BECKER
structural engineers, inc.

September 23, 2010

Ms. Tammy Munson
Code Enforcement Officer / Plan Reviewer
Inspection Services Division
City of Portland Maine
389 Congress Street
Portland, Maine 04101

CALCULATIONS FOR END WALL REVISIONS
RENOVATIONS TO FORMER CUMBERLAND COLD STORAGE BUILDING
PORTLAND, MAINE

Dear Tammy,

Per the City of Portland's request, we are formally submitting our calculations for the structural modifications to the end wall at the former Cumberland Cold Storage Building for your records. Proposed building renovations include adding large windows to the end wall (southeast wall) that decrease the seismic capacity of the wall by more than 10%. Per our previous discussion, and as outlined in our Code interpretations letter, dated August 9, 2010 (and attached), the City of Portland Code Enforcement (AHJ) will accept reinforcing the end wall to meet current Code requirements without upgrading other areas of the structure, provided we supply calculations that back up our end wall design. Our calculations are attached for your review and use.

The project has required new masonry stair and elevator shafts to be installed. We have detailed these as lateral force resisting members and they will act to stiffen the existing structure and serve as voluntary improvements to the lateral force resisting system. By adding these voluntary improvements, we have reduced the seismic forces in other surrounding elements and improved the seismic system of this building.

Please let us know any questions or comments. We would be happy to meet and discuss if desired.

Sincerely,
BECKER STRUCTURAL ENGINEERS, Inc.

STATE OF MAINE
PAUL B.
BECKER
NO. 6554
LICENSED
PROFESSIONAL ENGINEER

Paul B. Becker, P. E.
President

Cc: Chris Pachios
Winton Scott

BECKER
structural engineers, inc.

August 9, 2010

Ms. Tammy Munson
Code Enforcement Officer / Plan Reviewer
Inspections Services Division
City of Portland
389 Congress Street
Portland, Maine

STRUCTURAL BUILDING CODE AND CODE INTERPRETATIONS FOR PROPOSED RENOVATIONS TO FORMER CUMBERLAND COLD STORAGE BUILDING
PORTLAND, MAINE

Dear Tammy,

We are formally requesting to utilize the 2006 version of the International Building Code (IBC) for the structural design of above referenced project. We understand that the City of Portland is currently enforcing the 2003 version of the IBC Code. The following is our justification for the use of the newer version of the Code.

Our justification in using the 2006 Edition of the IBC Code pertains to the Seismic provisions included in the Codes. The Seismic Spectral Values used for the seismic design of buildings have been updated in the 2006 Edition of the IBC Code. The updated values are based on the 2004 Edition of the "National Earthquake Hazard Reduction Program (NEHRP) Recommended Provisions for Seismic Regulations for New Buildings and Other Structures – Part 1", Federal Emergency Management Agency (FEMA) Document 450. This document supersedes the 1998 version of the NEHRP/FEMA document, which is the basis of the 2003 Edition of the IBC Code. We understand that the updated FEMA guidelines are based on newer, more recent data provided by the United States Geological Survey (USGS). As design professionals we are of the opinion that use of the current values are appropriate for use in design of a building as they represent the latest science and data in the structural engineering field.

Our Code interpretations for seismic requirements, as they relate to this project are as follows:

1. As the change in occupancy does not place the existing structure in a higher occupancy category, a seismic upgrade of the lateral force resisting system to meet current Code requirements, due to change of occupancy, is not required.
2. As the proposed alterations will not increase the seismic force in any lateral force resisting element by more than 10%, or decrease the strength in any lateral force resisting element by more than 10% (with exception noted in item 3), a seismic upgrade of the lateral force resisting system to meet current Code requirements, due to alterations, is not required.
3. The southeast wall (water-side) is an exception to item 2. Proposed openings in this wall result in exceeding the 10%. Per previous conference call with Portland Code Enforcement, it is acceptable to reinforce the one affected wall to meet current Code requirements, without upgrading other areas of the structure, provided that we supply calculations that show the reinforced wall to meet current Code requirements.
4. New masonry stair and elevator shafts will be detailed as lateral force resisting members and will act to stiffen the existing structure and serve as voluntary improvements to the lateral force resisting system.

Thank you for your consideration and please let us know any questions or comments. We would be happy to meet and discuss further if desired.

Sincerely,
BECKER STRUCTURAL ENGINEERS, Inc.

Daniel S. Burne, P. E.
Associate

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Project: CCS	
W.O. 2314.10	Sheet 1 Of 21
Calculated By: NRM	Date 8-31-10
Checked By: PPB	Date 9.21.10

MODIFICATIONS TO EXISTING SOUTH WALL

SEE SKETCH BELOW FOR PROPOSED ALTERATIONS TO EXISTING SOUTH WALL. PER IBC 2006 SECTION 3403.2.3 "ALTERATIONS THAT ... DECREASE THE DESIGN STRENGTH OF ANY EXISTING STRUCTURAL ELEMENT TO RESIST SEISMIC FORCES BY MORE THAN 10 PERCENT ... SHALL NOT BE PERMITTED UNLESS THE ENTIRE SEISMIC-FORCE-RESISTING SYSTEM IS DETERMINED TO CONFORM TO ASCE 7 FOR A NEW STRUCTURE." PER CITY OF PORTLAND CODE ENFORCEMENT, IT IS ACCEPTABLE TO REINFORCE THE ONE AFFECTED WALL TO MEET CURRENT CODE REQUIREMENTS, WITHOUT UPGRADING OTHER AREAS OF THE STRUCTURE (SEE ATTACH LETTER/WAIVER).

PLEASE SEE ENCLOSE CALCULATIONS FOR SAID WALL REINFORC

SOUTH ELEVATION
1/8" = 1'-0"

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FROM ASCE 7-05 AS STATED IN IBC 2006

• SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD RESPONSE ACCELERATION PARAMETER (TABLE 11.6-1)

$S_{DS} = .481$ (SHEET 3)
OCCUPANCY CATEGORY II (TABLE 1-1)

⇒ SEISMIC DESIGN CATEGORY C

• SEISMIC DESIGN CATEGORY BASED ON 1-SECOND RESPONSE ACCELERATION PARAMETER (TABLE 11.6-2)

$S_{D1} = .179$ (SHEET 3)
OCCUPANCY CATEGORY II (TABLE 1-1)

⇒ SEISMIC DESIGN CATEGORY C

∴ SEISMIC DESIGN CATEGORY C SHALL BE USED TO DETERMINE DESIGN COEFFICIENTS & FACTORS FOR SEISMIC FORCE-RESISTING SYSTEMS (TABLE 12.2-1)

* TRY ORDINARY REINFORCED CONCRETE SHEARWALLS *

SEE 14.2 & 14.2.3.4 FOR DETAILING REQUIREMENTS

RESPONSE MODIFICATION FACTOR, $R = 4$
SYSTEM OVERSTRENGTH FACTOR, $\Omega = 2.5$
DEFLECTION AMPLIFICATION FACTOR, $C_d = 4$

DESIGN OF WALL GOVERNED BY ACI 318-05, CHAPTERS 1-18. NO SPECIAL DETAILING REQUIREMENTS MUST BE MET, WALL DESIGNED FOR APPLIED FORCES.

IBC 2006

CCS

2314.10 3 OF 21

NRM 8-31-10

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Spectral Response Accelerations Ss and S1
Zip Code - 4101
Zip Code Latitude = 43.658800
Zip Code Longitude = -070.257600
Ss and S1 = Mapped Spectral Acceleration Values
SiteClass B - Fa = 1.00, Fv = 1.00
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.2	0.314	Ss, SiteClass B
1.0	0.077	S1, SiteClass B

Period (sec)	Centroid Sa (g)	
0.2	0.314	Ss, SiteClass B
1.0	0.077	S1, SiteClass B

Period (sec)	Maximum Sa (g)	
0.2	0.317	Ss, SiteClass B
1.0	0.077	S1, SiteClass B

Period (sec)	Minimum Sa (g)	
0.2	0.313	Ss, SiteClass B
1.0	0.077	S1, SiteClass B

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Spectral Response Accelerations SMs and SM1
Zip Code - 4101
Zip Code Latitude = 43.658800
Zip Code Longitude = -070.257600
SMs = FaSs and SM1 = FvS1
Site Class E - Fa = 2.30, Fv = 3.50
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.2	0.721	SMs, Site Class E
1.0	0.269	SM1, Site Class E

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Spectral Response Accelerations SDs and SD1
Zip Code - 4101
Zip Code Latitude = 43.658800
Zip Code Longitude = -070.257600
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class E - Fa = 2.30, Fv = 3.50
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.2	0.481	SDs, Site Class E
1.0	0.179	SD1, Site Class E

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Project: CCS
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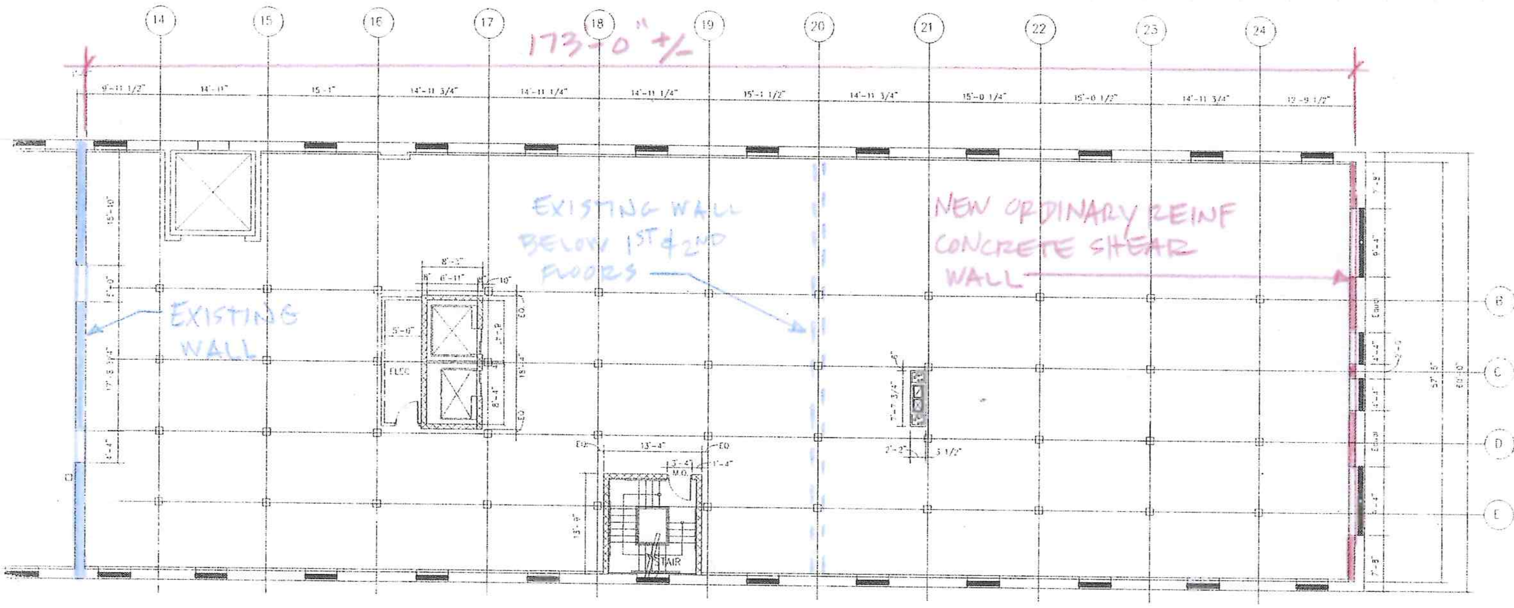
Checked By: PBB Date: 9.21.10

APPLIED SEISMIC FORCES

TRIBUTARY FLOOR AREA, $A = \left(\frac{173^{FT}}{2}\right)\left(60^{FT}\right) = 5190$ SF / FLOOR
2220 SF @ 1ST & 2ND

TRIBUTARY WALL AREA, $A_W = \left(173^{FT} + 60^{FT}\right)\left(12^{FT}\right) = 2800$ SF/FLO
1400 SF @ ROOF
1608 SF @ 1ST & 2ND

AVERAGE FLR-FLR HT
(SEE SHEET 1 FOR ELEV)



BUILDING WEIGHT (12.7.2)

20% FLAT ROOF SNOW

W_{ROOF} = 4.5 PSF + 0.7 PSF + 2.0 PSF + 6 PSF + 3 PSF + 3 PSF + 8.4 PSF = 27.6
6" RIGID INSUL, MEMBRANE, SHEATHING, ROOF JOISTS, ROOF BEAMS, GYP CEILING

PARTITION ALLOWANCE

W_{FLOOR} = 8 PSF + 14 PSF + 2 PSF + 2 PSF + 5 PSF + 10 PSF = 41 PSF (+63 PSF @ 1ST)
FLR DECK, FLR JOISTS, FLR BEAM, CARPET, FLOOR LEVELLING, CONC TOPPIN

W_{WALLS} = 115 PSF (3 WYTHE), 155 PSF (4 WYTHE), 190 PSF (5 WYTHE

W_{SW} = 100 PSF (8"), 150 PSF (12"), 200 PSF (16")

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BUILDING WEIGHT (CONT)

ROOF => $W = \underbrace{(28\ PSF)(5190\ SF)}_{ROOF} + \underbrace{(115\ PSF)(1400\ SF)}_{BRICK} + \overbrace{(200\ PSF)(360\ S}^{SHEARWALL}$

AREA OF SW

$W = 378.3^k$

5TH => $W = (41\ PSF)(5190\ SF) + (115\ PSF)(2800\ SF) + (200\ PSF)(720\ SF)$

$W = 678.8^k$

4TH => $W = (41\ PSF)(5190\ SF) + \left(\frac{115+155}{2}\right)(2800\ SF) + \left(\frac{150+200}{2}\right)(720\ SF)$

$W = 716.8^k$

3RD => $W = (41\ PSF)(5190\ SF) + (155\ PSF)(2800\ SF) + (150\ PSF)(720\ SF)$

$W = 754.8^k$

2ND => $W = (41\ PSF)(2220\ SF) + (155\ PSF)(1608\ SF) + (150\ PSF)(720\ SF)$

$W = 448.3^k$

1ST => $W = (104\ PSF)(2220\ SF) + \left(\frac{155+190}{2}\right)(1608\ SF) + (100\ PSF)(720\ SF)$

$W = 580.3^k$

TOTAL TRIBUTARY WEIGHT OF SHEARWALL

$W = 3557.3^k$ (NOT TAKING WINDOW PENETRATIONS INTO CONSIDERATION ∴ CONSERVATIVE)

- SEISMIC RESPONSE COEFFICIENT, $C_S = \frac{.481}{(4/1)} = .12$ (12.8-2)
- SEISMIC BASE SHEAR, $V = .12(3557.3^k) = 427.8^k$ (12.8-1) (ULTIMATE)

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VERTICAL DISTRIBUTION

VERTICAL DISTRIBUTION FACTORS (12.8-12)

$$C_{VR} = \frac{378.3^k (70^{Ft})}{136201^{k \cdot Ft}} = .194$$

$$C_{V5} = \frac{678.8^k (56^{Ft})}{136201^{k \cdot Ft}} = .279$$

$$C_{V4} = \frac{716.8^k (44^{Ft})}{136201^{k \cdot Ft}} = .232$$

$$C_{V3} = \frac{754.8^k (34^{Ft})}{136201^{k \cdot Ft}} = .188$$

$$C_{V2} = \frac{448.3^k (22^{Ft})}{136201^{k \cdot Ft}} = .072$$

$$C_{V1} = \frac{580.3^k (8^{Ft})}{136201^{k \cdot Ft}} = .034$$

LATERAL SEISMIC FORCES (12.8-11) - (ULTIMATE)

$F_R = .194 (427.8^k) = 83.0^k$

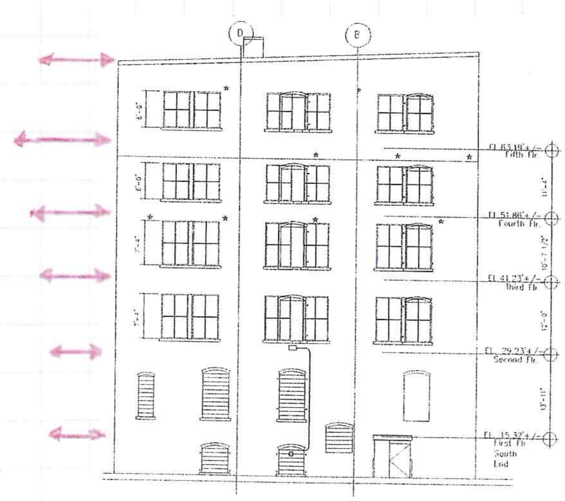
$F_5 = .279 (427.8^k) = 119.4^k$

$F_4 = .232 (427.8^k) = 99.3^k$

$F_3 = .188 (427.8^k) = 80.4^k$

$F_2 = .072 (427.8^k) = 30.8^k$

$F_1 = .034 (427.8^k) = 14.5^k$



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OVERTURNING MOMENT

$$M_o = 83^k(70^{Ft}) + 119.4^k(56^{Ft}) + 99.3^k(44^{Ft}) + 80.4^k(34^{Ft}) + 30.8^k(22^{Ft}) + 14.5^k(8^{Ft})$$

$M_o = 20392.8^{k \cdot FT}$ (ULT)
$14274.4^{k \cdot FT}$ (SERVICE)

$M_o = 5098.2^{k \cdot FT}$ (4 INDIVIDUAL WALLS)

RESISTING MOMENT

SELF WT OF WALL, $W_{SW} = (720\ SF)(115\ PSF + 200\ PSF)(2) + (720\ SF)(155\ PSF + 150\ PSF)(3) + (720\ SF)(190\ PSF + 100\ PSF)(1)$

$W_{SW} = 1321.2^k - (10^{Ft} \times 7^{Ft})(315\ PSF)(15)$

PENETRATION SIZE

\# OF PENETRATIONS

$W_{SW} = 990.5^k$ (16.5 KLF)

$M_R = .6(990.5^k)(30^{Ft}) = 17,829^{k \cdot FT}$ (SERVICE) $> M_o$ ∴ NO NET OVERTURNI[NG] & NO UPLIFT

CHORD FORCE

$T/C = 20392.8^{k \cdot FT} / 60^{FT} = 339.9^k$ (ULT)
$= 237.9^k$ (SERVICE)

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Project: CCS
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Checked By: PBB Date 9.21.10

8" ORDINARY REINFORCED CONCRETE SHEARWALL

$V_u = 427.8^k$ (ALL REFERENCES FOR WALL DESIGN FROM ACI 318-05)

SHEAR — CHECK FRACTURE PLANE @ WINDOWS

EFFECTIVE WALL LENGTH $L_{EFF} = 60^{FT} - (9'-4") - (10'-8") - (9'-4") = 30.66^{FT}$

$d = .8(30.66^{FT}) = 24.5^{FT}$ (11.10.4)

CONCRETE SHEAR STR. $V_c = 2\sqrt{f'_c}\,hd$ (11.10.5)

$= 2\sqrt{5000_{psi}}\,(8")(24.5^{FT} \times 12) = 332.6^k < 427.8^k$

$\phi V_c = .75\,(332.6^k) = 249.5^k$
(PER 9.3.2.3)

STEEL SHEAR STR. $\phi V_s = \underset{V_u}{427.8^k} - \underset{\phi V_n}{249.5^k} = 178.3^k$ (11.1.1)

TRY #5 REINF (HORIZONTAL STEEL) (11.10.9.1)

$s = \dfrac{(.31\,in^2)(60ksi)(24.5^{FT} \times 12)}{178.3^k} = 30.7"$ O.C. ∴ 12" O.C. OK✓

$V_n \leq 10\sqrt{f'_c}\,hd$

$\leq 10\sqrt{5000}\,(8")(24.5^{FT} \times 12) = 1663^k \gg V_u$ ∴ OK✓

$A_{V\,min} \geq .0025\,(12" \times 8") = .24\,in^2/FT < .31$ ∴ OK✓
(11.10.9.2)

VERT STEEL (11.10.9.4 & 11.10.9.5)

$\rho_l \geq .0025$ ∴ #5 @ 12" O.C. OK✓ $\rho_l = .0032$

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PROPORTIONING OF BOUNDARY ELEMENTS

CHORD FORCES, $T/C = 339.9^k$ (SHEET 7)

ASSUME MEMBERS W/ TIE REINFORCEMENT

COMP CAPACITY: PURE AXIAL $\phi P_n = .80\phi[.85f'_c(A_g - A_{ST}) + f_y(A_{ST})]$ (10.3.6.2)

$339.9^k = .8(.65)[.85(5\,ksi)(A_g)]$ (PER 9.3.2.2) ASSUME NO STL REINF (CONSERVATIVE)

$A_g = 153.8\ in^2$ ∴ 12"×16" BOUNDARY MEMBER OK✓ FOR COMP.

FLEXURAL TENSION CAPACITY $M_u = 20{,}392^{k\cdot FT}$

$\phi M_n = .9 A_s f_y(d - \frac{a}{2}) \geq M_u$

∴ $20392^{k\cdot FT} = .9(A_s)\,60\,ksi\,(648")$ → .9·d APPROXIMATION d = 60 FT

$A_s = 7.0\ in^2$

$A_{s\,ACTUAL} = .31\,in^2(60) = 18.6\ in^2$ → @12" o.c. d = 30 FT

$a = \frac{18.6\,in^2(60\,ksi)}{.85 \times 5 \times 8"} = 32.8"$ $d - \frac{a}{2} = 343.6"$

$\phi M_n = .9(18.6\,in^2)(60\,ksi)(343.6") = 28759.3^{k\cdot FT} > 20392$

CHECK MINIMUM STEEL (10.5.1)

$A_{s_{min}} = \max \begin{cases} \frac{3\sqrt{5000}}{60000} \times 8" \times 360" = 10.1\,in^2 < 18.6\,in^2 \\ \frac{200 \times 8" \times 360"}{60000} = 9.6\,in^2 \end{cases}$ OK✓

MAX STEEL (10.3.2-10.3.5) $\varepsilon_t = \frac{360" - 41"}{41"} \times .003 = .023 > .005$ ∴ $\phi = .9$ OK✓

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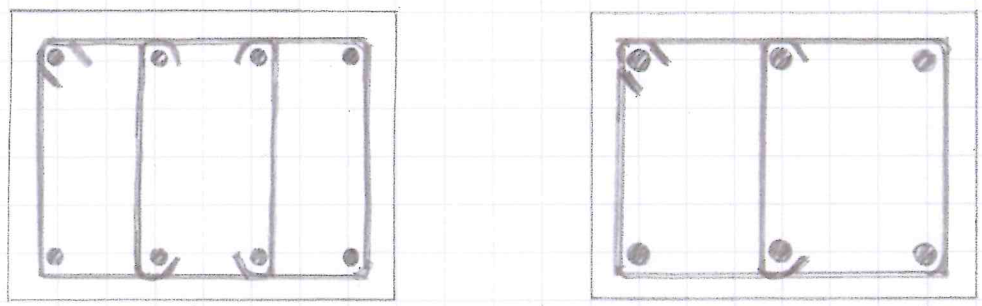
MINIMUM STEEL IN COMPRESSION MEMBR (10.9.1)

LONGITUDINAL STEEL $A_{ST} > .01\,A_g = .01(12" \times 16") = 1.92\ in^2$

$\therefore$ 8 - #5 OK✓ $A = 2.48\ in^2$
6 - #6 OK✓ $A = 2.64\ in^2$



LESS TIES $\therefore$ THIS DETAIL

CONFINEMENT STEEL #3 TIES (7.10.5.1)

$$s \le \min \begin{cases} 16 d_b = 16(.75) = 12" & (7.10.5.2) \\ 48(d_t) = 48(.375) = 18" \\ 12" \end{cases}$$

$\therefore$ SPACE @ 12" O.C

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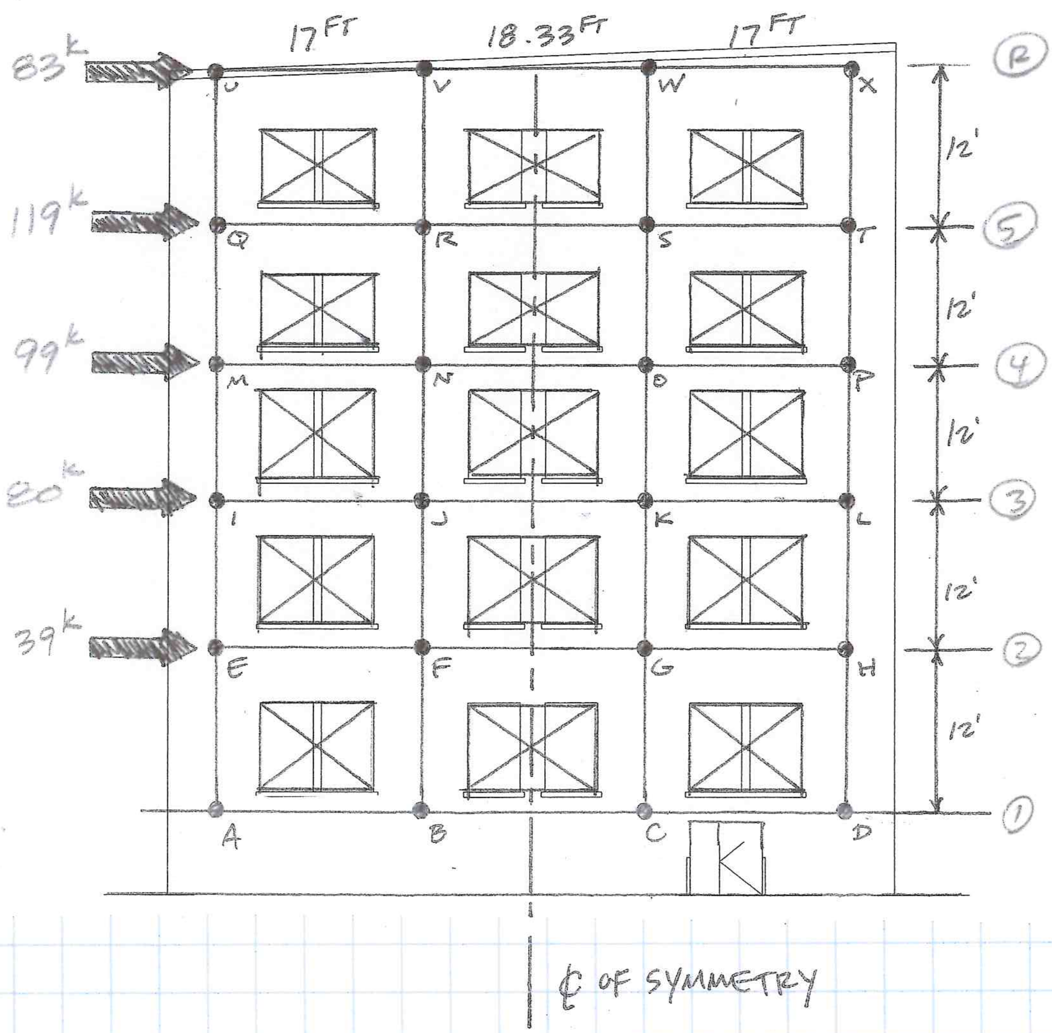
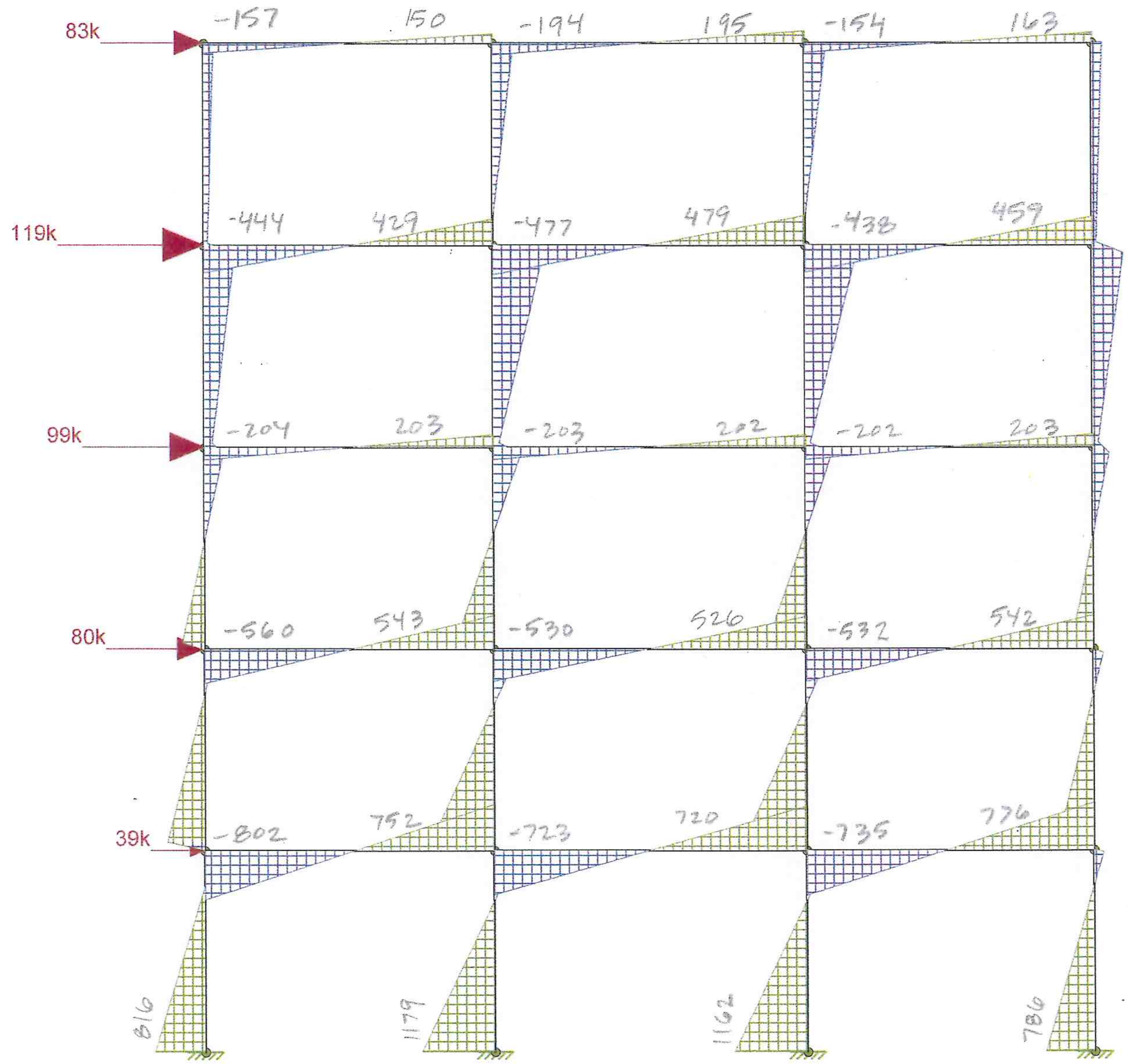
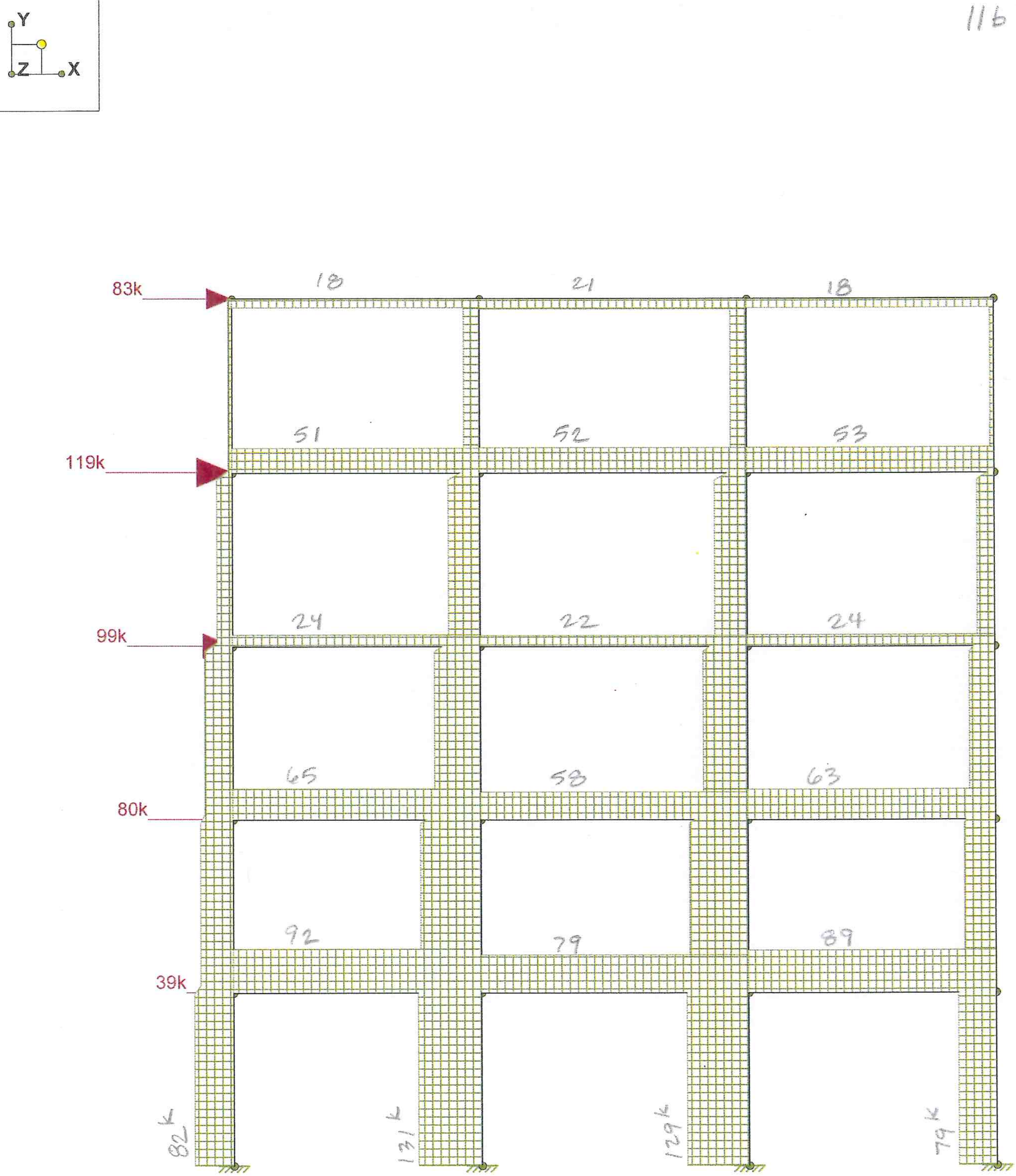
EVALUATION OF PENETRATIONS IN WALL

ASSUME WALL BEHAVES AS A RIGID FRAME. USING MODELING SOFTWARE, WE CAN DETERMINE FORCES IN COUPLING BEAMS & WALL SEGMENTS. SEE DIAGRAM BELOW FOR FRAME NODE APPROXIMATIONS & LOADING.

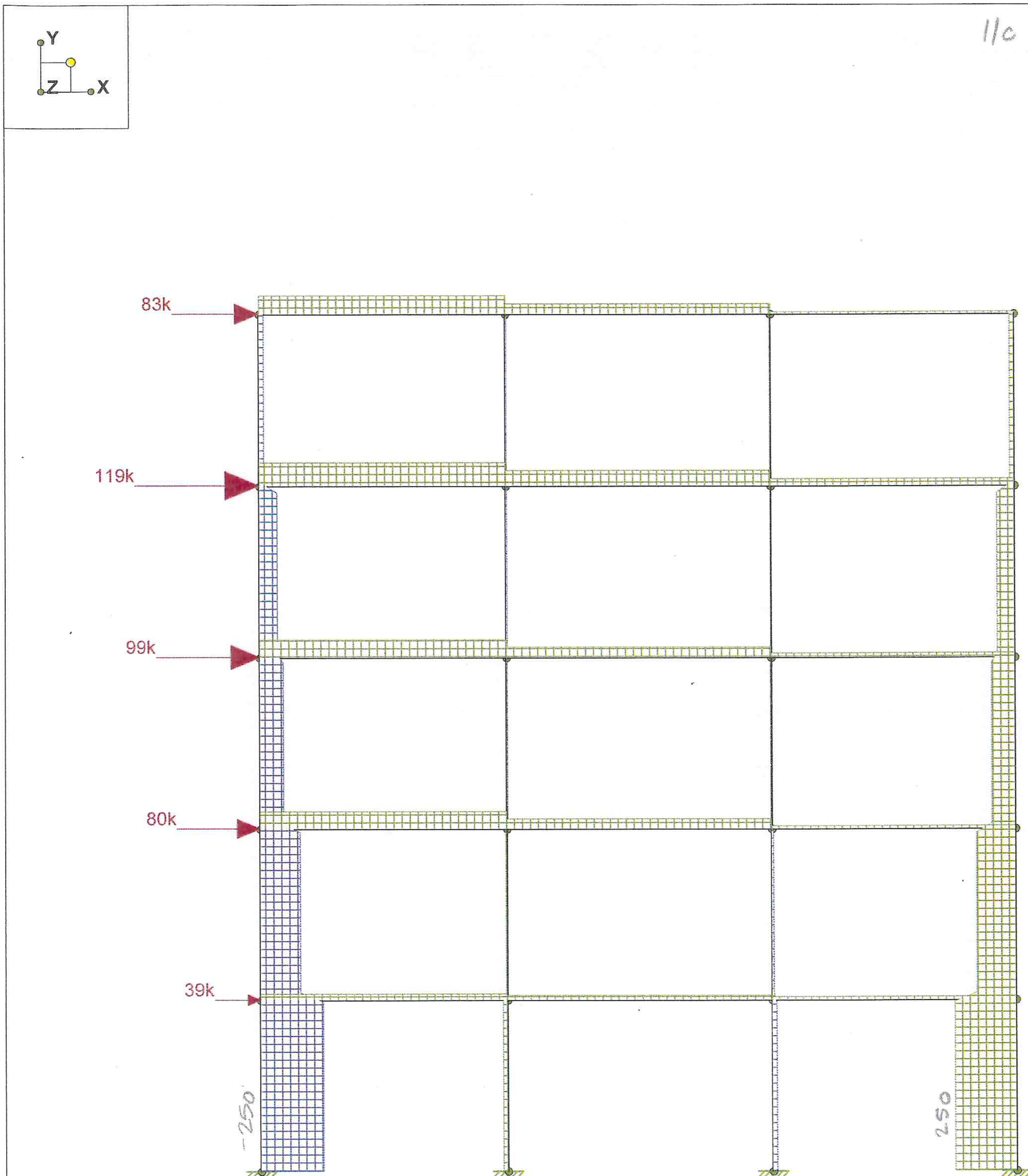




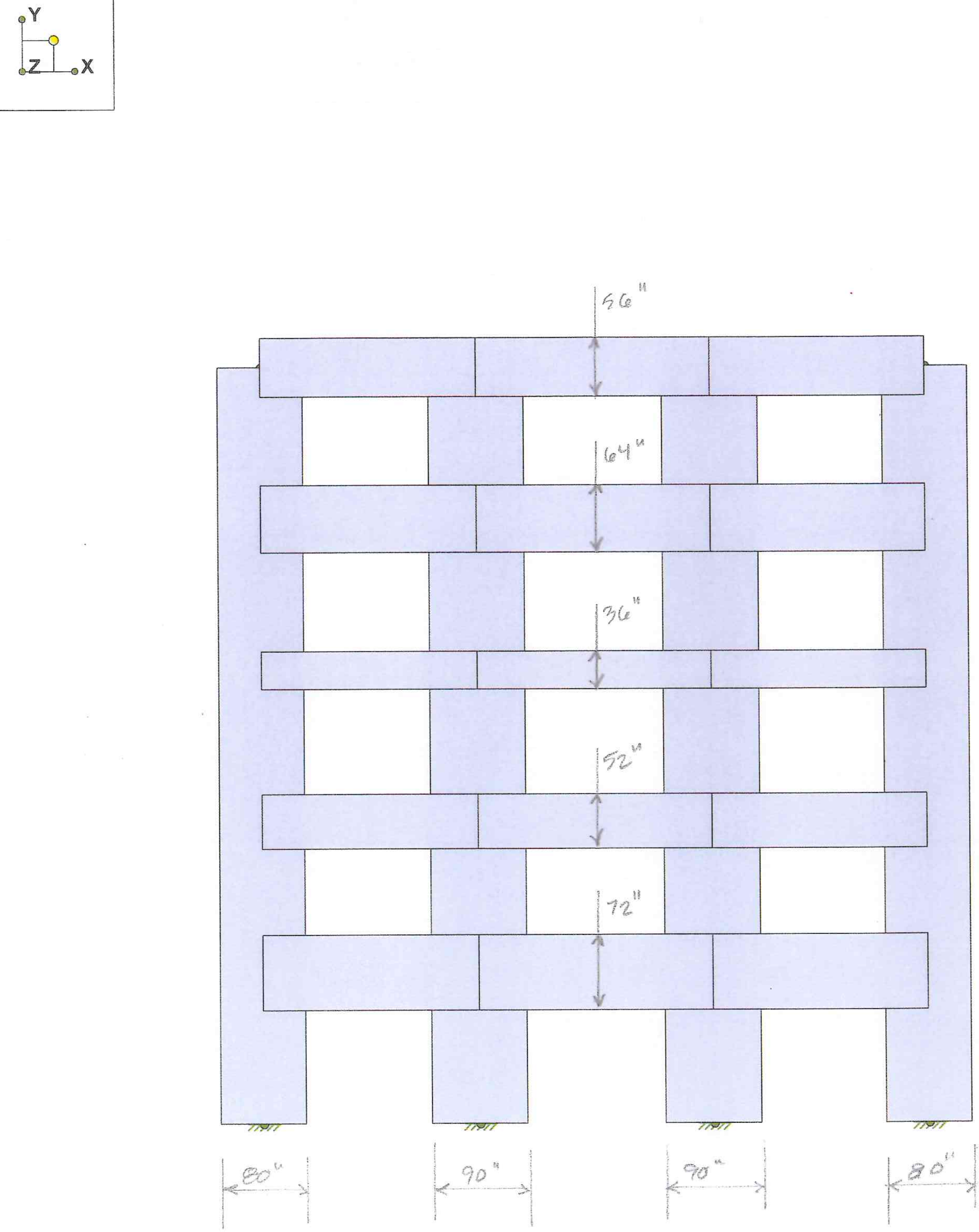
Loads: BLC 1, 1
Results for LC 1, 1
Member z Bending Moments (k-ft)



Loads: BLC 1, 1
Results for LC 1, 1
Member y Shear Forces (k)



Loads: BLC 1, 1
Results for LC 1, 1
Member Axial Forces (k)



Results for LC 1, 1

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WALL SEGMENT DESIGN

SHEAR $V_u = 427.8^k \approx 82^k + 131^k + 129^k + 79^k$ ∴ PREVIOUS SHEAR DESIGN OK✓

$\uparrow$ SEE SHEET 8 $\uparrow$ SEE SHEET 11b

AXIAL $P_u = 250^k$ +/- $< 339.9^k$ ∴ PREVIOUS AXIAL COMP DESIGN OK✓

$\uparrow$ SEE SHEET 9

$P_u = -250^k + 300\,PSF\,(72^{FT} \times 12^{FT}) = 9200^{\#}$ ∴ NO UPLIFT DESIGN OK✓ FOR AXIAL TENSION

$\uparrow$ WALL S.W. $\uparrow$ WALL SIZE

BENDING $M_u = 1180^{k \cdot FT} = 14160^{k\,in}$ (MIDDLE WALL)

$A_{s_{REQ'D}} = 14160^{k\,in} / (.9 \times 60\,ksi \times 72'') = 3.64\ in^2 \Rightarrow \underline{5\#8}$

$\uparrow$ $.9d = .9(80'')$ APPROXIMATION

$$a = \frac{3.95\,in^2 \times 60^{ksi}}{.85 \times 5\,ksi \times 8''} = 6.97''$$

$d = 90'' - 1.5'' - 3'' - 3'' = 82.5''$

$\uparrow$ COVER $\uparrow$ C-C SPACING

$$\phi M_n = .9\,(60\,ksi)(3.95\,in^2)\left(82.5'' - \frac{7''}{2}\right) = 16850^{k\,in} > 14160^{k\,in}\ \underline{OK}$$

$M_u = 816^{k \cdot FT} = 9792^{k\,in}$ (OUTER WALL)

$A_{s_{REQ'D}} = 9792^{k\,in} / (.9 \times 60\,ksi \times 63'') = 2.88\ in^2 \Rightarrow \underline{4\#8}$

$\uparrow$ $.9(70'')$

$$a = \frac{3.16\,in^2\,(60\,ksi)}{.85 \times 5\,ksi \times 8''} = 5.58''$$

$d = 76'' - 1.5'' - 3'' - 1.5'' = 70''$

$$\phi M_n = .9\,(60\,ksi)(3.16\,in^2)\left(70'' - \frac{5.6''}{2}\right) = 11467^{k\,in} > 9792\ \underline{OK✓}$$

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COUPLING BEAM DESIGN

CONTROLLING SECTION FOR BENDING, SHEAR & AXIAL FORCES CAN BE FOUND ON 11a - 11d

SHEAR

$V_u = 92^k$

$\phi V_c = .75\left(2\sqrt{5000} \times 8'' \times 72''\right) = 61^k$ (ACI 318-05, 11.10

$\phi V_s = 92^k - 61^k = 31^k$ ∴ $V_s = 41.3^k$

$s = \dfrac{(.31\ in^2)(60\ ksi)(72'')}{41.3^k} = 32.4''$ ∴ #5 @ 12" o.c.

(ACI 318-05, 11.10.9.1) OK✓

AXIAL

LOAD ENTERS WALL THRU DIAPHRAGM - NOT AS DISCRETE POINT LOAD EACH LEVEL

$P_u = 119^k / \underset{\uparrow}{3} = 40^k$

OF COUPLING BMS PER FLOOR

$A_g = \dfrac{40^k}{.8(.65)(.85 \times 5\,ksi)} = 17.9\ in^2$ (ACI 318-05, 10.3.6.2)

$A = 8'' \times 36'' = 288\ in^2 > 17.9$ ∴ OK✓

BENDING MOMENT

$M_u = 802^{k \cdot ft} = 9624^{k \cdot in}$

$A_s = \dfrac{9624^{k\,in}}{.9(60)(59'')} = 3.02\ in^2$ TRY 4 #8 $A = 3.16\ in^2$

↳ .9d (.9×66")

$d = 72'' - (1.5'' - 3'' - 1.5'') = 66''$ ← COVER

$a = \dfrac{3.16(60)}{.85 \times 5\,ksi \times 8''} = 5.58''$

$\phi M_n = .9(60)\left(66'' - \dfrac{5.6''}{2}\right)(3.16\ in^2) = 10784^{k \cdot in} > 9624^{k \cdot in}$ OK✓

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FOUNDATION LOADS

END WALL TO BE SUPPORTED @ MIDDLE & ENDS W/ (3) DISCRETE PILE GROUPS & PILE CAPS.

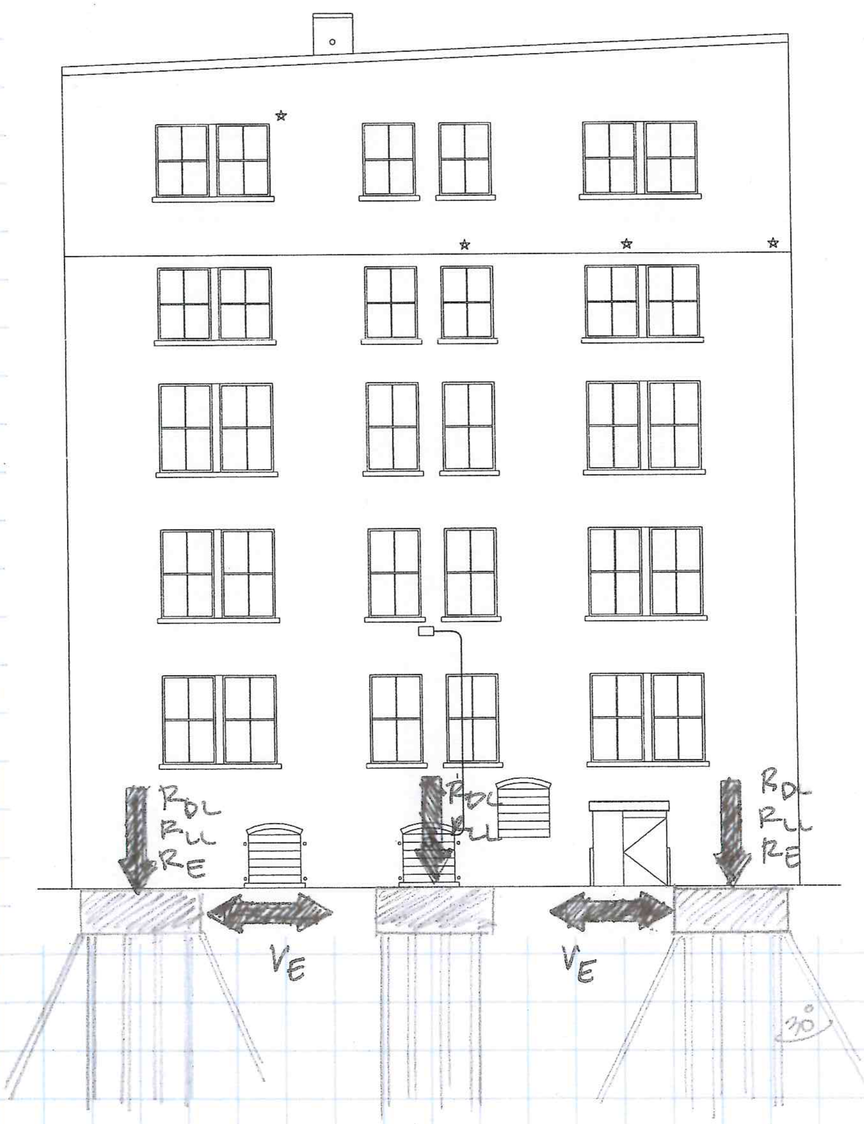
$W_{SW} = 992.5^k$ (SEE SHEET 7) + 31 PSF (6 FT)(60 FT)(5) = 1046^k

(5 = # FLOORS; 31 PSF = W_{FLOOR} MINUS PARTITION ALLOWANCE (SEE SHT 4))

$R_{DL} = 1046^k/2 = 523^k$ @ MIDDLE PILE CAP (261.5^k EA END)

$R_{LL} = \dfrac{65 \text{ PSF } (6\text{FT})(60\text{FT})(5) + 42 \text{ PSF } (6\text{FT})(40\text{FT})}{2} = 64^k$ @ MIDDLE (32^k EA END)

(65 PSF = LIVE LOAD + PARTITION ALLOWANCE)



$R_E = 237.9^k$ (EA END PILE CAP) (SERVICE) TO OCCUR @ ONLY (1) PILE CAP @ ANY GIVEN MOMENT (SHEET

$V_E = \dfrac{.7(427.8^k)}{2} = 150^k$ (EA END PILE CAP) (SERVICE)

ASSUMING 80^T PILE CAPACITY

$P_{MID} = 523^k + 64^k = 587^k$

$P_{END} = 589^k$

ASSUME $15^{FT} \times 12^{FT} \times 4^{FT}$ PILE CAP @ ENDS
$8^{FT} \times 12^{FT} \times 3^{FT}$ @ MID

$P_{MID} = 587^k + 104^k = 691^k$

∴ 5 PILES OK ✓ FOR GRAVITY

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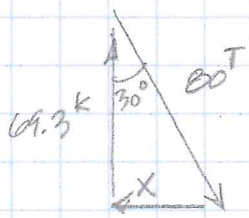
FOUNDATION LOADS (CONT)

IF BATTERED PILES @ 30° MIN SLOPE, AXIAL COMP ONLY



$X = 80^T \sin 30 = 40^T$

$\# \text{PILES} = 150^k / 40^T = 1.875$

∴ 2 BATTERED PILES EACH PILE CAP, EACH DIRECTION OK ✓

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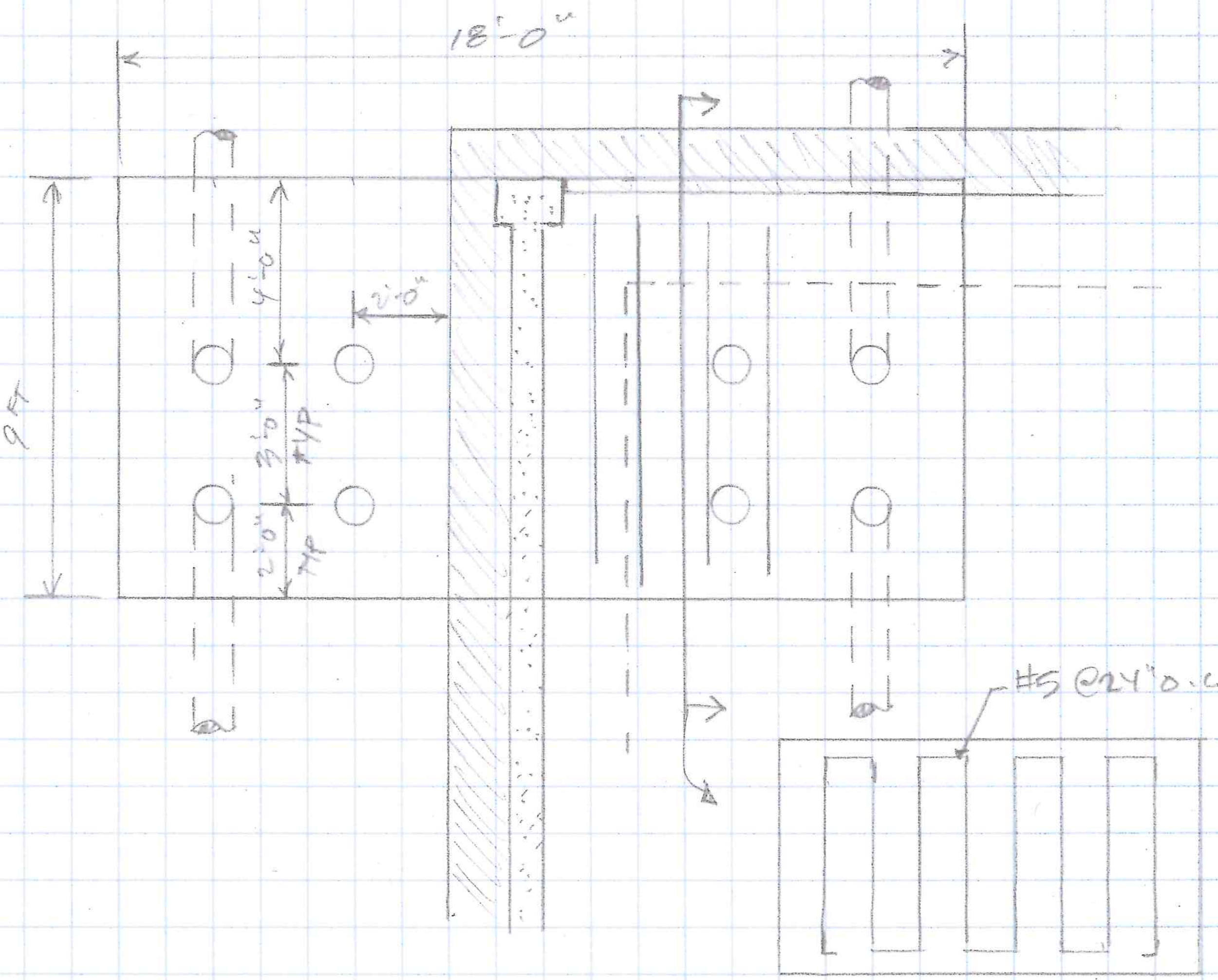
PILE & CAP LAYOUT @ WALL ENDS



PC8 18'-0" x 9'-0" x 5'-0" THICK
W/ 23 #8 L.W.B & 30 #8 S.W.B
W/ 180° HOOKS EA END

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PILE CAP DESIGN (PC8)

* ONE-WAY BEAM SHEAR

$$V_u = 1.6\left[(2)(160^k) + 160\cos 30\right] = 734^k$$

↑ VERT COMP OF BATTERED PILES

$$d = 60'' - 10'' = 50''$$

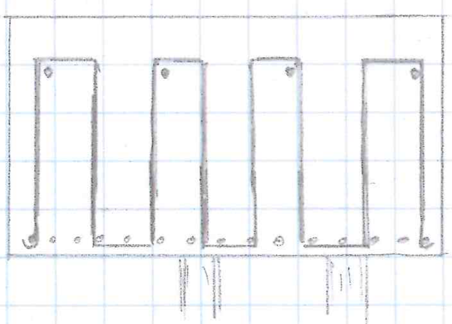
$$b_o = 9^{FT} \times 12 = 108''$$

$$\phi V_c = .85 \times 2 \times \sqrt{3000} \times 108'' \times 50'' = 503^k$$

$$V_s = \frac{734 - 503}{.85} = 272^k$$

$$S_{max} = \min \begin{cases} d/2 = 50''/2 = 25'' \\ 24'' \Leftarrow \text{CONTROLS} \\ \dfrac{f_y A_v}{50 b_w} = \dfrac{60(2.48)}{50(9 \times 12)} = 27.5'' \end{cases}$$

$$A_v = \frac{272^k(24'')}{60ksi(50'')} = 2.176 in^2 \therefore 8 \# 5$$



#5 BENT AS SHOWN = 8 #5

* PUNCHING SHEAR

NO PUNCHING SHEAR FAILURE MODE SINCE LOADING IS RESULT OF WALL, NOT DISCRETE COLUMN.

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* FLEXURAL REINFORCEMENT - LONG BARS

$M_u = 1.6\left[(2 \times 160^k \times 4^{FT}) + (160\cos 30 \times 7^{FT})\right] = 3600^{k \cdot FT} = 43200^{k \cdot in}$

$M_u = 4800^{k \cdot in}/FT$

$d = 60'' - 9.5 = 50.5''$

$A_s = \frac{4800}{.9(60ksi)(50.5'')} = 1.76\ in^2/FT$

TOTAL $A_s = 15.84\ in^2 < .0033(9^{FT})(12'')(50.5'') < \frac{4}{3}(15.84)$
$= 18\ in^2$ $= 21.12$

$\therefore A_s = 18\ in^2 \Rightarrow 23\ \#8$ w/ 180° HOOKS EA END

* FLEXURAL REINFORCEMENT - SHORT BARS

$M_u = 1.6\left[(2 \times 160^k \times 1.5^{FT}) + (160\cos 30 \times 1.5^{FT} \times 2)\right] = 1433^{k \cdot FT}$
$17197^{k \cdot in}$

$M_u = 955^{k \cdot in}/FT$

$d = 49.5''$

$A_s = \frac{955^{k \cdot in}}{.9(60ksi)(49.5'')} = .36\ in^2/FT$

A_s TOTAL $= 6.43\ in^2 < .0018(18^{FT} \times 12)(60'') = 23.3\ in^2$

$\therefore A_s = 23.3\ in^2 \Rightarrow 30\ \#8$ OR $54\ \#6$

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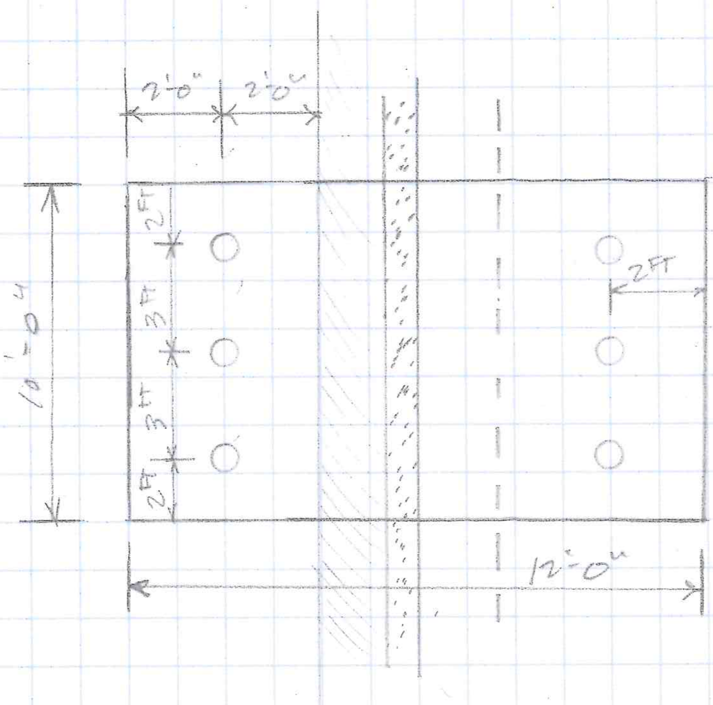
PILE & CAP LAYOUT @ WALL



PC6 12'-0" x 10'-0" x 5'-0" THICK
26 #8 L.W.B. & 20 #8 S.W.B.
W/ 180° HOOKS EA END
W/ 8 #5 VERT BARS FOR SHEAR

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PILE CAP DESIGN (PC6)

* FLEXURAL REINF - LONG BARS

$M_u = 1.6(3)(160^k)(4^{FT}) = 3072^{k \cdot FT} = 36864^{k \cdot in}$

$M_u = 36864^{k \cdot in} / 10^{FT} = 3686.4^{k \cdot in} / FT$

$d = 60'' - (6'' + 3'' + .5'') = 50.5''$

PILE EMBED, CLR, BAR THICKNESS (#8 ASSUMED)

$A_s = \dfrac{3686^{k \cdot in}}{.9(60ksi)(45.5'')} = 1.50 \ in^2/FT$

$45.5''$ = .9d

TOTAL $A_s = 1.50 \ in^2 (10'-0'') = 15.0 \ in^2 \left(\frac{4}{3}\right) > .0033(10^{FT} \times 12)(50.5'')$
$= 19.99 \ in^2$

$\therefore$ 20.0 in² OK

=> 26 #8 w/ 180° HOOKS EA END

* FLEXURAL REINF - SHORT BARS

$M_u = 1.6(2)(160^k)(3.0^{FT}) = 1536^{k \cdot FT} = 18432^{k \cdot in}$

$M_u = 18432^{k \cdot in} / 12^{FT} = 1536^{k \cdot in} / FT$

$d = 60'' - (6'' + 3'' + 1'' + .5'') = 49.5''$

$A_s = \dfrac{1536^{k \cdot in}}{.9(60ksi)(44.6)} = .64 \ in^2/FT$

44.6 = .9d

TOTAL $A_s = .64 \ in^2 (12^{FT}) = 7.65 \ in^2 \left(\frac{4}{3}\right) = 10.2 \ in^2 < .0018(12^{FT} \times 12)(60$
$= 15.55 \ in^2$

$\therefore A_s = 15.55 \ in^2$ => 20 #8 OR 36 #6 w/ 180° HOOKS EA END

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* PUNCHING SHEAR

NO PUNCHING SHEAR FAILURE MODE SINCE LOADING IS RESULT OF WALL, NOT DISCRETE COLUMN. SEE ONE-WAY BEAM SHEAR FOR SHEAR FAILURE MODE.

* ONE-WAY BEAM SHEAR

$$V_u = 1.6(3)(160^k) = 768^k$$

$$d = 50.5''$$

$$b_o = 10^{ft}(12'') = 120''$$

$$\phi V_c = .85 \times 2 \times \sqrt{3000} \times 120'' \times 50.5'' = 564^k < 768^k$$

NO GOOD

$$V_s = \frac{768^k - 564^k}{.85} = 240^k$$

$$S_{max} = \min \begin{cases} d/2 = 50''/2 = 25'' \\ 24'' \Leftarrow \text{CONTROLS} \\ \dfrac{f_y A_v}{50 b_w} = \dfrac{60(2.48 in^2)}{50(10 \times 12)} = 24.8'' \end{cases}$$

$$A_v = \frac{240^k (24'')}{60ksi(50'')} = 1.92 in^2 \therefore 7 \#5 \quad \text{USE } 8\#5$$