MOUNTAIN STATE
BLUE CROSS BLUE SHIELD
HEADQUARTERS

PARKERSBURG, WEST VIRGINIA

DOMINIC MANNO

STRUCTURAL OPTION

FACULTY CONSULTANT: DR. ANDRES LEPAGE

Final Thesis Report

4-7-09

MOUNTAIN STATE BLUE CROSS BLUE SHIELD HEADQUARTERS PARKERSBURG, WV

PROJECT TEAM

OWNER: WOOD COUNTY DEV. AUTHORITY
ARCHITECT: BURT HILL
STRUCTURAL: ATLANTIC ENGINEERING SERVICES
CIVIL: CTL ENGINEERING
CONTRACTOR: G. A. BROWN

MEP

2 ROOF TOP UNITS PROVIDING AIR AT 65000 CFM TO VAV BOXES WHICH CONDITION INTERIOR SPACES

SEVERAL ROOMS UTILIZE A SPLIT SYSTEM A/C UNIT

MAIN DISTRIBUTION PANEL IS 277/480 V 3 PHASE 4 WIRE 4000A BUSS SYSTEM

UNINTERRUPTIBLE POWER SUPPLY WITH 60 MIN. BATTERY BACKUP

ARCHITECTURE

OPEN FLOOR PLAN
FLR. TO FLR. HT.: 13'-4"
EXTERIOR BRICK VENEER WITH LARGE GLASS CURTAIN WALL SITS WELL IN DOWNTOWN PARKERSBURG
ROOF HOUSES MECHANICAL EQUIPMENT WITH SURROUNDING SCREEN WALL

BUILDING STATISTICS

SIZE: 128,496 SQ. FT.
HEIGHT: TOP OF STEEL 67' - 6.5"
COST: 18 MILLION (GMP)
DELIVERY METHOD: DESIGN BID BUILD

STRUCTURAL

STRUCTURAL STEEL FRAMING WITH 3.25" LW CONC. SLAB ON 2" STEEL COMPOSITE DECK

FOUNDATION SYSTEM: 4" SLAB ON GRADE WITH CAISSONS

4 BRACED FRAMES RESIST LATERAL LOADS

TYPICAL BAY SIZE: 30' X 30'

DOMINIC MANNO
STRUCTURAL OPTION
http://www.engr.psu.edu/ae/thesis/portfolios/2009/dam336/

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EXECUTIVE SUMMARY

This thesis evaluates the current Mountain State Blue Cross Blue Shield building for the addition of floor and then relocation to a high seismic region of Salt Lake City, Utah. The same type of lateral force resisting system will be maintained in order to see the effect that the seismic region has on the building. The depth study of this report includes the design of an additional floor for the building and the redesign of the lateral system for the new loads. Breadth studies investigate the necessary changes needed to be made architecturally for the new structural design to work while trying to keep the same laid out floor plan. The other breadth will look at the critical schedule impact and cost analysis of the additional floor and the new structural system.

The new gravity floor was redesigned in RAM resulting in minor changes to the columns. The new seismic loads for an additional floor were calculated and the lateral system was altered to keep the same bracing scheme for comparison to the building designed for Utah. Two braced frames were added to comply with code for the new seismic region based on height of the building. The frames needed to be changed from concentrically braced frames to special concentrically braced frames. Member sizes increased for all braced frames and typical X bracing was used in all frames in the Utah Building. Columns lines were also altered in certain locations in order to minimize the effect on the current architectural plans of the building.

The architectural breadth investigates the changes to the floor plans in order to accommodate the additional braced frames and new column locations. A couple of rooms needed rearranged and the entrance locations to certain rooms needed to be moved. Overall these changes did not affect the overall architectural scheme of the building, thus resulting in the same building in Utah as was planned.

The construction management breadth looked at the overall impact in the critical path schedule for the super structure due to the additional floor. When comparing the numbers to the original schedule an increase of 40 days would be needed in order to construct an additional floor. The cost analysis was then analyzed for the difference to add an additional floor, and the comparison to the new super structure needed in Utah. An increase of $0.53 per square foot was for the addition of a floor, and an increase of $1.06 per square foot was calculated in order to build the same building in Utah.

ACKNOWLEDGMENTS

The author wishes to extend gratitude, acknowledgements, and thanks to the following for their more than generous assistance with this thesis:

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Prof. Robert Holland
The entire AE Faculty and Staff

A special thanks to my friends, peers, and fellow students.

An extra special thanks to my family, especially my mother and father for everything they have provided me with throughout my entire life, and especially for the motivation to always be my best.

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.

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 6).

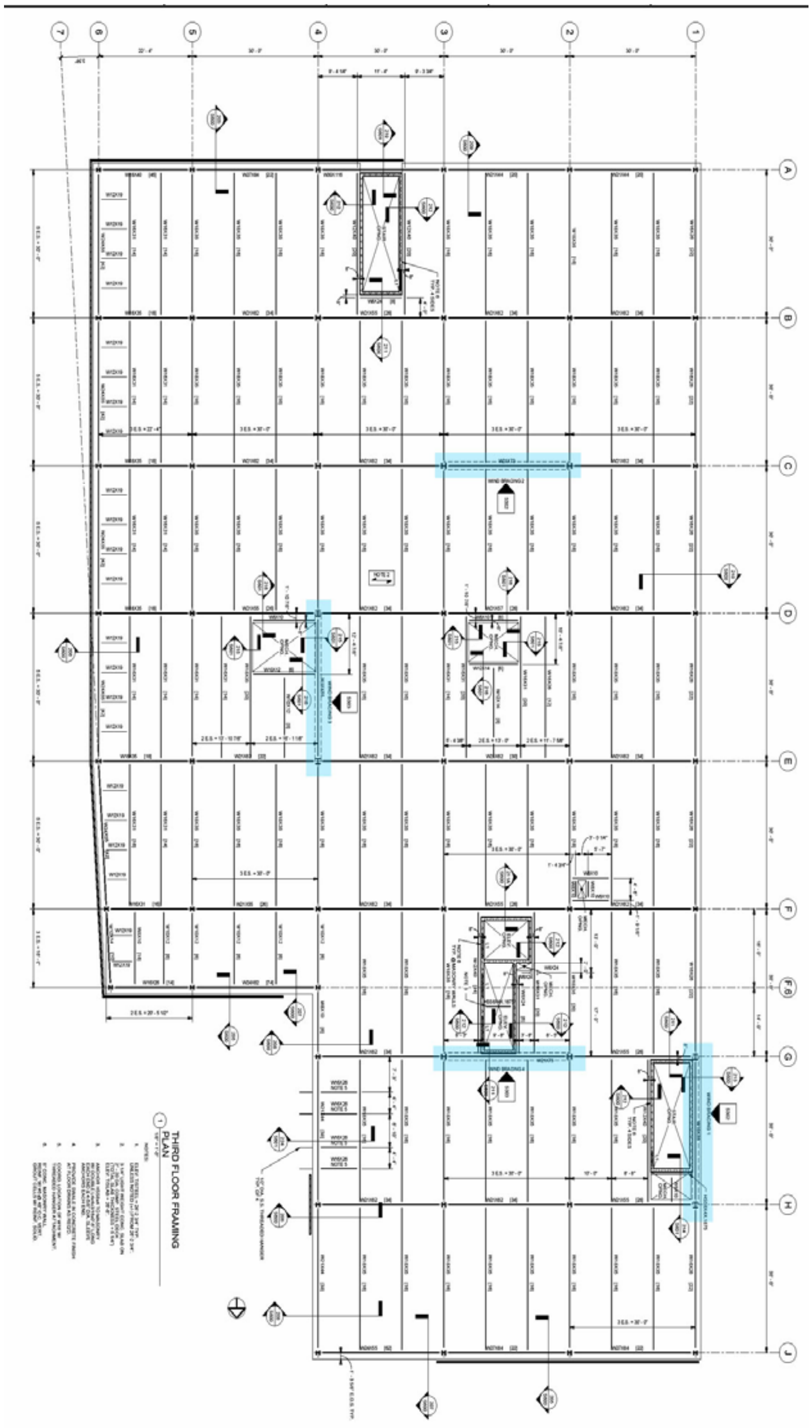

Figure 1: Lateral System Layout

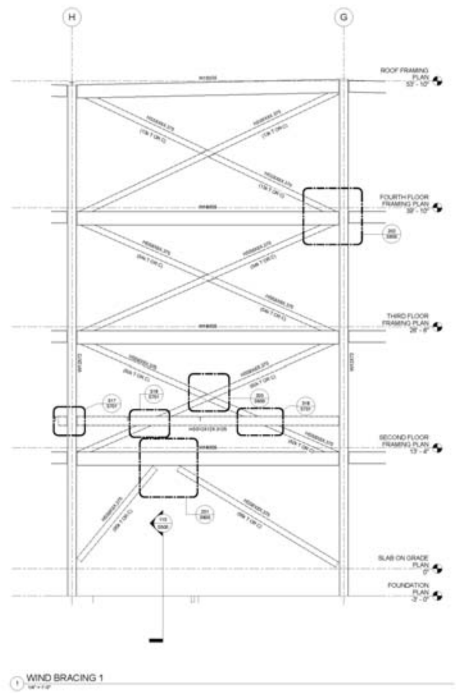

Figure 2: Braced Frame 4 Elevation

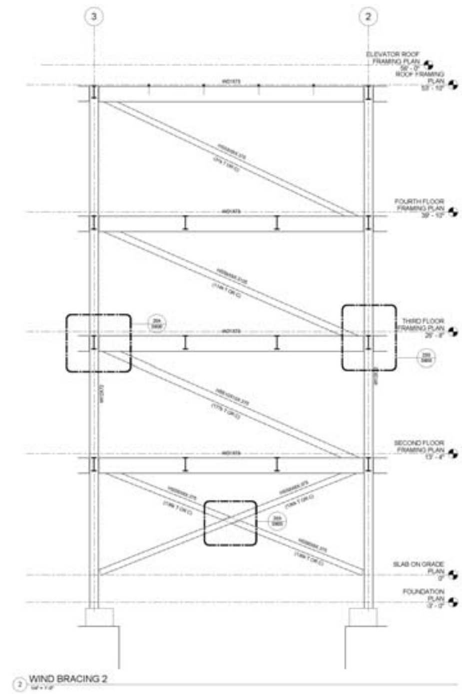

Figure 3: Braced Frame 2 Elevation

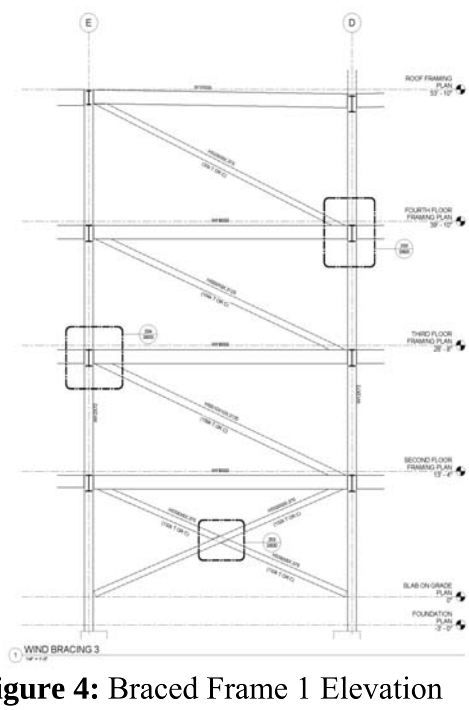

Figure 4: Braced Frame 1 Elevation

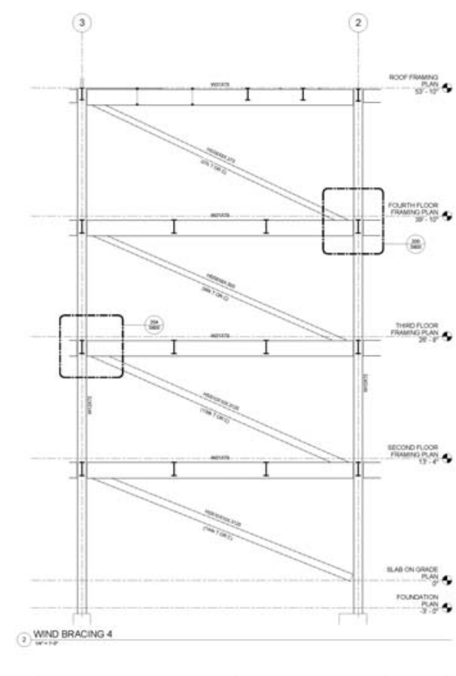

Figure 5: Braced Frame 3 Elevation

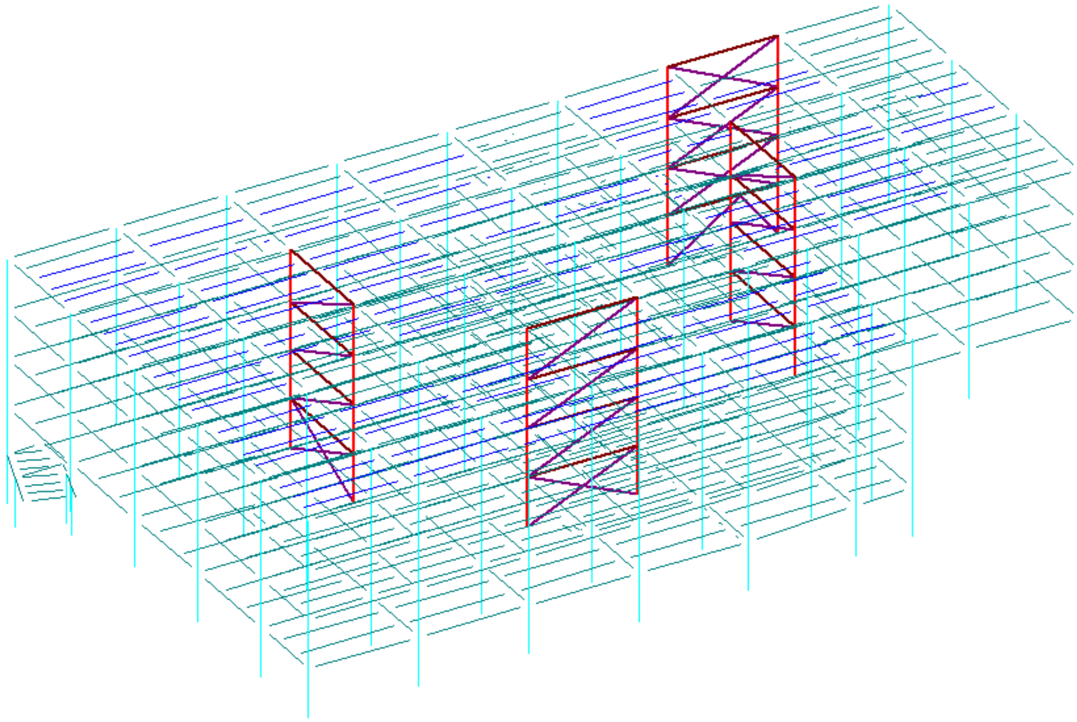

Figure 6: 3-D Layout of Structural System

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

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

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

PROBLEM STATEMENT

After investigating the current structural design in previous technical assignments I found that the system is appropriate for handling the gravity and lateral loads. I am trying to simulate that an owner would like to build almost the same building with an additional floor in Salt Lake City, Utah. Due to the additional weight of an additional floor and change in seismic zone, the building's columns and lateral system will have to be redesigned.

SOLUTION

The existing gravity members in the structure were adequate for the original design. The addition of another floor will implicate that the columns below the additional floor on the first level will need to be redesigned. The columns will be redesigned using AISC Steel Construction Manual, 13^{th} Edition. For a comparable building, the structure will be redesigned with an additional floor in Parkersburg, West Virginia, keeping the same bracing system. The building will then be redesigned to accommodate the change in seismic zone in Salt Lake City, Utah, optimizing the lateral system.

The change in building weight due to the addition of a floor, and the change in seismic zone will require an in depth analysis of the lateral system. The four steel braces that currently resist the lateral forces will need to be investigated for the new seismic loads. The redesign of the lateral system will be monitored for effects in floor to floor heights and also additional loads that the foundation system will need to handle. The braces were originally designed as diagonal and cross braces. In redesigning the system, I plan on looking into different bracing schemes that could be used uniformly in the four braces. The bracing systems will be compared against each other using story drifts. A RAM model will be created in order to determine the feasibility of the new lateral system. The gravity members of the additional floor will be designed according to ASCE 7-05 using RAM Structural System. .

BREADTH STUDIES

CONSTRUCTION MANAGEMENT

The scheduling, and cost impact of the additional floor, and change to the lateral system will be investigated in this breadth analysis. The schedule will be looked at for the increase in the amount of critical path time needed for the addition of another floor. The cost of the original design will be directly compared to the building with the additional floor in WV. The 5 story building in WV will then be compared to the redesigned 5 story building in Utah.

ARCHITECTURE

Due to the change in seismic zone the bracing system locations will most likely need to be looked at and the number of braced frames will need to be increased. The column grid will be changed to accommodate these changes. The floor plan will also be minimally altered to accommodate the need for additional braced frames. The changes in the architectural layout will be shown and compared to the original design.

MAE OPTION

In order to cover the requirements for the MAE program, the building's braced frames will be modeled in SAP 2000. This model will be used to determine the relative stiffness of each braced frame. The entire building will be modeled in RAM Structural System to evaluate the building's detailed gravity design and lateral design.

ADDITIONAL FLOOR

The purpose of this thesis is to design the gravity members for the addition of another floor, then redesign the lateral resisting system for a high seismic region in Salt Lake City, Utah. The original building was modeled using RAM Structural System. The purpose of the addition of another floor is to provide the owner with more office space. The additional floor was framed into the current column grid line and placed at the 3rd floor level shifting the current 3rd and 4th floors up. The gravity and design loads that were used in the design were the same as previously stated on page 15 of this report. This floor, (Figure 7), matched the existing floor layout (Figure 1, pg. 8).

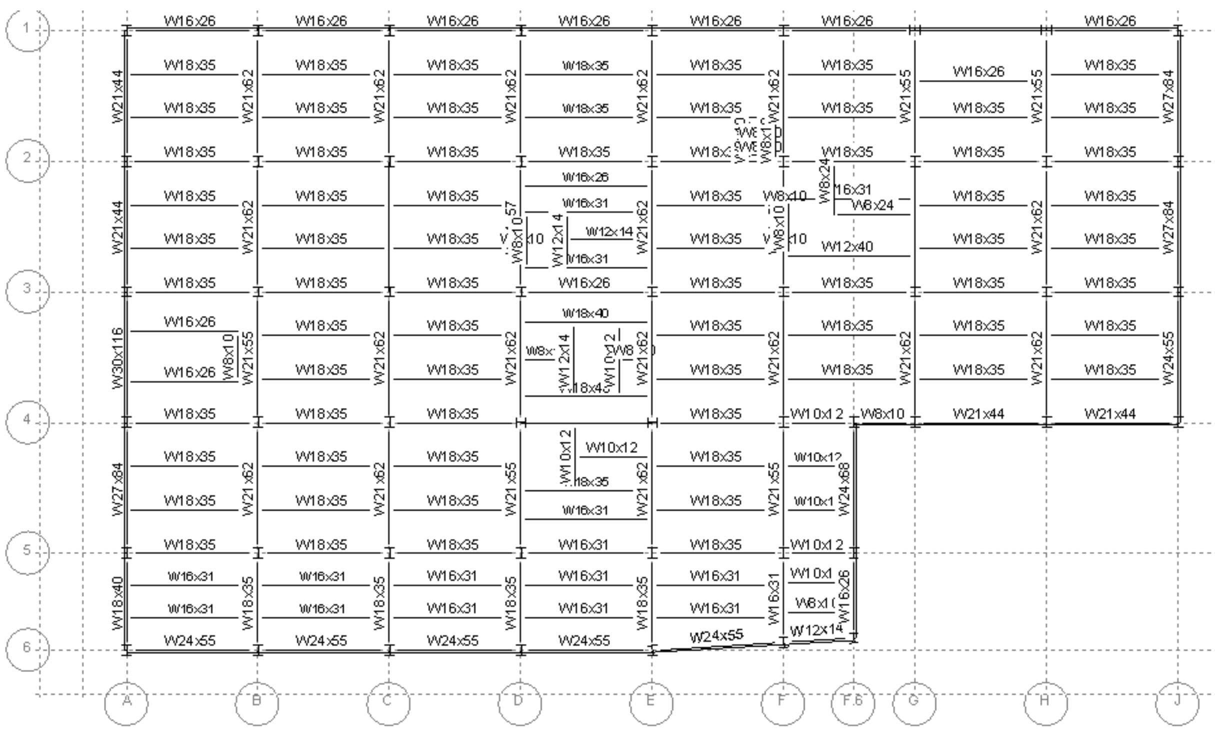

Figure 7: Additional Floor

LATERAL SYSTEM OF ADDITIONAL FLOOR IN PARKERSBURG, WV

The addition of this extra floor meant that the existing lateral system had to be redesigned. This was done so that when the building is redesigned for the high seismic region of Salt Lake City, Utah, a direct comparison could be done to the same 5 story building in West Virginia. In the original design seismic controlled. With the addition of this floor wind and seismic loads needed to be recalculated to determine the controlling lateral loads for the building. These hand calculations can be seen in Appendix B. Below is the resulting base shears and overturning moment for wind and seismic in both directions (Tables 1-3).

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
40	4	0.76	13.40	11.98	-3.79	12.22	-2.23
53.33	5	0.85	14.98	13.08	-3.79	13.35	-2.23
67.33	Roof	0.89	15.69	13.56	-3.79	13.85	-2.23

Table 1: Wind Design Pressures

Wind Design						
	Load (kips)		Shear (kips)		Moment (ft-k)	
Level	N/S	E/W	N/S	E/W	N/S	E/W
Roof	58.3	32.9	0.0	0.0	3537.08	1994.22
5	54.0	30.3	58.3	32.9	2517.62	1415.02
4	50.4	28.1	112.3	63.2	1681.19	937.59
3	48.1	26.7	162.7	91.3	961.46	533.07
2	43.0	23.5	210.8	118.0	286.50	156.48
Total	253.8	141.5	253.8	141.5	8983.86	5036.37

Table 2: Wind Base Shear and Overturning Moment

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	67.3	2662	2763331	0.40	197.61	0	13299.31
5	53.83	2883	2070308	0.30	148.05	197.61	7969.67
4	39.83	2883	1259481	0.18	90.07	345.66	3587.43
3	26.67	2883	649804.4	0.09	46.47	435.73	1239.33
2	13.33	2883	206925.7	0.03	14.80	482.20	197.25
Totals		14194	6949850	1.00	497.00	497.00	26292.98

Table 3: Seismic Base Shear and Overturning Moment

The Wind calculations do not include the 1.6 factor, but after factoring in the 1.6 the seismic loads still control the design of the lateral system.

RAM Structural System was used to design the lateral system for the new seismic loads. In the original model the controlling lateral base shear was 382 kips. The lateral system was redesigned to resist the lateral loads in Table 3. The same ordinary concentric braced frames were used including the additional floor. The braced frames below show the sizes needed to resist these loads (Figures 8-12).

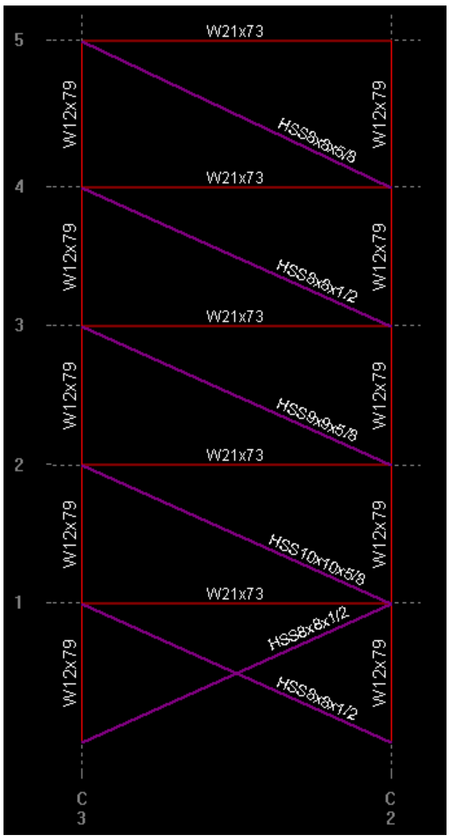

Figure 8: Braced Frame 1

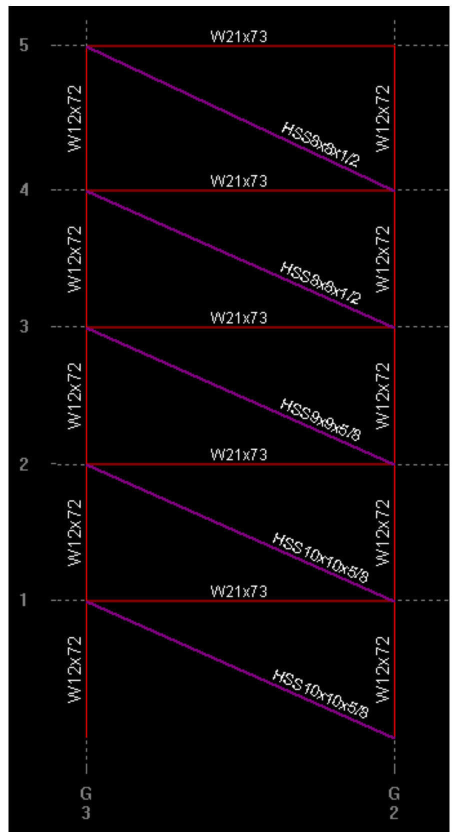

Figure 9: Braced Frame 3

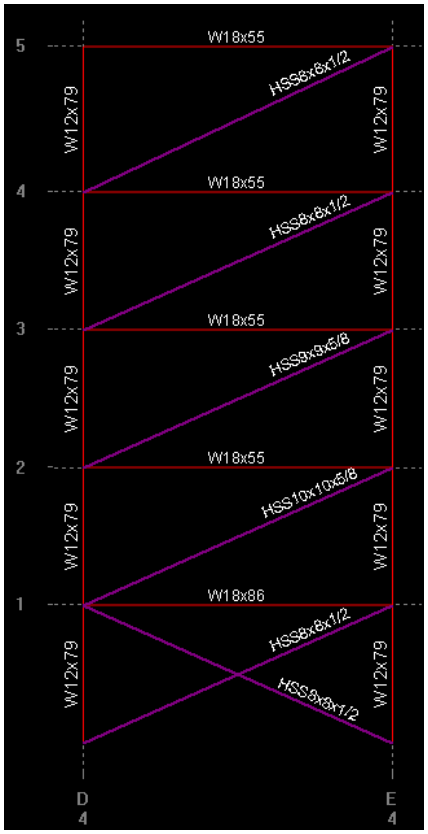

Figure 10: Braced Frame 2

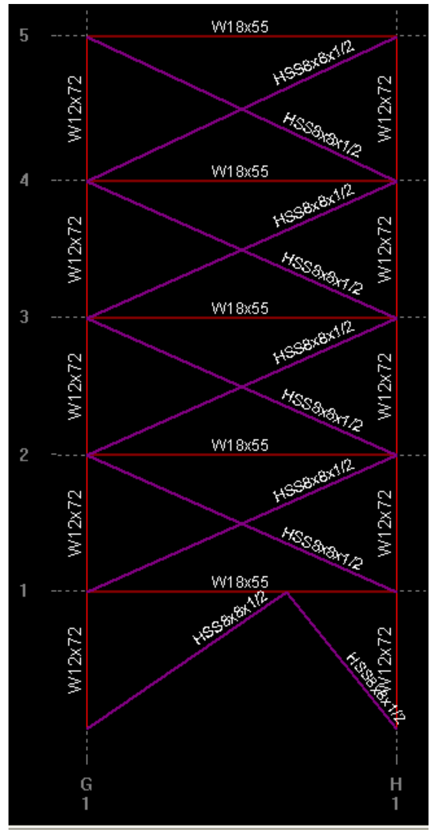

Figure 11: Braced Frame 4

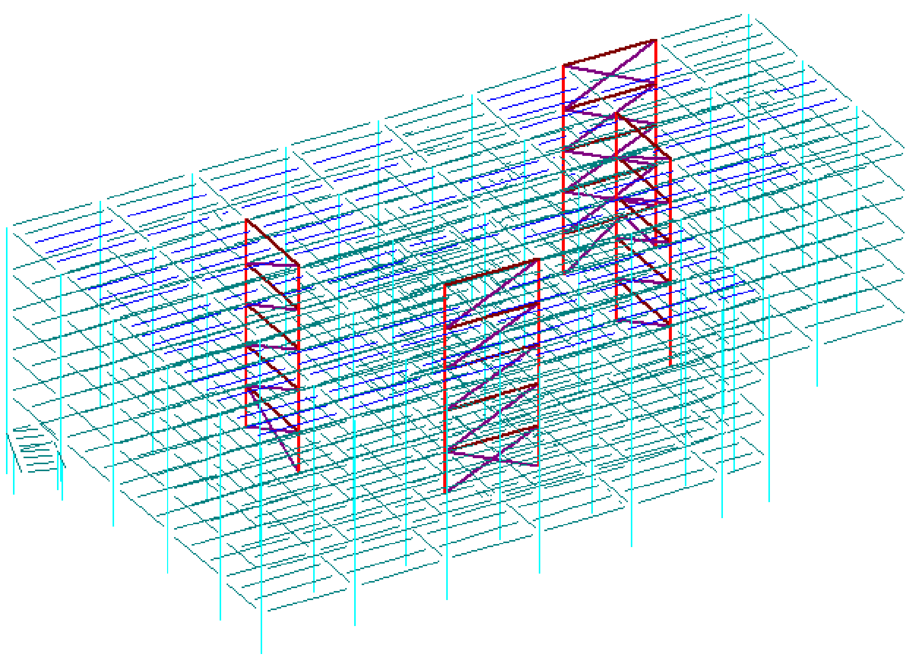

Figure 12: 3-D Layout of Structural System

SAP 2000 Frame Analysis

Sap 2000 was used to determine each braced frame's relative stiffness (Figure 13). A unit load of 100 kips was applied at the top level of the frames resisting loads in each direction. The diaphragm command was used to link the frames. This caused the deflection of the braced frames resisting loads in the same direction to be the same. Excel was then used to determine the frames relative stiffness and direct and torsional shear. Below is the relative stiffness of each frame and the direct shear and torsional shear associated with each frame (Table 4-9). Additional tables used in the calculations can be seen in Appendix C, including center of mass and centers of rigidity. In calculating the direct and torsional shear the base shear from the RAM Structural system was used. RAM's value was lower than my hand calculated base shear. This was due to the fact that I considered the beams and columns to be 15 psf. This was conservative and the actual value is lower resulting in a total weight of the building which is lower than the weight used in the hand calculations.

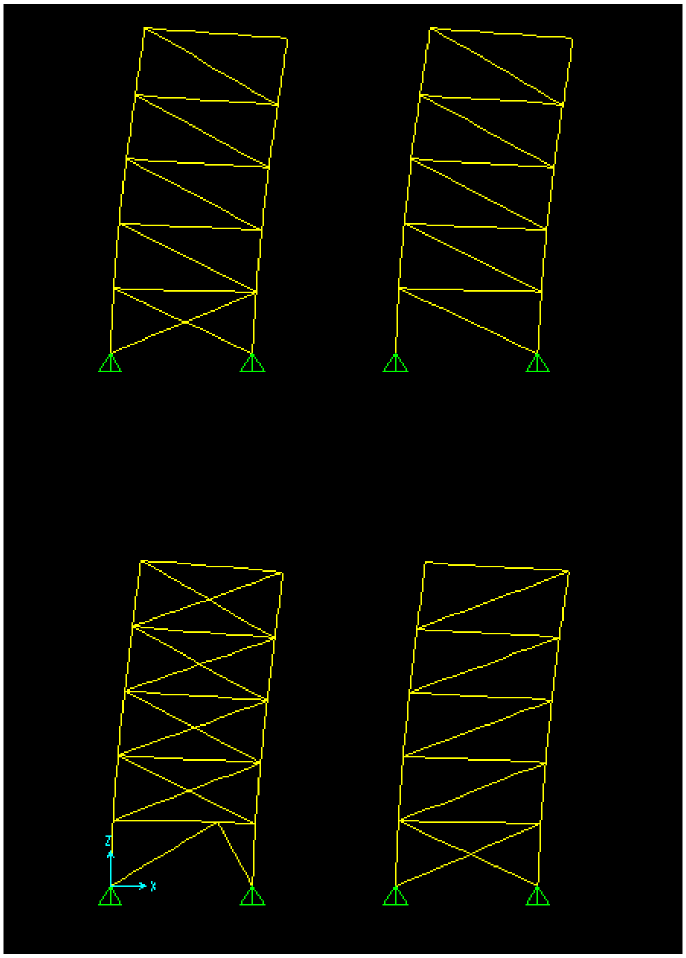

Figure 13: Deflected Shape of Braced Frames in SAP 2000

E/W Direction	Frame 1					
Level	Left Col Shear	Right Col Shear	Hor. Component of Brace	Sum of Forces	Floor Deflection	Relative Rigidity
Roof	0.48	0.49	56.49	57.46	0.3873	148.36
5th	0.46	0.45	57.46	58.37	0.2804	208.17
4th	0.32	0.4	57.23	57.95	0.1832	316.32
3rd	0.6	0.2	51.43	52.23	0.1017	513.57
2nd	0.05	-0.23	54.51	54.33	0.0378	1437.30
			Total of Sum of Forces	280.34		
			Relative Stiffness	0.55		

Table 4: Braced Frame 1 Relative Stiffness

E/W Direction	Frame 3					
Level	Left Col Shear	Right Col Shear	Hor. Component of Brace	Sum of Forces	Floor Deflection	Relative Rigidity
Roof	0.77	0.77	41.8	43.34	0.3873	111.90
5th	0.65	0.65	40.31	41.61	0.2804	148.40
4th	0.5	0.5	47.03	48.03	0.1832	262.17
3rd	0.57	0.57	46.62	47.76	0.1017	469.62
2nd	-0.04	-0.04	46.59	46.51	0.0378	1230.42
			Total of Sum of Forces	227.25		
			Relative Stiffness	0.45		

Table 5: Braced Frame 3 Relative Stiffness

N/S Direction	Frame 2					
Level	Left Col Shear	Right Col Shear	Hor. Component of Brace	Sum of Forces	Floor Deflection	Relative Rigidity
Roof	1.01	1.01	43.54	45.56	0.4389	103.80
5th	1.14	1.14	47.34	49.62	0.3197	155.21
4th	0.7	0.7	48.11	49.51	0.2049	241.63
3rd	0.76	0.76	48.08	49.6	0.1153	430.18
2nd	0.06	0.06	42.77	42.89	0.0455	942.64
			Total of Sum of Forces	237.18		
			Relative Stiffness	0.47		

Table 6: Braced Frame 2 Relative Stiffness

N/S Direction	Frame 4					
Level	Left Col Shear	Right Col Shear	Hor. Component of Brace	Sum of Forces	Floor Deflection	Relative Rigidity
Roof	1.07	1.07	53.07	55.21	0.4389	125.79
5th	1.21	1.21	47.92	50.34	0.3197	157.46
4th	0.73	0.73	49.01	50.47	0.2049	246.32
3rd	0.81	0.81	48.76	50.38	0.1153	436.95
2nd	0.05	0.05	56.99	57.09	0.0455	1254.73
			Total of Sum of Forces	263.49		
			Relative Stiffness	0.53		

Table 7: Braced Frame 4 Relative Stiffness

E/W Direction					
Direct Shear	V*Ri / ΣR		Torsional Shear	V*e*Ri*C/ΣR*C²	
V (k)	Frame 1	Frame 3	V (k)	Frame 1	Frame 3
55.24	30.38	24.86	55.24	4.31	7.42
164.74	90.61	74.13	164.74	12.73	22.29
113.74	62.56	51.18	113.74	9.08	14.99
66.88	36.78	30.10	66.88	5.57	8.45
22.65	12.46	10.19	22.65	2.11	2.54
			Total Shear		423.17

Table 8: E/W Direct, Torsion, and Total Shear

N/S Direction					
Direct Shear	V*Ri / ΣR		Torsional Shear	V*e*Ri*C/ΣR*C²	
V (k)	Frame 2	Frame 4	V (k)	Frame 2	Frame 4
55.25	25.97	29.28	55.25	5.44	5.66
164.36	77.25	87.11	164.36	16.01	17.03
113.39	53.29	60.10	113.39	11.14	11.65
66.69	31.34	35.35	66.69	6.70	6.69
22.65	10.65	12.00	22.65	2.61	2.25
			Total Shear		422.47

Table 9: N/S Direct, Torsion, and Total Shear

Drift calculations were done to determine if the story drift and total drift of the building was acceptable. This is a serviceability consideration and should be limited to as little amount of drift as possible to ensure the tenants no disruption. Deflections were taken from the RAM model and used to check against $\Delta_{SEISMIC} = 0.025h_{SX}$ at each level for seismic for the controlling load case in both directions. A Cd factor of 3.25 was also included in these calculations. Below you can see the comparisons in Tables 10-11. As you can see in the tables, the drifts at each level and overall total drift for each direction is far below the allowable amount. With the building being only 5 stories in height this is not a big surprise. After reviewing the results produced by RAM the members in each brace were designed well resulting with most members being well less than 1 when looking at the interaction equation produced in RAM.

Controlling Seismic Drift E/W Direction										
Story	Story Ht.(ft.)	Story Displacement (ft.)	Story Drift (ft.)	Allowable Story Drift (ft.) $\Delta_{SEISMC} = 0.025H_{sx}$			Total Drift (ft.)	Allowable Total Drift (ft.) $\Delta_{SEISMC} = 0.025H_{sx}$		
Roof	67.3	0.0900	0.00435	<	0.337	Acceptable	0.0220	<	1.68	Acceptable
5th	53.83	0.0800	0.00483	<	0.35	Acceptable	0.0176	<	1.35	Acceptable
4th	39.83	0.0583	0.00476	<	0.33	Acceptable	0.0128	<	0.996	Acceptable
3rd	26.67	0.0358	0.00436	<	0.33	Acceptable	0.0080	<	0.667	Acceptable
2nd	13.33	0.0150	0.00366	<	0.33	Acceptable	0.0037	<	0.333	Acceptable

Table 10: Drift Calculations E/W Direction

Controlling Seismic Drift N/S Direction										
Story	Story Ht.(ft.)	Story Displacement (ft.)	Story Drift (ft.)	Allowable Story Drift (ft.) $\Delta_{SEISMC} = 0.025H_{sx}$			Total Drift (ft.)	Allowable Total Drift (ft.) $\Delta_{SEISMC} = 0.025H_{sx}$		
Roof	67.3	0.0942	0.00455	<	0.337	Acceptable	0.0222	<	1.68	Acceptable
5th	53.83	0.0800	0.00483	<	0.35	Acceptable	0.0176	<	1.35	Acceptable
4th	39.83	0.0583	0.00476	<	0.33	Acceptable	0.0128	<	0.996	Acceptable
3rd	26.67	0.0358	0.00436	<	0.33	Acceptable	0.0080	<	0.667	Acceptable
2nd	13.33	0.0150	0.00366	<	0.33	Acceptable	0.0037	<	0.333	Acceptable

Table 11: Drift Calculations N/S Direction

Foundations:

The foundations needed to be evaluated for the new gravity loads and seismic loads caused by the additional floor. Below in Figure 14 I show the original design capacity of each caisson and the original load from MSBCBS Building. The new load to the foundations is then shown. The foundations which needed to be altered are highlighted in orange.

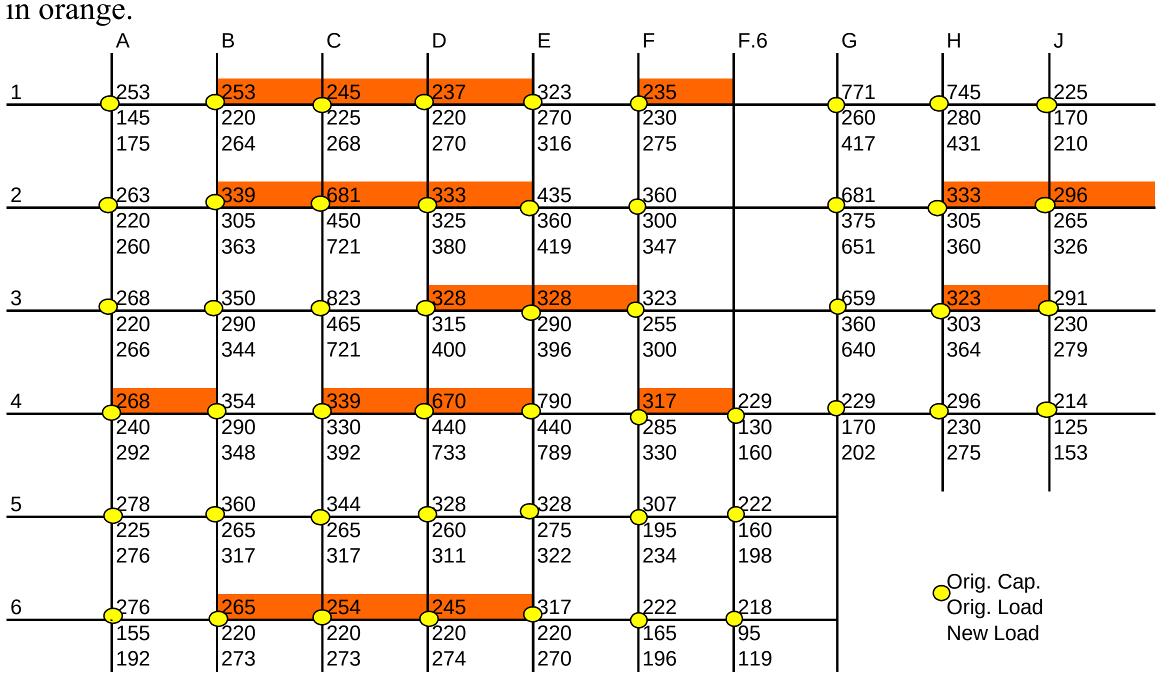

Figure 14: Foundation Loads and Capacities

The following table 12 shows the changes needed to make the allowable loads large enough to be acceptable for the new loads. When altering the foundations I increased the total depth of the caisson in order to save on cost because this method would result in less concrete needed. Table 13 shows the comparison in cubic yards of concrete between the original building and the additional floor for West Virginia.

Changes to Foundations		
Caisson	Additional Feet	Kips
A4	7	299
B1	3	265
B2	5	365
B6	3	276
C1	6	269
C2	4	735
C4	10	396
C6	5	274
D1	9	272
D2	9	381
D3	15	402
D4	7	735
D6	8	276
E3	14	397
F1	13	280
F4	4	333
H2	6	365
H3	9	370
J2	7	333

Table 12: Changes to Foundations

Building Comparison of Cubic Yards of Conc.	
Original Building	1227
Added Floor	1273
Total Difference	46

Table 13: Building Comparison of Cubic Yards of Concrete

REDESIGN OF LATERAL SYSTEM FOR SALT LAKE CITY, UTAH

The first thing that needed to be considered before starting to redesign the lateral system for Utah was to determine the new controlling seismic lateral load. Below is the new seismic criteria that was used in the calculations and detailed hand calculations can be found in Appendix B. The following table contains the new seismic design base shear and overturning moment for Mountain State Blue Cross Blue Shield (Table 14).

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………………………………………………………….…. 1.546
S1…………………………….. 0.602
T_L ………………………………………………………….. 8
Site Class…………….…………………………………..…… D
R……………………………………………………………. 6

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	67.3	2662	2763331	0.40	834.98	0	56194.25
5	53.83	2883	2070308	0.30	625.57	834.98	33674.65
4	39.83	2883	1259481	0.18	380.57	1460.56	15158.14
3	26.67	2883	649804.4	0.09	196.35	1841.13	5236.60
2	13.33	2883	206925.7	0.03	62.53	2037.47	833.47
Totals		14194	6949850	1.00	2100.00	2100.00	111097.11

Table 14: Seismic Base Shear and Overturning Moment for Utah

REDESIGN OF COLUMN GRID LAYOUT

After determining the controlling seismic load and referring to ASCE 7-05, 3 braced frames were needed in each direction of the building layout. With the current floor plan layout and column grid layout, there were no feasible places to position additional braces without rearranging the plan and column layout. After investigation I changed two column grid lines that had a minimal effect on the overall architectural layout of the building, which will be covered in depth in my architecture breadth. The RAM structural model was redesigned to accommodate these changes and the following figure displays the new column grid layout, new locations of the braced frames, and the new beam designs (Figure 15).

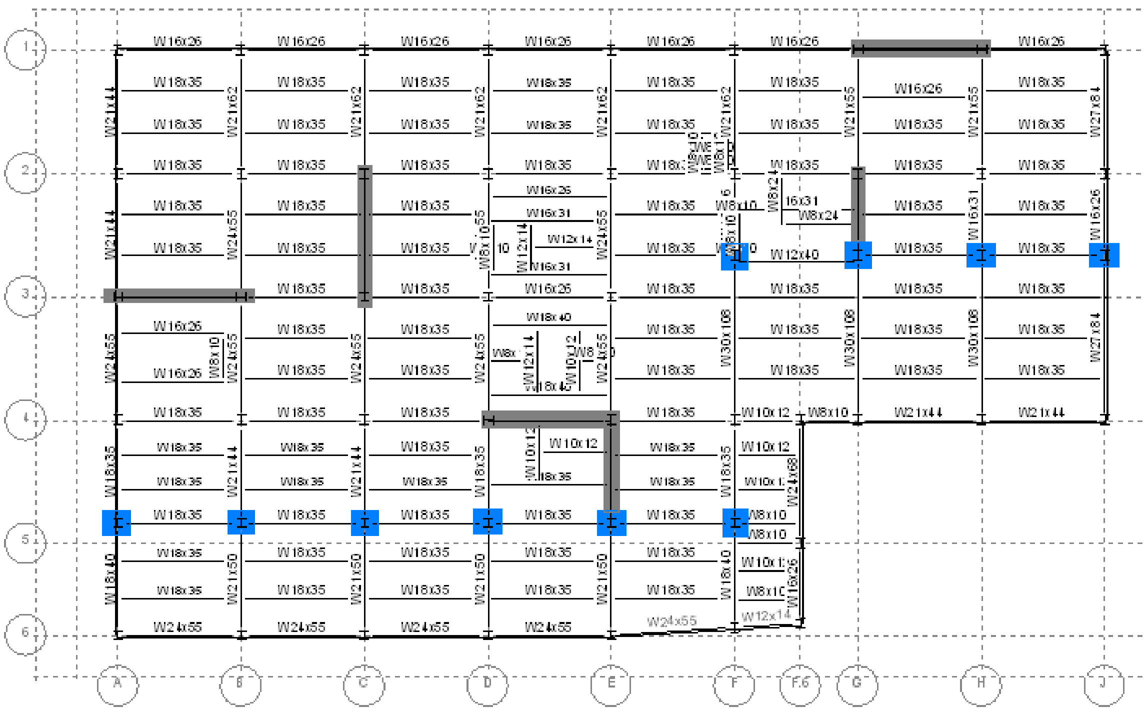

Figure 15: Revised Column and Braced Frame Layout

REDESIGN OF LATERAL SYSTEM

Reviewing the code for the new seismic design category D, ordinary steel concentrically braced frames could no longer be used. Special steel concentrically braced frames were needed in the new design of the building's lateral system. The new controlling lateral load is almost four times the amount of the original seismic design in West Virginia. Different bracing schemes were investigated to determine that using braced frames with a shorter length and angle closer to 45 degrees would result in a more efficient lateral resisting system. Ram structural system was used to design the lateral members assuming that the braces were pinned and the columns were also pinned at the base. The following Figures 16 – 22 show the new braced frames designs and layout.

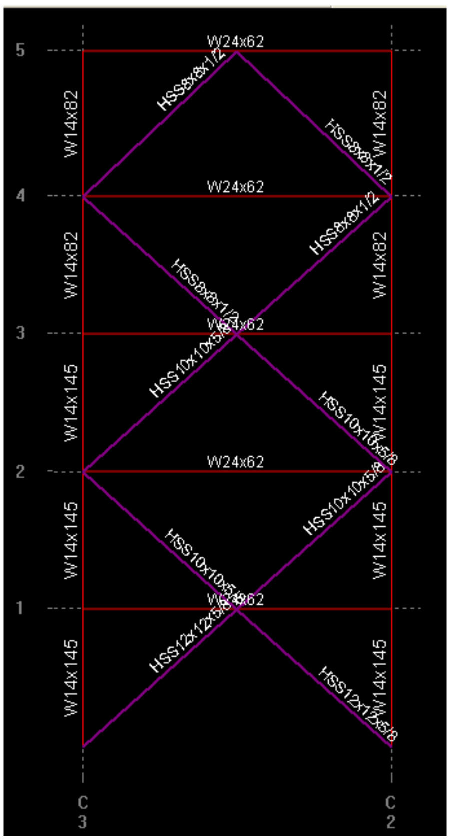

Figure 16: Braced Frame 1

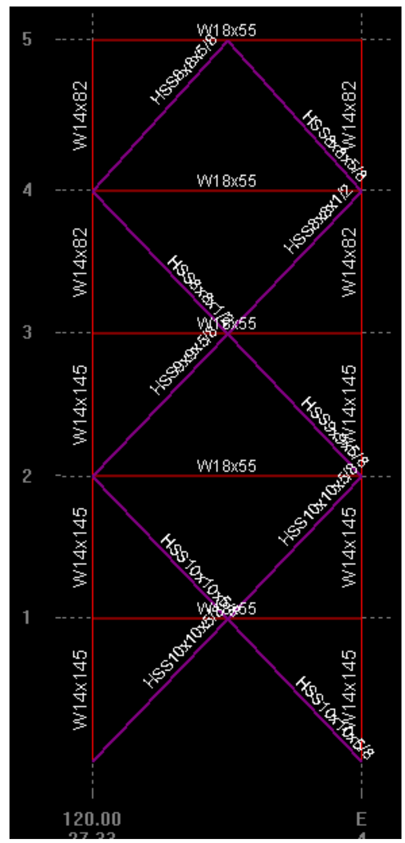

Figure 17: Braced Frame 2

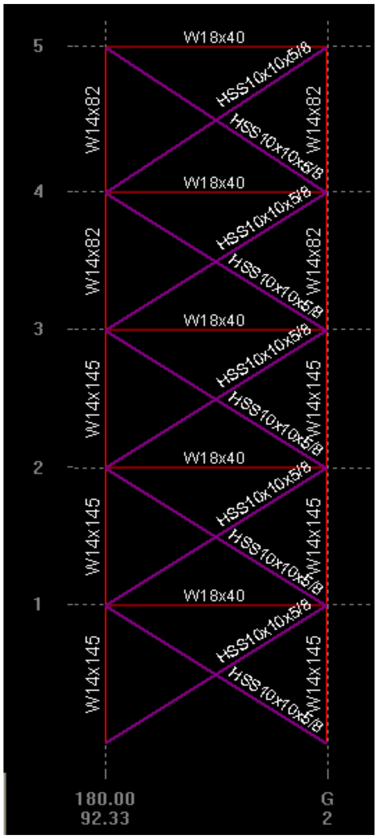

Figure 18: Braced Frame 3

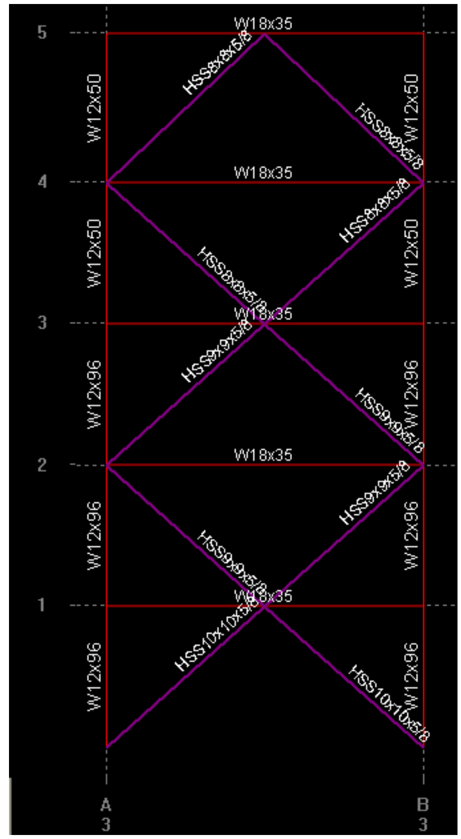

Figure 19: Braced Frame 4

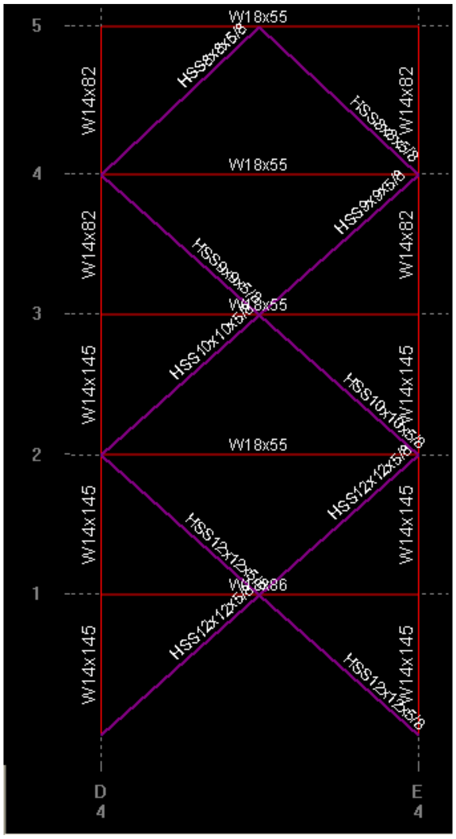

Figure 20: Braced Frame 5

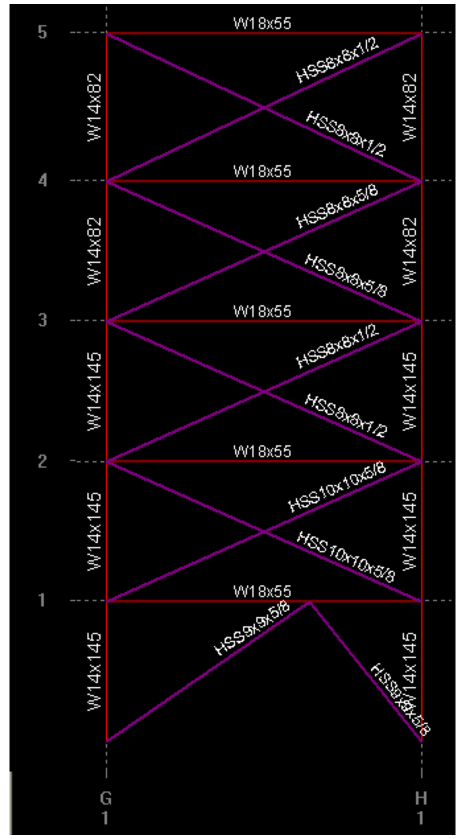

Figure 21: Braced Frame 6

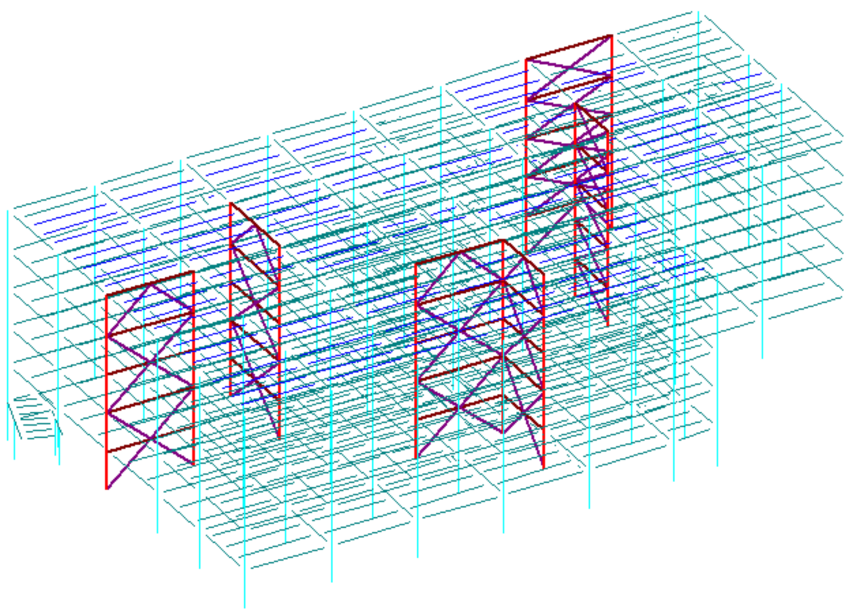

Figure 22: 3-D Layout of Structural System

Sap 2000 Frame Analysis

Sap 2000 was once again used to model the braced frames resisting lateral loads in each direction (Figure 23). The frames were linked together using the diaphragm command which enabled the 3 frames acting in each direction to deflect the same. There was a unit load of 1000 kips that was placed at the top level. Horizontal forces were taken from the model and placed into excel to determine each frames relative stiffness, direct and torsional shear. These forces included the shear in the columns at each level and the horizontal component of the axial force in the braces. Below are each braced frame's relative stiffness and the direct and torsional shear (Tables 15-22). For the direct and torsional shear analysis again the RAM Structural System base shear was used. This time the base shear value was much closer to the value I calculated by hand. This is due to the increase in size and added steel needed to resist the new lateral loads in Salt Lake City, Utah. Additional calculations used in the analysis including new centers of mass and rigidity can be seen in Appendix C.

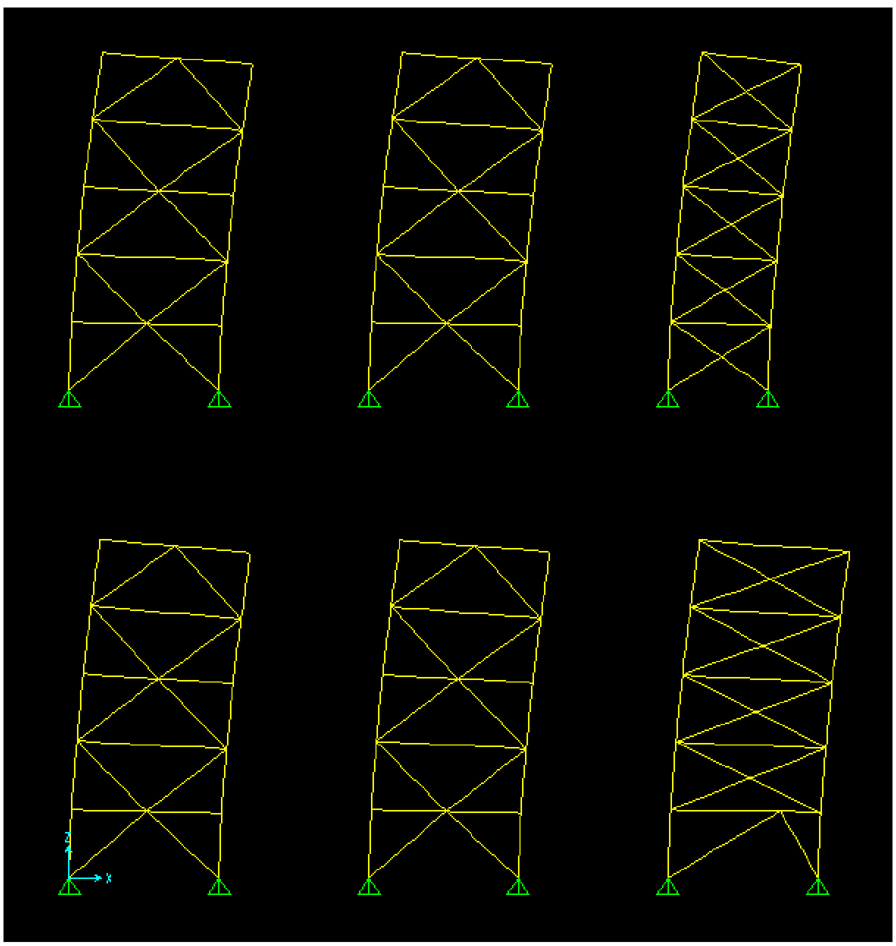

Figure 23: Deflected Shape of Braced Frames in Sap 2000

N/S Direction	Frame 1					
Level	Left Col Shear	Right Col Shear	Hor. Component of Brace	Sum of Forces	Floor Deflection	Relative Rigidity
Roof	4.81	4.81	354.33	363.95	1.5993	227.57
5th	5.19	5.19	333.78	344.16	1.1503	299.19
4th	1.95	1.95	441.86	445.76	0.7126	625.54
3rd	6.07	6.07	337.15	349.29	0.4025	867.80
2nd	-1.51	-1.51	446.94	443.92	0.1296	3425.31
			Total of Sum of Forces	1947.08		
			Relative Stiffness	0.39		

Table 15: Braced Frame 1 Relative Stiffness

N/S Direction	Frame 2					
Level	Left Col Shear	Right Col Shear	Hor. Component of Brace	Sum of Forces	Floor Deflection	Relative Rigidity
Roof	3.19	3.19	417.06	423.44	1.5993	264.77
5th	3.96	3.96	322.76	330.68	1.1503	287.47
4th	-0.54	0.54	394.68	394.68	0.7126	553.86
3rd	4.86	4.86	338.52	348.24	0.4025	865.19
2nd	-1.87	-1.87	365.21	361.47	0.1296	2789.12
			Total of Sum of Forces	1858.51		
			Relative Stiffness	0.37		

Table 16: Braced Frame 2 Relative Stiffness

N/S Direction	Frame 3					
Level	Left Col Shear	Right Col Shear	Hor. Component of Brace	Sum of Forces	Floor Deflection	Relative Rigidity
Roof	1	1	208.26	210.26	1.5993	131.47
5th	2.85	2.85	317.35	323.05	1.1503	280.84
4th	-0.85	-0.85	157.82	156.12	0.7126	219.09
3rd	4.02	4.02	292.31	300.35	0.4025	746.21
2nd	-2.11	-2.11	196.44	192.22	0.1296	1483.18
			Total of Sum of Forces	1182		
			Relative Stiffness	0.24		

Table 17: Braced Frame 3 Relative Stiffness

E/W Direction	Frame 4					
Level	Left Col Shear	Right Col Shear	Hor. Component of Brace	Sum of Forces	Floor Deflection	Relative Rigidity
Roof	1.29	1.29	294.67	297.25	1.5008	198.06
5th	1.31	1.31	234.82	237.44	1.0768	220.51
4th	0.57	0.57	331.75	332.89	0.68	489.54
3rd	2.13	2.13	220.32	224.58	0.3789	592.72
2nd	-0.84	-0.84	344.45	342.77	0.1222	2804.99
			Total of Sum of Forces	1434.93		
			Relative Stiffness	0.29		

Table 18: Braced Frame 4 Relative Stiffness

E/W Direction	Frame 5					
Level	Left Col Shear	Right Col Shear	Hor. Component of Brace	Sum of Forces	Floor Deflection	Relative Rigidity
Roof	3.01	3.01	366.92	372.94	1.5008	248.49
5th	2.86	2.86	350.15	355.87	1.0768	330.49
4th	0.99	0.99	414.63	416.61	0.68	612.66
3rd	4.7	4.7	354.29	363.69	0.3789	959.86
2nd	-1.36	-1.36	421.55	418.83	0.1222	3427.41
			Total of Sum of Forces	1927.94		
			Relative Stiffness	0.39		

Table 19: Braced Frame 5 Relative Stiffness

E/W Direction	Frame 6					
Level	Left Col Shear	Right Col Shear	Hor. Component of Brace	Sum of Forces	Floor Deflection	Relative Rigidity
Roof	3.46	3.47	321.06	327.99	1.5008	218.54
5th	3.17	3.13	398.74	405.04	1.0768	376.15
4th	1.29	1.58	245.63	248.5	0.68	365.44
3rd	4.83	3.36	401.92	410.11	0.3789	1082.37
2nd	-1.39	-2.46	240.31	236.46	0.1222	1935.02
			Total of Sum of Forces	1628.1		
			Relative Stiffness	0.33		

Table 20: Braced Frame 6 Relative Stiffness

N/S Direction							
Direct Shear	V*Ri / ΣR			Torsional Shear	V*e*Ri*C/ΣR*C²		
V (k)	Frame 1	Frame 2	Frame 3	V (k)	Frame 1	Frame 2	Frame 3
249.95	97.48	92.48	59.99	55.24	5.62	1.39	5.26
756.7	295.11	279.98	181.61	164.74	17.32	3.68	15.44
548.38	213.87	202.90	131.61	113.74	12.93	1.62	10.06
338.53	132.03	125.26	81.25	66.88	7.66	0.89	5.87
145.09	56.59	53.68	34.82	22.65	3.13	0.21	1.65
				Total Shear	2039.26		

Table 21: N/S Direct, Torsion, and Total Shear

E/W Direction							
Direct Shear	V*Ri / ΣR			Torsional Shear	V*e*Ri*C/ΣR*C²		
V (k)	Frame 4	Frame 5	Frame 6	V (k)	Frame 4	Frame 5	Frame 6
249.95	71.77	96.52	81.67	55.24	0.65	4.03	4.51
756.7	217.27	292.19	247.24	164.74	1.56	11.49	13.91
548.38	157.46	211.75	179.17	113.74	1.05	7.90	9.62
338.53	97.20	130.72	110.61	66.88	0.34	4.23	5.91
145.09	41.66	56.02	47.41	22.65	0.14	1.46	1.99
				Total Shear	2041.73		

Table 22: E/W Direct, Torsion, and Total Shear

After looking at these calculations you can see that for the N/S direction braced frames 1 and 2 are stiffer than braced frame 3. This is because braced frame 3 spans 20 feet compared to the 30 foot span of braced frame 1 and the 25 foot span of braced frame 2. The x bracing scheme of frame 3 was used to stiffen the frame. Since the moment of inertia is taken into account when determining stiffness. Addition of this steel resulted in a higher moment of inertia for the frame which helps take some of the force from the other two braced frames.

Drift calculations were also performed to ensure that the building stayed well within its limits. Deflections from RAM Structural System were used and to check against $\Delta_{SEISMIC} = 0.025h_{sx}$ at each level for seismic for the controlling load case in both directions. A Cd factor of 5.00 was also included in these calculations. Below you can see the comparisons in Tables 23-24.

Controlling Seismic Drift N/S Direction											
Story	Story Ht.(ft.)	Story Displacement (ft.)	Story Drift (ft.)	Allowable Story Drift (ft.) $\Delta_{SEISMC} = 0.025H_{sx}$			Total Drift (ft.)	Allowable Total Drift (ft.) $\Delta_{SEISMC} = 0.025H_{sx}$			
Roof	67.3	0.1530	0.01137	<	0.337	Acceptable	0.0511	<	1.68	Acceptable	
5th	53.83	0.1220	0.01133	<	0.35	Acceptable	0.0398	<	1.35	Acceptable	
4th	39.83	0.0875	0.01098	<	0.33	Acceptable	0.0284	<	0.996	Acceptable	
3rd	26.67	0.0530	0.00994	<	0.33	Acceptable	0.0174	<	0.667	Acceptable	
2nd	13.33	0.0200	0.00750	<	0.33	Acceptable	0.0075	<	0.333	Acceptable	

Table 23: Drift Calculations N/S Direction

Controlling Seismic Drift E/W Direction										
Story	Story Ht.(ft.)	Story Displacement (ft.)	Story Drift (ft.)	Allowable Story Drift (ft.) $\Delta_{SEISMC} = 0.025H_{sx}$			Total Drift (ft.)	Allowable Total Drift (ft.) $\Delta_{SEISMC} = 0.025H_{sx}$		
Roof	67.3	0.1480	0.01100	<	0.337	Acceptable	0.0468	<	1.68	Acceptable
5th	53.83	0.1290	0.01198	<	0.35	Acceptable	0.0358	<	1.35	Acceptable
4th	39.83	0.0967	0.01214	<	0.33	Acceptable	0.0238	<	0.996	Acceptable
3rd	26.67	0.0575	0.01078	<	0.33	Acceptable	0.0116	<	0.667	Acceptable
2nd	13.33	0.0023	0.00086	<	0.33	Acceptable	0.0009	<	0.333	Acceptable

Table 24: Drift Calculations E/W Direction

As you can see in the tables, the drifts at each level and overall total drift for each direction is far below the allowable amount. After looking into the results that RAM produced all members in the braced frames were designed well. The majority of the members were well below 1 when looking at the interaction equation.

Foundations

Foundations were then evaluated again for the new seismic loads and compared to the additional floor building in West Virginia. I assumed that the soil was the same in Utah as in West Virginia and that caissons were still used. Below in Figure 24 you can see the capacity of the caissons from the additional floor building in West Virginia compared to the new loads from Utah. Foundations that needed to change are highlighted in orange.

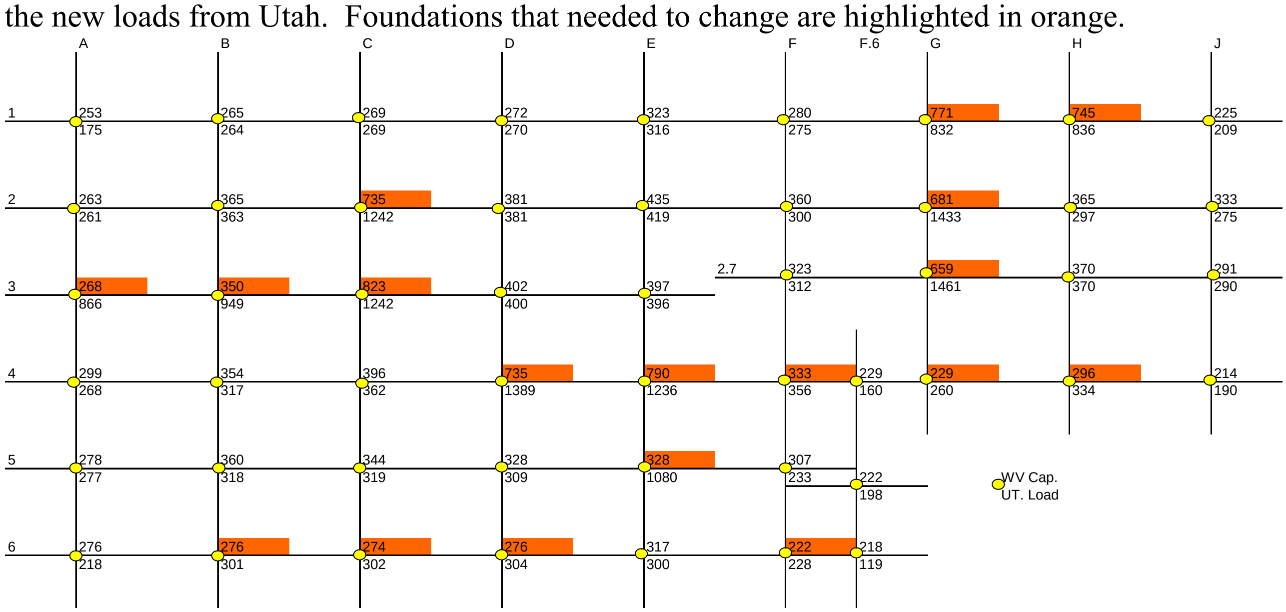

Figure 24: Foundation Loads and Capacities

In Table 25 you can see the changes made in order to make the foundations acceptable for the new loads. Increase in diameter size was needed in order to allow some foundations to work. In table 26 you can see the building comparison in cubic yards of concrete for the 5 story Utah building and the 5 story West Virginia building.

Changes to Foundations			
Caisson	Diam. Change	Additional Feet	Kips
A3	36" to 66"	none	889
B3	42" to 66"	8	954
B6	none	7	303
C2	60" to 72"	13	1244
C3	66" to 72"	17	1244
C6	none	8	303
D4	60" to 72"	22	1399
D6	none	8	307
E4	66" to 72"	30	1243
E5	42" to 72"	20	1088
F4	none	5	360
F6	none	2	229
G1	none	5	837
G2	60" to 76"	23	1489
G2.7	60" to 76"	25	1489
G4	none	8	261
H1	none	6	837
H4	none	8	338

Table 25: Changes to Foundations

Building Comparison of Cubic Yards of Conc.	
WV - 5 Stories	1273
UT - 5 Stories	1661
Total Difference	388

Table 26: Building Comparison of Cubic Yards of Concrete

DEPTH STUDY: SUMMARY

The purpose of this thesis was to redesign the original building in Parkersburg, WV with an additional floor and redesign the lateral system for a high seismic region (Salt Lake City, Utah). The original design did not change much with the addition of another floor. Only a couple of columns needed to be altered and the existing lateral system needed minor changes to accompany the additional floor. When redesigning the building for Salt Lake City, Utah, the column grid had to be redesigned in order to not affect the architecture drastically. The addition of 2 braced frames were needed to comply with code requirements and the braced frame sizes were much larger compared to the original design in Parkersburg, West Virginia. Several foundations needed to be changed in each phase of the redesign. These changes will be further evaluated in the construction management breadth to show the impact on cost.

ARCHITECTURE BREADTH

The purpose of this breadth is to investigate the changes in architecture that must be made in order to accompany the new lateral system needed for MSBCBS if it were built in Salt Lake City, Utah. In designing the lateral system for the building in Utah several different locations were investigated to include the addition of 2 braced frames. Also, since the bracing scheme changed from singular diagonal bracing to cross bracing, openings that were originally located within a braced frame needed to be investigated and altered to accompany the new bracing scheme. For this study, original floor plans are showed and compared to the new column layout and changes in the plan needed for the new lateral system.

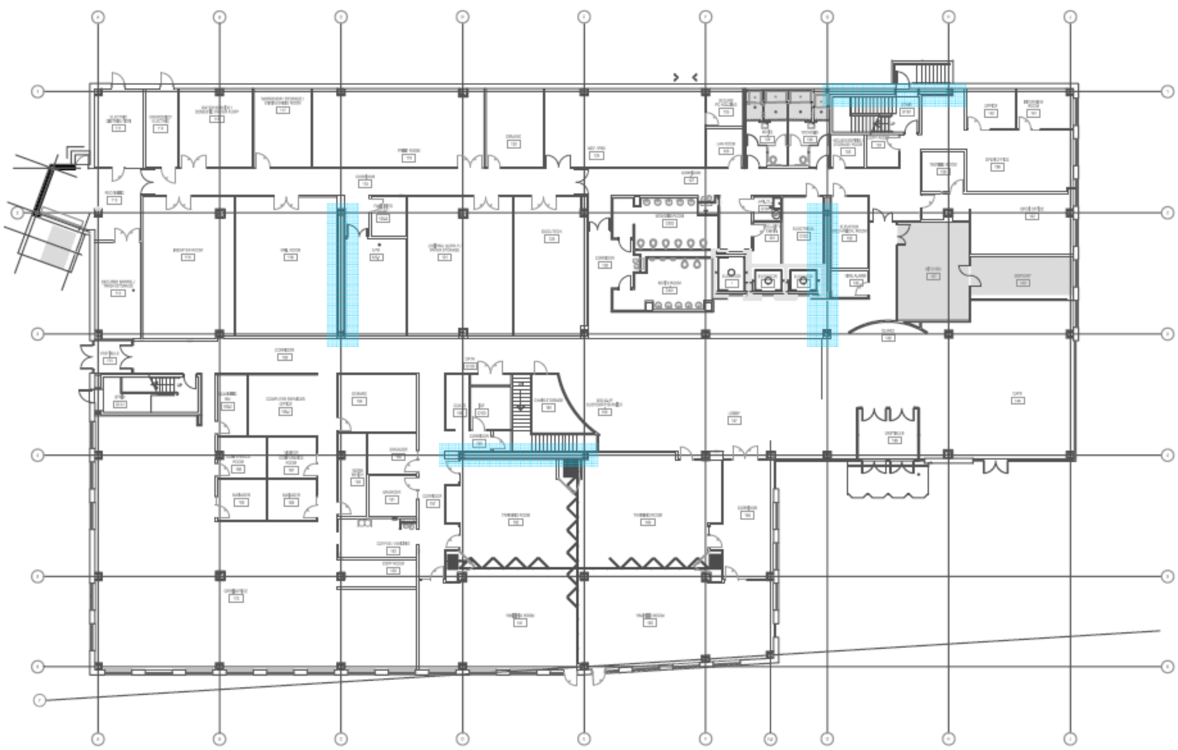

Figure 25: Original First Floor Plan and Location of Braced Frames

Above in Figure 25 you can see the original floor plan and the current location of the 4 braced frames which are highlighted in blue on the floor plan. Below in Figure 26 you can see how the floor plan was revised to include the 2 extra braced frames needed for Salt Lake City, Utah. In order to accompany these changes, I altered the locations of several columns and had to add a wall between grids 4 and 5. All changes are highlighted in purple.

Figure 26: Revised First Floor Plan for Salt Lake City, Utah

The above figure clearly shows the columns which were revised along with the added two braced frames. There was no major change in the actual layout of the floor plan. I was able to relocate these columns without changing the architecture much. The braced frame added at grid line E needed a wall to enclose the frame. This is the only change in the architecture that will be noticed. The retractable walls located within the four training rooms still works. There is just a separation from this wall of about 25 feet now between two of the training rooms. The other added braced frame fits well into the current walls laid out. You can also see here that the original braced frame was shortened by 10 feet in length in order to accompany the X bracing scheme I used in my structural layout.

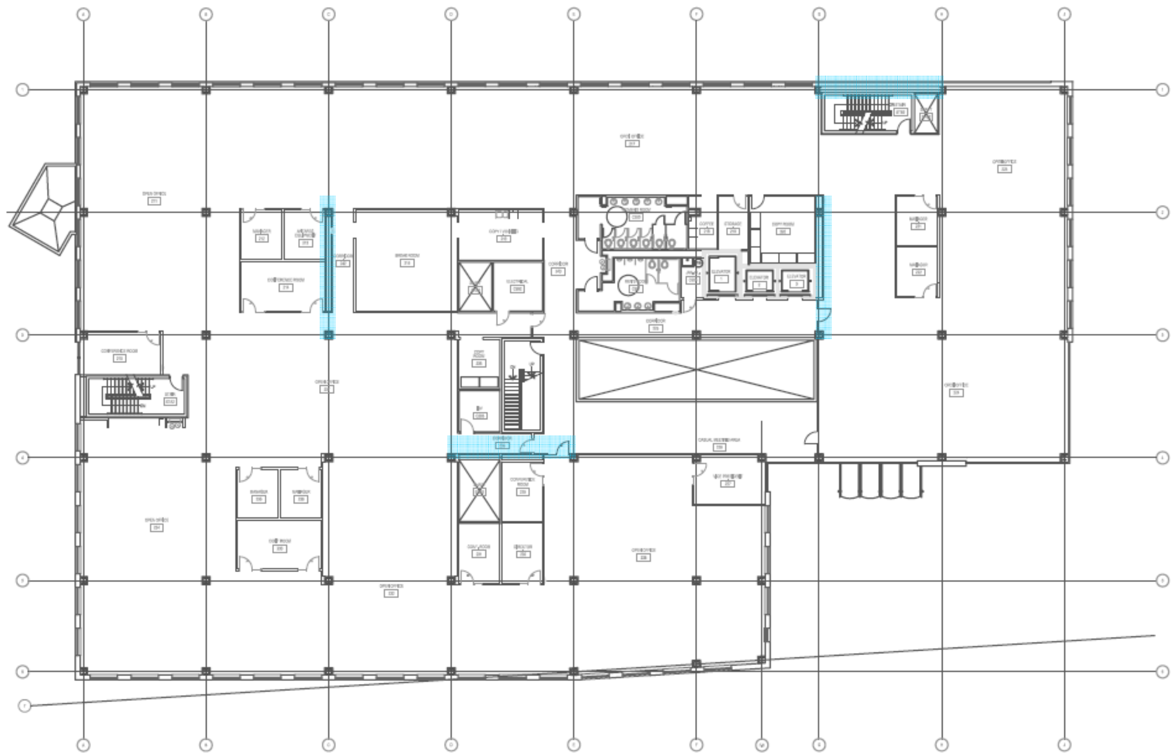

Figure 27: Original Second Floor Plan and Location of Braced Frames

Figure 28: Revised Second Floor Plan

On the previous page you can see the original and revised second floor plans (Figures 27 and 28). The second floor is mainly open office space with an opening to the lobby below on the first floor. In order to include the new bracing scheme and column layout 4 rooms needed to be rearranged. Between grids D and E, I reorganized the two conference rooms and director's room. This was done to integrate the wall needed to enclose the braced frame located on grid E. This caused no major change in the architecture and actually gave the director a bigger office and private hallway. The location of the entrance to the conference room located at grid 3 was moved also. This enabled me to enclose the braced frame along the grid line. Once again you can see that with the changes I made there were no major impacts to the architecture allowing the new structural system to remain hidden.

Figure 29: Remaining Original Floor Plans and Location of Braced Frames

Figure 30: Revised Remaining Floors

In the previous figures (29 and 30), you can see the remaining floors and changes made for the new structural layout. Again in these following floors the director's office and conference rooms were rearranged. The entrance to the other conference room highlighted in purple also was carried up from the previous floor.

BREADTH SUMMARY

The goal of this breadth was to keep the same architectural layout of MSBCBS building in the new design of the structural system. The changes shown are minor and affect the plan layout very little. This creates the same building in Salt Lake City, Utah. It also allows the ability to add two more braced frames which are needed in this high seismic region.

The purpose of this breadth was to look at the schedule impact the additional floor had on the building and determine the cost difference for the superstructure of the addition and the newly designed building in Salt Lake City, Utah. First I determined the increase in critical path time needed to add an additional floor to the original building in Parkersburg, West Virginia. In Table 27 you can see the days needed to complete a floor for each component of the structural system. These numbers were taken from the original schedule to date. I then calculated the total duration needed to complete the original building and with the additional floor. The difference in total days was found and divided by the total duration of the addition to get a percentage of increase. This percentage was then multiplied by the original critical path resulting in a total increase in critical path time of 40 days for the addition of a floor.

Super Structure	Days to Complete One Floor	Original Building	Additional Floor
Steel Fabrication	36	144	180
Steel Erection A-C	7	28	35
Steel Erection C-E	10	40	50
Steel Erection E-J	6	24	30
Deck and Detail A-C	14	56	70
Deck and Detail C-E	14	56	70
Deck and Detail E-J	30	120	150
Conc. Slab	32	128	160
Total Days	149	596	745

Total Difference	149
Percent Increase in Duration	20.00%
Critical Schedule (Original)	200
Critical Schedule (Addition)	240

Table 27: Schedule Impact of Floor Addition

The cost comparison was based on the current bid the owner used to construct the building. Values for the super structure were taken from each RAM model, and values for the foundations were taken from the original drawings and adjusted according to the structural changes needed for the caissons to be adequate. Table 28 shows the values of each structural component. Table 29 shows the cost per unit of each component based on the original building values. Table 30 shows the overall cost of the structural system for each building and the cost per ft^2. You can see that the cost for the addition in West Virginia only resulted in an increase of $0.53 per ft^2, and the design of the building in the high seismic region of Utah resulted in an increase of $1.06 per ft^2.

Structural Component	Original Building	WV Addition	UT Building
Beam / Joists	696392 lbs	879530 lbs	864401 lbs
Studs	14684 lbs	19548 lbs	19325 lbs
Columns	107416 lbs	141560 lbs	125413 lbs
Frame Members	87880 lbs	125061 lbs	211465 lbs
Floor Decking	93966 ft^2	125288 ft^2	125288 ft^2
Roof Decking	31322 ft^2	31322 ft^2	31322 ft^2
Slab On Deck	1523 yd^3	2030 yd^3	2030 yd^3
Slab On Grade	387 yd^3	387 yd^3	387 yd^3
Caisson Drilling	3153 Lin. Ft.	3297 Lin. Ft.	3512 Lin. Ft.
Caisson Concrete	1227 yd^3	1273 yd^3	1661 yd^3

Table 28: Amount of Structure in Specified Units

Structural Component	Cost
Steel Cost per Ton	$4,117.00
Decking Cost per ft^2	$2.28
Slab On Deck Cost per yd^3	$319.00
Slab On Grade Cost per yd^3	$621.00
Caisson Drilling Cost per Lin. Ft.	$122.74
Caisson Conc. Cost per yd^3	$154.60

Table 29: Cost per Specified Unit

Building	Total Cost for Super Structure	Cost per ft^2
Original Building	$3,454,297.29	$22.06
WV Addition	$4,246,040.83	$22.59
UT Building	$4,445,426.38	$23.65

Table 30: Total Cost and Cost per ft^2

CONCLUSION

This thesis focuses on the addition of floor to the original building in Parkersburg, West Virginia, and the relocation of the building to a high seismic region of Salt Lake City, Utah. The same type of lateral force resisting system was maintained in relocating the building to Utah. Two additional braced frames were needed to accommodate the relocation to Utah. A typical x bracing scheme was used to handle the building's lateral loads and member sizes were increased accordingly. Changes to the structure's column layout caused certain changes in the floor plans of the building. These changes did not however change the overall scheme that was originally planned for the building. Schedule impact showed that for the addition of a floor only an additional 40 days were needed for the buildings super structure in terms of critical path time. A cost difference of $ 0.53 per square foot resulted from the additional floor. A cost difference of $ 1.06 per square foot resulted from relocating the building to Salt Lake City, Utah.

Overall this is a complete structural analysis of the differences needed in order to relocate the original Mountain State Blue Cross Blue Shield building to Salt Lake City, Utah. I would recommend these changes to the owner if considering developing this building at this new location.

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.