CHRISTIANA HOSPITAL 2010 PROJECT

NEWARK, DE

Technical Report #1

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Structural Option
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Executive Summary

Christiana Hospital project is a 299,000 square foot addition to the Christiana Medical Campus which will expand its cardiovascular program along with adding extra beds, operating rooms, catheterization labs, emergency exam rooms, and an education center in partnership with the Delaware Academy of Medicine. The project is two-phase and is expected to be completed in 2007.

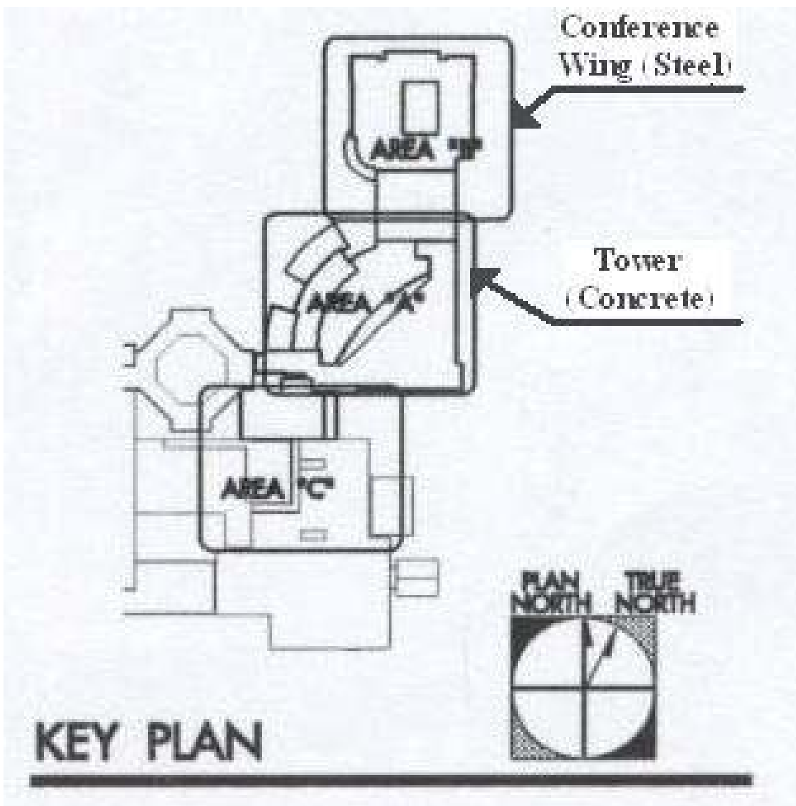
The structure is essentially L-shaped having a center tower joining the two legs. This design brings a contemporary feel to the medical campus with its unique shape and large spans of glass.

This paper is designed to explain the overall structural systems used in the Christiana Hospital project. These systems include the foundation, columns, floor systems and the lateral force resisting systems. In addition to an explanation, spot checks will be performed on these structural elements to both verify the loading patterns and to gain a better understanding of the structure.

Structural Overview

The building is mainly composed of structurally reinforced concrete with a stand alone adjacent steel framed conference wing. The concrete portion of the building stands 8 stories with one level underground and a penthouse roof. The structure contains varying spans which are created using a typical 9½ inch thick two-way flat slab with 5½ inch drops or shear caps. This slab transfers load to 24 inch square columns which in turn take the load down to a mat foundation. To prevent rotation and lateral displacement due to wind or seismic loading shear walls are strategically placed perpendicular to the buildings perimeter.

The conference wing is a 3 story structural steel frame with a majority of beams having pinned connections (figure 7 of Appendix) and spanning around 30 feet. In the center of this area is a larger span of over 60 feet. The buildings loads are transferred to the beams using a 3¼ inch, light weight concrete, structural slab over a 2 inch deep by 18 gage galvanized composite metal deck creating a total slab thickness of 5¼ inches. The load in the beams is transferred to steel girders which are attached using a pinned connection to W-shaped columns. These columns continue down to 4000 psi concrete spread footings. The wind and seismic loading in this area is distributed using concentrically braced frames.



Codes

Codes Used for Original Design

- International Building Code – 2000
- ASCE 7-98, American Society of Civil Engineers – Minimum Design Loads for Buildings and Other Structures
- ACI 318-99, American Concrete Institute – Building Code Requirements for Structural Concrete
- ACI Manual of Concrete Practice – Parts 1 through 5 – 1997
- Manual of Standard Practice – Concrete Reinforcing Steel Institute
- AISC Manual of Steel Construction – Allowable Stress Design, Ninth Ed., 1989
- AISC Manual of Steel Construction – Volume II Connections – ASD Ninth Ed./LRFD First Ed.
- AISC Detailing for Steel Construction
- American Welding Society – Structural Welding Code ANSI/AWS D1.1-96
- Steel Deck Institute – Design Manual for Floor Decks and Roof Decks

Codes Used for Thesis Design

- International Building Code – 2003
- AISC Manual of Steel Construction – Load and Resistance Factor Design, Third Ed., 2005

Structural System

Foundation:

The building consists of two separate types of foundations. In the concrete tower area the building rests on a 42” thick mat foundation. This mat is reinforced with #9’s at 12” o.c. each way, top and bottom, with additional reinforcing added where needed.

In the area of the conference wing, steel columns rest on concrete spread footings. These footings range in size from 4’x4’x 15” deep up to 16’x16’x 48” deep. The allowable soil bearing pressure for this site is 4000 psf.

Applications	Concrete Strengths (f'_c)
Footings	4000 psi
Mat Foundation	6000 psi
Grade Beams	4000 psi
Slab-On-Grade	3500 psi

Columns:

In the tower area a majority of the columns are 24”x24” reinforced concrete columns with only a few occurrences of 12”x24” columns. At the eighth floor nearly all the concrete columns stop and off of them W8 steel columns are posted. The 3 story conference wing is composed of W10 and W12 steel columns.

Applications	Material
Steel Columns	ASTM A992, Grade 50
Concrete Columns (Below Third Floor)	4000 psi
Concrete Columns (Above Third Floor)	5000 psi

Floor System:

Throughout the tower, spans are accomplished using 9½” thick two-way flat slabs with typical 5½” drops or shear caps at each column. Reinforcement for the slabs varies throughout the building.

The conference area uses a completely separate type of floor system. Here steel girders span between columns in one direction while beams, spanning in the opposite direction, frame into the girders. This steel framework works in composite action with the floor slab placed on top. The slab is constructed of 3¼" lightweight concrete over a 2" deep x 18 gage galvanized composite metal deck. The slab is then reinforced with 6x6-W2.1xW2.1 WWF. The bulk of the spans vary anywhere from 20 to 40 feet. Although, running across the middle, is a large 63 foot span made possible using W30x90 beams and the composite action. A spot check for this large span has been done later in this paper.

Lateral Force Resisting System:

The lateral forces acting on the building are resisted differently in the two areas of the building. In the concrete portion of the building, lateral forces are resisted by reinforced concrete shear walls which run the entire height of the building. These shear walls are placed in specific areas to also oppose the torsion effect that the lateral loads place on the building due to its L-shape.

In the conference wing lateral loads are taken care of with the use of concentrically braced frames. These frames are constructed using rectangular HSS steel. This framing is field welded to gusset plates. These gusset plates are attached in the fabrication shop, by means of a weld, to select beams. Refer to figure 4, 5, and 6 in the Appendix for examples of the frame and its corresponding connections.

Roof System:

The framing of the roof is done entirely with steel and metal decking. The decking used is a 1½" deep, wide rib, 20 gage galvanized metal deck. On top of the decking is a one hour fire rated roof construction. This consists of a 45 mill fully adhered roofing membrane on tapered insulation on 5/8" exterior gypsum board. The metal decking is also sprayed with a fireproofing at the soffits.

Gravity Loading

Floor Live Loads	
Occupancy or Use	**Uniform Live Load (psf)**
Assembly Space	100
Typical Hospital Floor	60
Corridor	80
Mechanical Rooms	150
Stair	100
Roof	15
Partition	20

Floor Dead Loads	
Occupancy or Use	**Dead Load**
Reinforced Concrete	150 pcf
Steel Members	Varies
Floor Superimposed	15 psf
Roof Superimposed	15 psf

Snow Loading	
Item	**Value**
Ground Snow Load (P_g)	25 psf
Exposure Category	B
Roof Exposure	Partially Exposed
Exposure Factor (C_e)	1.0
Thermal Factor (C_t)	1.0
Occupancy Category	IV
Importance Factor (I_s)	1.2
Flat-Roof Snow Load $P_f = 0.7C_eC_tI_sP_g$	21 psf

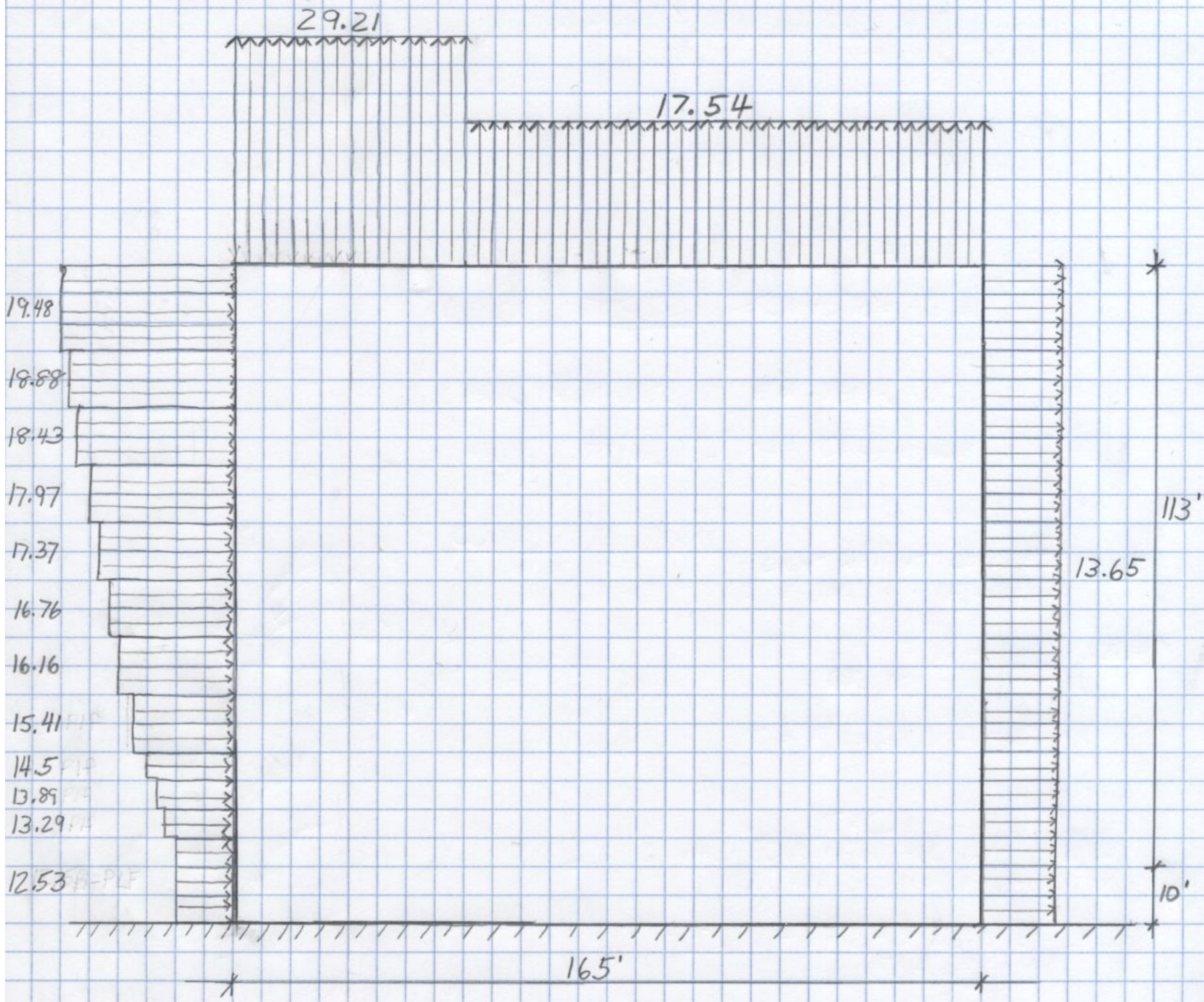
Wind Loading

Assumptions: For the wind loading calculations, only one side of the building was calculated. The side chosen was the plan North face of the building. This was done because it is both the longest and tallest side of the building. By doing this the largest wind loads were found. For simplicity these loads will then be applied to all other faces according to their heights.

Exposure Category	**K_{zt}**	**K_d**	**I**	**V (mph)**	**h (ft)**	**G**	**GC_{pi} (+/-)**
B	1	0.85	1.2	90	114	0.893	0.18

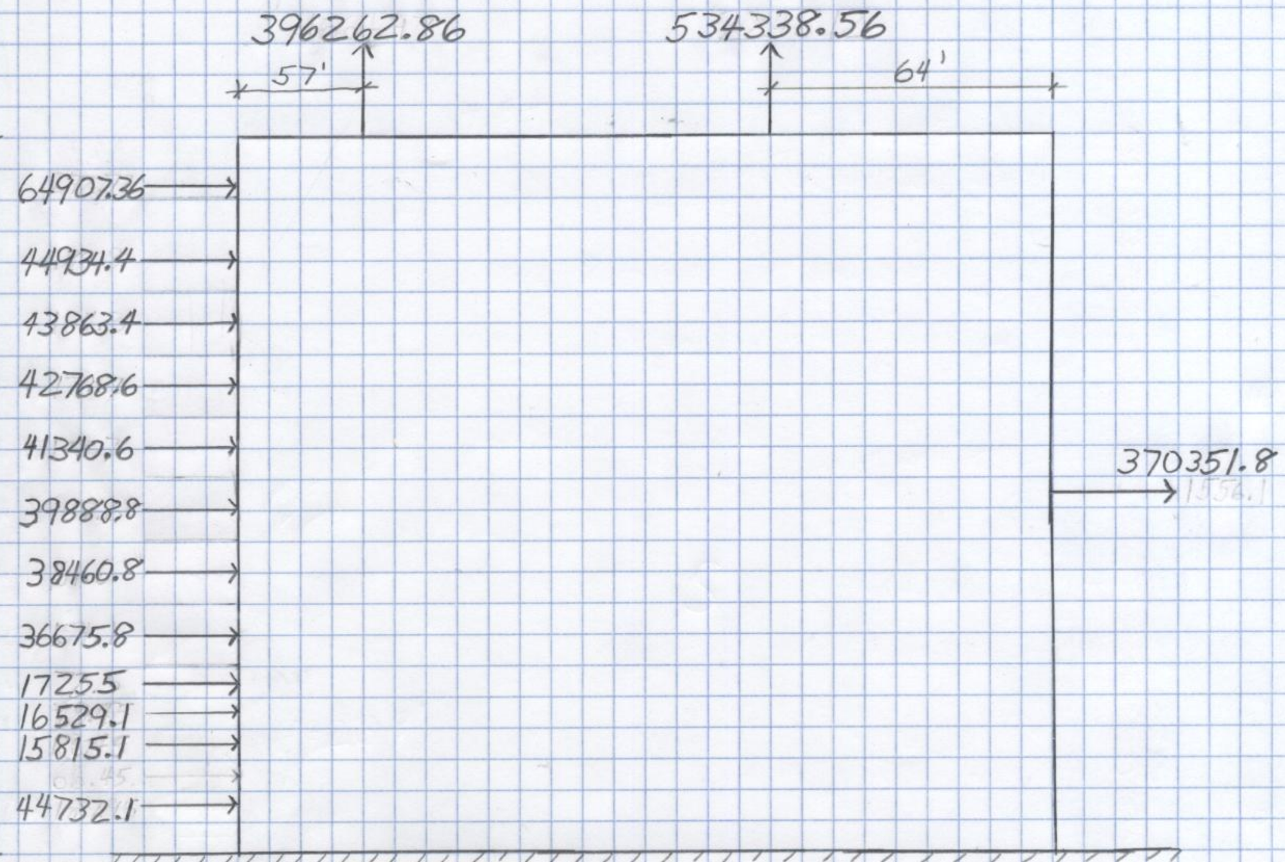
Wind Design Pressures							
			Windward	Leeward	Side Walls	Roof	
						0-57'	>57'
		C_p	0.8	-0.5	-0.7	-1.3	-0.7
h (ft)	K_z	q_z	p (psf)				
0-15	0.57	12.0559	12.53	-13.65	-17.54	-29.21	
20	0.62	13.1134	13.29	-13.65	-17.54	-29.21	
25	0.66	13.9595	13.89	-13.65	-17.54	-29.21	
30	0.7	14.8055	14.5	-13.65	-17.54	-29.21	
40	0.76	16.0745	15.41	-13.65	-17.54	-29.21	
50	0.81	17.1321	16.16	-13.65	-17.54	-29.21	
60	0.85	17.9781	16.76	-13.65	-17.54	-29.21	
70	0.89	18.8241	17.37	-13.65	-17.54		-17.54
80	0.93	19.6702	17.97	-13.65	-17.54		-17.54
90	0.96	20.3047	18.43	-13.65	-17.54		-17.54
100	0.99	20.9392	18.88	-13.65	-17.54		-17.54
114	1.03	21.7852	19.48	-13.65	-17.54		-17.54

WIND PRESSURES (PSF)



B=238'

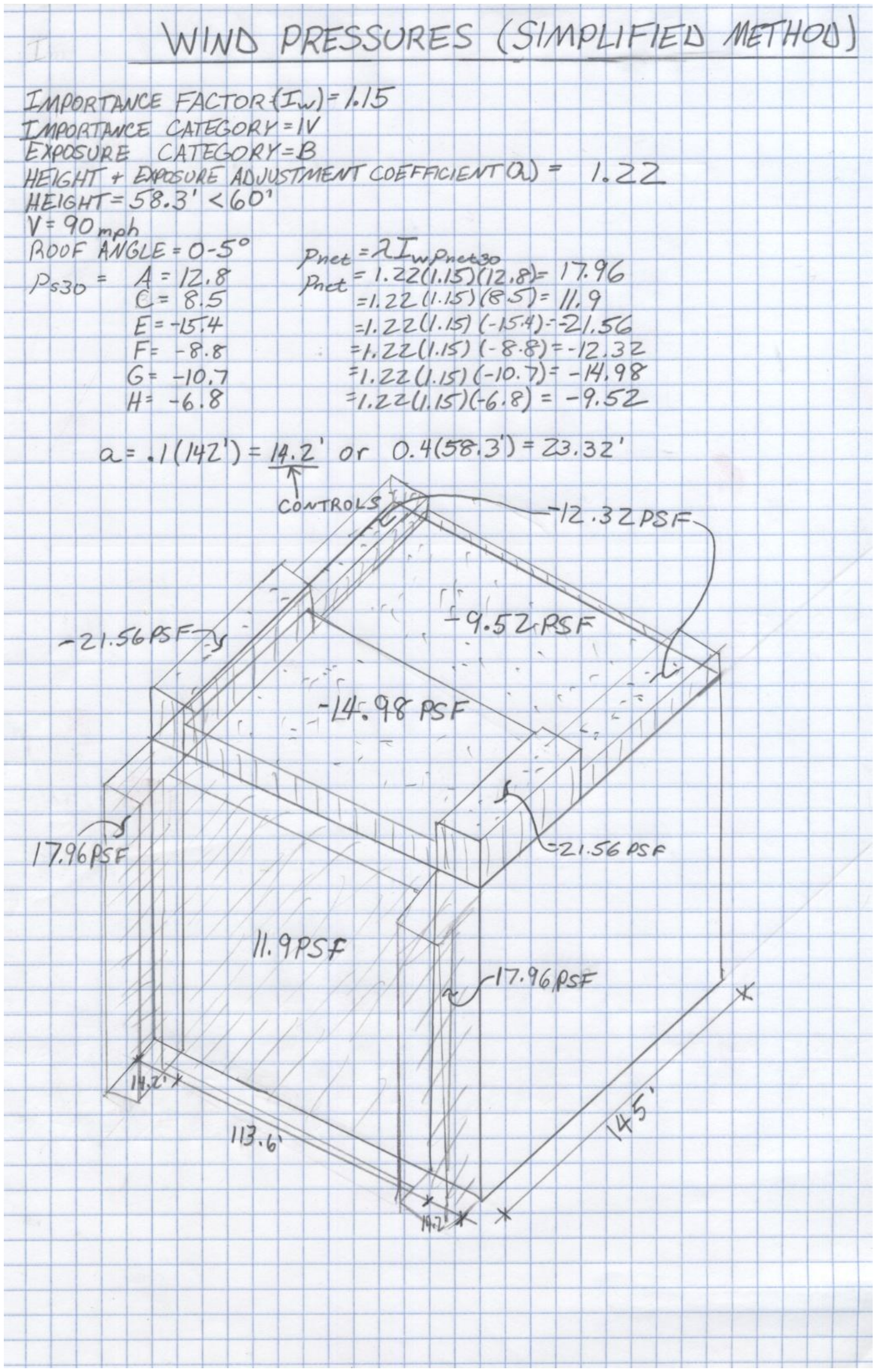
WIND PRESSURES (lbs.)



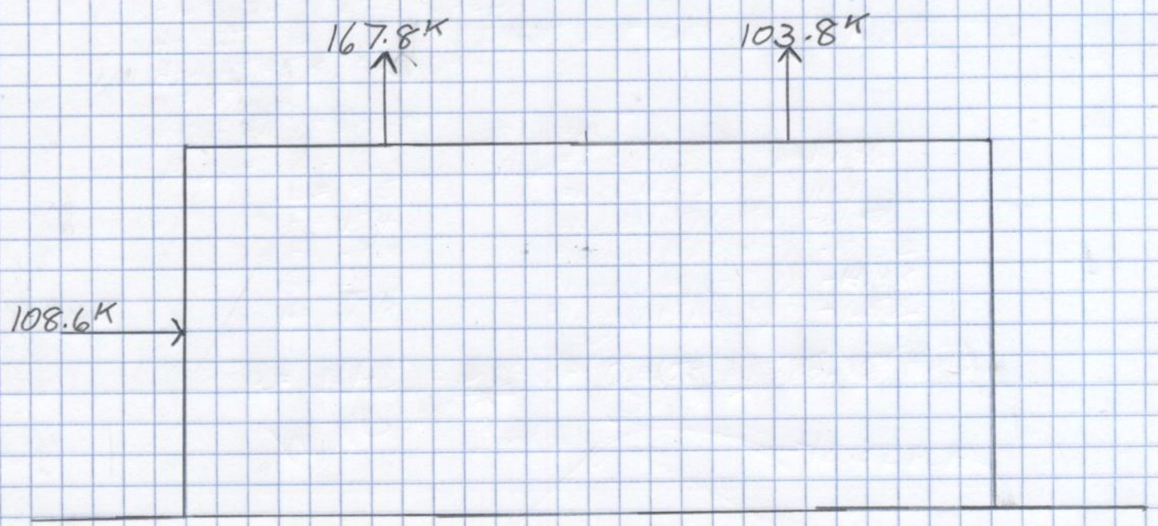
BASE SHEAR (V) = 817522.86 lbs = 817.52 kips

OVERTURNING MOMENT (M) = 44.73(7.5') + 15.82(17.5') + 16.53(22.5') + 17.26(27.5') + 36.68(35') + 38.46(45') + 39.89(55') + 41.34(65') + 42.77(75') + 43.86(85') + 44.93(95') + 64.91(107') + 396.26(108') + 534.34(64') + 370.35(57') = 125,606 ft-kips

When computing the wind pressures on the smaller steel portion of the building, the conference wing, the simplified method was used. This was done because this portion of the building met the simplified methods criterion and was less than 60 feet tall.



WIND PRESSURES (SIMPLIFIED METHOD) CONT.



$11.9\,PSF(58.3')(113.6') = 78812.3\,lbs = 78.81\,K$

$2(17.96\,PSF)(58.3')(14.2') = 29736.7\,lbs = 29.74\,K$

$78.81 + 29.74 = 108.6^K$

$-14.98\,PSF(113.6')(\frac{145'}{2}) = -123375.3\,lbs = -123.4^K$

$2(-21.56\,PSF)(14.2')(\frac{145}{2}) = -44392\,lbs = -44.4^K$

$-123.4 - 44.4 = -167.8^K$

$-9.52\,PSF(113.6')(\frac{145'}{2}) = -78406.7\,lbs = -78.4^K$

$2(-12.32\,PSF)(14.2')(\frac{145'}{2}) = -25366.9\,lbs = -25.4^K$

$-78.4 - 25.4 = -103.8^K$

BASE SHEAR $(V) = 108.6^K$

OVERTURNING MOMENT $(M) = 108.6^K(\frac{58.3'}{2}) + 167.8^K(\frac{72.5'}{2} + 72.5') + 103.8^K(\frac{72.5}{2})$

$= 25177\,ft\text{-}K$

Seismic Loading

Seismic Use Group	Importance Factor	Site Class	S_{MS}	S_{M1}	S_{DS}	S_{D1}
III	1.5	D (Stiff Soil)	0.468	0.192	0.312	0.128

Tower (Concrete Area)

$R = 5$ $C_s = 0.0589$ $k = 1.08$

$C_d = 2.5$ $T = 0.651$

Level	Height (ft)	w_x (k)	$h_x^k w_x$	C_{vx}	F_x (k)	M_x (ft-k)
B	0	0	0	0	0	0
1	14	5331	92177.93	0.029637	68.56384	959.8938
2	29.33	5163	198426.3	0.063799	147.5935	4328.919
3	40.66	4858	265679.6	0.085423	197.618	8035.147
4	52	4858	346530.2	0.111418	257.7563	13403.33
5	63.33	4858	428741.7	0.137851	318.9069	20196.38
6	74.66	4858	512144.5	0.164667	380.9436	28441.25
7	87.33	4932	615856.6	0.198013	458.0868	40004.72
8	100	3999	578031.4	0.185851	429.9516	42995.16
R	118	420	72590.85	0.02334	53.99457	6371.359
∑		39277	3110179			

Base Shear: V (kips) = 2313.4153
Overturning Moment: M (ft-kips) = 164736.162

Conference Center (Steel Area)

$R = 3$ $C_s = 0.156$ $k = 1$

$C_d = 2$ $T = 0.355$

Level	Height (ft)	w_x (k)	$h_x^k w_x$	C_{vx}	F_x (k)	M_x (ft-k)
B	0	0	0	0	0	0
1	32	2344	75008	0.44911	1038.99	33247.5
2	29.33	2355	69072.2	0.41357	956.764	28061.9
R	46.33	495	22933.4	0.13731	317.665	14717.4
∑		5194	167014			

Base Shear: V (kips) = 810.264

Overturning Moment: M (ft-kips) = 76026.8698

In both the concrete tower and steel conference wing the seismic loads ended up being larger than the wind loads. Due to this I shall use the seismic loads in my lateral analysis.

Column Spot Check

After completing a column load take down by means of tributary areas for Column Z.8-92, the final working load was compared to the final working load used by the engineer. In my calculations a working load of 1207 kips was obtained which can be rounded up to 1210 kips. The engineer had used this exact load for his working load which confirms my loading in this area of the building.

Level	Load Type	Height (ft)	Size (in x in)	Trib, A_T (ft^2)	Cum Trib (ft^2)	*K_{LL} (IBC)	Design LL (psf)	Allowable Reduction	Reduced LL (psf)	Slab t, (in)	P_{Slab} (kips)	P_{Drop} (kips)
Roof	Roof	18	W8 x 31	537	537	4	36	0.00	36	9.5	64	
8	Mechanical	12.67	24 x 24	537	1073	4	150	**0.00**	150	9.5	64	7
7	Floor	12.67	24 x 24	537	1610	4	80	0.44	35	9.5	64	7
6	Floor	11.33	24 x 24	537	2147	4	80	0.41	33	9.5	64	7
5	Floor	11.33	24 x 24	537	2684	4	80	0.40	32	9.5	64	7
4	Floor	11.33	24 x 24	720	3403	4	80	0.40	32	9.5	85	7
3	Assembly			292	3695	4	100	**0.00**	100	9.5	35	7
3	Floor	11.33	24 x 24	428	4123	4	80	0.40	32	9.5	51	7
2	Assembly			292	4415	4	100	**0.00**	100	9.5	35	7
2	Floor	15.33	24 x 24	428	4842	4	80	0.40	32	9.5	51	7
1	First Floor	14	24 x 24	720	5562	4	100	**0.00**	100	9.5	85	7
Ground/Foun	Floor				5562	4	80	**0.00**	80	SOG	0	

Level	P_{Col} (kips)	SDL (psf)	P_{SDL} (kips)	P_{LL} (kips)	P_{DL} (kips)	SP_{LL} (kips)	SP_{DL} (kips)	SP_{Total} (kips)	SP_u (kips)
Roof	**0.558**	15	8	19.32	72	19.3212	72	91.7	117.7
8	7.602	15	8	80.51	86	99.8262	159	258.4	350.0
7	7.602	15	8	18.76	86	118.585	245	363.4	483.6
6	6.798	15	8	17.68	85	136.269	330	466.6	614.4
5	6.798	15	8	17.17	85	153.444	416	569.2	744.4
4	6.798	15	11	23.03	110	176.471	526	702.1	913.2
3	0	15	4	29.21	46	205.681	572	777.3	1015.0
3	6.798	15	6	13.68	71	219.361	642	861.8	1121.9
2	0	15	4	29.21	46	248.571	688	937.0	1223.8
2	9.198	15	6	13.68	73	262.251	762	1023.9	1333.6
1	8.4	15	11	71.96	112	334.211	873	1207.4	1582.5
Ground/Foun	0	15	0	0	0	334.211	873	**1207.4**	**1582.5**

Below I have set up a column schedule for this particular column. In the Appendix I have supplied the interaction diagrams, generated using PCA Column, which I used for obtaining the amounts of steel used in these columns. For ease a large moment of 300 ft-kips was applied to the column to check if it would work. In the diagrams you will see that the columns have more than enough moment capacity to be considered safe. The actual column loads were used for the axial force.

For constructability purposes the rebar sizes were kept the same and the amount of rebar in the column was kept the same in two floor lifts. Where the concrete strength changes, at the third floor, is an exception to this two floor lift idea. 12#11's was unnecessary here. 8#11's is also slightly strong but it will be easier for the contractor to taper off 4 bars instead of 8, which would end up being the next amount possible to use.

Shown below is a column schedule comparing my rebar sizes alongside the engineer's. As you can see I agreed with all the sizes the engineer had chosen.

Size Reinforcement

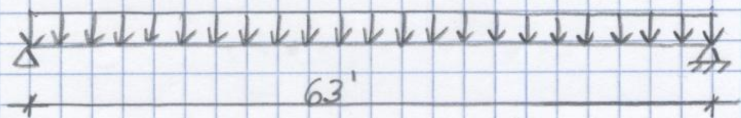
Column Z.8-92			
Floor	**Mine**	**Engineers**	
Roof			
Eighth	W8x35	W8x35	
Seventh	24x24 4#11	24x24 4#11	
Sixth	24x24 4#11	24x24 4#11	
Fifth	24x24 4#11	24x24 4#11	4000 psi
Fourth	24x24 4#11	24x24 4#11	
Third	24x24 4#11	24x24 4#11	
Second	24x24 8#11	24x24 8#11	
First	24x24 16#11	24x24 16#11	5000 psi
Ground	24x24 16#11	24x24 16#11	

Beam Spot Check

For the beam spot check I looked at a composite steel beam in the conference center. The beam looked at was a W30x90 which had a 63 foot span, the largest span in this area. On top of this beam sits a 2” metal deck with a 3¼” topping of lightweight concrete with a strength of 3000 psi. The composite action is formed using ¾” diameter studs.

In the check, a live load of 100 psf was used along with a superimposed dead load of 15 psf. After completing the calculation it was observed that the beam in combination with the composite action contained more than enough strength for the given span. The reason for the beam being larger than needed for strength purposes was most likely to account for the large deflections that are inherent with such a span. One other reason could be the vibrations that would occur in an assembly space with a smaller member.

STEEL BEAM DESIGN



LL = 100 psf
DL = 15 psf

$W = 1.2D + 1.6L = 1.2(15) + 1.6(100) = 182.5\ psf$

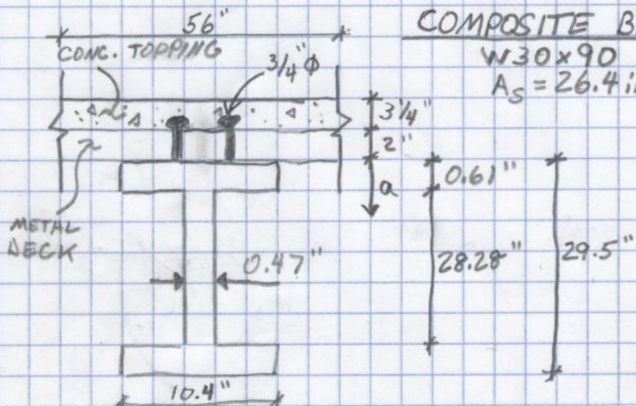
TRIB WIDTH = 9.33'

$w = 9.33'(182.5 psf) = 1703\ plf$

$V = \frac{wl}{2} = \frac{1703(63')}{2} = 53645\ lb = 53.6^K$

SHEAR VALUE USED BY ENGINEER = 52^K

$M = \frac{wl^2}{8} = \frac{1703(63')^2}{8} = 844901\ ft\text{-}lb = 845\ ft\text{-}k$



COMPOSITE BEAM

W30x90

$A_s = 26.4 in^2$

LIGHT WEIGHT CONCRETE

$f'_c = 3ksi \qquad F_y = 60ksi$

$C_c = 0.85 f'_c A = 0.85(3)(56)$
$= 143\ K$

$T_s = F_y A = 60(26.4) = 1584K$

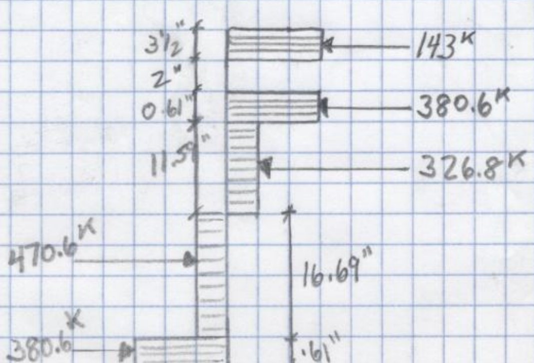
$T = C$

$143 + 60(10.4)(.61) + 60(.47)(a - .61) =$
$60(.47)(28.28 + 0.61 - a) + 60(10.4)(.61)$

$523.6 - 17.2 + 28.2a = 1195 - 28.2a$
$56.4a = 688.6$
$a = 12.2"$

$b_{eff} \leq 1/8\ SPAN = 1/8(63) \times 12 = 94.5"$

$b_{eff} \leq 1/2\ SPACING = \frac{1}{2}(9.33) \times 12 = 56"$



$M_n = 143(3.75) - 380.6(\frac{0.61}{2}) - 326.8(5.8 + .61)$
$+ 470.6(8.3 + 11.59 + .61) + 380.6(0.31 + 16.69 +$
$11.59 + 0.61) = 19086\ in\text{-}k$
$= 1591\ ft\text{-}k > 845\ ft\text{-}k \therefore OK$

STUDS

$Q_N = 14.6^K$

$\frac{143^K}{14.6^K} = 10\ STUDS \times \frac{2}{RIB} \times 2\ SIDES = 40\ STUDS$

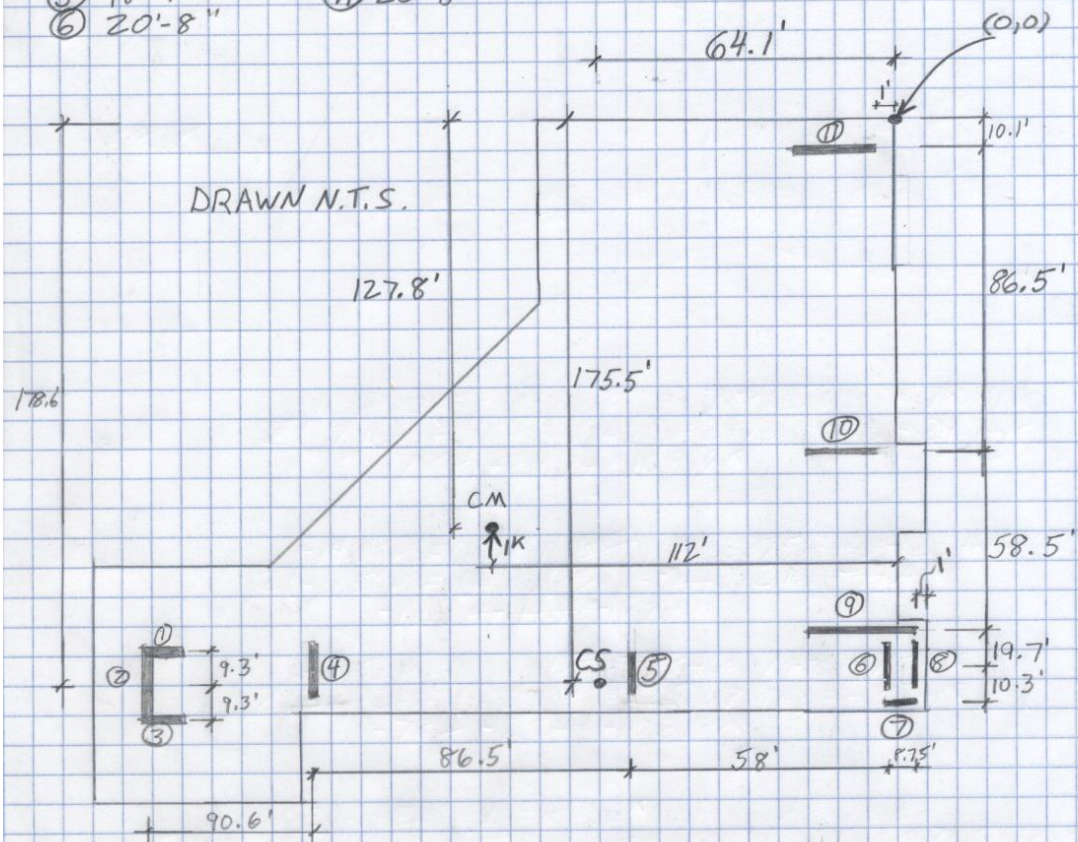
THEY USE 63 STUDS > 40 STUDS ∴ OK

Lateral Force Resisting System Spot Check

SHEAR WALL LATERAL DISTRIBUTION

SIZES (ALL SHEAR WALLS ARE 1' THICK)

①	12'-8"	⑦	8'-9"
②	19'-6"	⑧	20'-8"
③	12'-8"	⑨	26'-2"
④	23'-6"	⑩	18'-5"
⑤	18'-7"	⑪	20'-8"
⑥	20'-8"		



CENTER OF STIFFNES (CS):

$$y = \frac{(20'\text{-}8"+20'\text{-}8"+18'\text{-}7"+23'\text{-}6")\,174.8' + (19'\text{-}6")\,178.6'}{2(20'\text{-}8")+18'\text{-}7"+23'\text{-}6"+19'\text{-}6"} = \frac{18064}{102.9} = 175.5'$$

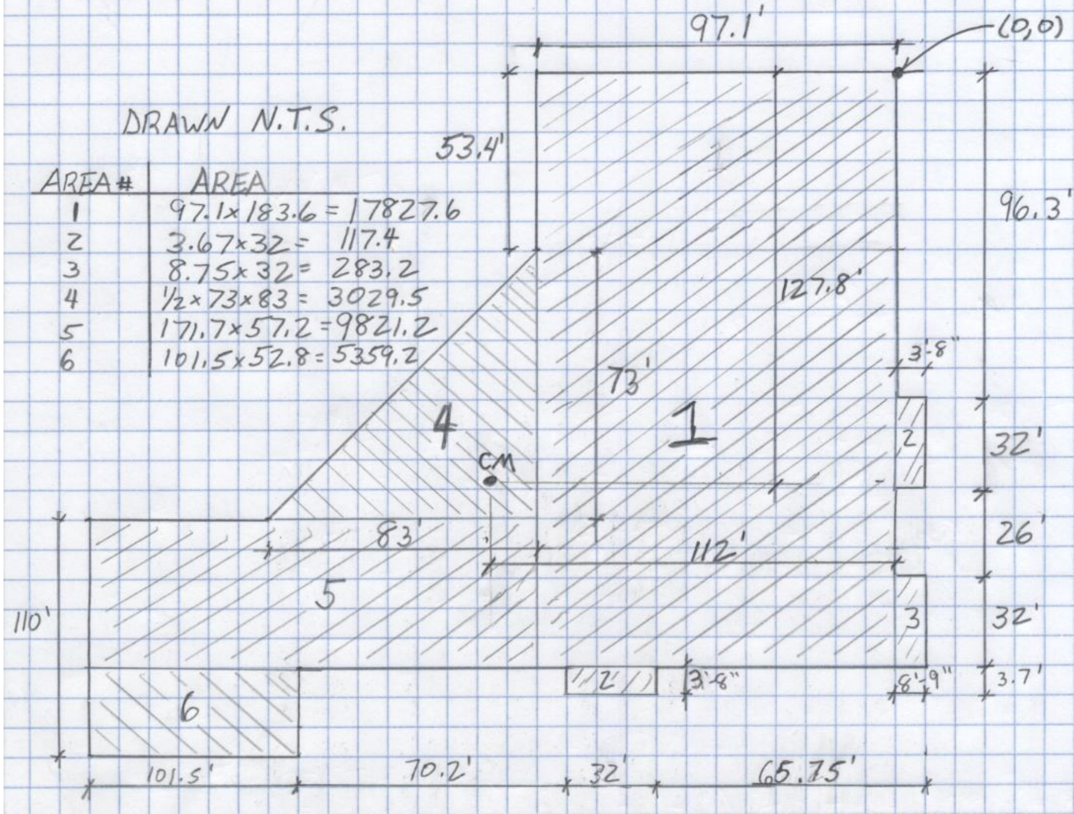
$$x = \frac{(12'\text{-}8"+12'\text{-}8")(230.3') + (20'\text{-}8")(11.3') + (18'\text{-}5")(10.2') + (26'\text{-}2")(5.3') + (8'\text{-}9")(-3.4)}{12'\text{-}8"+12'\text{-}8"+20'\text{-}8"+18'\text{-}5"+26'\text{-}2"+8'\text{-}9"}$$

$$= \frac{6365}{99.3} = 64.1'$$

SHEAR WALL LATERAL DISTRIBUTION CONTINUED

ASSUMPTION: COLUMN SPACINGS ARE FAIRLY UNIFORM AND SLAB THICKNESSES ARE THE SAME, THEREFORE TAKE CENTER OF MASS AS THE CENTER OF GEOMETRY.

MUST FIND THIS BECAUSE THE SEISMIC FORCES CONTROL.



AREA #	AREA
1	97.1×183.6 = 17827.6
2	3.67×32 = 117.4
3	8.75×32 = 283.2
4	½×73×83 = 3029.5
5	171.7×57.2 = 9821.2
6	101.5×52.8 = 5359.2

CENTER OF MASS (CM) =

$$y = \frac{17827.6\left(\frac{183.6}{2}\right) + 117.4(64.15) + 283.2(167.6) + 117.4(185.4) + 9821.2(155) + 5359.2(210) + 3029.5(53.4 + \tfrac{2}{3}\times 73)}{17827.6 + 2(117.4) + 283.2 + 3029.5 + 9821.2 + 5359.2}$$

$$= \frac{4670264}{36555.5} = 127.8'$$

$$x = \frac{17827.6(48.6) + 3029.5(124.8) + 9821.2(174.9) + 5359.2(210) + 117.4(73) - 117.4(1.8) - 283.2(4.4)}{36555.5}$$

$$= 4094776 / 36555.5 = 112'$$

SHEAR WALL
LATERAL DISTRIBUTION
CONTINUED

SPOT CHECK SHEAR WALL #4

TO FIND % OF FORCE ON WALL #4 APPLY
1 KIP LOAD AT CENTER OF MASS
USE 5% OFFSET

$$M = 1^K(112'-64.1')(1.05) = 50.3\,ft\text{-}K$$

POLAR MOMENT OF INERTIA (J)

$$\begin{aligned}J = \Sigma K_i d_i^2 &= (12'\text{-}8'')(9'\text{-}3'')^2 2 + (19'\text{-}6'')(90.6'-64.1')^2 + (23'\text{-}6'')(145.5-64.1)^2 \\ &+ (18'\text{-}7'')(64.1'-58')^2 + (20'\text{-}8'')(64.1'-1')^2 + (8'\text{-}9'')(185.1'-175.5')^2 \\ &+ (20'\text{-}8'')(64.1'+7.75')^2 + (26'\text{-}2'')(175.5'-155.1')^2 + (18'\text{-}5'')(175.5'-96.6')^2 \\ &+ (20'\text{-}8'')(175.5'-10.1')^2 = \\ &= 958425\end{aligned}$$

DIRECT SHEAR FOR WALL #4

$$1^K \; \frac{23'\text{-}6''}{23'\text{-}6''+19'\text{-}6''+18'\text{-}7''+20'\text{-}8''+20'\text{-}8''} = 0.228 \times 1^K$$

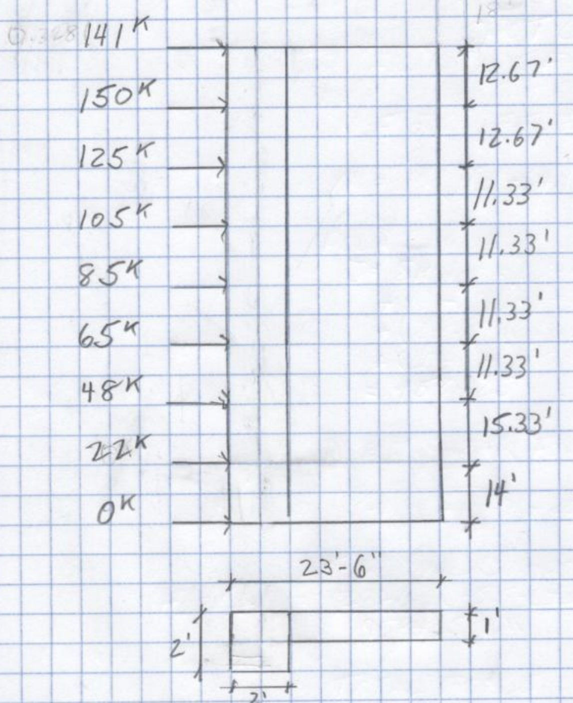
TORSIONAL COMPONENT ON WALL #4

$$F_4 = \frac{Kd}{J}M = \frac{(23'\text{-}6'')(145.5-64.1)}{958425}(50.3) = 0.1^K$$

TOTAL FORCE ON WALL #4

$$F_T = 0.1^K + 0.228^K = 0.328^K$$

SHEAR WALL CHECK



$M_u = 141(100') + 150(87.3') + 125(75)$
$+ 105(63) + 85(52) + 65(41)$
$+ 48(29.3) + 22(14)$
$= 51984 \text{ ft-K}$

$V_u = 741^K$

$P_u = 1873^K \leftarrow$ FOUND USING TRIB. AREA AND COL. LOAD TAKE DOWN

$$C_V = \frac{P_u}{2} + \frac{M_u}{d} = \frac{1873}{2} + \frac{51984}{23.5'} = 3149 = P_{u_{BE}}$$

BOUNDARY CHECK

$A_g = (1')(23.5') = 23.5 \text{ ft}^2$

$$I_g = \frac{(1')(23.5')^3}{12} = 1081 \text{ ft}^4$$

$$f_c = \frac{P_u}{A_g} + \frac{M_u \cdot \frac{h_w}{2}}{I_g} = \frac{1873}{23.5} + \frac{51984(23.5/2)}{1081}$$

$= 644.7 \text{ ksf} = 4.47 \text{ ksi}$

$0.2(f'_c) = 0.2(5 \text{ ksi}) = 1$

$f_c = 4.47 > 1 \therefore$ NEED BOUNDARY ELEMENT

LONG. & TRANVERSE REINFORCEMENT

$2A_{cv}\sqrt{f'_c} = 2(12 \times 23.5 \times 12)\sqrt{5000}/1000 = 479^K < 741^K \therefore$ NEED TWO CURTAINS

$A_{cv} = (12")(12") = 144 \text{ in}^2/\text{ft}$ 12" 12" #5 @12"

$A_{s_{\ell\,req'd}} = (0.0025)(144) = 0.36 \text{ in}^2/\text{ft}$

$A_{s\ell} = 2(0.31) = 0.62 \text{ in}^2/s$

$$\frac{0.36}{12"} = \frac{0.62}{s} \Rightarrow s_{req} = 20.6" \geq 12" \therefore OK$$

SHEAR WALL CHECK
CONT.

CHECK SHEAR

$V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_t f_y)$

$\frac{h_w}{\ell_w} = \frac{100'}{23.5'} = 4.26 > 2 \therefore \alpha_c = 2.0$

$A_{cv} = (12)(23.5 \times 12) = 3384 \text{ in}^2$

$\rho_t = \frac{2(0.31)}{(12)(12)} = 0.0043$

$V_n = 3384(2\sqrt{5000} + 0.0043(60000))/1000 = 1352^K$

$\phi V_n = 1352(.6) = 811^K > V_u = 741^K \therefore OK$

CHECK BOUNDARY ELEMENT CAPACITY

$16 \# 11 \quad A_{st} = 16 \times (1.56) = 24.96 \text{ in}^2$

$\rho_{st} = \frac{24.96}{(24)(24)} = 0.043 \quad \rho_{min} = 0.01 < \rho_{st} = 0.043 < \rho_{max} = 0.06$
$\therefore OK$

$\phi P_{n(max)} = 0.8\phi(.85 f'_c (A_g - A_{st}) + f_y A_{st})$
$= 0.8(.7)(.85(5)(576 - 24.96) + 60(24.96)) = 2150^K < 3149^K$
$\therefore$ NOT OK

I FEEL THIS BOUNDARY FAILURE IS MOST LIKELY DUE TO THE HIGH MOMENT. MY CALCULATED MOMENT MAY BE TO HIGH.

Appendix

Figure 1

Concrete Column
24" x 24"
f'c=5000 psi
Fy=60000 psi
16#11 Bars
P=1583 k
M=300 ft-k

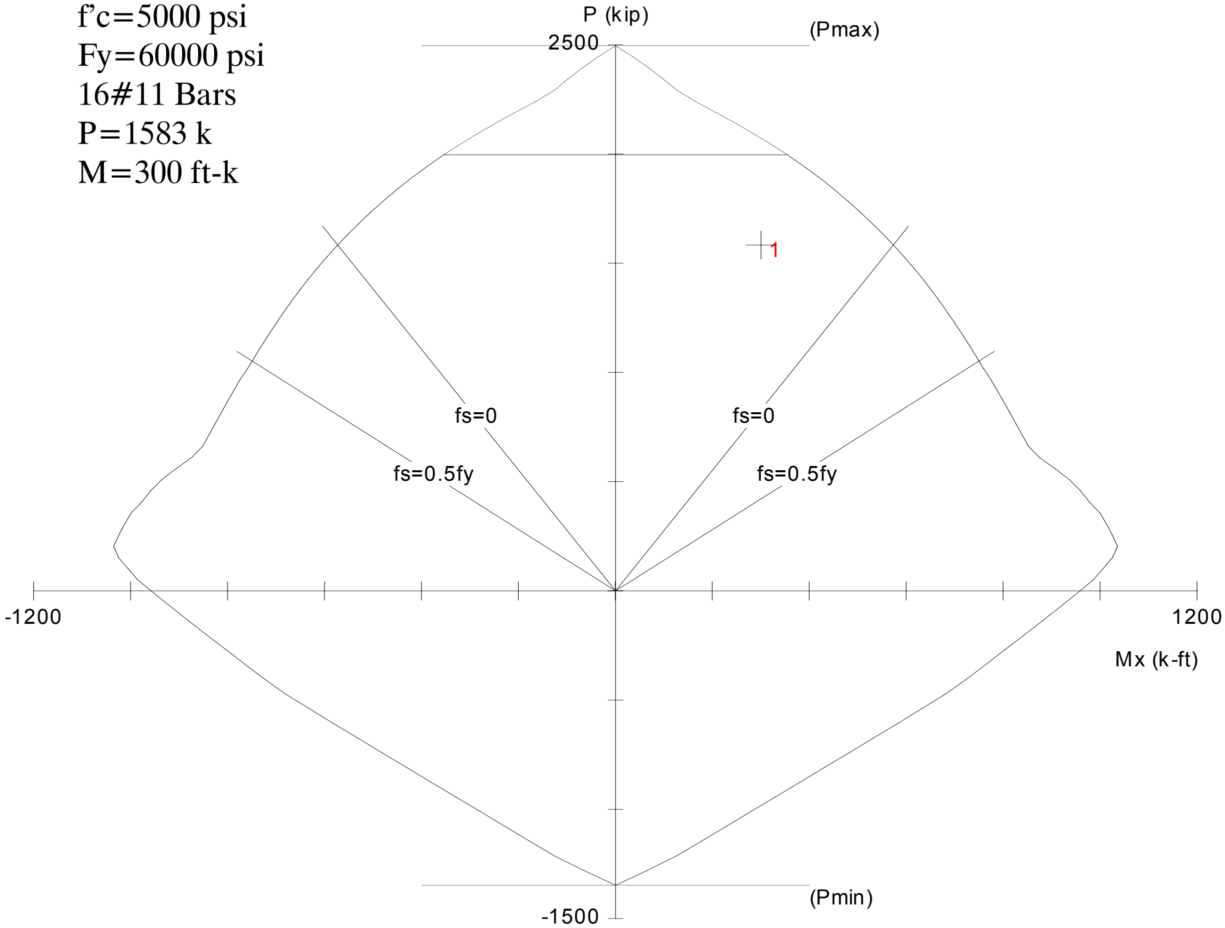


Figure 2

Concrete Column
24” x 24”
f’c=5000 psi
Fy=60000 psi
8#11 Bars
P=1122 k
M=300 ft-k

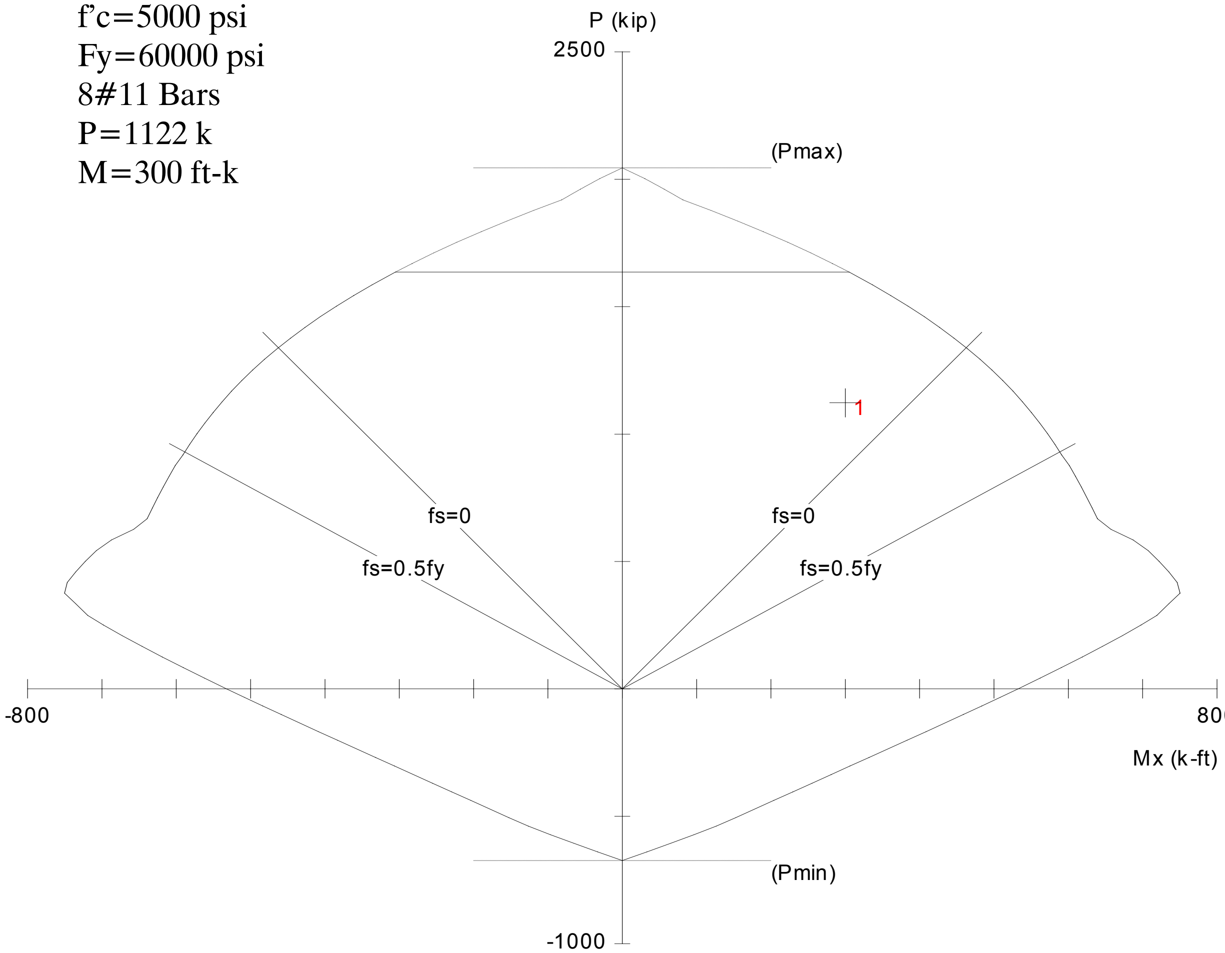
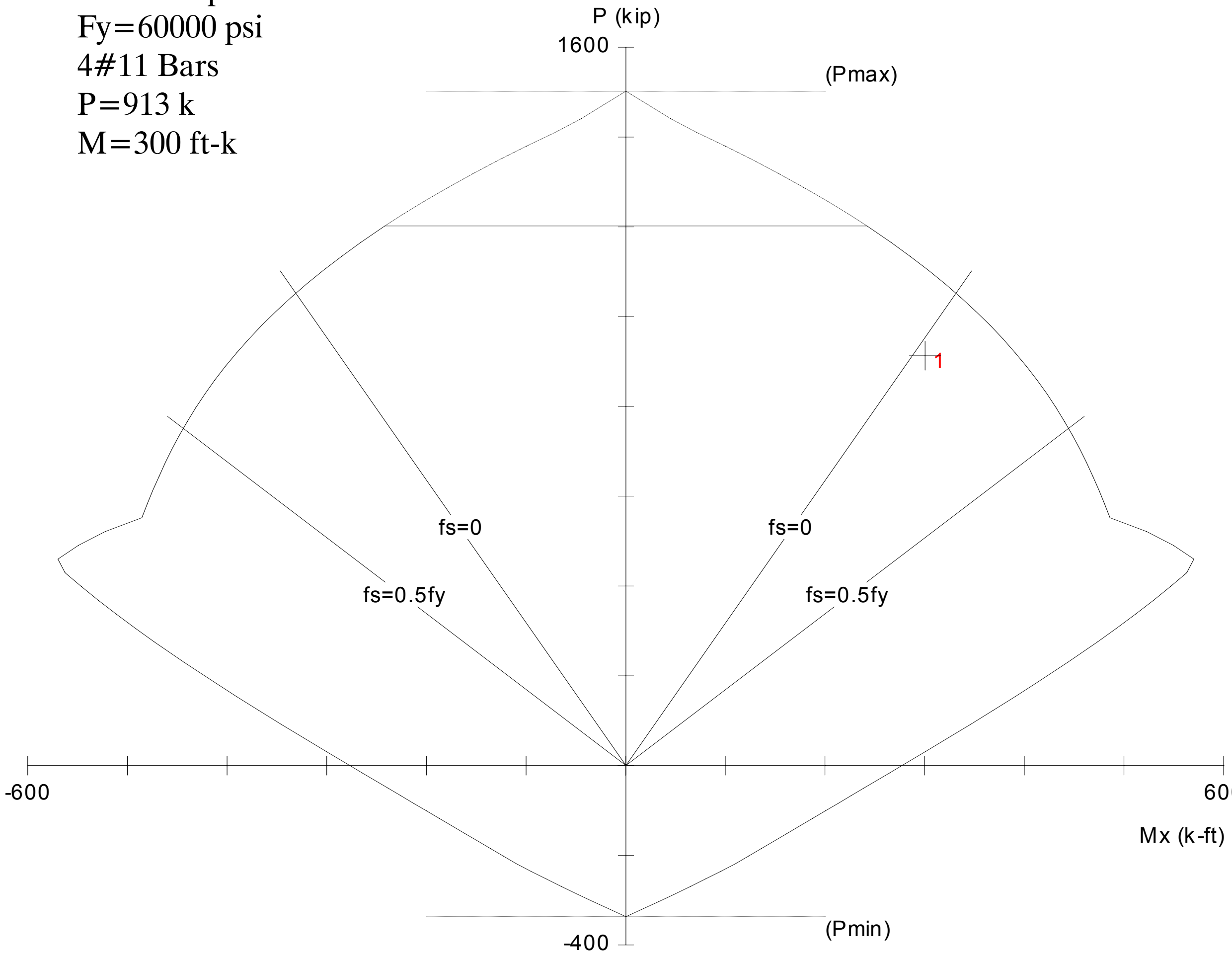


Figure 3

Concrete Column
24” x 24”
f’c=4000 psi
Fy=60000 psi
4#11 Bars
P=913 k
M=300 ft-k



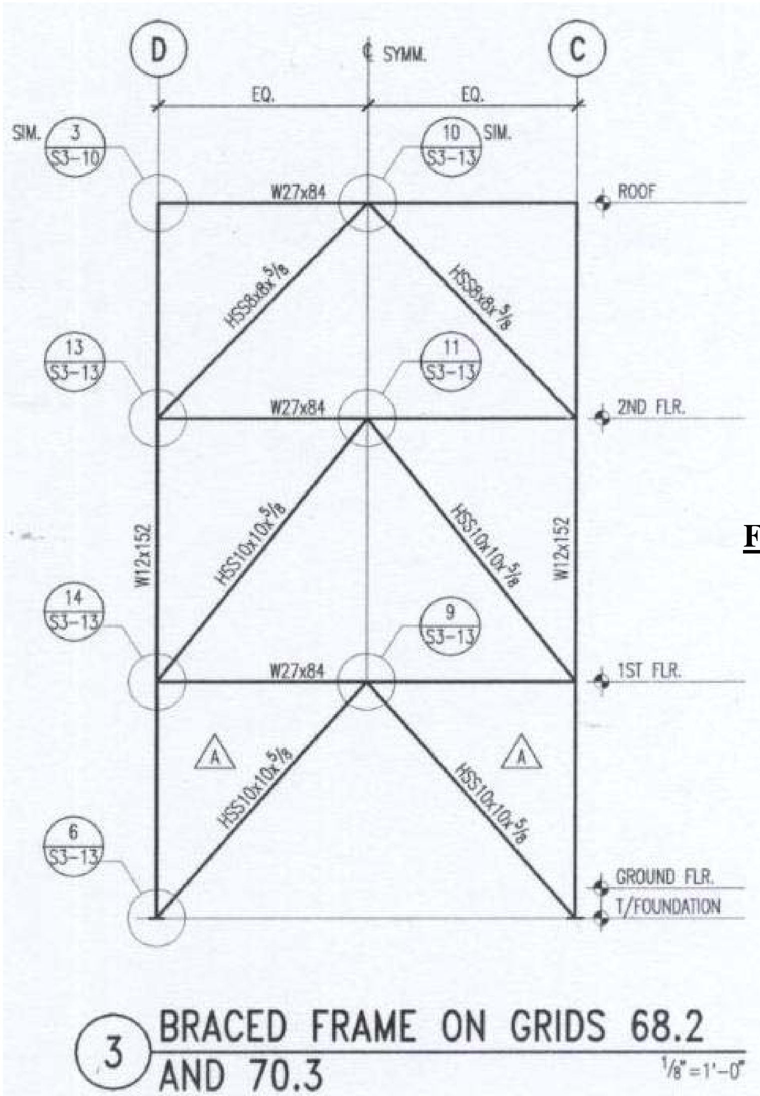


Figure 4

Figure 6

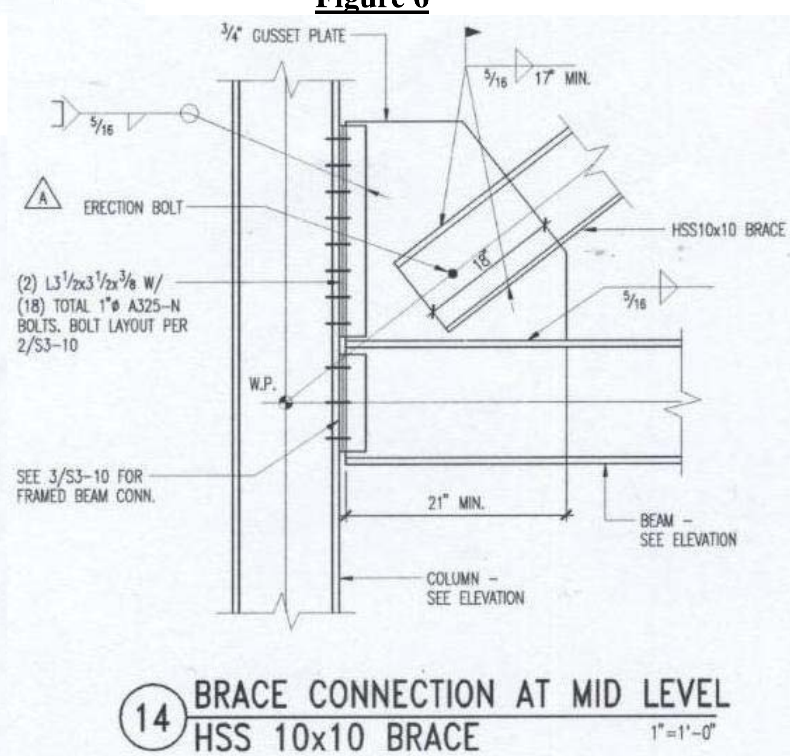
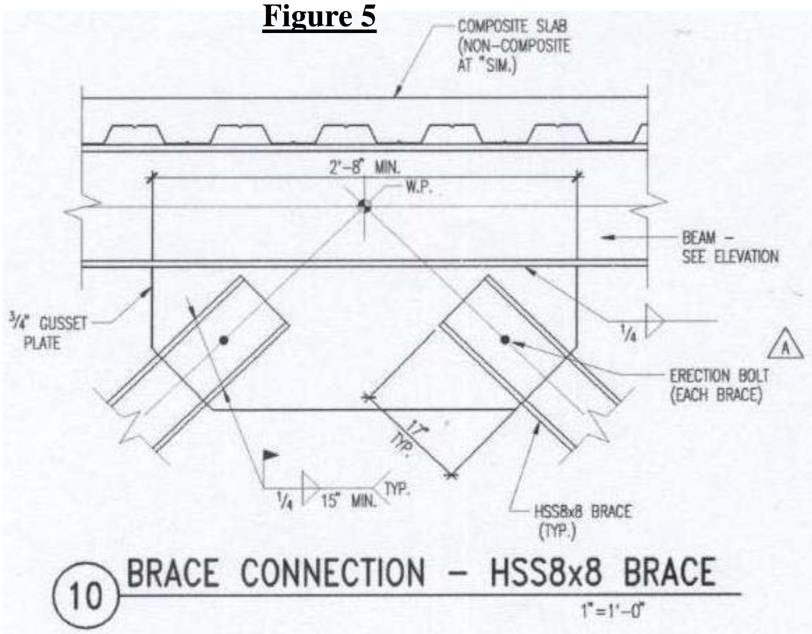
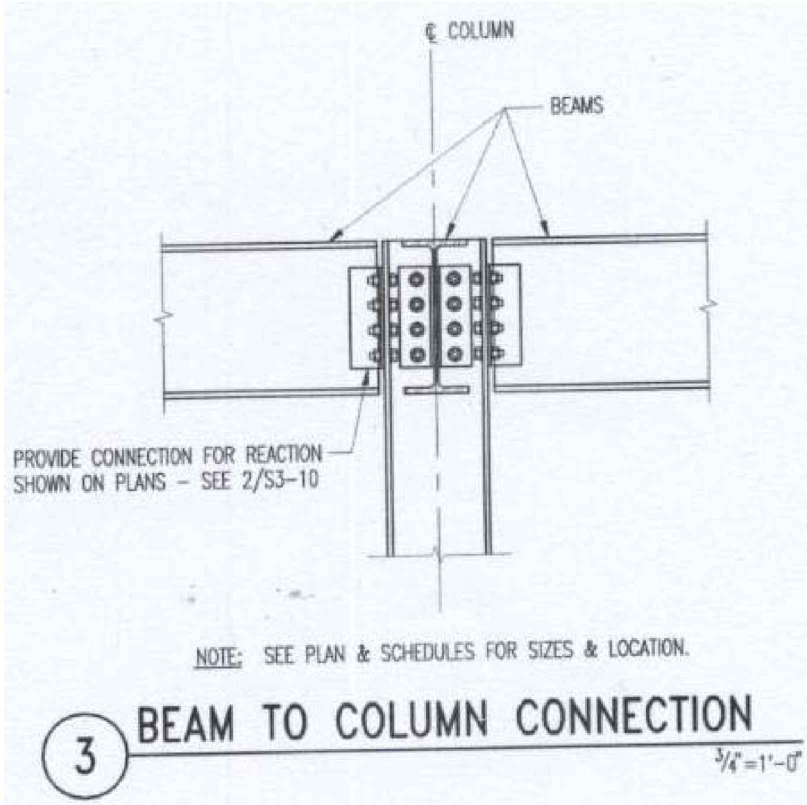


Figure 5





℄ COLUMN
BEAMS
PROVIDE CONNECTION FOR REACTION
SHOWN ON PLANS – SEE 2/S3-10
NOTE: SEE PLAN & SCHEDULES FOR SIZES & LOCATION.
3
BEAM TO COLUMN CONNECTION
3/4" = 1'-0"

Figure 7