ASHA National Office

Rockville, MD

Technical Report I

Photo Courtesy of Boggs & Partners Architects

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Structures Option

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Executive Summary

In this technical report, the existing structural system of the ASHA National Office building is discussed and analyzed. The report includes a detailed description of the building's structural system. The ASHA National Office is a five story office building with two floors of subgrade parking. The substructure is composed of a flat slab system with drop panels and the superstructure is composite steel. Images are used to allow for a better understanding of the system and its components. A list of building codes and standards used to design the building are also included in this report. The properties and strengths of the materials used for the structure of the building are also provided.

The wind and seismic loads were analyzed for the building using ASCE 7-10. The MWFRS Directional Procedure was used to determine the wind loads on the building in both directions. Seismic loads were calculated using The Equivalent Lateral Force Procedure. The wind loads in the North-South direction were found to control the design of the structure with a base shear of 435 kips.

Spot checks were done on a beam, girder and column to verify the sizes that were chosen by the structural engineer. A typical composite wide flange beam and girder on the second floor were checked, and a typical steel column was checked below the second floor and below the fourth floor. This was done because the columns are spliced above the third floor. The spot checked members were determined to be adequate for the gravity loads.

The appendix includes the hand calculations done for the snow, wind and seismic loads. The detailed spot check calculations are also shown. Typical framing plans are also provided in the appendix to help describe the structural framing of the building.

Introduction

The ASHA National Office building is a five story office building in Rockville, MD. The American Speech-Language-Hearing Association owns and operates the building. The building was designed with the employees in mind. There is a generous amount of workspace for the employees and the conference rooms are very flexible. A café and kitchen are provided for the employees on the first floor of the office building. There are two levels of subgrade parking beneath the building in addition to surface parking with a total of 446 spaces.

One of the main architectural themes that Boggs & Partners incorporated throughout the building is curves. This was done to mimic the sound waves in the ASHA logo. The pre-function space has the curve incorporated into it, and there is a curved piece of art on the landing of the stairway that leads from the lobby to the second floor. The exterior façade has large three story curved glass curtain wall above the main entrance, and the sidewalks on the exterior of the building are curved as well to further emphasize the main theme of the building.

The five story office building has a total floor area of 133,870 square feet and the roof the building is 69 feet above grade. The top of the penthouse roof is 85 feet above grade. The building façade of the office tower consists of a window wall system and precast concrete spandrels.

Structural System

Substructure

The substructure of the ASHA National Office building is comprised of two floors of subgrade parking. There is parking underneath the office tower along with a section of the parking structure that is adjacent to the office tower. See Figure 1: Overall Parking Floor Plan. The parking below the office tower is shown in blue and the parking adjacent to the office tower is shown in yellow.

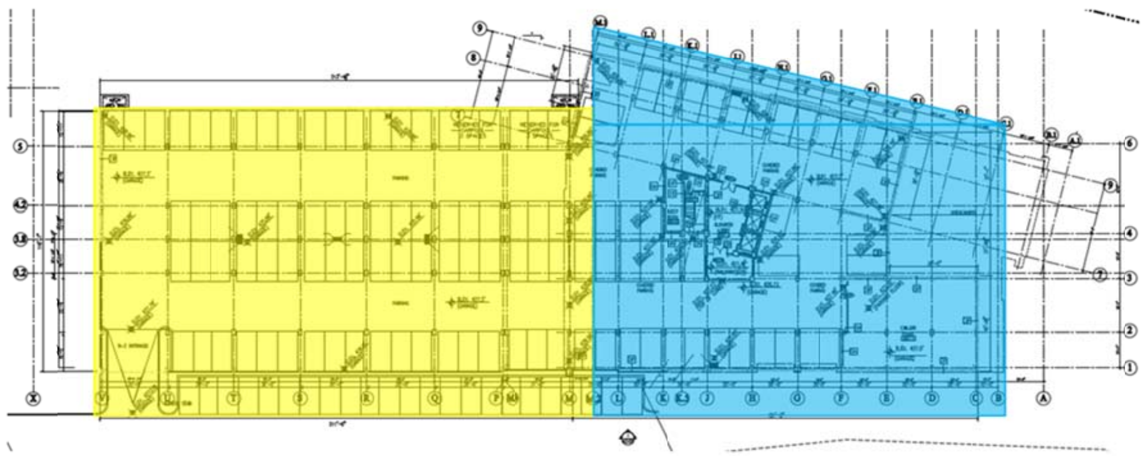


Figure 1: Overall Parking Floor Plan

Foundation

The foundation of the ASHA National Office building consists of a 5" thick reinforced concrete slab with strip footings around the perimeter of the building. There are also footings at the base of all concrete columns. The foundations for the building were designed in accordance with the recommendations included in the geotechnical report prepared by ESC Mid-Atlantic, LLC. See Figure 2: Partial Foundation Plan. The interior column footings are usually 6'x6' and range from 12" to 18" thick. See Figure 3: Column Footing Schedule.

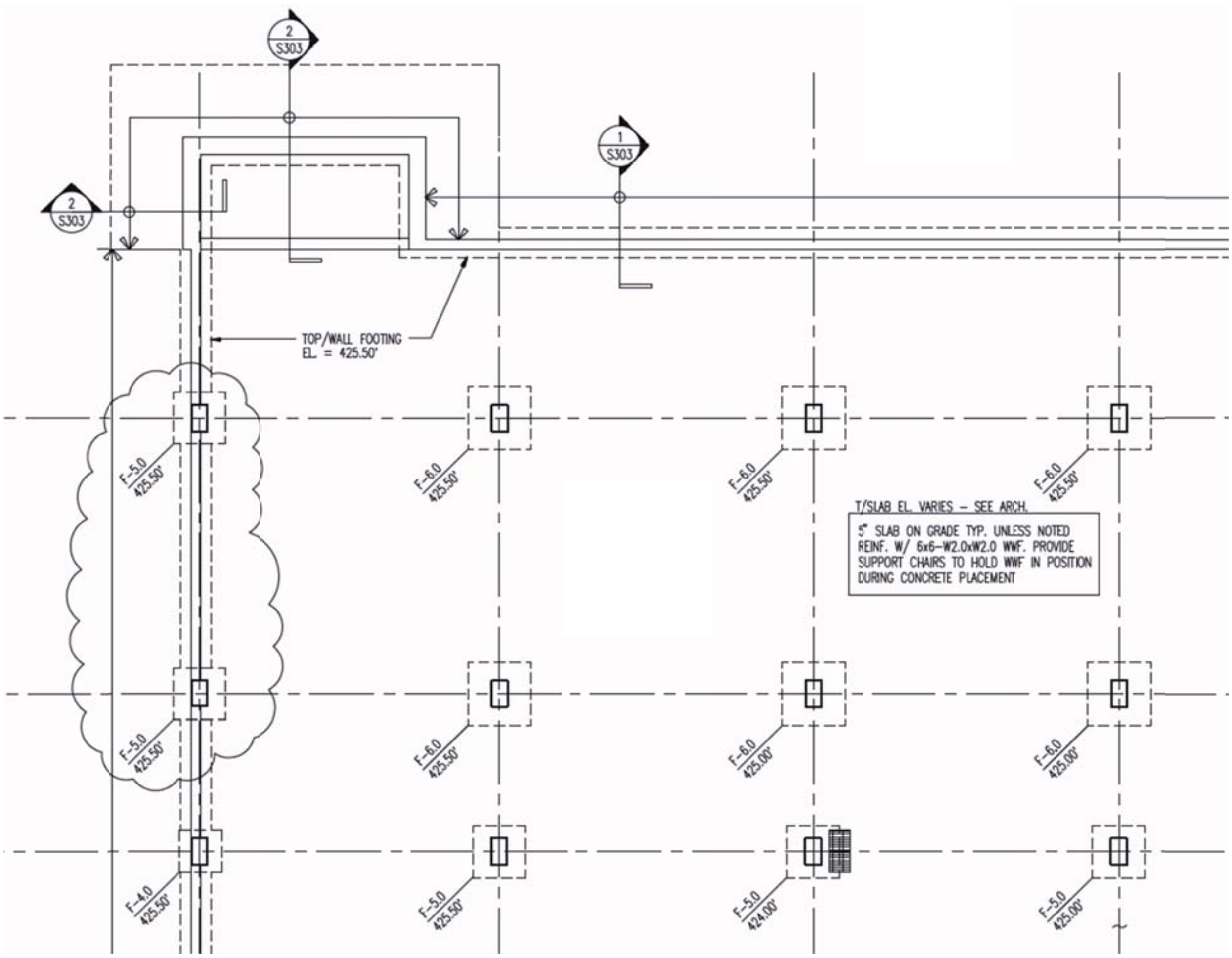


Figure 2: Partial Foundation Plan

COLUMN FOOTING SCHEDULE					
MARK	DIMENSIONS			REINFORCEMENT	REMARKS
	WIDTH	LENGTH	DEPTH		
F-4.0	4'-0"	4'-0"	12"	5#5 EWB	
F-4.5	4'-6"	4'-6"	15"	6#5 EWB	
F-5.0	5'-0"	5'-0"	15"	6#6 EWB	FOR F5.0A-SEE 2/S301 FOR F5.0B-SEE 3/S301
F-5.5	5'-6"	5'-6"	18"	7#6 EWB	
F-6.0	6'-0"	6'-0"	20"	8#6 EWB	FOR F6.0A-SEE 2/S301
F-7.0	7'-0"	7'-0"	24"	7#7 EWB	
F-7.5	7'-6"	7'-6"	26"	8#7 EWB	
F-8.0	8'-0"	8'-0"	27"	10#7 EWB	
F-8.5	8'-6"	8'-6"	29"	10#7 EWB	
F-9.0	9'-0"	9'-0"	30"	9#8 EWB	
F-9.5	9'-6"	9'-6"	31"	10#8 EWB	
F-10.0	10'-0"	10'-0"	33"	11#8 EWB	
F-10.5	10'-6"	10'-6"	36"	12#8 EWB	
F-11.0	11'-0"	11'-0"	36"	13#8 EWB	
F-3.0x8.0	3'-0"	8'-0"	18"	4#6 LWB 11#6 SWB	SEE PLAN FOR ORIENTATION

ABBREVIATIONS: EWB = EACH WAY BOTTOM EWT = EACH WAY TOP
SW = SHORT WAY LW = LONG WAY

NOTE: ALL FOOTINGS ARE DESIGNED FOR 8 KSF ALLOWABLE BEARING UNLESS OTHERWISE NOTED.

Figure 3: Column Footing Schedule

Floor Structure

The parking structure is composed of a two way reinforced concrete flat slab system that is comprised of a 9” thick slab and 5 ½” thick drop panels. Unless otherwise noted, the drop panels are 7’-0”x9’-0” and 10’-0”x10’-0”. The bay sizes vary depending on the part of the building, but the typical span ranges from 20’ to 40’. The bottom reinforcing mat consists of #5 bars at 12” or 14” each way. The top reinforcing bars vary depending on the location, but are typically #5, #6 or #7 bars. See Figure 4: Parking Level Framing Plan.

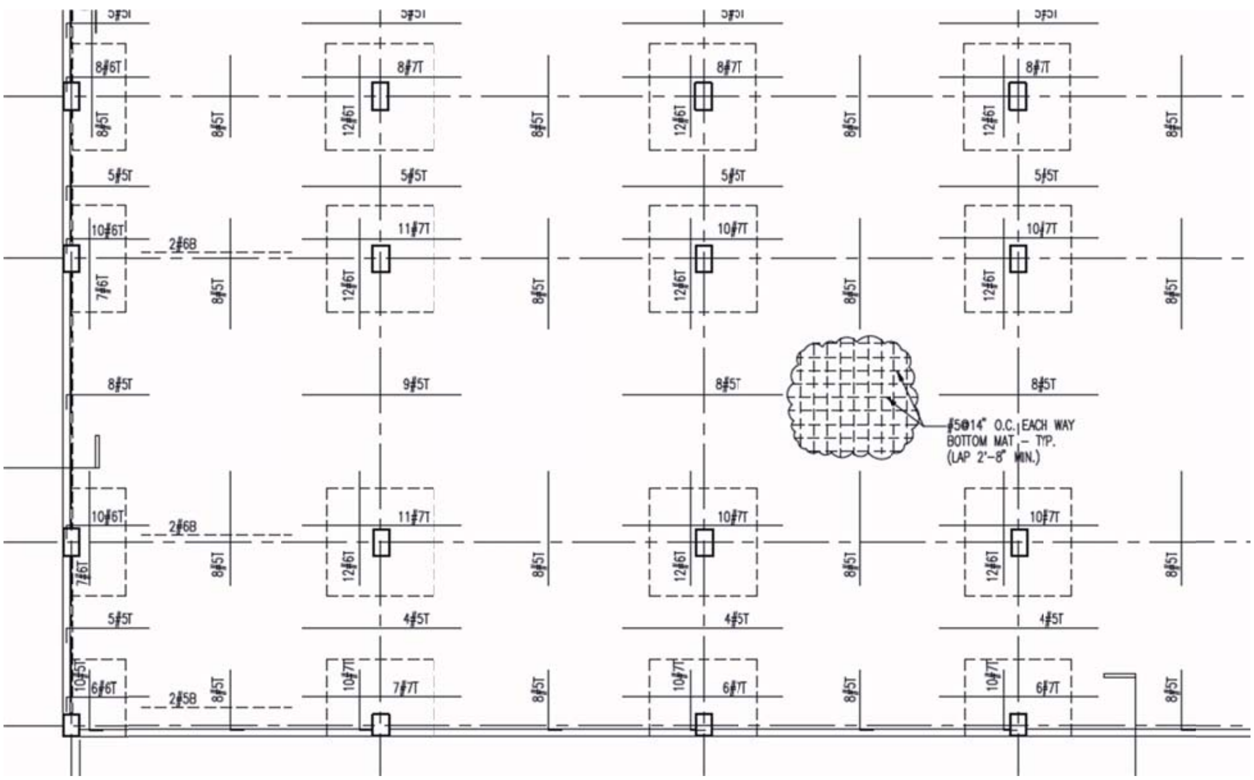


Figure 4: Parking Level Framing Plan

Columns

The concrete columns in the parking structure are generally 18"x30" with 10 #7 bars, and 24"x21" with 8 #8 bars. The columns have a minimum 28 day compressive strength of 4000 psi. See Figure 5: Partial Column Schedule. The concrete columns of the parking structure are connected to the steel columns in the office tower above with column base plates. See Figure 6: Baseplate Pocket Detail.

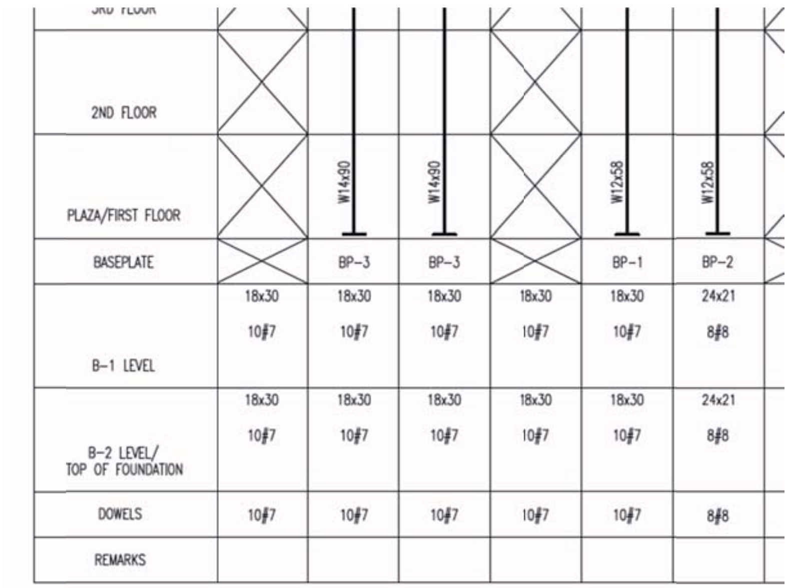


Figure 5: Partial Column Schedule

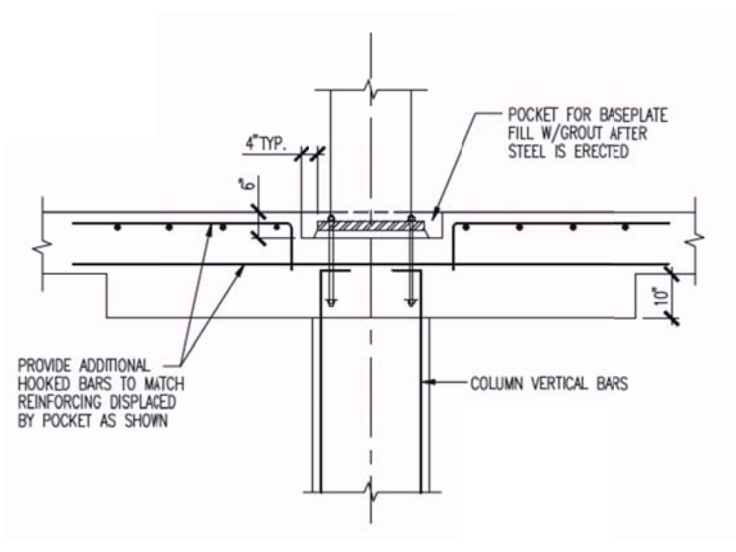


Figure 6: Baseplate Pocket Detail

Superstructure

A five story office tower is the superstructure of the ASHA National Office building. The first level has a large conference room that can be subdivided into five smaller conference rooms. The upper four floors are composed of offices in the central core of the building, and open office space with cubicles on the exterior of the building. There is a penthouse on top of the office tower that houses mechanical and elevator equipment.

Floor Structure

The floor structure for the tower consists of cambered steel beams with a composite concrete floor slab on metal deck. The composite slab consists of 3 ½” normal weight concrete on top of 2” deep 18 gauge galvanized composite steel deck. The composite beams are generally W21x44 and W14x22 members with ¾” diameter shear studs. The girders running along the exterior of the building vary in size, but are mostly W18x35 members. See Figure 7: Partial Framing Plan.

Figure 7: Partial Framing Plan

Columns

The columns for the office tower are steel wide flanges. The columns are all W12 and W14 members. The columns are spliced above level 3. The columns that extend to the penthouse roof are spliced again above level 5. See figure 8: Partial Column Schedule.

COLUMN / LEVEL	G–2	G–3	G.1–7	G.1–8	G.1–9	H–1
PENTHOUSE ROOF ROOF						
5TH FLOOR		W14x48	W14x48			
4TH FLOOR						
3RD FLOOR		W14x68	W14x68		W12x40	W12x40
2ND FLOOR						
PLAZA/FIRST FLOOR		W14x90	W14x90		W12x58	W12x58
BASEPLATE		BP–3	BP–3		BP–1	BP–2
B–1 LEVEL	18x30 10#7	18x30 10#7	18x30 10#7	18x30 10#7	18x30 10#7	24x21 8#8
B–2 LEVEL/ TOP OF FOUNDATION	18x30 10#7	18x30 10#7	18x30 10#7	18x30 10#7	18x30 10#7	24x21 8#8
DOWELS	10#7	10#7	10#7	10#7	10#7	8#8
REMARKS						

Figure 8: Partial Column Schedule

Roof System

The roof structure consists of "K" joists and wide flange members. The structural roof slab consists of 3 ½" normal weight concrete on top of 2" deep 18 gauge composite steel deck. See Figure 9: Partial roof framing plan.

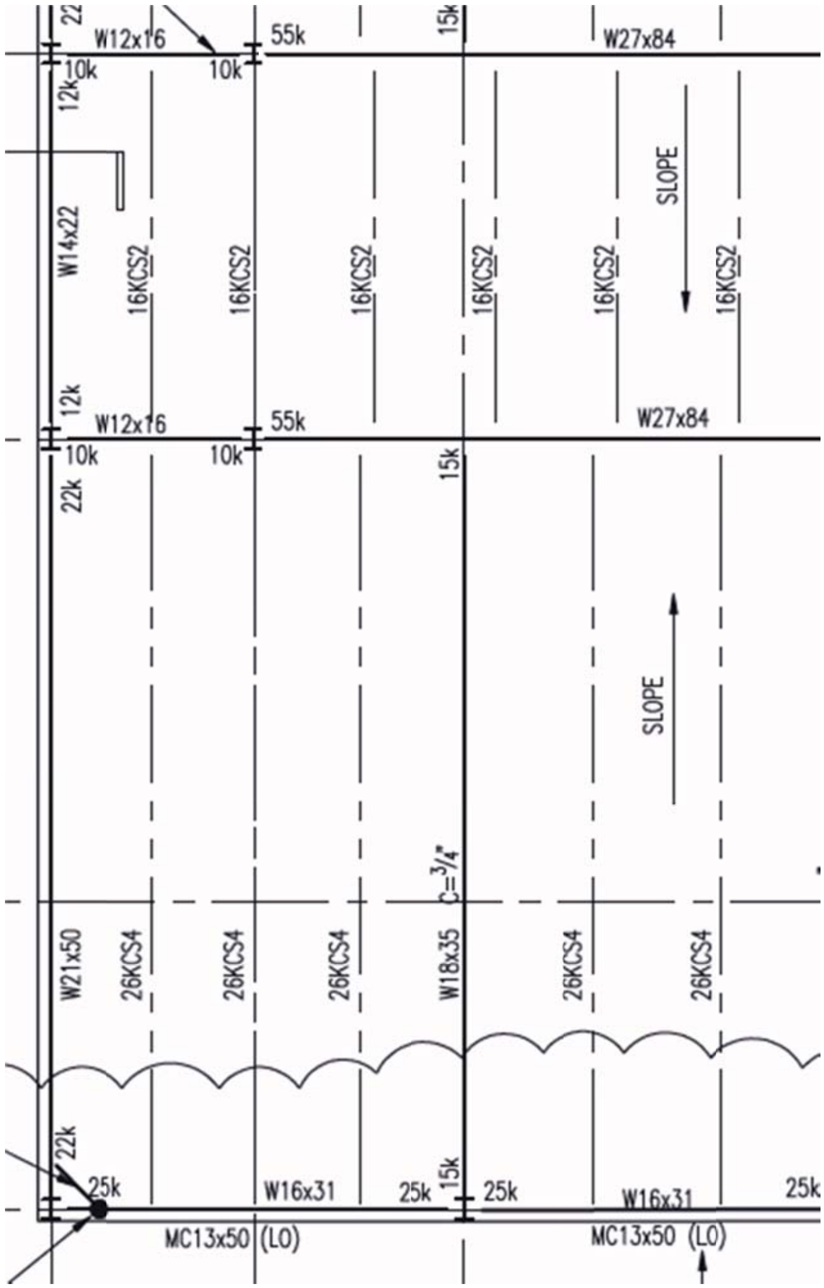


Figure 9: Partial Roof Framing Plan

Lateral System

The lateral force resisting elements in the ASHA National Office building consist of shear walls in the subgrade parking structure of the building and braced frames in the office tower. The shear walls below work in combination with the braced frames above to resist the lateral loads on the building. The wind loads are collected by the precast concrete spandrels that make up the façade of the building. These loads are then distributed to the composite floor slabs and beams which then are transmitted to the braced frames in the core of the building. These loads are then transfered to the shear walls below and to the footings at the base of the shear walls. See figure 10: Braced Frame and Shear Wall Elevation.

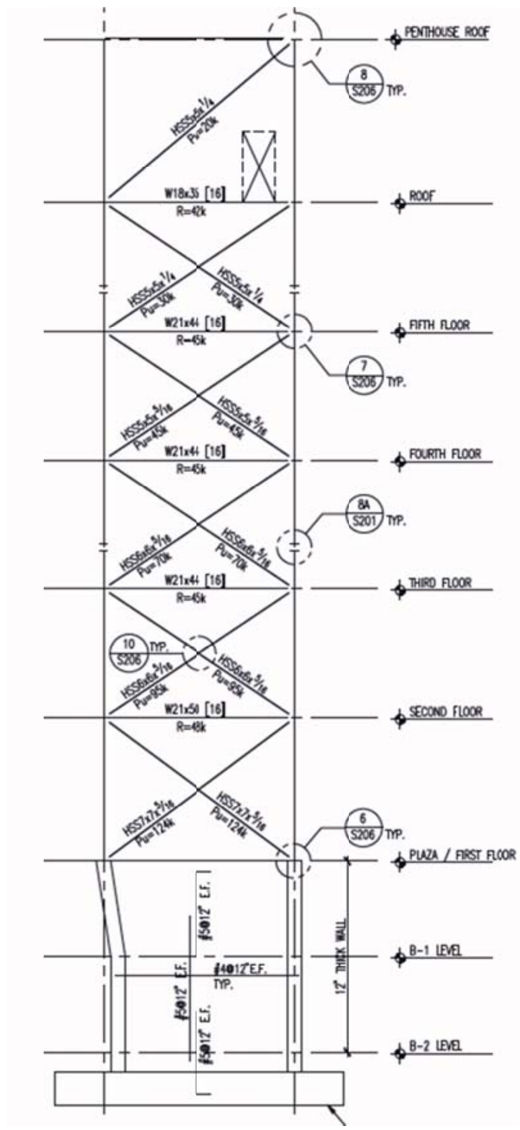


Figure 10: Braced Frame and Shear Wall Elevation

Codes and References

Design Codes and References

"The International Building Code – 2003", International Code Council.

"Minimum Design Loads for Buildings and Other Structures" (ASCE 7), American Society of Civil Engineers.

"Building Code Requirements for Structural Concrete, ACI 318-02", American Concrete Institute.

"ACI Manual of Concrete Practice – Parts 1 through 5", American Concrete Institute.

"Manual of Standard Practice", Concrete Reinforcing Steel Institute.

"Building Code Requirements for Masonry Structures (ACI 530, ASCE 5/ TMS 402)", American Concrete Institute, American Society of Civil Engineers, and The Masonry Society.

"Specifications for Masonry Structures (ACI 530.1/ASCE 6/TMS 602)", American Concrete Institute, American Society of Civil Engineers, and The Masonry Society.

"Manual of Steel Construction – Load and Resistance Factor Design", Third Edition, 2001, American Institute of Steel Construction (Including Specifications for Structural Steel Buildings, Specification for Structural Joints using ASTM A325 or A490 bolts, and AISC Code of Standard Practice.

"Detailing for Steel Construction", American Institute of Steel Construction.

"Structural Welding Code ANSI/AWS D1.1" American Welding Society.

"Design Manual for Floor Decks and Roof Decks", Steel Deck Institute.

"Standard Specifications for Open Web Steel Joists, K-Series", Steel Joist Institute.

"Standard Specifications for Longspan Steel Joists, LH-Series and Deep Longspan Steel Joists, DLH-Series", Steel Joist Institute.

Thesis Codes and References

Steel Construction Manual 13th edition, American Institute of Steel Construction (AISC).

Building Code Requirements for Structural Concrete, American Concrete Institute (ACI 318-08).

Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers (ASCE 7-10).

Material Properties

Minimum Concrete Compressive Strength (f'c)	
Member Type	28 Day Strength
Footings	3000 psi
Grade Beams	3000 psi
Foundation Walls	4000 psi
Shear Walls	4000 psi
Columns	4000 psi
Slabs-on-grade	3500 psi
Reinforced Slabs	5000 psi
Reinforced Beams	5000 psi
Parking Structure	5000 psi
Normal Weight on Steel Deck	3000 psi
Elevator Machine Room	4000 psi
Lightweight Topping	3000 psi

Reinforcement:

Deformed Reinforcing Bars	ASTM A615, Grade 60
Weldable Deformed Reinforcing Bars	ASTM A706
Welded Wire Reinforcement (WWF)	ASTM A185
Full Mechanical Connection Splices (Threadbar and Coupler)	Dywidag, Lenton or equal meeting ACI 318 Section 12.14.3
Adhesive Reinforcing Bar Dowels	Hilti HIT HY-150 System or equal
Slab Shear Reinforcement	Decon Studrails or equal

Steel:

Wide Flange Shapes and Tees	ASTM A992
Round Hollow Structural Shapes	ASTM A53, Grade B, Fy=35 ksi or ASTM A501, Fy=36 ksi
Square or Rectangular Hollow Structural Shapes	ASTM A500, Grade B, Fy=46 ksi
Base Plates and Rigid Frame Continuity Plates	ASTM A572, Grade 50
Other Structural Shapes and Plates	ASTM A36
High Strength Bolts	ASTM A325-N or ASTM F1852
Anchor Bolts	ASTM F1554, Grade 36
Galvanized Steel Floor Deck	ASTM A653 SS, Grade 33, G-60
Galvanized Steel Roof Deck	ASTM A653 SS, Grade 33, G-90
Grout	ASTM C1107, Non-Shrink, Non-Metallic f'c = 5000 psi

Gravity Loads

Live Loads		
Area	**Design Load**	**ASCE 7-10 Load**
Assembly Areas	100 psf	100 psf
Corridors	100 psf	100 psf
Corridors Above the First Floor	80 psf	80 psf
Mechanical Rooms	150 psf	-
Offices	80 + 20 psf	50 + 20 psf
Parking Garages	50 psf	40 psf
Stairs & Exitways	100 psf	100 psf
Storage (Light)	125 psf	125 psf
Roof (Minimum)	30 psf	20 psf

Snow Loads		
Load Type	**Design Load**	**ASCE 7-10 Load**
Flat Roof Snow Load p_f	21.0 psf	21.0 psf
Drift Surcharge Load p_d	-	55.5 psf

Superimposed Dead Loads	
Area	**Design Load**
Floors	10 psf
Roof	15 psf
Mech/Elec	15 psf

Composite Slab and Deck Weight			
Floor	**Area (sq. ft.)**	**Load (psf)**	**Weight**
2nd	24116	54	1302.3 k
3rd	24116	44	1061.1 k
4th	24116	44	1061.1 k
5th	23615	44	1039.1 k
Roof	23615	44	1039.1 k

Column Self Weight					
Floor	**Height Below (ft)**	**Height Above (ft)**	**Weight Below (plf)**	**Weight Above (plf)**	**Total Weight**
2nd	15	6.75	3097	3097	67.4 k
3rd	10.75	2.75	3097	2167	39.3 k
4th	6.75	6.75	2167	2167	29.3 k
5th	6.75	6.75	2167	2167	29.3 k
Roof	6.75	0	2167	0	14.6 k

Wind Loads

The wind loads were determined using ASCE 7-10. The MWFRS Directional Procedure was used to calculate the loads. When calculating the wind loads, the building was assumed to be a 210'x100' rectangle for simplification. The wind loads in the North-South Direction were found to control because the wind loads act upon a larger surface area, therefore creating a larger force on each story of the building. The total base shear came to be 434.7 kips. Detailed calculations of the wind loads are shown in Appendix B.

East-West Design Wind Pressures, p				
Wall	height z (ft)	kz	qz (psf)	p (psf)
Windward	0-15	0.57	16.40	15.63
	20	0.62	17.84	16.60
	25	0.66	18.99	17.37
	30	0.70	20.14	18.14
	40	0.76	21.87	19.31
	50	0.81	23.31	20.27
	60	0.85	24.46	21.05
	69	0.89	25.61	21.82
Leeward	All	0.89	25.61	-11.06

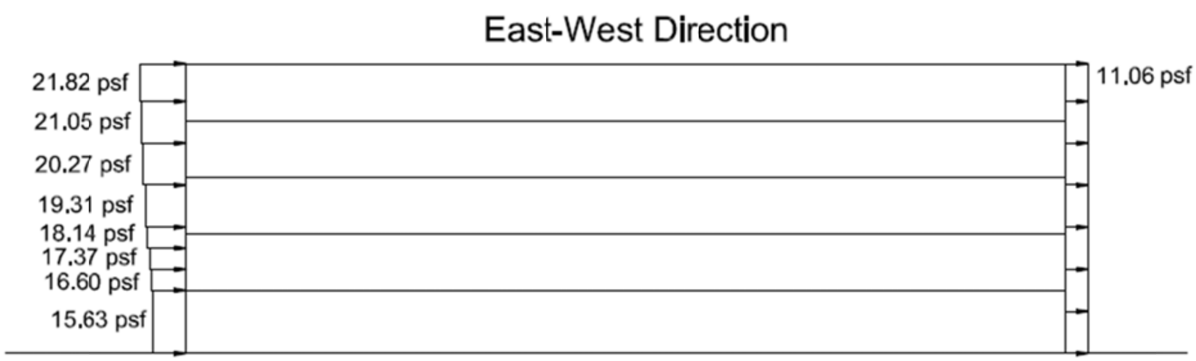


Figure 11: Wind Pressures East-West Direction

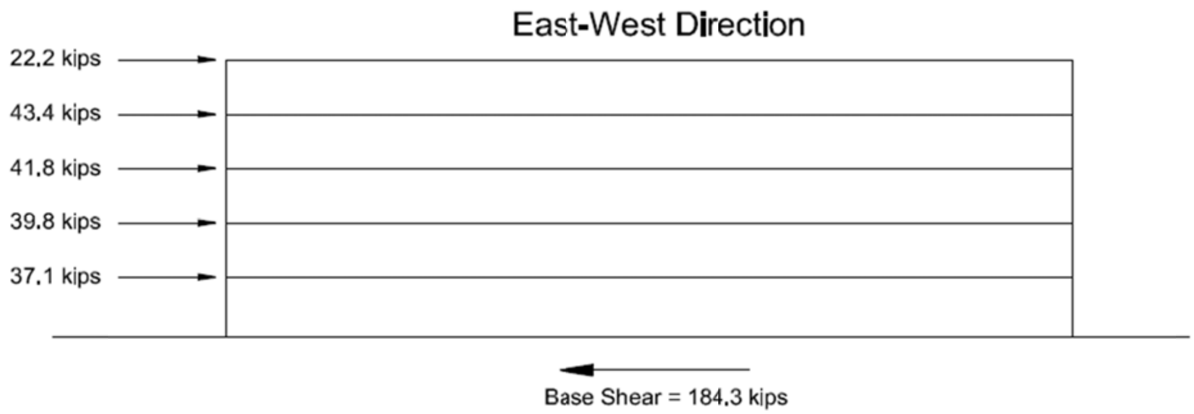


Figure 12: Wind Story Forces East-West Direction

North-South Design Wind Pressures, p				
Wall	height z (ft)	kz	qz (psf)	p (psf)
Windward	0-15	0.57	16.40	15.24
	20	0.62	17.84	16.17
	25	0.66	18.99	16.92
	30	0.70	20.14	17.66
	40	0.76	21.87	18.78
	50	0.81	23.31	19.71
	60	0.85	24.46	20.46
	69	0.89	25.61	21.21
Leeward	All	0.89	25.61	-14.98

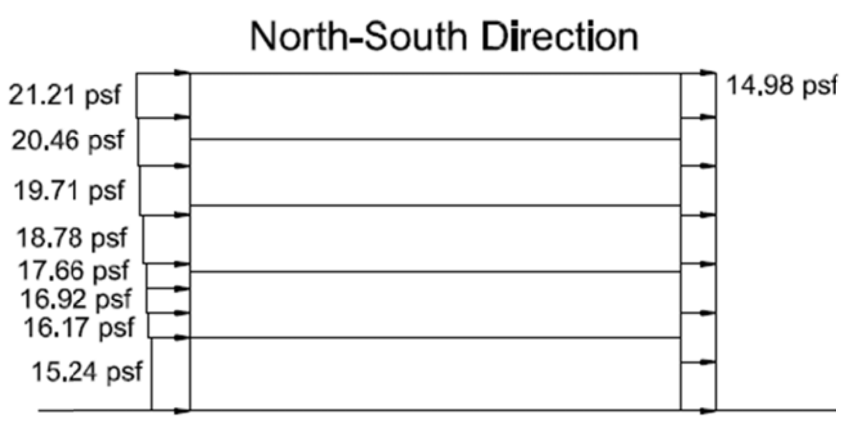


Figure 12: Wind Pressures North-South Direction

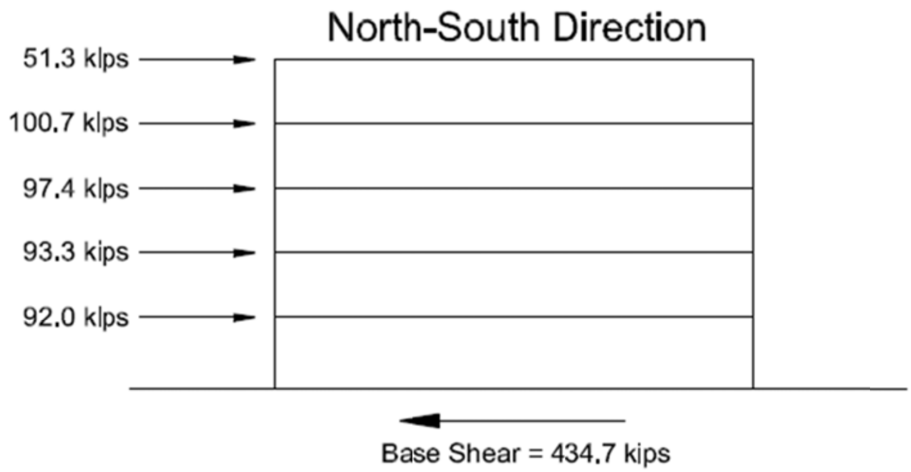


Figure 13: Wind Story Forces North-South Direction

Seismic Loads

The seismic loads on the building were calculated using The Equivalent Lateral Force Procedure of ASCE 7-10. The effective seismic weight of the building was estimated and used to calculate the total base shear of the building due to the seismic loads. The total base shear was calculated to be 288.3 kips which is very close to the base shear of 277 kips on the structural drawings. Detailed seismic load calculations are shown in Appendix B.

Effective Seismic Weight		
Floor	**Weight**	
2nd	2420.5	k
3rd	2135.0	k
4th	2125.0	k
5th	2102.9	k
Roof	2303.6	k
Total	11087.1	k

$V=C_sW=$ 288.3 k

Vertical Distribution of Seismic Forces						
Floor	**w_x**	**h_x (ft)**	**w_xh_x^k**	**C_{vx}**	**F_x**	
2nd	2035.566	15.0	48385.1	0.068	19.6	k
3rd	1773.251	28.5	89317.9	0.126	36.3	k
4th	1763.254	42.0	139803.0	0.197	56.8	k
5th	1741.21	55.5	191281.9	0.269	77.7	k
Roof	1700.687	69.0	241033.8	0.340	97.9	k
		Sum	709821.7	1.000	288.3	k

Overturning Moment 14778.4 ft-k

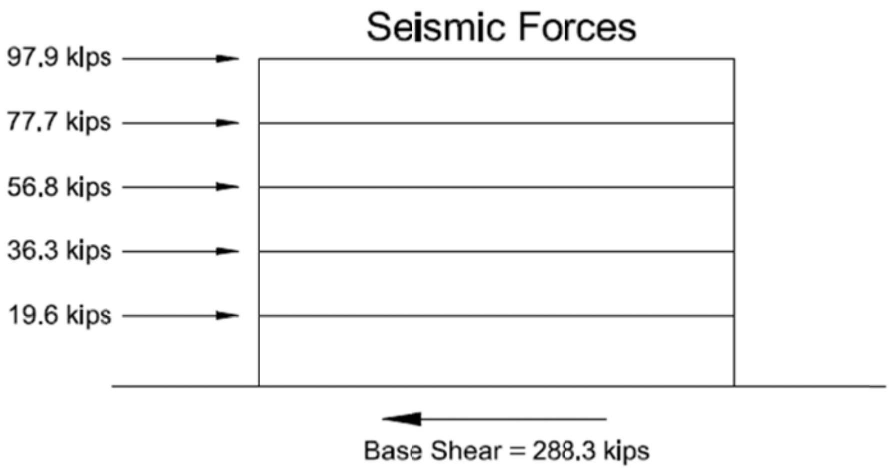


Figure 14: Seismic Story Forces

Comparison of Wind and Seismic Loads

Total Base Shear:

Wind East-West:	184.3 kips
Wind North-South:	**434.7 kips ← Controls**
Seismic Both Directions:	288.3 kips

Overturning Moment:

Wind East-West:	7387 ft-kips
Wind North-South:	**17258 ft-kips ← Controls**
Seismic Both Directions:	14778 ft-kips

Spot Checks

Spot checks were done on a typical composite beam and girder on the second floor. A spot check was also done for column H1. The members that were spot checked are shown below in Figure 15: Partial Second Floor Plan. The members in the actual design were determined to be adequate for the loads. The column was checked below the second floor and below the fourth floor because the steel columns are spliced 4' above the third floor. As seen in the table below, the *actual design* of the composite beam and girder is different than the *thesis design*. The fact that the actual designed members are larger may be attributed to the fact that these typical members are used throughout the building, including spaces that have higher live loads than the offices. It may also be because these members are used on other floors that have thinner composite slabs, so the strength of the composite members will be reduced.

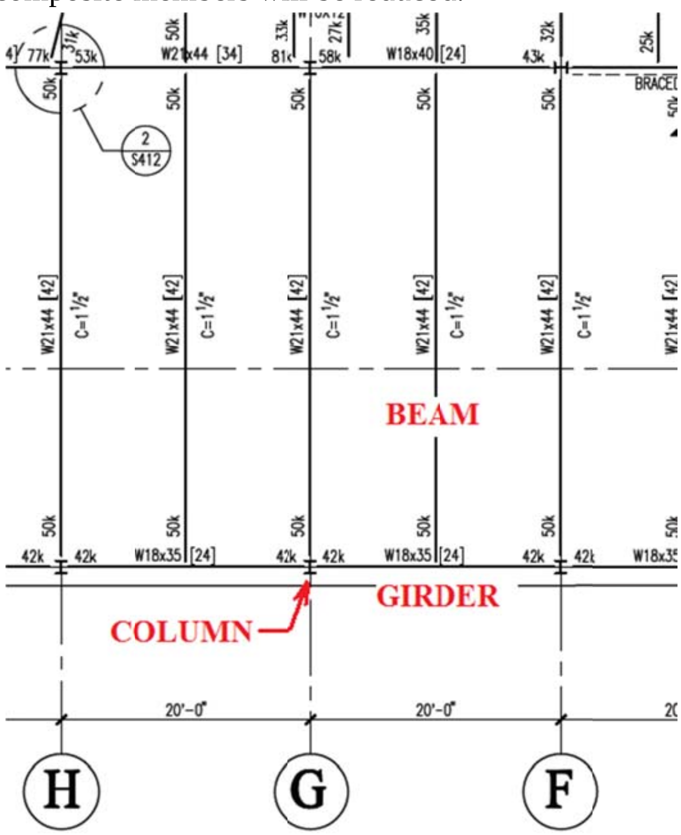


Figure 15: Partial Second Floor Plan

Spot Checks		
Member	Actual Design	Thesis Design
Composite Beam	W21x44 (42)	W18x40 (26)
Composite Girder	W18x35 (24)	W14x22 (32)
Column H1: Below Level 2	W12x58	W12x53
Column H1: Below Level 4	W12x40	W12x40

Appendix A: Typical Framing Plans

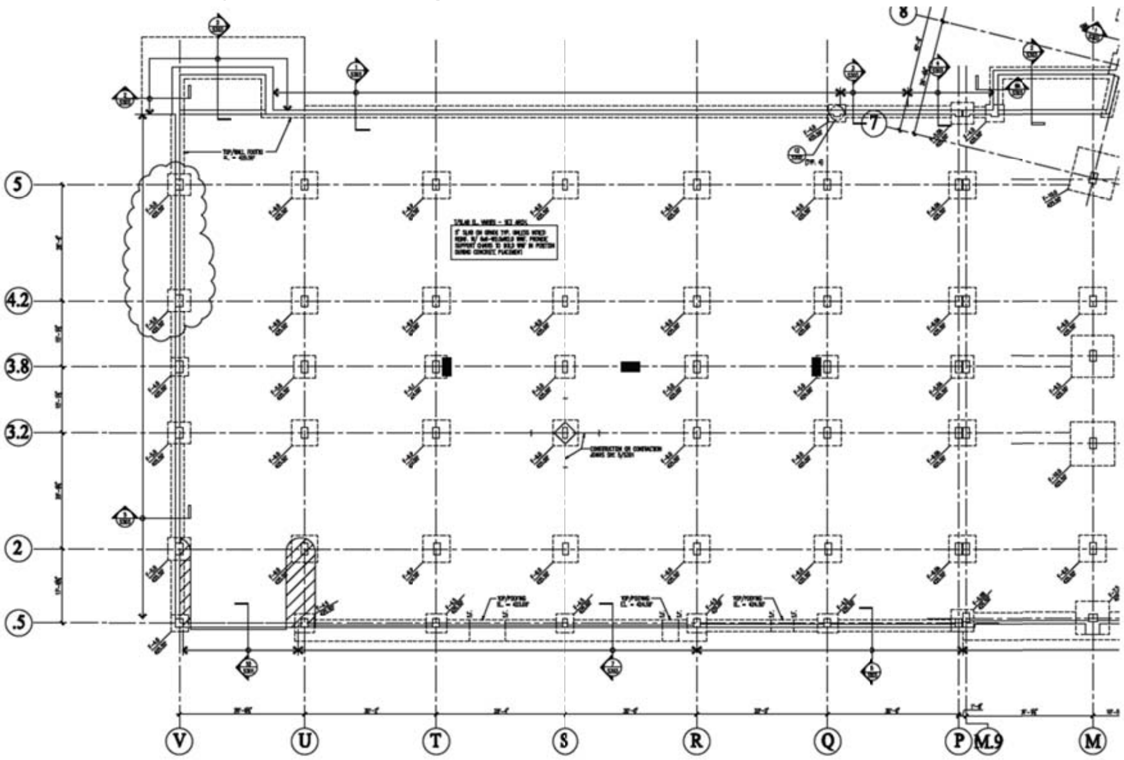


Figure 16: Foundation Plan Part A

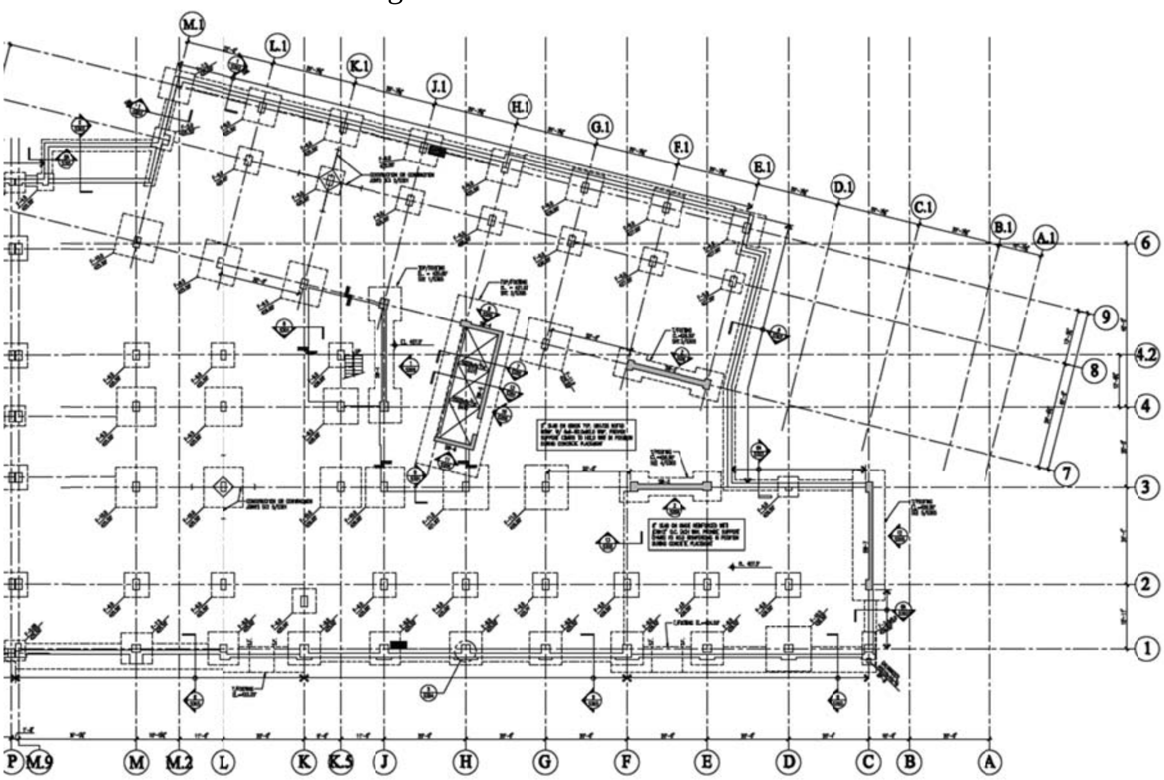


Figure 17: Foundation Plan Part B

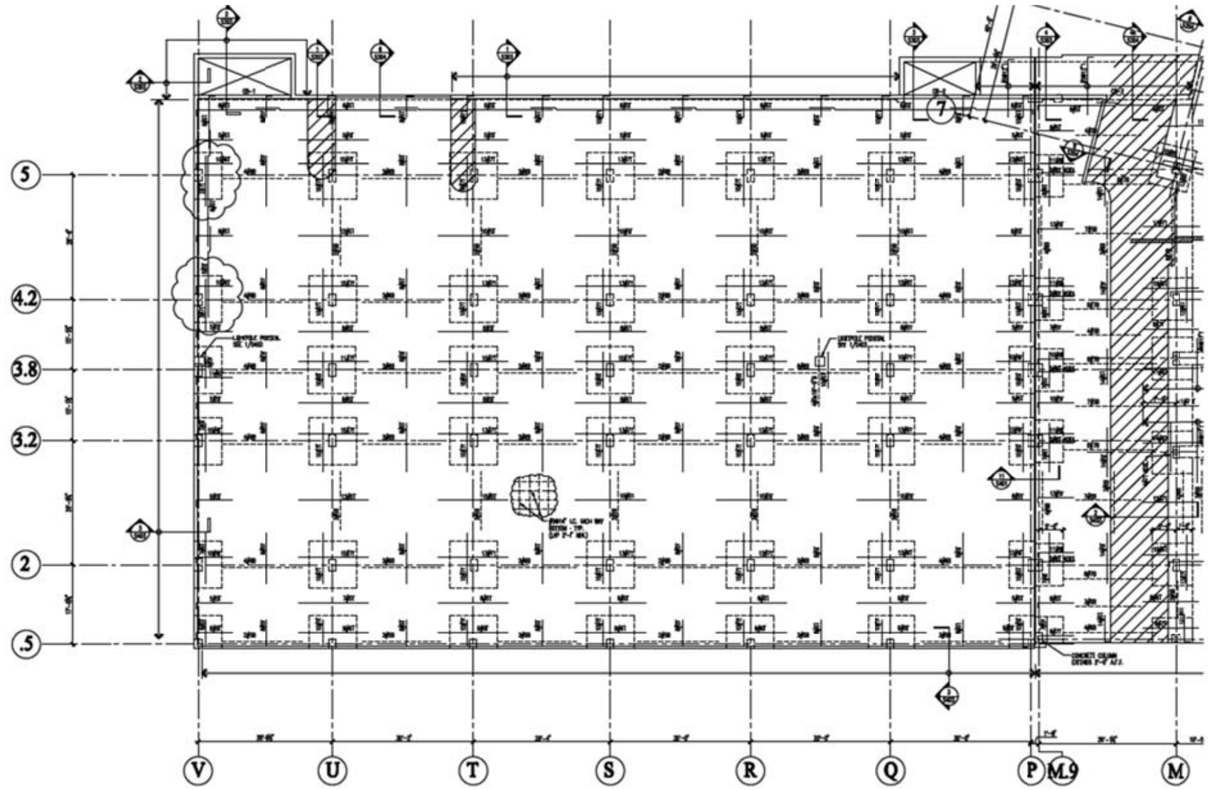


Figure 18: Plaza Level Framing Plan Part A

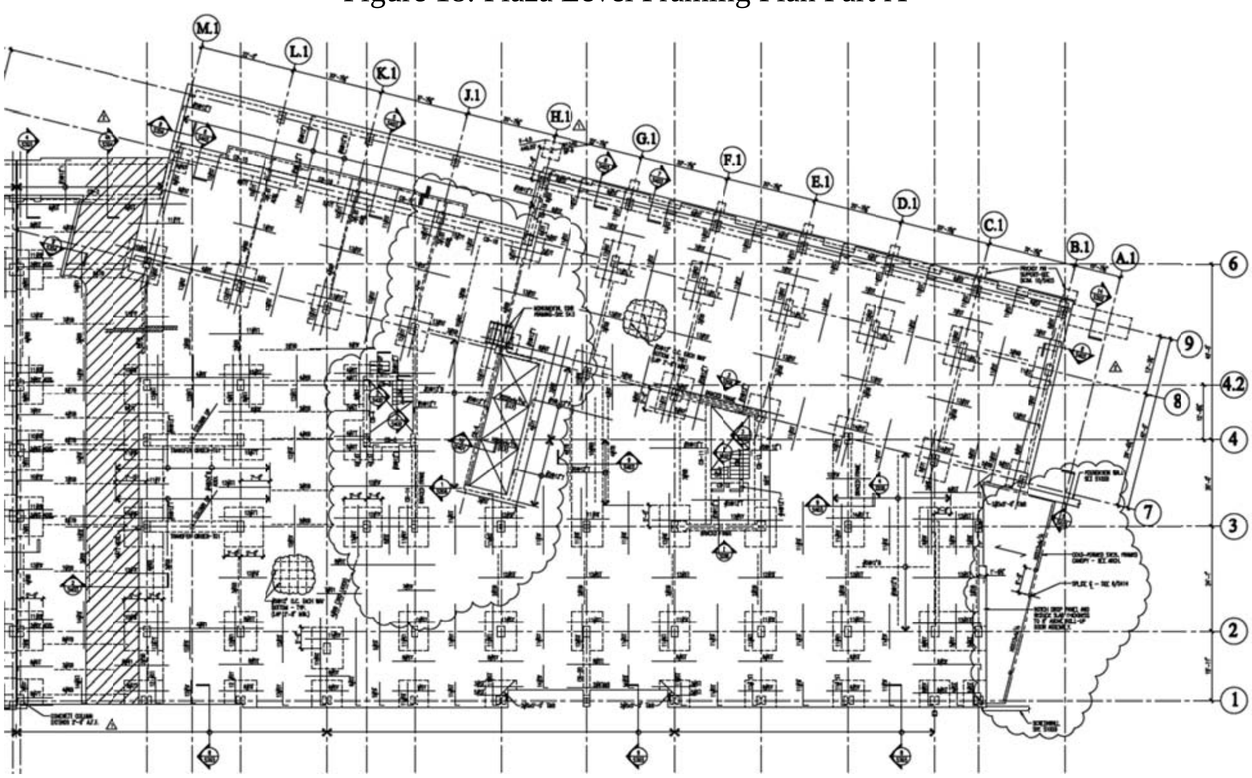


Figure 19: Plaza Level Framing Plan Part B

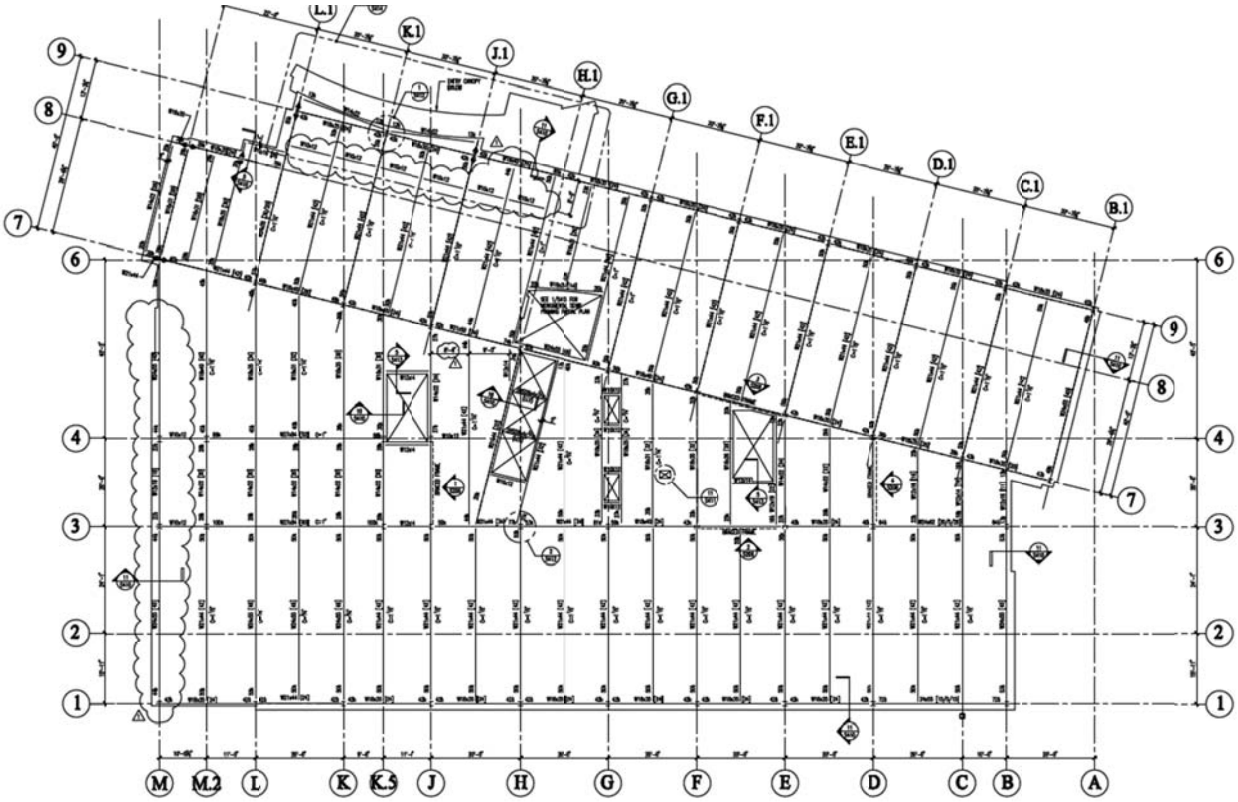


Figure 20: Second Floor Framing Plan

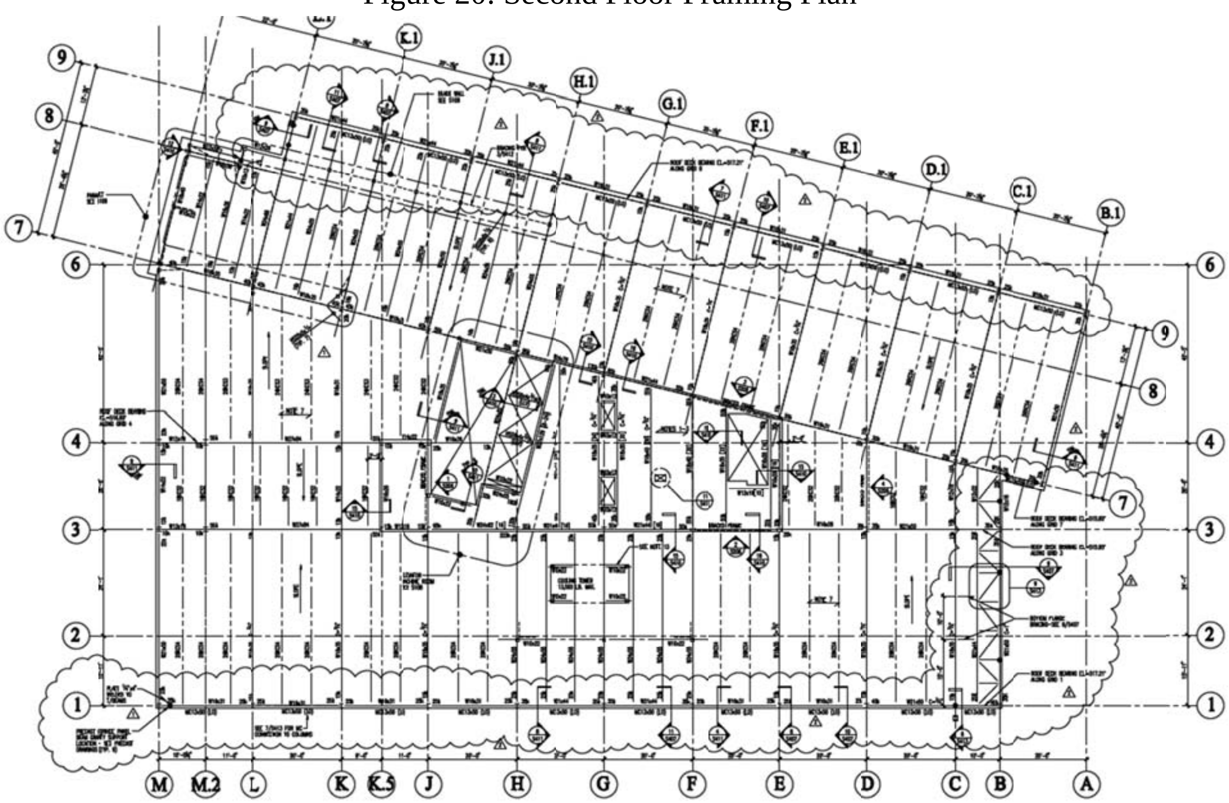


Figure 21: Roof Framing Plan

Appendix B: Calculations

Tech Report I	Snow Load Calcs	9/29/10

ASCE 7-10 Chapter 7 Snow Loads

Flat Roof Snow Load

$p_f = 0.7 C_e C_t I_s p_g$ (7.3-1)

$p_g = 30$ psf (Figure 7-1)

$C_e = 1.0$ (TABLE 7-2)

$C_t = 1.0$ (TABLE 7-3)

$I_s = 1.0$ (Table 1.5-2)

$p_f = 0.7 \cdot 1.0 \cdot 1.0 \cdot 1.0 \cdot 30 = 21$ psf

Drift Loads

$\gamma = 0.13 p_g + 14 = 0.13 \cdot 30 + 14 = 17.9$ pcf (7.7-1)

$h_b = \frac{p_s}{\gamma} = \frac{21}{17.9} = 1.17$ ft

Penthouse $h = 16'$

$h_c = 16 - 1.17 = 14.83'$

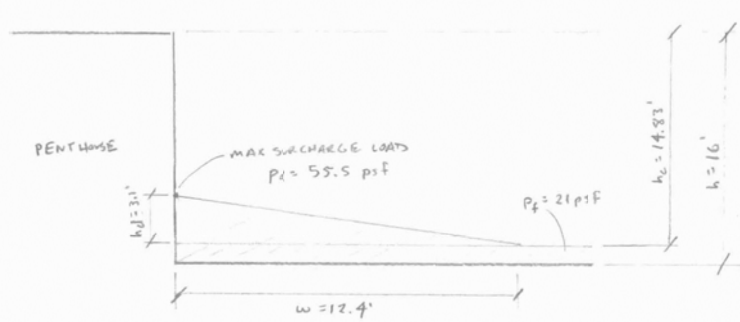
$\frac{h_c}{h_b} = \frac{14.83}{1.17} = 12.7 > 0.2$ DRIFT MUST BE DESIGNED FOR

$L_u = 90'$ $h_d = 3.1'$

$w = 4 h_d = 4 \cdot 3.1 = 12.4'$

Drift Surcharge Load

$p_d = h_d \gamma = 3.1 \cdot 17.9 = 55.5$ psf



Tech Report I | Dead Loads | 9/28/10

Columns

BELOW
58 ~~||||~~ ~~||||~~ ~~||||~~ ~~||||~~
65 ~~||||~~
53 ||
90 ~~||||~~ |
82 ||||
132 |
99 |
74 |
72 ||||
45 ||

Total plf = 3097

ABOVE
45 ~~||||~~
40 ~~||||~~ ~~||||~~ ~~||||~~ ~~||||~~ |||
53 ~~||||~~ ~~||||~~ |||
68 ||||
61 |

Total plf = 2167

2nd Floor

Column weight = $(7.5+6.75)\cdot 3097/1000 = 44.1^{K}$

3rd Floor

columns spliced 4' above 3rd floor

Column weight = $[(6.75+4)\cdot 3097 + 2.75\cdot 2167]/1000 = 39.3^{K}$

4th + 5th Floor

column weight = $13.5\cdot 2167/1000 = 29.3^{K}$

Roof

column weight = $6.75\cdot 2167/1000 = 14.6\,K$

Beam Self Weight Per Floor					
Beam Size		**Length**	**Number**	**Weight**	
W21x	44	40	35	61600	lb
W24x	55	40	5	11000	lb
W18x	35	20	20	14000	lb
W21x	44	20	5	4400	lb
W27x	84	40	2	6720	lb
W12x	14	10	4	560	lb
W10x	12	10	10	1200	lb
W18x	40	20	4	3200	lb
W24x	62	30	1	1860	lb
W14x	22	20	5	2200	lb
W10x	12	4	4	192	lb
W24x	55	20	1	1100	lb
W16x	26	15	1	390	lb
W21x	50	20	1	1000	lb
W16x	31	24	1	744	lb
W16x	40	20	1	800	lb
W14x	22	21	2	924	lb
W14x	22	26	1	572	lb
W16x	31	26	2	1612	lb
W18x	35	26	1	910	lb
W12x	16	12	1	192	lb
W12x	14	15	1	210	lb
W12x	16	18	1	288	lb
W14x	22	23	1	506	lb
W14x	22	25	1	550	lb
W12x	19	25	1	475	lb
W16x	26	28	1	728	lb
W18x	35	30	1	1050	lb
W16x	31	32	1	992	lb
W18x	35	34	2	2380	lb
W21x	44	37	1	1628	lb
W21x	44	40	2	3520	lb
W21x	44	43	1	1892	lb
W14x	22	25	1	550	lb
W16x	31	28	1	868	lb
W18x	35	30	1	1050	lb
W16x	31	32	1	992	lb
W18x	35	34	1	1190	lb
W18x	40	38	1	1520	lb
W21x	50	20	2	2000	lb
W14x	61	20	1	1220	lb
			Total	138785	lb
			Total	138.8	k

Tech Report I | WIND CALCS PG1 | RYAN DALRYMPLE 9/22/10

ASCE 7-10 WIND LOADS ON BUILDINGS - MWFRS (DIRECTIONAL PROCEDURE)

$V = 115$ mph (Figure 26.5-1A)
Exposure : B
Importance Factor, $I_w = 1.0$
Risk Category: II (Table 1.4-1)
$K_d = 0.85$ (Table 26.6-1)
$K_{zt} = 1.0$
$G = 0.85$ (26.9.1)
$GC_{pi} = \pm 0.18$ (Table 26.11-1)

$K_z = 2.01\left(z/z_g\right)^{2/\alpha}$ $\alpha = 7$ $z_g = 1200$

$z = 69$ ft : $K_z = 2.01\left(69/1200\right)^{2/7} = 0.89$

height z (ft)	K_z	q_z (psf)
0-15	0.57	16.40
20	0.62	17.84
25	0.66	18.99
30	0.70	20.14
40	0.76	21.87
50	0.81	23.31
60	0.85	24.46
69	0.89	25.61

$q_z = 0.00256 K_z K_{zt} K_d V^2$ (psf)

@20 ft $q_z = 0.00256(0.62)(1.0)(0.85)\,115^2 = 17.84$ psf

$n_a = 75/h = 75/69 = 1.1\text{ Hz} > 1.0\text{ Hz}$ ∴ Building is Rigid

27.4.1 Building is Rigid

$p = q\,GC_p - q(GC_{pi})$ (psf)

GUST EFFECT FACTOR 26.9.4 - Rigid Building

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

$g_Q, g_v = 3.4$

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

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

$\bar z = 0.6h = 0.6 \cdot 69 = 41.4$ ft
$z_{min} = 30$ ft

$c = 0.30$

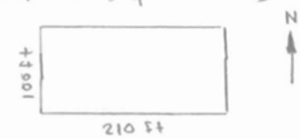
$I_{\bar z} = 0.3\left(\frac{33}{41.4}\right)^{1/6} = 0.29$

$L_{\bar z} = \ell\left(\frac{\bar z}{33}\right)^{\bar\epsilon}$ $\ell = 320$ $\bar\epsilon = 1/3$

$L_{\bar z} = 320\left(\frac{41.4}{33}\right)^{1/3} = 345$

Tech Report I | Wind Calcs PG 2 | Ryan Dalrymple 9/22/10

Simplified Shape of Building



$$Q_{E-W} = \sqrt{\frac{1}{1 + 0.63\left(\frac{100+69}{345}\right)^{0.63}}}$$

$$Q_{E-W} = 0.845$$

$$Q_{N-S} = \sqrt{\frac{1}{1 + 0.63\left(\frac{210+69}{345}\right)^{0.63}}}$$

$$Q_{N-S} = 0.803$$

$$G_{E-W} = 0.925\left(\frac{1 + 1.7(3.4)(0.28)(0.845)}{1 + 1.7\cdot(3.4)(0.29)}\right) = 0.84$$

$$G_{N-S} = 0.925\left(\frac{1 + 1.7\cdot(3.4)(0.28)(0.803)}{1 + 1.7(3.4)(0.29)}\right) = 0.81$$

External Pressure Coeffs. (C_p) – Figure 27.4-1

E-W $\quad L/B = \frac{210}{100} = 2.1$

WINDWARD WALL: $C_p = 0.8$

LEEWARD WALL: $C_p = -0.3$

N-S $\quad L/B = \frac{100}{210} = 0.48$

WINDWARD WALL: $C_p = 0.8$

LEEWARD WALL: $C_p = -0.5$

DESIGN WIND PRESSURES (P)

E-W:

WINDWARD WALL: $p = q_z G C_p - q_h (GC_{pi})$

$p = q_z \cdot 0.84 \cdot 0.8 - 25.61\,(\pm 0.18) = 0.672\,q_z + 4.61$ psf

LEEWARD WALL: $p = q_h G C_p - q_h (GC_{pi})$

$p = 25.61 \cdot 0.84 \cdot (-0.3) - 25.61\,(\pm 0.18) = -11.06$ psf

N-S:

WINDWARD WALL: $p = q_z \cdot 0.81 \cdot 0.8 - 25.61\,(\pm 0.18) = 0.648\,q_z + 4.61$ psf

LEEWARD WALL: $p = 25.61 \cdot 0.81 \cdot (-0.5) - 25.61\,(\pm 0.18) = -14.98$ psf

Tech Report I | WIND CALCS PG3 | Ryan Dalrymple 9/22/10

STORY WIND FORCES

E-W DIRECTION WINDWARD FORCES

STORY 1: $(15.63\text{ psf} \cdot 7.5\text{ ft} + 16.60\text{ psf} \cdot 5\text{ ft} + 17.37\text{ psf} \cdot 1.75\text{ ft}) \cdot 100\text{ ft}/1000 = 21.3\text{ k}$

STORY 2: $(17.37 \cdot 3.25 + 18.14 \cdot 5 + 19.31 \cdot 5.25) \cdot 100/1000 = 24.9\text{ k}$

STORY 3: $(19.31 \cdot 4.75 + 20.27 \cdot 8.75) \cdot 100/1000 = 26.9\text{ k}$

STORY 4: $(20.27 \cdot 1.25 + 21.05 \cdot 10 + 21.82 \cdot 2.25) \cdot 100/1000 = 28.5\text{ k}$

STORY 5: $(21.82 \cdot 6.75) \cdot 100/1000 = 14.7\text{ k}$

ADD LEEWARD FORCES

STORY 1: $21.3 + \frac{11.06 \cdot 14.25 \cdot 100}{1000} = 37.1\text{ k}$

STORY 2: $24.9 + \frac{11.06 \cdot 13.5 \cdot 100}{1000} = 39.8\text{ k}$

STORY 3: $26.9 + 14.9 = 41.8\text{ k}$

STORY 4: $28.5 + 14.9 = 43.4\text{ k}$

STORY 5: $14.7 + \frac{11.06 \cdot 6.75 \cdot 100}{1000} = 22.2\text{ k}$

N-S DIRECTION

STORY 1: $(15.24 \cdot 7.5 + 16.17 \cdot 5 + 16.92 \cdot 1.75) \cdot 210/1000 + \frac{14.98 \cdot 14.25 \cdot 210}{1000} = 92.0\text{ k}$

STORY 2: $(16.92 \cdot 3.25 + 17.66 \cdot 5 + 18.78 \cdot 5.25) \cdot 210/1000 + \frac{14.98 \cdot 13.5 \cdot 210}{1000} = 93.3\text{ k}$

STORY 3: $(18.77 \cdot 4.75 + 19.71 \cdot 8.75) \cdot 210/1000 + 42.5 = 97.4\text{ k}$

STORY 4: $(19.71 \cdot 1.25 + 20.46 \cdot 10 + 21.21 \cdot 2.25) \cdot 210/1000 + 42.5 = 100.7\text{ k}$

STORY 5: $(21.21 \cdot 6.75) \cdot 210/1000 + \frac{14.98 \cdot 6.75 \cdot 210}{1000} = 51.3\text{ k}$

BASE SHEAR AND OVER TURNING MOMENT

E-W: Base Shear $= 37.1 + 39.8 + 41.8 + 43.4 + 22.2 = 184.3\text{ k}$

$M = 37.1 \cdot 15 + 39.8 \cdot 28.5 + 41.8 \cdot 42 + 43.4 \cdot 55.5 + 22.2 \cdot 69$

$M = 7{,}387\text{ ft} \cdot \text{k}$

N-S: Base Shear $= 92 + 93.3 + 97.4 + 100.7 + 51.3 = 434.7\text{ k}$

$M = 92 \cdot 15 + 93.3 \cdot 28.5 + 97.4 \cdot 42 + 100.7 \cdot 55.5 + 51.3 \cdot 69$

$M = 17{,}258\text{ ft} \cdot \text{k}$

Tech Report I	Seismic Calcs	9/28/10

Seismic Requirements from Drawings:

$S_s = 0.16$ Site class C $R = 3$

$S_1 = 0.05$ $I_e = 1.0$ $C_s = 0.019$

$S_{DS} = 0.128$ Seismic Use group I

$S_{D1} = 0.06$ Design Category A PROCEDURE: Equiv. Lateral Force Procedure

Seismic Force Resisting System: Steel Braced Frames + Shear walls

Mapped Acceleration Parameters ASCE 7-10

$S_s = 0.14$ (Figure 22-1)

$S_1 = 0.05$ (Figure 22-2)

Spectral Response Acceleration Parameters - Site class C

$F_a = 1.2$ Table 11.4-1

$F_v = 1.7$ Table 11.4-2

$S_{MS} = F_a S_s = 1.2 \cdot 0.14 = 0.168$ (Eq. 11.4-1)

$S_{M1} = F_v S_1 = 1.7 \cdot 0.05 = 0.085$ (Eq. 11.4-2)

Design Spectral Acceleration Parameters

$S_{DS} = 2/3\, S_{MS} = 2/3 \cdot 0.168 = 0.112$ (Eq. 11.4-3)

$S_{D1} = 2/3\, S_{M1} = 2/3 \cdot 0.085 = 0.057$ (Eq. 11.4-4)

Seismic Design Category

Risk Category II $S_{DS} < 0.167$, $S_{D1} < 0.067$ $\therefore$ SDC A

Response Modification Coeff

$R = 3$ (TABLE 12.2-1)

Approximate Fundamental Period

$T_a = C_t h_n^x$ (Eq. 12.8-7)

$C_t = 0.03$ $x = 0.75$ $h_n = 85$ ft

$T_a = 0.03 \cdot 85^{0.75} = 0.84$ s

Tech Report I | Seismic Calcs | 9/28/10

Equivalent Lateral Force Procedure (Both Directions)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.112}{\left(\frac{3}{1}\right)} = 0.037 \qquad (\text{Eq. } 12.8\text{-}2)$$

$$T = 0.840\,s < T_L = 8\,s$$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.057}{0.84\left(\frac{3}{1}\right)} = \underline{0.026} < 0.037$$

$$\boxed{C_s = 0.026}$$

Vertical Distribution of seismic Forces

Sample calculation for effective seismic weight of a floor

$$w_2 = \underset{\text{SDL}}{0.025 \cdot 24116} + \underset{\text{BEAMS}}{118.8} + \underset{\text{SLAB}}{1302.3} + \underset{\text{COLUMNS}}{44.1} + \underset{\text{FACADE}}{\left[35\,(15 + 6.75)\cdot(2\cdot 100 + 2\cdot 210)/1000\right]}$$

$w_2 = 2420.5$ k	$h_2 = 15'$
$w_3 = 2135.0$ k	$h_3 = 28.5'$
$w_4 = 2125.0$ k	$h_4 = 42'$
$w_5 = 2102.9$ k	$h_5 = 55.5'$
$w_R = 2303.6$ k	$h_R = 69'$
$W = 11087.1$ k	

$$V = C_s W = 0.026 \cdot 11087.1$$

$$V = 288.3\ k$$

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k} \qquad k = \frac{0.84 - 0.5}{2.5 - 0.5}(2-1) + 1 = 1.17$$

$$C_{v2} = \frac{2420.5 \cdot 15^{1.17}}{2420.5\cdot 15^{1.17} + 2135\cdot 28.5^{1.17} + 2125\cdot 42^{1.17} + 2102.9\cdot 55.5^{1.17} + 2303.6\cdot 69^{1.17}}$$

$$C_{v2} = 0.068$$

$$F_x = C_{vx} V$$

$$F_2 = 0.068 \cdot 288.3$$

$$F_2 = 19.6\ k$$

Tech Report I | Spot Checks | 9/30/10

Typical Bay

B1 : W21x44 [42]
G1: W18x35 [24]

LOADS:
LL = 100 psf
SDL = 10 + 15 = 25 psf
SLAB/DECK = 54 psf
BEAM SELF WT = 5 psf

Composite Beam B1

$A_T = 10 \cdot 40 = 400\ \square$

$LL = 100\left(0.25 + \frac{15}{\sqrt{2 \cdot 400}}\right) = 78\ psf$

$W_u = 1.2\left(25 + 54 + 5\right) + 1.6 \cdot 78 = 225.6\ psf$

$w_u = 225.6 \cdot 10/1000 = 2.256\ klf$

$M_u = \frac{w_u l^2}{8} = \frac{2.256 \cdot 40^2}{8} = 451.2\ 'k$

Assume $a \approx 2"$ $Y_2 = 6.5 - \frac{2}{2} = 5.5"$

$b_{eff} = \min \begin{cases} 40 \cdot 12/4 = 120" \\ 10 \cdot 12 = 120" \end{cases}$ $b_{eff} = 120"$

TABLE 3-21
3/4" ϕ, ⊥ DECK, WEAK
$f'_c = 3000\ psi$
$Q_n = 17.2\ k$

Table 3-14

MEMBER	PNA	ϕM_n	ΣQ_n	# STUDS	Equiv. Wt.
W16x36	3	469	378	44	36·40+44·10 = 1880
W16x40	4	483	324	38	40·40+38·10 = 1980
W16x31	TFL	460	456	54	31·40+54·10 = 1740
W18x35	4	473	323	38	35·40+38·10 = 1780
W18x40	6	469	210	26	40·40+26·10 = 1860

Tech Report I | Spot Checks | 9/30/10

Try W16x31

$$a = \frac{\Sigma Q_N}{0.85 F'_c \cdot B_{eff}} = \frac{460}{0.85 \cdot 3 \cdot 120} = 1.50"$$

$$Y_2 = 6.5 - \frac{1.50}{2} = 5.75" > 5.5"$$

∴ CALCULATION IS CONSERVATIVE

Check unshored strength

W16x31 $\phi M_p = 203^{1k}$

$w_{LL} = (20 \cdot 10)/1000 = 0.2$ KIF

$w_{DL} = [54 \cdot 10 + 31]/1000 = 0.571$ KIF

$w_u = 1.4 \cdot 0.571 = 0.799$ KIF

$w_u = 1.2 \cdot 0.571 + 1.6 \cdot 0.2 = 1.01$ KIF

$$M_u = \frac{1.01 \cdot 40^2}{8} = 201^{1k} < \phi M_p = 203^{1k}$$

Check LL Deflection

$w_{LL} = 78 \cdot 10/1000 = 0.78$ KIF

$I_{LB} = 1200 \ in^4$ (Table 3-20)

$$\Delta_{LL} = \frac{5 w l^4}{384 EI} = \frac{5 \cdot 0.78 \cdot 40^4}{384 \cdot 29000 \cdot 1200} \cdot 1728 = 1.29" < \frac{L}{360} = \frac{40 \cdot 12}{360} = 1.33"$$

∴ OKAY

Check wet concrete Deflection

$w_{WC} = 0.571$ KIF $I_x = 375 \ in^4$

$$\Delta_{WC} = \frac{5 \cdot 0.571 \cdot 40^4 \cdot 1728}{384 \cdot 29000 \cdot 375} = 3.02" > \frac{L}{240} = \frac{40 \cdot 12}{240} = 2.0"$$

∴ NOT OKAY

Try 18x40

$$a = \frac{210}{0.85 \cdot 3 \cdot 120} = 0.583"$$

$$Y_2 = 6.5 - \frac{0.583"}{2} = 6.21" > 5.5"$$

∴ STRENGTH IS CONSERVATIVE

Check unshored Strength

W18x40 $\phi M_p = 294^{1k}$

$w_{LL} = 20 \cdot 10/100 = 0.2$ KIF

$w_{DL} = [54 \cdot 10 + 40]/1000 = 0.58$ KIF

$w_u = 1.4 \cdot 0.58 = 0.812$ KIF

$w_u = 1.2 \cdot 0.58 + 1.6 \cdot 0.2 = 1.02$ KIF

$$M_u = \frac{1.02 \cdot 40^2}{8} = 204^{1k} < \phi M_p = 294^{1k}$$

Tech Report I | Spot Checks | 9/30/10

Check LL Deflection

$I_{LB} = 1210 \text{ in}^4$

$\Delta_{LL} = \frac{5 \cdot 0.78 \cdot 40^4}{384 \cdot 29000 \cdot 1210} \cdot 1728 = 1.28'' < \frac{L}{360} = 1.33''$ ∴ okay

Check wet concrete Deflection

$w_{wc} = [540 + 40]/1000 = 0.580 \text{ klf}$

$I = 612 \text{ in}^4$

$\Delta_{wc} = \frac{5 \cdot 0.58 \cdot 40^4 \cdot 1728}{384 \cdot 29000 \cdot 612} = 1.88'' < \frac{L}{240} = 2.0''$ ∴ okay

B1: USE W18X40 [26]

Tech Report I | Spot Checks | 10/1/10

Girder G1   Typical Exterior Girder

$A_T = \left(20 + \frac{26}{12}\right) \cdot 20 = 443.3 \text{ ft}^2$

$LL = 100\left(0.25 + \frac{15}{\sqrt{2 \cdot 443.3}}\right) = 75.4 \text{ psf}$

FACADE LOAD

$35\text{psf}\,(7.5 + 6.75) = 498.8 \text{ plf}$

BEAM POINT LOAD

$44 \text{plf} \cdot 20 \text{ft} = 880 \text{ lb}$

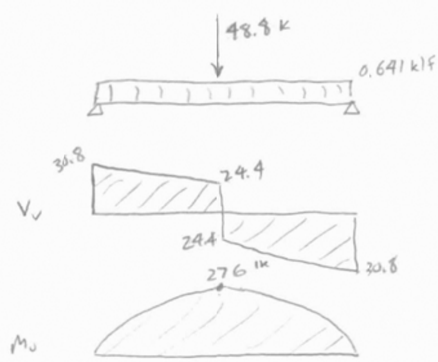
$W_u = 1.2(54 + 25) + 1.6 \cdot 75.4 = 215.4 \text{ psf}$

$P_u = 215.4 \cdot 10 \cdot 22.17/1000 + 1.2 \cdot 0.88 = 48.8 \text{ k}$

GIRDER SELF WEIGHT

Assume = 35 plf

$w_u = 1.2(35 + 498.8)/1000 = 0.641 \text{ klf}$



$M_u = 276^{\text{ik}}$

$Q_N = 18.3 \text{k}$

$b_{eff} = \min \begin{cases} 26 + \frac{20 \cdot 12}{8} = 56'' \\ 26 + 20 \cdot 12 = 266'' \end{cases}$ $b_{eff} = 56''$

Assume $a \simeq 2''$ $Y_2 = 6.5 - \frac{2}{2} = 5.5''$

MEMBER	PNA	ϕM_n	ΣQ_N	# STUDS	Equiv. Wt.
W12X26	3	281	259	30	820
W12X30	4	294	225	26	860
W14X22	2	284	283	32	760
W14X26	4	292	226	26	780

Try W14X22

Tech Report I | Spot Checks | 10/1/10

check unshored strength W14x22 $\phi M_p = 125^{1k}$

$w_u = 1.2(54+5) + 1.6 \cdot 20 = 102.8$ k

$P_u = \frac{102.8 \cdot 10 \cdot 40}{2} / 1000 = 20.6\text{k}$

$M_u = \frac{20.6}{2} \cdot 10 = 103^{1k} < \phi M_p = 125^{1k}$ ∴ OK

Check LL Deflection

$P_{LL} = \frac{75.4 \cdot 10 \cdot 40}{2} / 1000 = 15.1$ k

$I_{LB} = 662 \text{ in}^4$

$\Delta_{LL} = \frac{P\ell^3}{48EI} = \frac{15.1 \cdot 20^3 \cdot 1728}{48 \cdot 29000 \cdot 662} = 0.23'' < \frac{L}{360} = \frac{20 \cdot 12}{360} = 0.66''$ ∴ ok

Check wet concrete deflection

$P_{wc} = \frac{(54+5) \cdot 10 \cdot 40}{2} / 1000 = 11.8$ k

$I = 199 \text{ in}^4$

$\Delta_{wc} = \frac{11.8 \cdot 20^3 \cdot 1728}{48 \cdot 29000 \cdot 199} = 0.59'' < \frac{L}{240} = \frac{20 \cdot 12}{240} = 1.0''$ ∴ ok

G1: USE W14x22 [32]

Tech Report I | Spot Checks | 10/1/10

Check Column H1

W12×58
W12×40

$A_T = 20 \cdot 20 = 443.3 ft^2$

$LL = 100\left(0.25 + \frac{15}{\sqrt{4 \cdot 4 \cdot 443.3}}\right) = 43$ psf

$D_{Fl2} = 54$ psf $SDL_{ROOF} = 15$ psf $LL_{ROOF} = 21$ psf

$D_{Fl3-R} = 44$ psf $SDL_F = 10$ psf

$D_{BEAMS} = 10$ psf $SDL_{MEE} = 15$ psf

COLUMN BELOW 2ND FLOOR

$P_{uT} = \underbrace{3 \cdot 443.3\left[1.6 \cdot 43 + 1.2(44+25+10)\right]/1000}_{\text{FLOORS 3-5}} + \underbrace{443.3\left[1.6 \cdot 21 + 1.2(44+30+10)\right]/1000}_{\text{ROOF}}$

$+ \underbrace{443.3\left[1.6 \cdot 43 + 1.2(54+25+10)\right]/1000}_{\text{FLOOR 2}}$ 355.0 k

FACADE

$A_T = (69-7.5) \cdot 20 = 1230 ft^2$

$P_{uF} = 1.2 \cdot (1230 \cdot 35 psf) = 51.7$ k

kL = 15'

$P_u = 355.0 + 51.7 = 407$ k $\rightarrow \therefore$ USE W12×53 $\phi P_n = 477$ k

COLUMN BELOW 4th FLOOR LL = 50 psf

$P_{uT} = \underbrace{2 \cdot 443.3\left[1.6 \cdot 50 + 1.2(44+25+10)\right]/1000}_{\text{4th + 5th FLR}} + \underbrace{443.3\left[1.6 \cdot 21 + 1.2(44+25+15)\right]/1000}_{\text{ROOF}}$

$P_{uT} = 214.6$

FACADE

$A_T = (69-15-13.5-6.75) \cdot 20 = 675 ft^2$

$P_{uF} = 1.2(675 \cdot 35 psf) = 28.4$ k

kL = 13.5'

$P_u = 214.6 + 28.4 = 243$ k $\longrightarrow \therefore$ use W12×40 $\phi P_n = 280$ k

Ryan Dalrymple – Structures Option
Advisor: Dr. Thomas Boothby