Technical Report 3

Seneca Allegany Casino Hotel Addition

Salamanca, NY

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Structural Option

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Executive Summary

The purpose of this technical report is to analyze the lateral system of the Seneca Allegany Casino hotel addition. The overall effectiveness of the system will be checked by finding the resistive strength, stiffness, load distribution due to direct and torsional shear, serviceability and overturning moment effects on the foundations.

SAC Hotel's lateral system is made up of six concentrically braced frames in the N-S direction and two perimeter moment frames in the E-W direction.

A computer model was generated in RAM in order to effectively apply multiple load cases due to wind and seismic forces. These load cases were compared in RAM and with the aid of Excel spreadsheets. It was determined that Wind Load Case 2 controlled the design of the braced frames in the N-S direction, while seismic forces controlled the moment frames in the E-W direction.

The stiffness of each frame was found with the equation $K=P/\delta$, which concluded that Frame 3 was the stiffest in the N-S direction. With these stiffness values, both wind and seismic loads were distributed to all frames in each direction using the data collected from the RAM model. It was found that there exists an eccentricity in the E-W direction that would produce torsion in the N-S braced frames, thus torsional shear was included in calculations for the N-S direction. Load distribution concluded that Frame 3 took the most load in both wind and seismic cases, and that overall, wind controlled in the N-S direction and seismic controlled in the E-W direction, as proven in Technical Report 1.

Story drift was checked using provisions from ASCE 7-05 and unfactored loads. Using the RAM model, story displacements were found for the controlling wind and seismic load cases. These values were compared to H/400 for wind and .02h for seismic. All requirements were met for allowable displacement.

Lastly, overturning moments were found for wind and seismic in the controlling directions, and then compared to the overall resisting moments. It was determined that the resisting moments were much greater than the sum of the overturning moments, thus the foundations were adequate. A spot check was also performed for a ground level column and brace by checking axial and bending for the column and axial loads for the brace. Both members were determined to be capable of carrying the lateral loads applied.

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Building Introduction

The Seneca Allegany Casino has undergone multiple construction phases over the years, 5 in total, with the first being a pre-engineered metal building housing the original casino floor, built in 2004 and shown on the far right of Figure 1. Phase 2 consisted of an 8 level parking garage, built from precast concrete in 2005. Next came the first 11-story, 200 room hotel tower with a 2 story casino/restaurant addition, built in 2006 with a typical steel framing system. In 2007, Phase 4 was a renovation of phase 1, converting the original casino floor into an event center, which required new steel truss supports for partitions and concert lighting.

Figure 1 - Seneca Allegany Casino
Satellite Photo Courtesy of Bing.com

This thesis will focus on Phase 5, which is another 11-story, 200 room hotel tower with a structural steel framing system bearing on steel pile foundations. This tower ties into an existing portion of the Phase 3 tower, which was originally built to withstand gravity loads from Phase 5. Construction started in 2008, but construction was halted until 2011, with a projected completion date of Fall 2012. Phase 5 is shown in yellow in Figure 1.

Figure 2 shows the hotel tower sheathed in an insulated glass façade, reflecting the same aesthetic of the original hotel tower. The casino is located within the Seneca Indian Reserve in Salamanca, NY, a mountainous region with an average elevation of 1400 ft. above sea level. This high elevation allows for plenty of natural light and there are no other surrounding structures to shade the casino complex. The lower 3 levels of the addition consist of insulated metal panels backed by metal framing studs.

Figure 2 - South Elevation
Photo Courtesy of Jim Boje, PE (Wendel)

Structural System

Foundation

Drawing 1 shows a plan view for the steel pile foundations, with the perimeter of the hotel addition outline in red. The piles are HP12x53's designed for a working capacity of 200 kips and driven to bedrock. The pile caps are designed for a compressive strength of 4000 psi, reinforced with #9 and #11 bars, and range 42" to 72" in thickness. The caps rest on piles and strip and spread footings rest on subgrade with an allowable bearing capacity of 2000 psf. A section of a typical pile can be found in Appendix B.

The perimeter foundation consists of strip and spread footings designed for a compressive strength of 3000 psi, ranging from 5' to 16' in width, reinforced with #5-#8 grade 60 steel bars. The perimeter uses concrete frost walls up to the ground floor slab on grade, while interior column footings make use of piers tied to columns with steel plates and Gr. 36 and Gr. 55 steel anchor bolts. Sections of the strip footing and typical interior column footings with piers can be found in Appendix B. A fixed connection was assumed for the E-W moment frames and a pinned connection for the N-S braced frames.

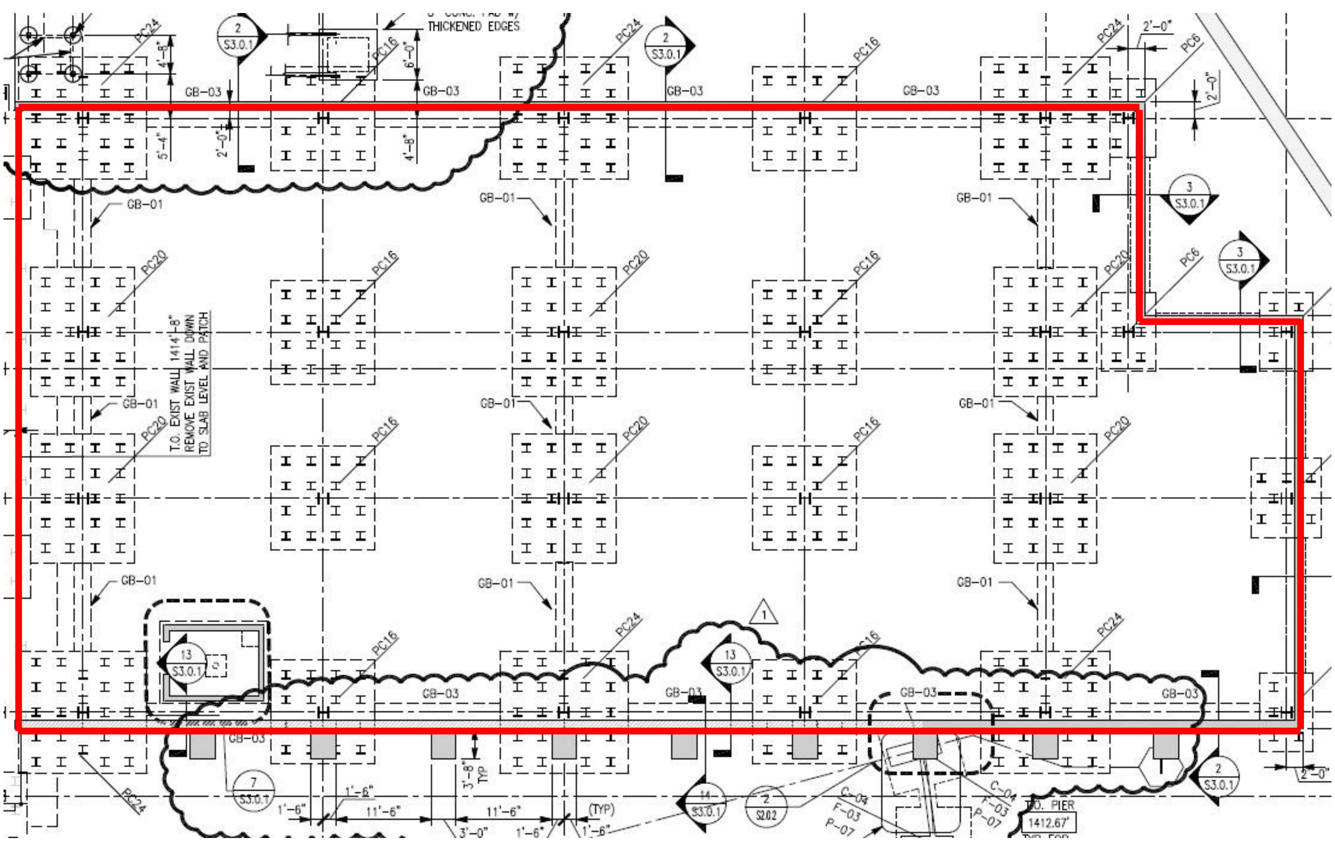


Figure 3 - Steel Pile/Pile Caps Plan
Drawings Courtesy of JCJ Architecture

Framing & Floors

Since this is a hotel tower, the bays are repetitive with the largest bay size a consistent 25'-9" by 29' from the lobby up through the 11th floor. The hotel rooms are located along the outer edges, between column lines 6.6 - 7.3 and 8.4- 9, shown here in Drawing 2. The middle section is the corridor, with a slightly smaller bay size of 20' by 29'. A complete framing plan can be found in Appendix A.

The most significant change in member sizes occurs in the columns and girders as the elevation increases. All structural steel is 50 ksi. The majority of floor beams in the hotel rooms are W16x26, with the exception of the 3rd floor, where they are W16x31 and the mezzanine level, where they are W18x35. The corridor also is consistent with W12x16's on the 3rd through 10th floors. The exception in sizes for the corridor is on the 2nd floor with W14x22's and on the 11th floor with W12x19's.

The floor system consists of concrete slabs on metal deck; 20 gage for hotel rooms and 18 gage for roof, with a 6.5" total depth, normal weight concrete (145 pcf) with compressive strength of 3500 psi and 6x6/W2.9xW2.9 wire mesh. At splices between deck and span changes, #4 rebar spaced at 12" is used. 3/4" diameter shear studs are spaced evenly along beams and girders, with the number shown in plan (see Appendix A). Drawing 3 shows a typical deck section.

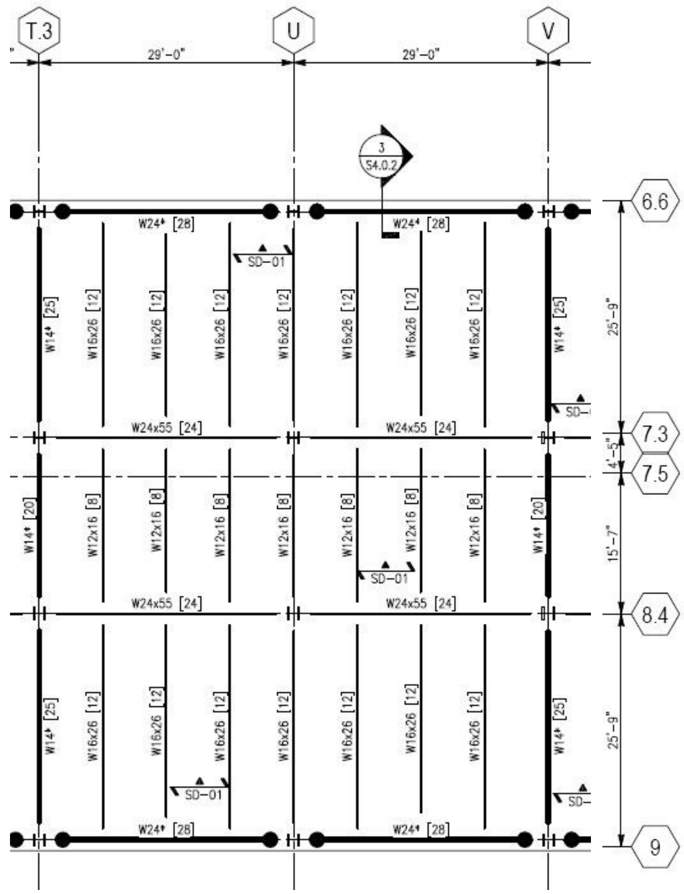


Figure 4 - Section of 4th—10th Floor Framing Plan
Drawings Courtesy of JCJ Architecture

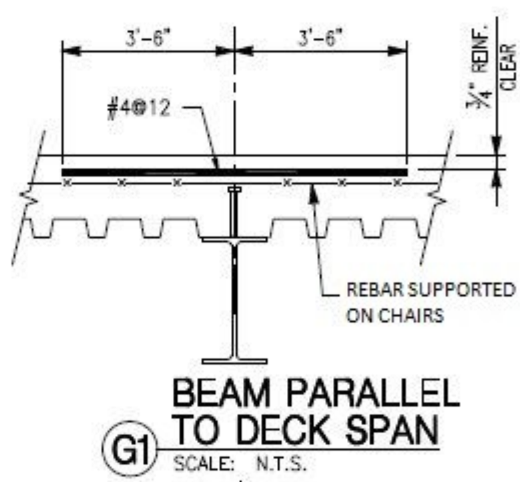


Figure 5 - Typical Composite Metal Deck Section
Drawings Courtesy of JCJ Architecture

Columns

The SAC Hotel addition uses wide flange columns throughout the entire addition. The weight of the columns decrease as the elevation increases, with a small range of sizes used. Figure 6 below shows the column schedule. All columns are in accordance with ASTM A992, 50 ksi steel.

Columns connect to the foundation by use of ASTM A572, 50 ksi base plates, and vary in attachments, whether it be with or without column piers, or directly to frost walls along the perimeter. Anchor bolts conform to ASTM F1554, 55 ksi.

STEEL COLUMN SCHEDULE								
COLUMN MARK	COL. SIZE	BASE PLATE			ANCHOR BOLTS			REMARKS
		T (in.)	W (in.)	L (Ft.–in.)	QTY	SIZE (DIA)	ASTM F1554	
C–01	16"øx0.50" PIPE	2"	24"	2'–0"	4	1 1/4"	GR55	.
C–02	W14x68	1"	22"	1'–10"	4	1"	GR36	.
C–03	W14x90	1 1/2"	22"	1'–10"	4	1"	GR36	.
C–04	W14x132	2"	28"	2'–4"	4	1 1/4"	GR55	20" WIDE BASE ℄ AT HOTEL LOBBY

Figure 6
Drawings Courtesy of JCJ Architecture

Lateral System

The lateral systems used in the SAC Hotel consist of moment frames in the long spans (E-W) directions and diagonally braced frames in the short (N-S) directions. For the moment frames, moment connections occur at columns and girders, shown below in Figures 7 and 8. Typical frame elevations can be found in the lateral analysis section.

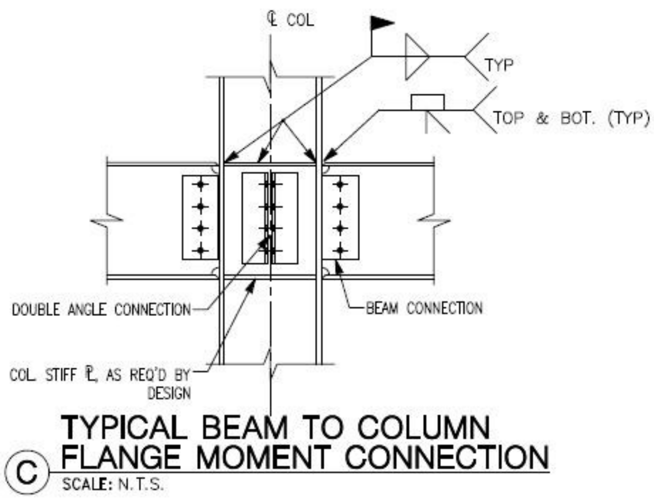


Figure 7 - Typical Moment Connection
Drawings Courtesy of JCJ Architecture

Figure 8 - Typical Moment Connection
Photo Courtesy of Jim Boje, PE (Wendel)

The diagonal bracing is used in specific column lines, Q, S, T.3, V, W, and X. (Framing plan can be found in Appendix B.) Wide flange shapes are used, ranging in size from W14's at the lower floor levels to W10's for the 4th through 10th floor. Column line W has only one bay diagonally braced the entire height of the building to account for the stairwell. The bracing is tied into the frame by use of steel plates embedded in slab deck at beams and columns, shown by Figures 9 and 10. Sections of the diagonal bracing tied into the foundation can be seen in Appendix B.

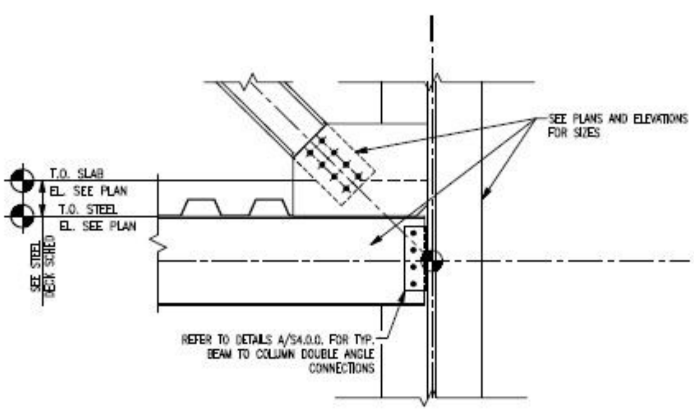


Figure 9 - Diagonal Brace Connection at Column
Drawings Courtesy of JCJ Architecture

Figure 10 - Diagonal Brace Connection at Column
Photo Courtesy of Jim Boje, PE (Wendel)

Roof

The roof structure is consistent with the hotel floor framing, with no change in bay sizes, or location of moment frames, and uses similar metal deck to the hotel floors, with a larger gauge of 20. Slightly larger W shapes are used to account for the extra roof snow load, (40 psf), with the majority of members being W18x35's. A 5' parapet surrounds the perimeter, framed with HSS 14x10x3/16 members embedded within. A detailed parapet section is shown in Figure 11, with the HSS outlined in red. A more detailed roof framing plan can be found in Appendix A. The roof also supports window washing machines, with anchors embedded in the deck. The locations of these can also be seen in Appendix A.

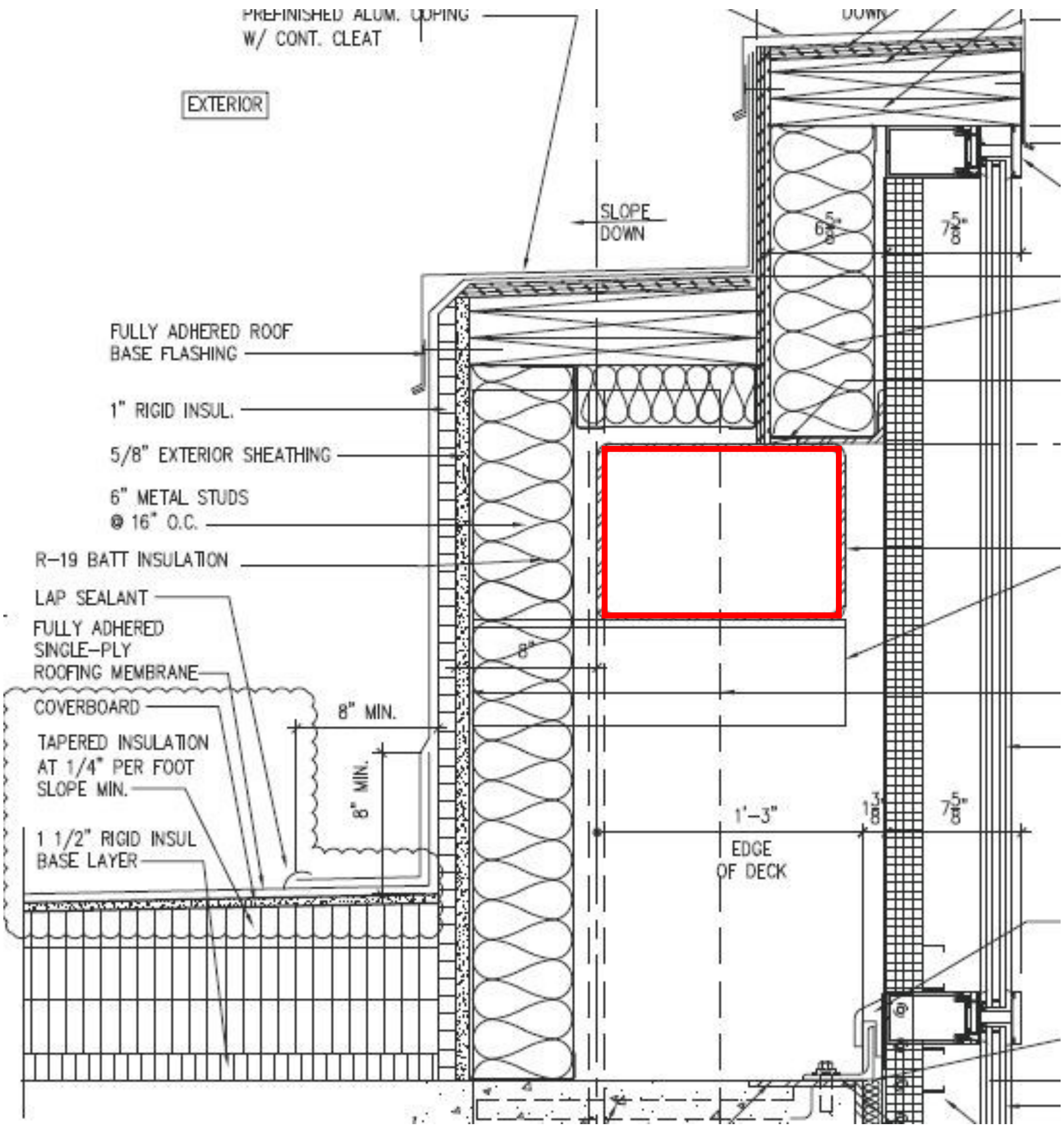


Figure 11 - Roof Parapet Section
Drawings Courtesy of JCJ Architecture

Expansion Joint

The addition to the SAC Hotel requires that the structure tie into the existing structure of the original 11-story hotel tower. This was accomplished using a 12” expansion joint beginning at the 4th floor and at each floor up through the roof level, shown below in Figure 12 and 13. The joint provides a flexible connection which allows the new addition to move independent of the existing tower, resisting wind and seismic loads through the moment and braced frames with no effect on the existing tower.

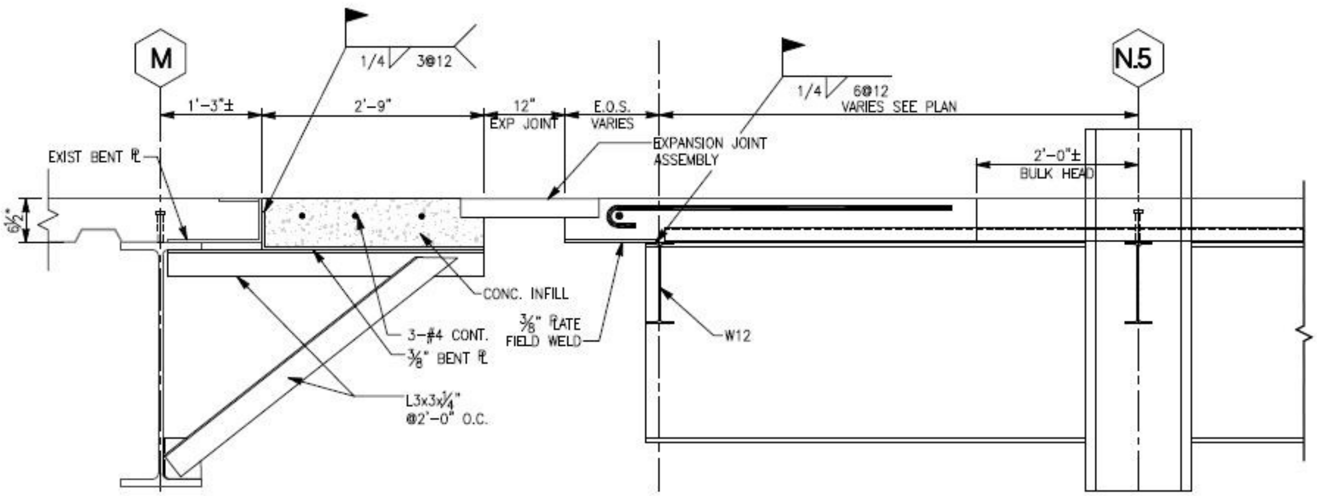


Figure 12 - Expansion Joint Section
Drawing Courtesy of JCJ Architecture

Figure 13 - Expansion Joint Section
Photo Courtesy Jim Boje, PE (Wendel)

Design Codes

Construction of the 2nd SAC Hotel tower began in 2008, and was put on hold until 2011. The following codes were used in the design process:

- 2006 International Building Code
- 2010 New York State Building Code
- ASCE 7-05
- ACI 318-08
- AISC, 13th edition
- Building code requirements for concrete masonry structures ACI-530 and ACI-530.1

For this technical report, the following code editions were used for calculation checks:

- 2009 IBC
- ASCE 7-05
- AISC, 14th edition
- Vulcraft 2008 Decking Catalogue

Material Properties

Concrete

Pilecaps, Piers, and Grade Beams	4000 psi
Footings and Frost Walls	3000 psi
Interior Slabs	4000 psi
Concrete in Slabs on Metal Deck	3500 psi

Masonry

Hollow Masonry Units	ASTM C90, 1900 psi
Mortar	Type S, ASTM C270, 1800 psi
Grout	ASTM C476, 3000 psi

Metal Deck

Hotel Floors	2", 20 Gauge, NWC
Mezzanine and Roof	2", 18 Gauge, NWC

Reinforcement

Reinforcing Bars	ASTM 615, Grade 60
Welded Wire Fabric	ASTM A185
Lap Splices and Spacing	ACI 318

Structural Steel

Connections	Bolts, ASTM A325 or A490
Columns, Beams & Girders	50 ksi, ASTM A992
Tubular Shapes	46 ksi, ASTM A500, Grade B
Round Shapes	36 ksi, ASTM A53, Grade B
Plates	50 ksi, ASTM A572
All Other Steel	36 ksi, ASTM A36
Anchor Bolts	55 ksi, ASTM F1554 (U.O.N.)

Cold Formed Metal Framing

12, 14 and 16 Gage Studs	ASTM C955, Fy = 50 ksi
18 and 20 Gage Studs	ASTM C955, Fy = 33 ksi
Track, Bridging and Accessories	ASTM C955, Fy = 33 ksi

Gravity Loads

Below is an overview of the design loads used in this analysis of the SAC Hotel addition, including loads provided in the specifications and estimations used for calculations.

Dead Loads		
Superimposed	15 psf	Partitions/Façade Estimate
MEP	10 psf	Specs
Ceiling	5 psf	Specs
Metal Deck	69 psf	Vulcraft 2008 Deck Catalog

Live Loads		
	Design Loads	*ASCE 7-05*
Ground Floor	250 psf	
Typical Hotel Rooms	80 psf	40 psf
Hotel 2nd Floor	125 psf	
11th Floor Suites	125 psf	40 psf
Roof and Mezzanine	200 psf	20 psf
Corridors, Stairs, Lobbies	100 psf	100 psf
Mechanical Rooms	200 psf	

Note: Due to drastic differences in ASCE 7-05 values and the Design Loads listed in the specifications, the provided design loads were always used in calculations.

Snow Loads		
	Design Loads	*ASCE 7-05*
Roof Snow Load	40 psf	38.5 psf
Ground Snow Load	50 psf	CS
Drift Snow Load	-	20.5 psf

Note: CS in ASCE 7-05 stands for Case Study snow loads, which is why the 50 psf Design Load was used in calculations, taken from the specifications for the 2010 New York State Building Code.

Lateral Analysis

N-S Seismic Loads

A seismic analysis was performed with ASCE 7-05. A rigid structure was assumed since the N-S direction is braced with concentric steel bracing and the E-W direction makes use of moment connections in the perimeter frames. The design specifications for the SAC Hotel addition provided the seismic response coefficients (Cs) for the short and long term effects, but a check was performed to reproduce these coefficients. The full calculations for the seismic analysis can be found in Appendix C, as well as a screen shot of the USGS report with values used in calculations. The (Cs) value for the N-S direction was found to be 0.028, which is close to the provided value of 0.026. The base shear was found to be 530.4 kips. The following tables provide exact numbers used with a force distribution diagram on the following page.

Floor Weights		
Floor	Mass (Kips)	Sum of Above Weight (kips)
Roof	1396.3	1396.3
11	1578.46	2974.76
10	1578.46	4553.22
9	1578.46	6131.68
8	1578.46	7710.14
7	1578.46	9288.6
6	1578.46	10867.06
5	1578.46	12445.52
4	1578.46	14023.98
3	1583.76	15607.74
2	1586.56	17194.3
Mezz.	1582.36	18776.66
1	166	18942.66

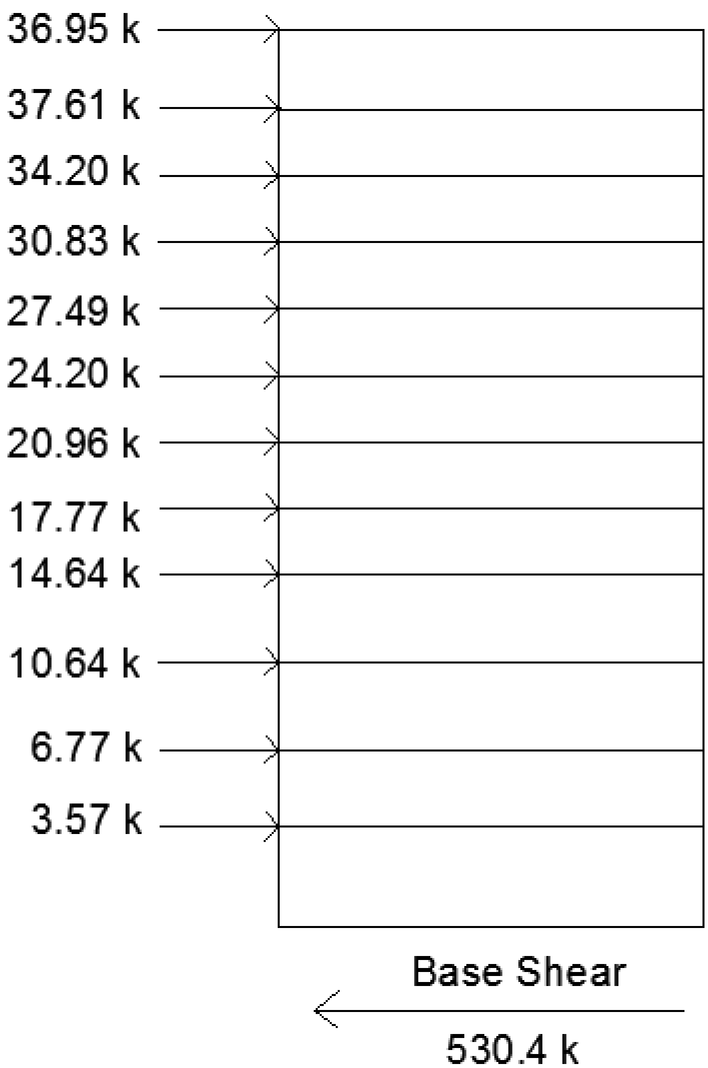
Cs	0.028
$\sum w_i h_i^{1.12}$	2804282.544
V (kips)	530.4
M (ft.-k)	57321.6

Figure 14

Forces				
Floor	Height (ft)	C_{vx}	F_x (kips)	M (ft-k)
Roof	153	0.139319	73.89	11305.9
11	139.33	0.141821	75.22	10480.7
10	128	0.128969	68.41	8755.9
9	116.67	0.116254	61.66	7194.0
8	105.33	0.103674	54.99	5792.0
7	94	0.091267	48.41	4550.4
6	82.67	0.079039	41.92	3465.7
5	71.33	0.067	35.54	2534.9
4	60	0.0552	29.28	1756.7
3	45	0.04013	21.28	957.8
2	30	0.025528	13.54	406.2
Mezz.	17	0.013477	7.15	121.5
		1.00	531.3	57321.6

Figure 15

N-S Seismic Forces



Lateral Analysis

E-W Seismic Loads

The (Cs) value was found to be 0.02. The E-W base shear was found to be 378.8 kips and tables below show the exact numbers found, with a story force distribution diagram on the following page.

Floor Weights		
Floor	Mass (Kips)	Sum of Above Weight (kips)
Roof	1396.3	1396.3
11	1578.46	2974.76
10	1578.46	4553.22
9	1578.46	6131.68
8	1578.46	7710.14
7	1578.46	9288.6
6	1578.46	10867.06
5	1578.46	12445.52
4	1578.46	14023.98
3	1583.76	15607.74
2	1586.56	17194.3
Mezz.	1582.36	18776.66
1	166	18942.66

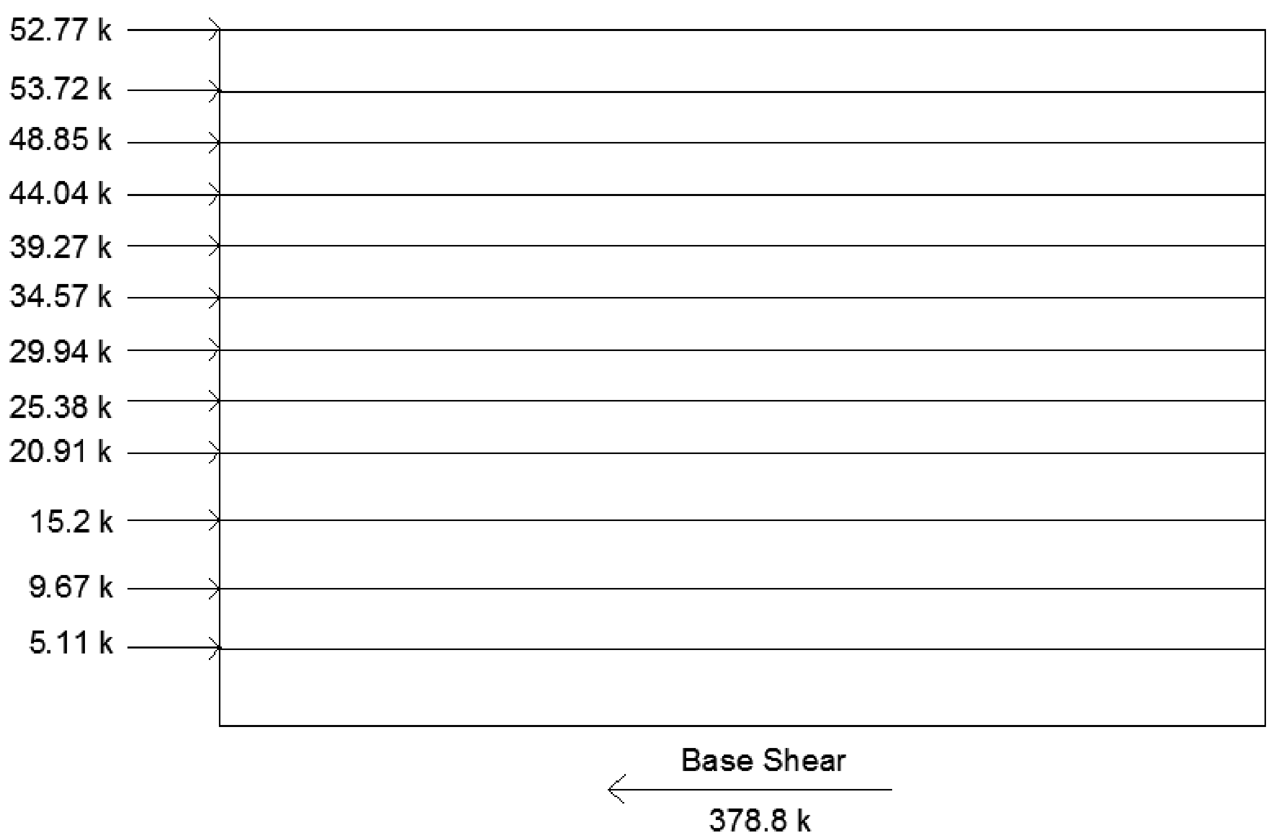
Cs	0.02
$\sum w_i h_i^{1.12}$	2804282.544
V (kips)	378.8
M (ft-k)	40937.8

Figure 16

Forces				
Floor	Height (ft)	C_{vx}	F_x (kips)	M (ft-k)
Roof	153	0.139319	52.77	8074.4
11	139.33	0.141821	53.72	7485.1
10	128	0.128969	48.85	6253.3
9	116.67	0.116254	44.04	5137.8
8	105.33	0.103674	39.27	4136.5
7	94	0.091267	34.57	3249.8
6	82.67	0.079039	29.94	2475.1
5	71.33	0.067	25.38	1810.3
4	60	0.0552	20.91	1254.6
3	45	0.04013	15.20	684.1
2	30	0.025528	9.67	290.1
Mezz.	17	0.013477	5.11	86.8
		1.00	379.4	40937.8

Figure 17

E-W Seismic Forces



Wind Loads

A wind analysis was also performed using the ASCE 7-05 MWFRS procedure. After calculating wind and seismic loads, it was found that wind force controls the N-S lateral system's member sizes, producing a base shear of 912.5 kips and an overturning moment of 73452 ft.-k. This is due to the larger surface area on the 230' side of the building. The base shear in the E-W direction was found to be 325.4 kips with an overturning moment of 14759 ft.-k.

Figures 18-20 show the tabulated values of pressures and story forces, and diagrams of the force and pressure distribution. Full calculations can be found in Appendix C.

Pressures (All Directions)			
Height (h)	Kz	qz (psf)	p (WW) (psf)
0-15	0.85	14.98	14.57
20	0.9	15.86	15.17
25	0.94	16.57	15.65
30	0.98	17.27	16.13
40	1.04	18.33	16.84
50	1.09	19.21	17.44
60	1.13	19.92	17.92
70	1.17	20.62	18.4
80	1.21	21.33	18.88
90	1.24	21.86	19.24
100	1.26	22.21	19.48
120	1.31	23.09	20.08
140	1.36	23.97	20.68
153	1.38	24.32	20.92

p (LW)(N-S) = 10.58 psf
p (LW)(E-W)= 14.72 psf

Figure 18

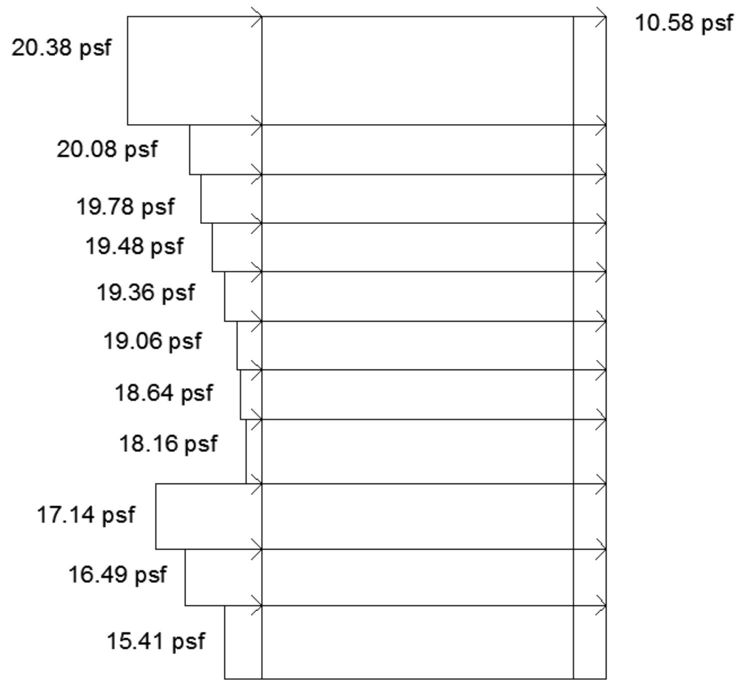
Story Forces (N-S)	
Mezz.	Force (kips)
1	78.65
2	93
3	96
4	74.4
5	75.9
6	77.1
7	78
8	78.3
9	79.4
10	79.9
11	98.2

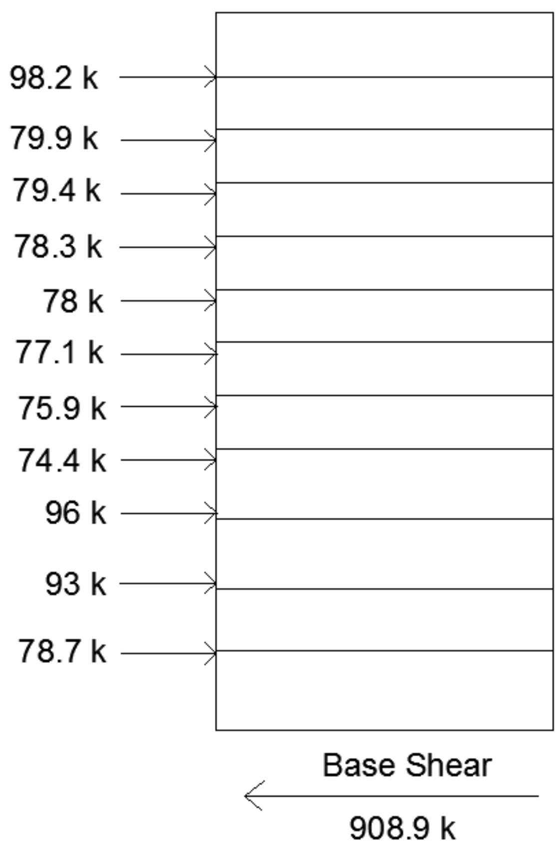
Figure 19

Story Forces (E-W)	
Mezz.	Force (kips)
1	28.5
2	33.6
3	34.5
4	26.7
5	27.1
6	27.5
7	27.8
8	28.1
9	28.2
10	28.6
11	34.8

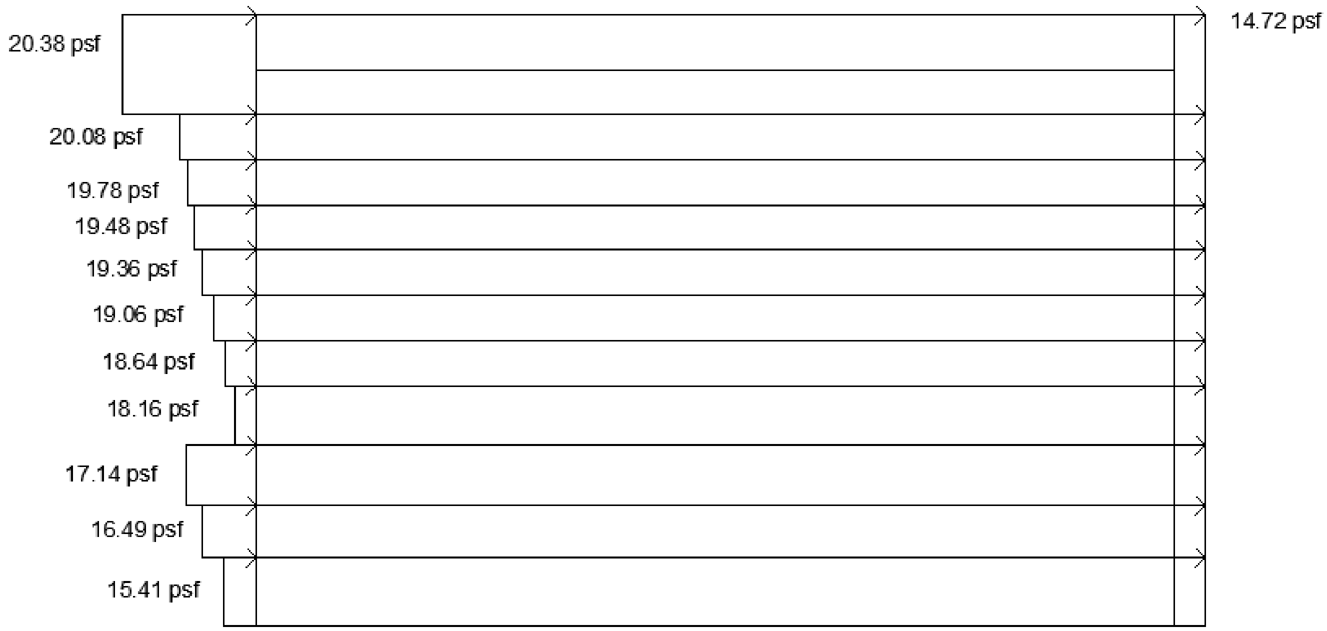
Figure 20

North-South Wind Pressures and Forces





East-West Wind Pressures and Forces



Lateral Load Comparison

	Wind		Seismic	
	N-S	E-W	N-S	E-W
Base Shear (k)	908.9	325.4	530.4	378.8
Overturning Moment (ft-k)	73452	14759	28660	38621

It was determined that the N-S direction is controlled by wind forces, while the E-W direction is controlled by seismic forces, as shown in the table above.

Computer Model

A computer model was developed using RAM Structural System software by Bentley. The model was used to determine frame displacements, stiffness, lateral drifts and to confirm controlling load cases found in Technical Report 1. The composite floor system was modeled as a rigid diaphragm on each floor level.

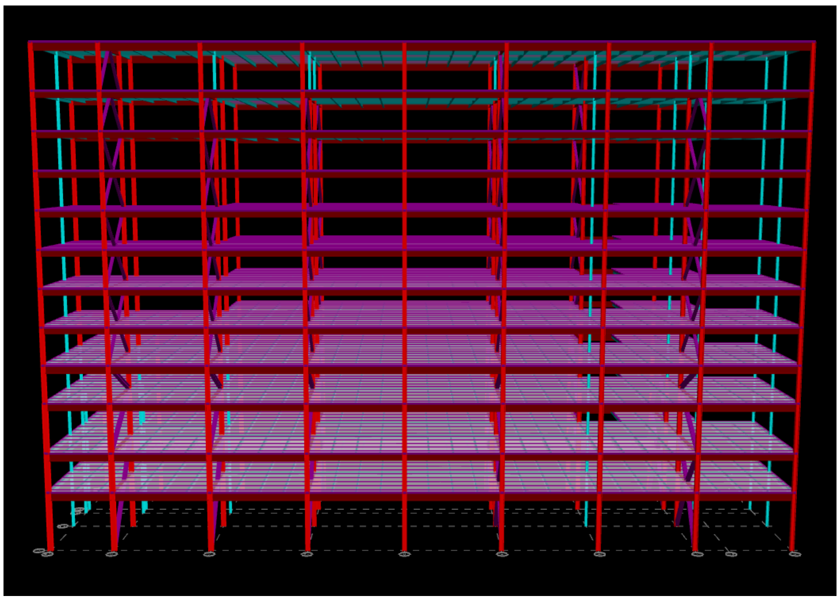


Figure 21 - RAM E-W Frames

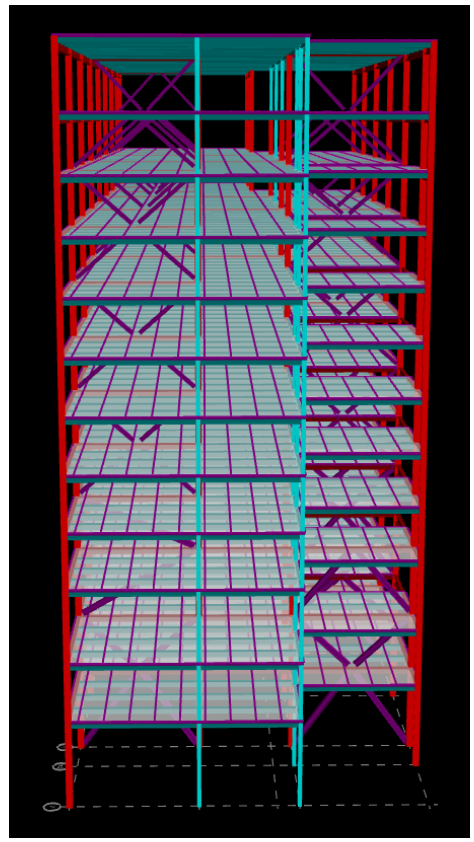


Figure 22 - RAM N-S Frames

Computer Model

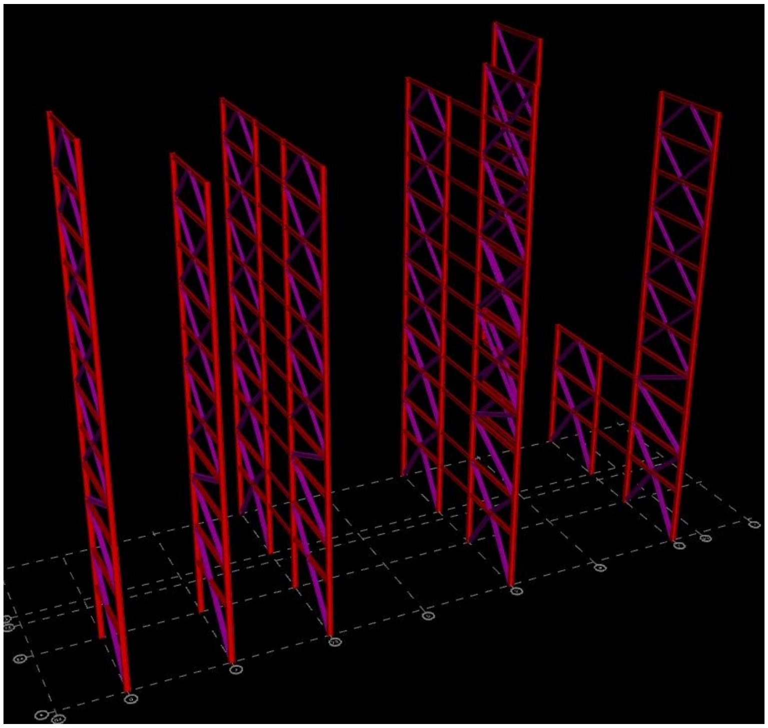


Figure 23 - RAM Braced Frames

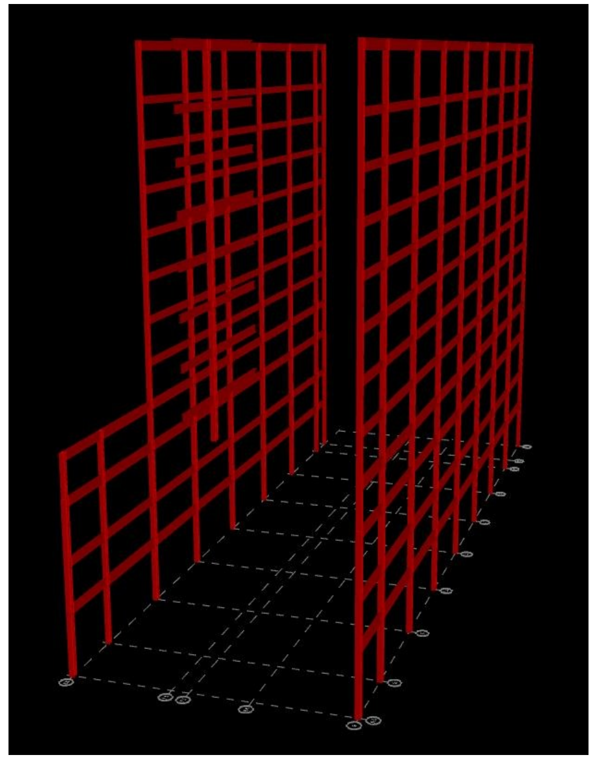


Figure 24 - RAM Moment Frames

All members were modeled as the same sizes as the provided drawings for the SAC Hotel. A section of the first 3 floors are framed into the existing building, but were modeled as a part of the new addition for simplification.

Relative Stiffness

Braced Frames

The SAC Hotel uses two types of lateral framing systems: concentric braced frames in the N-S direction and moment frames along the perimeter in the E-W direction. The first system analyzed using the RAM model was the braced frames. By placing a 1000 kip load at the 11th floor, the displacements at each frame were found. With the deflection, the stiffness of each frame could be found using the equation K=P/δ. The table below shows the relative stiffness for each braced frame, with frame 3 as the stiffest. Figure 26 below shows frame locations and numbering scheme used throughout the rest of this report. All values are for the 10th floor since the majority of braced frames reach only to the 10th floor.

Stiffness of Braced Frames				
Frame	Load at 10th Floor (kip)	10th Floor Displacement (in)	Stiffness (k/in)	Relative Stiffness (k/in)
1	1000	0.25335	3947.1	0.163
2	1000	0.14736	6786.1	0.281
3	1000	0.04138	24166.3	1.000
4	1000	0.17058	5862.4	0.243
5	1000	0.27656	3615.9	0.150
6	1000	0.38254	2614.1	0.108

Figure 25

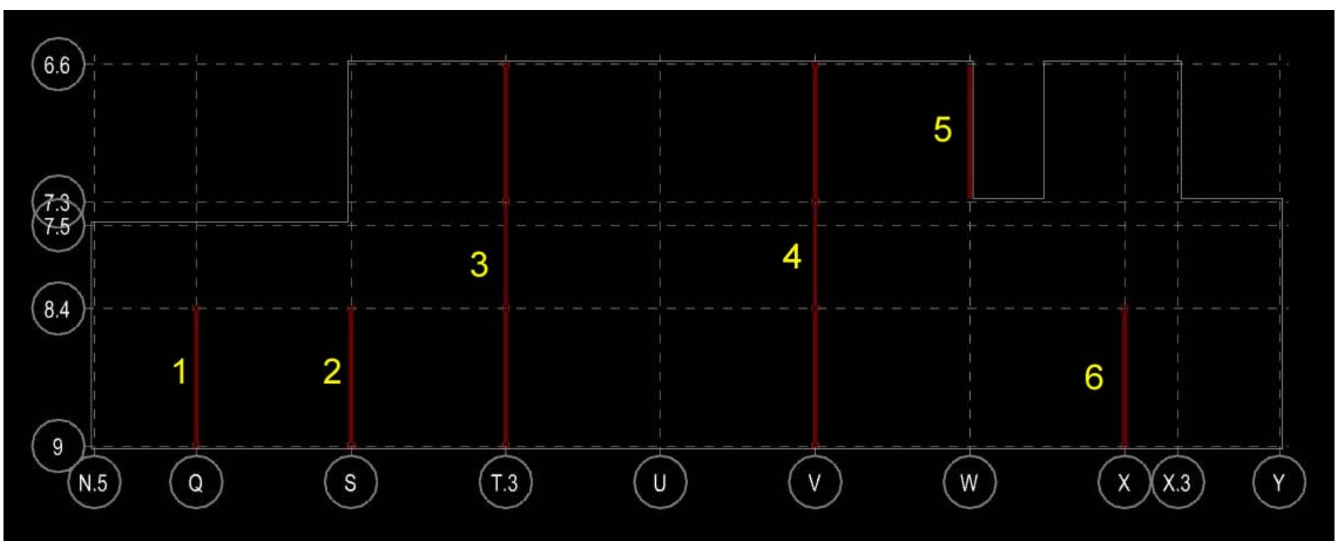


Figure 26 - RAM Braced Frames Plan View

Relative Stiffness

Moment Frames

The same 1000 kip load was applied in the E-W direction for the moment frames, and it was found that both frames had almost the same stiffness due to symmetry. The table and plan view below show the RAM values and frame locations.

Stiffness of Moment Frames				
Frame	Load at 10th Floor (kip)	10th Floor Displacement (in)	Stiffness (k/in)	Relative Stiffness (k/in)
7	1000	12.5	80	0.9792
8	1000	12.24	81.7	1

Figure 27

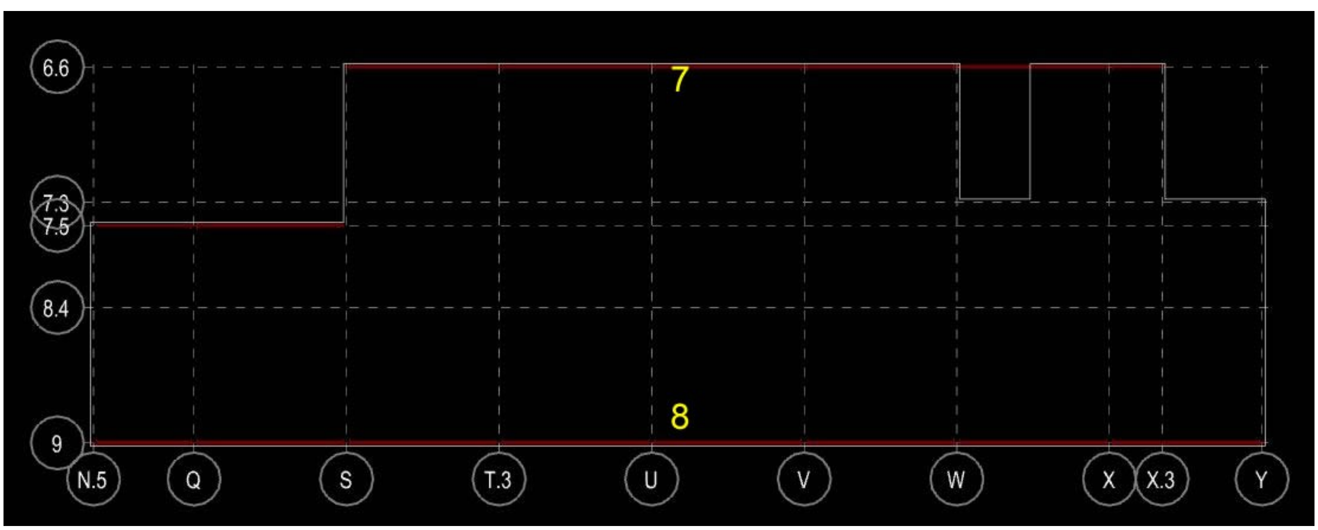


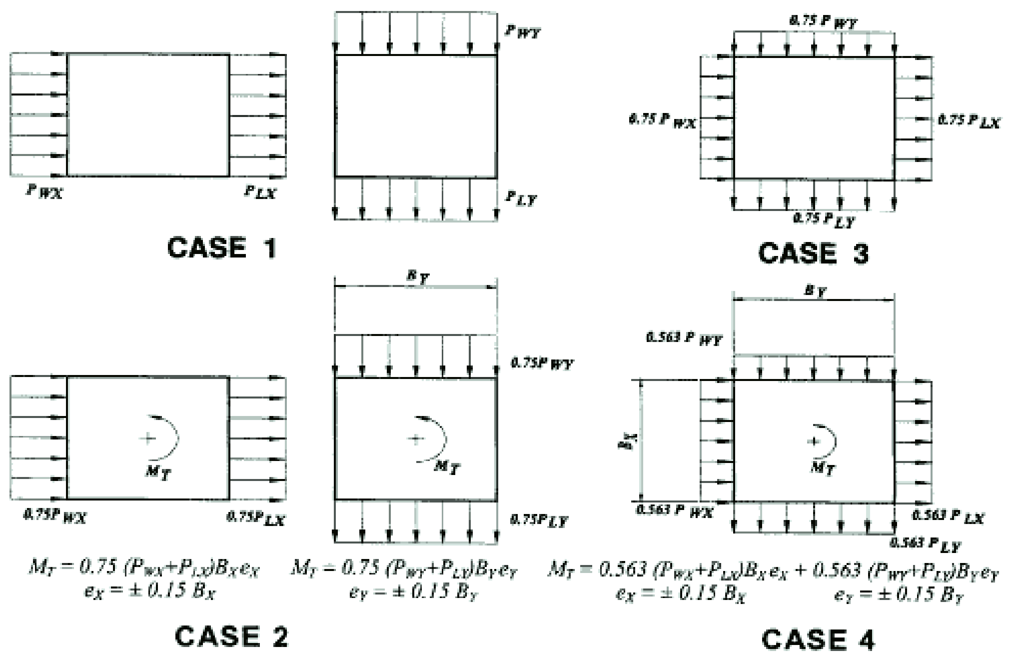
Figure 28 - RAM Moment Frames Plan View

Wind Load Cases

Braced Frames

As previously determined in Technical Report 1, the design of the N-S braced frames are controlled by wind loads. ASCE 7-05 considers four different load cases for wind, shown below in Figure 29, that will produce the worst case scenario. The load case values are shown on the next page.

Main Wind Force Resisting System—Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY} : Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY} : Leeward face design pressure acting in the x, y principal axis, respectively.
   - e (e_X, e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T : Torsional moment per unit height acting about a vertical axis of the building.

Figure 29

Case 1			
X Direction (Total)		Y Direction	
Frame	Force (k)	Frame	Force (k)
1	0.27	1	14.06
2	0.39	2	13.32
3	0.37	3	37.27
4	0.46	4	38.95
5	0.04	5	13.94
6	0.19	6	31.25

Case 2			
X Direction (Total)		Y Direction	
Frame	Force (k)	Frame	Force (k)
1	0.4	1	28.11
2	0.6	2	33.33
3	0.56	3	55.9
4	0.68	4	58.42
5	0.06	5	20.91
6	0.28	6	26.2

Case 3	
Total	
Frame	Force (k)
1	2.22
2	0.75
3	0.52
4	1.65
5	0.61
6	1.31

Case 4	
Total	
Frame	Force (k)
1	3.34
2	1.13
3	0.78
4	2.48
5	0.92
6	1.96

From the RAM results, Case 2 was found to produce the largest loads in the Y direction, which lines up with the wind analysis results from Technical Report 1. The calculations from Tech 1 for wind can be found in Appendix C.

Load Combinations

In the following load distribution, drift, and overturning moment analyses, the following load combinations from ASCE 7-05 were used. The combinations specifically used in calculations were numbers 6 and 7.

2.3.2 Basic Combinations. Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

Chapter 2 ASCE 7-05

Lateral Load Distribution

Direct Shear

From the prior wind load analysis performed in RAM, it was determined that Case 2 controlled the design of the braced frames in the N-S direction. The analysis from RAM also showed that seismic loads controlled in the E-W direction. The direct shear forces were checked in both directions using the ASCE 7-05 load combinations.

Torsional Shear

The SAC Hotel's center of rigidity was found to have an eccentricity in the E-W direction, thus, torsional shear was only accounted for in the N-S braced frames. Calculations for the center of rigidity and sample calculations for the load distributions can be found in Appendix C.

North-South Direction (Wind)									
Frame	Rigidity (K)	Lateral Force (kips)	e_x (ft)	d (ft)	k*d	Direct Shear (k)	Torsional Force Percentage	Torsional Shear (k)	Total Shear (k)
1	3947.1	1532.4	16.8	69.24	273297.2	128.57	0.05450	-83.518	45.05
2	6786.1	1532.4	16.8	40.24	273072.7	221.28	0.05446	-83.4494	137.83
3	24166.3	1532.4	16.8	11.24	271629.2	788.11	0.05417	-83.0083	705.11
4	5862.4	1532.4	16.8	46.76	274125.8	191.24	0.05467	83.77121	275.01
5	3615.9	1532.4	16.8	75.76	273940.6	117.84	0.05463	83.7146	201.56
6	2614.1	1532.4	16.8	104.76	273853.1	85.201	0.05461	83.68787	168.89
									1533.44

$\sum(k*d^2)$=	**84243540**

North-South Direction (Seismic)									
Frame	Rigidity (K)	Lateral Force (kips)	e_x (ft)	d (ft)	k*d	Direct Shear (k)	Torsional Force Percentage	Torsional Shear (k)	Total Shear (k)
1	3947.1	1459.7	16.8	69.24	273297.2	122.47	0.05450	-79.5557	42.91
2	6786.1	1459.7	16.8	40.24	273072.7	210.78	0.05446	-79.4904	131.29
3	24166.3	1459.7	16.8	11.24	271629.2	750.72	0.05417	-79.0702	671.65
4	5862.4	1459.7	16.8	46.76	274125.8	182.17	0.05467	79.79694	261.97
5	3615.9	1459.7	16.8	75.76	273940.6	112.25	0.05463	79.74302	191.99
6	2614.1	1459.7	16.8	104.76	273853.1	81.159	0.05461	79.71755	160.88
									1460.70

Lateral Load Distribution Cont'd

East-West Direction (Wind)			
Frame	Rigidity (K)	Lateral Force (kips)	Direct Shear (k)
7	80	1381	683.595
8	81.7	1381	697.405
			1381

East-West Direction (Seismic)			
Frame	Rigidity (K)	Lateral Force (kips)	Direct Shear (k)
7	80	1405	695.475
8	81.7	1405	709.525
			1405

After including effects of torsion in the N-S direction, it was found that wind loads still controlled the design of the N-S braced frames, while seismic controlled the design of the E-W moment frames.

Drift

The RAM model was used to identify story displacements due to the controlling wind and seismic load cases and then compared to the value of H/400 for wind and .02h for seismic, with H as the height of each story. The tables below show these values.

Story Drift Due to Unfactored Wind Loads						
	Height (in)	Actual Displacement		H/400	Story Drift	
Story		X (in)	Y (in)	(in)	X (in)	Y (in)
Mezz	17	0.23	0.001	0.51	0.23	0.001
2	13	0.37	0.002	0.39	0.14	0.001
3	15	0.47	0.002	0.45	0.10	0.000
4	15	0.57	0.002	0.45	0.10	0.000
5	11.33	0.67	0.003	0.34	0.11	0.001
6	11.33	0.77	0.004	0.34	0.10	0.001
7	11.33	0.86	0.005	0.34	0.09	0.001
8	11.33	0.94	0.007	0.34	0.08	0.001
9	11.33	1.00	0.008	0.34	0.06	0.001
10	11.33	1.05	0.009	0.34	0.05	0.001
11	13.67	1.09	0.012	0.41	0.04	0.003

Story Drift Due to Unfactored Seismic Loads						
	Height (in)	Actual Displacement		.02h	Story Drift	
Story		X (in)	Y (in)	(in)	X (in)	Y (in)
Mezz	17	0.36	0.0003	4.1	0.36	0.0003
2	13	0.61	0.0004	3.1	0.25	0.0001
3	15	0.78	0.001	3.6	0.17	0.0006
4	15	0.98	0.004	3.6	0.19	0.0025
5	11.33	1.20	0.005	2.7	0.22	0.0016
6	11.33	1.42	0.007	2.7	0.22	0.0016
7	11.33	1.63	0.009	2.7	0.21	0.0019
8	11.33	1.83	0.011	2.7	0.20	0.0024
9	11.33	2.01	0.013	2.7	0.18	0.0020
10	11.33	2.15	0.015	2.7	0.14	0.0020
11	13.67	2.27	0.019	3.3	0.12	0.0040

Foundation Stability

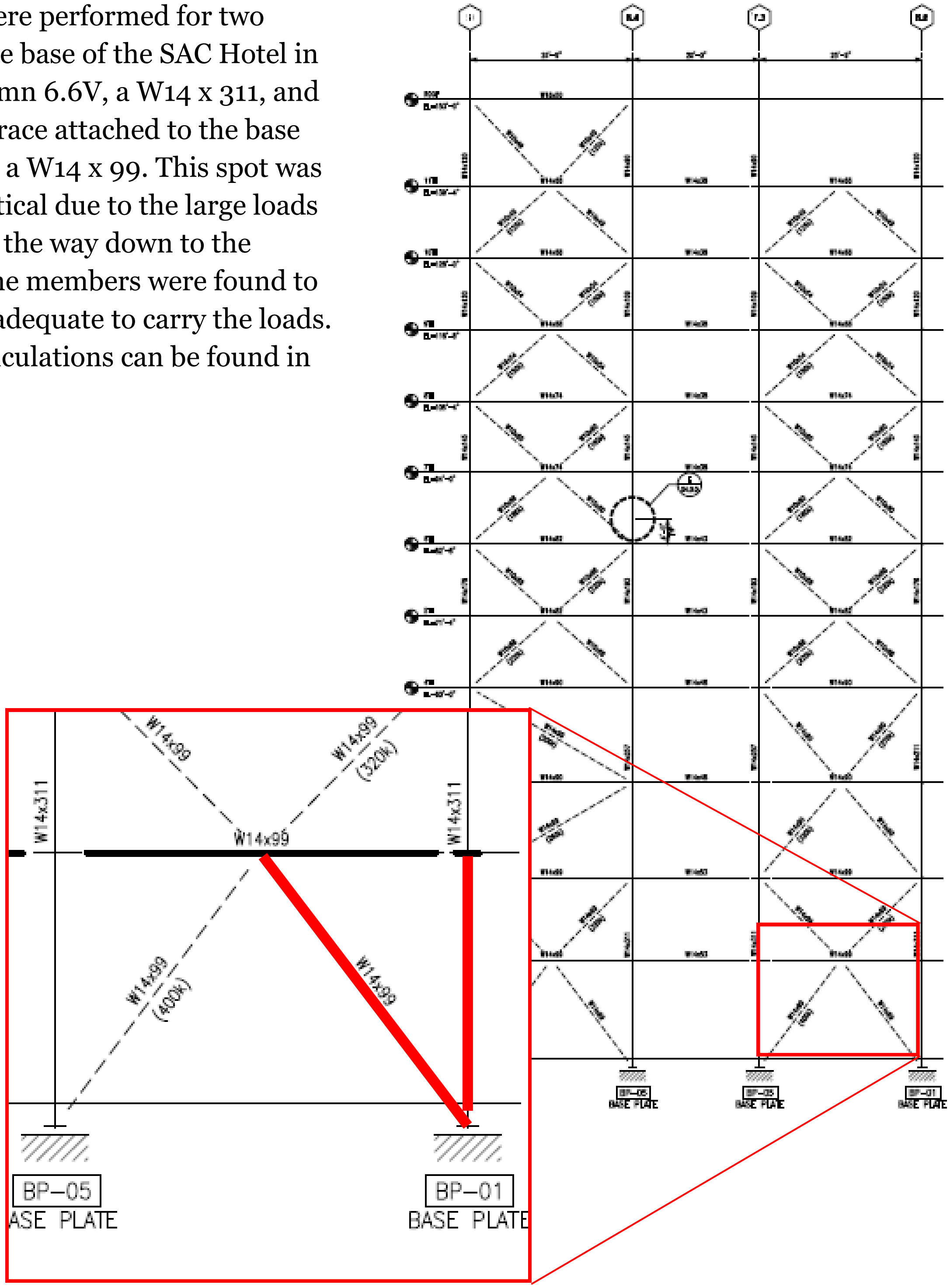
Lateral loads that are applied to buildings can create large moments that must be carried by the foundations. In taller buildings such as the SAC Hotel, foundation design is extremely important. Below is a table that shows the results of the lateral forces in each direction that determined the overturning moments.

Overturning and Resisting Moments					
		N-S Wind		E-W Seismic	
Floor	Height (ft)	Lateral Force (k)	Moment (ft-k)	Lateral Force (k)	Moment (ft -k)
11	139.3	71.47	9957.9	45.07	6279.6
10	128	128.63	16464.6	39.18	5015
9	116.7	115.38	13464.8	34.98	4082.1
8	105.3	114.12	12016.8	30.66	3228.4
7	94	112.5	10575	26.87	2525.7
6	82.7	110.64	9149.9	22.47	1858.2
5	71.3	108.86	7761.7	18.79	1339.7
4	60	106.09	6365.4	13.56	813.6
3	45	104.91	4720.9	11.83	532.3
2	30	115.64	3469.2	10.53	315.9
Mezz	17	114.5	1946.5	4.55	77.3
Overturning Moment			95892.93		26068.28
Resisting Moment			21878330		681912

As stated previously, wind loads controlled design in the N-S direction while seismic controlled in the E-W direction. In order to find the resisting moment, the total load of the building, (found in Technical Report 1 to be approximately 19,000 kips), was multiplied by half the length of the SAC Hotel in each direction. It was found that the resisting moments were much larger than the overturning moments, thus the foundations could adequately carry the lateral forces.

Member Checks

Spot checks were performed for two members at the base of the SAC Hotel in Frame 5: Column 6.6V, a W14 x 311, and the diagonal brace attached to the base of the column, a W14 x 99. This spot was considered critical due to the large loads transferred all the way down to the foundation. The members were found to be more than adequate to carry the loads. Supporting calculations can be found in Appendix C.



Conclusion

With the aid of a computer model of the SAC Hotel in RAM Structural Systems, it was determined that the braced frames and moment frames were adequate in providing lateral resistance from wind and seismic loads. Along with the computer model, Excel spreadsheets were used extensively to simplify complicated hand calculations in order to produce desirable results and to reduce error as much as possible.

The RAM model was used to determine frame stiffness, controlling wind load cases, load distribution due to direct shear and torsional shear, allowable story drift and overturning moments and their impact on the SAC Hotel's foundations. A spot check was performed on a ground level column and brace to determine their individual strengths, which were found to be capable of carrying the applied loads.

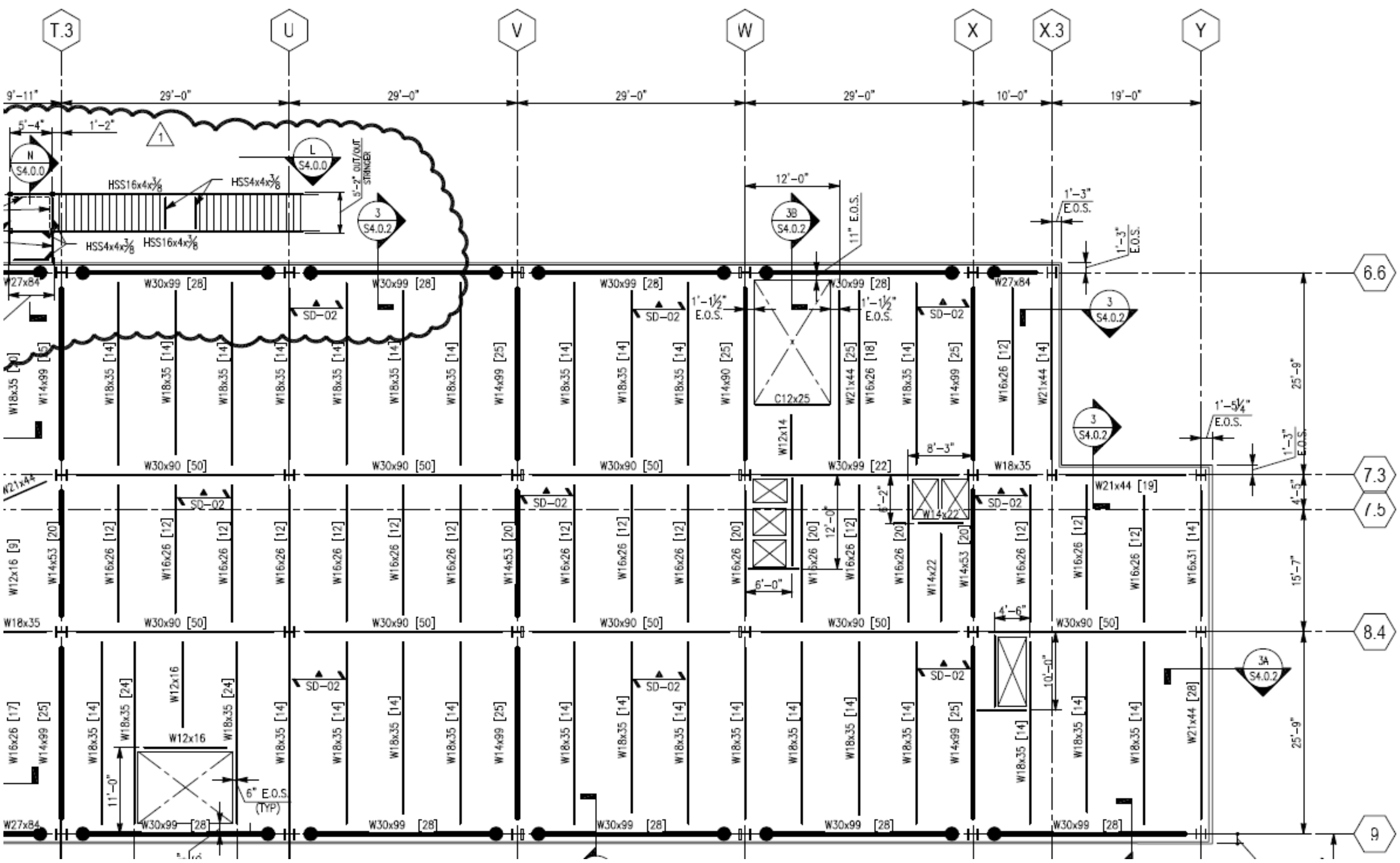
Each analysis was done at the 10th floor of the SAC Hotel since it is the highest floor that is reached by all braced frames. It was determined that Wind Load Case 2 controlled in the N-S direction while seismic loads controlled in the E-W direction, which confirmed the results of Technical Report 1.

Load distribution calculations confirmed that Frame 3 had the highest stiffness in the N-S direction since it carried the most load from wind and seismic forces. Torsional forces were only applicable to the N-S direction due to eccentricity in the E-W direction. The resisting moments were found to be much larger than overturning moments in both directions as well, showing that the foundations were adequately designed. The spot check performed proved that the column and brace selected were also adequately designed, though capacities were much larger than the applied loads. This could be due to the fact that only lateral loads were checked and not the impact of gravity loads.

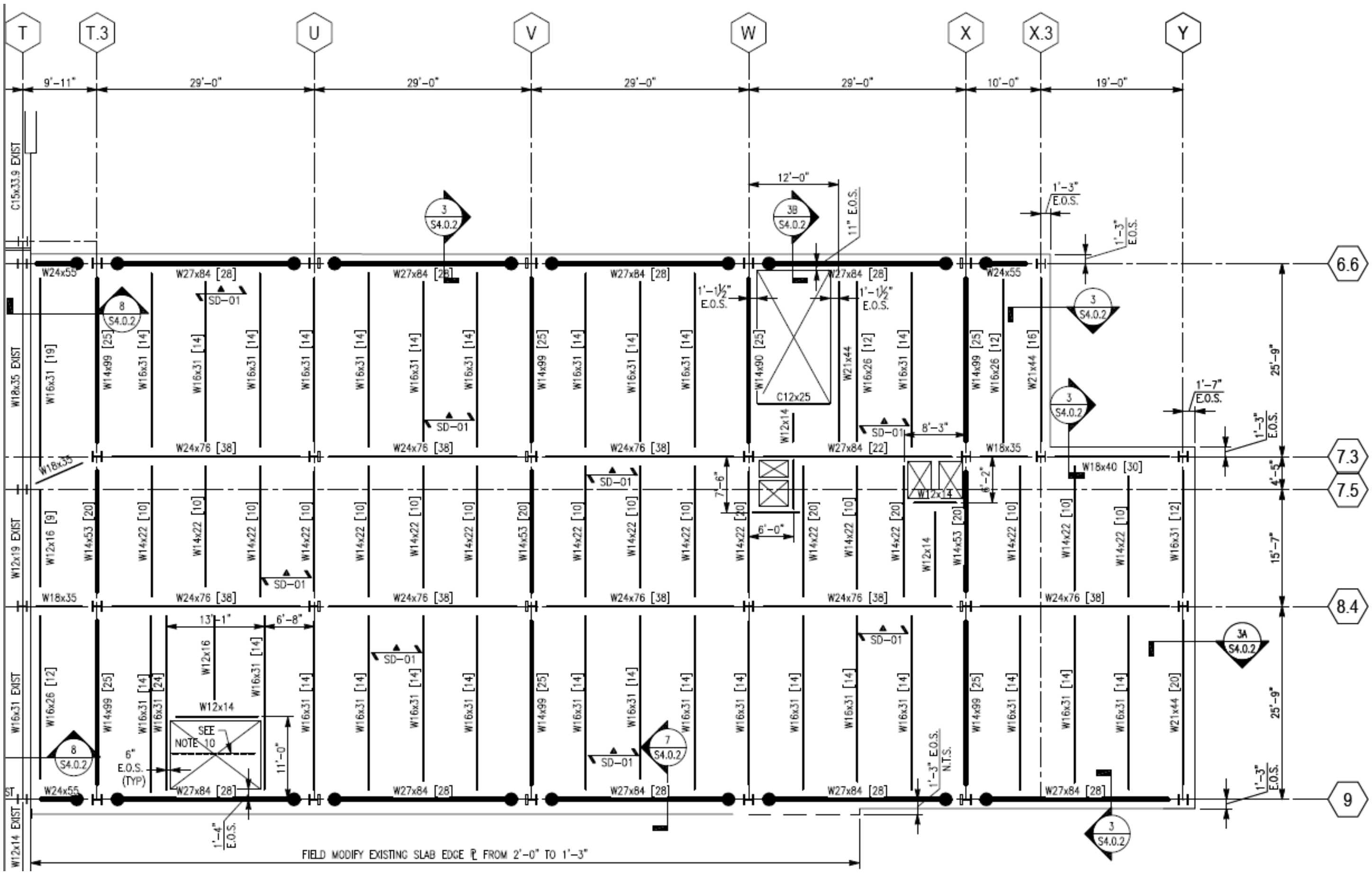
Appendices

Appendix A - Floor Plans

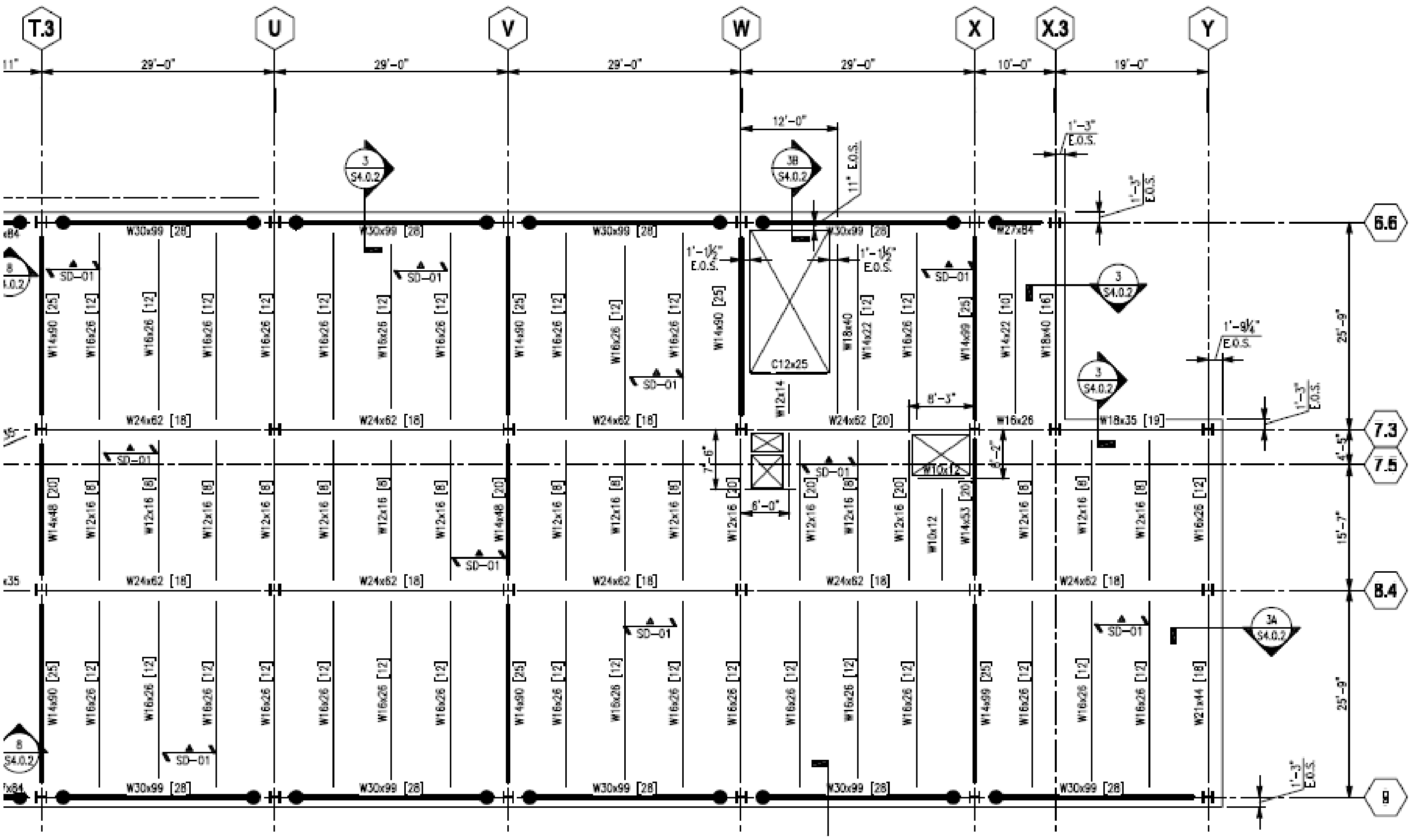
Mezzanine



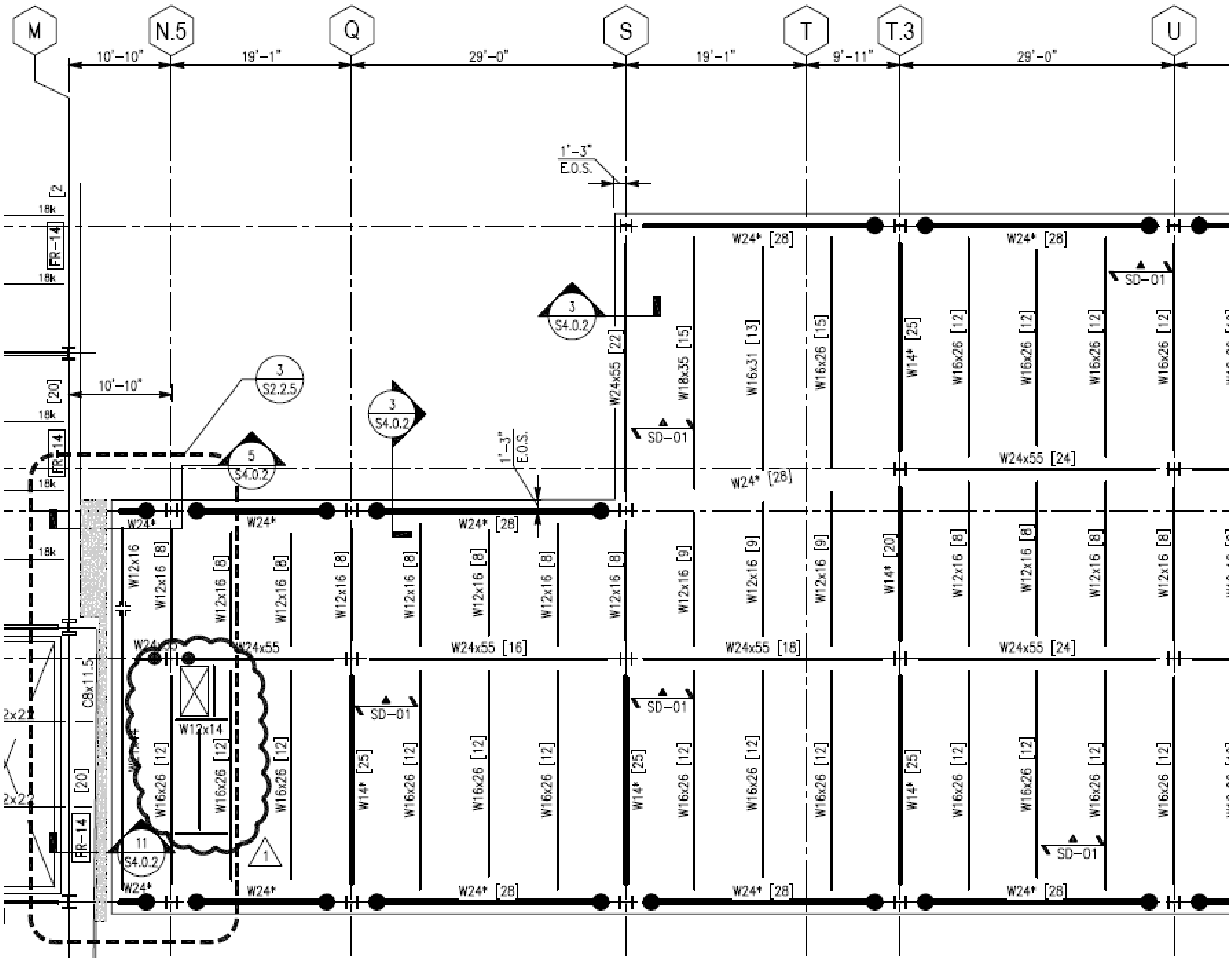
2nd Floor


T
T.3
U
V
W
X
X.3
Y
6.6
7.3
7.5
8.4
9
W27x84 [28]
W24x76 [38]
SD-01
S4.0.2
FIELD MODIFY EXISTING SLAB EDGE ℄ FROM 2'-0" TO 1'-3"

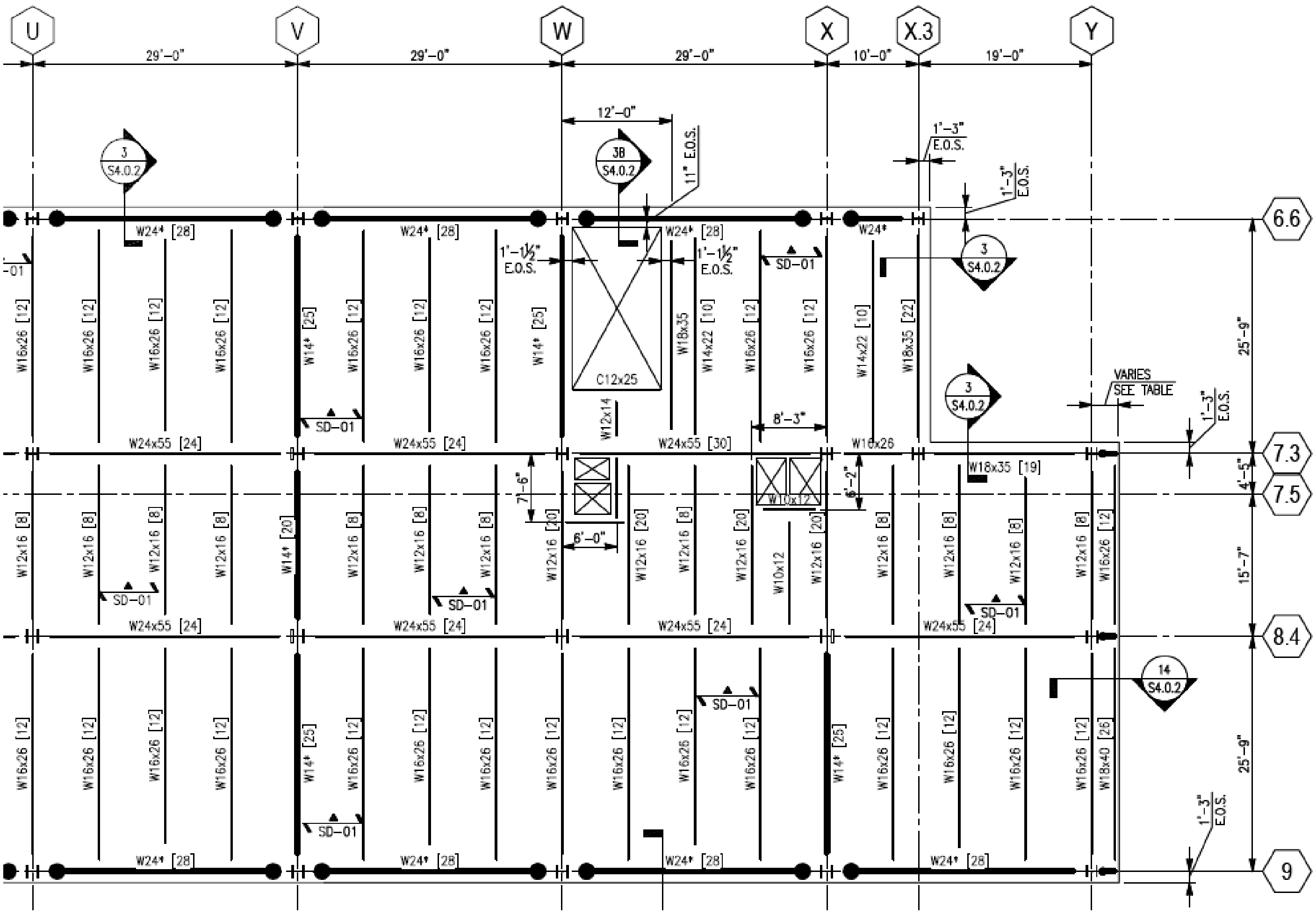
3rd Floor


T.3
U
V
W
X
X.3
Y
29'-0"
29'-0"
29'-0"
29'-0"
10'-0"
19'-0"
12'-0"
6.6
7.3
7.5
8.4
9
25'-9"
15'-7"
25'-9"
W30x99 [28]
W24x62 [18]
SD-01
C12x25
S4.0.2

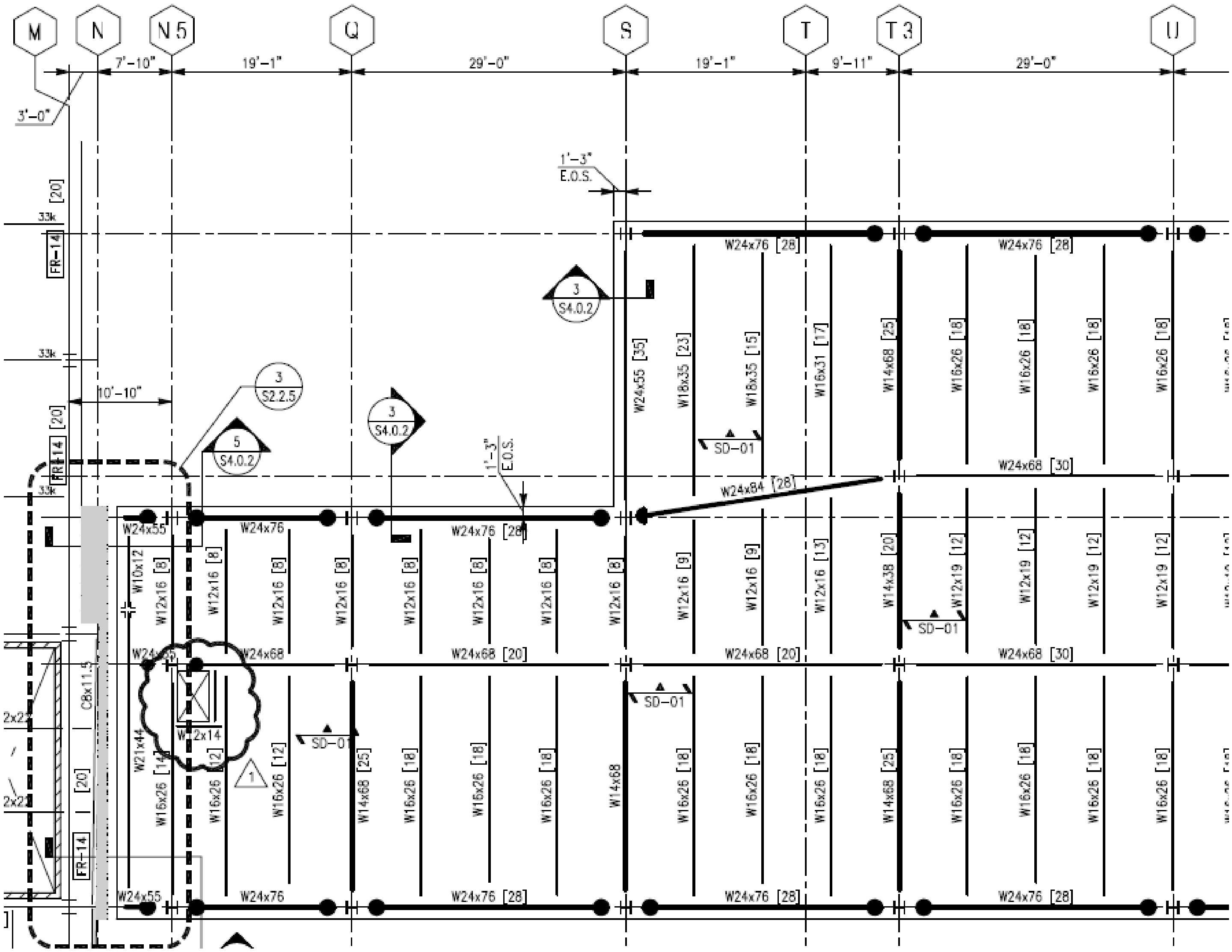
4th thru 10th Floor (Partial)



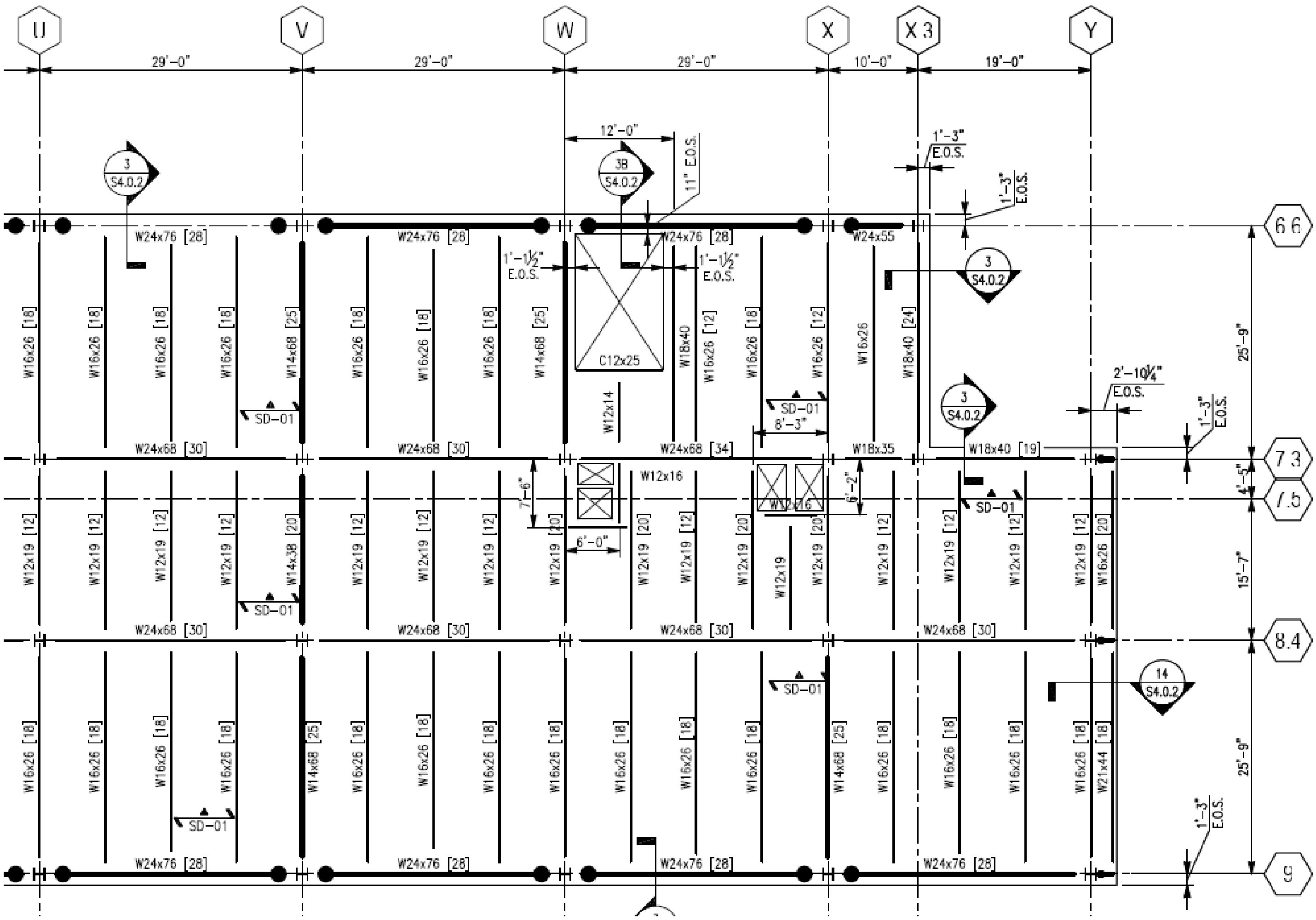
4th thru 10th Floor (Partial)



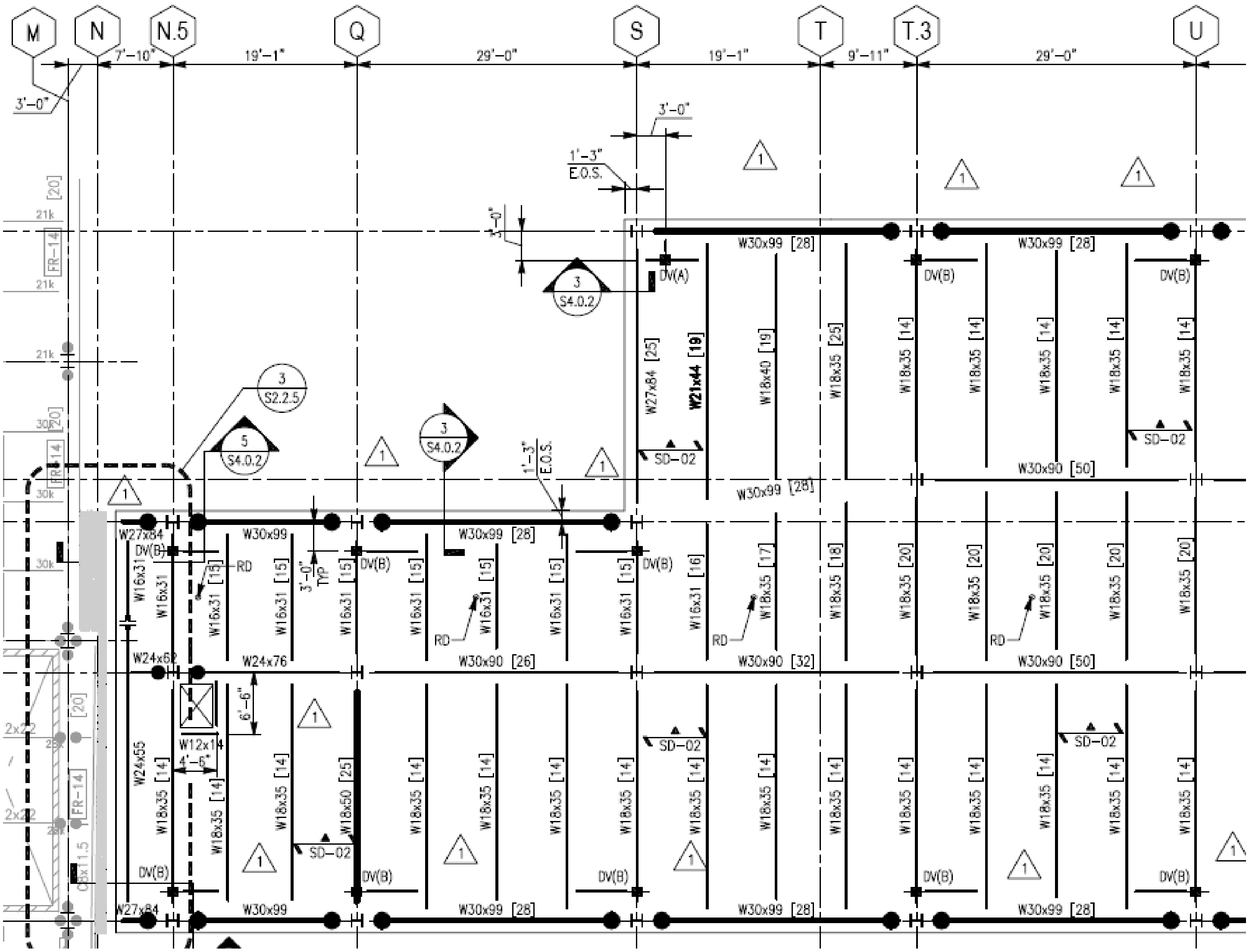
11th Floor (Partial)



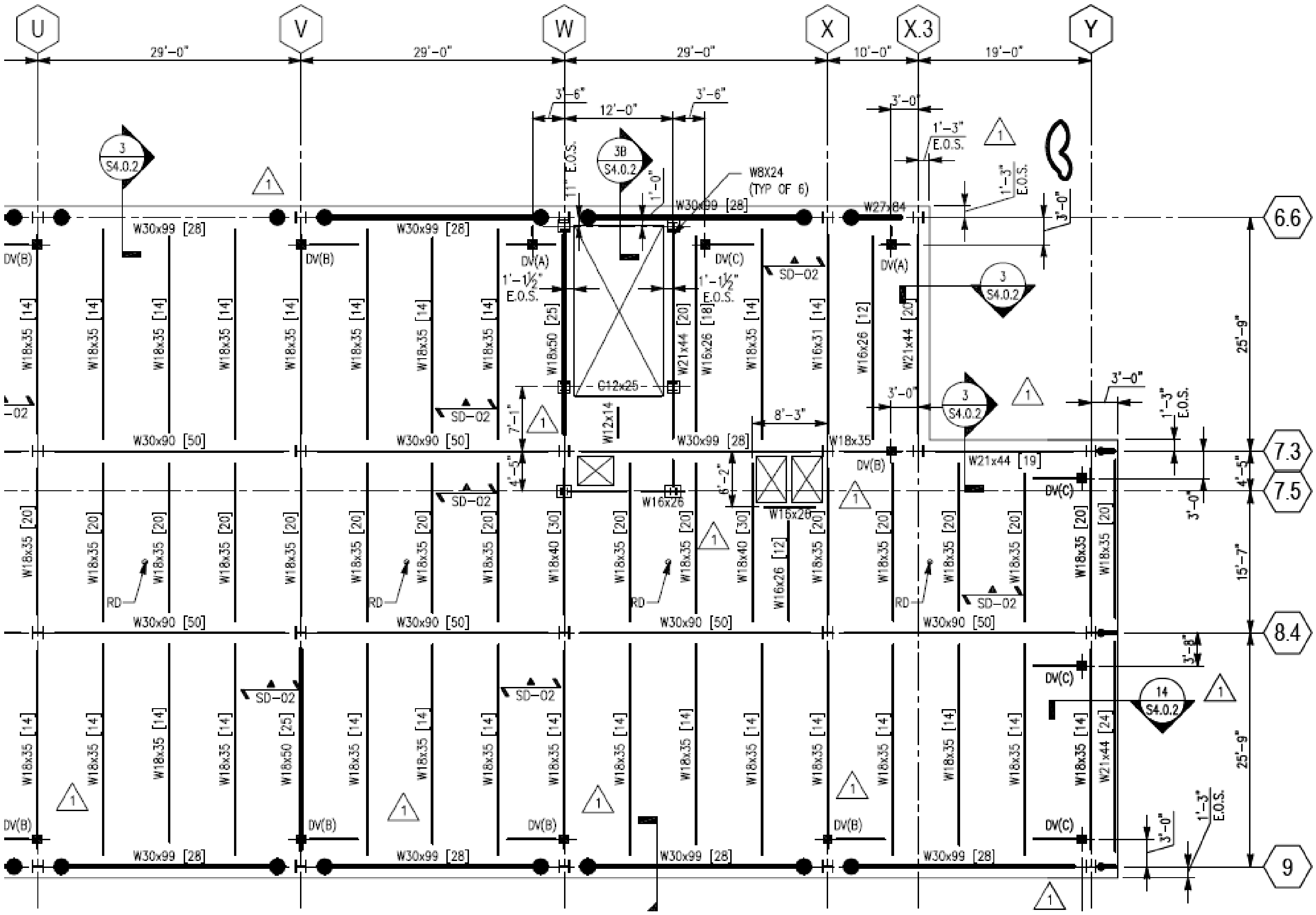
11th Floor (Partial)



Roof (Partial)

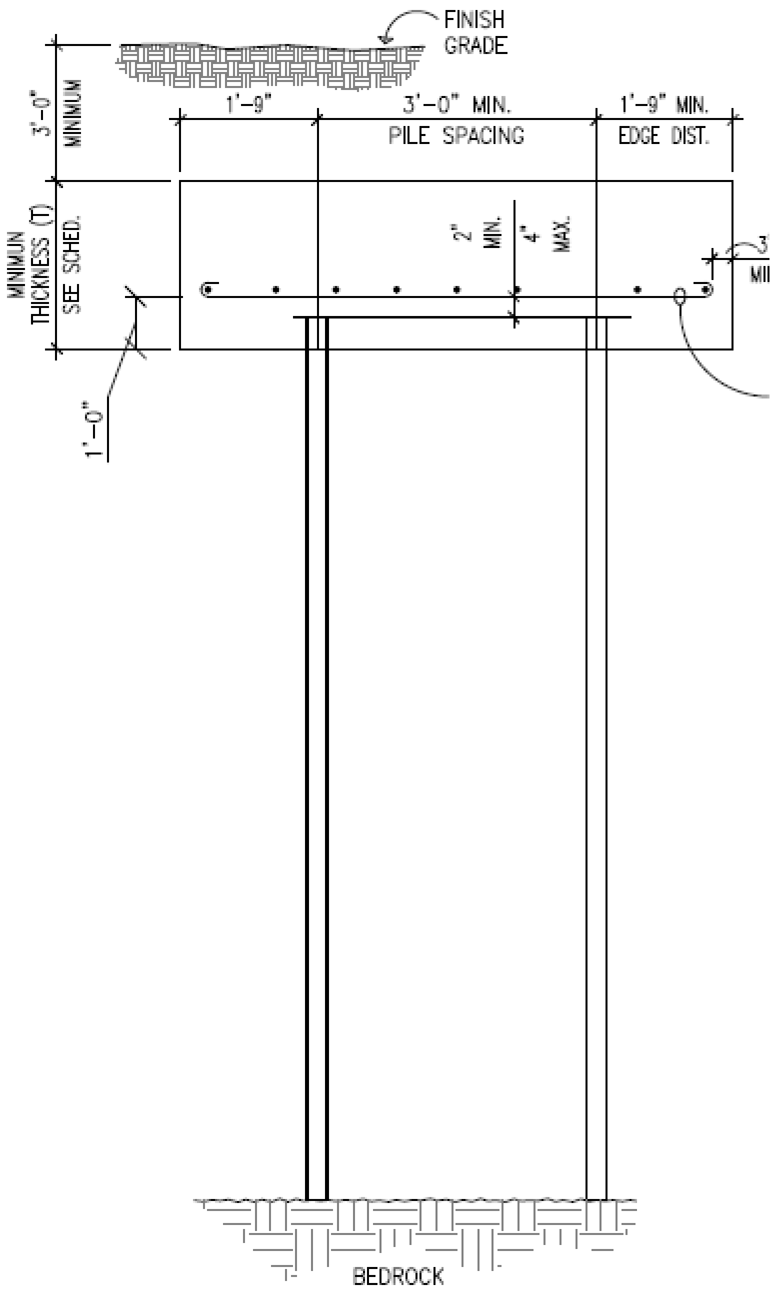


Roof (Partial)

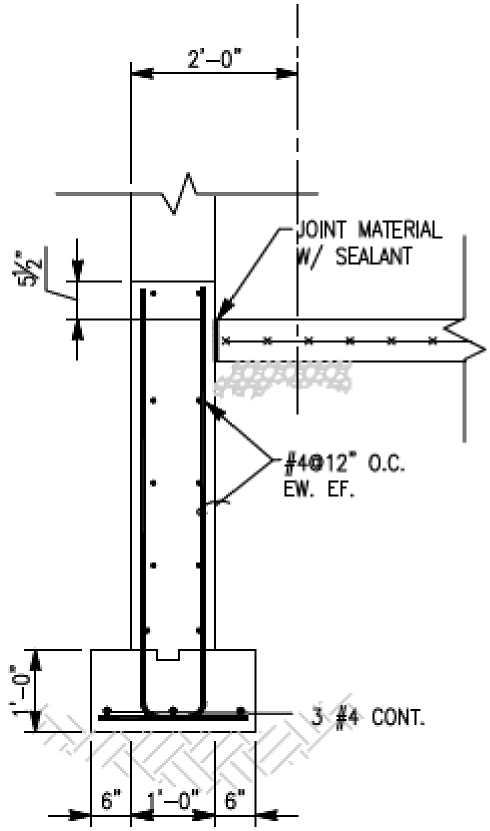


Appendix B - Sections

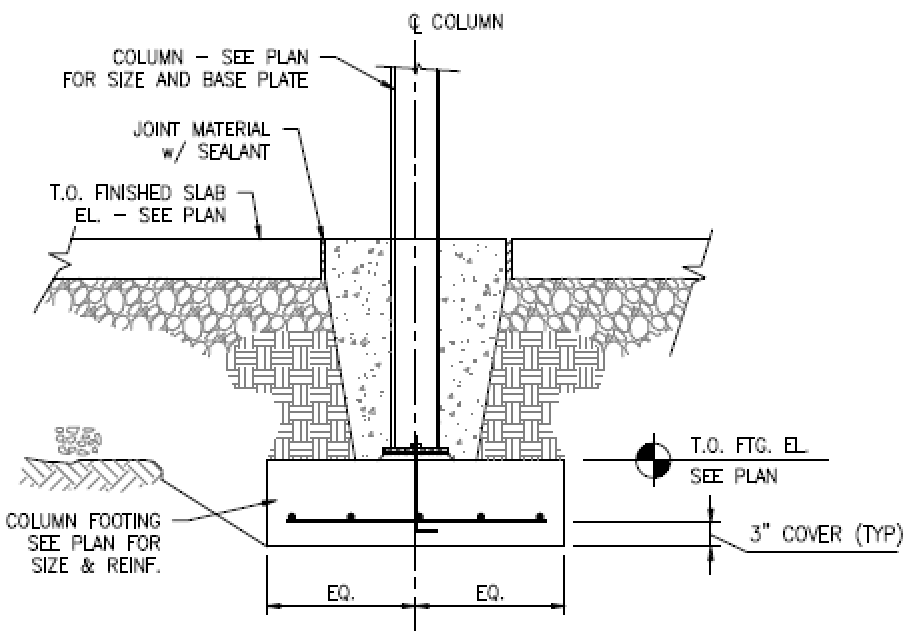
Steel Piles



Footings

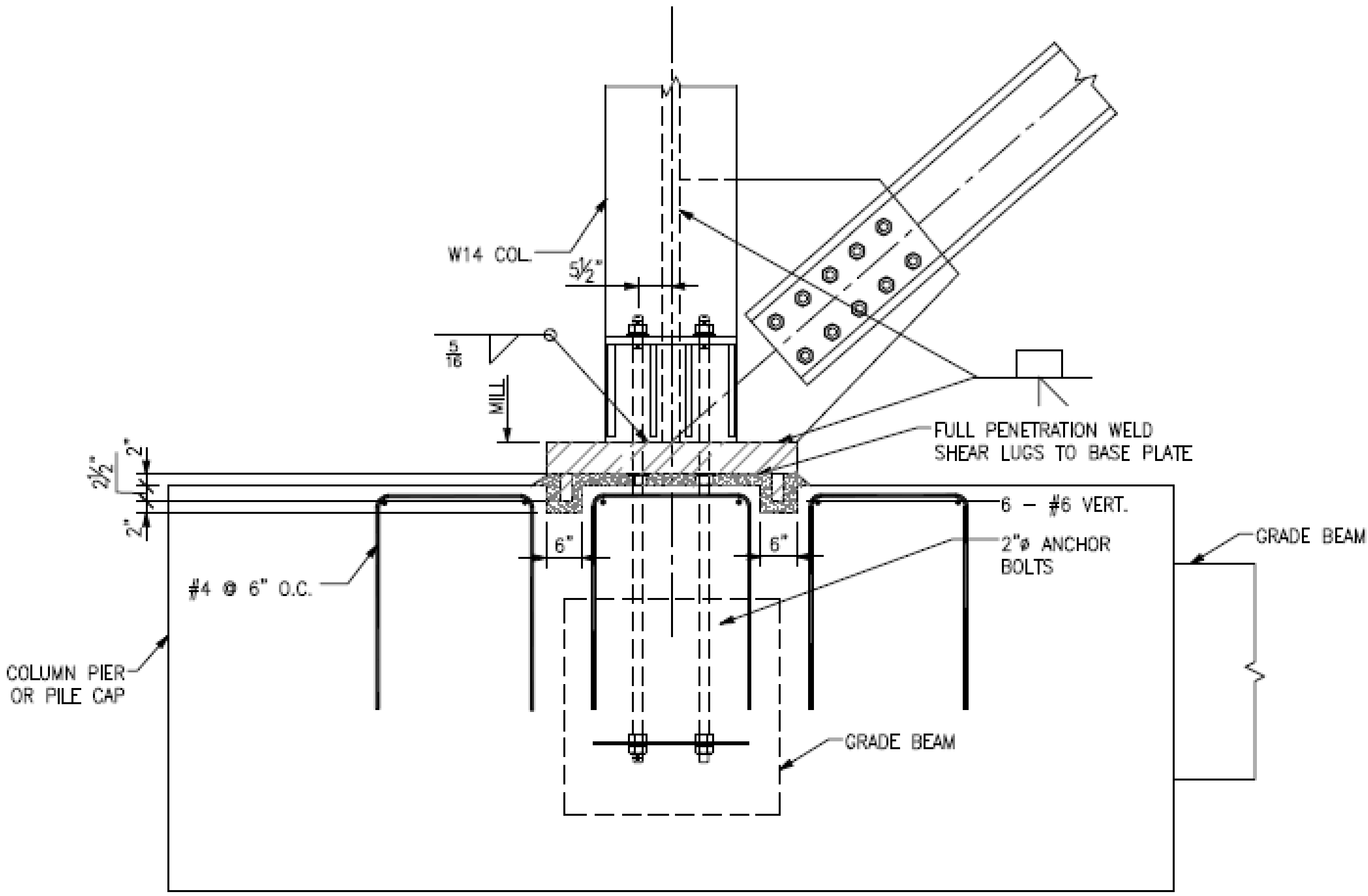


Typical Strip Footing



Typical Interior Footing Without Pier

Brace At Foundation



Nick Reed | Tech Report 1 | Wind Loads 1/5

Location: Salamanca, NY   Occupancy Category III

$\boxed{\text{ASCE 7-05}}$

Exposure C, MWFRS   Total height = 153'

V = 90mph (Fig. 6-1) $K_z = 1.36 + (153 - 140)\left(\frac{1.39 - 1.36}{160 - 140}\right) = 1.38$

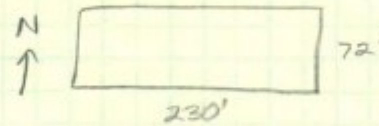
I = 1.15 (Table 6-1)

$K_d = 0.85$ (Table 6-4)   * See attached spreadsheet for q_z calcs *

$K_{zt} = 1.0$

G = 0.85

$GC_{pi} = \pm 0.18$ (Fig. 6-5)



Simplified Shape

Design Wind Pressures for MWFRS

Wall Cp

WW = 0.8   L/B = 2.2 ⇒ LW = −0.3 (N−S)

SW = −0.7   L/B = 0.46 ⇒ = −0.5 (E−W)

Roof Cp

$h/L = 153/155 = 0.99$ $C_p = -1.3$ 0 to h/2   Second $C_p = -0.18$

-0.7 > h/2

Sample Calc for Roof Level (153') ⇒ $q_z = 24.32$

N−S

WW wall: $p = 24.32(.85)(.8) - 24.32(\pm 0.18)$

$p = 16.54 \pm 4.38$ psf

LW wall: $p = 24.32(.85)(-.3) - 24.32(\pm 0.18)$

$p = -6.2 \pm 4.38$ psf wind normal to 230' wall

E−W

WW wall: $p = 24.32(.85)(.8) - 24.32(\pm 0.18)$

$p = 16.54 \pm 4.38$ psf

LW wall: $p = 24.32(.85)(-0.5) - 24.32(\pm 0.18)$

$p = -10.34 \pm 4.38$ psf

Nick Reed | Tech Report 1 | Wind Loads 2/5

* See attached spreadsheet for pressures at each floor *

Story Wind Forces (N-S)

Mezzanine (17'-30')

$$\frac{\left[(15.17\,psf)(3') + (15.65\,psf)(5') + (16.13\,psf)(5')\right](230') + (10.58)(13')(230')}{1000}$$

$= 78.65\ k$

2nd Floor (30'-45')

$[(16.13)(10') + (16.84)(5')](230) + (10.58)(15')(230') = 93\ k$

3rd Floor (45'-60')

$[(16.84)(5') + (17.44)(10')](230') + (10.58)(15')(230') = 96\ k$

4th Floor (60'-71'4")

$[(17.92)(10') + (18.40)(1' + \frac{4}{12})](230') + (10.58)(11' + \frac{4}{12})(230') = 74.4\ k$

5th Floor (71'4"-82'8")

$[(18.40)(8.67') + 18.88(2.67')](230') + (10.58)(11.33')(230') = 75.9\ k$

6th Floor (82'8"-94')

$[(18.88)(7.33') + (19.24)(4')](230') + (10.58)(11.33')(230') = 77.1\ k$

7th Floor (94'-105'4")

$[(19.24)(6') + (19.48)(5.33')](230') + (10.58)(11.33)(230) = 78\ k$

8th Floor (105'4"-116'8")

$(19.48)(11.33')(230') + (10.58)(11.33)(230) = 78.3\ k$

Nick Reed | Tech Report 1 | Wind Loads 3/5

Story Wind Forces (N-S) cont'd

9th Floor (116'8"-128')

$[(19.48)(3.33') + (20.08)(8')](230') + (10.58)(11.33')(230')$
$= 79.4\ k$

10th Floor (128'-139'4")

$(20.08)(11.33')(230) + (10.58)(11.33)(230) = 79.9\ k$

11th Floor (139'4"-153')

$[(20.08)(\frac{8}{12}') + (20.68)(13')](230') + (10.58)(13.67')(230) = 98.2\ k$

Parapet (Slab to top of Parapet, 153'-158')

$K_z = 1.36 + (158-140)\left(\frac{1.39-1.36}{160-140}\right) = 1.39$

$q_z = 0.00256(1.39)(0.85)(90^2) = 24.5\ psf$

$(24.5)(5')(230') + 8.58(5')(230') = 38\ k$

Story Wind Forces (E-W)

Mezzanine (17'-30')

$[(15.17)(3') + (15.65)(5') + (16.13)(5')](72') + (14.72)(13')(72') = 28.5k$

2nd Floor (30'-45')

$[(16.13)(10') + (16.84)(5')](72') + (14.72)(15')(72') = 33.6k$

3rd Floor (45'-60')

$[(16.84)(5') + (17.44)(10')](72') + (14.72)(15')(72') = 34.5k$

4th Floor (60'-71'4")

$[(17.92)(10') + (18.4)(1.33)](72') + (14.72)(11.33')(72) = 26.7k$

5th Floor (71'4"-82'8")

$[(18.40)(8.67') + (18.88)(2.67)](72') + (14.72)(11.33)(72) = 27.1k$

Nick Reed | Tech Report 1 | Wind Loads 4/5

Story Wind Forces (E-W) cont'd

6th Floor (82'8" - 94')

$[(18.88)(7.33) + (19.24)(4')](72) + (14.72)(11.33)(72) = 27.5k$

7th Floor (94' - 105'4")

$[(19.24)(6') + (19.48)(5.33)](72) + (14.72)(11.33)(72') = 27.8k$

8th Floor (105'4" - 116'8")

$(19.48)(11.33)(72) + (14.92)(11.33)(72) = 28.1k$

9th Floor (116'8" - 128')

$[(19.48)(3.33') + (20.08)(8')](72) + (14.72)(11.33)(72) = 28.2k$

10th Floor (128' - 139'4")

$(20.08)(11.33)(72) + (14.92)(11.33)(72) = 28.6k$

11th Floor (139'4" - 153')

$[(20.08)(.67') + (20.68)(13')](72') + (14.72)(13.67')(72) = 34.8k$

Nick Reed | Tech. Report 1 | Wind Loads 5/5

Base Shear, Overturning Moment (N-S)

$V = 78.65 + 93 + 96 + 74.4 + 75.9 + 77.1 + 78 + 78.3 + 79.4 + 79.9 + 98.2$

$= 908.9\ k$

$M = 78.65(17') + 93(30') + 96(45') + 74.4(60') + 75.9(71.33') + 77.1(82.67) + 78(94) + 78.3(105.33') + 79.4(116.67') + 79.9(128) + 98.2(139.33')$

$= 73452\ ft{\cdot}k$

(E-W)

$V = 28.5 + 33.6 + 34.5 + 26.7 + 27.1 + 27.5 + 27.8 + 28.1 + 28.2 + 28.6 + 34.8$

$= 325.4\ k$

$M = 28.5(17') + 33.6(30') + 34.5(45') + 26.7(60') + 27.1(71.33') + 27.5(82.67) + 27.8(94) + 28.1(105.33') + 28.2(116.67) + 28.6(128) + 34.8(139.33)$

$= 14759.4\ ft{\cdot}k$

N-S controls

USGS Design Maps Summary Report

Print   View Detailed Report

User–Specified Input

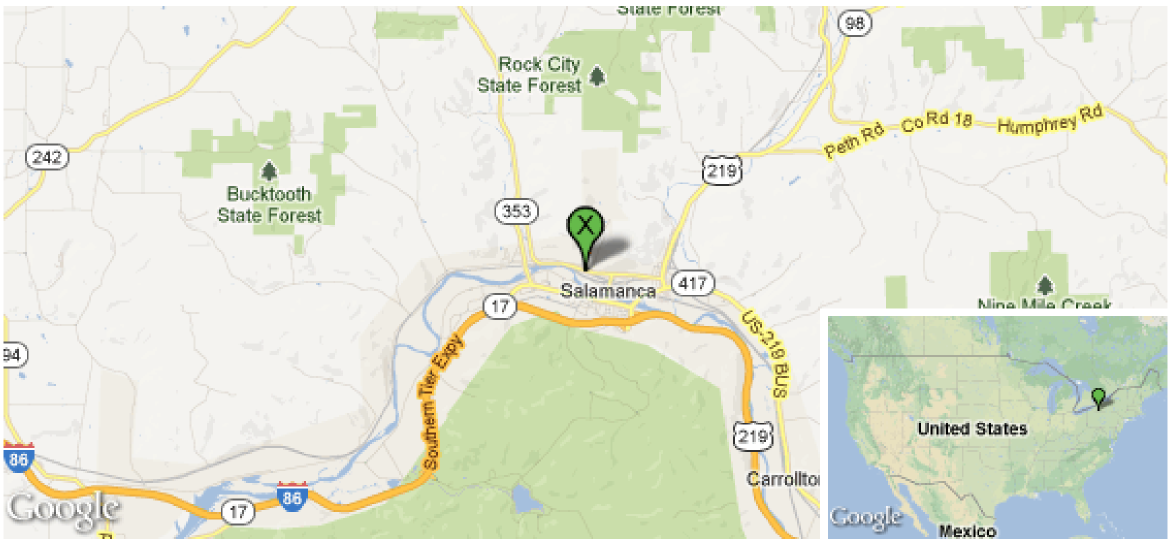
Report Title SAC Hotel
Wed November 7, 2012 17:36:35 UTC

Building Code Reference Document ASCE 7-05 Standard
(which makes use of 2002 USGS hazard data)

Site Coordinates 42.16311°N, 78.72324°W

Site Soil Classification Site Class D – "Stiff Soil"

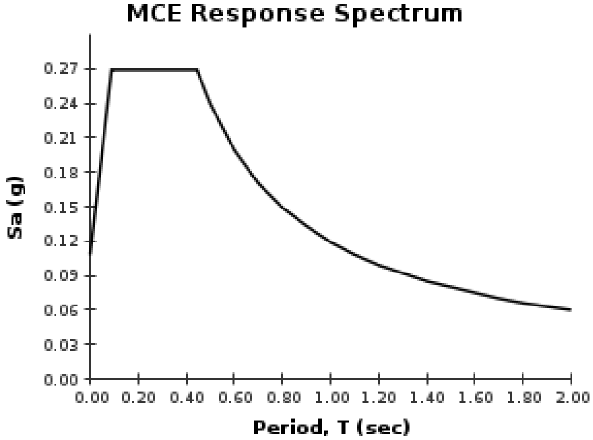
Occupancy Category Occupancy Category III



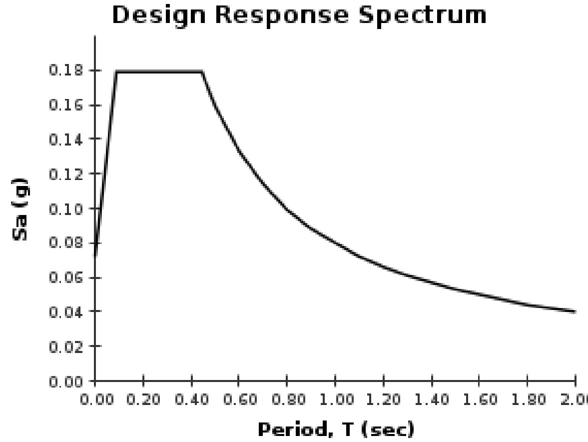
USGS–Provided Output

S_S =	0.168 g	S_{MS} =	0.269 g	S_{DS} =	0.179 g
S_1 =	0.050 g	S_{M1} =	0.119 g	S_{D1} =	0.080 g

MCE Response Spectrum



Design Response Spectrum



Nick Reed | Tech Report 3 | Seismic Revised 1/2

Salamanca, NY - Site Class D

Occupancy Category III

Importance Factor = 1.25

N-S

Seismic Design Category B ⇒ $\rho = 1.0$

$R = 3.25$ From USGS website (printout in Appendix)

$\Omega = 2$

$C_d = 3.25$

$S_s = 0.168$

$S_1 = 0.05$

$T = C_t h_n^x$

$h = 153'$ $C_t = 0.02$ $x = 0.75$ (ASCE 7-05 Table 12.8-2)

$T = .02(153)^{.75} = .87s$ $T_L = 6s$ (Fig 22-15)

$T < T_L = C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)}$ $F_v = 2.4$ (Table 11.4-2)

$S_{D1} = \frac{2}{3} F_v S_1$

$= \frac{2}{3}(2.4)(.05)$

$S_{D1} = .08$ = USGS website value

$C_s = \frac{.08}{.87\left(\frac{3.25}{1.25}\right)} = .035 > .01$ OK

$S_{DS} = \frac{2}{3} F_a S_s$ $F_a = 1.6$ (Table 11.4-1)

$= \frac{2}{3}(1.6)(.168)$

$S_{DS} = .179$ = USGS website value

$C_s = \frac{.179}{(8/1.25)} = .028 > 0.1$ OK

$C_s = .028$ in N-S dir.

Nick Reed | Tech Report 3 | Seismic Revised 2/2

E-W (Moment Frames)

$R = 3.5$ (Table 12.2-1)

$I = 1.25$

$\rho = 0$

$C_t = .028 \quad x = 0.8$ (Table 12.8-2)

$T = .028(153)^{.8} = 1.57s$

$T < T_L$

$S_{D1} = \frac{2}{3}(2.4)(.05) = .08$

$C_s = \frac{.08}{1.57\left(\frac{3.5}{1.25}\right)} = .02 > .01$ OK

$S_{DS} = \frac{2}{3}(1.6)(.168) = .18$

$C_s = \frac{.18}{(3.5/1.25)} = .06 > .01$ OK

$C_s = .02$ in E-W

Nick Reed | Tech Report 1 | Seismic Loads 3/4

Total weight

Roof:

Mech. Suspended From ceiling = (15 psf)(16560 ft²) = 248.4 k

Deck = (69 psf)(16560 ft²) = 1142.6 k

Beams = (35 plf/30')(16560 ft²) = 19.3 k

Parapet = (10 plf)(532') = 5.3 k

Total = 1396.3 k

4th – 11th Floor:

SDL = (15 psf)(16560 ft²) = 248.4 k

Deck = 1142.6 k

Beams = (26 plf/30')(16560) = 14.4 k

Facade = (10 psf)(230')(2') + 10 psf(72')(1') = 5.3 k

Total = (248.4 + 1142.6 + 14.4 + 5.3)(8 Floors) = 11285.6 k

3rd Floor:

SDL = 248.4 k

Deck = 1142.6 k

Beams = (26 plf/30')(16560 ft²) = 14.4 k

Facade = 20 psf(230')(2') + 20 psf(72') = 10.64 k

Total = 1416 k

2nd Floor:

SDL = 248.4 k

Deck = 1142.6 k

Beams = (31 plf/30)(16560 ft²) = 17.11 k

Facade = 10.64 k

Total = 1418.8 k

Mezzanine:

SDL = 248.4 k

Deck = 1142.6 k

Beams = (35 plf/30)(11160 ft²) = 13 k

Facade = 10.64 k

Total = 1414.6 k

Nick Reed | Tech Report 1 | Seismic Loads 4/4

Columns: Average of 180 plf, Average height of 12'
35 cols per Floor

$(180\text{ plf})(12')(11) = 23.8\text{ k}$

Partitions:

$(10\text{ psf})(16560\text{ ft}^2)(12\text{ Floors}) = 1987\text{ k}$

Building Total = 18942 k

N-S

$V = C_sW = .028(18942) = 530.4\text{ k}$

E-W

$V = C_sW = .02(18942) = 378.8\text{ k}$

See spreadsheets for story forces and overturning moments

Overturning Moments

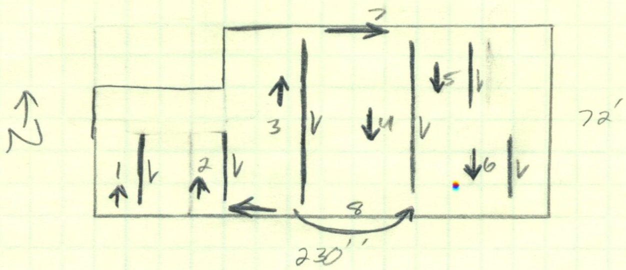
N-S: $3.57(17') + 6.77(30') + 10.64(45') + 14.64(60') + 17.77(71.33') + 20.96(82.67') + 24.2(94') + 27.49(105.33') + 30.83(116.67') + 34.2(128') + 37.61(139.33) + 36.95(153')$

$= 28{,}660\text{ ft·k}$

E-W: $5.11(17') + 9.67(30') + 15.2(45') + 20.91(60') + 25.38(71.33') + 29.94(82.67') + 34.57(94') + 39.27(105.33') + 44.04(116.67) + 48.85(128') + 53.72(139.33') + 52.77(153')$

=

Nick Reed | Tech Report 3 | Center of rigidity + Shears



$K_1 = 3947.1$
$K_2 = 6786.1$
$K_3 = 24166.3$
$K_4 = 5862.4$
$K_5 = 3615.9$
$K_6 = 2614.1$
$K_7 = 80$
$K_8 = 81.7$

$$x_r = \frac{3947.1(30') + 6786.1(59') + 24166.3(88') + 5862.4(146') + 3615.9(175') + 2614.1(200')}{3947.1 + 6786.1 + 24166.3 + 5862.4 + 3615.9 + 2614.1}$$

$x_r = 98.24'$

$y_r = 36'$ $\quad e_x = 115 - 98.24 = 16.8'$

$e_y = 0$

$x_m = 115'$

$y_m = 36'$

Direct Shear (N-S)

$$F_1 = \frac{K_1}{K_1 + \dots + K_6} \cdot F = \frac{3947.1}{46991.9} \cdot F = .0839F \downarrow$$

$$F_2 = \frac{K_2}{46991.9} \cdot F = 0.1444F \downarrow$$

$$F_3 = \frac{K_3}{46991.9} \cdot F = 0.5143\,F \downarrow$$

$$F_4 = \frac{K_4}{46991.9} \cdot F = 0.1248\,F \downarrow$$

$$F_5 = \frac{K_5}{46991.9} \cdot F = 0.07695F \downarrow$$

$$F_6 = \frac{K_6}{46991.9} \cdot F = .05563F \downarrow$$

(E-W)

$$F_7 = \frac{K_7}{161.7} = 0.495\,\underset{\rightarrow}{F}$$

$$F_8 = \frac{K_8}{161.7} = 0.505\,\underset{\rightarrow}{F}$$

Nick Reed | Tech Report 3 | CR + Shears

Torsional

$$\Sigma(kd^2) = [k_1(98.24-29')^2] + [k_2(98.24-58')] + [k_3(98.24-87')] + [k_4(145'-98.24')] + [k_5(174'-98.24)] + [k_6(203'-98.24')]$$

$$= 3947.1(4794.18) + 6786.1(1619.3) + 24166.3(126.3) + 5862.4(2186.5) + 3615.9(5739.6) + 2614(10974.7)$$

$$= 84243540$$

Sample Calc

$$F_{v_1} = \frac{F e_x (d_1 k_1)}{\Sigma(kd^2)} = \frac{F(16.8)(69.24)(3947.1)}{84243540} = .0545F \uparrow$$

See spreadsheet for other forces and forces due to seismic

Nick Reed | Tech Report 3 | Spot Checks

Column (Frame 5)

W14×311

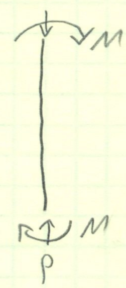
$L_b = 17'$

From RAM model

$P = 957\text{k}$ (From Tech 1)

$M_{rx} = 13.37$

$M_{ry} = 15.02$



AISC 14

W14×311

$p \times 10^3 = 0.289$

$b_x \times 10^3 = 0.396$

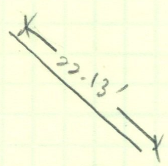
$b_y \times 10^3 = 0.78$

$p P_r + b_x M_{rx} + b_y M_{ry} \leq 1.0$

$0.289 \times 10^{-3}(957) + .396 \times 10^{-3}(13.37) + 0.78(15.02) \leq 1.0$

$0.294 \leq 1.0$ OK

Diagonal Brace (Frame 5)



From structural drawings

$P = 400\text{k}$

W14×99

Compression: $\phi P_n = 904\text{k} > 400\text{k}$ OK

Tension: $\phi P_n = 1310\text{k}$ (Yielding) $> 400\text{k}$

$\phi P_n = 1060\text{k}$ (Rupture) $> 400\text{k}$ OK