MOUNTAIN STATE
BLUE CROSS BLUE SHIELD
HEADQUARTERS

PARKERSBURG, WEST VIRGINIA

DOMINIC MANNO

STRUCTURAL OPTION

FACULTY CONSULTANT: DR. ANDRES LEPAGE

Technical Report 3

11-21-08

TABLE OF CONTENTS

EXECUTIVE SUMMARY

In this third technical report, detailed lateral analysis for Mountain State Blue Cross Blue Shield Headquarters was investigated using RAM Structural System and hand calculations. The building utilizes a composite steel system to handle all gravity loads and has 4 steel braces, 2 in each direction, which handle the lateral loads. This report takes a detailed look into the seismic and wind forces developed by hand calculations and RAM Structural System. Sap 2000 and hand calculations were done and used to compare to the complicated analysis done by RAM Structural System.

The results confirm that seismic controls the design of the lateral members in both directions. The computed story shears were similar to the hand calculations with the computer analysis being more precise. Strength and serviceability checks were performed using these loads to ensure that the members were well within code limits. The building was acceptable for story drifts and total drift. Spot checks led me to conclude that drift controlled the design of columns and braces in the lateral members. It has enabled me to conclude, that by using a different bracing layout these members may be able to be reduced achieving a more economical design. This is something I may look into for my proposal.

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. It was designed by Burt Hill Architects. Its main purpose for being built was to expand to include an extra 170 employees that are to be hired this year. G.A. Brown was hired as the contractor and began construction in March of 2008 and is expected to be completed by April of 2009. 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 building's 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.

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

(ASCE7-05) Minimum design loads for Buildings and other Structures
American Society of Civil Engineers

Steel Deck Institute, Design Manual 2001

DEFLECTION CRITERIA per IBC 2006

Δ_{WIND} = H/400 Allowable Building Drift

$\Delta_{SEISMIC} = 0.025h_{SX}$ Allowable Story Drift

LOAD CASES AND COMBINATIONS per IBC 2006

The following are the load cases considered for this analysis per IBC 2006, Section 1605:

1.4(Dead)
1.2(Dead) + 1.6(Live) + 0.5(Roof Live)
1.2(Dead) + 1.6(Roof Live) + (1.0 Live or 0.8 Wind)
1.2(Dead) + 1.6(Wind) + 1.0(Live) + 0.5(Roof Live)
1.2(Dead) + 1.0(Seismic) + 1.0(Live)
0.9(Dead) + 1.6(Wind)
0.9(Dead) + 1.0(Seismic)

Total Combinations generated by the RAM computer analysis were 313. These combinations were applied at different eccentricities from various directions.

CODE / REFERENCES USED IN ORIGINAL DESIGN

2003 International Building Code

(ACI 318-05) 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

(ASCE7-05) 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

EXISTING STRUCTURAL SYSTEM

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 of No. 7 to No. 8 vertical reinforced bars, and have ties that are No. 3 reinforcing. 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. 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 a 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. Below is the layout of the lateral braces and elevations (Figures 1 through 7).

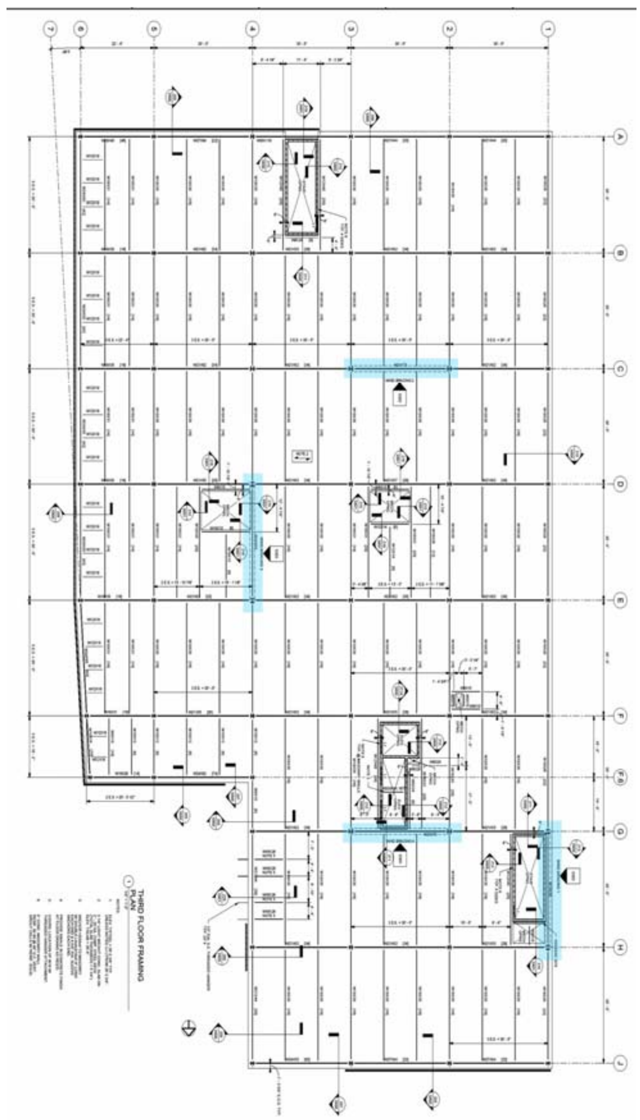


Figure 1: Lateral System Layout

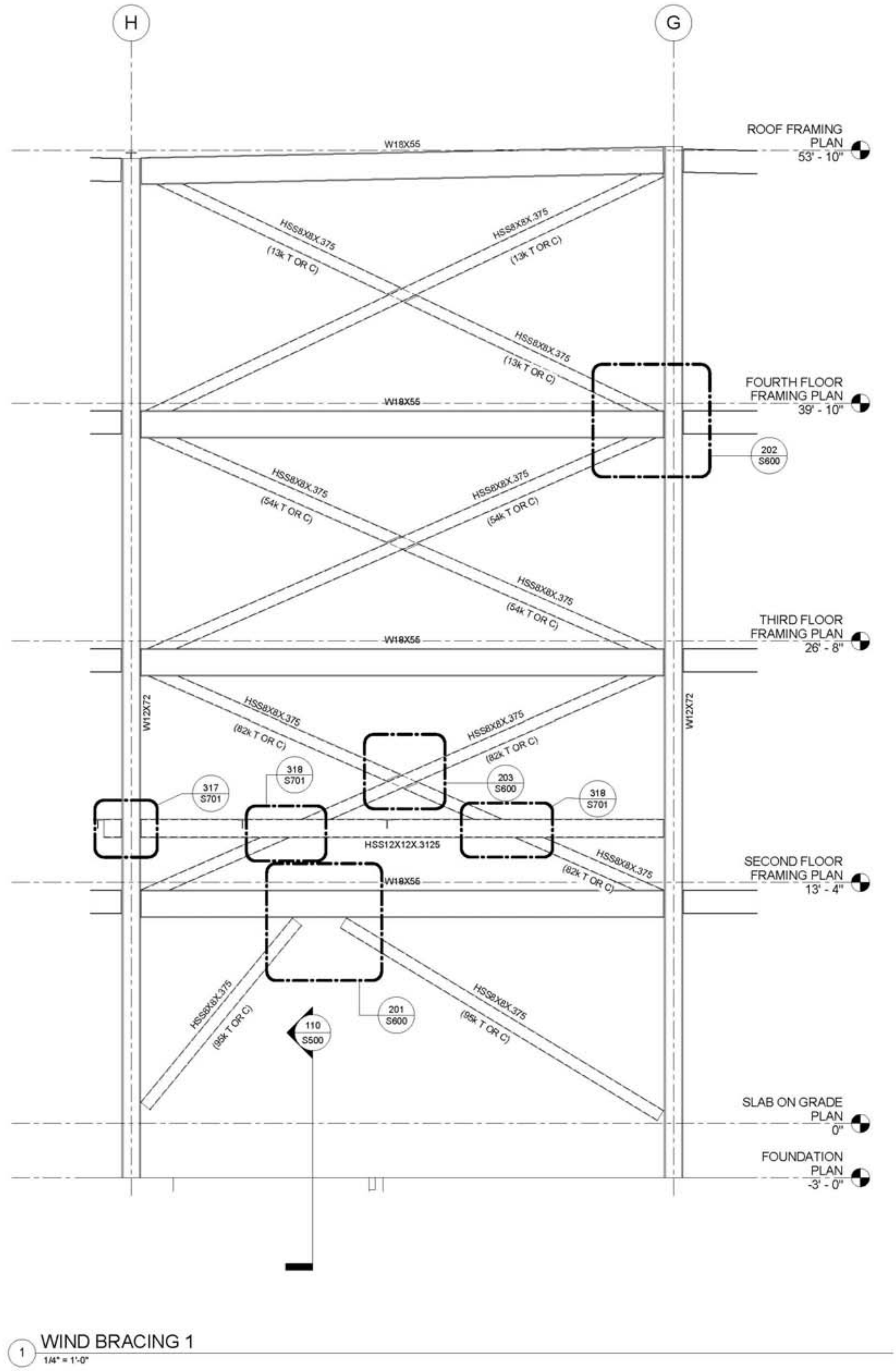


Figure 2: Lateral Brace 1 Elevation

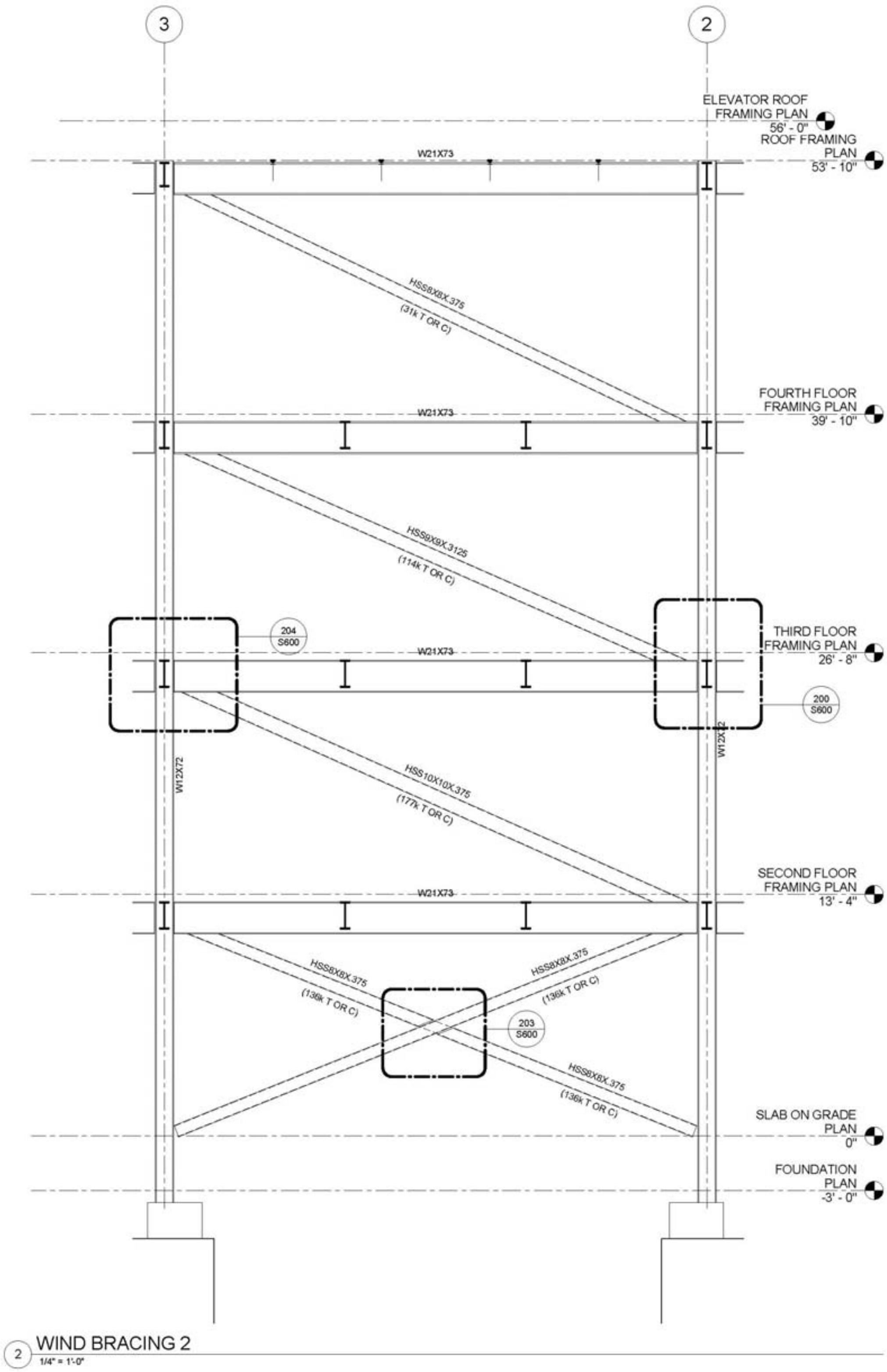


Figure 3: Lateral Brace 2 Elevation

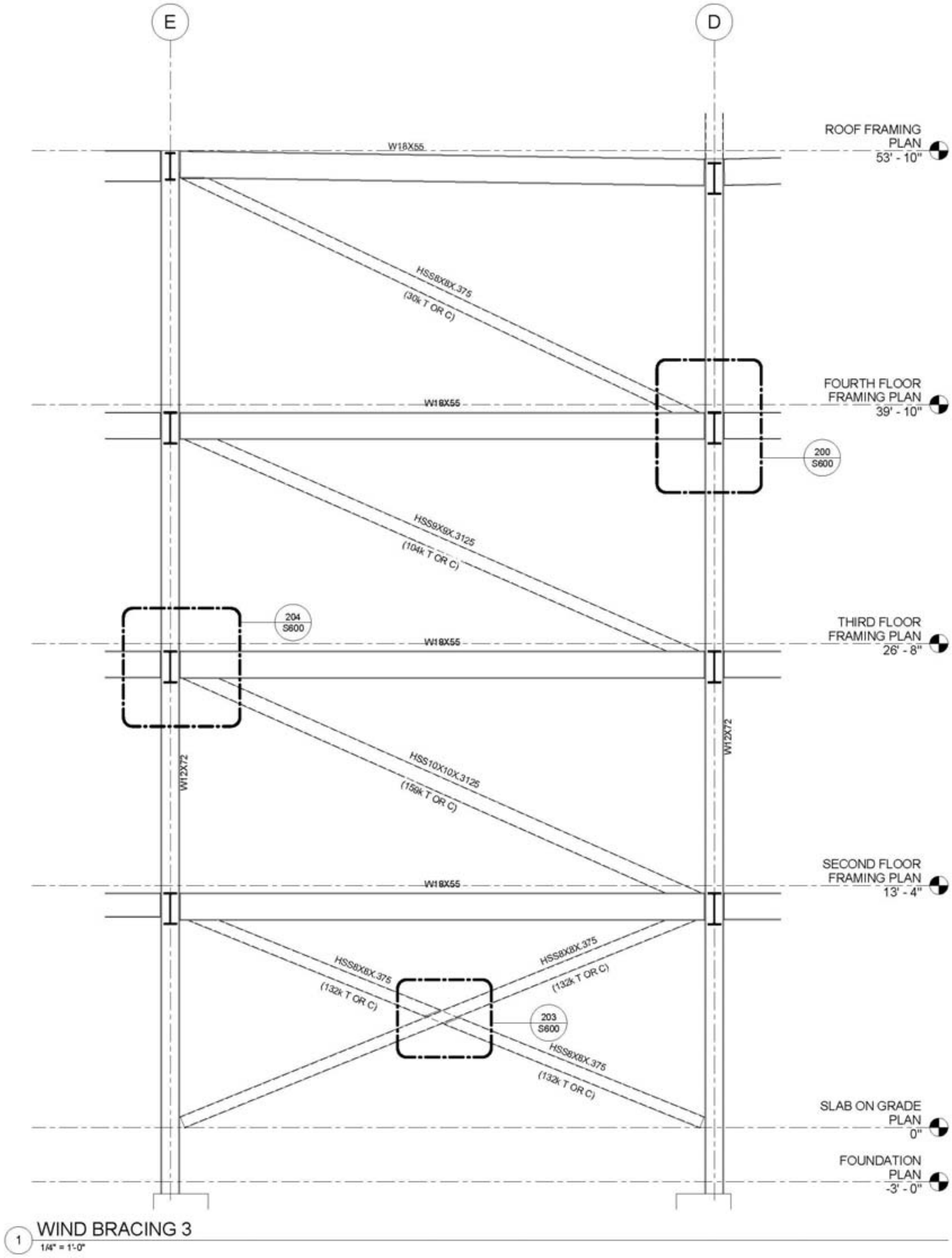


Figure 4: Lateral Brace 3 Elevation

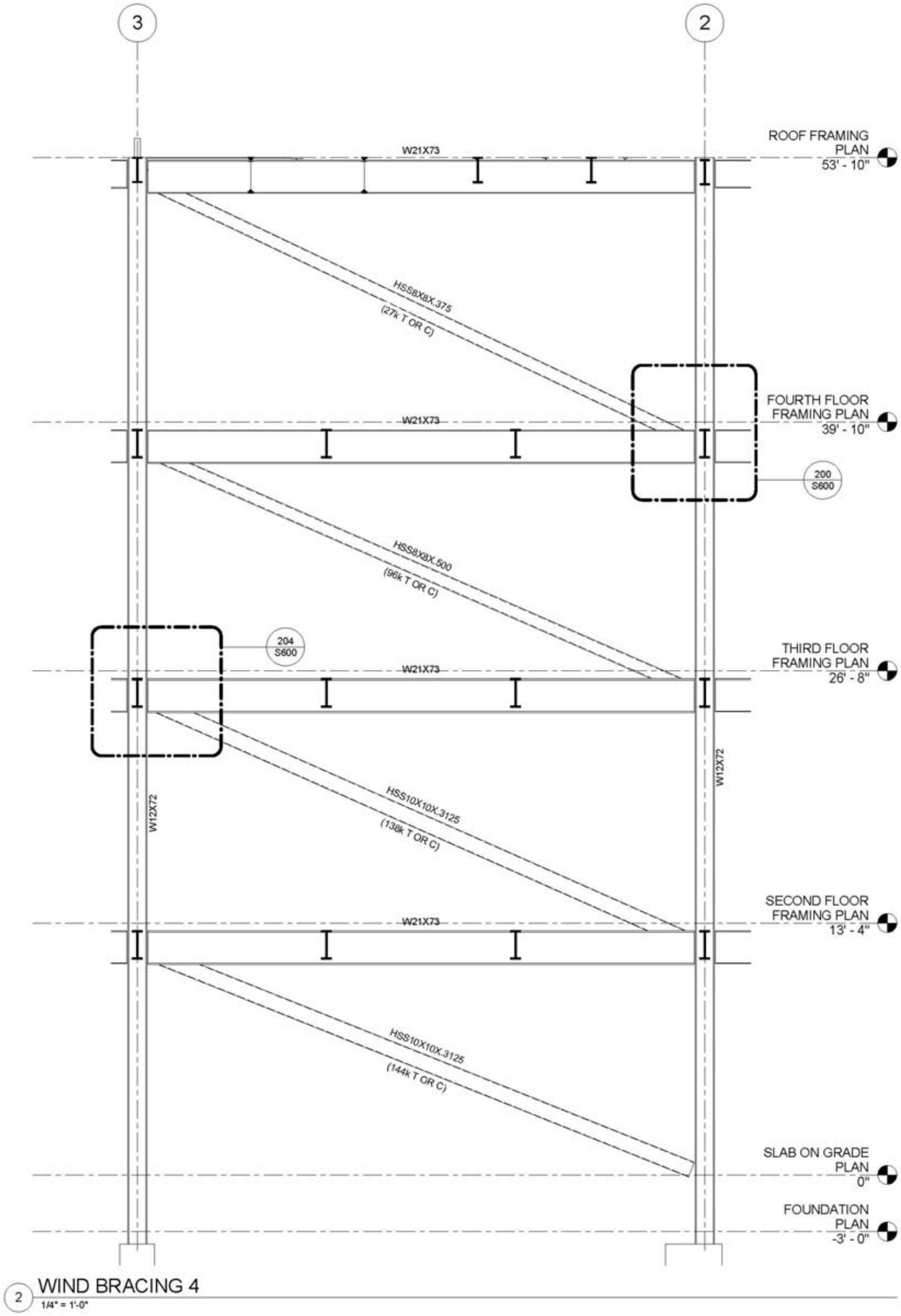


Figure 5: Lateral Brace 4 Elevation

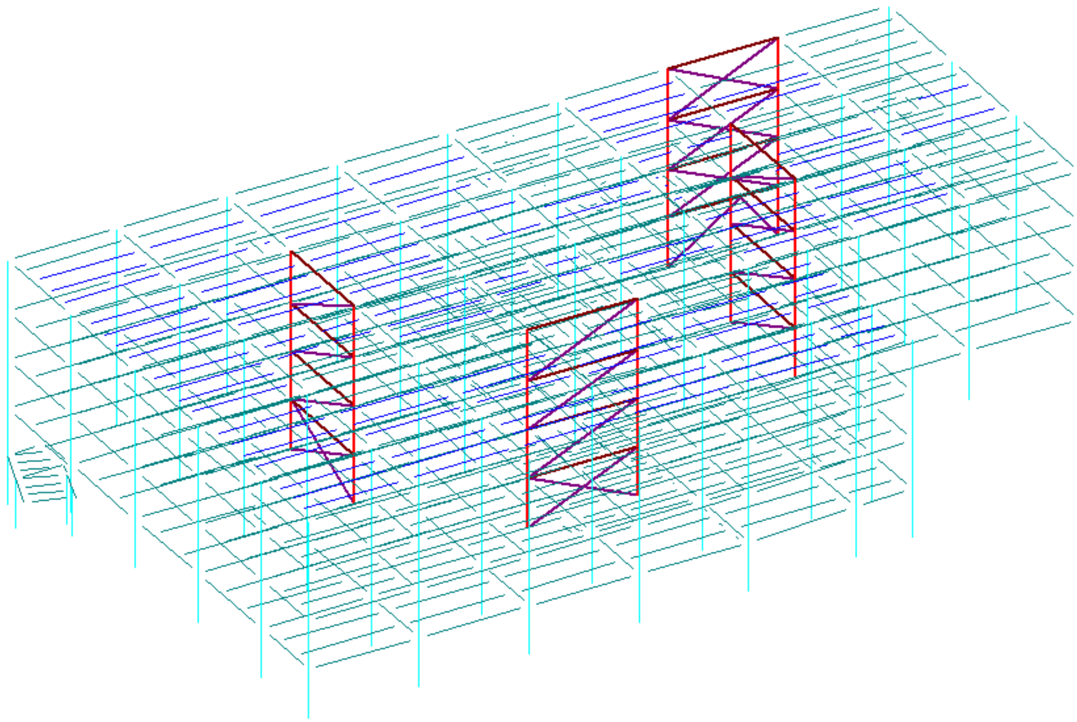


Figure 6: 3-D Layout of Structural System

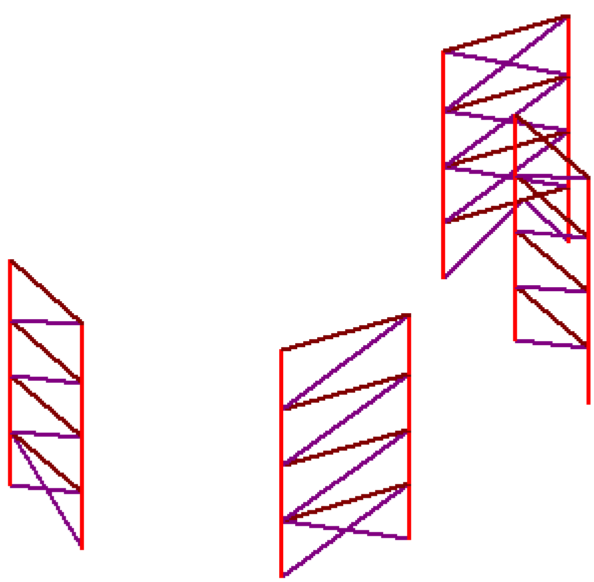


Figure 7: 3-D Layout of Lateral System

LATERAL LOADS

In technical report one, loads which I calculated for Mountain State Blue Cross Blue Shield were relatively high due to the fact that I was trying to be conservative. I decided to use the loads that RAM produced to compare to the originals by the design engineer for this report. This resulted in lateral loads that are far more accurate than ones previously calculated in technical report 1 by hand.

Wind Design Criteria

Wind loads were analyzed using ASCE7 -05. These assumptions were inputted into RAM to determine wind loads.

Basic Wind Speed ***V***…………………………………………… 90 mph
Exposure Category……………………………………………… B
Importance Factor……………………………………………… 1.0
Building Category……………………………………………….. I
Internal Pressure Coefficient ***GCpi***……………………… +/- 0.18

Resulting story shears in the x and y direction due to wind and overturning moments for each direction are shown below in figures 8 and 9.

RAM - Wind Y, (N-S)		Force	Story Shear	Moment
54	Roof	26.55	0	1433.7
40	4	51.4	26.55	2056
26.67	3	48	77.95	1280.16
13.33	2	45.31	125.95	603.9823
	Base	171.26	171.26	5373.842

Figure 8: Wind Story Shears N/S Direction

RAM - Wind X, (E-W)		Force	Story Shear	Moment
54	Roof	14.4	0	777.6
40	4	27.81	14.4	1112.4
26.67	3	25.63	42.21	683.5521
13.33	2	22.16	67.84	295.3928
	Base	90	90	2868.945

Figure 9: Wind Story Shears E/W Direction

Seismic Criteria

These were the assumptions made in finding seismic forces for MSBCBS. They were also calculated using ASCE7-05.

Seismic Occupancy Category……………………………………I
Importance Factor………………………………………… 1.0
Spectral Response Accelerations
Ss………………………………………………………….…. 0.141
S1……………………………... 0.058
T_L ……………………………………………………………..12
Site Class……………..……………………………...…… D
R………………………………………………………….3.25

Resulting story shears in the x and y direction due to seismic and overturning moments for each direction are shown below in figures 10 and 11.

RAM - Seismic Y, (N-S)		Force	Story Shear	Moment
54	Roof	61.35	0	3312.9
40	4	171.26	61.35	6850.4
26.67	3	107.39	232.61	2864.0913
13.33	2	42.32	340	564.1256
	Base	382.32	382.32	13591.5169

Figure 10: Wind Story Shears N/S Direction

RAM - Seismic X, (E-W)		Force	Story Shear	Moment
54	5	61.29	0	3309.66
40	4	171.05	61.29	6842
26.67	3	107.65	232.34	2871.0255
13.33	2	40.66	339.99	541.9978
	Base	380.65	380.65	13564.6833

Figure11: Wind Story Shears E/W Direction

It was concluded that after investigating the Ram model under these loads that seismic controlled the design of the lateral system in both directions even after the 1.6 load combination coefficient was taken into account for wind.

LATERAL LOAD DISTRIBUTION AND ANALYSIS

For this technical report when investigating MSBCBS's lateral system, two computer programs were used. SAP 2000 was used to determine each frame's relative stiffness. RAM Structural Systems was used to perform a detailed lateral analysis. The building is a composite steel deck floor system and for computer modeling it was treated as a rigid diaphragm. This means that the building distributes the lateral loads to each of the braces depending on the relative stiffness of the frame.

SAP 2000

While preparing this model I assumed that the connections between the columns and foundations were a pinned connection not fixed. I placed the two frames that resist load in each direction in the same 2d plane. I then proceeded to link the nodes at each level to a diaphragm. The deflections for each floor would now be identical between the two frames in each direction. A unit load of 100 kips was added to the top level of a frame and analyzed to determine shear in the columns, axial force in the braces, and deflection at each floor. For the purpose of determining torsion effects on the building, centers of rigidity and mass were taken from the RAM. Below is a summary of the calculations to determine each frame's relative stiffness, actual rigidity, direct shear, and torsion shear (Figures 12 to 22).

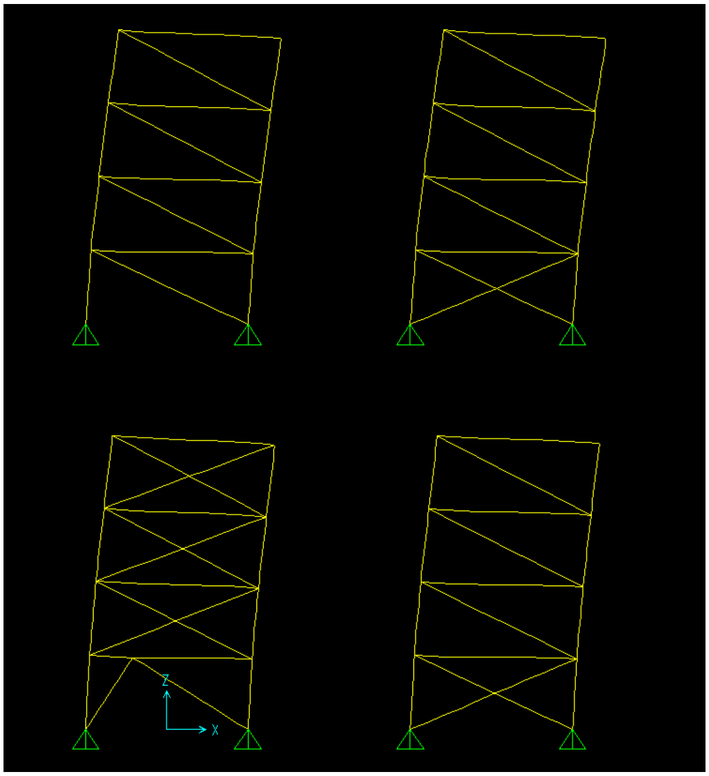


Figure 12: Deflected Shape of Braces in SAP 2000

E/W Direction	Frame 1					
Level	**Shear Col Left**	**Shear Col Right**	**Horizontal Component of Brace**	**Sum**	**Floor Deflection**	**Actual Rigidity**
Roof	0.71	0.72	58.85	60.28	0.2971	336.59
4th	0.63	0.56	61.39	62.58	0.2051	487.57
3rd	0.39	0.73	56.64	57.76	0.1175	851.06
2nd	-0.14	0.09	50.73	50.68	0.0463	2159.83
			Total	231.3		
		Relative Stiffness	**% Frame Takes**	0.58		

Figure 13: Frame 1 Relative Stiffness

E/W Direction	Frame 3					
Level	**Shear Col Left**	**Shear Col Right**	**Horizontal Component of Brace**	**Sum**	**Floor Deflection**	**Actual Rigidity**
Roof	1.27	1.27	37.27	39.81	0.2971	336.59
4th	1.04	1.04	35.42	37.5	0.2051	487.57
3rd	0.98	0.98	40.34	42.3	0.1175	851.06
2nd	0.09	0.09	49.5	49.68	0.0463	2159.83
			Total	169.29		
		Relative Stiffness	**% Frame Takes**	0.42		

Figure 14: Frame 3 Relative Stiffness

N/S Direction	Frame 2					
Level	**Shear Col Left**	**Shear Col Right**	**Horizontal Component of Brace**	**Sum**	**Floor Deflection**	**Actual Rigidity**
Roof	1.64	1.64	46.99	50.27	0.3686	271.30
4th	1.12	1.12	40.76	43	0.2534	394.63
3rd	1.29	1.29	51.3	53.88	0.1502	665.78
2nd	0.14	0.14	65.08	65.36	0.0604	1655.63
			Total	212.51		
		Relative Stiffness	**% Frame Takes**	0.53		

Figure 15: Frame 2 Relative Stiffness

N/S Direction	Frame 4					
Level	Shear Col Left	Shear Col Right	Horizontal Component of Brace	Sum	Floor Deflection	Actual Rigidity
Roof	1.63	1.63	46.54	49.8	0.3686	271.30
4th	1.11	1.11	54.85	57.07	0.2534	394.63
3rd	1.3	1.3	43.6	46.2	0.1502	665.78
2nd	0.14	0.14	34.45	34.73	0.0604	1655.63
			Total	187.8		
		Relative Stiffness	% Frame Takes	0.47		

Figure 16: Frame 4 Relative Stiffness

RAM	Center of Rigidity	
LEVEL	x	y
Roof	118.37	106.79
4	117.1	107.04
3	109.96	104.35
2	101.83	93.97

Figure 17: Center of Rigidity

RAM	Center of Mass	
LEVEL	x	y
Roof	111.41	84.29
4	108.98	74.54
3	108.98	74.53
2	107.79	74.23

Figure 18: Center of Mass

COR Y				
Frame 2 - C	$R*C^2$	Frame 4 -C	$R*C^2$	J
58.37	5394359.801	61.63	5332941.841	10727301.64
57.1	5162175.201	62.9	5554996.669	10717171.87
49.96	3951894.872	70.04	6887708.945	10839603.82
41.83	2770360.286	78.17	8579514.961	11349875.25
COR X				
Frame 1 - C	$R*C^2$	Frame 3 - C	$R*C^2$	J
36.21	2916460.331	54.46	4777223.884	7693684.215
35.96	2876327.875	54.71	4821184.487	7697512.362
38.65	3322752.708	52.02	4358740.528	7681493.236
49.03	5347155.458	41.64	2792812.39	8139967.849

Figure 20: J -Torsional Rigidity

Direct Shear	X Direction	
V	Frame 1	Frame 3
61.29	35.5482	25.7418
171.05	99.209	71.841
107.65	62.437	45.213
40.66	23.5828	17.0772
Torsional Shear	X Direction	
V	Frame 1	Frame 3
61.29	4.600467522	5.010399762
171.05	12.74414037	14.04038041
107.65	8.638454691	8.419343064
40.66	3.905932716	2.402121012
Total Shear	380.666751	

Figure 21: E/W Direct, Torsion, and Total Shears

Direct Shear	Y Direction	
V	Frame 2	Frame 4
61.35	32.5155	28.8345
171.26	90.7678	80.4922
107.39	56.9167	50.4733
42.32	22.4296	19.8904
Torsional Shear	Y Direction	
V	Frame 2	Frame 4
61.35	6.36885466	5.963288228
171.26	17.40842268	17.0057562
107.39	9.443227084	11.73994681
42.32	3.210305121	5.320109208
Total Shear	382.3282909	

Figure 22: N/S Direct, Torsion, and Total Shears

RAM STRUCTURAL SYSTEMS

Ram Structural Systems was used to perform a detailed lateral analysis of Mountain State Blue Cross Blue Shield. Accurate centers of mass and centers of rigidity were used in the above calculations for torsion and direct shear. Story shears were also taken from Ram. The wind loads were less than the original values that I calculated in technical report one because the loads I assumed were extremely conservative. The seismic loads calculated by Ram were above hand calculated. The loads analyzed by Ram are far more accurate because of the calculation of the actual weight of the building and therefore were used in my above calculations. The major difference in my hand calculations and the Ram model loads was at the roof level. This is mainly because the Ram model makes a more precise distribution of the loads. The Ram model proved after running all load combinations for LRFD that seismic controlled the design of lateral members in both directions of the building. Drift values were taken from Ram and checked against $\Delta_{SEISMIC} = 0.025h_{SX}$ at each level for seismic for the controlling load case in both directions. Below you can see the comparison in Figures 23 and 24.

Controlling Seismic Drift Y Direction									
Story	Story Ht.(ft.)	Story Drift (in.)	Allowable Story Drift (in.) $\Delta_{SEISMIC} = 0.025H_{sx}$			Total Drift (in.)	Allowable Total Drift (in.) $\Delta_{SEISMIC} = 0.025H_{sx}$		
Roof	53.33	0.135	<	0.333	Acceptable	0.9319	<	1.33	Acceptable
4	40	0.2225	<	0.333	Acceptable	0.7969	<	1	Acceptable
3	26.66	0.299	<	0.333	Acceptable	0.5744	<	0.667	Acceptable
2	13.33	0.2754	<	0.333	Acceptable	0.2754	<	0.333	Acceptable

Figure 23: Drift Calculations N/S Direction

Controlling Seismic Drift X Direction									
Story	Story Ht.(ft.)	Story Drift(in.)	Allowable Story Drift (in.) $\Delta_{SEISMIC} = 0.025H_{sx}$			Total Drift (in.)	Allowable Total Drift (in.) $\Delta_{SEISMIC} = 0.025H_{sx}$		
Roof	53.33	0.109	<	0.333	Acceptable	0.686	<	1.33	Acceptable
4	40	0.188	<	0.333	Acceptable	0.577	<	1	Acceptable
3	26.66	0.221	<	0.333	Acceptable	0.389	<	0.667	Acceptable
2	13.33	0.168	<	0.333	Acceptable	0.168	<	0.333	Acceptable

Figure 24: Drift Calculations E/W Direction

As you can see in the charts, the drifts at each level and overall total drift for each direction is acceptable. With the building only being 4 stories high this is not a surprise. After looking at the member code check in Ram I was able to see that the members in each brace were designed well, with most members being well less than 1 when looking at the interaction equation produced in Ram. Hand calculations were done to spot check a brace, column, and beam to ensure that Ram was correct. The same result was proven which can be seen in Appendix B. These calculations also proved that most of the bracing members and columns were designed for drift rather than strength. The interaction equations resulted in 82% for the brace and 64% for the columns. These numbers are more than acceptable for strength. The beams were only 12% meaning that drift most likely did not control their design. With a different brace configuration in these braces the column sizes and braces may be able to be reduced and still keep the building within acceptable limits for drift.

Overturning Check

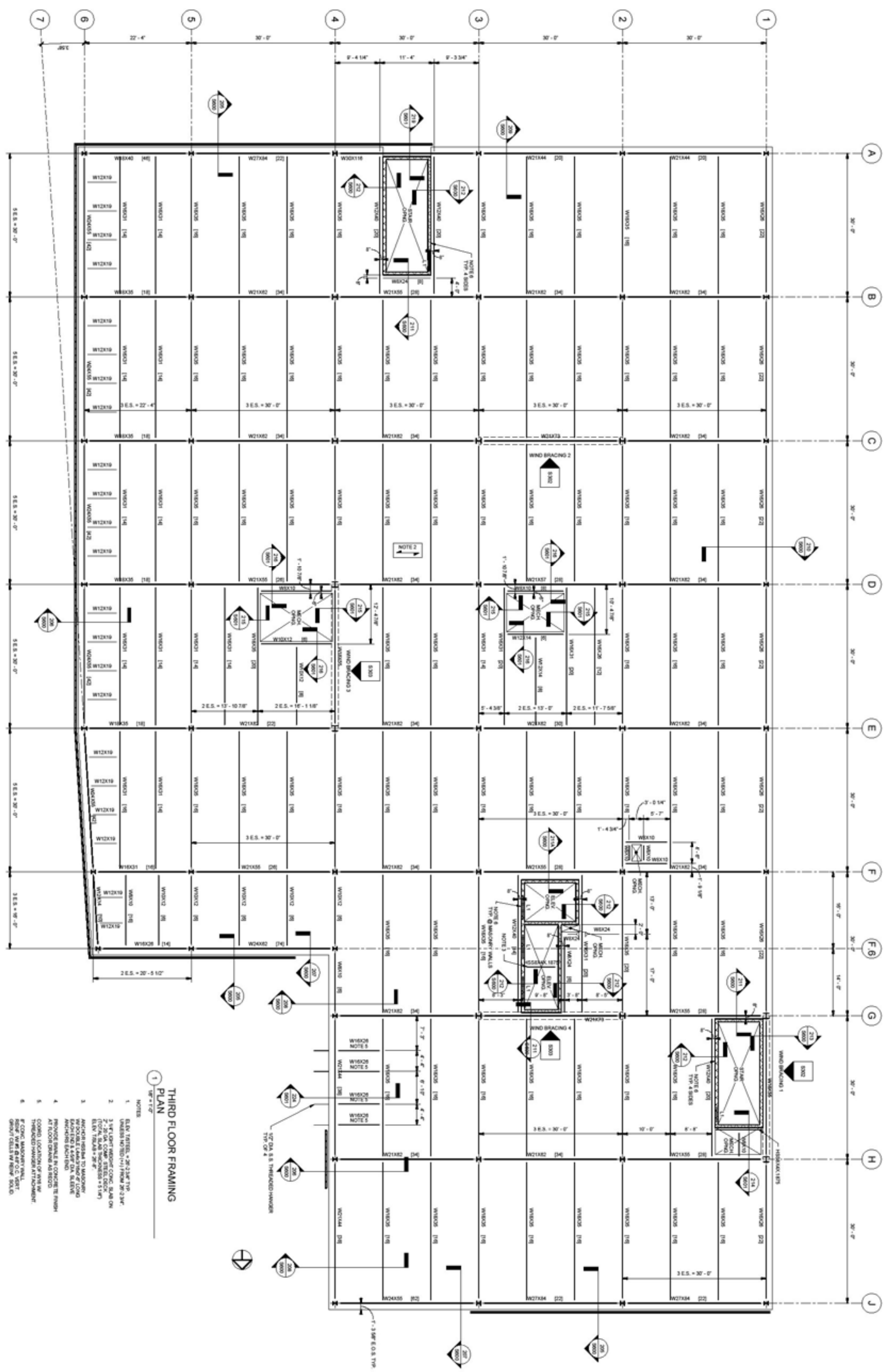
The overturning moment caused by the controlling seismic load case was approximately 13,500 ft-kips. The foundation system that handles this is a drilled caisson system with depths reaching 59 to 74 feet. The mass of the building and the depths of these caissons will handle these uplift forces by inspection. Therefore there is no issue here.

CONCLUSION

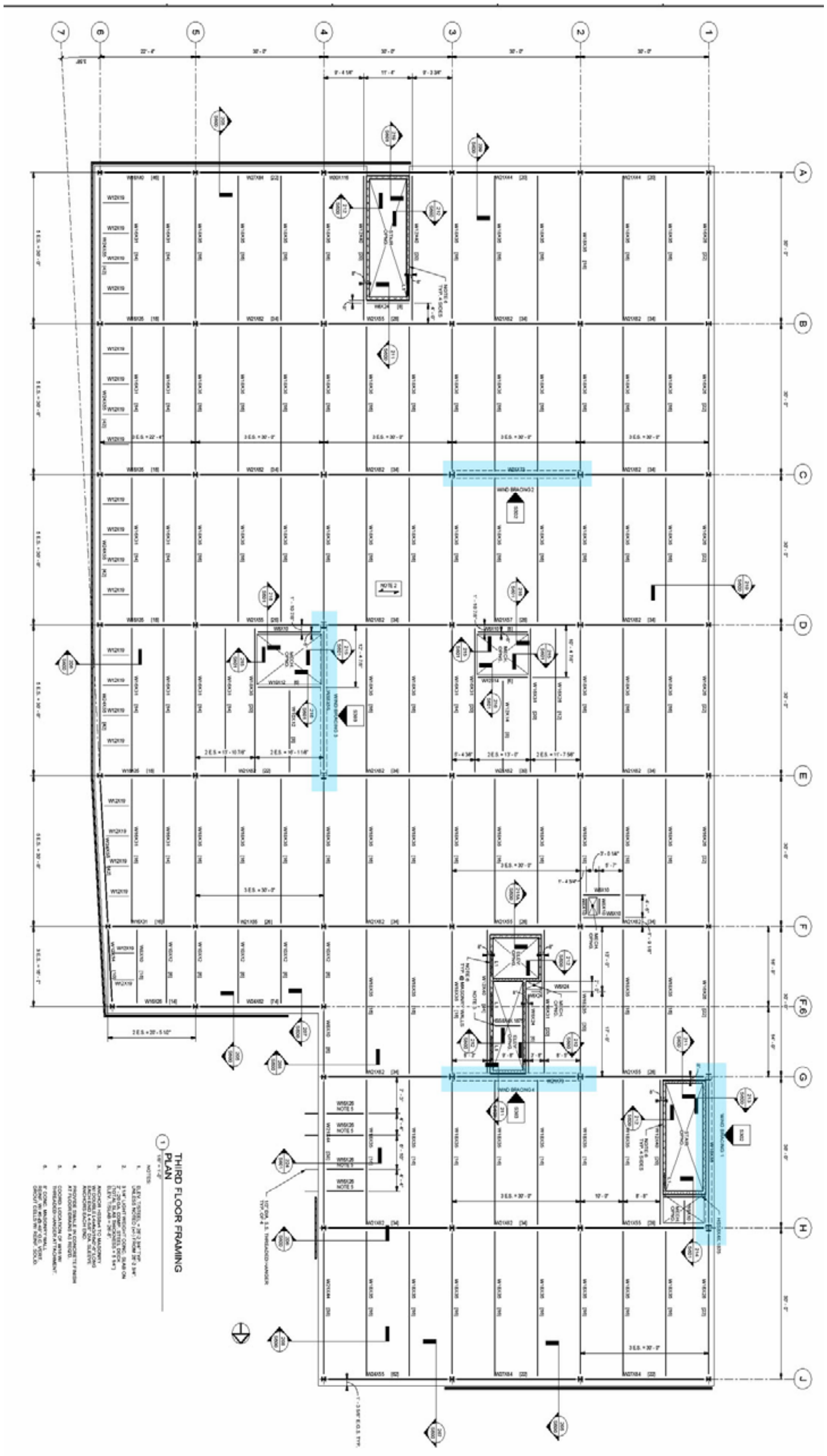
This technical report analyzed Mountain State Blue Cross Blue Shield's lateral force resisting system. Computer models were used and hand calculations were made to ensure that the results received from the models were correct. The hand calculations verified that the computer models' output was correct and more precise. The computer was able to take into account every opening in the building and all irregularities in geometry. Hand calculations were also used to ensure the building was within acceptable drift limits and to check the strength of the members used by the computer program. The braces used to resist lateral loads in the building are designed well to handle the loads. The interaction equation proves that with a different configuration for these braces, columns and braces may be able to be reduced to be more economical. The design strength for all the members was well within their ultimate capacity.

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 Plan

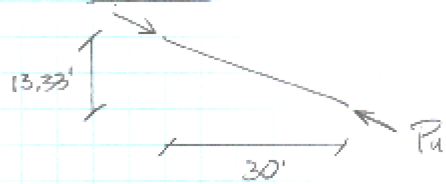


Layout of Braced Frames

APPENDIX B:
MEMBER SPOT CHECKS

MEMBER SPOT CHECKS

BRACE



2nd FLOOR BRACE 2
HSS 10 x 10 x 3/8

$F_y = 46$ ksi
$P_u = 233^k$
$L_b = 32.83'$

DESIGN CHECK

$$\frac{KL}{r} = \frac{1.0\,(32.83')\,(12''/ft)}{3.92} = 100.5 < 200 \ \underline{\underline{ok}}$$

STEEL MANUAL TABLE 4.4

$$\phi P_n = 285^k > P_u = 233^k \quad \underline{\underline{ok}}$$

STRESS RATIO:

$$\frac{P_u}{\phi P_n} = \frac{233}{285} = 0.82 < 1.0 \quad \underline{\underline{ok}}$$

BEAM

2nd FLOOR BM
W 18 x 55

$M_x = 7.31^k$ $\qquad V_u = 18.17^k$
$P_u = 58.31^k$ $\qquad M_y = 0$

$$\frac{KL_x}{r_x} = \text{FULLY BRACED}$$

$$\frac{KL_y}{r_y} = \frac{(1.0)(8.0)(12''/ft)}{1.67} = 57.5 < 200 \ \underline{\underline{ok}}$$

SHEAR CHECK

$\phi V_n = \phi\, 0.6 F_y\, d\, t_w$

$\phi V_n = 1.0(0.6)(50)(18.1)(0.39) = 212^k > 13.17^k$ OK

AXIAL CHECK

$L_b = 8'$

TABLE 6-1

$p = 1.76 \times 10^{-3}$

$pP_u = (1.76 \times 10^{-3})(58.31) = 0.103 < 1.0$ OK

BENDING CHECK

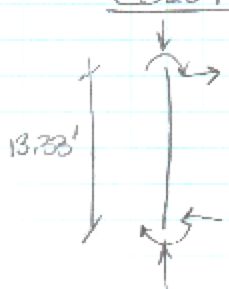
$L_b = 8'$

FROM TABLE 6-1

$b_x = 2.27 \times 10^{-3}$ $\quad b_y = 12.8 \times 10^{-3}$

$0.103 + (2.27 \times 10^{-3})(7.31) + 0 = 0.12 < 1$ OK

COLUMN



$P_u = 347$ $\quad M_x = 5.64$ $\quad$ 2nd FLOOR BRACE 4

$V_u = 0.67$ $\quad M_y = 0.22$

W12X72

$P_u = 347$ $\quad M_x = 2.61$

$V_u = 0.67$ $\quad M_y = 0.46$

$$\frac{KL_y}{r_y} = \frac{1\,(13.33')(12''/ft)}{3.04} = 52.6 < 200 \ \underline{\underline{ok}}$$

SHEAR CHECK

$$\phi V_n = \phi \cdot 0.5\, F_y\, d\, t_w$$

$$\phi V_n = (1)(0.5)(50)(12.3)(0.43) = 132 > 0.67 \ \underline{\underline{ok}}$$

COMBINED LOADING CHECK

$$B_1 = C_m / 1 - \alpha P_r / P_{e1} \geq 1.0$$

$$C_{m_x} = 0.6 - 0.4\left(\frac{5.64}{-2.61}\right) = 1.4$$

$$C_{m_y} = 0.6 - 0.4\left(\frac{-0.22}{-0.46}\right) = 0.41$$

$$P_{e1} = \frac{\pi^2 EI}{(KL)^2} \qquad P_{e1_x} = \frac{\pi^2 (29000)(597)}{(160)^2} = 6{,}675^k$$

$$P_{e1_y} = \frac{\pi^2 (29000)(195)}{(160)^2} = 2180^k$$

$$B_{1x} = 1.4 \qquad B_{1y} = 1.0 \qquad \text{(BY INSPECTION)}$$

$$B_2 = \frac{1}{1 - \frac{\alpha \Sigma P_{st}}{\Sigma P_{e2}}} \geq 1.0$$

$\Delta H_x = 0.221$

$\Delta H_y = 0.229$

$$\Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta H} \ ; \ R_M = 1.0$$

$$\Sigma P_{e2x} = \frac{1.0(107.7)(160)}{0.221} = 77973^k$$

$$\Sigma P_{e2y} = \frac{1.0(107.4)(160)}{0.229} = 75039^k$$

$$B_{2x} = \frac{1}{1-\left(\frac{380.7}{77973}\right)} = 1.01$$

$$B_{2y} = \frac{1}{1-\left(\frac{382.32}{75039}\right)} = 1.01$$

$P_r = 1.4(347) = 486^k$

$M_x = 1.4(5.64) = 7.9'^k$

$M_y = 1.4(0.46) = 0.64'^k$

$p = 1.28 \times 10^{-3}$ $\quad b_x = 2.28 \times 10^{-3}$ $\quad b_y = 4.82 \times 10^{-3}$

$pP_r = 1.28 \times 10^{-3}(486) = 0.62 > 0.2$ $\quad$ H1-1a

$0.62 + (2.28 \times 10^{-3})7.9 + (4.82 \times 10^{-3})(0.64) = 0.643 < 1.0$

$\underline{\underline{\text{ok}}}$

APPENDIX C:
RAM OUTPUT

Center of Rigidity

RAM Frame v12.1
DataBase: 06265_00^RAM 11/18/08 17:43:36

CRITERIA:
Rigid End Zones: Ignore Effects
Member Force Output: At Centerline of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Wall Mesh Criteria :
Max. Allowed Distance between Nodes (ft) : 8.00

		Centers of Rigidity		Centers of Mass	
Level	**Diaph. #**	**Xr**	**Yr**	**Xm**	**Ym**
		ft	**ft**	**ft**	**ft**
Roof	1	118.37	106.79	111.41	84.29
4th Floor	1	117.10	107.04	108.98	74.54
3rd Floor	1	109.96	104.35	108.96	74.53
2nd Floor	1	101.83	93.97	107.79	74.23

Story Displacements

RAM Frame v12.1
DataBase: 06265_00^RAM
Building Code: IBC

11/19/08 21:30:0

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Centerline of Joint
P Delta: Yes Scale Factor: 1.00
Ground Level: Base
Wall Mesh Criteria :
Max. Allowed Distance between Nodes (ft) : 8.00

LOAD CASE DEFINITIONS:

E2	Seismic 1	EQ_IBC06_X_-E_F
E4	Seismic 1	EQ_IBC06_Y_-E_F

Level: Roof, Diaph: 1

Center of Mass (ft): (111.41, 84.29)

LdC	Disp X in	Disp Y in	Theta Z rad
E2	0.82063	-0.03224	0.00045
E4	-0.06713	0.89450	-0.00023

Level: 4th Floor, Diaph: 1

Center of Mass (ft): (108.98, 74.54)

LdC	Disp X in	Disp Y in	Theta Z rad
E2	0.73860	-0.03692	0.00039
E4	-0.07966	0.76251	-0.00019

Level: 3rd Floor, Diaph: 1

Center of Mass (ft): (108.96, 74.53)

LdC	Disp X in	Disp Y in	Theta Z rad
E2	0.49148	-0.00870	0.00025
E4	-0.03828	0.50958	-0.00009

Level: 2nd Floor, Diaph: 1

Center of Mass (ft): (107.79, 74.23)

LdC	Disp X in	Disp Y in	Theta Z rad
E2	0.19902	0.00470	0.00009
E4	-0.00625	0.21886	-0.00002

Building Story Shears

RAM Frame v12.1
DataBase: 06265_00^RAM 11/19/08 21:46:05

CRITERIA:

Rigid End Zones:		Ignore Effects	
Member Force Output:		At Centerline of Joint	
P-Delta:	Yes	Scale Factor:	1.00
Ground Level:	Base		

Wall Mesh Criteria :

Max. Allowed Distance between Nodes (ft) : 8.00

Load Case: E2 Seismic 1 EQ_IBC06_X_-E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	61.29	-0.04
4th Floor	1	232.34	-0.87
3rd Floor	1	339.99	-0.79
2nd Floor	1	250.49	0.47
2nd Floor	None	130.16	0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	61.29	61.29	-0.04	-0.04
4th Floor	232.34	171.05	-0.87	-0.83
3rd Floor	339.99	107.65	-0.79	0.08
2nd Floor	380.65	40.66	0.47	1.26

Load Case: E4 Seismic 1 EQ_IBC06_Y_-E_F

Level	Diaph. #	Shear-X kips	Shear-Y kips
Roof	1	-0.06	61.35
4th Floor	1	-1.24	232.61
3rd Floor	1	-1.75	340.00
2nd Floor	1	-8.95	382.32
2nd Floor	None	8.46	0.00

Summary - Total Story Shears

Level	Shear-X kips	Change-X kips	Shear-Y kips	Change-Y kips
Roof	-0.06	-0.06	61.35	61.35
4th Floor	-1.24	-1.18	232.61	171.26
3rd Floor	-1.75	-0.51	340.00	107.39
2nd Floor	-0.49	1.26	382.32	42.32