MOUNTAIN STATE
BLUE CROSS BLUE SHIELD
HEADQUARTERS

PARKERSBURG, WEST VIRGINIA

DOMINIC MANNO

STRUCTURAL OPTION

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9-29-08

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EXECUTIVE SUMMARY

This first technical report investigates the existing building conditions for Mountain State Blue Cross Blue Shield Headquarters in Parkersburg, West Virginia. A detailed discussion of the foundation, floor, column, and lateral systems are included. Along with these descriptions, plans and elevations are provided for a better understanding of how the building is laid out structurally. A summary of the codes used and material strengths are listed. A detailed lateral load analysis was done to determine wind and seismic loads according to ASCE 7-05. Wind was analyzed using Method 2 of chapter 6, and seismic was determined by chapters 11 and 12. After running the calculations I found that seismic controlled the design. Spot checks of a typical floor bay and column were performed. I found that my loads were conservative and the floor system checked. The lowest level of the column used by the designer was a size smaller than my calculation. Appendices are provided to make available my calculations, figures, and tables.

INTRODUCTION TO MOUNTAIN STATE BLUE CROSS BLUE SHIELD HEADQUARTERS

Mountain State Blue Cross Blue Shield Headquarters Building consists of 4 stories that sit above grade and is mainly office space. Its main purpose for being built was to expand to include the extra 170 employees that are to be hired this year. MSBCBS is located in Parkersburg, WV, which sits on the north-western area of the state near the Ohio border. The building has a brick veneer façade which sits well into the site of downtown Parkersburg. It also has a large glass curtain wall which emphasizes the buildings entrance and gives the building a modern appeal.

The building is approximately 130,000 square feet and has mainly an open floor plan. The buildings top of steel is at a height of 67' – 6.5" above grade due to the screen wall located on the roof for the mechanical units. The floor to floor height of the building is approximately 13'-4". The typical bay size is 30' x 30' being made by composite steel structure and concrete slab on steel decking. The lateral system of the building is made up of four braced frames, two in the north/south and two in the east/west building direction. The foundation contains caissons which extend approximately 70' ft. The ground level consists of a 4" slab on grade with grade beams surrounding the perimeter of the buildings footprint.

STRUCTURAL SYSTEMS

FOUNDATIONS

The foundation system is drilled caissons that range from 30” in diameter to 66”. They were designed to have an allowable skin friction of 550 psf. They contain a variation No. 7 to No. 8 vertical reinforced bars, and have ties that are No. 3 reinforced bars. Depending on the location on the plan the caissons are driven into the ground 59’ to 74’ below grade. The caissons support the steel framed system and the 4” concrete slab on grade. The grade beams surrounding the perimeter of the building are 24” x 30”.

FLOOR SYSTEM

MSBCBS has a composite system with 30’ x 30’ typical bay size. A 3-1/4” light weight concrete slab sits on a 2” – 20 gauge composite steel decking with ¾” studs. The deck is supported by mainly W18 x 35 beams that are spaced 10’ center to center. The majority of the girders are W21 x 62 which transfer the loads from the beams to the columns. This floor system is used for all floors except for the roof and the 4” slab on grade. The roof is made up of an 1-1/2” 20 gauge wide rib galvanized steel deck and is 3 spans continuous with 3” of concrete. The roof floor system is mainly supported by K-series joists that are spaced 6’ center to center.

COLUMNS

The gravity columns for MSBCBS are typically W10’s. The gravity base plates have a 4 bolt connection and have a thickness varying from 1” to 1-5/8”. The lateral columns are W12’s. The lateral base plates typically have a 12 bolt connection with a thickness of 1-1/2” to 2-1/2”. The mechanical screen roof is composed of HSS 12 x 12 x 3/8 post, which connects to the beam, with a 1” thick base plate.

LATERAL SYSTEM

Four braced frames make up the lateral force resisting system for the building. The placements of these braces were based on the location of interior walls throughout the building. The purpose was to be able to conceal the braces within the walls. Several different types were used, from diagonal bracing to x bracing to uneven inverted chevron bracing. All of these braces are laid out in between floor to floor spaces. The braces range from HSS 8x8’s to HSS 10x10’s. The braces are connected using gusset plates with a minimum thickness of the beam’s web thickness. Typical base plates for these lateral columns are 2-1/2” thick with large caissons to transfer the shear forces.

CODE

CODE / REFERENCES

2006 International Building Code

(ACI 318-08) Building Code Requirements for Structural Concrete

Specification for the Design, Fabrication and Erection of Structural Steel Buildings
Allowable Steel Design, 13th Edition, American Institute of Steel Construction

(ASCE - 07) Minimum design loads for Buildings and other Structures
American Society of Civil Engineers

Steel Deck Institute, Design Manual

MATERIALS

Concrete

Foundations	f'c = 4000 PSI
Slab On Grade	f'c = 4000 PSI
Exterior Slabs	f'c = 4500 PSI
Interior Slabs on Metal Deck	f'c = 4000 PSI

Reinforcement

Deformed Bars	ASTM A615, Grade 60
Welded Wire Fabric	ASTM A185

Steel

Structural "W" Shapes	ASTM A992
Structural "M," "S," and "HP" Shapes	ASTM A572, Grade 50
Channels	ASTM A572, Grade 50
Steel Tubes (HSS Shapes)	ASTM A500, Grade B
Steel Pipe (Round HSS)	ASTM A500, Grade B
Angles and Plates	ASTM A36

Metal Deck and Shear Studs

Composite Floor	2" 20 Gauge
Roof Deck	1 ½ " Galvanized
Studs	¾" Diam. 4 ½" Tall

GRAVITY AND DESIGN LOADS

DEAD LOADS

Construction Dead Loads

Concrete	150 PCF
Light Weight Concrete	110 PCF
Steel	490 PCF
Partitions	20 PSF
M.E.P.	10 PSF
Finishes and Misc.	5 PSF
Windows and Framing	20 PSF
Roof	20 PSF

LIVE LOADS

Public Areas	100 PSF
Lobby	100 PSF
Office First Floor Corridor	100 PSF
Office Corridors above First Floor	80 PSF
Offices	50 PSF
Light Storage	125 PSF
Heavy Storage	250 PSF
Mechanical	150 PSF
Stairs	100 PSF

LATERAL LOADS

This section reviews the lateral loads considered for wind and seismic analysis per code ASCE 7-05. For a detailed hand calculation of the loads please refer to Appendix B. Below is a diagram of a typical floor (Figure1), emphasizing where the lateral braces are located in the building by the highlighted blue area. Figures 2 and 3 show elevations of the four braces.

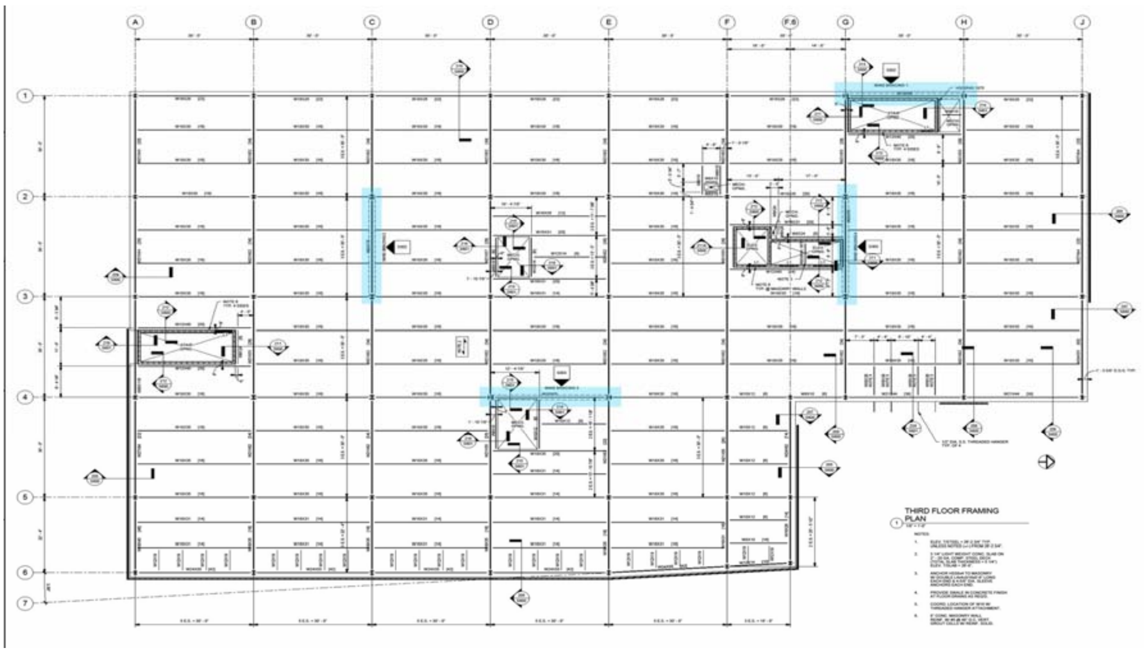


Figure 1: Lateral System Layout

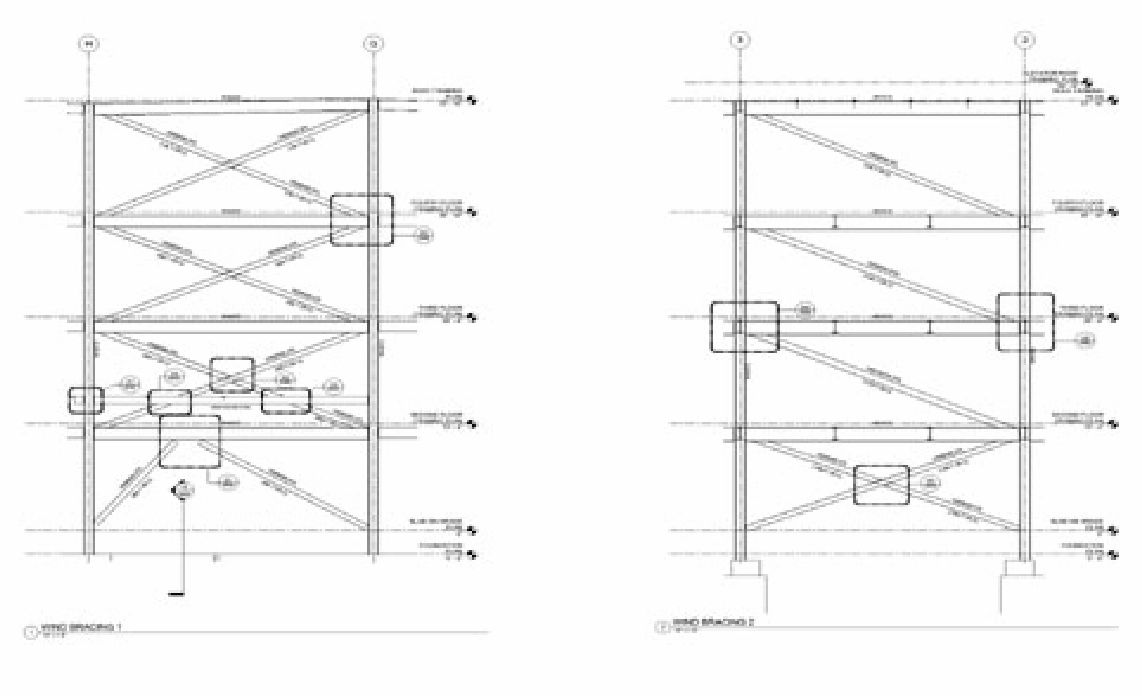


Figure 2: Lateral Bracing 1 and 2 Elevations

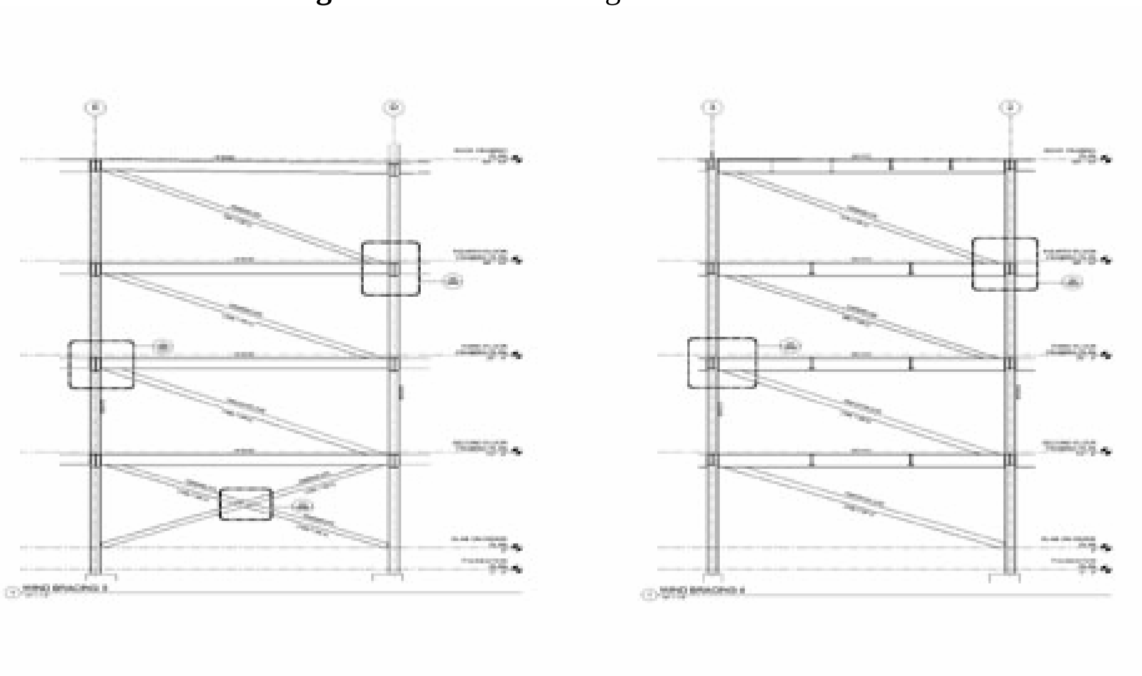


Figure 3: Lateral Bracing 3 and 4 Elevations

WIND

Wind loads were calculated using Section 6.5 from ASCE 7-05 (Method 2: Analytical Procedure. Factors used for this analysis were based off the location of the site and the mean roof height of the building. For the purpose of this calculation the screen roof for mechanical equipment was ignored because its effect on the overall outcome is negligible. Since the building is shaped relative to a rectangle wind coming from both the north-south, and east-west directions had to be considered in the analysis. When calculating the loads, I took the building footprint to be perfectly rectangle ignoring the open layout in the lower right hand corner of the plan. It was also assumed that the building was rigid in nature, which proves to be correct in my results (Appendix C). On the next page you can see the wind pressures for the building in both directions and see that for the wind design the north-south direction controlled with a total of approximately 198 kips for base shear (Figures 4 and 5). Figure 6 shows the distribution of forces according to each floor.

Wind Pressures (psf)							
Floor Heights	Level	Kz	qz	N-S (windward)	N-S (leeward)	E-W (windward)	E-W (leeward)
13.33	2	0.57	10.05	9.66	-3.79	9.84	-2.23
26.67	3	0.70	12.34	11.24	-3.79	11.47	-2.23
39.83	4	0.76	13.40	11.98	-3.79	12.22	-2.23
53.83	Roof	0.85	14.98	13.08	-3.79	13.35	-2.23

Figure 4: Windward and Leeward Wind Pressures at each Floor

Wind Design						
	Load (kips)		Shear (kips)		Moment (ft-k)	
Level	N/S	E/W	N/S	E/W	N/S	E/W
Roof	56.7	31.8	0.0	0.0	2655.26	1489.19
4	49.8	27.8	56.7	31.8	1655.85	924.35
3	48.1	26.7	106.5	59.6	961.52	533.73
2	43.0	23.5	154.6	86.3	286.38	156.51
Total	197.6	109.8	197.6	109.8	5559.01	3103.79

Figure 5: Total Base Shears for each Direction

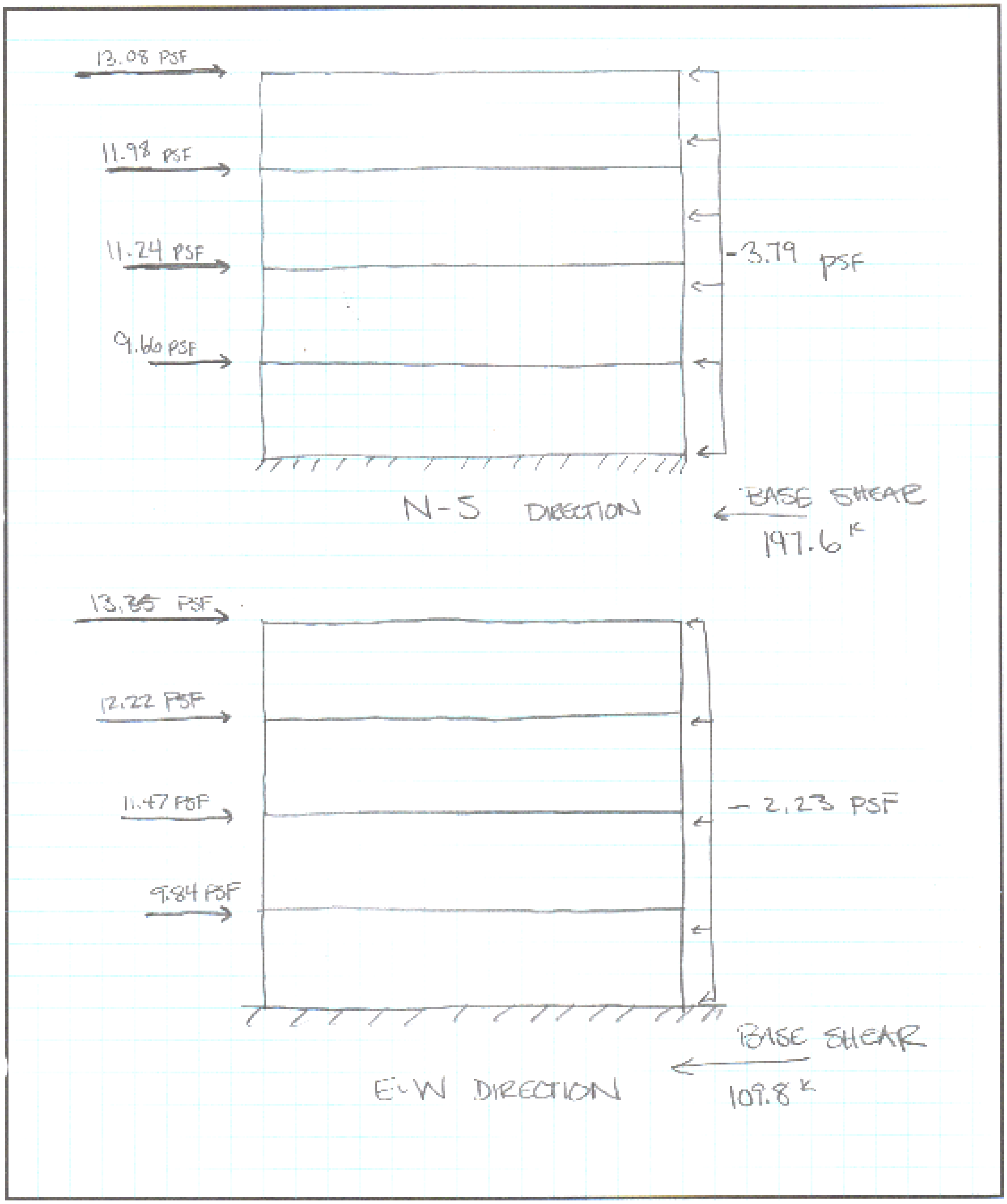


Figure 6: Distribution of Forces per Floor in each Direction

SEISMIC

Seismic loads were analyzed using Chapters 11 and 12 of the code ASCE 7-05. The factors that were used were based on location and related specifically to the building. The floor loads and roof loads were also used to calculate total base shear. A detailed description of these loads and analysis are located in Appendix C. Since there are 2 of the same concentric steel braces in each direction the base shear for will be the same in each direction of the building. Figure 7 below shows the total base shear of 339 kips and an overturning moment of approximately 13,800 ft-kips.

Base Shear and Overturning Moment Distribution							
Level	h_x	Floor Load	$h_x^k w_x$	C_{vx}	$F_x=C_{vx}V$	V_x(k)	M_x (ft-k)
Roof	53.83	2662	282167.2	0.41	138.42	0	7451.10
4	39.83	2883	214827.9	0.31	105.39	138.42	4197.50
3	26.67	2883	134366.7	0.19	65.91	243.80	1757.94
2	13.33	2883	59689.22	0.09	29.28	309.72	390.32
Totals		11311	691051	1.00	339.00	339.00	13796.86

Figure 7: Base Shear and Overturning Moment

REACTION TO LATERAL LOADS

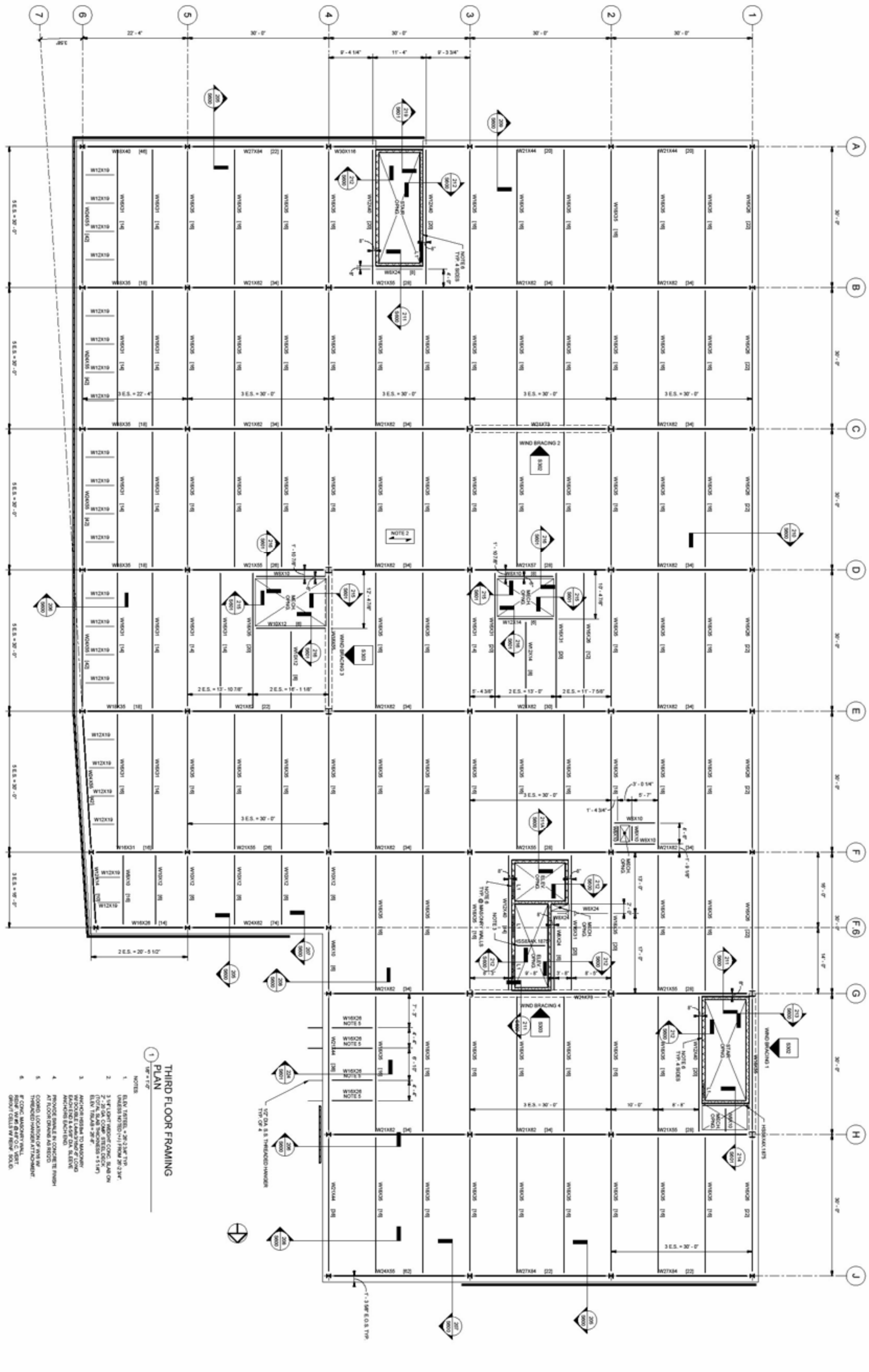
After running both the wind and seismic analysis I was able to conclude that the seismic base shear controlled the overall design. I was also able to conclude, after consulting with the design engineer, that the design value that they used was 338 kips controlled by seismic. I was also able to find out that ASCE 7-05 was the code that was followed which ensures me that my calculations are correct and that the loads I used were fairly close to the ones the engineer used.

CONCLUSION

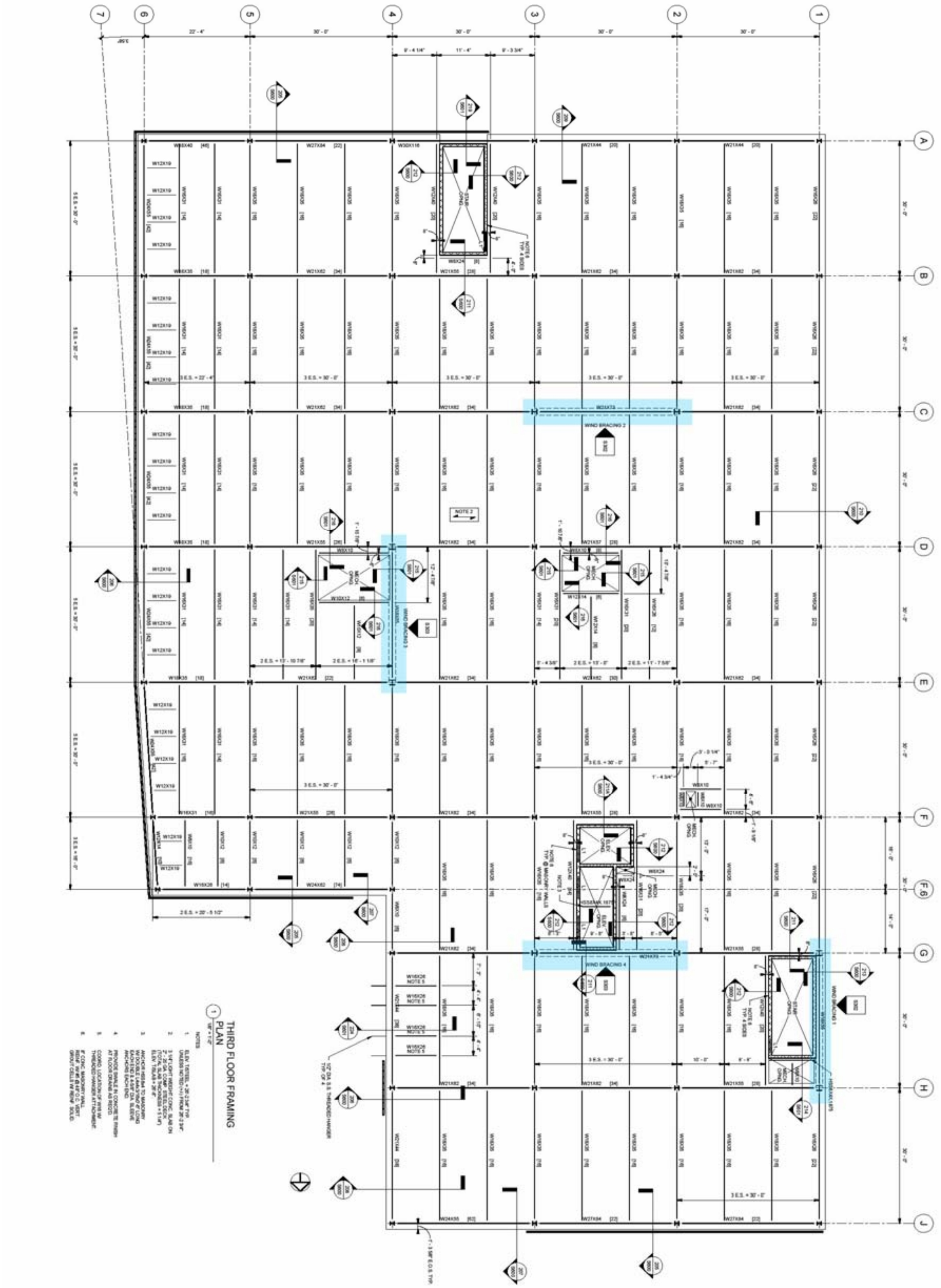
In technical report one; Mountain State Blue Cross Blue Shield Headquarters' existing building conditions were expressed in descriptions of the building's foundation, floor, lateral bracing, and columns. The material strengths used throughout the building were stated. Codes that were used in evaluation of loads were listed. Detailed calculations were taken to determine lateral loads that the building was designed to withstand. Spot checks of the building's floor and column system were completed to compare the final design with the loads that I considered. The majority of the loads I used were fairly conservative. The beam and girder that were used by the engineer in final design, both check in design but the stud value used was off of the values I calculated. The original design was analyzed using ASD and I analyzed them using LRFD. This is a possible reasoning behind the difference in stud numbers. After analyzing the column I found that the size used in the final design worked for the top 3 levels. At the lowest level I found that it needed to be bumped up one size in order to cover the load I calculated. I believe that the difference between my analysis and the one by the engineer may differ based on the conservative loads that I used especially on the roof. In later calculations, I will also look into second order effects and column stiffness which may change my results. A detailed analysis of the buildings lateral loads were calculated in this first report. I concluded that seismic controlled the design. The four lateral braces that carry these loads will be analyzed in a later report to determine the effectiveness of each frame throughout the building.

All design values used were in accordance with the codes referenced. Detailed calculations and notes are available for review in the appendices. Any questions or comments can be aimed at Dominic Manno via email: dam336@psu.edu.

APPENDIX A: BUILDING LAYOUT



Typical Floor Layout



Lateral System Layout

-End of Section-

APPENDIX B: WIND ANALYSIS

WIND DESIGN

ASCE 7-05 METHOD 2

BASIC WIND SPEED $V = 90$ MPH (FIGURE 6-1)

IMPORTANCE FACTOR $I = 1.0$ (TABLE 6-1)

CATEGORY II

EXPOSURE CATEGORY - B (SEC 6.5.6.3)

VELOCITY PRESSURE EXPOSURE COEFFICIENT, K_z

(TABLE 6-3)

FLOOR	ACTUAL HT.	EST. HT.	K_z
2	13'-4"	15'	0.57
3	26'-8"	30'	0.70
4	39'-10"	40'	0.76
ROOF	53'-10"	60'	0.85

TOPOGRAPHIC FACTOR $K_{zt} = 1.0$

FLAT SITE

GUST EFFECT FACTOR, $G = 0.925\left(\frac{(1+1.7g_Q I_z Q)}{1+1.7g_v I_z}\right)$ (EQ 6-4)

$h = 54'$

$c = 0.30$ (TABLE 6-2)

$z_{min} = 30'$ (TABLE 6-2)

$\bar{z} = 0.6h = 32.4'$

$I_z = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{32.4}\right)^{1/6} = 0.3$ (EQ 6-5)

$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}}$ (EQ 6-6)

GUST EFFECT FACTOR CONT.

$$B = 240' \text{ OR } 146'$$

$$L_{\bar{z}} = \ell \left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{32.4}{33}\right)^{1/3.0} = 318.05 \text{ ft}^2$$

$\ell = 320'$ (TAB 6-2)

$\bar{\epsilon} = 1/3.0$ (TAB 6-2)

$$Q_{B=240} = \sqrt{\frac{1}{1+0.63\left(\frac{240+54}{318.05}\right)^{0.63}}} = 0.788$$

$$Q_{B=146} = \sqrt{\frac{1}{1+0.63\left(\frac{146+54}{318.05}\right)^{0.63}}} = 0.825$$

$$G_{N\text{-}S,\ B=240} = 0.925\left(\frac{(1+1.7(3.4)(0.3)(0.788))}{1+1.7(3.4)(0.3)}\right) = 0.866$$

$$G_{E\text{-}W,\ B=146} = 0.925\left(\frac{(1+1.7(3.4)(0.3)(0.825))}{1+1.7(3.4)(0.3)}\right) = 0.889$$

ENCLOSURE CLASSIFICATION = ENCLOSED (SEC 6-2)

INTERNAL PRESSURE COEFFICIENT

$$GC_{pi} = \pm 0.18 \quad \text{(FIGURE 6-5)}$$

EXTERNAL PRESSURE COEFFICIENT

WINDWARD WALL, $C_p = 0.8$

LEEWARD WALL, $C_p \Rightarrow L/B = 146/240 = 0.61$ (B=240), $C_p = -0.5$ (N-S)

$C_p \Rightarrow L/B = 240/146 = 1.64$ (B=146), $C_p = -0.37$ (E-W)

SIDEWALL: $C_p = -0.7$

VELOCITY PRESSURE (EQ 6-15)

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \quad (\text{lb}/\text{ft}^2)$$

SEE SPREADSHEETS

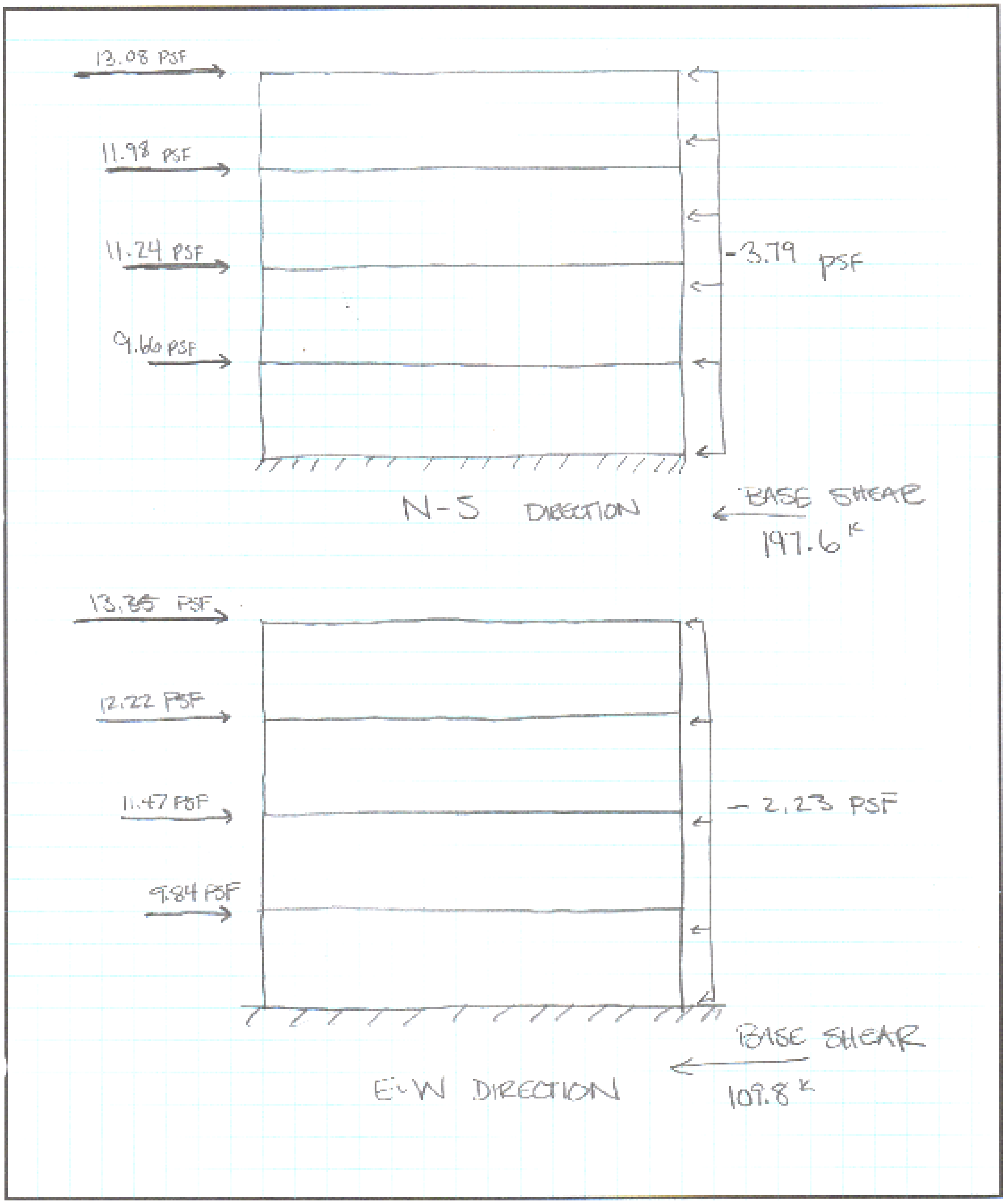
DESIGN WIND PRESSURES, P (EQ 6-17)

$$P = qGC_p - q_i(GC_{pi}) \quad (\text{lb}/\text{ft}^2)$$

SEE SPREADSHEETS

Wind Pressures (psf)							
Floor Heights	Level	Kz	qz	N-S (windward)	N-S (leeward)	E-W (windward)	E-W (leeward)
13.33	2	0.57	10.05	9.66	-3.79	9.84	-2.23
26.67	3	0.70	12.34	11.24	-3.79	11.47	-2.23
39.83	4	0.76	13.40	11.98	-3.79	12.22	-2.23
53.83	Roof	0.85	14.98	13.08	-3.79	13.35	-2.23

Wind Design						
	Load (kips)		Shear (kips)		Moment (ft-k)	
Level	N/S	E/W	N/S	E/W	N/S	E/W
Roof	56.7	31.8	0.0	0.0	2655.26	1489.19
4	49.8	27.8	56.7	31.8	1655.85	924.35
3	48.1	26.7	106.5	59.6	961.52	533.73
2	43.0	23.5	154.6	86.3	286.38	156.51
Total	197.6	109.8	197.6	109.8	5559.01	3103.79



Distribution of Forces from each Direction

-End of Section-

APPENDIX C: SEISMIC ANALYSIS

SEISMIC DESIGN ASCE 7-05

LATITUDE - 39.35 LONGITUDE 81.43

OCCUPANCY CATEGORY II

IMPORTANCE FACTOR (I) = 1.0

USGS:

$S_S = 0.137$

$S_1 = 0.057$

SOIL SITE CLASS D

SITE COEFFICIENTS

$F_a = 1.6$ (TABLE 11.4-1)

$F_V = 2.4$

$S_{MS} = F_a S_s = 1.6(0.137) = 0.219$ (EQ 11.4-1)

$S_{M1} = F_V S_1 = 2.4(0.057) = 0.137$ (EQ 11.4-2)

DESIGN ACCELERATIONS

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.219) = 0.146$ (EQ 11.4-3)

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.137) = 0.091$ (EQ 11.4-4)

SEISMIC DESIGN CATEGORY $\Rightarrow$ B

$S_{DS} = 0.15 \longrightarrow$ A

$S_{D1} = 0.09 \longrightarrow$ B CONTROLS

RESPONSE MODIFICATION FACTOR

$R = 3.25$ (TABLE 12.2-1)

$\Omega = 2 \quad C_d = 3.25$

$$C_s = \frac{S_{DS}}{(R/I)} \geq 0.01$$

$$\frac{S_{D1}}{T\left(\frac{R}{I}\right)} \quad T \leq T_L \quad \geq 0.01$$

$$\frac{S_{D1} T_L}{T^2\left(\frac{R}{I}\right)} \quad T > T_L \quad \geq 0.01$$

(EQ 12.8-7) $\cdot\ T_a = C_t h_n^x = 0.03(53.83)^{0.75} = 0.60$

(TABLE 12.8-2) $C_t = 0.03$

$x = 0.75$ $\qquad T_L = 12 \Rightarrow$ (FIG 22-15)

$C_u = 1.4$ (TABLE 12.8-1)

$T = C_u T_a \Rightarrow$ (SEC 12.8.2)

$T = 0.84$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \Longrightarrow \frac{0.09}{(0.84)\left(\frac{3.25}{1}\right)} = 0.03 \quad \text{OK}$$

$$C_s = 0.03$$

$$V = C_s W$$

WEIGHT OF BUILDING

TYP FLOOR AREA = 31,322 ft^2

TYP FLOOR LOADS	
PARTITIONS	20 PSF
FIN & MISC	5 PSF
MEP	10 PSF
5.25" SLAB/DECK	42 PSF
BEAMS / COL	15 PSF
	92 PSF

ROOF LOADS	
MEP	10 PSF
ROOF MAT	20 PSF
SLAB / DECK	40 PSF
MISC	5 PSF
BEAM / JOISTS	10 PSF
	85 PSF

ROOF : $31,322\ ft^2 (85\ psf) = 2662^k$

FLOORS : $3(31,322\ ft^2)(92\ psf) = 8645^k$

$WT = 11,307^k$

$V_s = 0.03(11,307)$

$= 339^k$

$k = 0.75 + 0.5(0.84) = 1.17$

Base Shear and Overturning Moment Distribution							
Level	**h_x**	**Floor Load**	**$h_x^k w_x$**	**C_{vx}**	**$F_x = C_{vx} V$**	**V_x(k)**	**M_x (ft-k)**
Roof	53.83	2662	282167.2	0.41	138.42	0	7451.10
4	39.83	2883	214827.9	0.31	105.39	138.42	4197.50
3	26.67	2883	134366.7	0.19	65.91	243.80	1757.94
2	13.33	2883	59689.22	0.09	29.28	309.72	390.32
Totals		11311	691051	1.00	339.00	339.00	13796.86

-End of Section-

APPENDIX D: SNOW ANALYSIS

SNOW LOAD

ASCE-7 05

$p_f = 0.7\,C_e C_t I p_g$

$C_e = 1.0$ (TABLE 7-2)

$C_t = 1.0$ (TABLE 7-3)

$I = 1.0$ (TABLE 7-4)

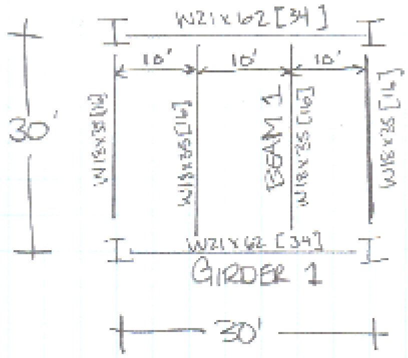
$p_g = 20$ PSF (FIG 7-1)

$p_f = 0.7(1.0)(1.0)(1.0)(20) = 14$ PSF

-End of Section-

APPENDIX E: FLOOR AND COLUMN ANALYSIS

SPOT CHECKS



LOADS

LIVE LOADS: ASSUME 50 PSF FOR OFFICE SPACE

DEAD LOADS:

PARTITIONS :	20 PSF
FIN & MISC. :	5 PSF
M.E.P. :	10 PSF
SLAB & DECK :	42 PSF
SW BMS :	15 PSF
TOTAL :	92 PSF

CHECK BEAMS

LRFD:

$1.2D + 1.6L$

$1.4D$

$$W_u = 1.2(92) + 1.6(50) = 190 \text{ PSF}$$

TRIB WIDTH = 10'

$w_u = 1900$ PLF

$$M_u = \frac{w_u l^2}{8} = \frac{1900(30')^2}{8} = 214^{\prime K}$$

b_{eff} = min | SPACING: 120"
| SPAN/4 = 90" CONTROLS

$$\Delta_{CONST} = \frac{5\, w_{cons}\, \ell^4}{384\, EI}$$

$$w_{CONST} = 115\left(\frac{5.25'}{12}\right)(10') = 500 \text{ PLF}$$

$$\Delta_{CONST} = \ell/360 = 1.0$$

$$I_{min} = \frac{5\, w_{cons}\, \ell^4}{384\, E\, \Delta_{cons}}$$

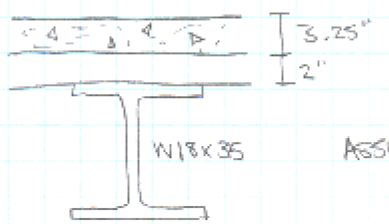
$$I_{min} = \frac{5(0.5 \text{ KLF})(30)^4(1728)}{(384)(29000)(1)} = 314 \text{ in}^4$$

ASSUME $W_{cons} = 500$ PLF $\quad W_L = 20$ PSF $\Rightarrow W_u = 0.7$ KLF

$$M_{u\,min} = \frac{(0.7 \text{ KLF})(30)^2}{8} = 78.75 \text{ ft-k}$$

NON COMP NEEDS TO MEET THIS DURING
CONST. PHASE

W16x45 MIN. BEAM FOR CONST.



$T = A_s f_y = 10.3(50 \text{ ksi})$

$= 515^{k} = \Sigma Q_n$

ASSUME $T = C$

$$a = \frac{\Sigma Q_n}{0.85(f'_c)(b_{eff})}$$

$$a = \frac{515}{0.85(4)(90)} = 1.68 \approx 2$$

$$Y_2 = 5.25 - \frac{2}{2} = 4.25''$$

@ $Y_2 = 4.25''$ TFL $\phi M_n = 506'^k$

$506'^k > 214'^k$ OK

$I_b = 1395$ TABLE 3-20

$$\Delta_L = \frac{l}{360} = 1.0 = \frac{5\, w_u\, l^4}{384 E I_b}$$

$$\frac{5(1.9\,\text{k/ft})(30)^4(1728)}{384(29000)(1395)} = 0.86 < 1.0 \quad \text{OK}$$

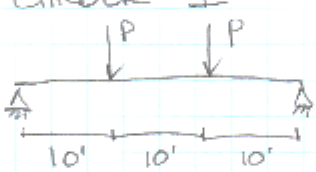
NUMBER OF STUDS: 25 EACH SIDE

THE DESIGN OF 16 STUDS FAILS MEANING T+C. MY VALUES MAY BE OFF, BUT THIS WAS ORIGINALLY DESIGNED IN ASD, WHICH COULD RESULT IN THE DIFFENCE IN STUD NUMBERS.

Atlantic Engineering Services
650 Smithfield Street • Suite 1200
Pittsburgh • Pennsylvania 15222
AES

JOB
SHEET NO. ____ OF ____
CALCULATED BY ____ DATE ____
CHECKED BY ____ DATE ____
SCALE ____

GIRDER 1



LOAD: 1.9 KLF

$P = \frac{1.9(30)}{2} = 28.5^K$

$M_{max} = Pa = 28.5'(10) = 285'^K$

STEEL ONLY TRY: $W21\times62 \Rightarrow 540'^K$

$B_{EFF} = \frac{30(12)}{4} = 90''$

ASSUME $a = 2''$ $Y_2 = 4.25$

PNA #3 $\Sigma Q_n = 659$ $\phi M_n = 925$

CHECK $a = \frac{659}{0.85(4)(90)} = 2.15 \approx 2$

$Y_2 = 5.25 - \frac{2}{2} = 4.25$ OK

$Q_n = \frac{659}{21.2} = 31$ STUDS EACH SIDE

CONST. DEAD LOAD Δ

$\Delta = \frac{PL^3}{28EI} = \frac{15(30)^3(144)}{28(29000)(1330)} = 0.05 < \frac{l}{360} = 1$ OK

$w = 1.6(50)(30') = 2.4$ KLF

$P = 2.4(30) = 72^K$

Δ_{LL} $I_b = 2995$ (TABLE 3-20)

$\Delta = \frac{PL^3}{28EI_b} = \frac{(144)(72)(30)^3}{28(29000)(2995)} = 0.12 < \frac{l}{360} = 1$ OK

W21X62 [31] CHECKS ✓

COLUMN SPOT CHECK

ROOF COLUMN

$A_I = 3600\ ft^2 \quad A_T = 900\ ft^2 \quad KL = 14'$

NO LIVE REDUCTION ASSUME SNOW LOADS

LOAD: $1.2(85\ psf) + 1.6(14\ psf) = 124$

$P_u = 124(900) = 112^k$

W10x49 $\phi P_n = 471^k$ CHECKS ✓

4th FLOOR COLUMN

$A_I = 3600\ ft^2 \quad A_T = 900\ ft^2 \quad KL = 13.17'$

$L = L_o\left(.25 + 15/\sqrt{3600}\right) = 0.5 > 0.4$ OK

LOAD = $1.2(92\ psf) + 1.6(0.5(50)) = 150$ PSF

$P_u = 135^k + 112^k = 247$

W10x49 $\phi P_n = 489^k$ CHECKS ✓

3RD FLOOR COLUMN

$A_I = 3600$ $A_T = 900$ $KL = 13.33'$

$L_R = 0.5 L_0$

LOAD: SAME AS 4TH. 150 PSF

$135 + 247 = 382^K$

W10x49 $\phi P_n = 487^K$ CHECKS ✓

2nd FLOOR COLUMN

$A_I = 3600$ $A_T = 900$ $KL = 13.33$

LOAD: SAME AS ABOVE

$P_u = 135 + 382 = 517^K$

W10x49 $\phi P_n = 487^K$ NG

W10x54 $\phi P_n = 539^K$ OK

THIS LEADS ME TO BELIEVE SOME OF OUR LOADS MAY BE DIFFERENT OR DIFFERENCE FROM ASD TO LRFD MAY HAVE IMPACT.

-End of Section-