TECHNICAL REPORT I

STRUCTURAL CONCEPTS/STRUCTURAL EXISTING CONDITIONS

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October 24, 2008

TABLE OF CONTENTS

EXECUTIVE SUMMARY

Technical Report I is an existing conditions report for the Layfield Tower, a hospital expansion in Salisbury, Maryland. In this report descriptions are made of the building codes, materials and the structural systems. From this information it was then possible to perform a structural analysis of the building. The building was checked for dead and live loading, snow loading, wind loading, and seismic loading. Also, a spot check was performed on an interior beam. The results were compared with those of the original design and found to be within a reasonable tolerance. Many of the results had slight differences with the original design and this is likely because of minor differences in the assumptions made.

INTRODUCTION

The Layfield Tower is part of an expansion and renovation project at Peninsula Regional Medical Center. It is located at 100 East Carroll Street in Salisbury, MD. It is a 200,000 square foot facility that will house a new emergency/trauma center, pediatric unit, intensive care unit, cardiac and thoracic and vascular unit and a neurosciences and stroke unit. The building also features a helipad on the lower roof with access to the third floor of the main tower. There is a connection to the existing hospital at the northeast corner. Construction on Layfield Tower was completed in 2008.

The structure is divided into two parts: the east side (Area A) with three stories and the west (Area B) with one story. An expansion joint connects the two sections of the building.

This report will give an overview of the existing structural system of the building as well as a general structural analysis.

CODES AND MATERIAL PROPERTIES

Codes

The structural design of the Layfield Tower conforms to the requirements of the Maryland Building Performance Standards (MBPS) which has adopted the 2003 International Building Code (IBC) and ASCE 7-02. Structural steel design used the AISC Manual of Steel Construction Load and Resistance Factor Design, Second Edition, 2003. Concrete design used American Concrete Institute, ACI 318-02.

For this report, the latest versions of these codes were used. IBC 2006 and ASCE 7-05 were used for design loads and structural analysis. ACI 318-08 was used for structural concrete design and AISC Manual of Steel Construction, Load and Resistance Factor Design, Fourth Edition 2007 for structural steel.

Material Properties

Steel Members	
W-Shapes	ASTM A992, Grade 50
Channels, Angles, Plates, Bars	ASTM A36 or A572, Grade 50
HSS Sections	ASTM A500, Grade B
Structural Pipe	ASTM A53, Type E or S, Grade B
Braced Frame Members	ASTM A992 or A36
Steel Reinforcement	Grade 60

Concrete		
Footings	3000 psi	145 pcf
Slab-on-grade	3500 psi	145 pcf
Foundation walls	4000 psi	145 pcf
Suspended slabs	4000 psi	145 pcf
Slabs on Metal Deck	3000 psi	115 pcf
Building frame members	4000 psi	145 pcf
Building walls	4000 psi	145 pcf
Precast panels	5000 psi	145 pcf or 115 pcf

STRUCTURAL SYSTEM

Foundation

The foundation of Layfield Tower consists of cast-in-place reinforced concrete walls with spread footings along the perimeter of the building and spread footings underneath all interior columns. The wall footings vary from 6’-0” to 24’-0” and are 24 inches thick. Typical column footings in Area A are 12’-6” x 12’-6” and 35 inches thick. In Area B typical column footings are 8’-0” x 8’-0” and 24 inches thick. On the south side of both Areas A and B the basement floor is either 2’-0” or 6’0” below the basement floor on the north so the footings on the south side are all lower than those on the north side.

Superstructure

The main structural system is made up of structural steel W-shape members. Most connections are shear connections. The typical beam size in Area A (Figure 1) is W18x35 space at 10’-0” on center and in Area B (Figure 2) it is W 18x35 also spaced at 10’-0” on center. Girders are typically W21x50 in both areas. Columns in Area A are various W12 sizes. In Area B the typical column size is W12x53. The most typical bay is 30’0” by 30’0”, but there are also column spacings of 28’0”, 27’-8”, and 26’0”.

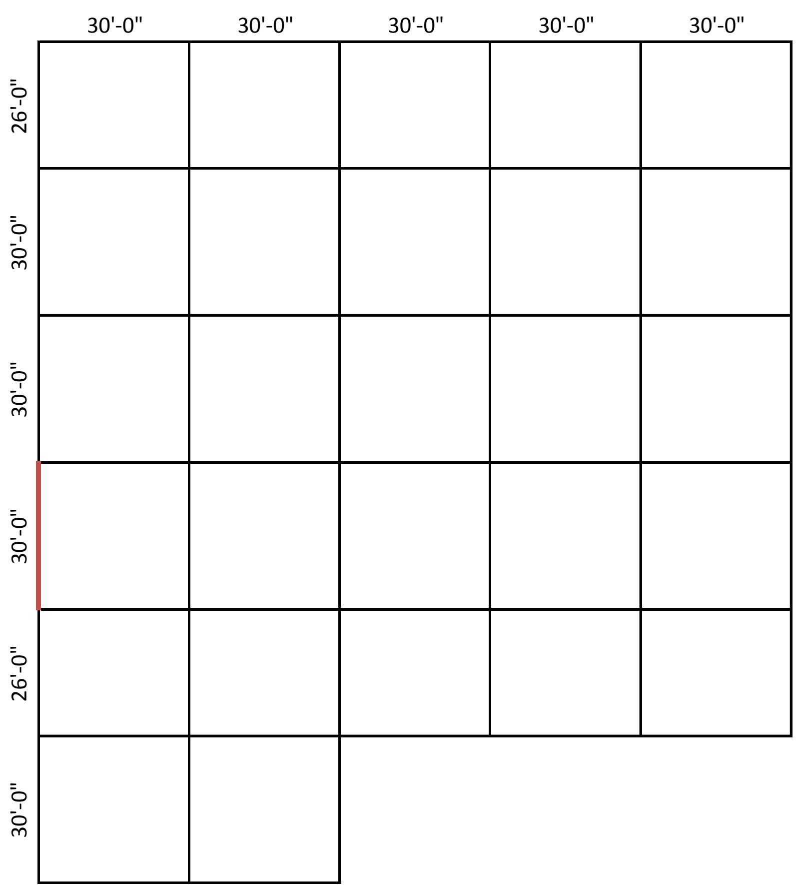


Figure 1, Area A

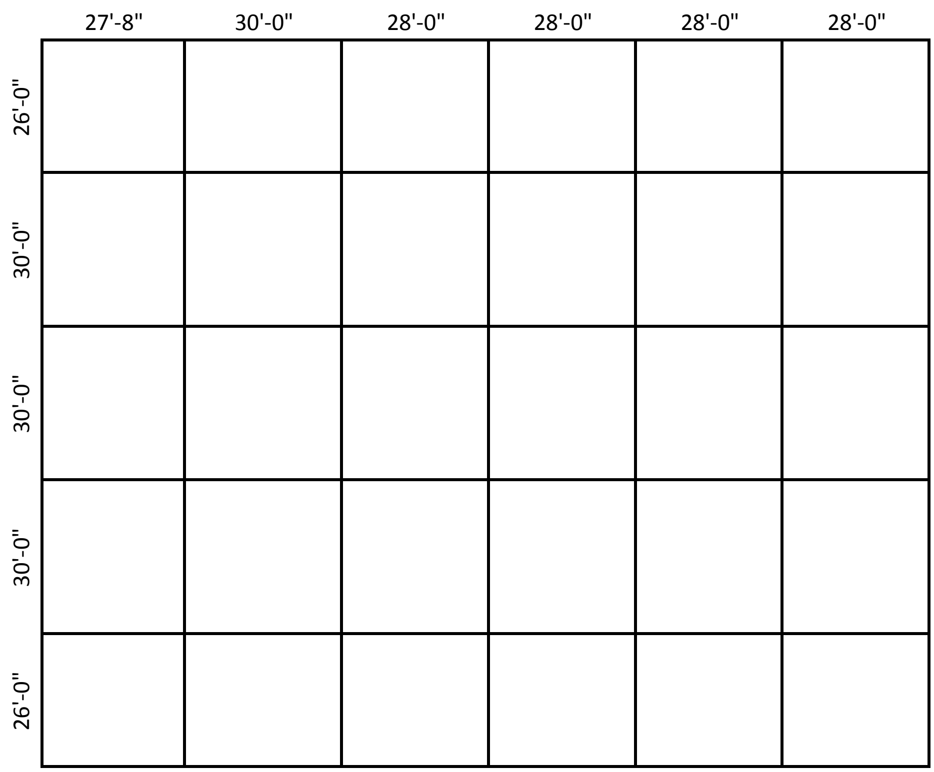


Figure 2, Area B

The north canopy is has a lower canopy comprised of W10x33 and W12x60members and a cantilevered canopy with w10x33's. Its columns are either w10X33 or W12x106. The south canopy has W24x68 girders and W12x35 beams. The south canopy columns are HSS10x10x5/8.

Floor System

Floor slabs are 3-1/4" lightweight concrete on 3" deep 20 gage, galvanized composite metal deck for a total thickness of 6-1/4". They are reinforced with 6x6 W2.1xW2.1 welded wire fabric. All shear studs are 3/4" x 5 3/16". The floor slab of the connector corridor is 4-1/2" normal weight concrete on 3" deck.

Roof System

The roof structural system of Area A is similar to the other floors. The roof of Area B has a typical beam size of W16x26 spaced at 10'-0 on center. Girders are similar to the rest of the building.

Lateral System

The lateral structural system is composed of braced frames, one in each direction. W12's are the typical members for the braced frames. All of the main frames are one bay wide, extending the full height of the building, and most are located along the perimeter walls of both Areas A and B. In Area A there is one near the elevator shafts located in the center of the building. Figures 3 and 4 show the locations of the braced frames by the orange highlighted lines. All penthouses as well as the heliport are braced along all sides.

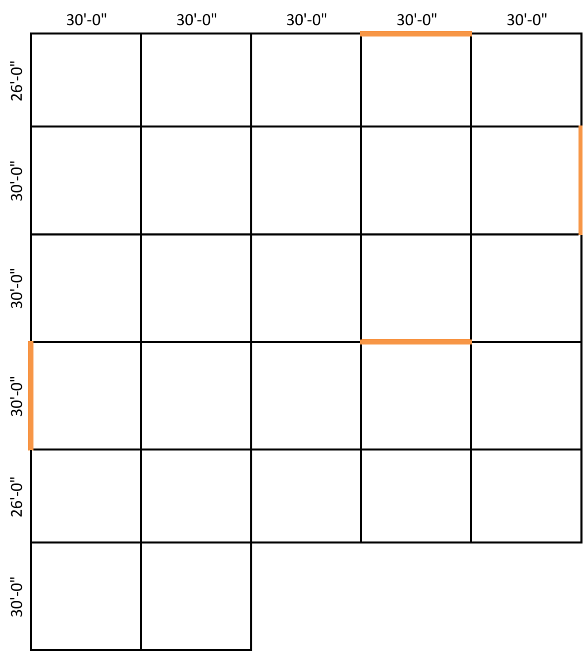


Figure 3, Lateral Braces Area A

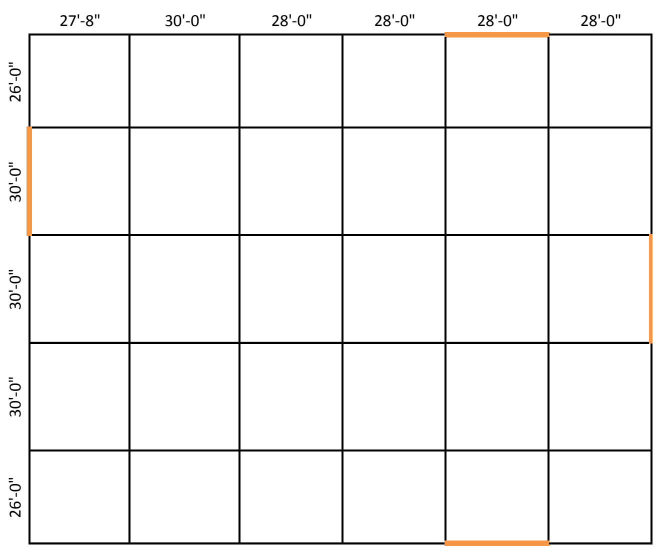


Figure 4, Lateral Braces Area B

STRUCTURAL ANALYSIS

Dead Loads

Floor Area	Load (psf)
Partitions	20
Suspended Ceilings	3
Ductwork and Piping	5
Lights	2
Sprinklers	2
Fireproofing	2
Structural Steel Framing	8
6 1/4" Floor Slab (LW)	47
7 1/2" Floor Slab (NW)	75
Hanging Load in Mechanical Rooms	65

Roof Area	Load(psf)
Suspended Ceilings	3
Ductwork and Piping	5
Lights	2
Sprinklers	2
Fireproofing	2
Structural Steel Framing	8
Metal Roof Deck	2
Roofing and Insulation	12
Re-roofing Allowance	5
Joists and Bridging	3
6 1/4" Floor Slab (LW)	47
Hanging Load in Mechanical Rooms	65

Live Loads

Floor Area	Load(psf)
Elevator Penthouse	150
Mechanical Rooms	15
Office Areas	50
Toilets	60
Corridors	80
Minimum for Design	80

Roof Area	Load(psf)
Minimum	20
Rain	26
Rain-on-Snow Surcharge	5

Snow Load

Snow load was calculated in accordance with section 7 of ASCE 7-05. The roofs were also checked unbalanced snow loads and drifting snow.

50-year Recurrence Ground Snow Load, p_g	20 psf
Exposure Factor, C_e	0.9
Thermal Factor, C_t	1.0
Importance Factor, I	1.2
Flat-Roof Snow Load, p_f	24 psf

Wind Loads

Wind loads were designed in accordance with section 6 of ASCE 7-05. Areas A and B are separated by an expansion joint so they were treated as individual buildings in this analysis. The connecting corridor is also assumed to be part of Area A so the projected surface area in the north-south direction is larger than actual. Also, interference of the wind forces from the existing hospital structures is neglected. These assumptions will produce conservative results in the analysis. Method 2 was used for the analysis of Area A. Below is a list of the factors obtained based on the assumptions.

- Basic Wind Speed, V = 110 mph
- Wind Directionality Factor, K_d = 0.85
- Building category = IV
- Importance Factor = 1.15
- Exposure Category = C
- Topographic Factor, K_{zt} = 1.0
- External Pressure Coefficient, $C_{p,w}$ = 0.8
- External Pressure Coefficient, $C_{p,l}$ = -0.5
- External Pressure Coefficient, $C_{p,s}$ = -0.7

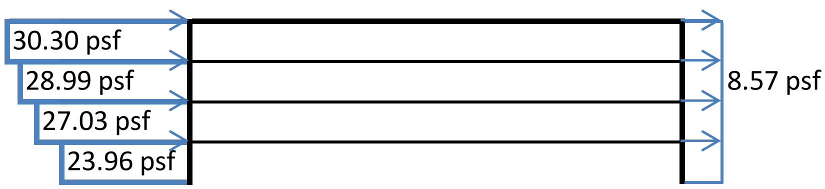


Figure 5. East-West Wind Pressures

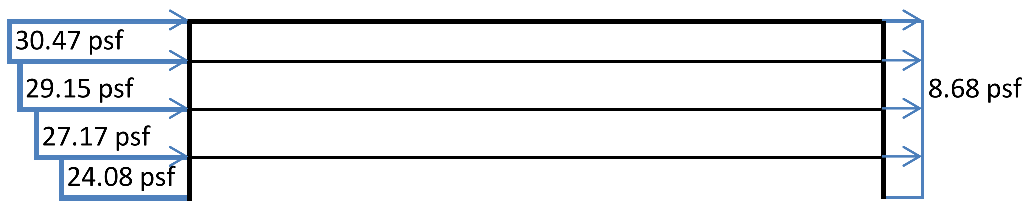


Figure 6, North-South Wind Pressures

Method 1 was used for Area B since its mean roof height is only 16’-0”. Below is a list of the factors used in this analysis.

- Basic Wind Speed, V = 110 mph
- Importance Factor, I = 1.15
- Exposure Category = C
- Mean Roof Height = 16’-0”
- Adjustment Factor, λ = 1.22

Horizontal Pressures (psf)				**Vertical Pressures (psf)**			
A	B	C	D	E	F	G	H
19.2	-10	12.7	-5.9	-23.1	-13.1	-16	-10.1
Adjusted Pressures (psf)				**Adjusted Pressures (psf)**			
23.424	-12.2	15.494	-7.198	-28.182	-15.982	-19.52	-12.322

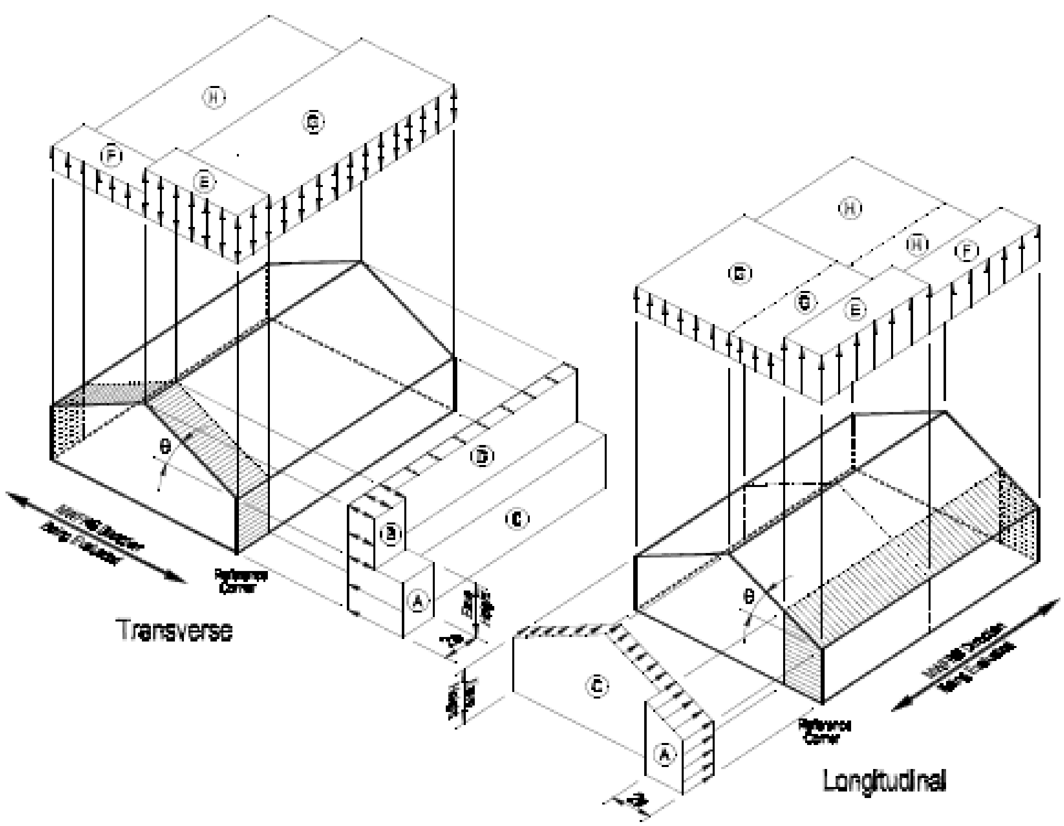


Figure 7, ASCE-7 Fig. 6-2 MWFRS Method 1

Seismic Loads

Seismic Loads were designed in accordance with sections 11 and 12 of ASCE 7-05. The geotechnical information was unavailable for this report so the site class was assumed to be site class D because that is what was used in the original design of the building. The short period spectral response acceleration, S_s, and the one-second period acceleration, S_1, were found using the USGS Seismic Design Values for Buildings found at http://earthquake.usgs.gov/research/hazmaps/design. By doing so, these values are lower than those used by the structural engineer and thus conservative in comparison.

- Ss = 0.124
- S1 = 0.045
- SDS = 0.132
- SD1 = 0.072
- Seismic Design Category C
- Response Modification Factor, R = 3
- Importance Factor, I = 1.5

	Area A	Area B
Base Shear	630 kips	592 kips
Overturning Moment	79061 ft-kips	24234 ft-kips

CONCLUSION

The analysis performed was found to be comparable to the original analysis. Some assumptions may have caused minor differences in some of the results. Building Category IV was used in the analysis while the original design was a category III. This seemed to have no effect on the outcome of the calculations. Wind and seismic loads were found to be within a five percent tolerance of the results found by the structural engineer. A spot check was performed on the typical interior beam which was sized just as in the original design.

APPENDIX

Snow Load

p_g = 20 psf (Figure 7-1)
C_e = 0.9 (Table 7-2)
C_t =1.0 (Table 7-3)
I = 1.2 (Table 7-4)
$P_g \leq 20$ psf, therefore $p_f = Ip_g$
$p_f = Ip_g = 1.2(20) = 24$ psf

Rain Load

$R = 5.2(d_s + d_h)$
d_s = 4.0 in.
d_h = 1.0 in.
$R = 5.2(4.0 + 1.0) = 26.0$ psf

WIND LOADS | AREA A | North-South

Mean Roof Height, $h = 70$ ft ∴ Use Method 2

Basic Wind Speed, $V = 110$ mph

$B = 223.4'$

$L = 172'$

$K_d = 0.85$

Bldg. Category = IV

Importance Factor, $I = 1.15$

Surface Roughness = C

Exposure = C

$K_{zt} = 1.0$

$n_1 = \frac{75}{H} = \frac{75}{70} = 1.07\ Hz > 1.0\ Hz$ ∴ Bldg is rigid

Conservative

$g_Q = 3.4$

$g_v = 3.4$

$\bar{z} = 0.6h = 0.6(70) = 42\text{ ft.} > z_{min} = 15\text{ ft}$ ∴ Use 42 ft.

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$

$c = 0.20$

$= 0.20\left(\frac{33}{42}\right)^{1/6} = 0.1921$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$

$\ell = 500'$

$\bar{\epsilon} = 1/5.0$

$= 500\left(\frac{42}{33}\right)^{1/5.0} = 524.71$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{223.4+70}{524.71}\right)^{0.63}}} = 0.8343$$

$$G = 0.925\left(\frac{1+1.7g_Q I_{\bar{z}} Q}{1+1.7g_v I_{\bar{z}}}\right) = 0.925\left(\frac{1+1.7(3.4)(0.1921)(0.8343)}{1+1.7(3.4)(0.1921)}\right)$$

$= 0.8444$

Area A, North-South Wind Loads

Location	Height(ft)	K_z, K_h	q_z	External Pressure			Internal Pressure			Combined Pressure	
				G	C_p	qG_fC_p	q_i	GC_{pi}	q_iGC_{pi}	$(+Gc_{pi})$	$(-Gc_{pi})$
Windward	70	1.17	35.43	0.844	0.8	23.92	35.43	0.18	6.377	17.54	30.30
	54	1.106	33.49	0.844	0.8	22.61	35.43	0.18	6.377	16.23	28.99
	35	1.01	30.58	0.844	0.8	20.65	35.43	0.18	6.377	14.27	27.03
	16	0.86	26.04	0.844	0.8	17.58	35.43	0.18	6.377	11.21	23.96
Leeward	ALL	1.17	35.43	0.844	-0.5	-14.95	35.43	-0.18	-6.377	-21.33	-8.57
Side Wall	ALL	1.17	35.43	0.844	-0.7	-20.93	35.43	-0.18	-6.377	-27.31	-14.55

Area A, East-West Wind Loads

Gust effect factor assumed to be 0.85, because structure is rigid.

Location	Height(ft)	K_z, K_h	q_z	External Pressure			Internal Pressure			Combined Pressure	
				G_f	C_p	qG_fC_p	q_i	GC_{pi}	q_iGC_{pi}	$(+Gc_{pi})$	$(-Gc_{pi})$
Windward	70	1.17	35.43	0.850	0.8	24.09	35.43	0.18	6.377	17.71	30.47
	54	1.106	33.49	0.850	0.8	22.77	35.43	0.18	6.377	16.40	29.15
	35	1.01	30.58	0.850	0.8	20.80	35.43	0.18	6.377	14.42	27.17
	16	0.86	26.04	0.850	0.8	17.71	35.43	0.18	6.377	11.33	24.08
Leeward	ALL	1.17	35.43	0.850	-0.5	-15.06	35.43	-0.18	-6.377	-21.43	-8.68
Side Wall	ALL	1.17	35.43	0.850	-0.7	-21.08	35.43	-0.18	-6.377	-27.46	-14.70

SEISMIC

Site Class = D

Seismic Use Group = III

$I = 1.5$

$S_S = 0.124$ (from USGS)

$S_1 = 0.045$

$F_a = 1.6$

$F_v = 2.4$

$S_{MS} = F_a S_S = 1.6(0.124) = 0.198$

$S_{M1} = F_v S_1 = 2.4(0.045) = 0.108$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.198) = 0.132g < 0.167g \therefore$ SDC-A

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.108) = 0.072g > 0.067g \therefore$ SDC-C ← CONTROLS

Seismic Design Category → C

$R = 3 \quad T_L = 8$

AREA A

$T = \frac{1}{n_1} = \frac{1}{0.742} = 1.348$

$T_a = C_t h_n^x$

$C_t = 0.02$

$x = 0.75$

$= 0.02(70)^{0.75}$

$= 0.484$

$C_u = 1.7$

$C_u T_a = 1.7(0.484) = 0.8228$ ← CONTROLS

$$C_S \geq \begin{cases} S_{DS}/(R/I) = \frac{0.132}{(3/1.5)} = 0.066 \\ S_{D1}/[T(R/I)] = \frac{0.072}{0.823(\frac{3}{1.5})} = 0.0437 \leftarrow \text{controls} \\ \frac{S_{D1} T_L}{T^2(R/I)} = \frac{0.067(8)}{(0.823)^2(\frac{3}{1.5})} = 0.396 \end{cases}$$

AREA B

$T = \frac{1}{2.416} = 0.414$

$T_a = 0.02(76)^{0.75}$

$= 0.160$

$C_u = 1.7$

$C_u T_a = 1.7(0.16) = 0.272$ ← CONTROL

$$C_S \geq \begin{cases} \frac{0.132}{3/1.5} = 0.066 \leftarrow \text{controls} \\ \frac{0.072}{0.272(3/1.5)} = 0.132 \\ \frac{0.067(8)}{(0.272)^2(3/1.5)} = 3.62 \end{cases}$$

Conterminous 48 States
2005 ASCE 7 Standard
Zip Code = 21801
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.05 deg grid spacing

Period (sec)	Centroid Sa (g)	
0.2	0.120	(Ss, Site Class B)
1.0	0.044	(S1, Site Class B)

Period (sec)	Maximum Sa (g)	
0.2	0.124	(Ss, Site Class B)
1.0	0.045	(S1, Site Class B)

Period (sec)	Minimum Sa (g)	
0.2	0.118	(Ss, Site Class B)
1.0	0.044	(S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Zip Code = 21801
Spectral Response Accelerations SMs and SM1
SMs = FaSs and SM1 = FvS1
Site Class D - Fa = 1.6 ,Fv = 2.4

Period (sec)	Sa (g)	
0.2	0.192	(SMs, Site Class D)
1.0	0.107	(SM1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Zip Code = 21801
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.6 ,Fv = 2.4

Period (sec)	Sa (g)	
0.2	0.128	(SDs, site Class D)
1.0	0.071	(SD1, site Class D)

Area A Seismic Calculations

Ss	0.124
S1	0.045
Fa	1.6
Fv	2.4
Sms	0.148
Sm1	0.108
SDs	0.132
SD1	0.072
SDC	C
T	1.348
Ta	0.484
Cu	1.7
CuTa	0.8228
Cs	0.0437

Floor	Height (ft.)	Area (sf)	Load (ksf)	Weight (kips)	k	$w_x h_x^k$	C_{vx}	F_x	Moment ft-kips
1	0	23319	0.154	3591	1.424				35291
2	16	23319	0.154	3591	1.424	186165	0.0556	35.0	25215
3	35	23319	0.154	3591	1.424	567523	0.1695	106.7	13914
5	54	23319	0.154	3591	1.424	1052348	0.3143	197.9	4642
Roof	70	23319	0.156	3638	1.424	1542603	0.4607	290.1	0
			Total =	14411		3348639		629.8	79061

Overturning Moment 79061

Base Shear 630 kips

Area B Seismic Calculations

Ss	0.124
S1	0.045
Fa	1.6
Fv	2.4
Sms	0.148
Sm1	0.108
SDs	0.132
SD1	0.072
SDC	C
T	0.414
Ta	0.16
Cu	1.7
CuTa	0.272
Cs	0.066

Floor		Area (sf)	Load (ksf)	Weight (kips)	k	$w_x h_x^k$	C_{vx}	F_x	Moment ft-kips
1	0	24093	0.154	3710	2				16853
Roof	16	24093	0.156	3759	2	962178	0.3437	203	7382
Helipad	35			1500	2	1837500	0.6563	389	0
			Total =	8969		2799678		592	24234

Base Shear 592 kips

Typ. Beam Spot Check — Tower Part A

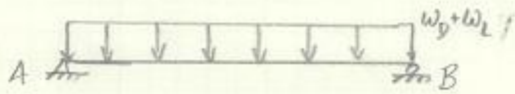
Composite steel

$f'_c = 3000$ psi

spacing = 10'

span = 30'

$t_{slab} = 6.25''$



$w_D = 89$ psf

$w_L = 80$ psf

$w_u = 1.2w_D + 1.6w_L = 1.2(89) + 1.6(80) = 234.8 \text{ psf}(10) = 2.35 \text{ k/ft}$

$M_u = \frac{w_u l^2}{8} = \frac{2.35(30)^2}{8} = 264.4 \text{ ft·k}$

$\Delta_{max} = \frac{l}{300} = \frac{30(12)}{300} = 1.2''$

$\Delta = \frac{5 w_L l^4}{384 EI} \Rightarrow I = \frac{5(0.80)(30)^4(1728)}{384(29000)(1.2)} = 419 \text{ in}^4$

From Table 5-2

Use W18×35, $I = 510 \text{ in}^4 > 419 \text{ in}^4$

$R_A = \frac{w_u l}{2} = \frac{2.35(30)}{2} = 35.25^k$