Technical Report 1

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 was to analyze the existing structure of the Seneca Allegany Casino Hotel addition, starting with a general overview of the structure as a whole. By explaining the foundation, floor, lateral, framing and roof system, a better understanding was gained of how loads are distributed throughout the structure. Included in the analysis are multiple figures to provide a better understanding of the systems being described, as well as appendices for more detailed plans and calculations.

The figures and specifications included in this report are from the full set of drawings courtesy of JCJ Architecture. Differing editions of AISC and IBC codes were used, but the lateral analysis in the actual design and in this report made use of ASCE 7-05. Through calculations of wind and seismic, it was concluded that wind controls the lateral design, with the N-S direction controlling the overall wind design. The SAC Hotel uses moment frames in the E-W direction, and concentrically braced frames in the N-S direction.

Spot checks of a typical 29'x25.75' bay were conducted to check sizing of a beam, girder and column member. The floors are framed with composite metal deck. These spot checks were done using LRFD, while it was known that the SAC Hotel addition was designed with ASD. It was found that member sizes greatly differed in the girder check, since LRFD takes larger loads into account. Composite sections were also checked with LRFD, and it was found that the number of required shear studs was double what is shown in framing plans. Columns were found to be overdesigned with LRFD, but moment connections and construction loads were not considered, thus it was determined that the columns were sized accordingly.

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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 the added load of 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

Figure 3 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 sub-grade with an allowable bearing capacity of 2000 psf. A section of a typical pile can be found in Appendix C.

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 C.

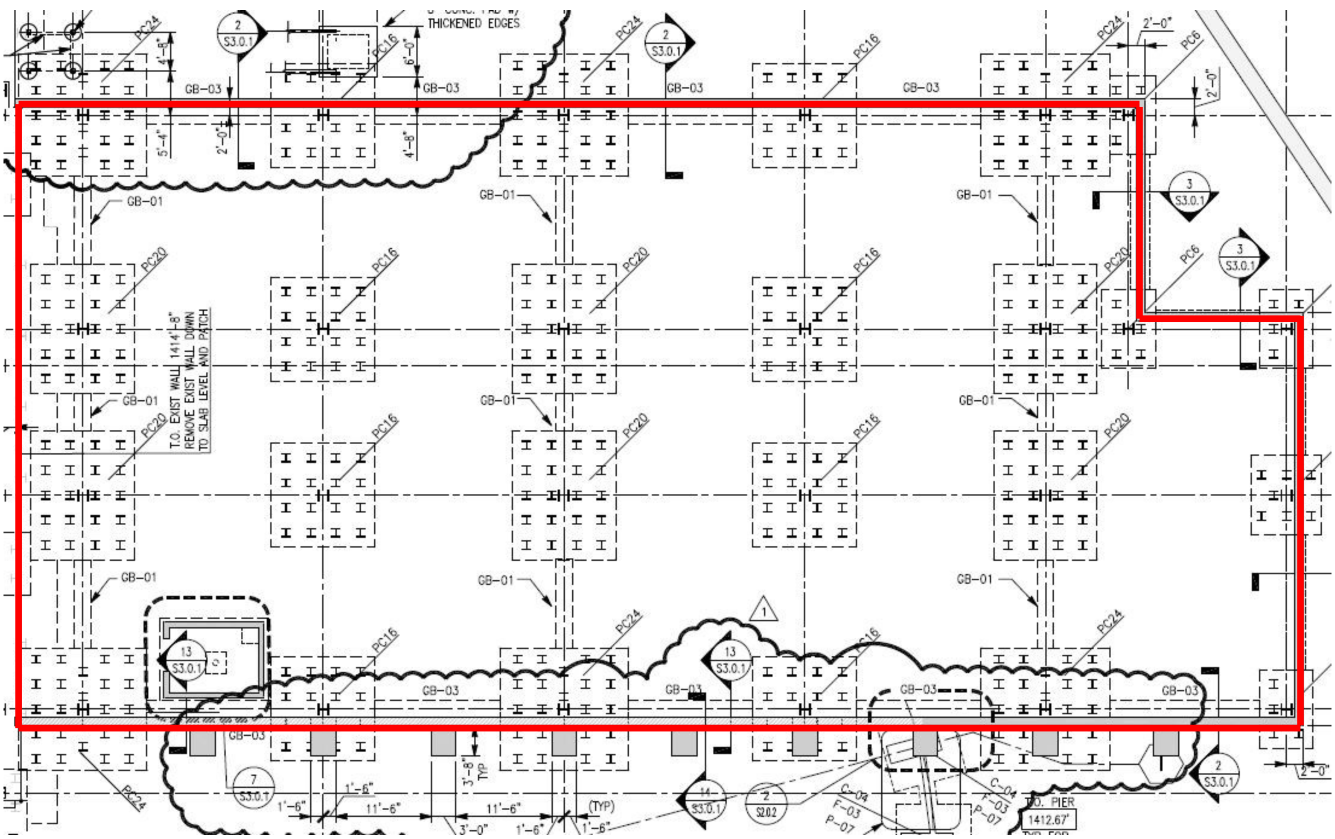


Figure 3 - Steel Pile/Pile Caps Plan
Drawings Courtesy of JCJ Architecture

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 Figure 4. The middle section is the corridor, with a slightly smaller bay size of 15'7" 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. 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, 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). Figure 5 shows a typical deck section.

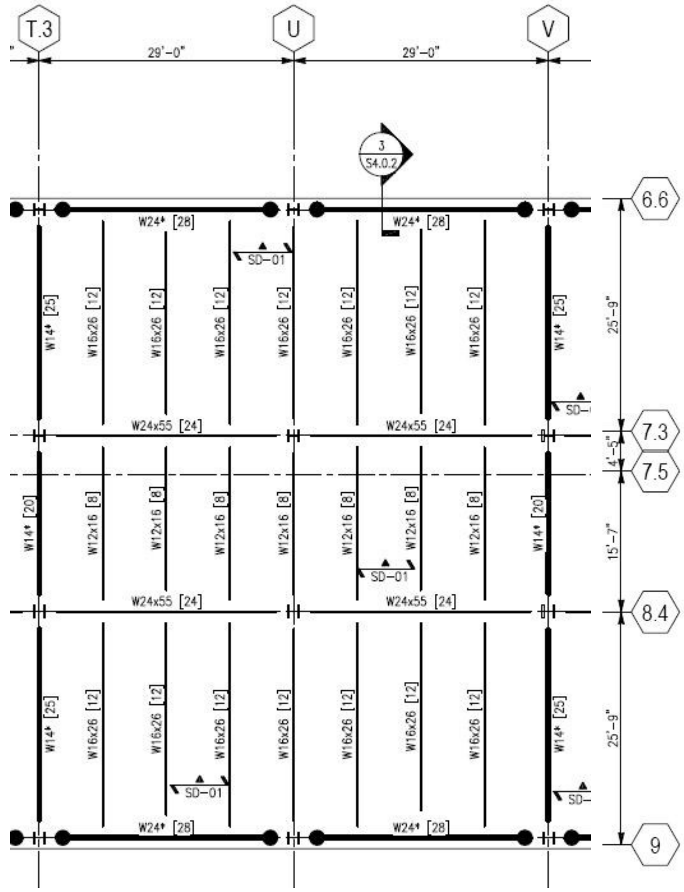


Figure 4 - Section of 4th—10th Floor Framing Plan
Drawings Courtesy of JCJ Architecture

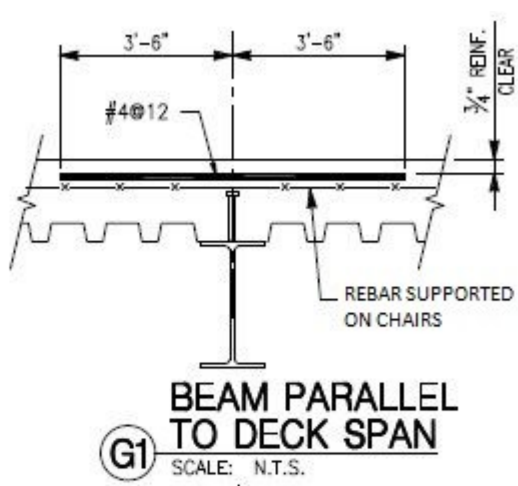


Figure 5 - Typical Composite Metal Deck Section
Drawings Courtesy of JCJ Architecture

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. Column elevations can be found in Appendix B.

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 PL 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 Appendix B.

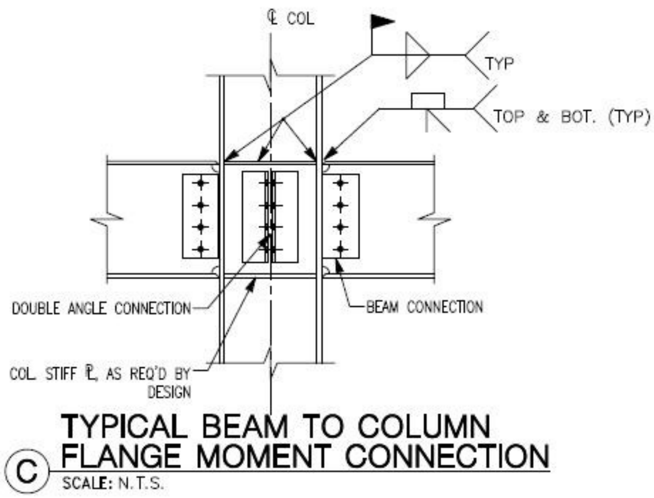


Figure 7 - Typical Moment Connection
Drawings Courtesy of JCJ Architecture

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 C.

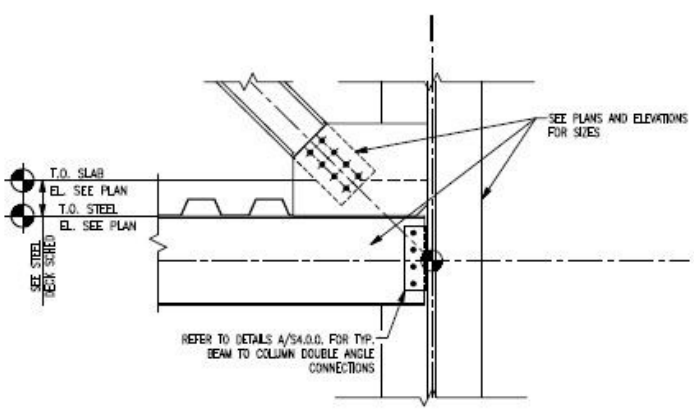


Figure 9 - Diagonal Brace Connection at Column
Drawings Courtesy of JCJ Architecture

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.

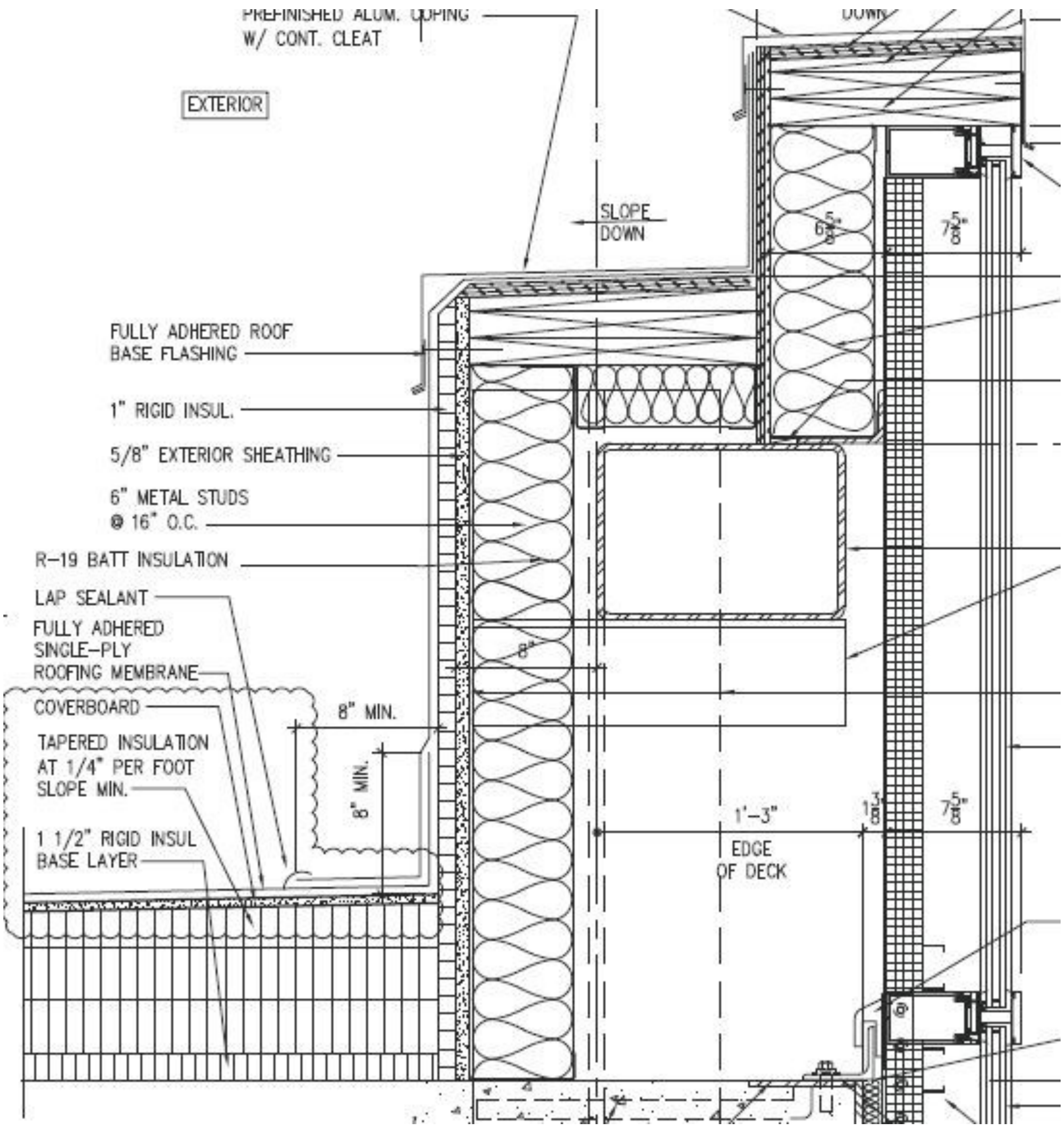


Figure 11 - Roof Parapet Section
Drawings Courtesy of JCJ Architecture

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, transmitting wind and seismic loads through the moment and braced frames with no effect on the other tower.

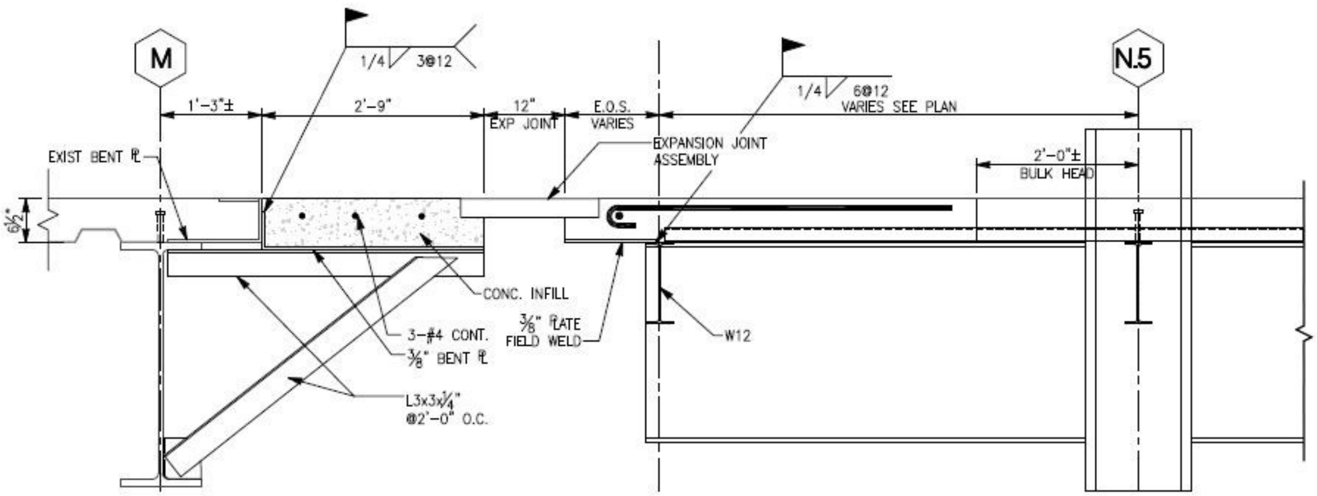


Figure 12 - Expansion Joint Section
Photo Courtesy of JCJ Architecture

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

Existing Hotel Lobby & Load Path

On top of connecting to the original hotel tower's structure, the new SAC Hotel addition has 9 floors, including the roof, partially resting on a 3-story lobby that was constructed with the original hotel tower. The lobby was over designed with the new addition in mind to carry the large loads. Figure 14 shows the existing frames in column line 9.

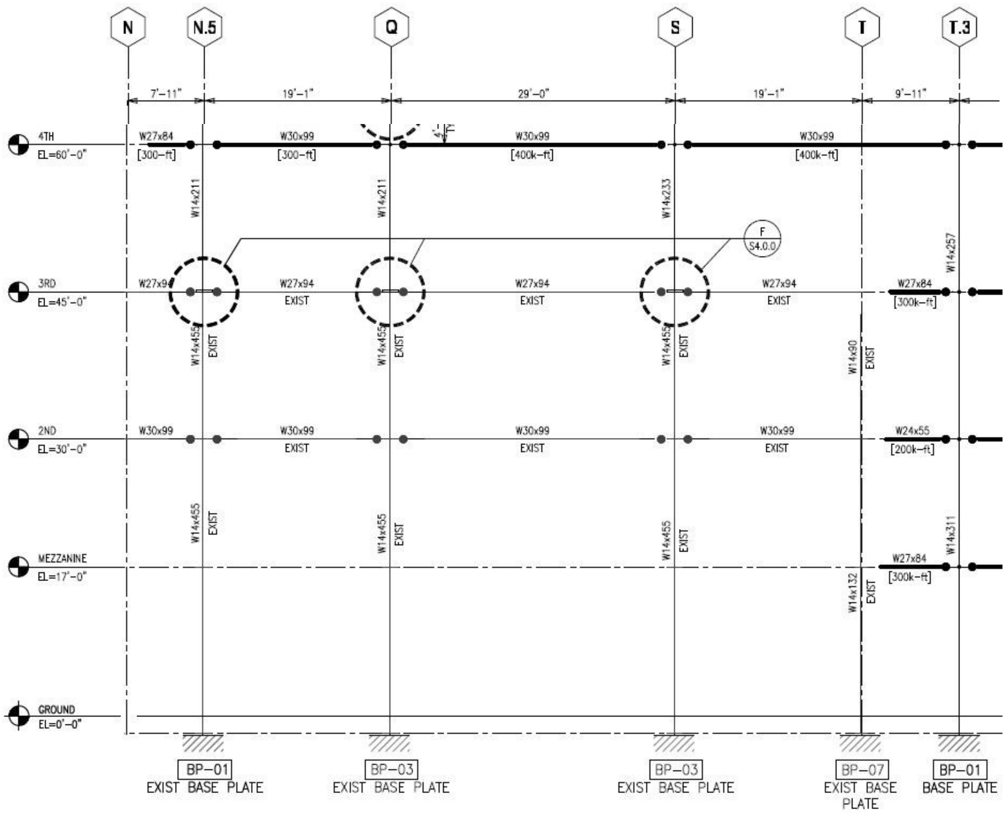


Figure 14 - Existing Lobby Frame (Col. Line 9)
Photo Courtesy of JCJ Architecture

For the overall load distribution, gravity loads are taken by the composite slabs, then to beams and girders and then columns. The column loads terminate in the foundations at ground level. For lateral loads, wind and seismic in the E-W direction are transferred through moment frames along the perimeter of the building, down to the foundations. In the N-S direction, diagonally braced frames in specific column lines, (stated in the Lateral section, pg. 8), carry the loads to the foundation.

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 Cite Specific 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.

Spot Checks

Prior to any calculations, it was found that the design of the SAC Hotel addition was accomplished using ASD over LRFD. This analysis was done using LRFD. Therefore, any discrepancies in final loads, moments, etc., could be a result of this fact. Figure 15 below shows the members in a typical 29' by 25.75' bay, along with an exterior column, that were analyzed for this section of the report. It was found that live load reduction did not apply for the beam and girder due to small tributary areas.

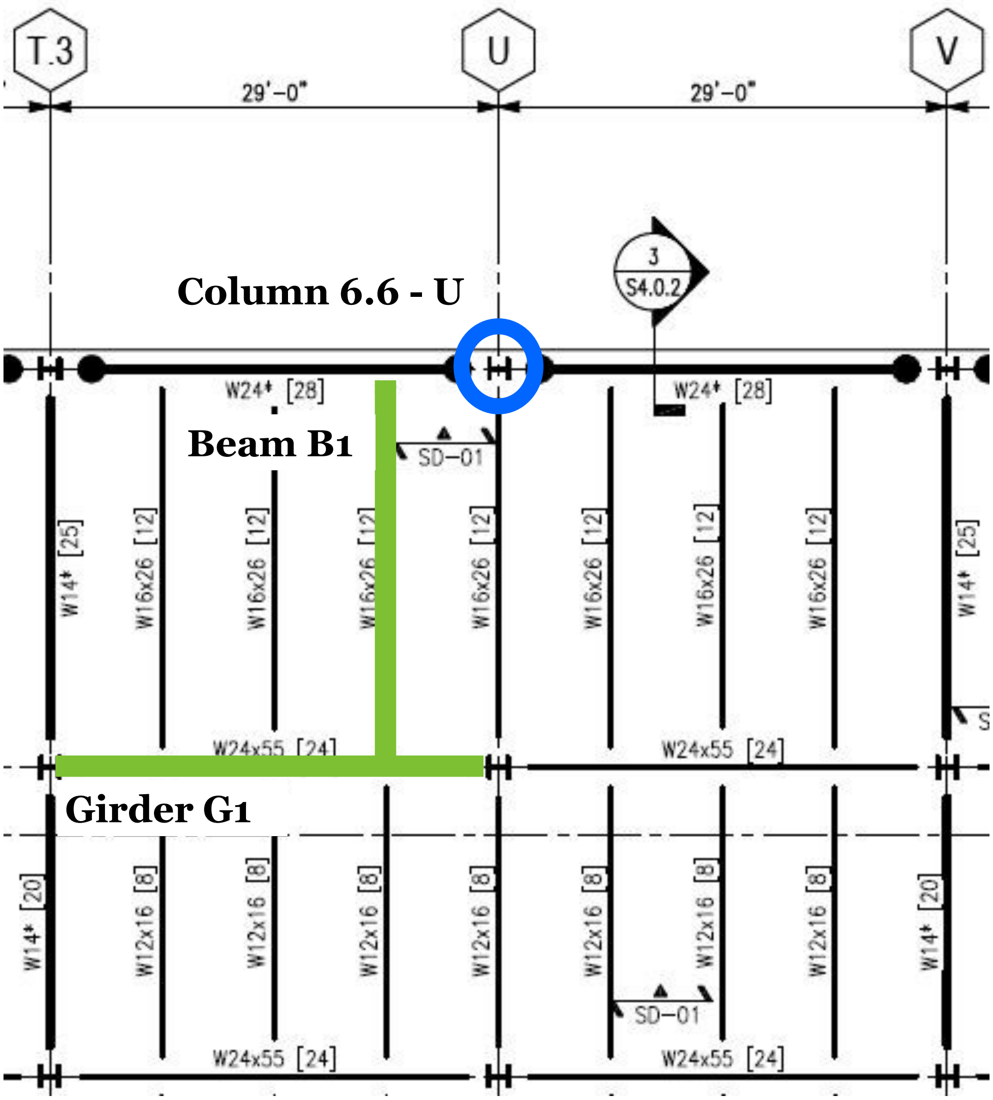


Figure 15 - Typical Bay of 4th Floor for Spot Checks
Photo Courtesy of JCJ Architecture

Decking

In the SAC Hotel addition, the metal deck schedule calls for 2 different sizes of composite deck, 18 and 20 gage, with normal weight concrete. The bay in Figure 15 requires 20 gage, 6.5" in total depth. For this analysis, 2VLI20 deck was selected from the Vulcraft 2008 Deck Catalogue as a close approximation. The total weight of the 2VLI20 is 69 psf, with a clear span of 7'6", (>7.25' in the bay used for analysis), capable of carrying a 400 psf superimposed live load, while design specifications only require 80-125 psf superimposed live load.

Beams

Beams span in the N-W direction in the 29' by 25.75' bay, with W16x26's as the most common and repetitive members for the hotel room floors. For all load calculations, a superimposed dead load of 15 psf accounts for suspended mechanical equipment and ceilings. Initially, Beam B1 was checked as simply supported and was found to meet the required loads, with a slight overdesign most likely due to LRFD. Next, B1 was checked as partially composite to determine the number of shear studs required. It was found that 24 shear studs would be required, which is double what the SAC Hotel frame plans show. Differences could again be due to LRFD over ASD, and the fact that the addition was designed with an older version of the AISC manual. As far as deflections, the W16x26 met the requirements for live and total load deflections.

Girder

In Figure 15, Girder G1 spans 29' in the E-W direction, carrying beams that support the hotel rooms and the corridor. The girder was also checked with simply supported and then partially composite. It was found that the W24x55 was unable to meet any of the required loads. Due to LRFD, the two different live loads, (80 psf for hotel rooms and 100 psf for corridors), being carried by the beams framing into the girder greatly increase the required moments that ASD would otherwise account for. The required moment was found to be 12% higher than the W24x55 can carry as composite.

Supporting calculations can be found in Appendix D.

Columns

A 4th floor exterior column from the bay in Figure 15, column 6.6-U, was chosen for this spot check. From the 4th floor to the roof, shape sizes range from W14x211's to W14x120's. All columns were found to be more than adequately designed, although the effects of moment connections were not explored. Also, some loads could have been estimated as not conservative enough. Figures 16 and 17 below show the member sizes associated with floor levels and which loads were applied. Table 4-1 of the AISC manual was used to find the critical stresses associated with the effective lengths listed. Sample calculations can be found in Appendix D and frame elevations can be found in Appendix B.

Column Loads					
Floor	**A_t (ft^2)**	**DL (psf)**	**LL (psf)**	**Reduced LL (psf)**	**SL (psf)**
4	373.5	100	80	51	0
5	373.5	100	80	51	0
6	373.5	100	80	51	0
7	373.5	100	80	51	0
8	373.5	100	80	51	0
9	373.5	100	80	51	0
10	373.5	100	80	51	0
11	373.5	100	80	51	0
Roof	373.5	100	200	127.6	40

Figure 16

Column Capacities					
Floor	**P_u (k)**	**$\sum P_u$ (k)**	**Member**	**ϕP_n (k)**	**Unbraced**
4	75.30	730.92	W14x211	2580	11.33
5	75.30	655.62	W14x176	2150	11.33
6	75.30	580.32	W14x176	2150	11.33
7	75.30	505.03	W14x145	1770	11.33
8	75.30	429.73	W14x145	1770	11.33
9	75.30	354.43	W14x120	1450	11.33
10	75.30	279.13	W14x120	1450	11.33
11	75.30	203.84	W14x120	1450	11.33
Roof	128.54	75.29	W14x120	1450	13.67
∑	**730.92**				

Figure 17

Lateral Analysis

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 D. The Cs value found was about half the provided value in the design specs, with a N-S base shear of 265 kips controlling the lateral system for seismic. The following Figures 18 and 19 provide the exact numbers used, and Figure 20 provides a diagram of the force distribution.

Floor Weights		
Floor	Weight (Kips)	Sum of Above
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	0.014
$\sum w_i h_i^{1.12}$	2804282.544
V (kips)	265.2
M (ft.-k)	28661

Figure 18

Forces				
Floor	Height (ft.)	C_{vx}	F_x (kips)	M (ft.-k)
Roof	153	0.139319	36.95	5653.0
11	139.33	0.141821	37.61	5240.3
10	128	0.128969	34.20	4377.9
9	116.67	0.116254	30.83	3597.0
8	105.33	0.103674	27.49	2896.0
7	94	0.091267	24.20	2275.2
6	82.67	0.079039	20.96	1732.9
5	71.33	0.067	17.77	1267.4
4	60	0.0552	14.64	878.3
3	45	0.04013	10.64	478.9
2	30	0.025528	6.77	203.1
Mezz.	17	0.013477	3.57	60.8
		1.00	265.6	28660.8

Figure 19

Seismic Forces

N-S Controls

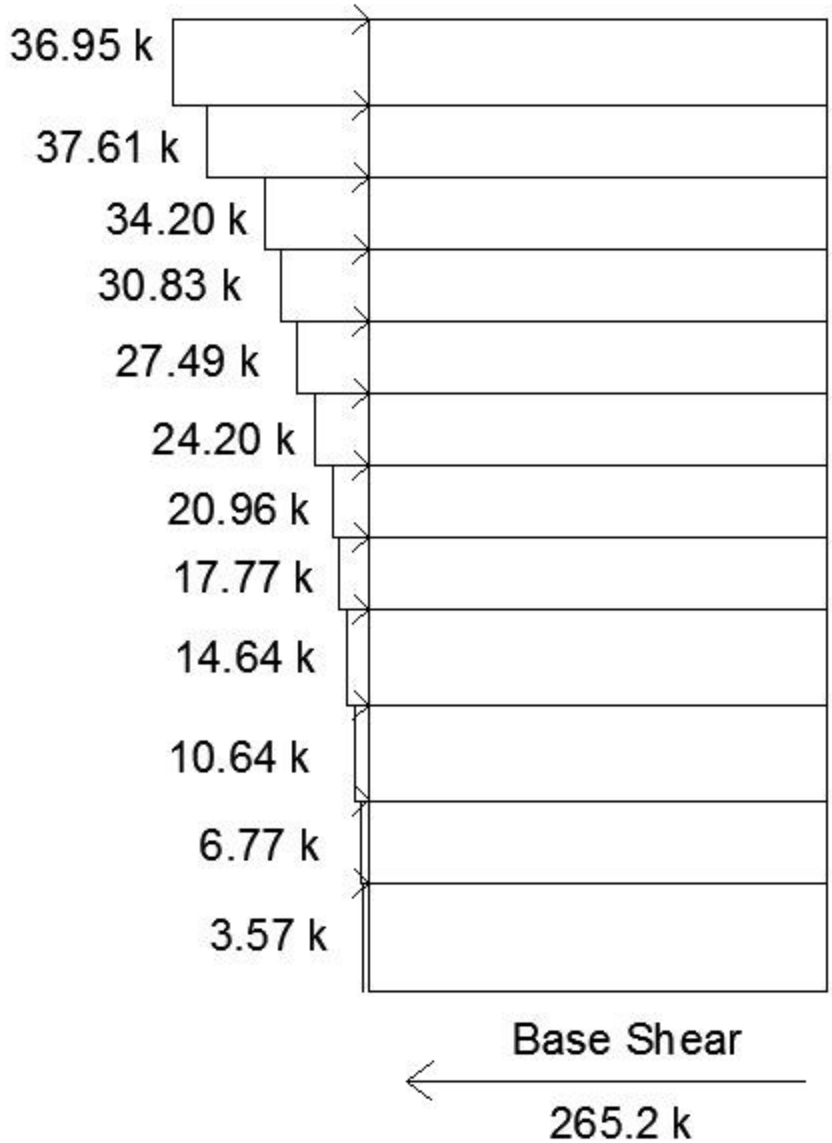


Figure 20 - Seismic Forces and Base Shear, (N-S)

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 overall lateral system in the N-S direction, producing a base shear of 908.9 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 21 - 27 show the tabulated values of pressures and story forces, and diagrams of the force and pressure distribution. Full calculations can be found in Appendix D.

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 21

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 22

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 23

North-South Wind Pressures and Forces

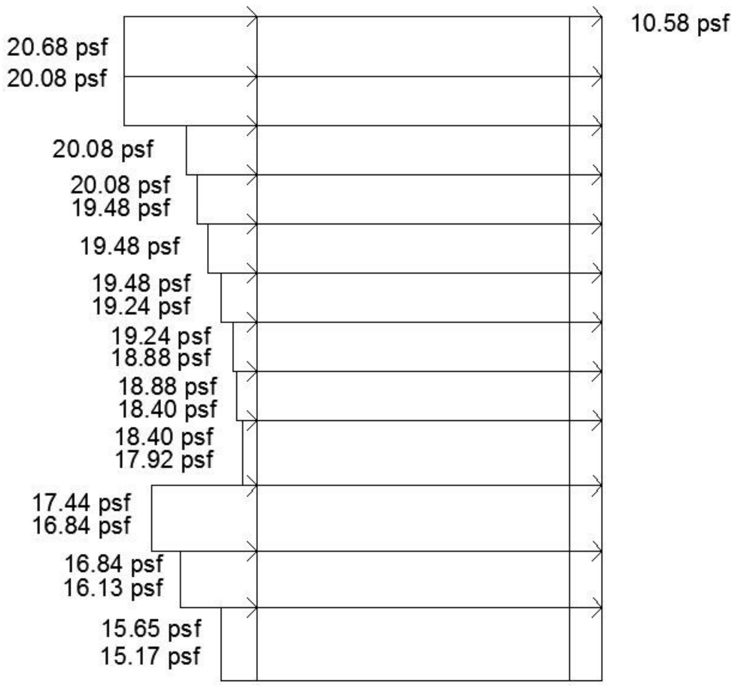


Figure 24

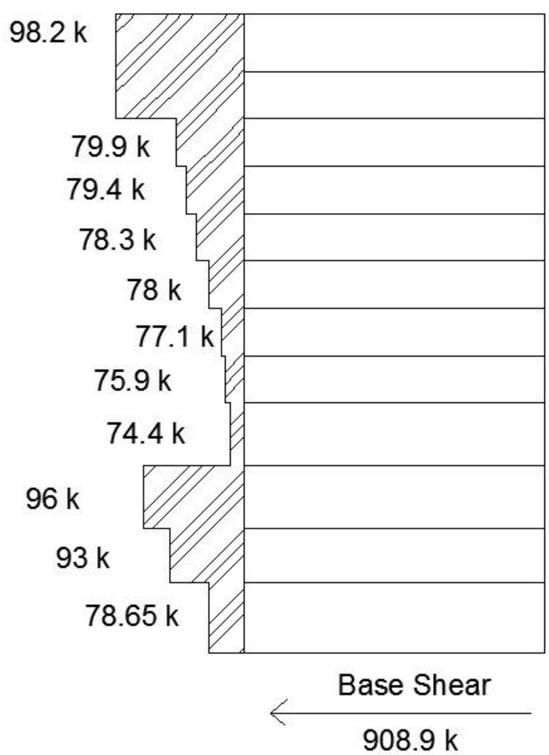


Figure 25

East-West Wind Pressures and Forces

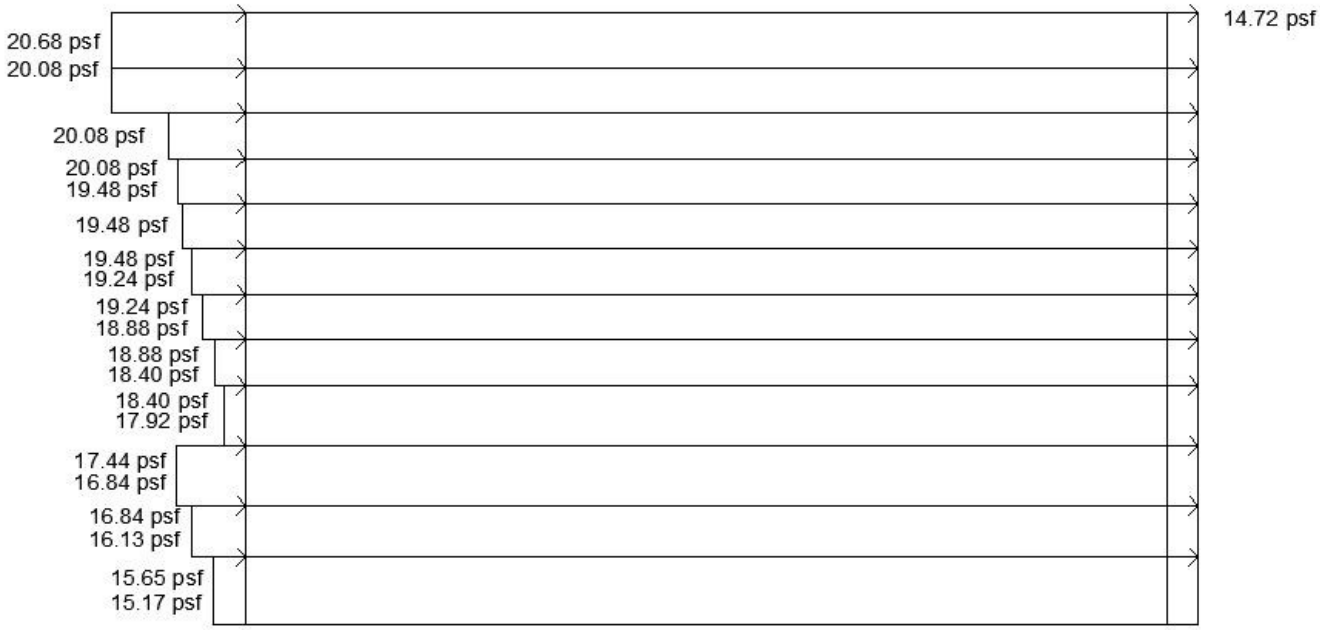


Figure 26

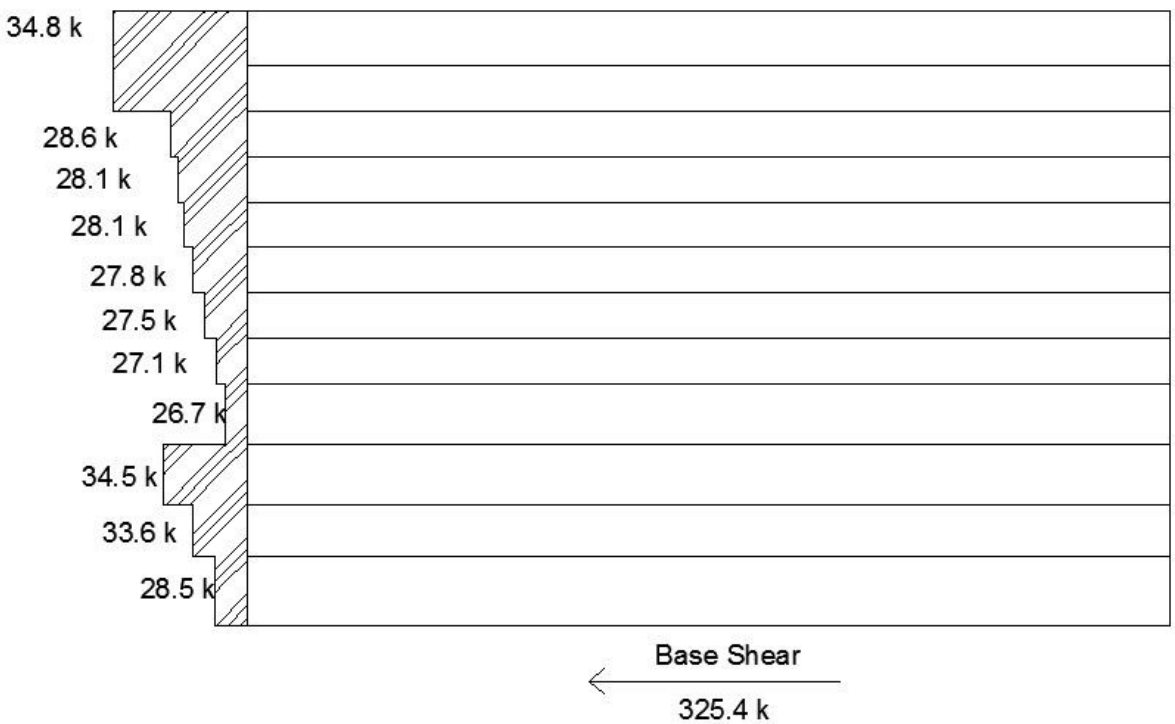


Figure 27

Lateral Load Comparison

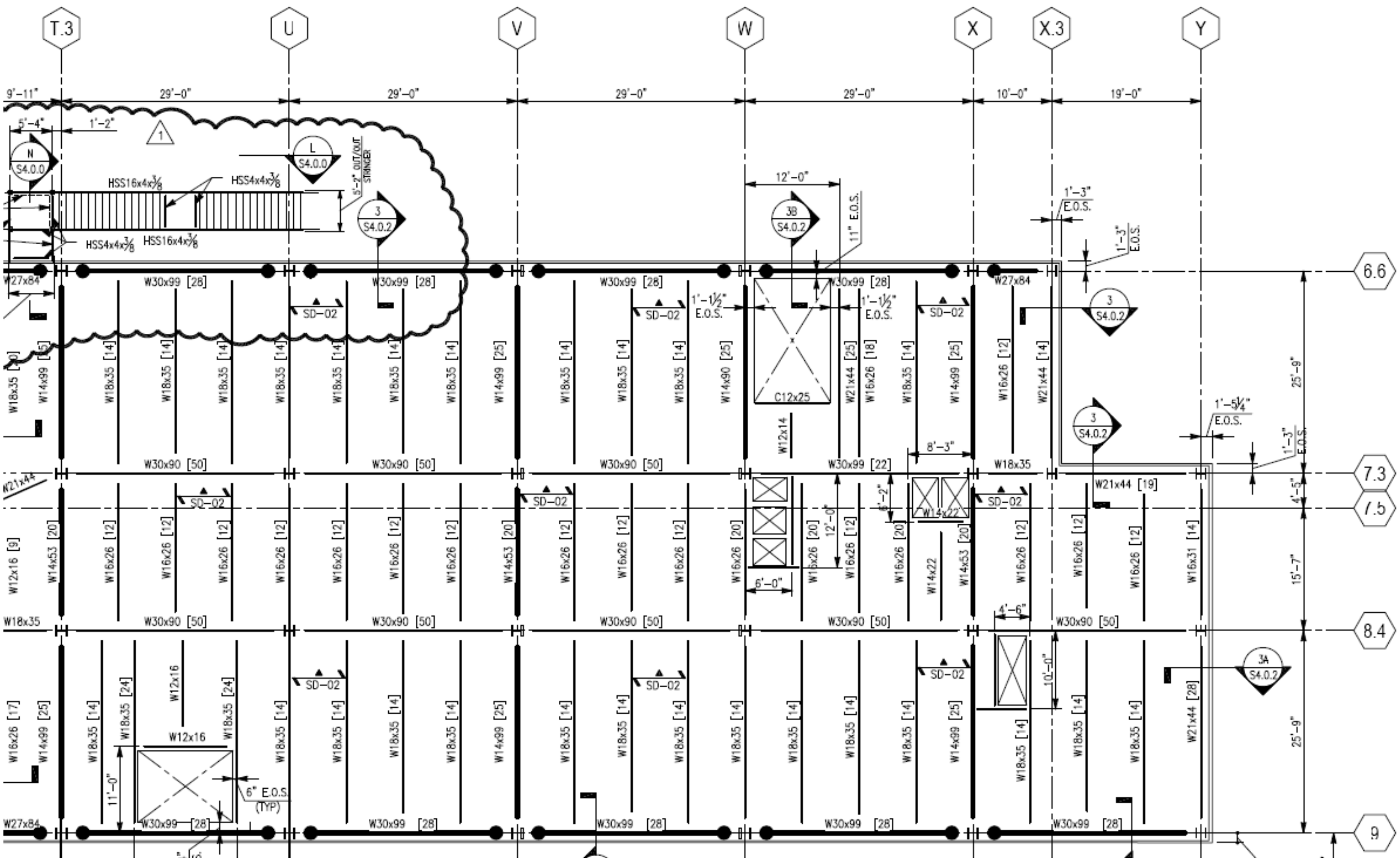
	Wind		Seismic
	N-S	E-W	
Base Shear (k)	908.9	325.4	265.2
Overturning Moment (ft.-k)	73452	14759	28661

It was determined that the N-S direction controlled the overall design of the lateral system, as shown in the above table.

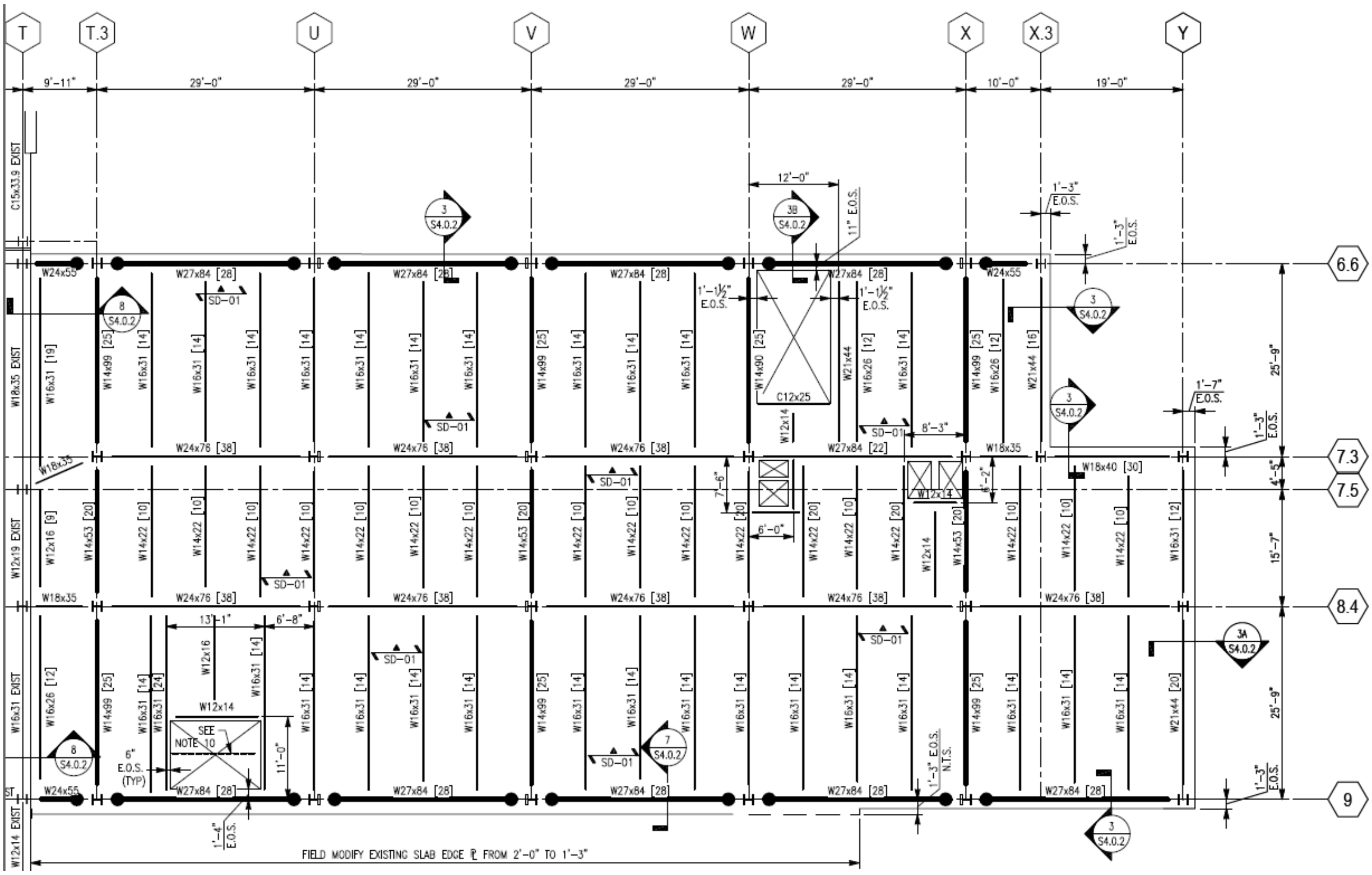
Appendices

Appendix A - Floor Plans

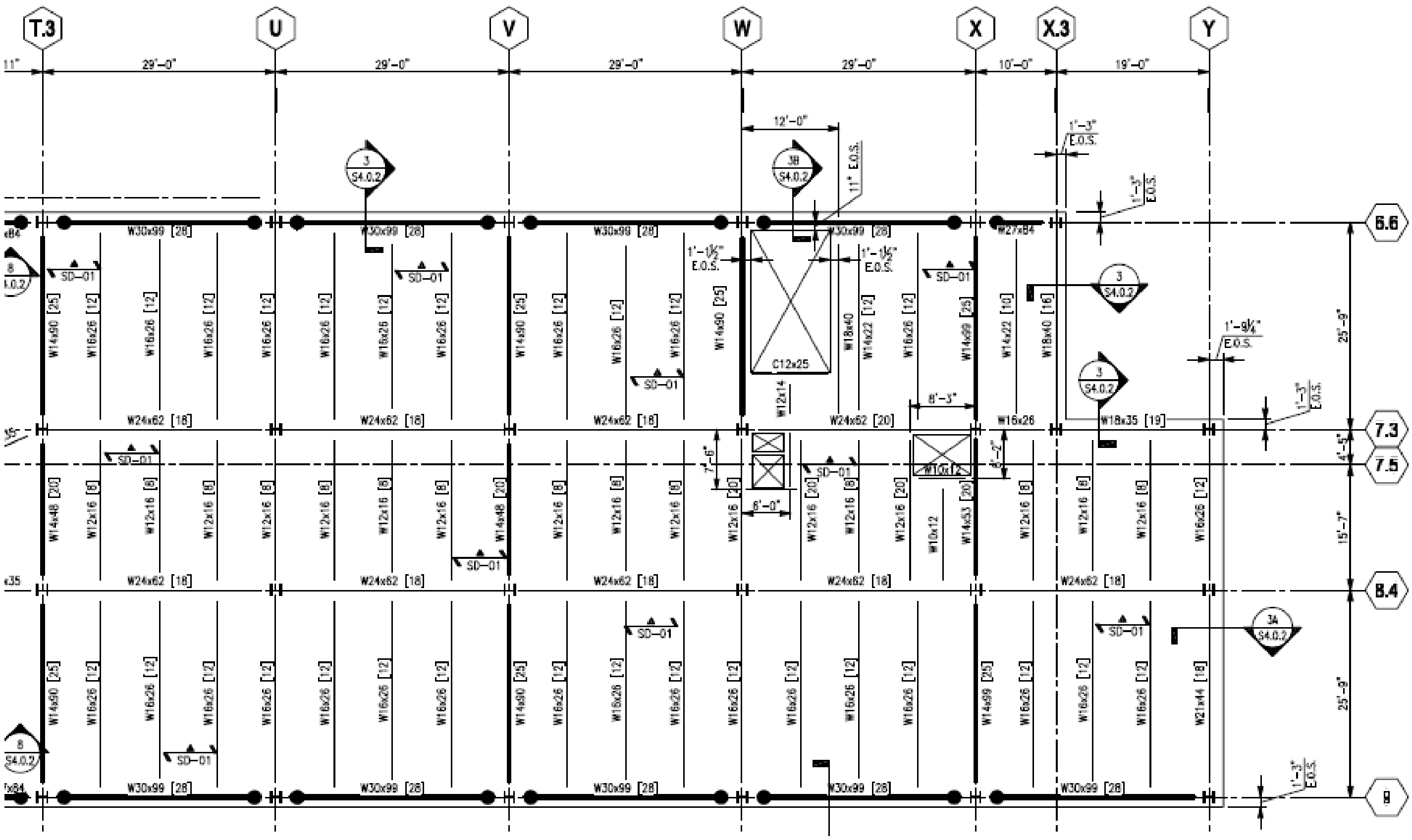
Mezzanine



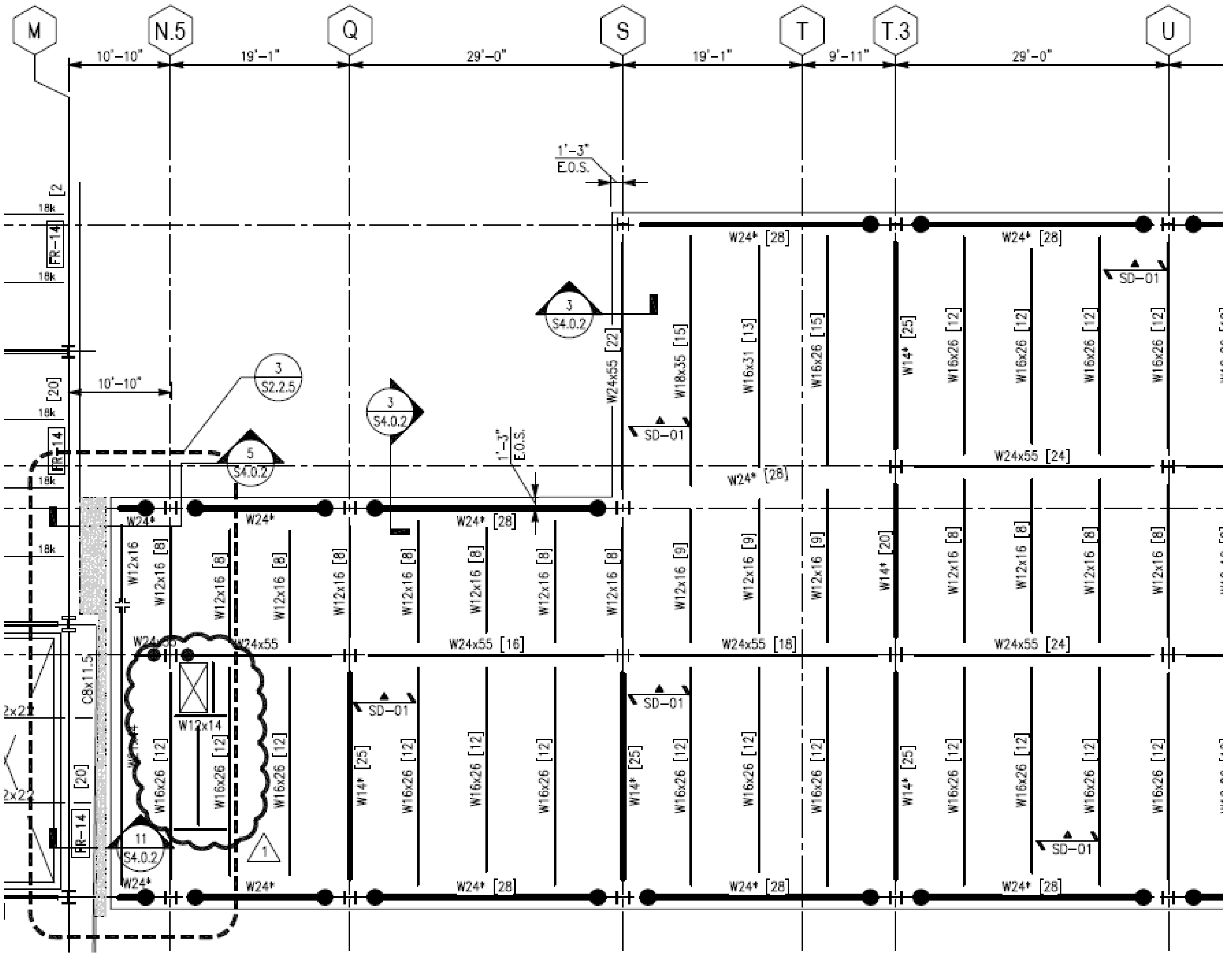
2nd Floor


T
T.3
U
V
W
X
X.3
Y
9'-11"
29'-0"
29'-0"
29'-0"
29'-0"
10'-0"
19'-0"
6.6
7.3
7.5
8.4
9
25'-9"
4'-5"
15'-7"
25'-9"
12'-0"
W24x55
W27x84 [28]
W24x76 [38]
W18x35
W18x40 [30]
C12x25
W12x14
SD-01
S4.0.2
SEE NOTE 10
6" E.O.S. (TYP)
11'-0"
13'-1"
6'-8"
1'-4" E.O.S.
1'-3" E.O.S. N.T.S.
C15x33.9 EXIST
W18x35 EXIST
W12x19 EXIST
W16x31 EXIST
W12x14 EXIST
FIELD MODIFY EXISTING SLAB EDGE ℄ FROM 2'-0" TO 1'-3"

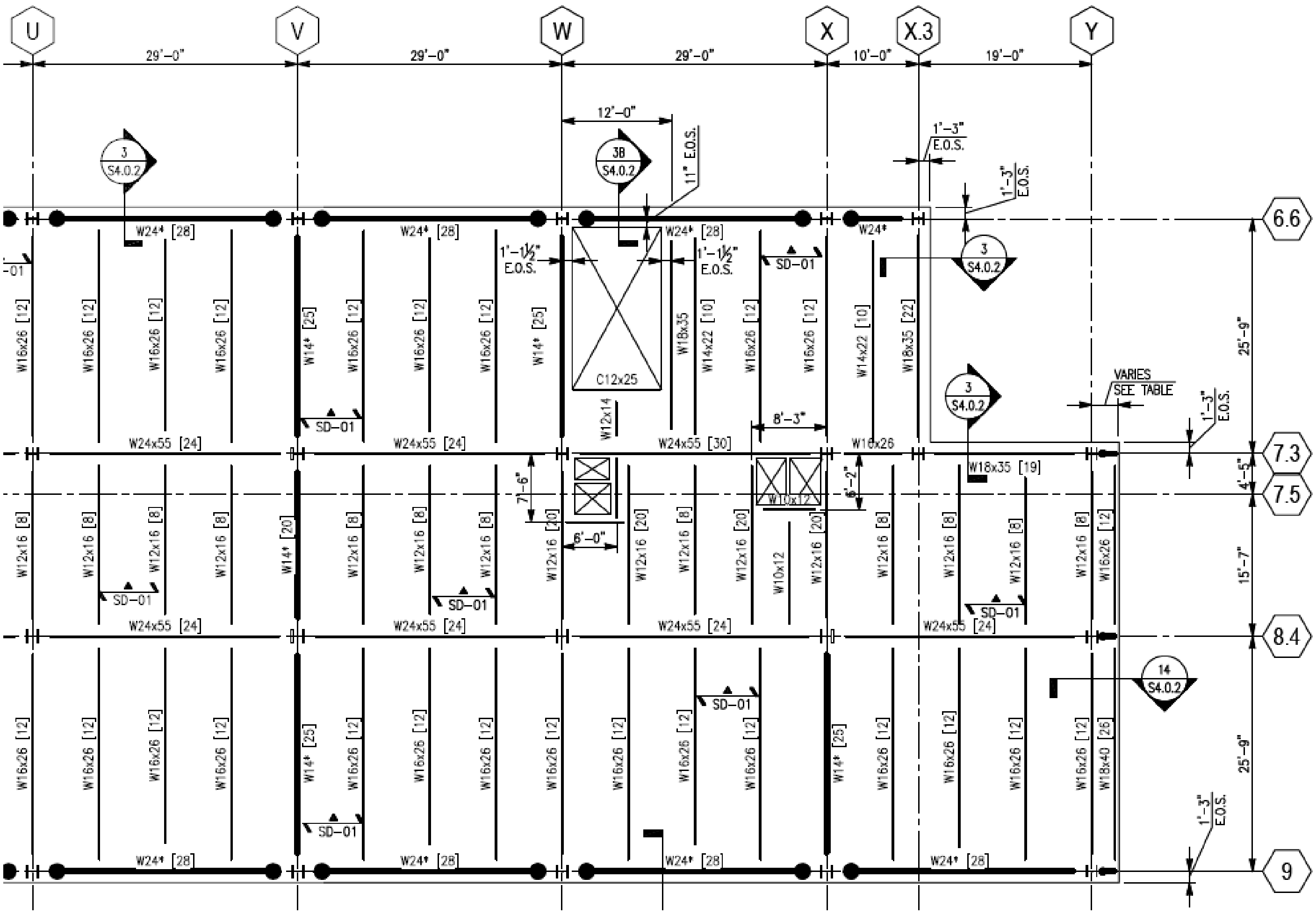
3rd Floor



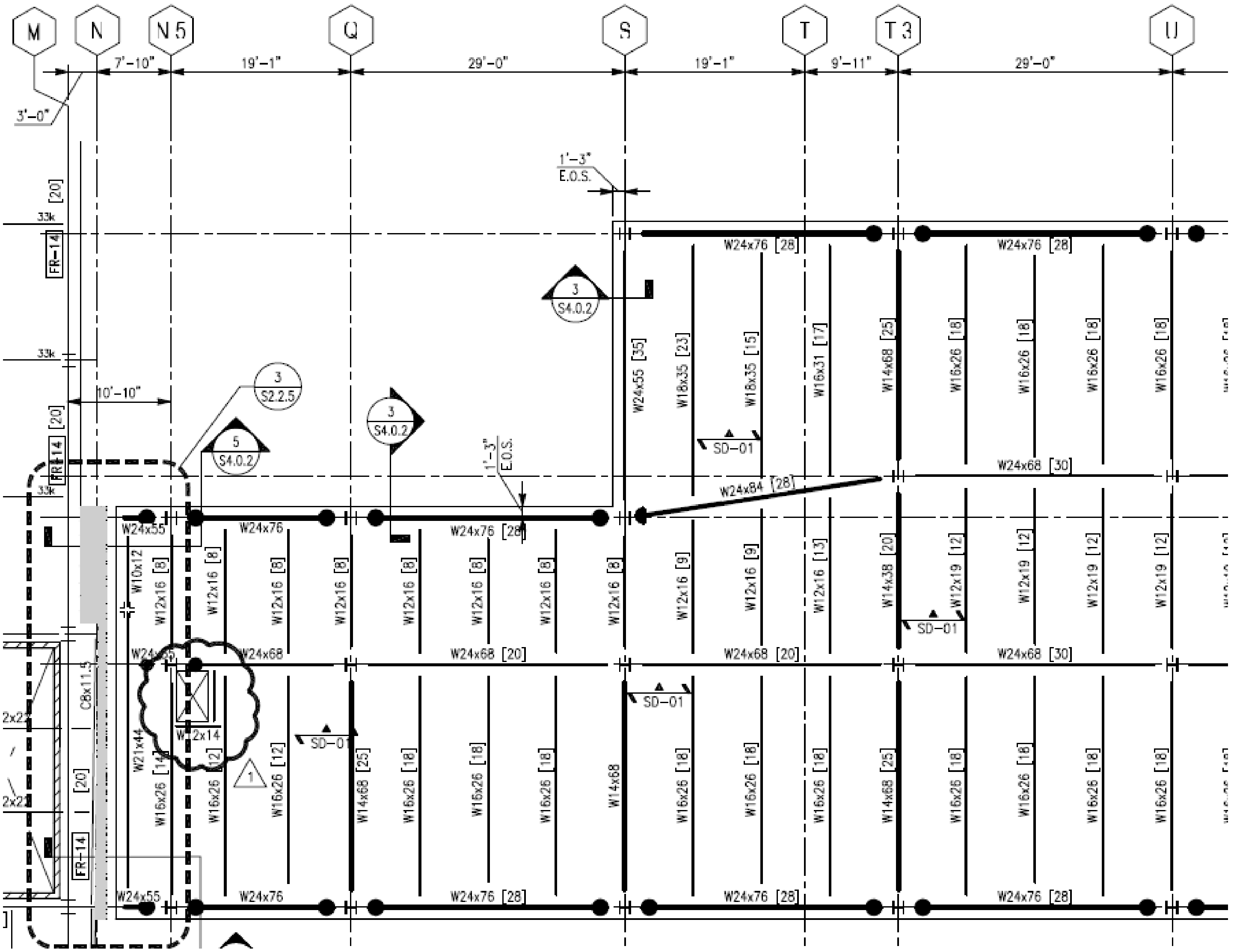
4th thru 10th Floor (Partial)



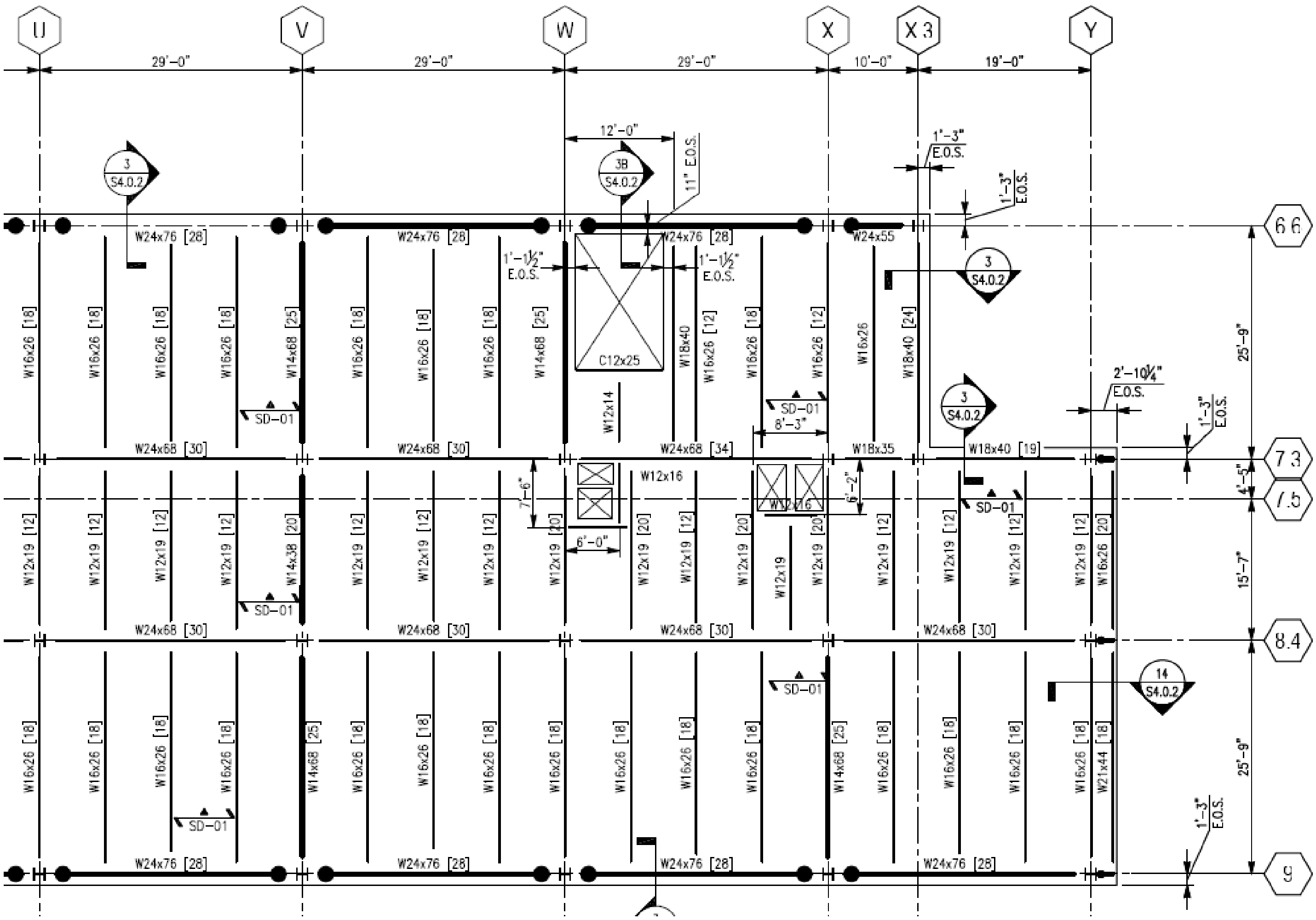
4th thru 10th Floor (Partial)



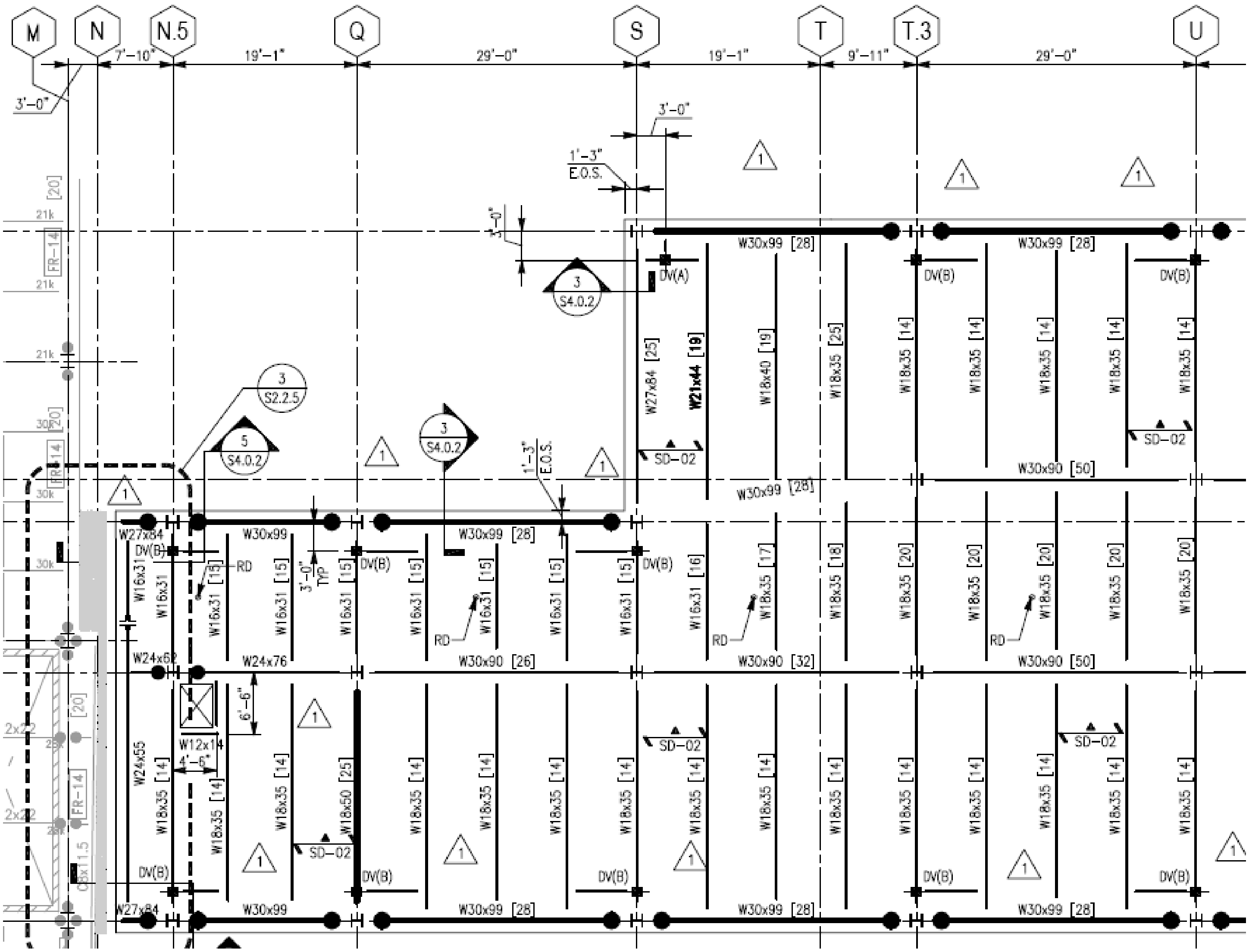
11th Floor (Partial)



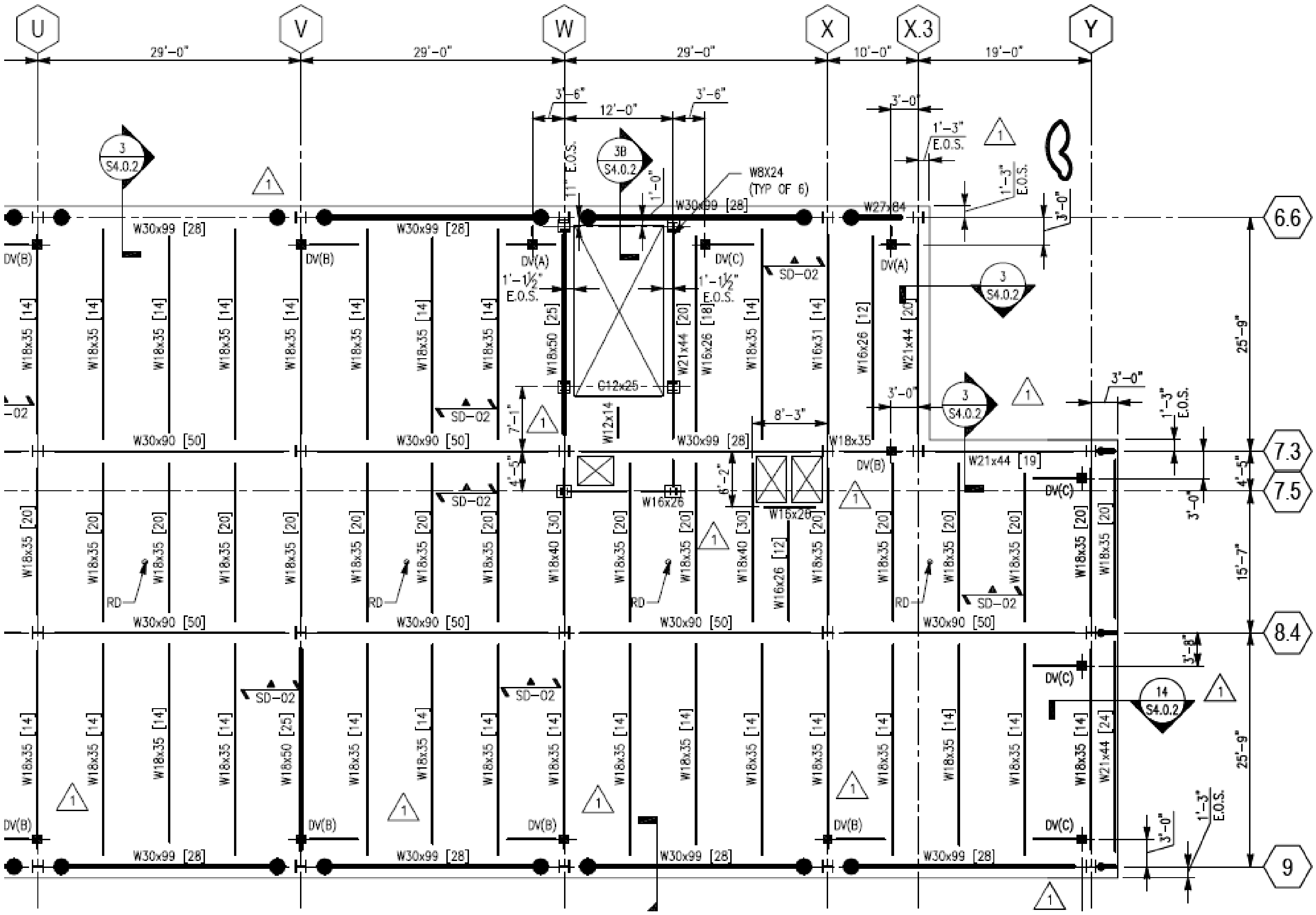
11th Floor (Partial)



Roof (Partial)

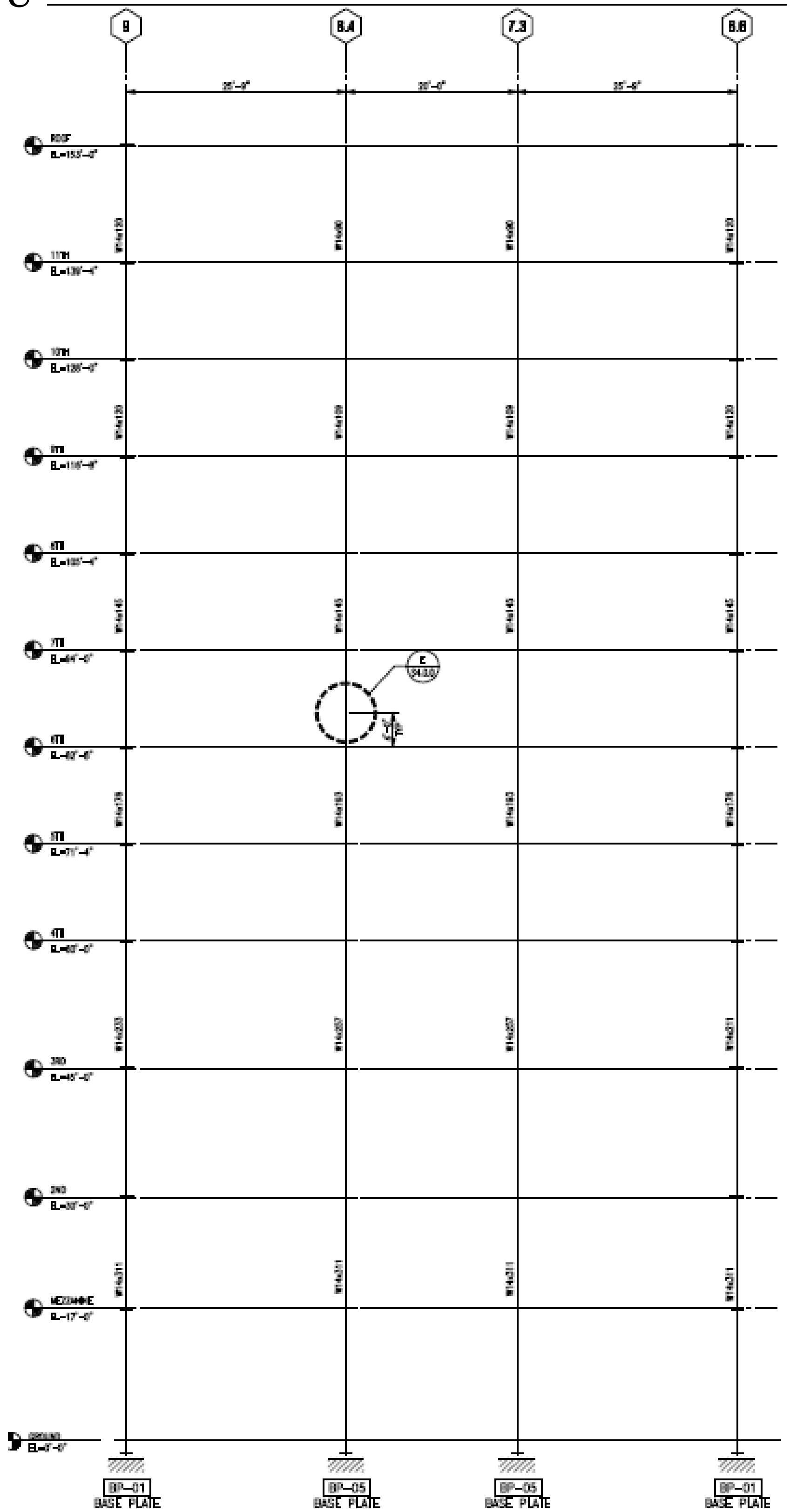


Roof (Partial)

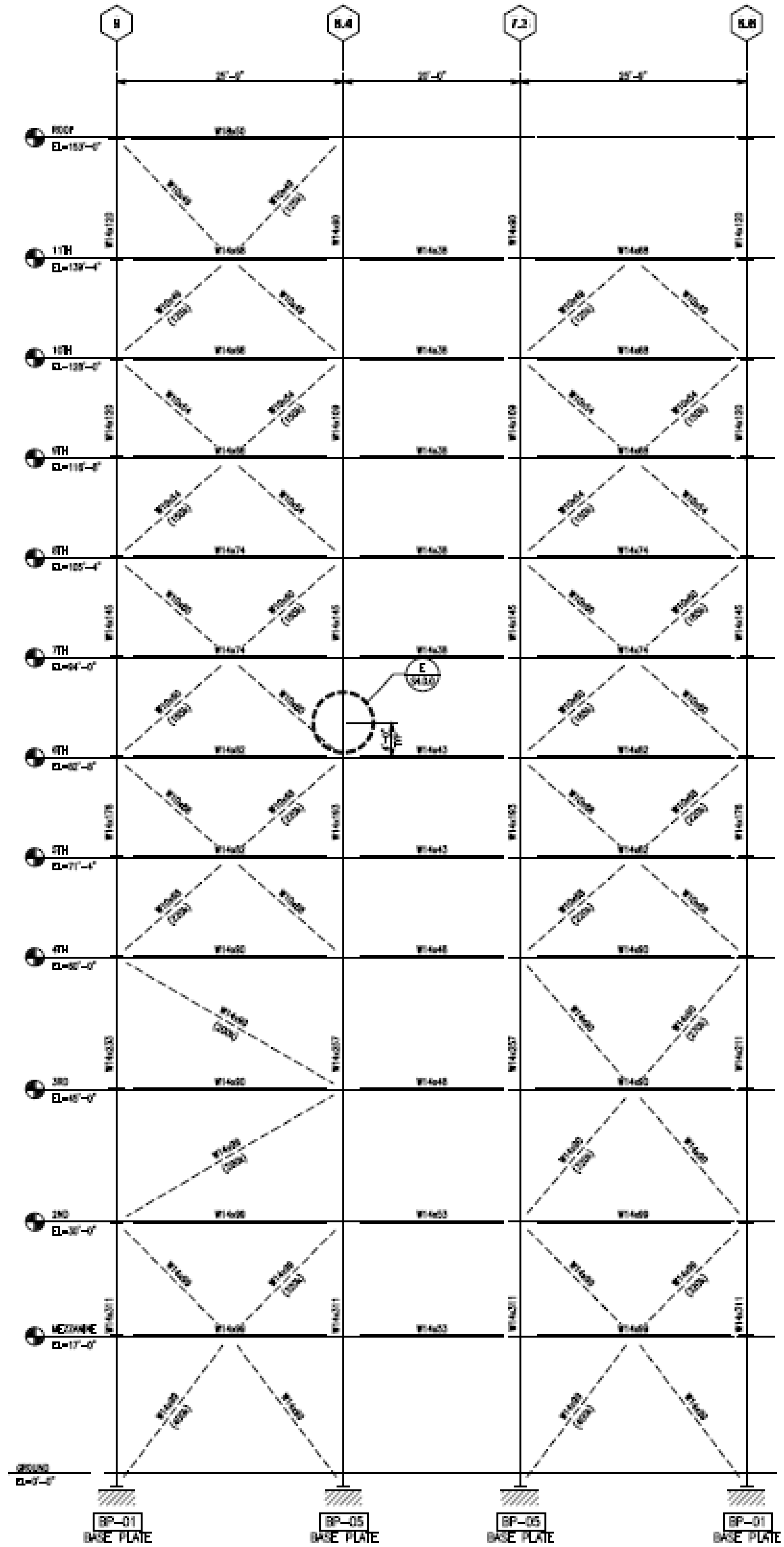


Appendix B - Elevations

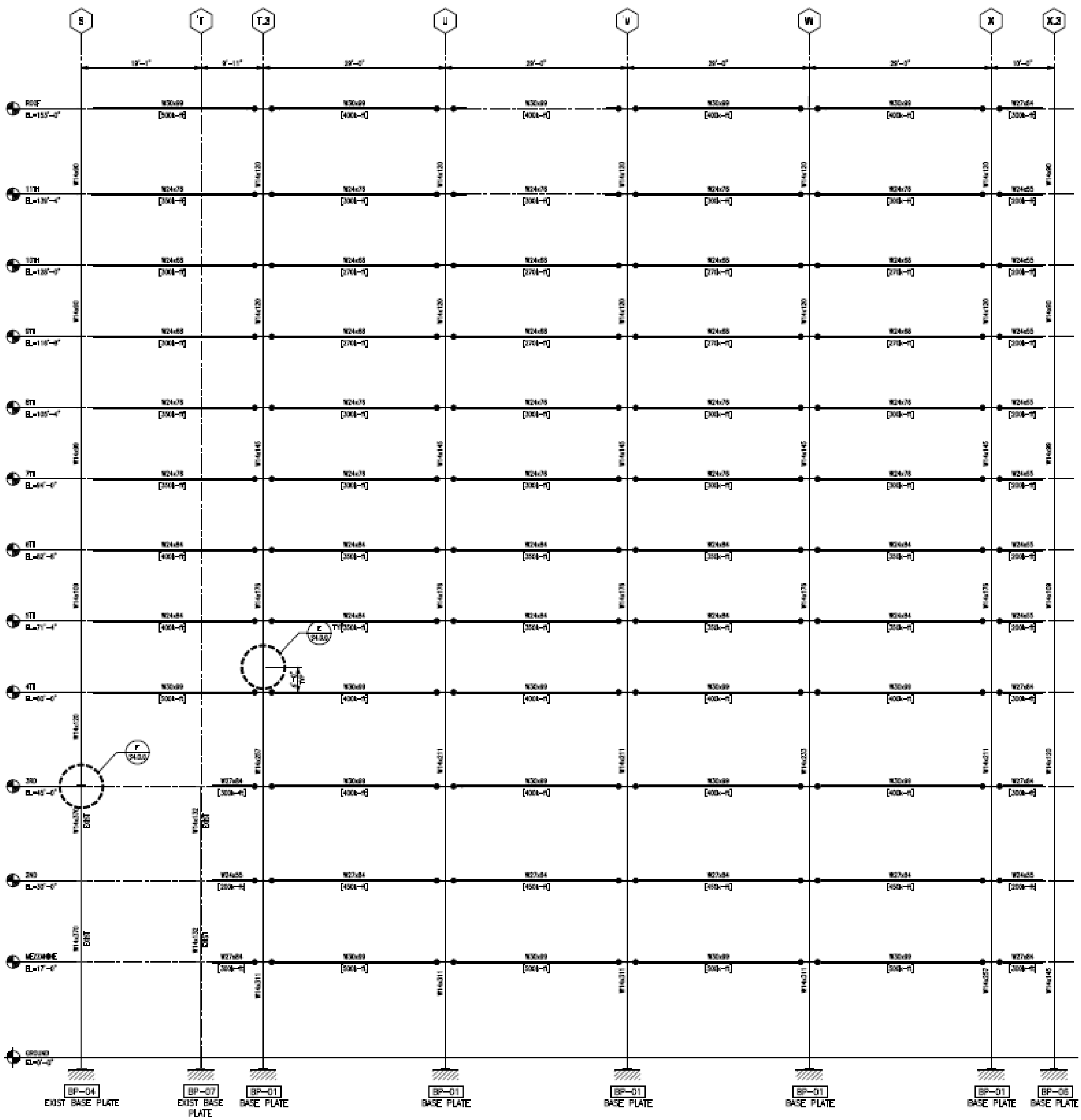
Column Line U



Column Line V

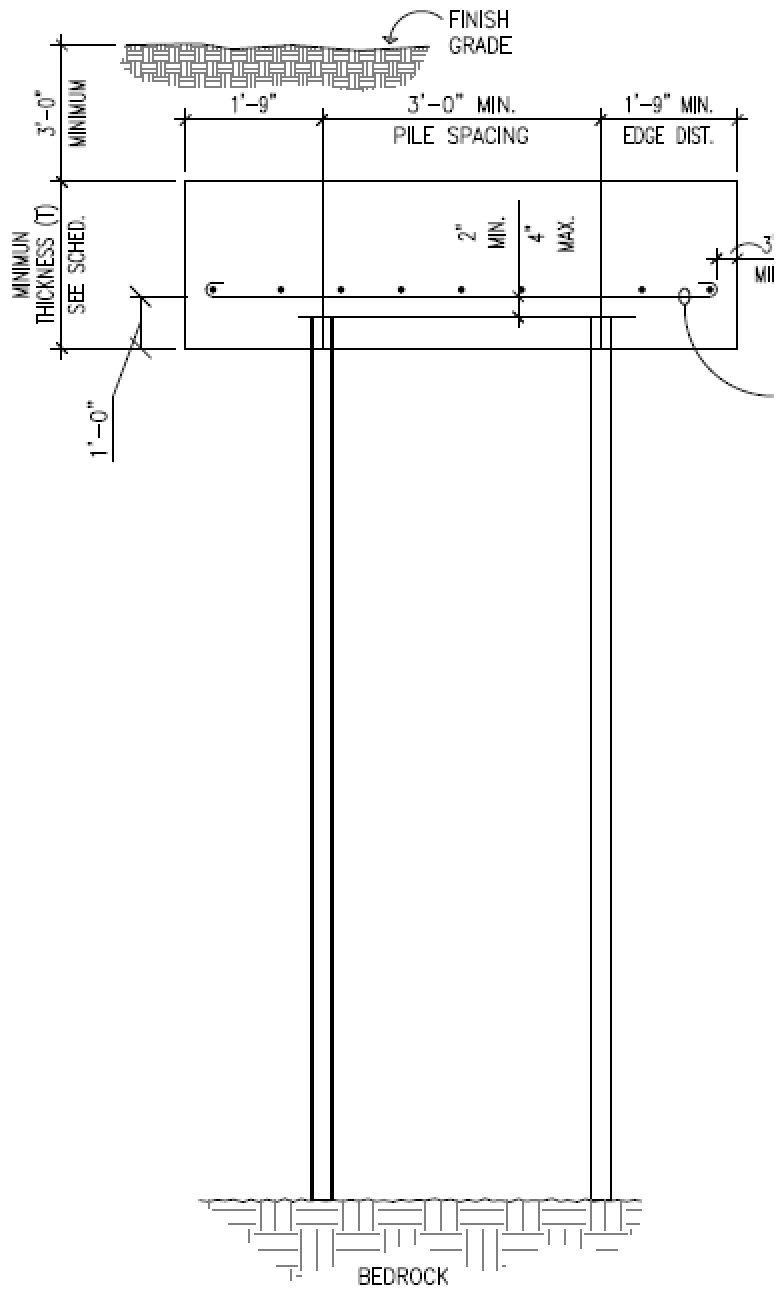


Column Line 6.6

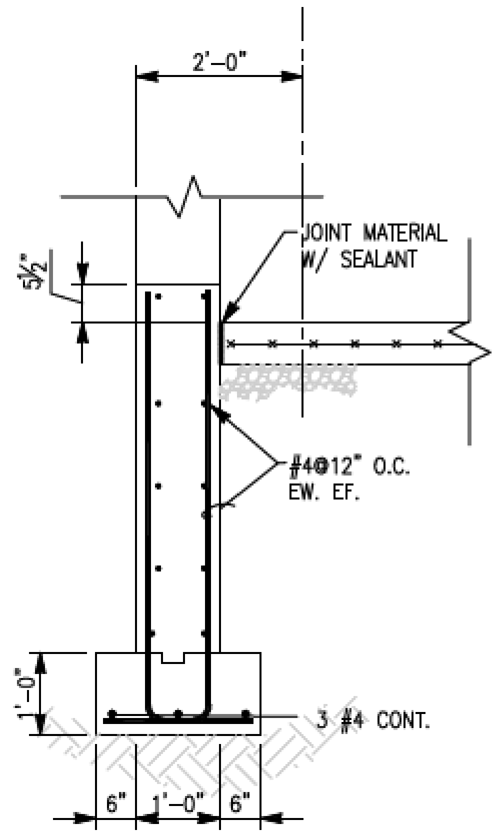


Appendix C - Sections

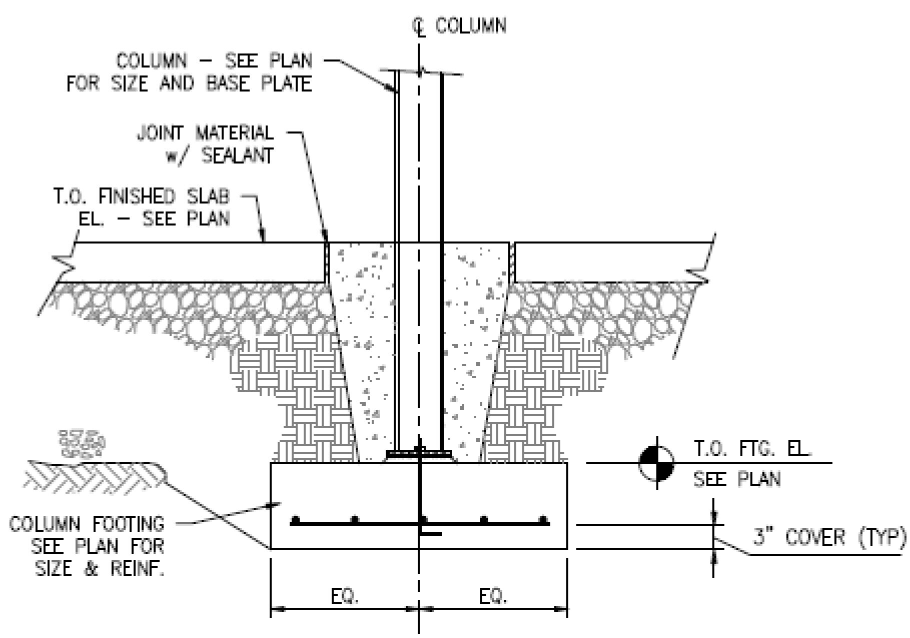
Steel Piles



Footings

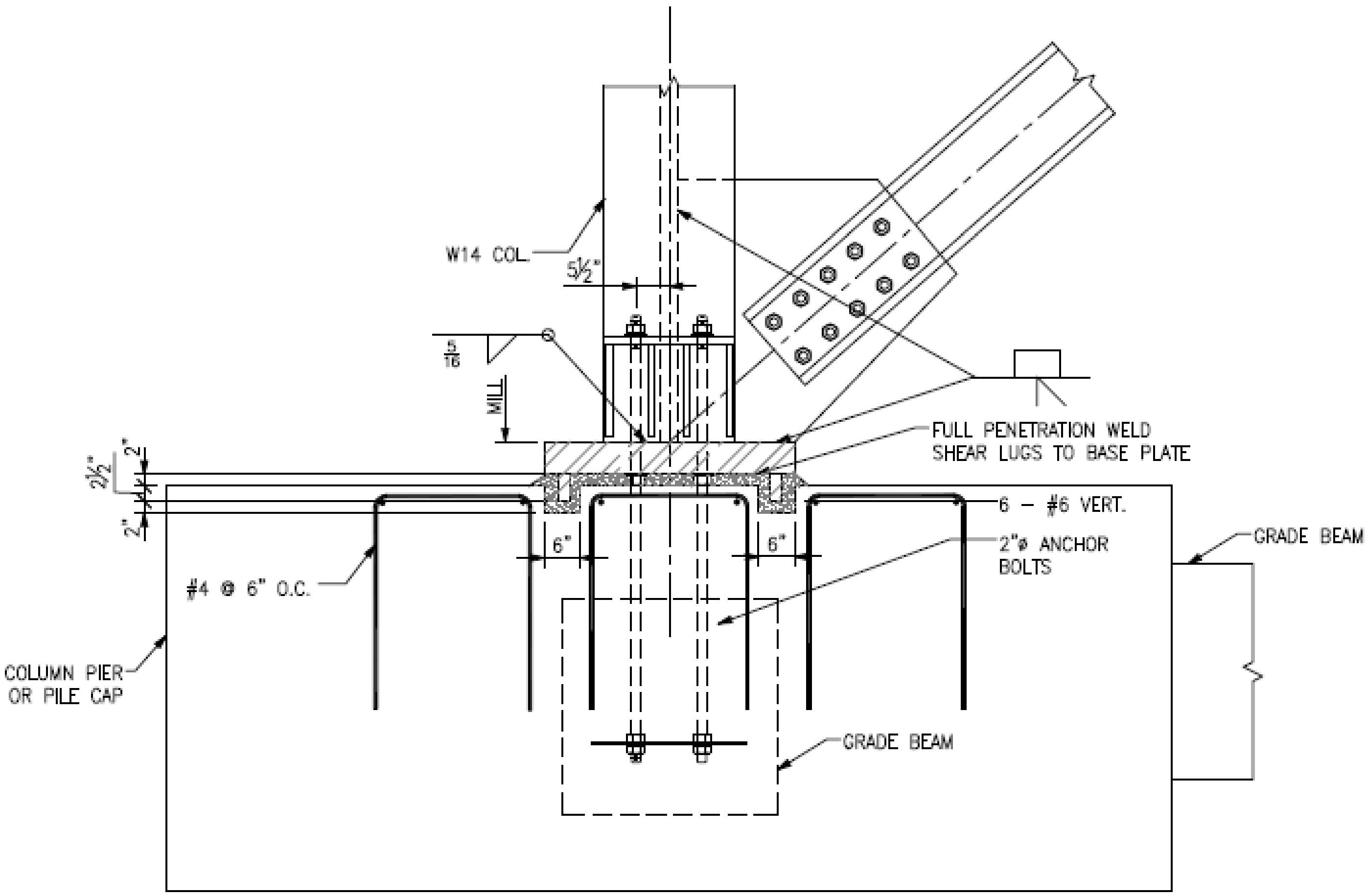


Typical Strip Footing



Typical Interior Footing Without Pier

Brace At Foundation



Appendix D - Calculations

Nick Reed | Tech. Report 1 | Snow Loads

Roof (Flat)

From specs

$C_e = 1.0$

$C_t = 1.0$

$I = 1.1$

$p_g = 50\text{psf}$

$p_f = 0.7 C_e C_t I p_g$

$p_f = 0.7(1)(1)(1.1)(50)$

$p_f = 38.5\text{psf} \approx 40\text{psf}$ from specs

Drift

$\gamma = 0.13 p_g + 14$

$= 0.13(50) + 14 = 20.5\text{ psf} < 30\text{ pcf}$ OK

$h_b = \frac{p_s}{\gamma}$ $\quad p_s = C_s p_f \quad C_s = 1.0$

$p_s = 38.5\text{ psf}$

$h_b = \frac{38.5}{20.5} = 1.88'$

$h_{penthouse} \approx 8'$

$\frac{h_c}{h_b} = \frac{6.12}{1.88} = 3.26 > 0.2$

Account for drift

$h_c = 8 - 1.88 = 6.12'$

$l_u = 10'$

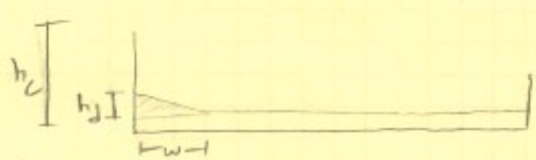
$h_d = 0.43 \sqrt[3]{10} \sqrt[4]{50+10} - 1.5 = 1.08'$ $\quad h_d < h_c$

$w = 4h_d < 8h_c$

$w = 4(1.08) = 4.32' < 8(6.12) = 49'$ OK

4.32' less than 25'-9" available for drift

$p_d = 1.08(38.5) = \boxed{41.6\text{ psf}}$



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

Location: Salamanca, NY Occupancy Category III

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

Exposure C, MWFRS

$V = 90$ mph (Fig. 6-1)

$I = 1.15$ (Table 6-1)

$K_d = 0.85$ (Table 6-4)

$K_{zt} = 1.0$

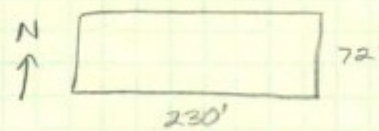
$G = 0.85$

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

Total height = 153'

$K_z = 1.36 + (153 - 140)\left(\frac{1.39 - 1.36}{160 - 140}\right) = 1.38$

* See attached spreadsheet for q_z calcs *



Simplified Shape

Design Wind Pressures for MWFRS

Wall Cp

WW = 0.8 $L/B = 2.2 \Rightarrow$ LW = -0.3 (N-S)

SW = -0.7 $L/B = 0.46 \Rightarrow$ = -0.5 (E-W)

Roof Cp

$h/L = 153/155 = 0.99$ $C_p = -1.3$ 0 to h/2; -0.7 > h/2 Second $C_p = -0.18$

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

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')

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

$= 78.65$ 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')

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

2nd Floor (30'-45')

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

3rd Floor (45'-60')

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

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

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

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

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

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

Story Wind Forces (E-W) cont'd

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

$[(18.88)(7.33) + (19.24)(4')](72) + (14.72)(11.33)(72) = 27.51\text{ k}$

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

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

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

$(19.48)(11.33)(72) + (14.92)(11.33)(72) = 28.1\text{ k}$

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

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

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

$(20.08)(11.33)(72) + (14.92)(11.33)(72) = 28.61\text{ k}$

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

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

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

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

Salamanca, NY – Site Class D

Occupancy Category III

Importance Factor 1.25, Seismic Design Category B ⇒ $\rho = 1.0$

$R = 3.25$ $\quad S_s = 0.2$ (Fig 22-1)

$\Omega = 2$ $\quad S_1 = .046$ (Fig 22-2)

$C_b = 3.25$

$T = C_t h_n^x$ $\quad h = 153'$ $\quad C_t = .03$ $\quad x = 0.75$ (Table 12.8-2)

$T = .03(153)^{.75} = 1.31s$

$T_L = 6s$ (Fig 22-15) $\quad T < T_L = C_s = \dfrac{S_{D1}}{T\left(\frac{R}{I}\right)}$

$S_{D1} = \frac{2}{3} S_{M1}$ $\quad F_v = 1.6$ (Table 11.4-2)

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

$= \frac{2}{3}(1.6)(.046)$

$S_{D1} = .049$

$C_s = \dfrac{.049}{(1.31)\left(\frac{3.25}{1.25}\right)} = .014 > 0.01$

$S_{DS} = \frac{2}{3} S_{MS}$ $\quad F_a = 1.6$

$= \frac{2}{3} F_a S_s$

$= \frac{2}{3}(1.6)(0.2)$

$S_{DS} = 0.21$

$C_s = \dfrac{0.21}{(8/1.25)} = .033 > 0.01$

$C_s = .014$ in N-S Direction

Nick Reed | Tech Report 1 | Seismic Loads 2/4

Moment-Frames in E-W

$R = 3.5$ (Table 12.2-1)

$\Omega = 3$

$C_d = 3$

$I = 1.25$

$\rho = 0$

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

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

$T < T_L\ (6s)$

$S_{D1} = \frac{2}{3}(1.6)(.046) = .049$

$C_s = \dfrac{.049}{1.57\left(\frac{3.5}{1.25}\right)} = .011 > 0.01$

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

$= 0.21$

$C_s = \dfrac{0.21}{(3.5/1.25)} = .075 > 0.01$

$C_s = 0.011$ in E-W direction

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

Total weight

Roof:

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

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) = 248.4k

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

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

Partitions:

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

Building Total = 18942 k

N-S

$V = C_s W = .014(18942) = 265.2\,k$ ← Controls
(Wind controls overall)

E-W

$V = C_s W = .011(18942) = 208.4\,k$

$T = 1.31\,s$

$k = 0.5 + (1.31 - 1.0)\left(\frac{2.5 - 0.5}{2 - 1}\right) = 1.12$

$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$ Sample Calc For 11th Floor

$\Sigma w_i h_i^k = 2804282.544$

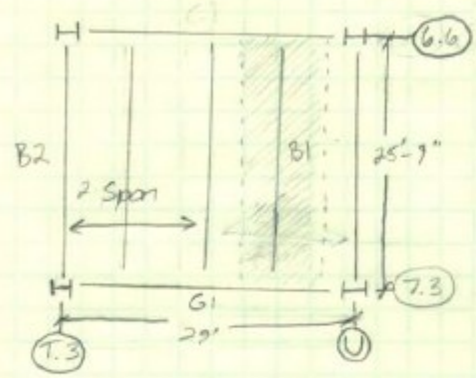
$C_{vx} = \frac{(1578.46)(130.33')^{1.12}}{2804282.544} = 0.141821$

$F = C_{vx} V = .141821(265.2) = 37.61\,k$

Full calcs in spreadsheet in Seismic section
including over turning moments

Nick Reed | Tech. Report 1 | Spot Checks 1/4

Typical Bay (Hotel Room, Floor 6)



G1: W24x55
B1: W16x26
B2: W14x82

$\frac{14.5}{2} = 7.25'$

Composite Deck, 6.5" total depth
(2 span, NWC, 2VLI20, allow span = 8'9")

Loads

Deck = 69 psf
SDL = 10(susp. mech.) + 5(ceiling) = 15 psf
Beam Self WT = 5 psf
LL = 80 psf (includes partitions according to specs.)

Beam B1

$A_t = 25.75' \times 7.25'$

$A_t = 186.7\ ft^2$

$K_{LL} = 2 \Rightarrow K_{LL}A_T = 2(186.7) = 373.4\ ft^2$ No reduction needed

Total DL = 89

$W_u = 1.2(89) + 1.6(80) = 234.8\ psf$

$W_u = 234.8(7.25')/1000$

$W_u = 1.70\ klf$

$M_u = \frac{w_u l^2}{8} = \frac{1.70(25.75)^2}{8} = 140.9\ ft\cdot k < 166\ ft\cdot k$

Drawing show 1 stud/rib, assume 3/4"φ, Deck ⊥ to beam, weak pos.

$f'c = 3500\ psi \Rightarrow Q_n = 17.2\ k$ (Table 3-21)

b_{eff}

$b_1' = \min\left[\begin{array}{l} 25.75(12)/8 \\ \frac{7.25(12)}{2} \end{array}\right] = \min\left[\begin{array}{l} 38.63" \\ 43.5" \end{array}\right]$ $\quad b_2' = b_1'$

$b_{eff} = 77.26"$

Nick Reed | Tech Report 1 | Spot Checks 2/4

Beam B1 cont'd

Let a = 1.0 $Y2 = t - a/2 = 6.5 - 1/2 = 6''$

$A_{sc} F_u = \pi((3/8))^2(65) = 28.72k$

$F_u = 65$ ksi

$R_p = 0.6$ (weak pos.) $Q_n = \min \begin{cases} 17.2 \\ (1.0)(0.6)(28.72) = 17.23 \end{cases}$

$R_g = 1.0$

W16x26 $M_u = 311$ ft·k (Table 3-19)

$\Sigma Q_n = 194$ k

$194/17.2 = 11.3 \Rightarrow 12$ studs

Check a

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{194}{0.85(3.5)(77.26)} = .84 < 1'' \text{ OK}$$

Deflections

$W_{LL} = 80 \times 7.25 = .580$ klf

$I_{LB} = 795$ (Table 3-20)

$$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(.580)(25.75)^4(1728)}{384(29000)(795)} = 0.25''$$

$$\Delta_{allow} = \frac{L}{360} = \frac{25.75(12)}{360} = 0.86'' > .25 \text{ OK}$$

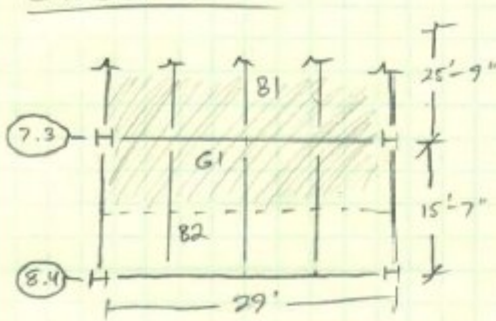
$$\Delta_{TL} = \frac{5(1.70)(25.75)^4(1728)}{384(29000)(795)} = 0.73''$$

$$\frac{L}{240} = \frac{25.75(12)}{240} = 1.29'' > 0.73'' \text{ OK}$$

$\boxed{\text{W16x26 is adequate for loads}}$

Nick Reed | Tech Report 1 | Spot Checks 3/4

Girder (G1)



A smaller bay frames into G1 (corridor)

From B1 calcs,

$w_u = 1.7$ klf

$P_u = \frac{1.7(25.75)}{2} = 21.89$ k

B2 loads

Deck = 69 psf

SDL = 15 psf

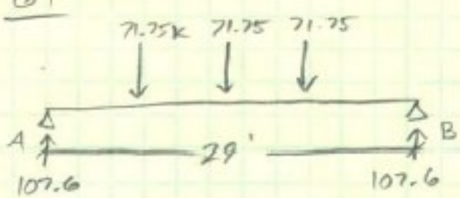
B.S.W = 5 psf

LL = 100 psf

$w_u = 1.2(5175\text{psf}) + 1.6(100) = 6.4$ klf

$P_u = \frac{6.4(15.58)}{2} = 49.86$ k

G1



Simplified Reaction EQ's Table 3-22a

$R_A, R_B = 1.5(71.75) = 107.6$ k

$M = 0.5(71.75)(29')$

$M = 1040$ ft·k >> 503

Check composite

$b_1' = \min\left[\begin{array}{l}\frac{29(12)}{8} = 43.5 \\ \frac{25.75(12)}{2} = 154.5\end{array}\right.$

$b_2' = \min\left[\begin{array}{l}\frac{29(12)}{8} = 43.5 \\ \frac{15.58(12)}{2} = 93.5\end{array}\right.$

$b_{eff} = 87"$

Let $a = 1.0$, $Y2 = 6"$, $A_{sc}F_u = 28.72$k

Table 3-19, W24x55 $M_u = 915$ ft·k

$\Sigma Q_n = 456$ K

studs $= 456/17.2 = 26$ studs

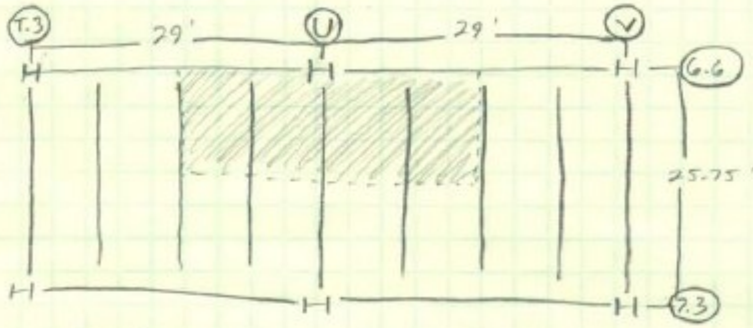
Check a

$a = \frac{456}{0.85(3.5)(87)} = 1.8 > 1$ NG

No need to check deflections

Nick Reed | Tech Report 1 | Spot Checks 4/4

Column (4th Floor)



$A_T = (29')(12.88') = 373.5\ ft^2$

$K_{LL} = 4$ (Ext. column w/o cant.)

$K_{LL} A_T = 4(373.5) = 1494\ ft^2 > 400\ ft^2$ Reduce LL

LL = 80 psf in hotel rooms

$L = 80\left[.25 + \frac{15}{\sqrt{1494}}\right] = 51\ psf$

Dead Loads

SDL = 10 + 5 + 15 = 30 psf (facade)

Deck = 69 psf

Total = 99 psf ≈ 100 psf

Snow = 40 psf

See spreadsheet for total loads on floors above 4th

Mech Live Load = 200 psf

$L = 200\left[.25 + \frac{15}{\sqrt{1494}}\right] = 127.6\ psf$

Load combo = 1.2D + 1.6L + 0.5S