Milton S. Hershey Medical Center Biomedical Research Building
Hershey, Pennsylvania

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17 September 2012

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Executive Summary:

A detailed analysis was conducted in order to get a general feel of the building, the codes it was designed under relative to the current codes used today. It was found that there is a good chance that the building is out of date in relation to modern codes, and thus it is difficult to discern which popular design method, ASD or LRFD was used to design the building. In regards to the spot checks performed, the caisson passed the first axial check at its 84 inch diameter section with a load of 2425 kips being supported by a maximum of 4708 kips provided by the caisson. It failed at the second section, with a load of 2425 kips being support by a maximum 1946 kips. The column supports its axial load of 623 kips with a maximum of 2000 kips, but fails in the maximum possible moment should a severe loading difference occur with a maximum moment of 175 ft kips supporting a moment of 479 ft kips. Checking the punching shear of the slabs found that the slab can support the shear force about the shear reinforcement about the column, with a 453 kip maximum supporting a 267 kip shear force. It, however, fails at the face of the column, with 297 kips pushing through 202 kips supplied by the face of the support. The beam checked failed at the center of the span, with it being designed for only 293 ft kips, but having a positive moment of 330 ft kips being applied, while the 479 ft kip negative moment was successfully supported by reinforcement that could provide a maximum of 559 ft kips. Shear reinforcement was not sufficient, with a 124.5 kip load pushing through a slightly less 122.1 kip reinforcement. Development length of the negative moment was checked as well, and it was found that the 7 foot 1 inch provided length was greater than the 5 foot 2 inch requirement. Deflection was determined not necessary to calculate given the depth of the beam. There were a number of hits and misses with being up to code, and some aspects that were not up to code, but that can simply be a difference of codes utilized, different loads applied, different design methods being used, ASD vs. LRFD, and human error. Finally, checks between seismic and wind loads applied at each floor level, it was determined that seismic forces control over the wind forces that could be seen by this building.

Building Summary:

The Milton S. Hershey Medical Center Biomedical Research Building in Hershey, Pennsylvania, is an education and research facility. It is owned by the Milton S. Hershey Medical Center, and is part of Penn State Hershey, and thus is a branch campus of Pennsylvania State University. It is a 110' tall structure with 7 stories and 245000 total square feet of floor space. It was constructed by Alexander Building and Shoemaker Construction Companies and managed by Alvin H. Butz, Inc. between 1991 and 1993, costing $49 million. It was designed by Geddes Brecher Qualls Cunningham, and engineered by The Sigel Group and Earl Walls Associates. The most distinguishing architectural aspect of the building is a large cylinder that extends from the 2nd floor up to the roof on one of the corners of the building.

Foundation System:

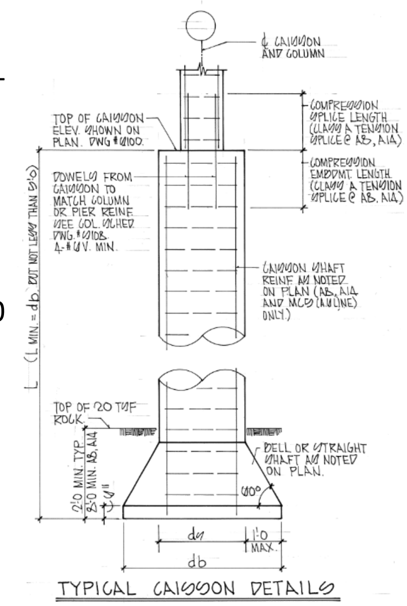


Figure 1. Typical Caisson Detail

The Biomedical Research Building at Penn State Hershey utilizes a simple monolithic concrete structure to serve its load distribution needs. This structure stands on a series of large, 3 to 7 and a half foot diameter caissons which loads ranging from 250 kips to 1610 kips, with most loads around 1000 kips expected by the building's original engineers. These caissons have a 40 kip per square foot requirement, using 3000 psi 28 day strength concrete, and are set into the bedrock below. It should be noted that even though 3000 psi concrete was called for, there was an instance where 1000 psi concrete was called for in the plans. A variety of different sized 60ksi steel rebar are utilized in reinforcing both the caissons and the grade beams, with clear cover at 2.5 inches, given its exposure to ground.

Caissons were chosen as the building's foundation, as the area is known to have large sink holes develop within the limestone deposits. This prevents future sinkhole development underneath or nearby to have any drastic effect on the Biomedical Research Building's safety, especially as sinkholes are not usually detected until it is too late. As seen in figure 2, grade beams act to transfer forces from the columns into the caissons when columns and caissons do not line up, and to further the idea of sink hole damage prevention, using beams varying from 14 inches wide by 30 inches deep to 7 feet by 16 foot 8 inches deep.

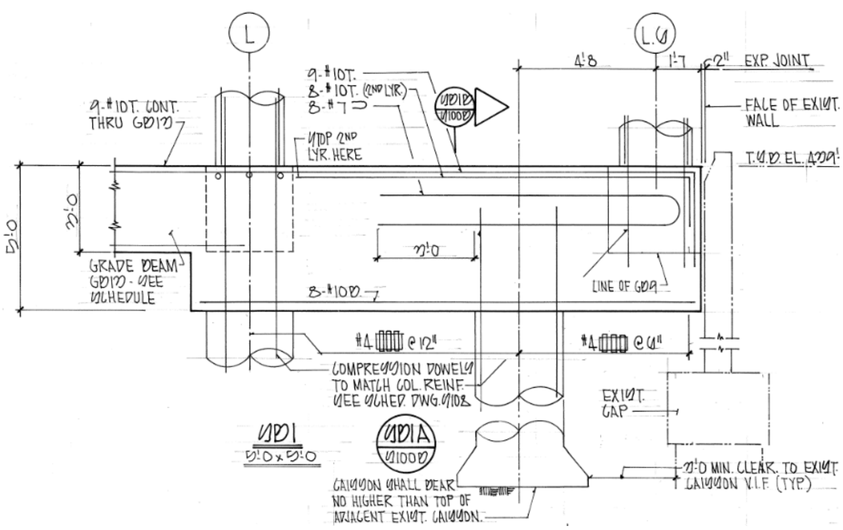


Figure 2. Example of caisson and column misalignment

General Floor Framing:

Floors of the Biomedical Research building are supported by large beams typically spanning 20' that predominately go in the longitudinal direction of the building for the central part, and in the far ends of the building. These beams vary from 12 to 36 inches deep, and 3 to 8 feet wide. There obviously were some depth restrictions where the 8 foot wide beams are located. Shown in Figure 3 on the next page, the building is effectively cut into 3 sections by two set of three openings in the floors, with columns and beams on all sides of these openings. These openings are to serve the building in its HVAC, plumbing and electrical needs. Additional openings in the floor are directly adjacent to these service openings, for elevator shafts that serve the entirety of the building. These elevator shafts have two additional columns to help support the concentrated load of the elevator and its machinery, distributing the load around the openings.

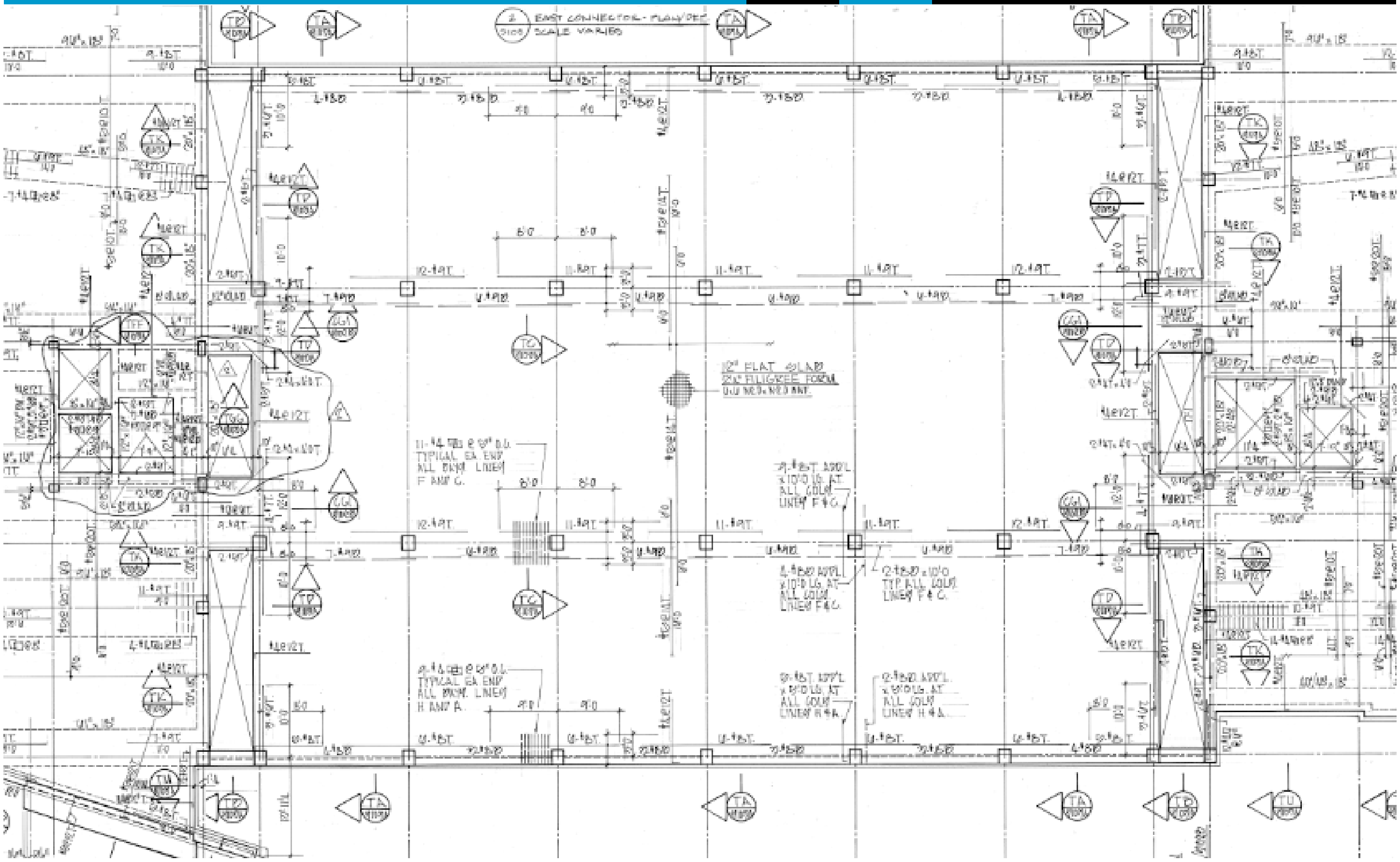


Figure 3. Typical Floor Plan - The three vertical openings on each side are for HVAC, electrical, and mechanical usage, and the openings just to the outside of these openings are elevator shafts.

Beams use rebar at the top and bottom of the beam to resist positive and negative moments, and such reinforcement is usually discontinued at some point after development length has been achieved. Shear reinforcement is used in the form of stirrups, using #3 or #4 sized rebar with 40ksi steel. There are no drop panels used, and as found in the calculations on page # in Appendix #, the building would benefit from drop panels.

Supporting the beams are a multitude of columns, averaging about 2 feet by 2 feet in dimension. Circular columns are also used, and average about 30 inches in diameter. 60ksi rebar are used to reinforce the columns, with varied sizes and number of rebar utilized. Clear cover for the columns and beams inside of the building is at 1.5 inches.

Floor Systems:

On these beams are a system of one way slabs designed to support 100 to 125 psf floor loads, using 4000 psi 28 day strength concrete, with temperature reinforcement and a 6x6 W2.0xW2.0 WWF. The one way slabs are oriented perpendicular to the beams, and are treated as beams in that direction. On the ground level, where large mechanical equipment is located, slabs are thickened according to the size and weight of the machinery, as applicable.

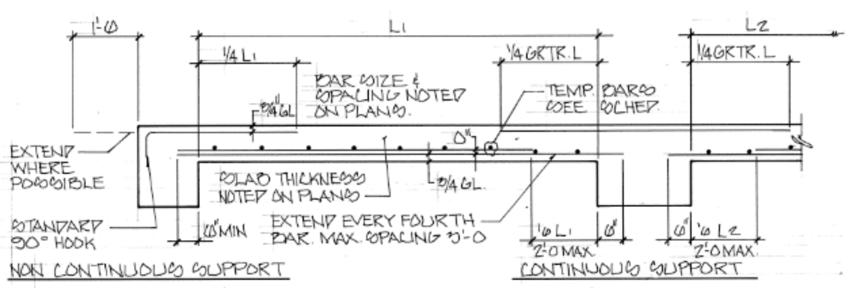


Figure 4. Typical Slab Detail

Expansion joints:

There are no expansion joints, but there is temperature reinforcement to handle the stresses of expansion and contraction of the building. In addition, there are also control joints that are designed to mitigate and control potential cracking in the building, which would include crack development due to temperature change. A typical control joint detail is shown below.

TEMPERATURE BARS	
SLAB THK.	REINF.
4" LESS THAN 5"	#3@12"
5" - 6"	#4@18"
6" - 7"	#4@15"
7" - 8"	#4@15"
8" - 9"	#4@11"
9" - 10"	#4@10"

Figure 5. Temperature Reinforcement Schedule

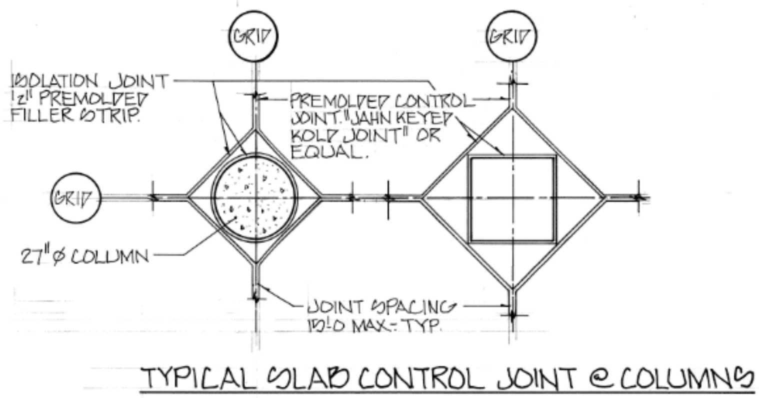


Figure 6. Typical Control Joint Detail

Roof system:

On the roof, elevator machinery and miscellaneous other HVAC machinery is stationed here, that must be supported in addition to snow loads, and were designed also to manage rain water, and divert it to drainage pipes on the roof. There are parapets of varying heights also located on the roof, preventing water run off on the sides of the building. The 8 inch thick roof is sloped slightly to aid in rain water management, preventing it from pooling, and potentially causing a collapse. Calculations on page # in Appendix # for snow loads show that the design load of 30 psf is in excess of the 21 psf snow load that would accumulate on the roof should snow drifts come into play during winter months.

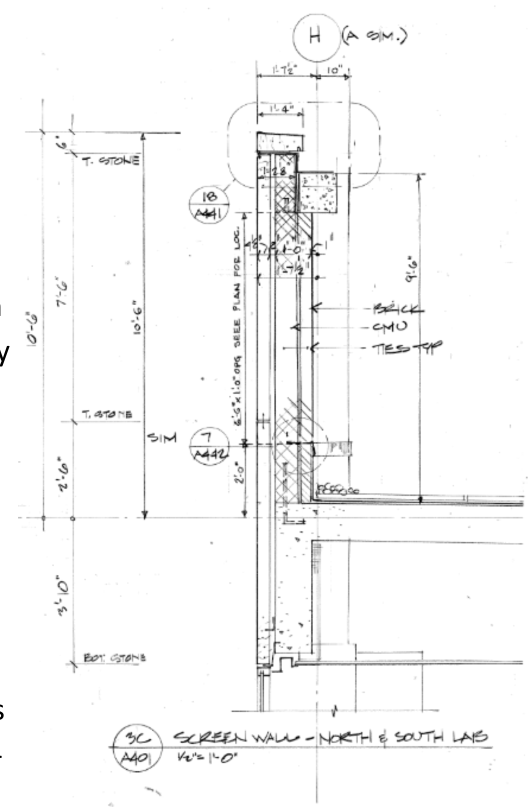


Figure 7. Example Section of a Parapet.

Secondary Structural System for Mechanical Equipment:

As mentioned before, for the ground level, slabs are thickened for the additional weight, and elevator equipment has its own columns around the elevator shaft to handle both the weight of the machinery, the elevator carriage, and the people that may be using the elevator at any given time.

Support of Curtain Walls:

Curtain walls and cladding for this building consist of limestone, granite and glass panels. These are often anchored directly into the concrete structure where they are applied. Two inches of clearing between the panel and the building are in place to insure that moisture has a way to trickle out and not accumulate behind the panel. Slabs have beams or some other support at the edge of their spans of varying depths and widths to support additional weight where panels are installed.

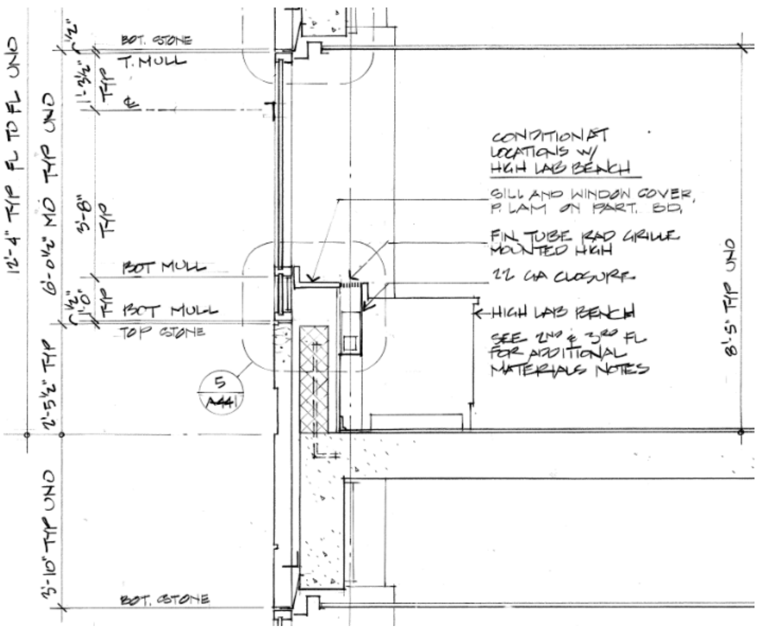


Figure 8. Example Section of Curtain Wall

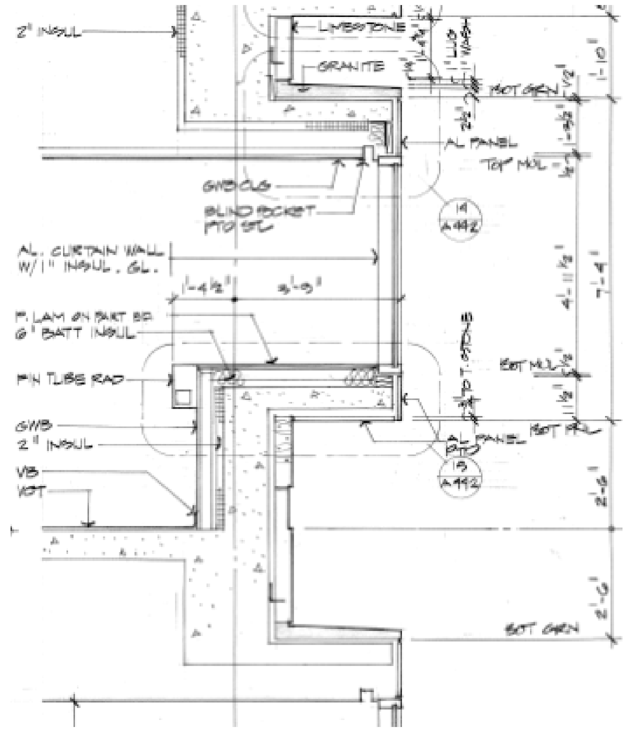


Figure 9. Example Section of Exterior Cladding

Support of Architectural Cylinder on Corner of Building:

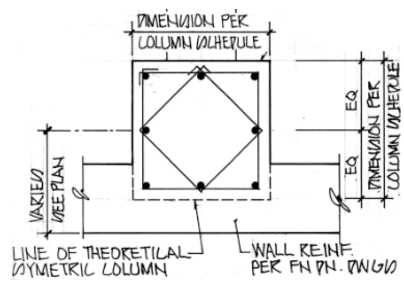


Figure 10. Illustration of Column Used for Support of Architectural Cylinder

There is an architectural cylinder on the corner of the building that is supported by 4 - 33" by 33"columns reinforced with 8 #11's as in Figure 10. The column is 125% larger than the columns above it, possibly from a safety standpoint. From the 2nd floor to the roof, the slabs on the interior support its glass, granite and limestone facade, and on the other face, a solid wall supports additional aesthetic wall panels along the stairwell, as seen in a section in Figure 11.

Lateral system:

Wind plays a large factor in the surrounding buildings, especially the Crescent, the main hospital building of the Hershey Medical Center. Its long and unique shape plays a direct role in sheltering the Biomedical Research Building from direct wind, as well as other surrounding buildings in the area. As for the Biomedical Research building, it has an oblong shape, making wind forces to be manageable in one direction by a smaller area for wind to push up, and a large structure to resist this wind load, but leaves a larger area to resist a larger wind load with shear walls. Wind forces are directly resisted by the curtain on the building, and forces are then transferred to the 8”-12” thick concrete slabs. Slabs then transfers the load into the columns and shear walls, and eventually down into the ground, through the caissons. For the short side of the building, there are large concrete beams that would play a strong role in resist wind forces.

Overall Interaction of Systems:

Ultimately, all existing systems rely heavily on the largely straightforward concrete structure, with lateral forces, going through the curtain walls, and most live and gravity loads behind handled by the floor slabs. The one way slabs transfer the loads to the beams and shear walls, and subsequently into various columns, which also support equipment loads and resulting roof loads. Excessive cracking in the slabs are controlled by control joints, temperature reinforcement maintains the effectiveness of the slabs under various temperature related stresses. Large grade beams then take the loads from the columns, as well as the thickened ground slab, supporting various heavy machinery, and redistribute the loads to the caissons below.

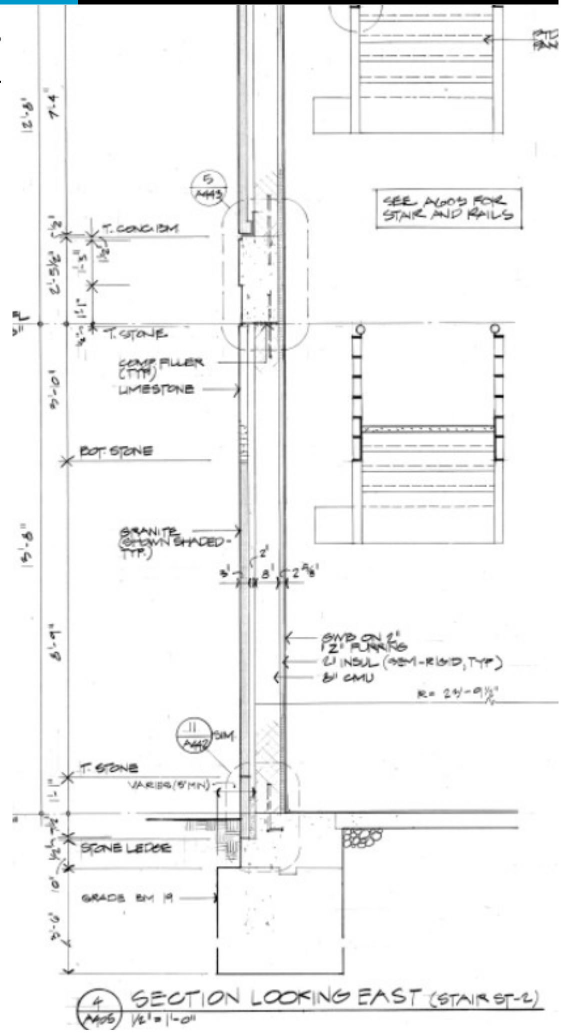


Figure 11. Section of Stairwell

Design Codes:

The original codes used by the original plans were BOCA, 1987 Edition, ACI 318-83, AISC, 1980 Edition, A. W. S. D1.1, 1986 or 1988 Edition and CRSI, 1986 edition. This technical report uses ACI 318-08, and ASCE-05 for its reference calculations.

Typical Materials Used:

Typical materials that were utilized were varying strengths of concrete. Those specifically specified in the typical details were 4000-5000 psi 28 day strength concrete, with most concrete being 4000 psi strength, while further investigation into the plans revealed at least one call for 1000 psi concrete for use in caissons. Reinforcing steel bars for #4-#11 sizes were to adhere to ASTM A615-60, and stirrups being #3 and #4 were to be of grade 40 steel. For the one way slabs, unless 6x6-w2.0xw2.0 WWF was called for, 6x6-w2.9xw2.9 WWF was the typical wire mesh used.

Gravity Loads:

Gravity loads were a combination of dead, live, and superimposed loads. Dead loads were calculated based on existing slab thicknesses and a 150 pcf concrete density. Live loads from plans were used, 125 psf for laboratories, and 100 psf for everywhere else, but for simplicity’s sake, 125 psf was used for all locations except the roof. A 30 psf roof load was used for a guideline for calculated snow drift loads. Lastly, a 15 psf superimposed dead load was included for miscellaneous lighting, electrical, HVAC, and plumping fixtures that may have been otherwise excluded from calculations.

Spot Checks:

Four checks were performed, including a typical column, a typical beam, punching shear for a typical slab, and a caisson. Figures are included below for reference for where these checks were performed.

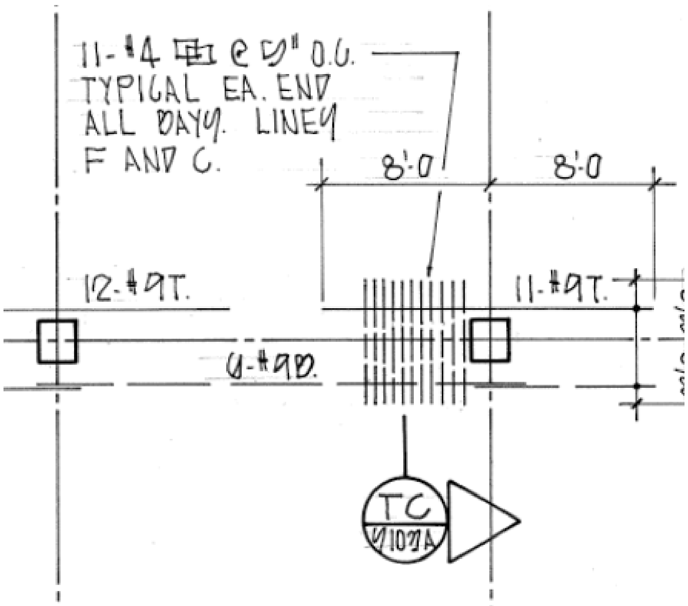


Figure 12. Beam between lines 9 and 10 along C on the 5th floor. Punching shear was checked for this slab around the right column.

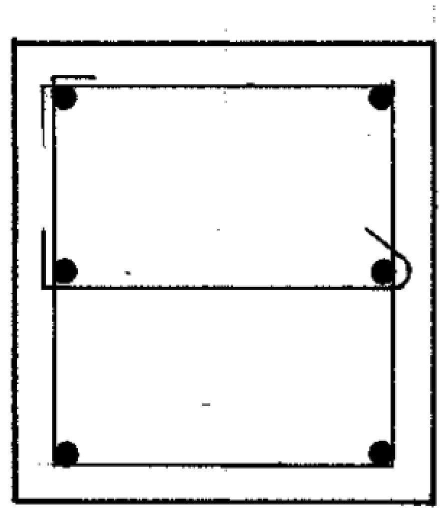


Figure 13. Typical section of column calculated. Column is located at F10 on the 5th floor.

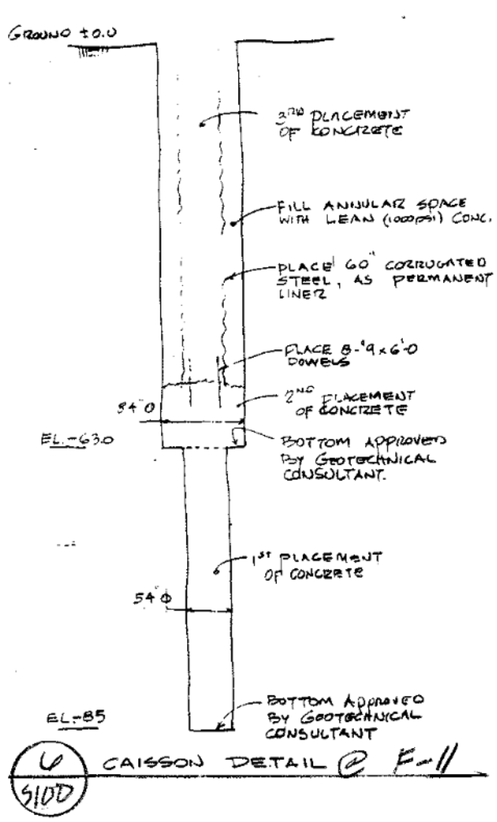
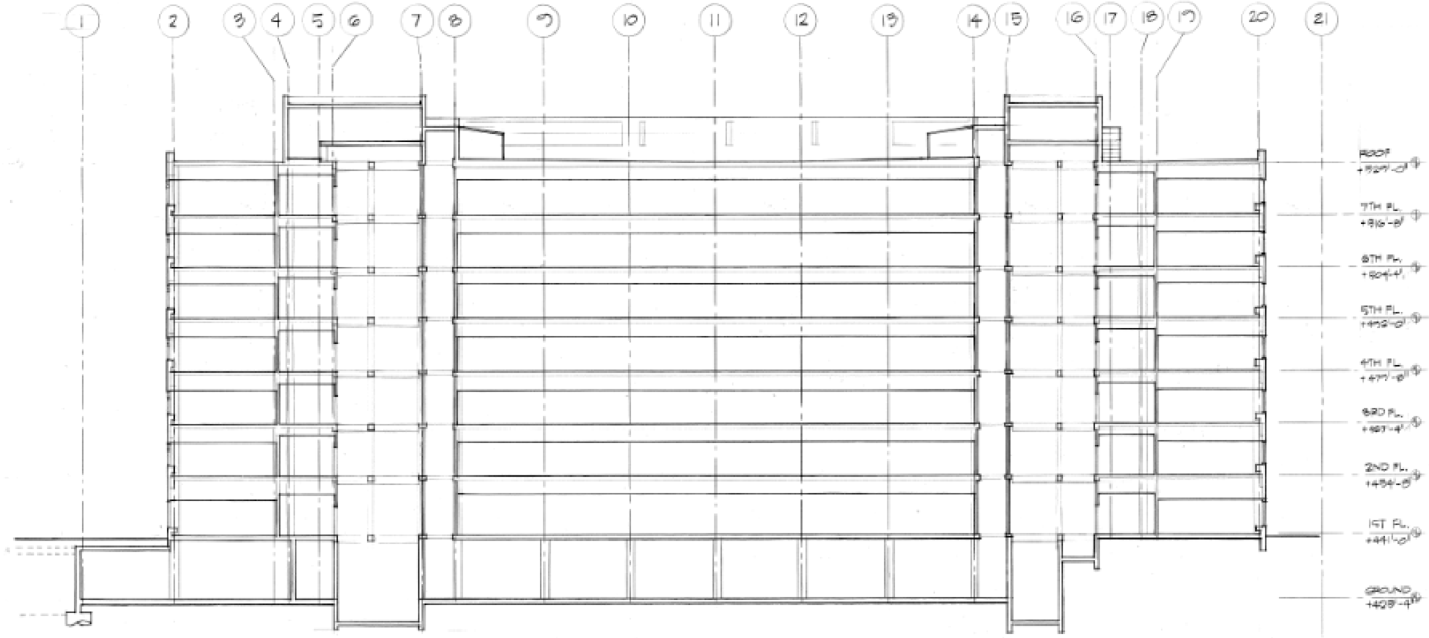


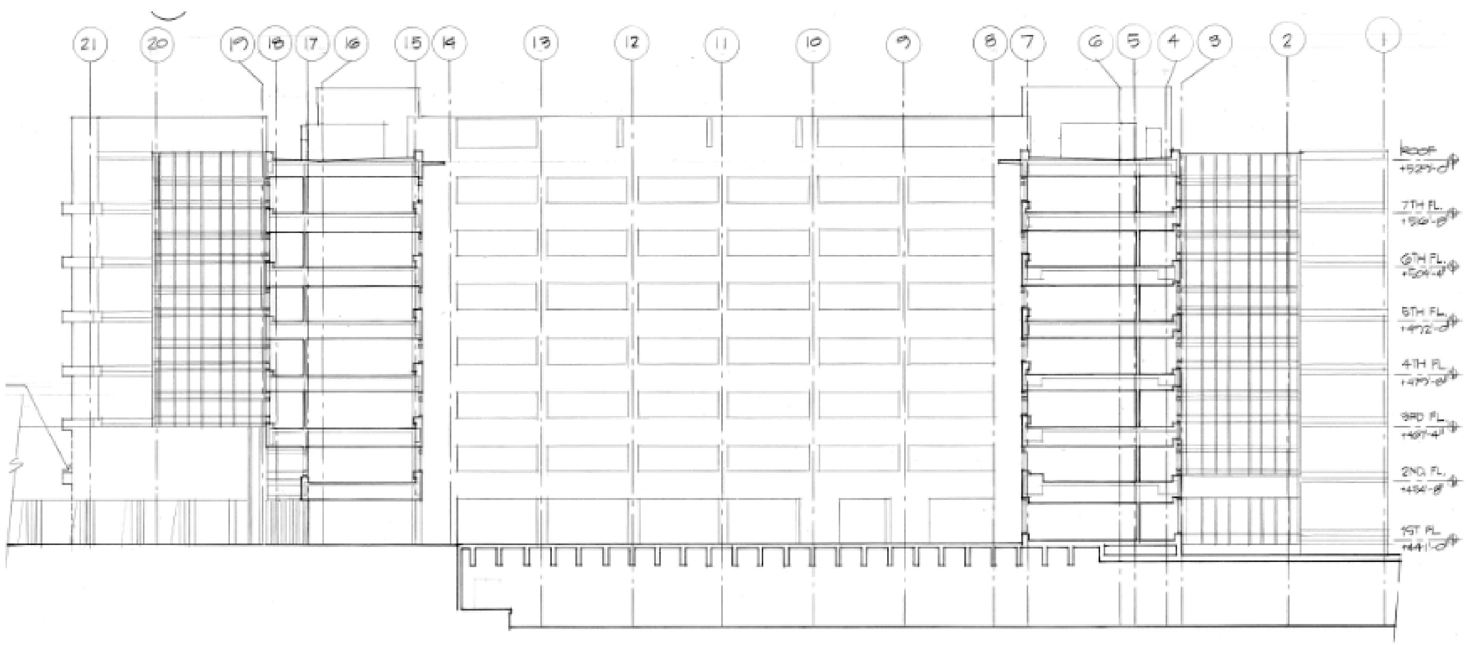
Figure 14. Caisson section that was checked. Bending moments were assumed to be negligible.

Appendix

Elevations

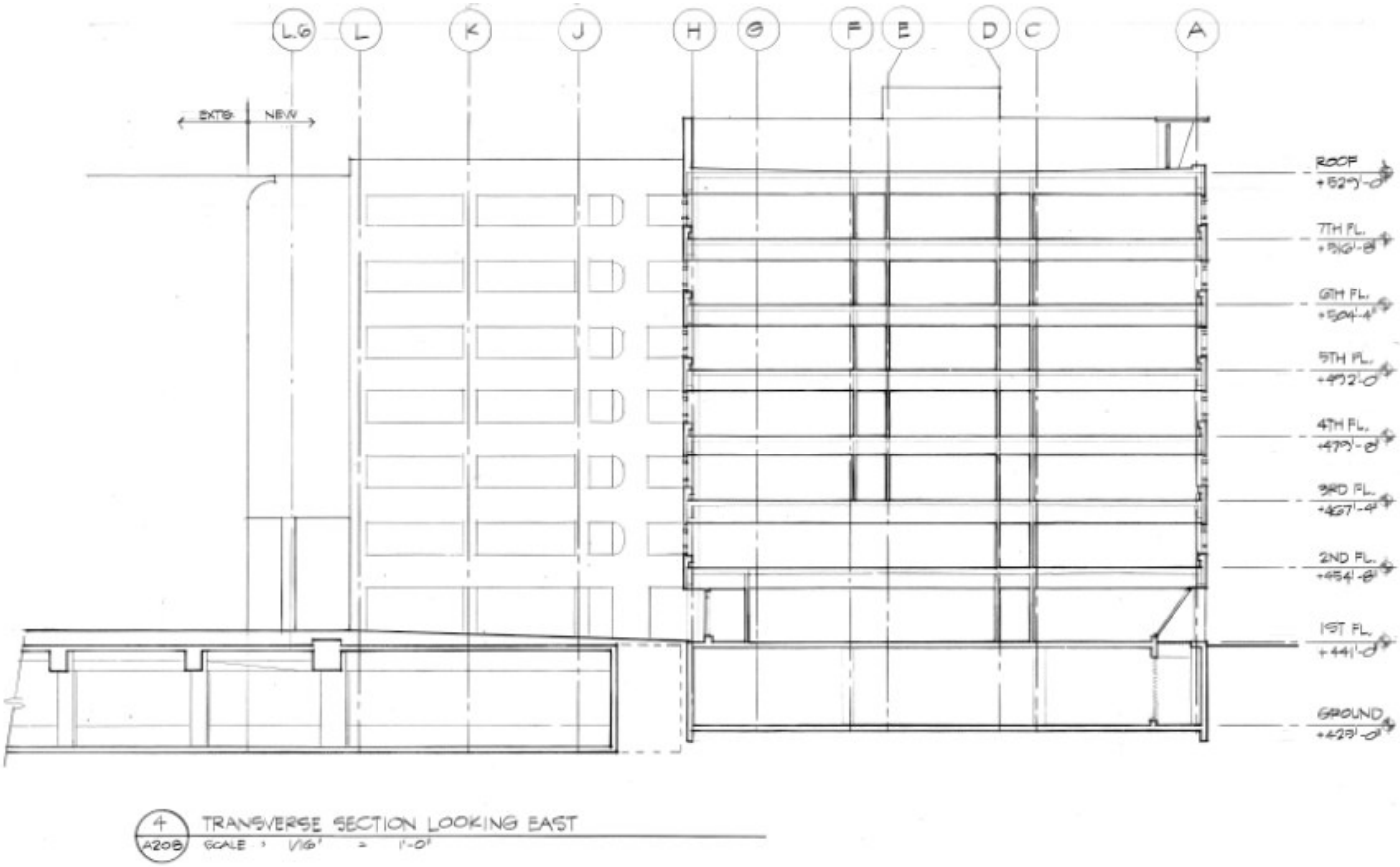


1 A208 LONGITUDINAL SECTION LOOKING NORTH
SCALE : 1/16" = 1'-0"

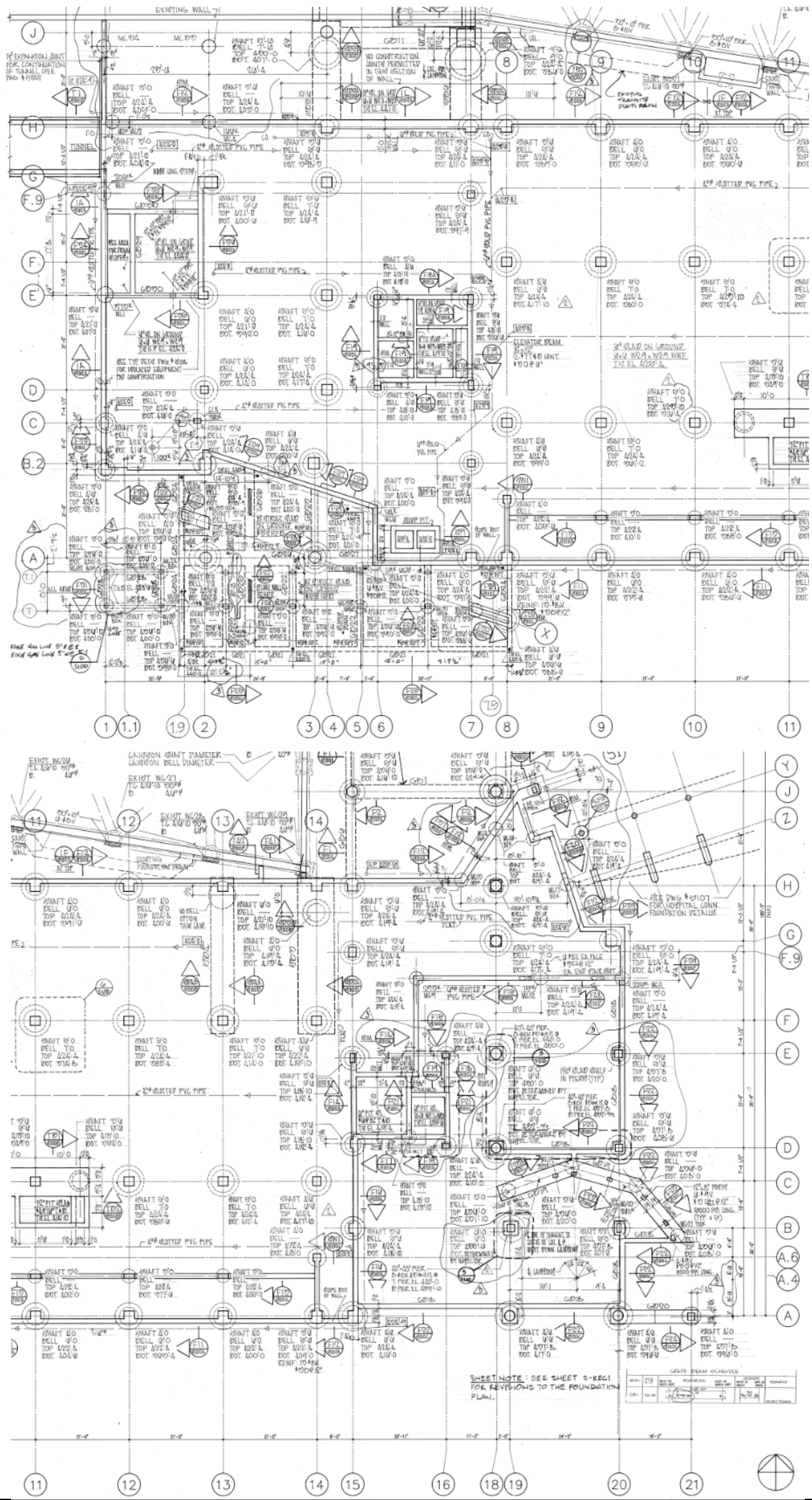


2 A208 SECTION THRU CONNECTORS LOOKING SOUTH
SCALE : 1/16" = 1'-0"

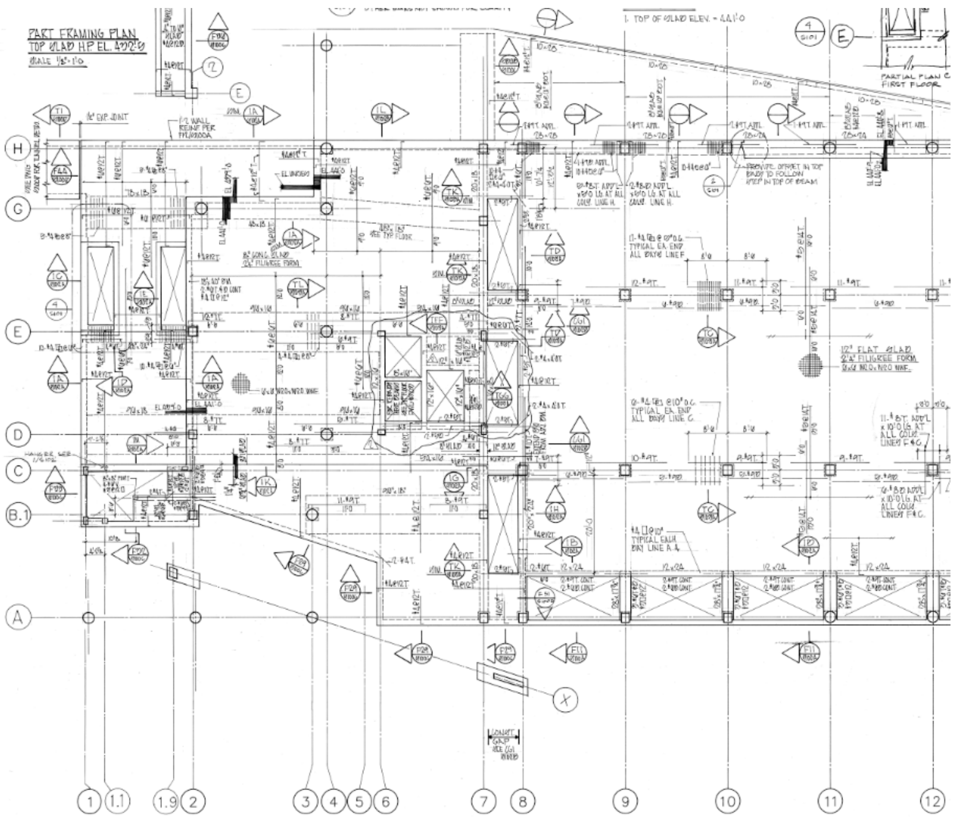
Elevations

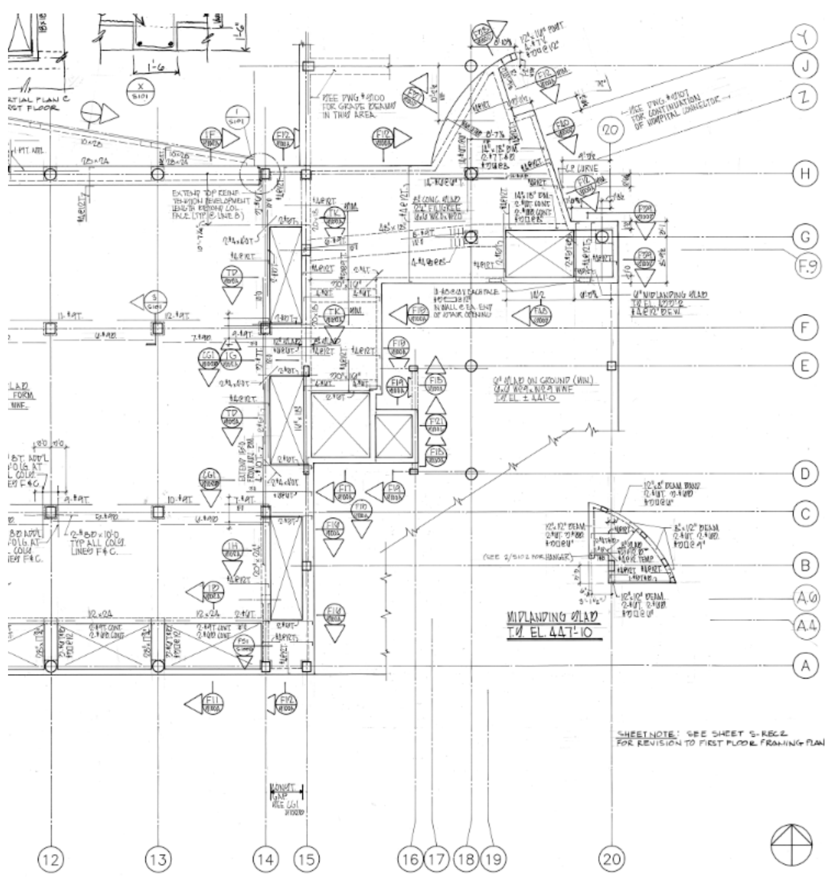

L.6
L
K
J
H
G
F
E
D
C
A
EXTG.
NEW
ROOF
+529'-0"
7TH FL.
+516'-8"
6TH FL.
+504'-4"
5TH FL.
+492'-0"
4TH FL.
+479'-8"
3RD FL.
+467'-4"
2ND FL.
+454'-8"
1ST FL.
+441'-0"
GROUND
+429'-0"
4
A208
TRANSVERSE SECTION LOOKING EAST
SCALE : 1/16" = 1'-0"

Foundation Plan (Ground Floor)

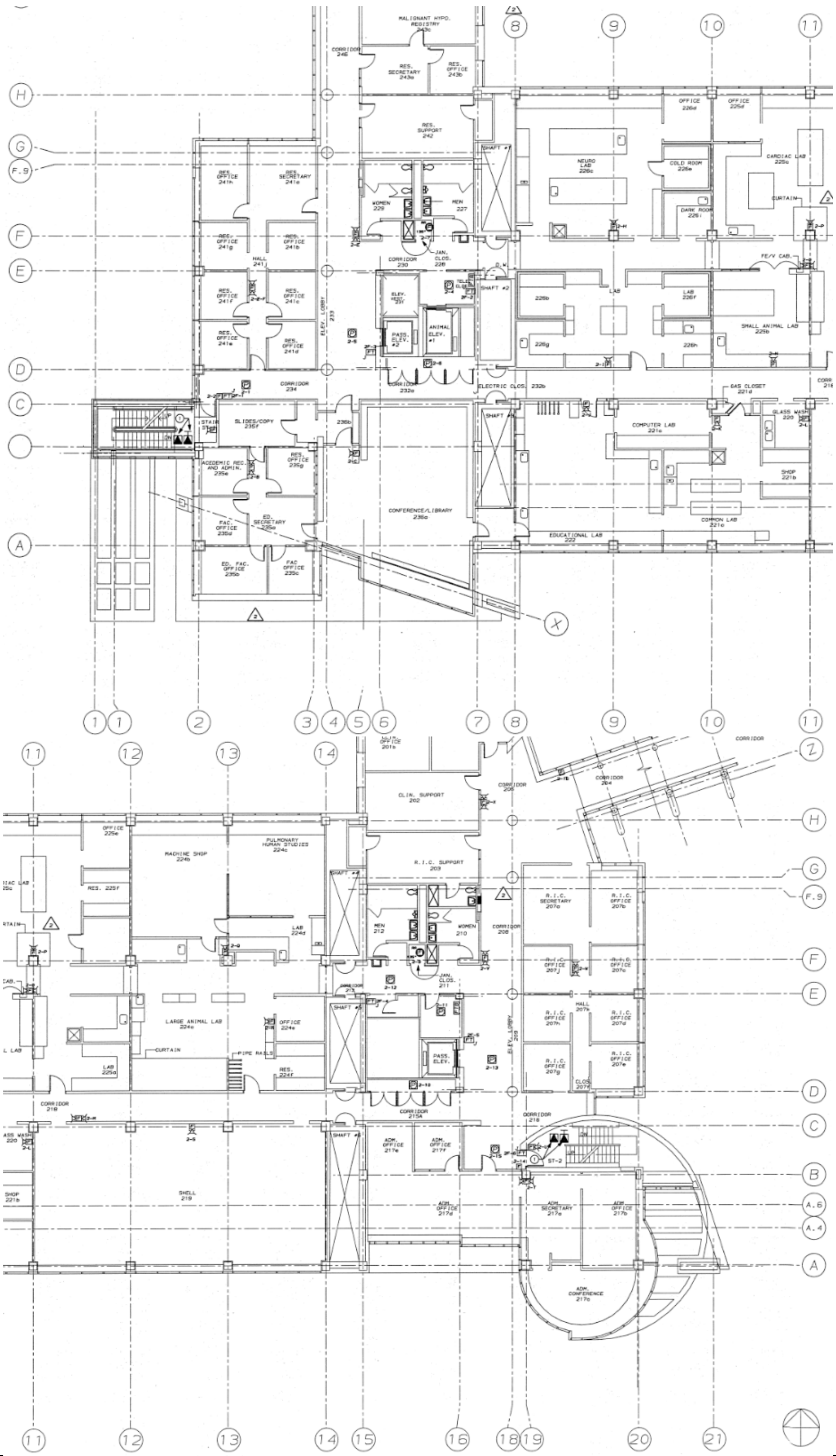


First Floor Plan

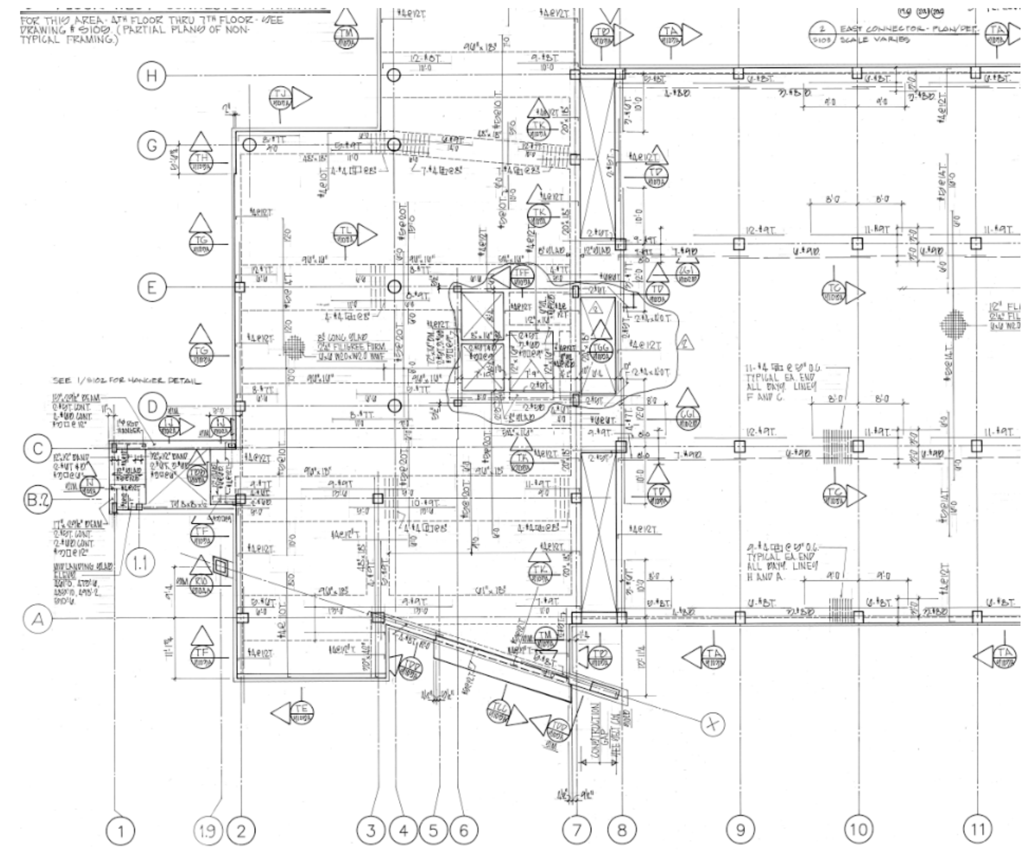


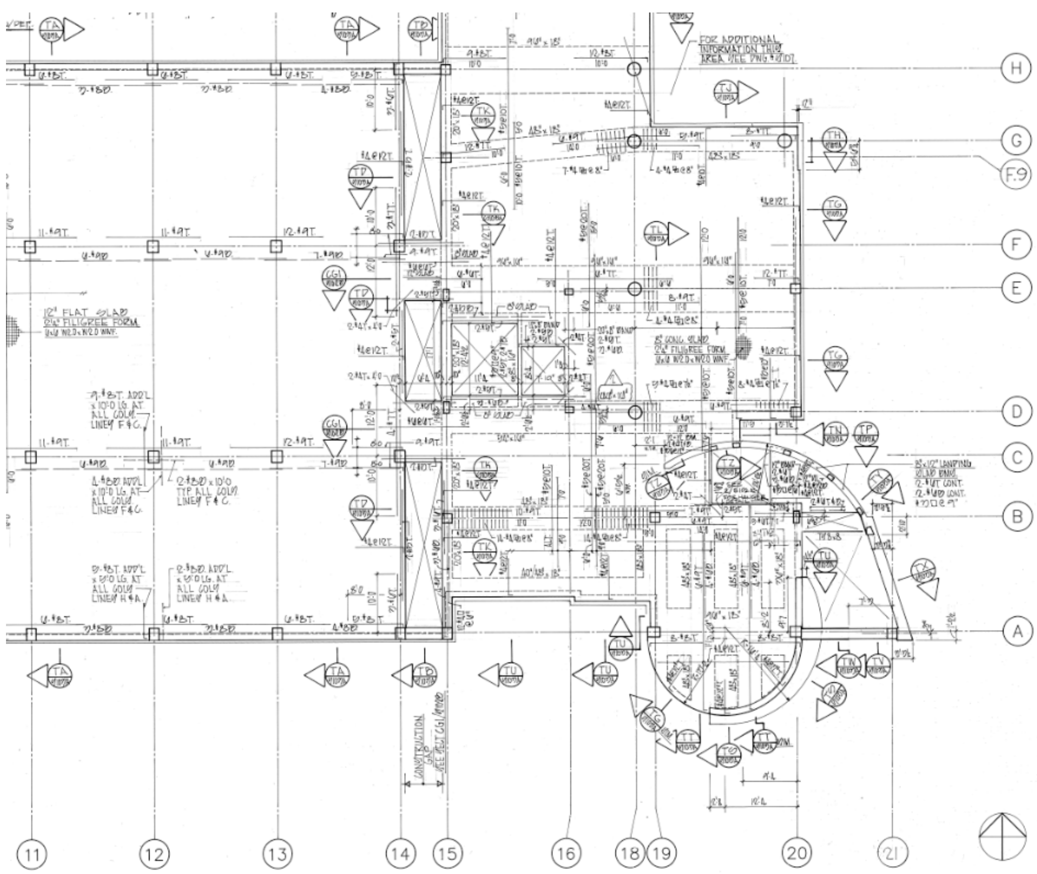


Second Floor Plan



Typical 3rd through 7th Floor Plans





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Self weight of building:

Assume 8" slabs typical for floors 3-7.

Assume 12" slabs for 1st floor. Assume 6" slab for ground floor.

Assume 12" slabs for 2nd floor. Assume 8" slab for roof

Assume 150 lbs per cubic foot of concrete.

Assume columns are uniform in size along entire length

Assume curtain walls to be typical – 6' 3.5" high limestone sections, 4" thick
– 6' 4.5" glass, 1" thick

Approximate area of ground floor:

(95.75)·(264.75) = 25350 sq ft.

Approximate area of 1st floor:

(88.2)(286) = 25225 sq ft.

Approximate area of 2nd floor:

(257)(96) = 24672 sq ft.

Approximate area of 3rd – 7th floors:

(282.75)(96) = 27144 sq ft.

5·27144 = 135720 sq ft.

total area: 211000 sq ft

Approximate Roof Area: 25000 sq ft.

height of columns: 110' from top of caissons to bottom of roof.

Average column size is 20"x20". => 1.6'x1.6'

# of columns: 67

volume of columns: 18870 ft^3

volume of floors + roof:

$$\frac{25350}{2}\,ft^3 + 25225\,ft^3 + 24670\,ft^3 + 2\left(\frac{135720}{3}\right)ft^3 + 2\left(\frac{25000}{3}\right)ft^3 = 169700\,ft^3$$

Perimeter of building: 752 ft

Joshua Zolko | Tech Report 1 | Seismic Loads | 2/3

Site location: Hershey, PA

1. Determine Design Spectral Response Acceleration

Values obtained from usgs.gov, referencing ASCE 7-10

Assume Class "B" Soil Classification, Risk Category facility

$S_S = .154g$ $S_{MS} = .154g$ $S_{DS} = .103g$

$S_1 = .055g$ $S_{M1} = .055g$ $S_{D1} = .037g$

2. Determine Seismic Design Category (SDC)

Building is Category IV so $I = 1.5$ (table 1.5-2)

For Hershey, PA

For $S_{DS} = .103g$ and Category IV ⇒ SDC = A (table 11.6-1)

For $S_{D1} = .037g$ and Category IV ⇒ SDC = A (table 11.6-2)

3. Identify the analysis procedure

Equivalent Lateral Force Procedure

Seismic base Shear

$V = C_S W$ $R = 3$ (Table 12.2-1, Ordinary reinforced concrete moment frame)

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{.103}{\left(\frac{3}{1.5}\right)} = .0515$$

$$T = C_T h_n^x \quad C_T = .016 \quad x = .9 \text{ (Table 12.8-2)}$$

$$= .016(110)^{.9} \quad h_n = 110'$$

$$= 1.100 < T_L = 6 \checkmark$$

$$C_S < \frac{S_{D1}}{\left(\frac{R}{I}\right)T} = \frac{.037}{\left(\frac{3}{1.5}\right)1.1} = .017$$

C_S is not lower than .017, therefore C_S is .017.

$C_S = .017 > .01 \checkmark$

4. Calculate total building weight

Surface area of building:

752·110 = 82720 Sq ft.

$\frac{82720}{2}\cdot\frac{1}{3}\cdot 150 + \frac{82720}{2}\cdot\frac{1}{12}\cdot 150 = 3{,}100{,}000$ lbs curtain wall weight

(169700 + 18870)150 = 28300000 lbs

Total Approximate building weight: 31400000 lbs

roof LL = 30 psf (from plans)

.2·30 = 6 psf

6·25000 = 150000 lbs

$w = 31400000 + 150000 = 31450000$ lbs => 31450 Kips

$V = C_s \cdot w = .017 \cdot 31450$

V = 529 Kips

6. Determine Vertical distribution of seismic forces

$F_x = C_{vx} \cdot V$ V = 529 Kips

$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$ K = 2, for T = 1.175s

C_{vx} calculated on spreadsheet.

F_x Calculated on spreadsheet.

Overturning moment also calculated on spreadsheet, at 42808 ft-Kips

Seismic Spreadsheet Calculations

Cvx Calculations					
Floor	Wx	Hx	EWiHi^K	K	Cvx
g	2535	0	0	2	0
1	3784	14.25	768388.5	2	0.00838
2	3700	27.91667	2883559	2	0.031448
3	2714	40.58333	4469977	2	0.048749
4	2714	52.91667	7599671	2	0.082882
5	2714	65.25	11555025	2	0.126019
6	2714	77.58333	16336037	2	0.17816
7	2714	89.91667	21942709	2	0.239306
Roof	2500	102.25	26137656	2	0.285056
Total :					1

Fx Calculations		
Cvx	V	Fx
0.01	529	5.29
0.03	529	15.87
0.05	529	26.45
0.08	529	42.32
0.13	529	68.77
0.18	529	95.22
0.24	529	126.96
0.29	529	153.41
Total Shear		534.29

Overturning Moment		
Fx (kip)	H (Feet)	M (kip ft)
5.29	14	75.3825
15.87	28	443.0375
26.45	41	1073.429
42.32	53	2239.433
68.77	65	4487.242
95.22	78	7387.485
126.96	90	11415.82
153.41	102	15686.17
Total:		42808

Joshua Zolko | Tech Report 1 | Wind Loads | 1/2

Location: Hershey, PA

Category IV (Table 1-1)

Exposure C (section 6.5.6)

V=90mph (Figure 6-1)

I=1.15 (Table 6-1)

K_d=.85 (Table 6-4)

K_{zt}=1.0 (flat elevation)

$K_z = \frac{1.26+1.31}{2} = 1.29$ (Table 6-3) (h=110') (varies)

qz calculated on spread sheet using:

$q_z = .00256 K_z K_{zt} K_d V^2 I$

For $p = qGC_p - q_i(GC_{pi})$;

$G = .85$
C_p from table 6-6
$q_i = q$
GC_{pi} from table 6-5

values found in Spread sheet.

L/B for 95' side: $95/277 = .34 < 1$; $C_p = -.5$

L/B for 277' side: $277/95 = 2.92$; $C_p = -.3$

Wind Spreadsheet Calculations

Spreadsheet for qz									
Floor	H	C	Kz	Kzt	Kd	V	V^2	I	qz
1	14.3	0.00256	0.85	1	0.85	90	8100	1.15	17.22902
2	13.7	0.00256	0.97	1	0.85	90	8100	1.15	19.66136
3	12.7	0.00256	1.04	1	0.85	90	8100	1.15	21.08022
4	12.3	0.00256	1.09	1	0.85	90	8100	1.15	22.09369
5	12.3	0.00256	1.15	1	0.85	90	8100	1.15	23.30986
6	12.3	0.00256	1.2	1	0.85	90	8100	1.15	24.32333
7	12.3	0.00256	1.24	1	0.85	90	8100	1.15	25.13411
Parapet	1.5	0.00256	1.26	1	0.85	90	8100	1.15	25.53949

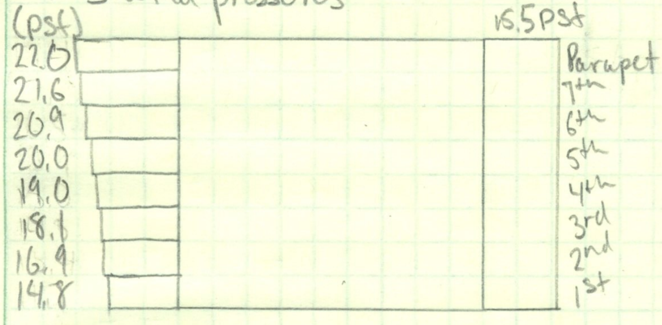
Windward Pressures							
q	G	Cp	qi	Gcpi (+/-)	Pressure	(+/-)	Resultant
17.22902	0.85	0.8	17.22902	0.18	11.71574	3.101224	14.81696
19.66136	0.85	0.8	19.66136	0.18	13.36972	3.539044	16.90877
21.08022	0.85	0.8	21.08022	0.18	14.33455	3.794439	18.12899
22.09369	0.85	0.8	22.09369	0.18	15.02371	3.976864	19.00057
23.30986	0.85	0.8	23.30986	0.18	15.8507	4.195774	20.04648
24.32333	0.85	0.8	24.32333	0.18	16.53986	4.378199	20.91806
25.13411	0.85	0.8	25.13411	0.18	17.09119	4.524139	21.61533
25.53949	0.85	0.8	25.53949	0.18	17.36686	4.597109	21.96397

Leeward Pressures							
LW (95' side)							
q	G	Cp	qi	Gcpi (+/-)	p	(+/-)	Resultant
25.53949	0.85	-0.5	25.53949	0.18	-10.8543	4.597109	-15.4514
LW (277' side)							
q	G	Cp	qi	Gcpi (+/-)	p	(+/-)	Resultant
25.53949	0.85	-0.3	25.53949	0.18	-6.51257	4.597109	-11.1097

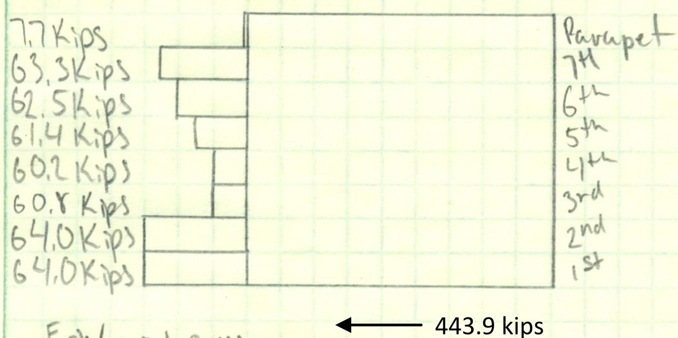
Resultant wind loads							
	WW (95')	WW (277')	H	LW (95')	LW (277')	R (95')	R (277')
1	20058.46	58486.25	14.3	-20917.3	-43852.7	40.97579	63.91115
2	21953.22	64010.96	13.7	-20061.1	-42057.5	42.01428	64.01076
3	21815.21	63608.57	12.7	-18593.2	-38980.2	40.40839	60.79538
4	22262.34	64912.29	12.3	-18103.9	-37954.4	40.36622	60.21671
5	23487.79	68485.44	12.3	-18103.9	-37954.4	41.59167	61.44216
6	24509	71463.07	12.3	-18103.9	-37954.4	42.61288	62.46337
7	25325.96	73845.18	12.3	-18103.9	-37954.4	43.42985	63.28033
Parapet	3129.865	9126.028	1.5	-2201.82	-4616.07	5.331689	7.745937

Joshua Zolko | Tech Report 1 | Wind Loads | 2/2

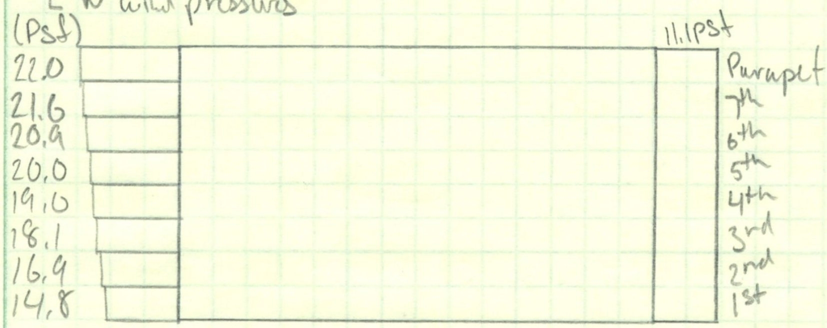
N-S wind pressures



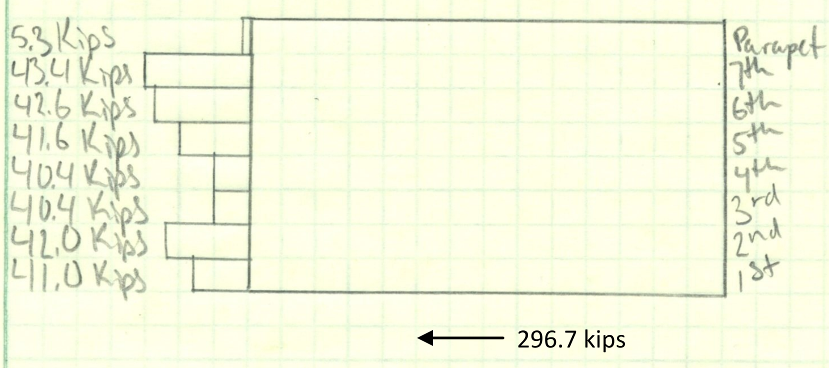
N-S wind forces



E-W wind pressures



E-W wind forces



Joshua Zolko | Tech Report 1 | Snow loads | 1/1

Roof: flat

$p_f = .7 C_e C_t I p_g$

$C_e = 1.0$ (table 7-2)

$C_t = 1.0$ (table 7-3)

$I = 1.2$ (table 7-4)

$p_g = 25$ (figure 7-1)

$p_f = .7(1.0)(1.0)(1.2)(25)$

$= 21$ psf

Drift

$\gamma = .13p_g + 14 \quad p_g = 25$

$= .13(25) + 14$

$= 17.25$ pcf < 30 pcf ✓

$h_b = \frac{p_s}{\gamma} \quad p_s = C_s p_f \quad C_s = 1.0$ (figure 7-2)

$p_s = 1.0(21) \quad p_f = 21$

$h_b = \frac{21}{17.25} \quad p_s = 21$

$h_b = 1.22'$

$h = 8.75'$ (from roof projects in a404.)

$h_c = 8.75 - 1.22 = 7.53' \quad \frac{h_c}{h_b} = \frac{7.53}{1.22} = 6.17 > .2$, check drift

$l_u = 56$

$h_d = 2.3 \quad h_d < h_c$

$w = 4h_d < 8h_c$

$= 4(2.3) < 8(7.53)$

$= 9.2 < 60.2$ ✓

$p_d = h_d(p_s)$

$= 2.3(21) = \underline{48.3 \text{ psf}}$ higher than design roof load on drawings!

Joshua Zolko | Tech Report 1 | Spot Checks | 1/a

Beam Check

Rebar: $f_y = 60,000$ psi Stirrups: $f_y = 40,000$ psi $f'c = 4000$ psi

LL = 125 psf (Lab spaces require 125 psf as specified on drawings)

DL = 150 psf (1 ft deep concrete slab)

Superimposed DL = 15 psf

$1.2D + 1.6L = w$

$1.2(150 + 15) + 1.6(125) = 398 \text{ psf} \approx 400 \text{ psf}$

Trib length: 35'-9" Clear span: $21 - 22/12 = 19'\text{-}2''$

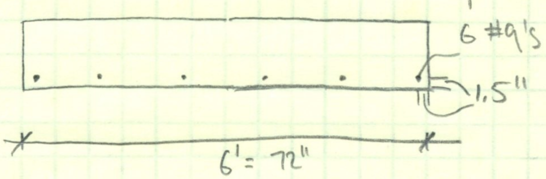
Distributed load:

$35.75 \cdot 400 = 14.3$ Kip/ft

$M^+ = \dfrac{14.3(19.2)^2}{16} = 329.5$ ft·Kips

$M^- = \dfrac{14.3(19.2)^2}{11} = 479.2$ ft·Kips

Section of beam @ maximum positive moment: 329.5 ft·Kips



note: beam is continuous with slab; both have a depth of 12"

$d = 12 - 1.5 = 10.5''$

$A_s = 6 \cdot (1) = 6 \text{ in}^2$ $f_y = 60000$ psi $f'c = 4000$ psi (assuming rebar is yielding)

$a = \dfrac{A_s \cdot f_y}{.85(f'c)(b)} = \dfrac{6 \cdot 60000}{.85(4000)(72)} = 1.47''$ $c = \dfrac{a}{.85} \Rightarrow \dfrac{1.47}{.85} = 1.73''$

Check assumption:

$.003\left(\dfrac{d-c}{c}\right) = .003\left(\dfrac{10.5 - 1.73}{1.73}\right) = .015 > .002$ ✓ and $.015 > .004$, section is allowed.

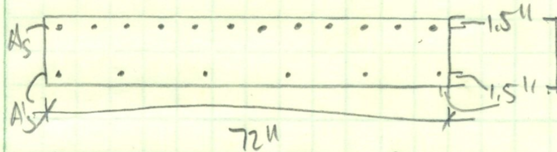
$M_n = A_s \cdot f_y (d - a/2)$

$= 6 \cdot 60000(10.5 - 1.47/2) = 4055400$ lbs·in $\Rightarrow 293$ ft Kips

293 ft·Kip < 329.5 ft·Kip ✗ section fails

Joshua Zolko | Tech Report 1 | Spot Checks | 2/9

Section of beam @ maximum negative moment = 479.2 ft·Kips



Treat section as upside-down, checking negative section.

$A'_s = 6\,in^2$ $d' = 1.5"$ $f_y = 60000\,psi$
$A_s = 12\,in$ $d = 10.5"$ $f'_c = 4000\,psi$

Assume both rows of rebar are yielding.

$a = \frac{A_s f_y - A'_s f_y}{.85 f'_c (b)} = \frac{12(60000) - 6(60000)}{.85(4000)(72)} = 1.47"$ (measured from bottom of section)

$c = \frac{1.47}{.85} = 1.73"$

Verify assumptions:

$\varepsilon'_s = \frac{.003}{c}(c-d') = \frac{.003}{1.73}(1.73-1.5) = .00039 < .002$; not true

$.85 f'_c \beta_1 c \cdot b + A'_s \frac{.003}{c}(c-d')E_s = A_s f_y$ $E_s = 29000000\,psi$

Solve quadratic

$.85 f'_c \beta_1 c^2 b + A'_s .003(c-d')E_s - A_s f_y c = 0$

$.85 f'_c \beta_1 c^2 b + A'_s .003 c E_s - A_s f_y c - A'_s .003(d')E_s = 0$

$.85(4000)(.85)(72)c^2 + 6 \cdot .003 \cdot 29000000c - 12 \cdot 60000 \cdot c - 6 \cdot .003(29000000)1.5 = 0$

$208080c^2 - 198000c - 783000 = 0$

$c = 2.47"$

Check yielding:

$\varepsilon_s = \frac{.003}{2.47}(10.5-2.47) = .0097 > .002$ ✓ and $.0097 > .004$, section is allowed.

$M_n = A'_s \frac{.003}{c}(c-d')E_s(d-d') + .85 f'_c a b (d - a/2)$ $a = \beta_1 c = 2.1"$

$M_n = 6 \cdot \frac{.003}{2.47}(2.47-1.5)(29000000)(10.5-1.5) + .85(4000)(2.1)(72)(10.5 - 2.1/2)$

.97 9 9.45

$= 6703020\,lb \cdot in \Rightarrow 559\,ft \cdot Kips$

$559\,ft\,Kips > 479.2\,ft \cdot Kips$ ✓

Joshua Zolko | Tech Report 1 | Spot Checks | 3/9

Check shear reinforcement: Plans call for 11 #4's ☐ @ 5" o.c. (4 legs) ⇒ $A_v = .8 in^2$

$V_c = 2\sqrt{f'c}(b)(d)$ $f'c = 4000$ $b = 72"$ $d = 10.5"$

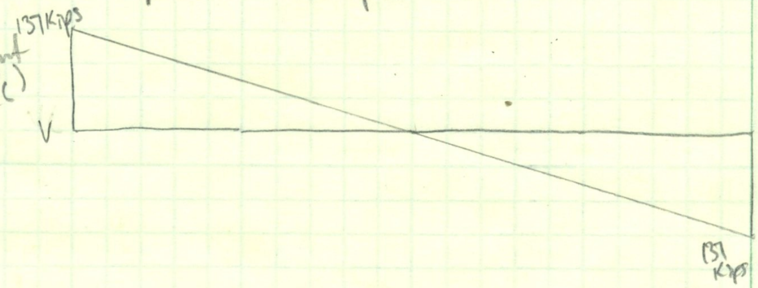
$= 2\sqrt{4000}(72)(10.5)$ $R = 137$ Kips   Clear span = 19'-2"

$V_c = 95.6$ Kips (no reinforcement required below V_c)

$\phi V_n = .5\phi V_c$ $\phi = .75$

$= .5(.75)(95.6)$

$\phi V_n = 35.9$ Kips



$V_s = \frac{V_u}{\phi} - V_c$ $V_u = 137 - \frac{10.5}{12} \cdot 14.3 = 124.5$ Kips

$V_s = \frac{124.5}{.75} - 95.6 = 70.4$ Kips

Maximum spacing

$V_s \leq 4\sqrt{f'c}\, b_w d = 4\sqrt{4000}(72)(10.5)$

$V_s \leq 191.3$ ✓

therefore $S_{max} = \min\begin{cases} d/2 \\ 24" \end{cases}$

$S_{max} = \min\begin{cases} 10.5/2 \\ 24" \end{cases}$

$= 5.25" > 5"$ called for on plans ✓

Check reinforcement discontinue:

Appears shear reinforcement starts at 5" of support.

$137 - \frac{11.5}{12} \cdot 14.3 = 71.5$ Kips

$V_c = 95.6 > 71.5$ ✓

reinforcement discontinued at appropriate spot.

minimum shear reinforcement

$A_{vmin} = \max\begin{cases} .75\sqrt{f'c}\, b_w S/f_{yt} \\ 50 b_w S/f_{yt} \end{cases} = \max\begin{cases} .75\sqrt{4000}(72)(5)/40000 \\ 50(72)(5)/40000 \end{cases} = \max\begin{cases} .43 \\ .45 \end{cases}$

$A_{vmin} = .45 in^2 < .8 in^2$ ✓

Check required reinforcement:

$\phi V_n = \phi V_c + \phi V_s = .75\left(95.6 + \frac{.8(40)(10.5)}{5}\right) = 122.1$ Kips

122.1 Kips < 124.5 Kips ✗ shear reinforcement fails

Joshua Zolko | Tech Report 1 | Spot Checks | 4/9

Deflection check.

$h_{min} = \frac{L}{21} = \frac{19.2 \cdot 12}{21} = 10.97'' < 12''$ ✓ unnecessary to check deflection.

Check development length of top reinforcement. (16' long total, so 8' into beam from center of column.)

$L_d = \frac{3}{40} \frac{f_y}{\lambda\sqrt{f'c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{C_b + K_{tr}}{d_b}\right)} d_b \left(\frac{A_{s,req'd}}{A_{s,prov}}\right) \geq 12''$

$C_b = 1.5''$ $K_{tr} = 0$ (conservative)

$f_y = 60000\,psi$

$\frac{C_b + K_{tr}}{d_b} \leq 2.5$

$d_b = 1.128''$ $\frac{A_{s,req'd}}{A_{s,prov}} = 1$ (conservative)

$\lambda = 1.0$

$\frac{1.5 + 0}{1.128} = 1.3 < 2.5$ ✓

$f'c = 4000\,psi$

$\psi_t = 1.0$ $\psi_e = 1.0$ (assume uncoated bars, reasonable given this is an interior bar)

$\psi_s = 1.0$

$L_d = \frac{3}{40} \frac{60000}{1.0\sqrt{4000}} \frac{1.0 \cdot 1.0 \cdot 1.0}{1.3} (1.128) \geq 12$

$L_d = 62'' > 12$ ✓

$\frac{62}{12} = 5'\text{-}2''$

Length given $= 96 - 11 = 7'\text{-}1''$

$5'\text{-}2'' < 7'\text{-}1''$ ✓ length is sufficient for reinforcement development.

Joshua Zolko | Tech Report 1 | Spot Checks | 5/9

Typical slab check:

Punching Shear:

At face of support:

$b_0 = 22 \cdot 2 + 24 \cdot 2 + 4 \cdot 12 = 140''$

$V_n = \left(21 \cdot 35.75 - \left(\frac{22+12}{12} \cdot \frac{24+12}{12}\right)\right) 400\,psf$

$= 297\ Kips$

$\lambda = 1.0 \quad \phi = .75 \quad A_v = 1.6 in^2 \quad f_y = 40000\,psi$

$s = 5'' \quad d = 10.5$

$V_u = \phi 2 \lambda \sqrt{f'c}\, b_0 d + \frac{\phi A_v f_y d}{s} \leq \phi 6 \sqrt{f'c}\, b_0 d$

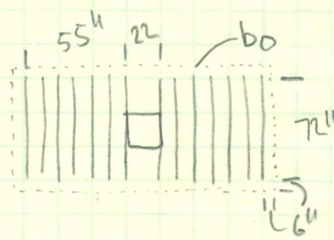
$V_u = .75(2)\sqrt{4000}(140)(10.5) + \frac{.75(40000)(10.5)}{5} \leq .75(6)\sqrt{4000}(140)(10.5)$

$= 201.9\ Kips \leq 416.7\ Kips\ \checkmark$

but,

$V_n = 297 > 201.9\ Kips$ fails

At end of Stirrups:



$b_0 = 22 \cdot 2 + 110 \cdot 2 + 72 \cdot 2 + 12 \cdot 4$

$= 456 in$

$V_n = (21 \cdot 35.75 - 12 \cdot 7)400$

$= 266.7\ Kips$

$V_u = \phi 2 \lambda \sqrt{f'c}\, b_0 \cdot d$

$= .75(2)\sqrt{4000}(456) \cdot 10.5$

$V_u = 452.5\ Kips$

$V_n = 266.7 < 452.5$ ✓ does not fail to punching shear.

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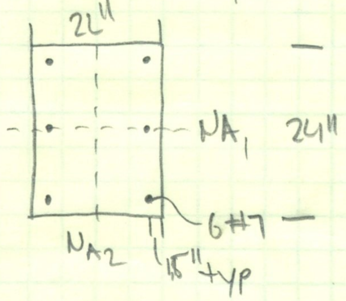
Column Check (typical)



$A_s = 6 \cdot .6 = 3.6 in^2$ $f_y = 60000 psi$

Pure axial Strength: $A_c = 22 \cdot 24 - 3.6 = 524.4 in^2$ $f'c = 4000 psi$

$P_o = .85 f'c \cdot A_c + A_s \cdot f_y$

$= .85(4000)524.4 + 3.6(60000)$

$P_o = 2000 Kips$

$P_n = \underbrace{2 \cdot 400 \cdot 35.75 \cdot 21}_{\text{2 floors of load 6th + 7th}} + \underbrace{30 \cdot 35.75 \cdot 21}_{\text{roof load}}$

$P_n = 1623 Kips$

$P_o > P_n$

$2000 > 623$ ✓

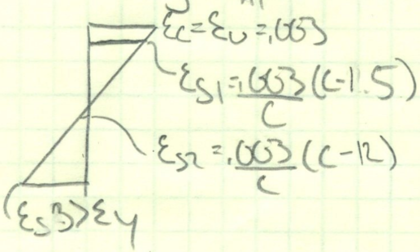
Pure axial tension:

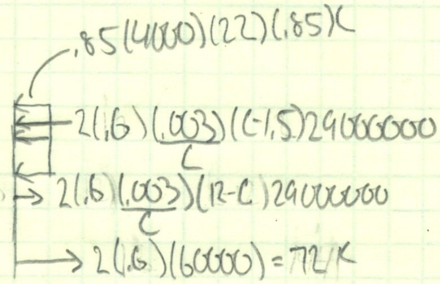
$T_o = A_s \cdot f_y$

$= 3.6 \cdot 60000$

$T_o = 216 Kips$

Pure bending @ N_{A1}





Strain Force

$\frac{104400}{c}(c-1.5)$

$\frac{104400}{c}(c-12)$

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$$63580c + \frac{104400}{c}(c-1.5) = \frac{104400}{c}(c-12) + 72000$$

$$63580c^2 + 104400c - 156600 = 104400c - 1252800 + 72000c$$

$$63580c^2 + 72000c - 1096200 = 0$$

$$c = 4.8''$$

$$\varepsilon_{s1} = \frac{.003}{4.8}(4.8-1.5) = .00207 = \varepsilon_y \qquad \varepsilon_y = \frac{60}{29000} = .00207$$

$$F_{s1} = 60\text{ksi} \Rightarrow 72\text{ Kips}$$

$$\varepsilon_{s2} = \frac{.003}{4.8}(4.8-12) = -.0045$$

$$F_{s2} = -60\text{ksi} \Rightarrow -72\text{ Kips} \qquad a = .85c = .85(7.7) = 6.5''$$

$$\varepsilon_{s3} = \frac{.003}{4.8}(4.8-24) > \varepsilon_y$$

$$F_{s3} = -60\text{ksi} \Rightarrow -72\text{ Kips}$$

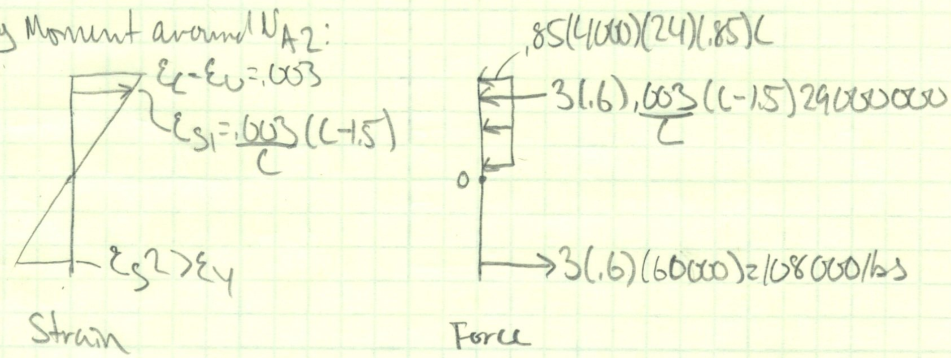
$$\text{Concrete} = .85(4000)(22)(6.5) = 486.2\text{ Kips}$$

Pure bending moment around NA_1:

$$M_0 = 486.2\left(12 - \frac{6.5}{2}\right) + 72(12-1.5) + 72(12-1.5)$$

$$M_0 = 480\text{ ft}\cdot\text{Kips}$$

Bending Moment around NA_2:



$$.85(4000)(24)(.85)c + 3(.6)\frac{.003}{c}(c-1.5)29000000 = 108000$$

$$69360c^2 + 156600c - 234900 = 108000c$$

$$69360c^2 + 48600c - 234900 = 0$$

$$c = 1.5''$$

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$c = 1.5''$

$\varepsilon_{s1} = \frac{.003}{1.5}(1.5 - 1.5) = 0$

$F_{s1} = 0$ Concrete $= .85(4000)(24)(.85)(1.5)$

$F_{s2} = 108$ Kips $= 164$ Kips

Moment about 0 at NA_2:

$M_0 = 164\left(11 - \frac{.85(1.5)}{2}\right) + 108(11 - 1.5)$

$M_0 = 175.3$ ft·Kips

$479.2 > 175.3$ X 479.2 Kipft from negative moment on beam.

but 479.2 would include almost 1/2 DL, mixed with different codes and Design methods, could account for drastic difference.

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Caisson Check

Pure axial: (Assume negligible moment, as caisson is in ground.)

$P_n = 8 \cdot 400 \cdot 35.75 \cdot 21 + 30 \cdot 35.75 \cdot 21$

$P_n = 2424.9$ Kips (plans only mention a load of 1550K)

Plans call for 1000 psi concrete ⇒ $f'c = 1000$ psi

$84'' \phi \Rightarrow \pi 42^2 = A_c = 5539 \text{ in}^2$

$.85 \cdot 1000 \cdot 5539 = 4708$ Kips > 2425 Kips ✓

second part of caisson below first is $54'' \phi \Rightarrow \pi 27^2 = A_c = 2289.06 \text{ in}^2$

$.85(1000) \cdot 2289 = 1945.7$ Kips < 2425 Kips ✗ fails, but works with 1550 Kip load marked on plans. Also, step created by shrinking of diameter + caisson is 85' deep with step at 63' deep.