\Delta$ estimate
$\Delta = 5 M_u h^2 / 48 E_m I_{cr}$

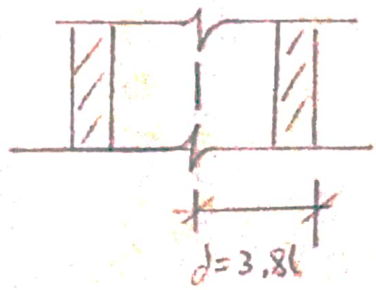
$I_n = bh^3/12 = 281.7$
$I_{cr} = .05 I_n = 14.1$
$E_m = 1,800,000$ psi

$\Delta = 5(4668.78)(14 \times 12)^2 / 48(1800000) 14.1$
$= .54$

$P\Delta = .54(741.4) = 400.97$

$M_u(est) = 4,668.72 + 400.97 = 5,069.7$ lb-in

Est Reinf As(flexure)



$a = d - \sqrt{d^2 - (2(P_u(d - t/2) + M_u) / \phi .8 f'_m b}$

$a = 3.81 - \sqrt{3.81^2 - (2 M_u) / .8(.9) 2000(7.625)}$

$a = .123$

Estimate steel Area

$As = [.8 f'_m b a - P_u/\phi] / f_y$

$As = [.8(2000)7.625(.123) - 741.4/.9] / 60000$
$= .011$

Try #6 AT 12" O.C.

$As = .44(12)/12 = .44 > .011$ ✓

$M_{cr} = (f_r + P_u/A_n)(I_n/t_{/2})$ $f_r = 153$ psi
$= (153 + 741.4/58.14)(281.7/3.81)$ $A_n = 58.14$
$M_{cr} = 12,255$ $I_n = 281.7$

Cracked section Properties:

$c = A_s f_y + P_u / .64 f'_m b$
$= .44(60000) + 741.4 / .64(2000)7.625 = 2.78$

$n = 16.11$

$I_{cr} = n[A_s + P_u(t)/f_y(2d)](d-c)^2 + \frac{bc^3}{3}$

$= 16.11[.44 + 741.4(7.625)/60000(7.625)](3.81-2.78)^2 + \frac{7.625(2.78)^3}{3}$
$= 54.82$

$$M_u = \frac{\frac{15.88(14\times12)^2}{8} + \frac{5(12,255)14^2(12)^2}{48(1800000)} \times \left(\frac{1}{281.7} - \frac{1}{54.82}\right)}{1 - (5\times741.4\times14^2\times12^2)/48(1800000)54.82}$$

$M_u = 56,024$

ϕM_n : $a = (.44(60,000) + 741.4/.9)/(.8(2000)7.625)$

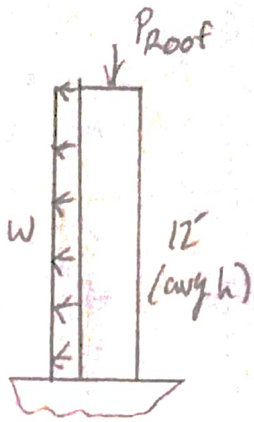
$a = 2.23$

$M_n = [741.4/.9 + (.44\times60000)](7.625 - 2.23/2)$

$M_n = 73,436.14$

$\phi M_n = 66,092.53$

Out of Plane Force Walls N/S



$t = 7.625"$ $f_y = 60$ ksi
$W_{wall} = 78$ psf SDS = .509

Load on Wall:
$W_{wall} = 78(12/2) = 468$ plf
Roof = 99 plf

$P_u = 468 + 99 = 567$ plf

Earthquake: $w_u = .4\,S_{DS}(I)\,W_p$

$w_u = .4(.509)\,78(1') = 15.88$ plf

Slender Wall:
$P/A < .05 f'_m$
$567/7.625^2 = 9.75 < 100$ ✓

1st order Mu:

$M_u(oop) = w_u L^2/8$
$= 15.88(12)^2 \times 12 / 8 = 3,430.08$ lb-in

2nd order Mu:
PΔ (est)
$\Delta = 5 M_u h^2 / 48 E_m I_{cr}$
$= 5(3,430.08)(12)^2(12)^2 / (48 \times 1800000 \times 14.1)$
$\Delta = .292$

$I_n = bh^3/12 = 281.7$
$I_{cr} = .05 I_n = 14.1$
$E_m = 1,800,000$ psi

$P\Delta = .292(567) = 165.52$

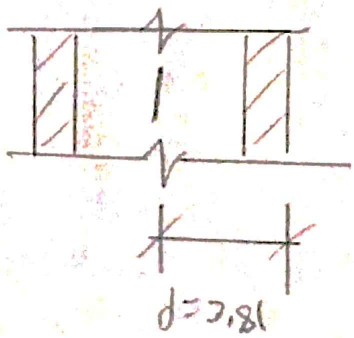
$M_u(est) = 165.52 + 3,430.08 = 3595.6$ lb-in

Est Reinf As(flexure)

$a = d - \sqrt{d^2 - 2(P_u(d - t/2) + M_u) / (.8\phi f'_m b)}$

$a = 3.81 - \sqrt{3.81 - (2 \times 3595.6) / (.8(.9)2000(7.625))}$

$a = .087$



Estimate steel Area:

$A_s = [.8 f'_m b a - P_u/.9] / f_y$

$A_s = [.8(2000)7.625(.087) - 567/.9] / 60000$
$= .0072$

Try #5 @ 12" O.C.

$A_s = .31(12)/12 = .31 > .0072$ ✓

$M_{cr} = (f_r + P_n/A_n)(I_n/t/2)$

$f_r = 153$ psi
$A_n = 58.14$
$I_n = 281.7$

$M_{cr} = (153 + 567/58.14)(281.7/3.81)$
$= 12,033.42$

Cracked section properties:

$c = A_s f_y + P_u / .64 f'_m b = .31(60000) + 567 / (.64(2000)7.625)$

$c = 1.96$

$n = 16.11$

$I_{cr} = n[A_s + P_u(t)/f_y(2d)](d-c)^2 + \frac{bc^3}{3}$

$= 16.11[.31 + (567(7.625)/60000(7.625))](3.81-1.96)^2 + \frac{7.625(1.96)^3}{3}$
$= 36.75$

$$M_u = \frac{\frac{15.88(12 \times 12)^2}{8} + \frac{5(12,033.42)12^2(12)^2}{48(1800000)} \times \left(\frac{1}{281.7} - \frac{1}{36.75}\right)}{1 - (5 \times 567 \times 12^2 \times 12^2 / 48(1800000)36.75)}$$

$M_u = 41,937.05$

ϕM_n: $a = (.31(60000) + 567/.9) / .8(2000)7.625$

$a = 1.58$

$M_n = [567/.9 + (.31 \times 60000)](7.625 - 1.58/2)$

$M_n = 58,122.68$

$\phi M_n = 52,310.4$

Column Footing:

Assume soil f'c = 3000 psi (Tbl 1808.8.1 IBC)

P = 7,294.88

Col weight: 1.2 (27.48 x 15.833) = 522 #

P_u = 7,294.88 + 522.11 = 7817 #s

f'c = P/A -> A = P/f'c

A = 7817/3000 = 2.6

b = $\sqrt{2.6}$ = 1.614

use 1'-9" x 1'-9" Col Foundation

Wall continous footing:

-Wall D: •Wwall (115 x .67 x 14) = 1,078.7 plf
Roof loads: (71810.5 x 9)/80 = 1328.68 plf

$P_{(per\ foot)}$ = 2407.4 plf

3000 = 2407.4 / (w x 1')

w = 2407.4 / 3000 = .8'

use 1' wide continous Foundation

-Wall E: Wwall (115 x .67 x 10) = 770.5 plf
Roof loads = (4294.7 x 9)/80 = 483.2 plf

$P_{(per\ foot)}$ = 1253.7 plf

W = 1253.7/3000 = .42' ≈ 5"

use 9" continous foundation

N/S walls: (based on interior loads)

Wwall = (115 x .67 x 12') = 924.6 plf
Roof load = 380 plf + (48.85 x 1.2) = 438.62

P = 1363.22 -> W = 1363.22/3000 = .45 ≈ 5.5"
use 9" contionous Foundation

Connections:

- Metal Deck to HSS BM

T/C = 15,587.2 lbs (Diaph forces)

15,587.2 / 25 = 623.5 plf

ARC SPOT WELD3,4 Capacity = 783 lbs (from Vulcraft Catalog 2014)

USE ARC SPOT WELD3,4 AT 12" O.C.

BM to CMU: (based on highest load) = 10,450 lbs

PL thickness: $\phi P_n \geq P_u$ (Assume A36) $f'_c = 3ksi$ $\phi = .65$

$.65(.85(3)A_1) \geq 10.45 \rightarrow A_1 = 6.3 \ in^2$

PL min size = to BM Dimensions $b \times h = 43.6 \ in^2$

Try 7" x 7" PL = 49 in² > 43.6 in²

$m = 7 - [.95(6.6)/2] = 3.865$

$n = 7 - [.8(6.6)/2] = 4.36$

$\lambda = 1$ (assumed

$n' = \frac{1}{4}\sqrt{6.6 \times 6.6} = 1.65$

$l = 4.36$

$t = 4.36\sqrt{2(10.45)/.9 \times 7^2 \times 36} \rightarrow t = 0.5"$

USE $7 \times \frac{1}{2} \times 7$ (A36) PL

Min weld leg (AISC m pg 16.1-119):

PL = ½" → min leg = 3/16" < 7/16" ✓

Max leg: ½ − 1/16 = 7/16"

Weld capacity: 7/16" Weld

$\phi R_n = 1.392 \times 7 = 9.74$ k-in

USE 7/16" fillet welds

Bm to CMU continued...

use (4) SSTB16 5/8" diameter Simpson Anchor bolts

Capacity = 4(2865) = 11,460 lbs > 10,450 lbs ✓

Bm to GRDR conection:

use 7"x1/2"x7" PL with 7/16" Fillet welds

(same calcs as Bm to CMU)

GRDR to Column:

P_u = 3,040 lbs

PL thickness: $\phi P_n \geq P_u$ (Assume A36) $f'_c = 3ksi$ $\phi = .65$

$.65(.85(3)A_1) \geq 3.04 \rightarrow A_1 = 1.83$

min PL size = BM dimensions $b \times h = 43.6$

Try 9"x7" PL = 63 in² > 43.6 ✓

$m = 9 - [.95(6.6)/2] = 5.865$
$n = 7 - [.8(6.6)/2] = 4.36$
$\lambda = 1$
$n' = 1/4\sqrt{6.6 \times 6.6} = 1.65$
$l = 4.36$

$t = 5.865\sqrt{2(3.04)/.9 \times 9 \times 7 \times 36} = .32"$

use (2) 1/4" PL 1/4" fillet welds

From AISC M Table 7-3:

use (2) 5/8" diam bolt $\phi R_n = 16.3$ k (each)

Column to Footing:

Required bearing area A_1: $\phi P_p \geq P_u$

$P_p = .85 f'_c A_1 \sqrt{A_2/A_1} \leq 1.7 f'_c A_1$

$P_u = 7294.88 + (27.48 \times 15.833)1.2 = 7816.99 \approx 7817$ lbs

$.65(.85(3)A_1)\sqrt{21 \times 21 / A_1} \geq 7.817$

$A_1/\sqrt{A_1} \geq .225 \rightarrow A_1 \geq .05$ in^2

but $P_u = \phi P_n \leq \phi c(1.7 f'_c A_1)$

$A_1 \geq P_u / \phi 1.7 f'_c \rightarrow A_1 \geq 7.817/(.65 \times 1.7 \times 3) = 2.36$ in^2

Min PL dimensions = Colmn $b \times h = 25$ in^2

Try 8x8 PL $A = 64$ in^2

$m = 8 - (.95(5)/2) = 5.625$
$n = 8 - (.8(5)/2) = 6$
$n' = \frac{1}{4}\sqrt{5 \times 5} = 1.25$
$\lambda = 1$

$l = 6$

$t = 6\sqrt{2(7.817)/.9 \times 8^2 \times 36} \rightarrow t = .52'' \approx 3/4''$

use 8"x 3/4"x 8" (A36) PL

Anchor bolts: (AISC-M pg 7-23)

Try A 307 bolts (4) 5/8" Diameter

$\phi r_n = 4(26) = 104k > 7.8k$

use (4) 5/8" ϕ A 307 bolts

GENERAL

1. ALL NEW CONSTRUCTION SHALL COMPLY WITH THE CONTRACT DOCUMENTS AND THE CURRENT EDITION OF THE IBC AND ACI.

2. THESE GENERAL NOTES SUPERSEDE THE REQUIREMENTS OF THE PROJECT SPECIFICATIONS. IN CASE OF CONFLICT BETWEEN THE PLANS AND SPECIFICATIONS, CONTACT THE OWNER'S REPRESENTATIVE.

3. REFERENCE TO CODES, RULES, REGULATIONS, STANDARDS, MANUFACTURER'S INSTRUCTIONS OR REQUIREMENTS OF REGULATORY AGENCIES IS TO THE LATEST PRINTED EDITION OF EACH IN EFFECT AT THE DATE OF SUBMISSION OF BID UNLESS THE DOCUMENT DATE IS SHOWN.

4. TYPICAL DETAILS AND GENERAL NOTES APPLY TO ALL PARTS OF THE WORK EXCEPT WHERE SPECIFICALLY DETAILED OR UNLESS NOTED OTHERWISE (U.N.O.)

5. THE STRUCTURAL DRAWINGS ILLUSTRATE THE NEW STRUCTURAL MEMBERS. REFER TO ARCHITECTURAL, MECHANICAL AND ELECTRICAL DRAWINGS FOR NON-STRUCTURAL ITEMS WHICH REQUIRE SPECIAL PROVISIONS DURING THE CONSTRUCTION OF THE STRUCTURAL MEMBERS.

6. REFER TO ARCHITECTURAL DRAWINGS FOR FLOOR DEPRESSIONS, EDGE OF SLAB, OPENINGS, SLOPES, DRAINS, CURBS, PADS, EMBEDDED ITEMS, NON-BEARING PARTITIONS, ETC. REFER TO MECHANICAL AND ELECTRICAL DRAWINGS FOR SLEEVES, OPENINGS, AND HANGERS FOR PIPES, DUCTS AND EQUIPMENT.

7. THE CONTRACTOR SHALL VERIFY AND BE RESPONSIBLE FOR COORDINATING THE WORK OF ALL TRADES AND SHALL VERIFY ALL DIMENSIONS AND CONDITIONS WHICH IMPACT THE WORK. FIELD VERIFY SIZES, ELEVATIONS, HOLE LOCATIONS, ETC. PRIOR TO FABRICATION.

8. DRAWING DIMENSIONS ARE TO FACE OF FINISH, JOINT CENTERLINE OR COLUMN GRID CENTERLINE UNLESS NOTED OTHERWISE. DO NOT SCALE THE DRAWINGS.

9. CONTRACTOR SHALL CAREFULLY REVIEW THE DRAWINGS TO IDENTIFY THE SCOPE OF WORK REQUIRED. VISIT THE SITE TO RELATE THE SCOPE OF WORK TO EXISTING CONDITIONS AND DETERMINE THE EXTENT TO WHICH THOSE CONDITIONS AND PHYSICAL SURROUNDINGS WILL IMPACT THE WORK.

10. EXISTING CONDITIONS AS SHOWN ON THESE PLANS ARE FOR REFERENCE ONLY. CONTRACTOR IS REQUIRED TO FIELD VERIFY ALL EXISTING CONDITIONS PRIOR TO CONSTRUCTION. CONTRACTOR SHALL REPORT CONDITIONS THAT CONFLICT WITH THE CONTRACT DOCUMENTS TO THE OWNER'S REPRESENTATIVE. DO NOT DEVIATE FROM THE CONTRACT DOCUMENTS WITHOUT WRITTEN DIRECION FROM THE OWNER'S REPRESENTATIVE.

11. THE CONTRACTOR SHALL RESOLVE ANY CONFLICTS ON THE DRAWINGS OR IN THE SPECIFICATIONS WITH THE OWNER'S REPRESENTATIVE BEFORE PROCEEDING WITH THE WORK.

12. ANY DEVIATION, MODIFICATION & SUBSTITUTION FROM THE APPROVED SET OF STRUCTURAL DRAWINGS SHALL BE SUBMITTED TO THE OWNER'S REPRESENTATIVE FOR REVIEW/APPROVAL PRIOR TO ITS USE OR INCLUSION ON THE SHOP DRAWINGS & PRIOR TO PROCEEDING WITH THE WORK.

13. THE CONTRACTOR SHALL PROVIDE ALL NECESSARY SHORES, BRACES AND GUYS REQUIRED TO SUPPORT ALL LOADS TO WHICH THE BUILDING STRUCTURE AND COMPONENTS, SOILS, OTHER STRUCTURES AND UTILITIES MAY BE SUBJECTED DURING CONSTRUCTION. SHORING SYSTEMS SHALL BE DESIGNED AND STAMPED BY A CIVIL ENGINEER LICENSED IN THE STATE OF CALIFORNIA. VISITS TO THE SITE BY THE OWNER'S REPRESENTATIVE WILL NOT INCLUDE OBSERVATION OF THE ABOVE NOTED ITEMS.

14. THE CONTRACTOR SHALL PROVIDE MEANS, METHOD, TECHNIQUES, SEQUENCE AND PROCEDURE OF CONSTRUCTION AS REQUIRED. SITE VISITS PERFORMED BY THE OWNER'S REPRESENTATIVE DO NOT INCLUDE INSPECTIONS OF MEANS AND METHODS OF CONSTRUCTION PERFORMED BY CONTRACTOR.

15. THE CONTRACTOR SHALL PROTECT ALL WORK, MATERIALS AND EQUIPMENT FROM DAMAGE AND SHALL PROVIDE PROPER STORAGE FACILITIES FOR MATERIALS AND EQUIPMENT DURING CONSTRUCTION.

16. STRUCTURAL OBSERVATIONS PERFORMED BY ENGINEER DURING CONSTRUCTION ARE NOT THE CONTINUOUS AND SPECIAL INSPECTION SERVICES AND DO NOT WAIVE THE RESPONSIBILITY FOR THE INSPECTIONS REQUIRED OF THE BUILDING INSPECTOR OR THE DEPUTY INSPECTOR. OBSERVATIONS ALSO DO NOT GUARANTEE CONTRACTOR'S PERFORMANCE AND SHALL NOT BE CONSIDERED AS SUPERVISION OF CONSTRUCTION.

17. CONTRACTORS SHALL REVIEW SHOP DRAWINGS FOR COMPLETENESS AND COMPLIANCE WITH CONTRACT DOCUMENTS. CONTRACTOR SHALL STAMP SHOP DRAWINGS PRIOR TO SUBMISSION TO OWNER'S REPRESENTATIVE.

18. REVIEW OF THE SHOP DRAWINGS SHALL NOT BE CONSTRUED AS AN AUTHORIZATION TO DEVIATE FROM CONTRACT DOCUMENTS.

19. SHOP DRAWINGS WILL NOT BE PROCESSED DUE TO INCOMPLETENESS, LACK OF CO-ORDINATION WITH RELEVANT PORTION OF CONTRACT DOCUMENTS, LACK OF CALCULATIONS IF REQUIRED AND WHERE DEVIATIONS, MODIFICATIONS AND SUBSTITUTIONS ARE INDICATED WITHOUT PRIOR WRITTEN APPROVAL FROM OWNER'S REPRESENTATIVE.

20. ALLOW FOURTEEN WORKING DAYS FOR PROCESSING SHOP DRAWINGS AFTER RECEIPT.

CONCRETE

1. CONCRETE IS REINFORCED AND CAST-IN-PLACE UNLESS OTHERWISE NOTED. WHERE REINFORCING IS NOT SPECIFICALLY SHOWN OR WHERE DETAILS ARE NOT GIVEN, PROVIDE REINFORCING SIMILAR TO THAT SHOWN FOR SIMILAR CONDITIONS, SUBJECT TO REVIEW BY THE OWNER'S REPRESENTATIVE.

2. ALL STRUCTURAL CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH AT 28 DAYS AS FOLLOWS:

SLAB ON GRADE	4000 PSI NORMAL WEIGHT
ALL OTHER CONCRETE	4000 PSI NORMAL WEIGHT

3. ALL STRUCTURAL CONCRETE MIXES SHALL BE TYPE II CEMENT AND SHALL BE DESIGNED BY AN APPROVED LABORATORY.

4. NORMAL WEIGHT CONCRETE AGGREGATES SHALL CONFORM TO ASTM C-33. LIGHT WEIGHT CONCRETE AGGREGATES SHALL CONFORM TO ASTM C-330.

5. NO MORE THAN ONE GRADE OF CONCRETE SHALL BE ON THE JOB SITE AT ANY ONE TIME.

6. THOROUGHLY CLEAN AND ROUGHEN ALL HARDENED CONCRETE AND MASONRY SURFACES TO RECEIVE NEW CONCRETE. INTERFACE SHALL BE ROUGHENED TO A FULL AMPLITUDE OF 1/4" UNLESS NOTED OTHERWISE.

7. KEY AND DOWEL POUR JOINTS AS SHOWN ON THE PLANS. ANY DEVIATION FROM POUR JOINTS SHOWN ON THE PLANS MUST BE APPROVED BY THE OWNER'S REPRESENTATIVE.

8. DEFECTIVE CONCRETE (VOIDS, ROCK POCKETS, HONEYCOMBS, CRACKING, ETC.) SHALL BE REMOVED AND REPLACED AS DIRECTED BY THE OWNER'S REPRESENTATIVE.

REINFORCEMENT

1. REINFORCING TO CONFORM TO THE FOLLOWING, UNLESS OTHERWISE NOTED:

LOCATION	TYPE
REINFORCING STEEL U.N.O.	ASTM A615, 60 KSI
REINFORCING STEEL TO BE WELDED AND IN CONCRETE SHEAR WALL BOUNDARY ELEMENTS	ASTM A706, 60 KSI
WELDED STEEL WIRE FABRIC	ASTM A185, 70 KSI
SMOOTH DOWELS IN SLAB ON GRADE	ASTM A36, 36 KSI

2. REINFORCING BARS SHALL HAVE THE FOLLOWING MINIMUM COVERAGE. PLACE BARS AS NEAR TO THE CONCRETE SURFACE AS THESE MINIMUMS PERMIT WHEREVER POSSIBLE UNLESS NOTED OTHERWISE:

	MIN. CONCRETE COVER
CONCRETE POURED AGAINST EARTH	3"
FORMED CONCRETE IN CONTACT WITH EARTH	2"
EXPOSED TO WEATHER (#6 AND LARGER)	2"
EXPOSED TO WEATHER (#5 AND SMALLER)	1.5"
SLABS & WALLS NOT EXPOSED TO WEATHER	3/4"

3. #5 AND LARGER REINFORCING BARS SHALL NOT BE SPLICED EXCEPT AS LOCATED AND DETAILED ON THE DRAWINGS. #4 AND SMALLER BARS WITH LENGTH NOT SHOWN SHALL BE CONTINUOUS, LAPPING 1'-6" MINIMUM IN CONCRETE (SEE TYPICAL DETAILS). HORIZONTAL WALL SPLICES SHALL BE STAGGERED. VERTICAL BARS SHALL NOT BE SPLICED EXCEPT AT HORIZONTAL SUPPORT, SUCH AS FLOOR OR ROOF, UNLESS DETAILED OTHERWISE. ALL BARS ENDING AT THE FACE OF A WALL, COLUMN, OR BEAM SHALL EXTEND TO WITHIN 2" OF THE FAR FACE AND HAVE A 90 DEGREE HOOK UNLESS OTHERWISE SHOWN.

4. BARS SHALL BE FIRMLY SUPPORTED AND ACCURATELY PLACED AS REQUIRED BY THE A.C.I. STANDARDS, USING TIE AND SUPPORT BARS IN ADDITION TO REINFORCEMENT SHOWN WHERE NECESSARY FOR FIRM AND ACCURATE PLACING. ALL DOWELS SHALL BE ACCURATELY SET IN PLACE BEFORE PLACING CONCRETE.

5. DRAWINGS SHOW TYPICAL REINFORCING CONDITIONS. CONTRACTOR SHALL PREPARE DETAILED PLACEMENT DRAWINGS OF ALL CONDITIONS SHOWING QUANTITY, SPACING, SIZE, CLEARANCES, LAPS, INTERSECTIONS AND COVERAGE REQUIRED BY STRUCTURAL DETAILS, APPLICABLE CODE AND TRADE STANDARDS. CONTRACTOR SHALL NOTIFY REINFORCING INSPECTOR OF ANY ADJUSTMENTS FROM TYPICAL CONDITIONS THAT ARE PROPOSED IN PLACEMENT DRAWINGS TO FACILITATE FIELD PLACEMENT OF REINFORCING STEEL AND CONCRETE.

6. NO WELDING OF REINFORCEMENT (INCLUDING TACK WELDING) SHALL BE DONE UNLESS SHOWN ON THE DRAWINGS. WHERE SHOWN ON THE DRAWINGS, WELDING OF REINFORCING STEEL SHALL BE PERFORMED BY WELDERS SPECIFICALLY CERTIFIED FOR REINFORCING STEEL. USE E90XX ELECTRODES.

CONCRETE
- MASONRY

A. CONCRETE MASONRY TO BE SUPPLIED PER 2018 CBC SECTION 2105.22 AND PLACED PER SECTION 2104.
B. ASSEMBLY STRENGTH F'm - 2000 psi AT 28 DAYS.
C. UNITS: MEDIUM WEIGHT 2 CELL BLOCKS CONFORMING TO ASTM C90. SHRINKAGE OF BLOCKS SHALL NOT EXCEED 0.065% WHEN TESTED PER ASTM C426.
D. MORTAR: ASTM C270, TYPE M.
E. GROUT: ASTM C476. COMPRESSIVE STRENGTH AS REQUIRED TO ATTAIN SPECIFIED ASSEMBLY STRENGTH. ALL CELLS SHALL BE FULLY GROUTED.
F. USE LOW LIFT CONSTRUCTION WITH MAXIMUM GROUT POUR HEIGHT OF 4 FT. HIGH LIFT GROUTING IS ACCEPTABLE IF APPROVED IN WRITING BY THE ENGINEER.
G. ALL MASONRY TO BE REINFORCED UNLESS SPECIFICALLY MARKED "NOT REINFORCED".
H. SEE PLAN FOR LOCATIONS OF VERTICAL CONTROL JOINTS. HORIZONTAL BOND BEAM AND LINTEL REINFORCING SHALL BE CONTINUOUS ACROSS VERTICAL CONTROL JOINTS.
I. ALL CELLS, SHALL BE GROUTED SOLID. REINFORCING STEEL SHALL BE SECURED IN PLACE PRIOR TO GROUTING.
J. MASONRY BUILDING WALLS HAVE BEEN DESIGNED TO SPAN VERTICALLY AS SIMPLE SPANS FROM FLOOR TO ROOF AND ARE DEPENDENT UPON THE COMPLETED ROOF STRUCTURE AND COMPLETION OF ALL MASONRY WALLS FOR STABILITY AND FOR RESISTANCE TO WIND AND SEISMIC FORCES. THE CONTRACTOR IS SOLELY RESPONSIBLE FOR PROVIDING ALL NECESSARY TEMPORARY BRACING AS REQUIRED FOR CONSTRUCTION LOADS, FOR STABILITY AND FOR RESISTANCE TO WIND AND SEISMIC FORCES UNTIL THE ENTIRE STRUCTURE IS COMPLETE. THE SHORING SHALL NOT RELY ON ANY MOMENT RESISTANCE CAPACITY OF THE FOOTINGS.

INSPECTION / TESTING

1. CONTINUOUS SPECIAL INSPECTION IS REQUIRED PER IBC. CONTINUOUS SPECIAL INSPECTION IS REQUIRED FOR THE FOLLOWING WORK AS DESCRIBED IN IBC:

1.1 ALL CONCRETE WORK
1.2 BOLTS INSTALLED IN CONCRETE
1.3 EXPANSION & ADHESIVE ANCHORS
1.4 REINFORCING STEEL AND PRESTRESSING STEEL TENDONS
1.5 WELDING OF REINFORCING STEEL
1.6 JUST PRIOR TO PLACING CONCRETE FOUNDATIONS TO ENSURE SUBGRADE IS SUITABLE, FREE FROM LOOSE SOIL, AND FOUNDATIONS ARE OF PROPER DIMENSIONS

JOURNEYMAN INTERNATIONAL

SEAL:

DRAWINGS TO BE REVIEWED BY IN-COUNTRY ARCHITECT AND ENGINEER
DRAWINGS NOT FOR CONSTRUCTION

PROJECT: Take Heart School in Kenya

SITE: Migori County, Kenya

REVISIONS

No.	DESC.	DATE

DRAWN BY:

CHECKED BY:

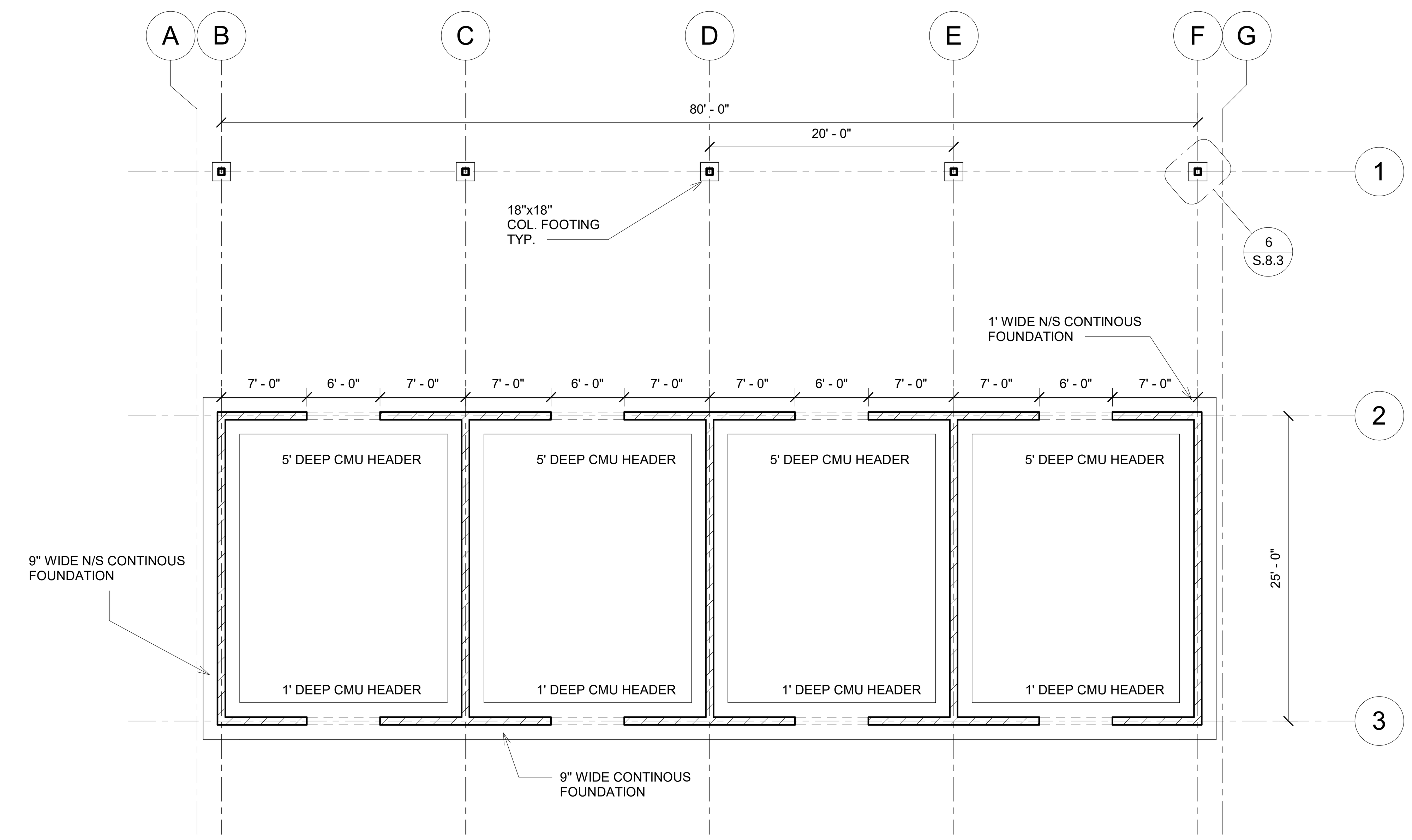
PLOT DATE: 6/10/2021 8:15:27 PM

SHEET NAME:

SCALE:

SHEET No.:

NOTES:
1. SEE GENERAL NOTES
2. SEE SHEETS S.8 FOR WALL AND HEADER DETAIL AND SCHEDULE



1 Level 1
1/8" = 1'-0"

SEAL:

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Migori County, Kenya

REVISIONS

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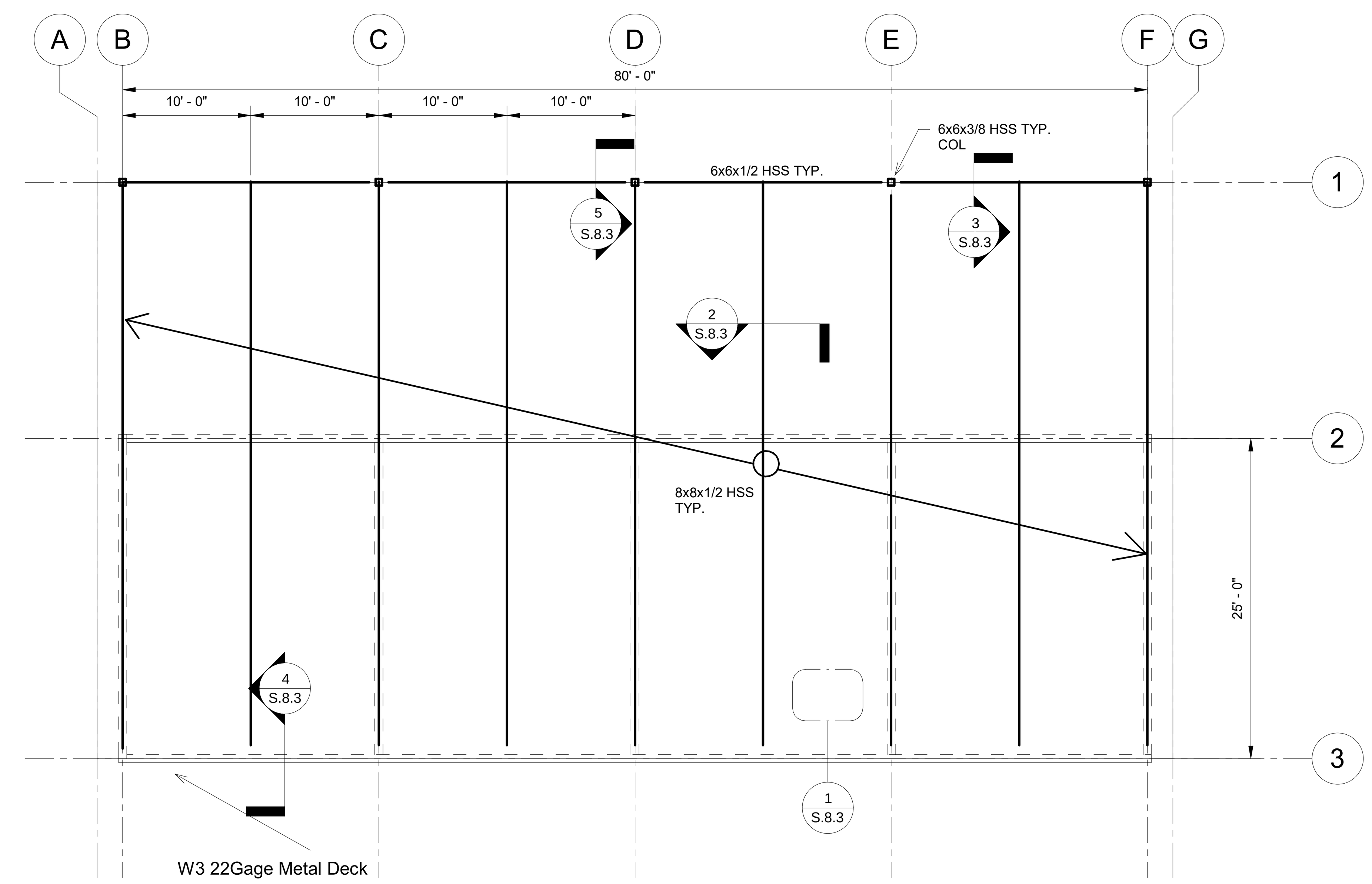

1 Level 2
1/8" = 1'-0"

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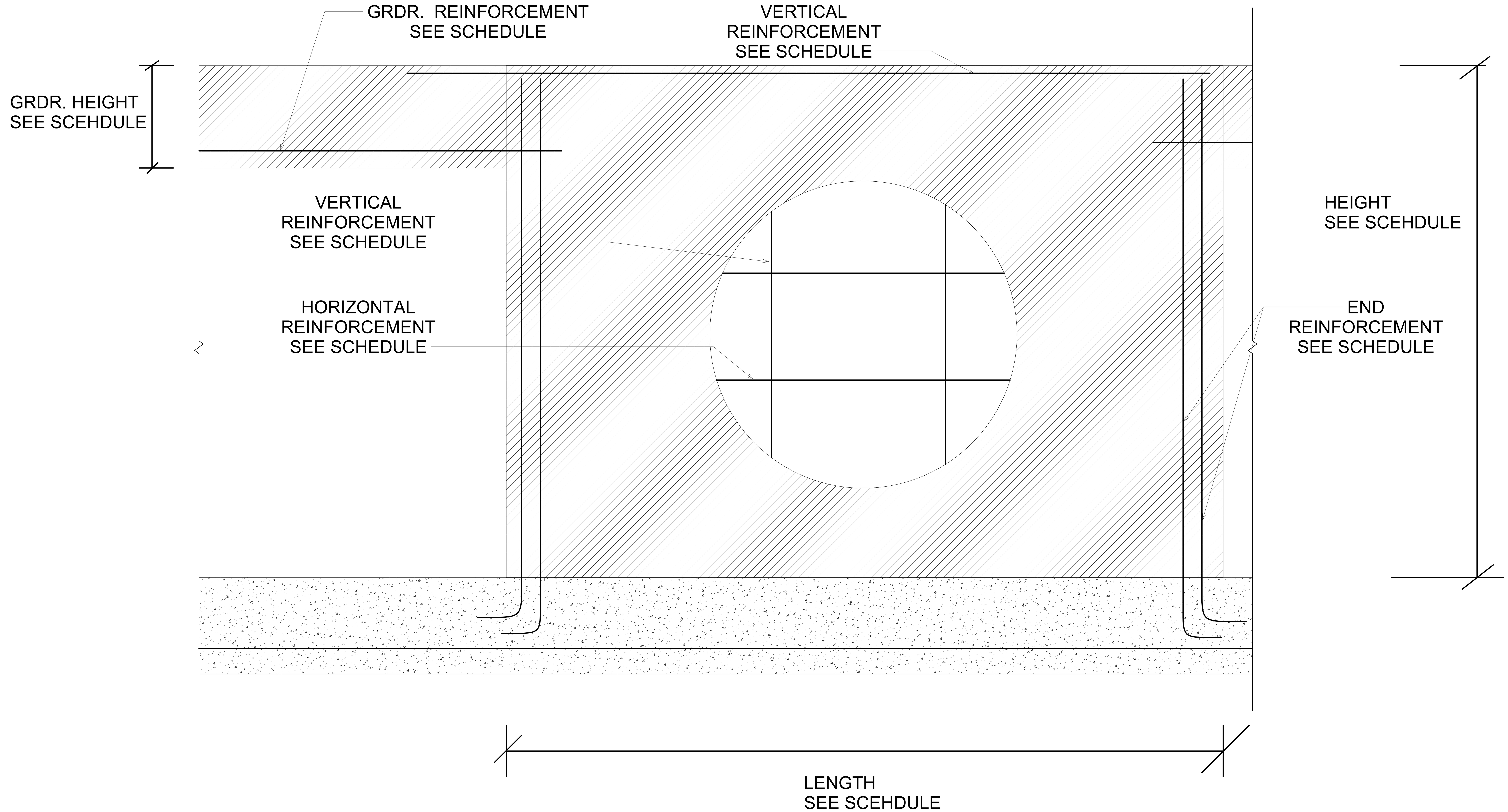

1 E/W Wall
1" = 1'-0"

E/W SHEAR WALL SHEDULE						
Grid Line	Height	Length	Vert. Reinf.	Horiz. Reinf.	Top Reinf.	End Reinf.
1	14'	14'	#5 AT 56"O.C.	#5 AT 56"O.C.	#5	(2) #6
2	10'	14'	#5 AT 40"O.C.	#5 AT 40"O.C.	#5	(2) #6

HEADER SHEDULE			
Grid Line	Height	Length	Horiz. Reinf.
1	6'	6'	(2) #6
2	2'	6'	#5

2 E/W Shedule
1" = 1'-0"

JOURNEYMAN INTERNATIONAL

SEAL:

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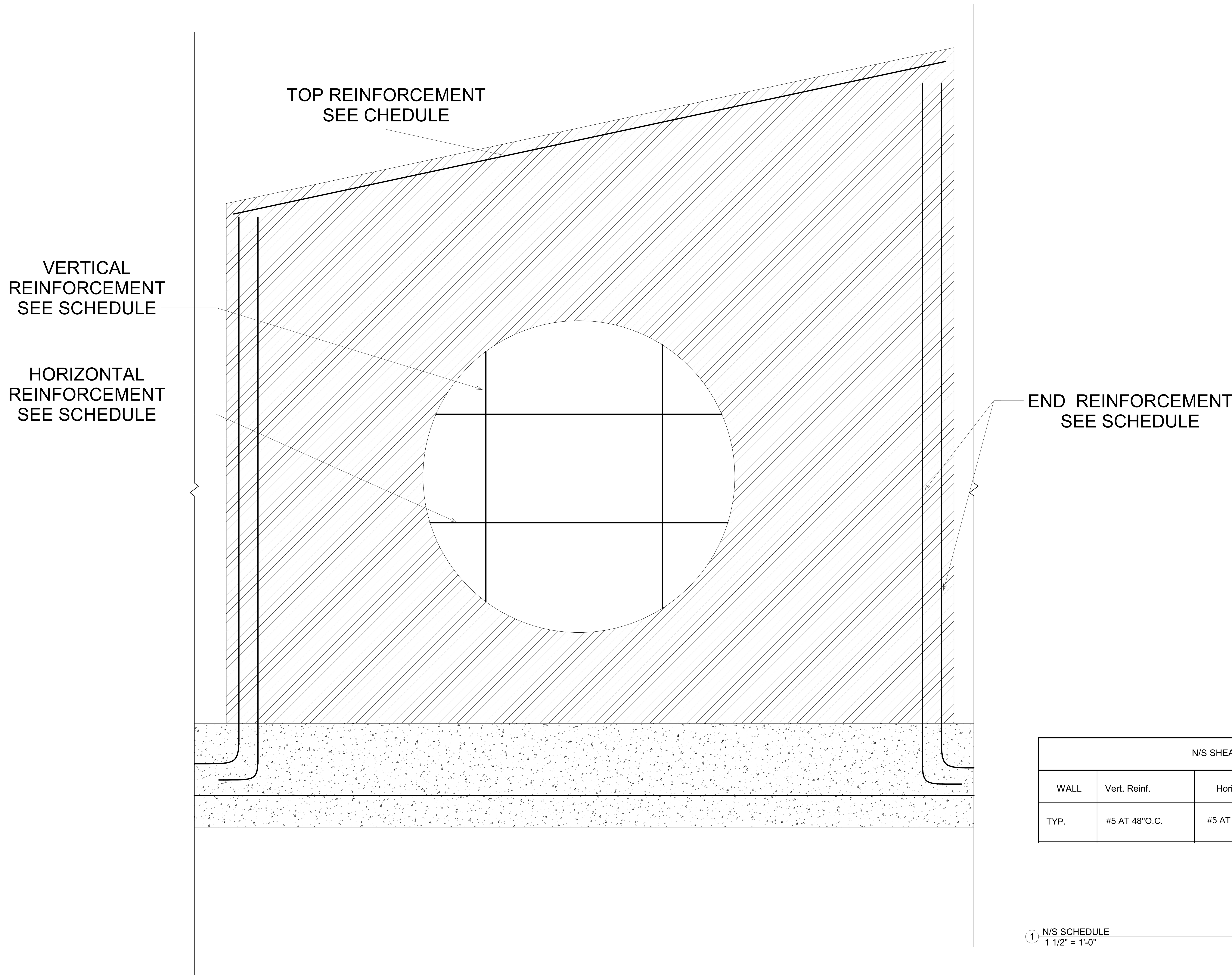

2 N/S Wall
1" = 1'-0"

N/S SHEAR WALL SHEDULE				
WALL	Vert. Reinf.	Horiz. Reinf.	Top Reinf.	End Reinf.
TYP.	#5 AT 48"O.C.	#5 AT 48"O.C.	#5	(2) #8

1 N/S SCHEDULE
1 1/2" = 1'-0"

SEAL:

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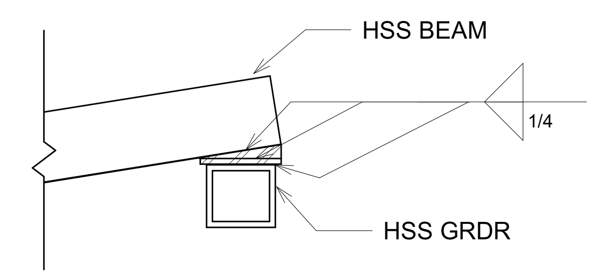
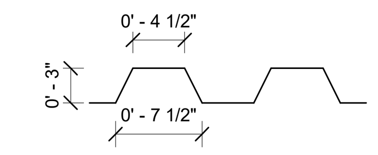
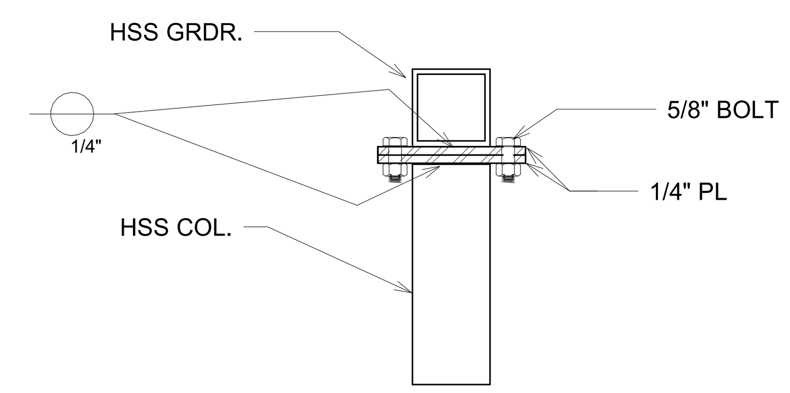


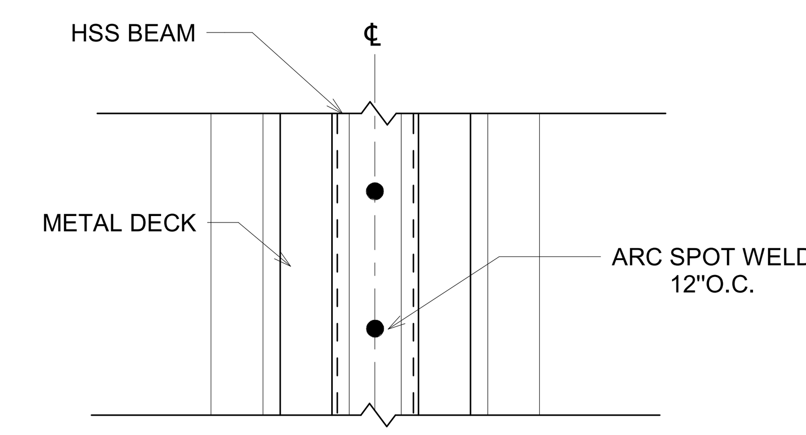
(4) BEAM TO GRDR
1" = 1'-0"



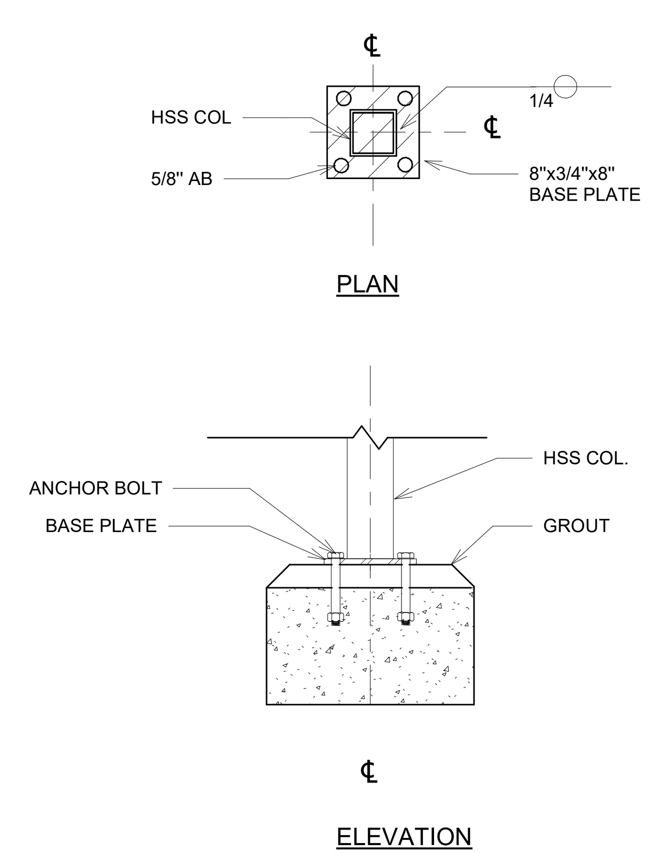
(1) TYP. METAL DECK SECTION
1" = 1'-0"



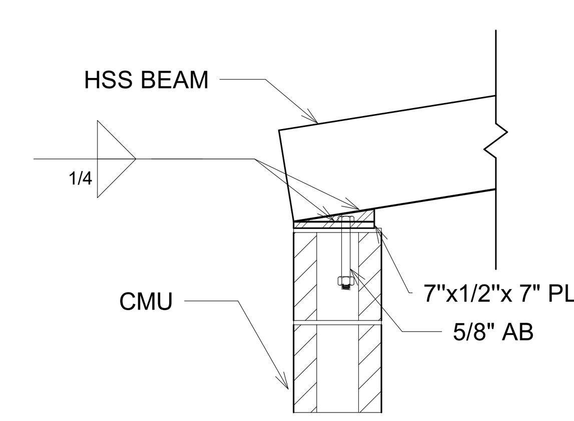
(5) GRDR TO COLUMN
1 1/2" = 1'-0"



(2) METAL DECK TO BEAM
1" = 1'-0"



(6) COLUMN TO FOOTING
1" = 1'-0"



(3) BEAM TO CMU WALL
1" = 1'-0"

SEAL:

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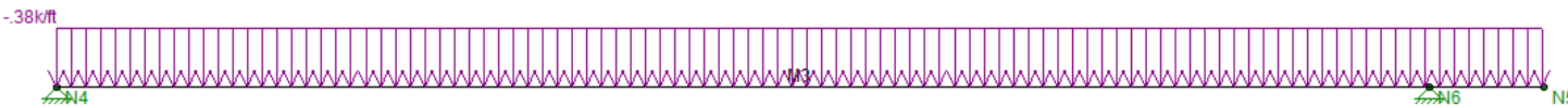
RISA DATA

Building 1 BM



Beam Deflections (By Combination)

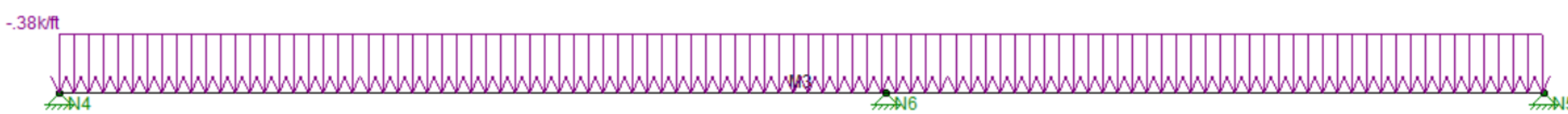
Service | Strength | Beam Defl | Beam Check

	L...	Member Label	S...	Locati...	y' [in]	(n) L'/y'...
1	1	M3	1	12.512	-1.169	258
2			2	27.3	.308	163

Joint Reactions (By Combination)

	L...	Joint Label	X [k]	Y [k]	MZ [k-ft]
1	1	N4	0	4.755	0
2	1	N6	0	5.619	0
3	1	Totals:	0	10.374	
4	1	COG (ft):	X: 13.65	Y: 3	

Building 2 BM



Beam Deflections (By Combination)

Service | Strength | Beam Defl | Beam Check

	L...	Member Label	S...	Locati...	y' [in]	(n) L'/y'...
3	1	M3	1	11.325	-.603	501
4			2	37.75	-.119	2027

Joint Reactions (By Combination)

	L...	Joint Label	X [k]	Y [k]	MZ [k-ft]
4	1	N4	0	3.786	0
5	1	N6	0	10.865	0
6	1	N5	0	2.563	0