ASHA National Office

Rockville, MD

Technical Report III

Photo Courtesy of Boggs & Partners Architects

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Structures Option

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Executive Summary

The ASHA National Office building is an office building located in Rockville, MD. The office tower is five stories and there are two floors of subgrade parking. The parking structure is composed of a flat slab system with drop panels and the superstructure is composite steel. The lateral system consists of four braced frames in the office tower with shear walls in the subgrade parking garage. The gross area of the building is 133,870 square feet.

This report includes a description of how the lateral loads are distributed to each of the braced frames. The lateral loads are transferred through the floor slab and into the braced frames. The loads are then transferred from the braced frames into the shear walls in the subgrade parking structure, and then into the foundations below. The relative stiffness of each of the braced frames was calculated by modeling each of them in ETABS and applying a 1 kip load at the top. The stiffness of each frame was then used to determine how much direct load and torsional load each frame takes.

The ASCE 7-10 load cases that were considered in this report are shown. Different load cases govern depending on the building element that is being analyzed. Both gravity and lateral loads are considered in the load cases. Due to the complex building plan, an ETABS model was used to analyze the building. The braced frames and subgrade shear walls were modeled, along with the floors which were modeled as rigid diaphragms. Building torsion was analyzed in this report. Both inherent torsion due to the eccentricity of the center of rigidity and accidental torsion are considered. The torsion due to the seismic loads in both directions was determined. Overturning moment was also calculated. It was found that the seismic loads and the wind loads in the N-S direction are considerable moments, and may affect the design of the foundations. This may have to be looked at more in the spring semester.

The story drifts and displacements due to the lateral loads were measured in order to determine if the building meets serviceability requirements. The ETABS model output was used to obtain these displacements. It was determined that the building meets the ASCE 7-10 code requirements and engineering standards. Members were checked to make sure that they are adequate for the applied gravity and lateral loads. A cross bracing HSS member, a W-Flange column and a concrete column were checked. Different load cases controlled depending on which member was being analyzed. All three members were found to be acceptable for the loads applied. The building meets the story drift and displacement serviceability requirements, and the members meet the strength requirements, it was determined that the lateral system is adequate for the ASHA National Office 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. There are 201 parking spaces in the subgrade parking structure and 224 spaces above grade.

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 which is shown below. 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 a 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.

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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.

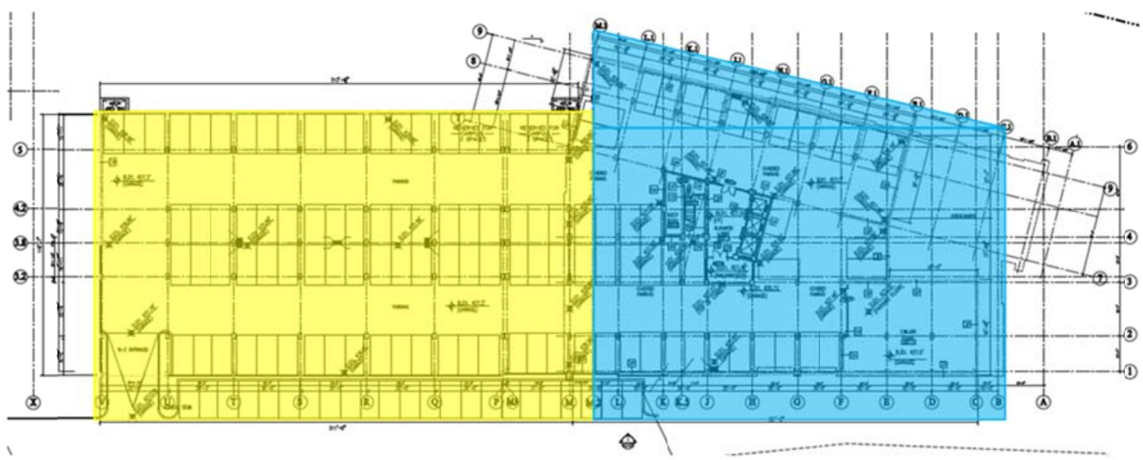

Figure 1: Overall Parking Floor Plan

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 generally 6'x6' and range from 12" to 18" thick. See Figure 3: Column Footing Schedule.

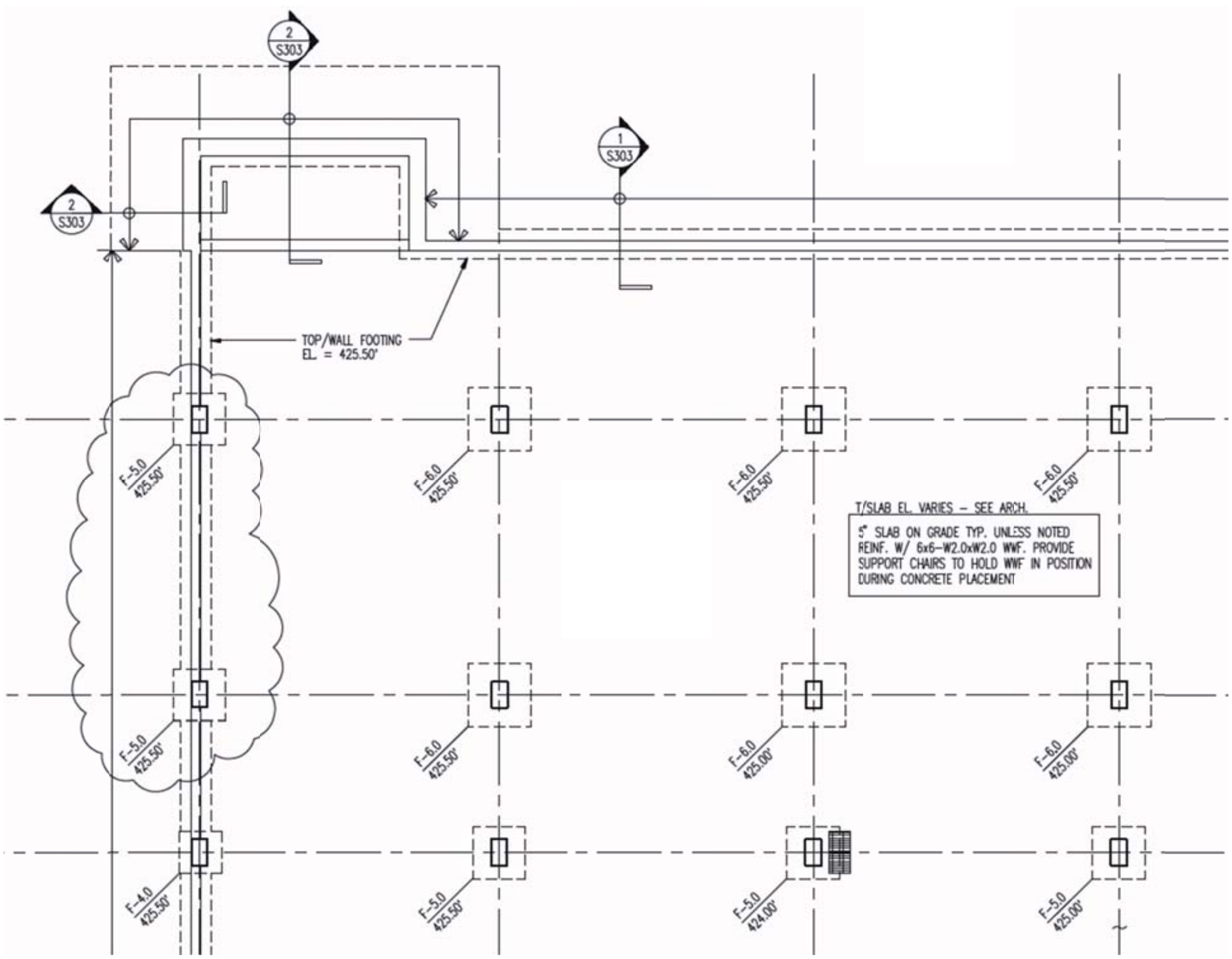

Figure 2: Partial Foundation Plan

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 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 on the plans, 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.

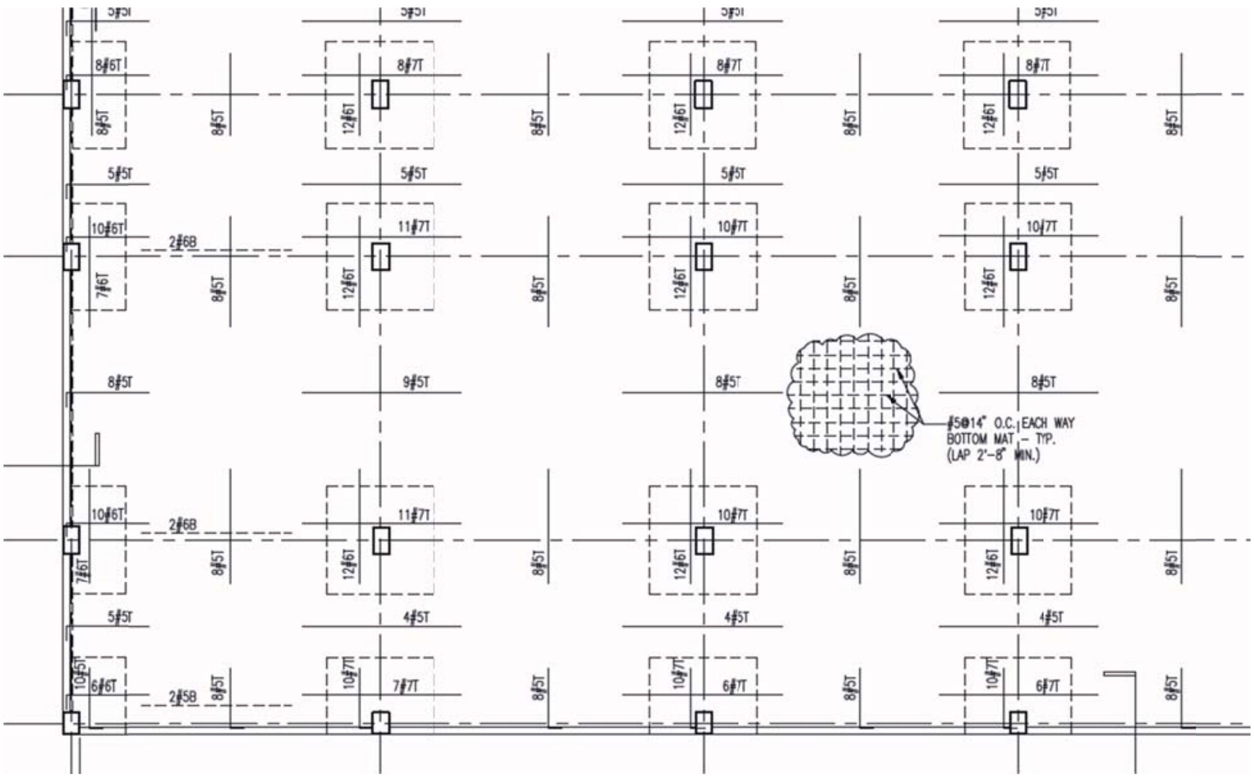

Figure 4: Parking Level Framing Plan

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.

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 5: Partial Column Schedule

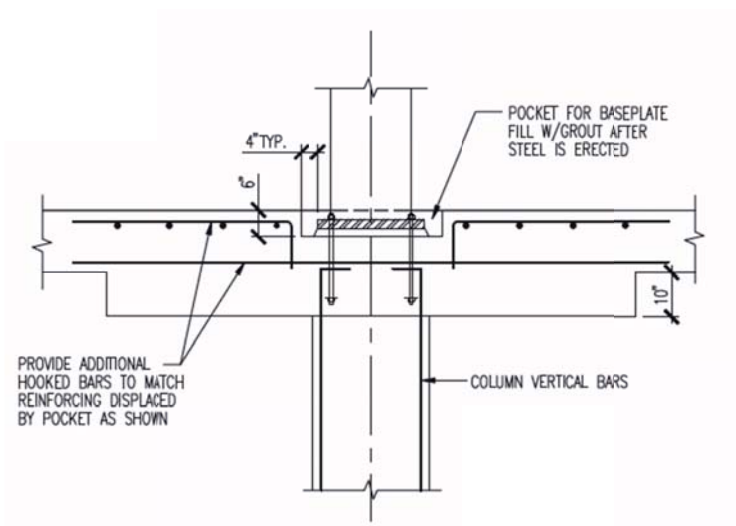

Figure 6: Baseplate Pocket Detail

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's. See Figure 7: Partial Framing Plan.

Figure 7: Partial Framing Plan

Columns

The columns for the office tower are steel wide flange shapes. 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.

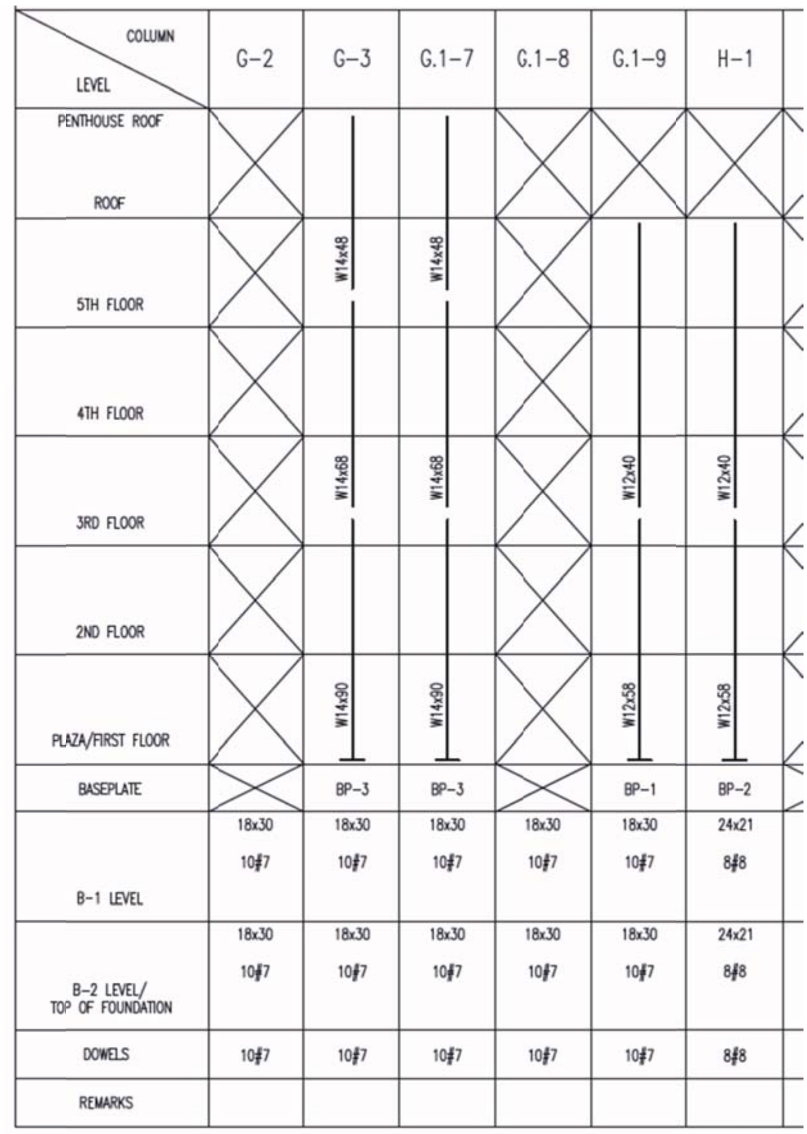

Figure 8: Partial Column Schedule

Roof System

The roof structure consists of K series open web joists and wide flange shapes. 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.

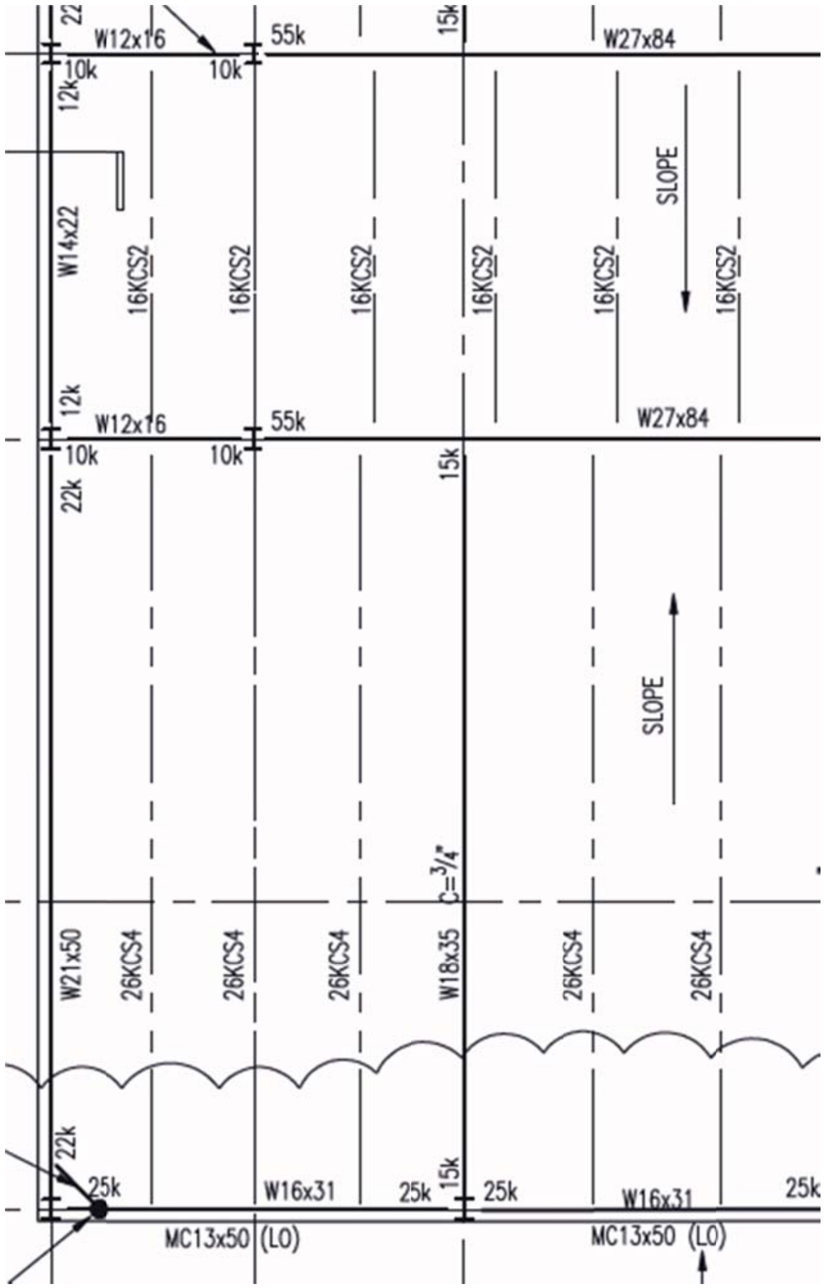

Figure 9: Partial Roof Framing Plan

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.

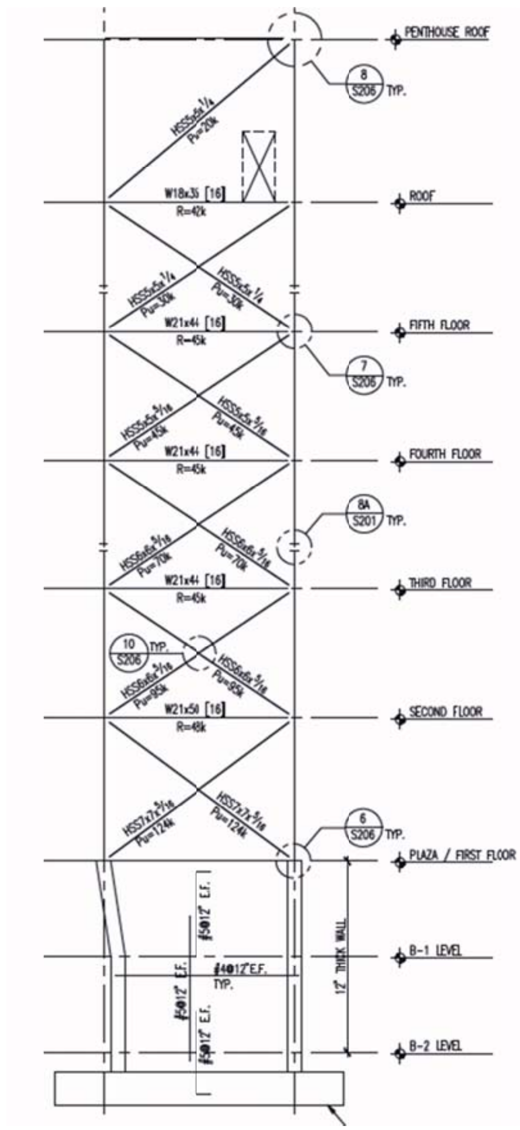

Figure 10: Braced Frame and Shear Wall Elevation

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 + 15 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 315.6 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	11.02
	20	0.62	17.84	11.99
	25	0.66	18.99	12.76
	30	0.70	20.14	13.53
	40	0.76	21.87	14.70
	50	0.81	23.31	15.66
	60	0.85	24.46	16.44
	69	0.89	25.61	17.21
Leeward	All	0.89	25.61	-6.45

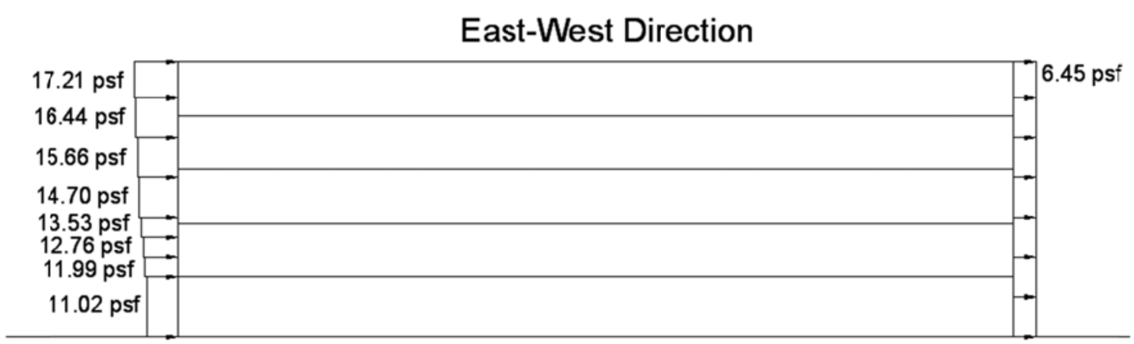

Figure 11: Wind Pressures East-West Direction

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	10.63
	20	0.62	17.84	11.56
	25	0.66	18.99	12.31
	30	0.70	20.14	13.05
	40	0.76	21.87	14.17
	50	0.81	23.31	15.10
	60	0.85	24.46	15.85
	69	0.89	25.61	16.60
Leeward	All	0.89	25.61	-10.37

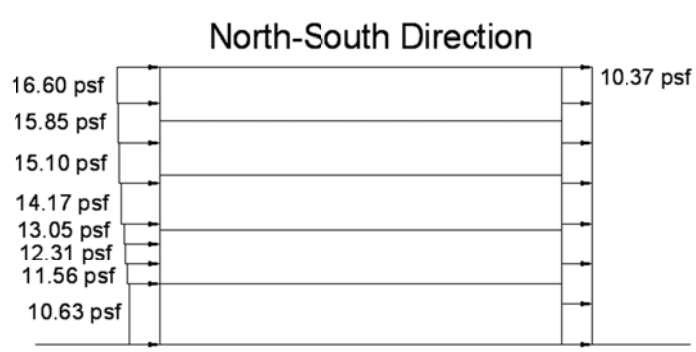

Figure 12: Wind Pressures North-South Direction

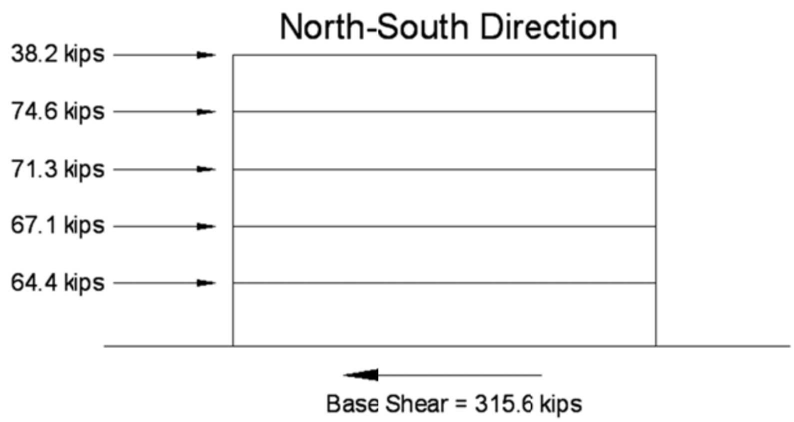

Figure 13: Wind Story Forces North-South Direction

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

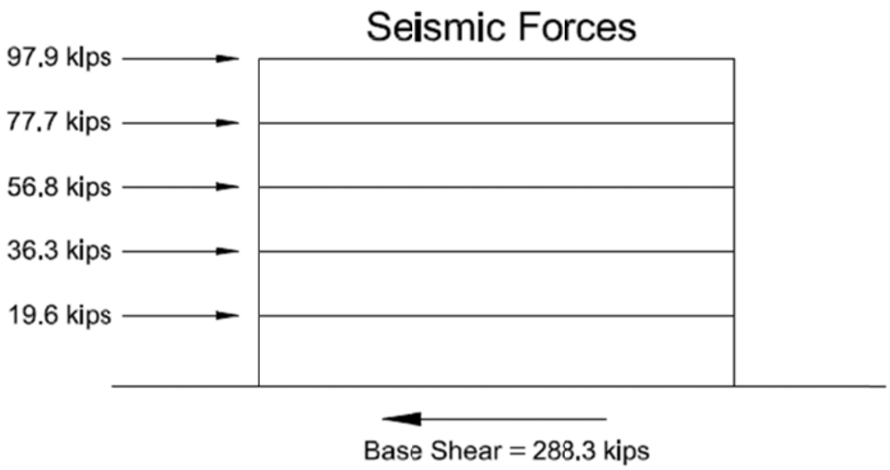

Figure 14: Seismic Story Forces

Lateral Load Distribution

The lateral loads are resisted by the four braced frames in the office tower and the shear walls in the subgrade parking structure. These four brace braced frames are highlighted in red and labeled in Figure 15 below. For this report, the floor slabs were modeled in ETABS as rigid diaphragms. The lateral loads are transferred through the floor slab and into the braced frames. The loads are then transferred from the braced frames into the shear walls in the subgrade parking structure, and then into the foundations below. The relative stiffness of each of the braced frames was determined by modeling each of them in ETABS and applying a 1 kip horizontal at the top of the braced frame, and determining the displacement of the frame. By determining the relative stiffness of each braced frame, the total load that each braced frame takes can be determined. The table below shows the stiffness of each of the four braced frames. ETABS output for each braced frame can be seen in Appendix B.

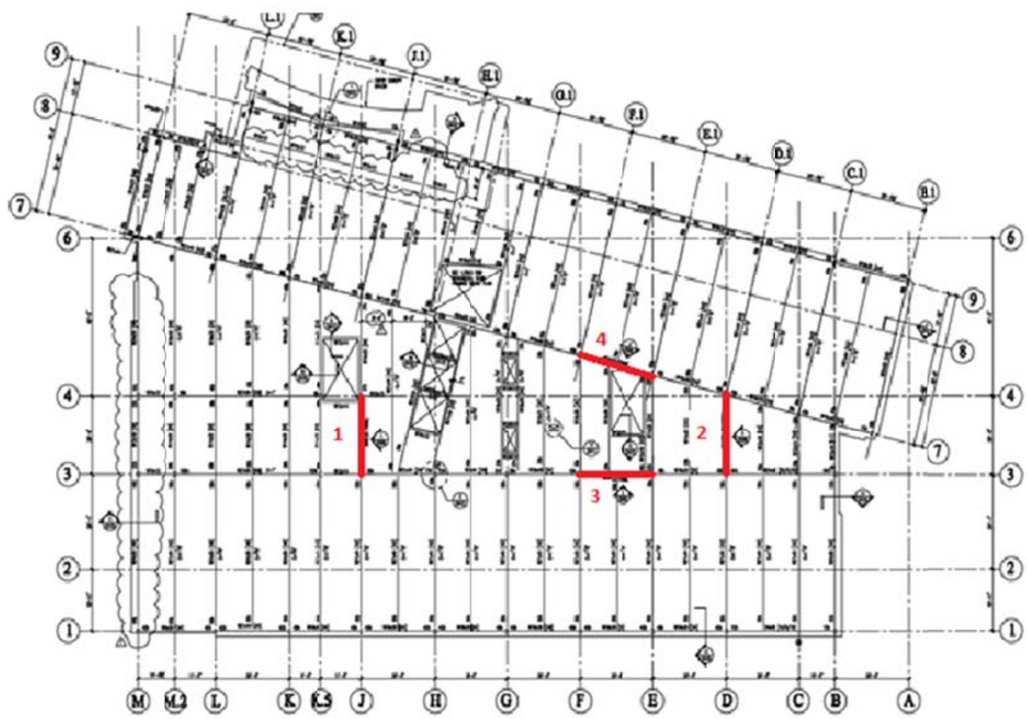

Figure 15: Floor Plan with Braced Frames

Braced Frame Stiffness				
Braced Frame #	**P (kip)**	**Dp (in)**	**K (k/in)**	**Rel K**
1	1	0.034415	29.06	26.73
2	1	0.049499	20.20	18.58
3	1	0.033432	29.91	27.51
4	1	0.033836	29.55	27.18

Hand calculations were done to determine the distribution of the lateral loads to the four braced frames of the building. The calculations were done for the wind loads in both directions and for the seismic loads in both directions. The calculations include torsion due to the fact that the center of rigidity and center of mass are at different locations. In order to simplify the calculations, braced frame #4 was assumed to run in the E-W direction even though it is at angle. The detailed hand calculations can be seen in appendix B.

E-W Wind Load Distribution to Braced Frames

Braced Frame	Rigidity K (k/in)	Total Lateral Load (kip)	e (ft)	d (ft)	k*d^2	Direct Shear (kip)	Torsional Shear (kip)	Total Shear (kip)
#1	29.06	156.0	2.4	46.5	62835.0	0.0	3.9	**3.9**
#2	20.20	156.0	2.4	53.5	57817.5	0.0	3.1	**3.1**
#3	29.91	156.0	2.4	15.5	7185.9	42.9	1.3	**44.3**
#4	29.55	156.0	2.4	8.5	2135.0	42.4	0.7	**43.1**
				Sum	129973.3			

N-S Wind Load Distribution to Braced Frames

Braced Frame	Rigidity K (k/in)	Total Lateral Load (kip)	e (ft)	d (ft)	k*d^2	Direct Shear (kip)	Torsional Shear (kip)	Total Shear (kip)
#1	29.06	315.6	13.4	46.5	62835.0	84.4	44.0	**128.3**
#2	20.20	315.6	13.4	53.5	57817.5	58.6	35.2	**93.8**
#3	29.91	315.6	13.4	15.5	7185.9	0.0	15.1	**15.1**
#4	29.55	315.6	13.4	8.5	2135.0	0.0	8.2	**8.2**
				Sum	129973.3			

E-W Seismic Load Distribution to Braced Frames

Braced Frame	Rigidity K (k/in)	Total Lateral Load (kip)	e (ft)	d (ft)	k*d^2	Direct Shear (kip)	Torsional Shear (kip)	Total Shear (kip)
#1	29.06	288.3	2.4	46.5	62835.0	0.0	7.2	**7.2**
#2	20.20	288.3	2.4	53.5	57817.5	0.0	5.8	**5.8**
#3	29.91	288.3	2.4	15.5	7185.9	79.3	2.5	**81.8**
#4	29.55	288.3	2.4	8.5	2135.0	78.4	1.3	**79.7**
				Sum	129973.3			

N-S Seismic Load Distribution to Braced Frames

Braced Frame	Rigidity K (k/in)	Total Lateral Load (kip)	e (ft)	d (ft)	k*d^2	Direct Shear (kip)	Torsional Shear (kip)	Total Shear (kip)
#1	29.06	288.3	13.4	46.5	62835.0	77.1	40.2	**117.2**
#2	20.20	288.3	13.4	53.5	57817.5	53.6	32.1	**85.7**
#3	29.91	288.3	13.4	15.5	7185.9	0.0	13.8	**13.8**
#4	29.55	288.3	13.4	8.5	2135.0	0.0	7.5	**7.5**
				Sum	129973.3			

ETABS Model

Due to the irregular floor plan of the ASHA National Office building, a computer model was created using ETABS. Only the four braced frames, shear walls, and the basement walls were modeled because these are the elements in the building that resist lateral loads. Each floor was modeled using rigid diaphragms, with an added area mass to account for the self-weight. The shear walls were modeled as shell elements and meshed into areas with a maximum dimension of 24”x24”. The shear walls and basement walls are located below grade, therefore their moments of inertia were not reduced. Line elements were used to model the columns, beams and cross bracing. The beams and columns are W-Flange shapes and the cross bracing consists of square HSS shapes. Two grids were created for this model. One of the grids is rotated 14.04 degrees clockwise off of the global axis. Figure 16 and 17 show three dimensional views of the ETABS model that was created for this report. Figure 18 is a typical floor plan of the ETABS model that shows the locations of the four braced frames.

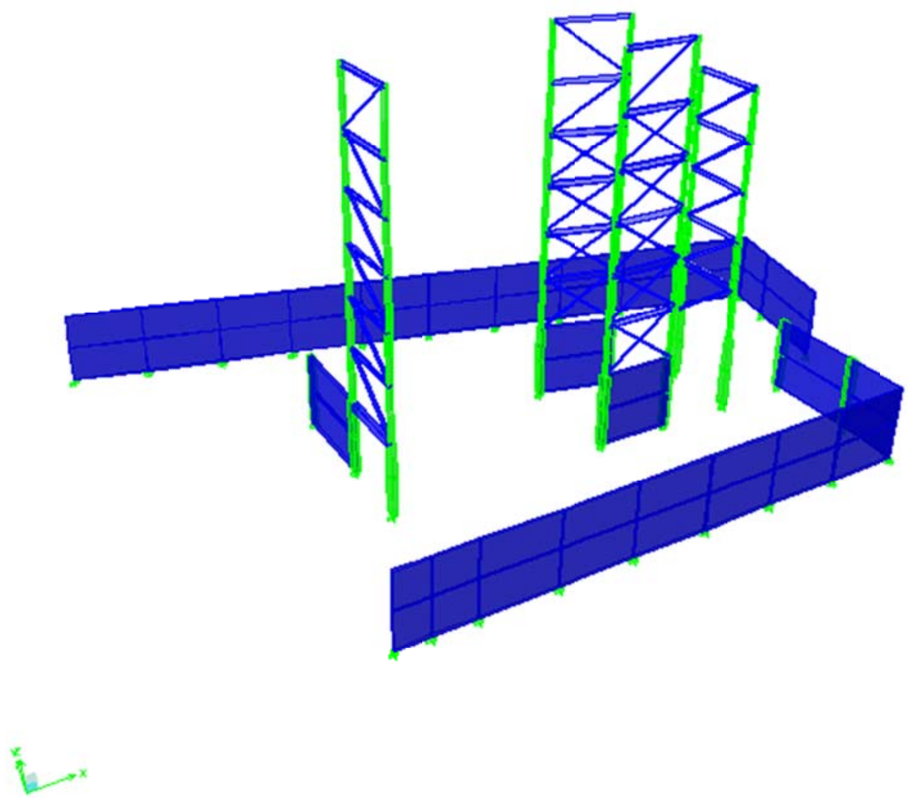

Figure 16: ETABS Model (Diaphragms Not Shown)

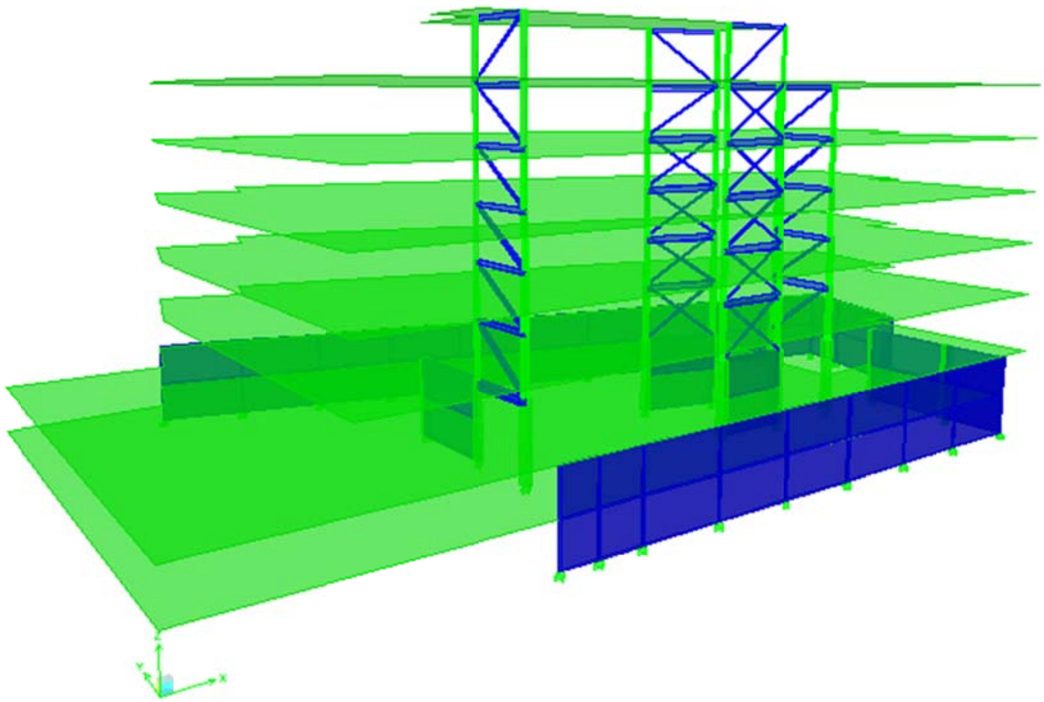

Figure 17: ETABS 3D Model (Diaphragms Shown)

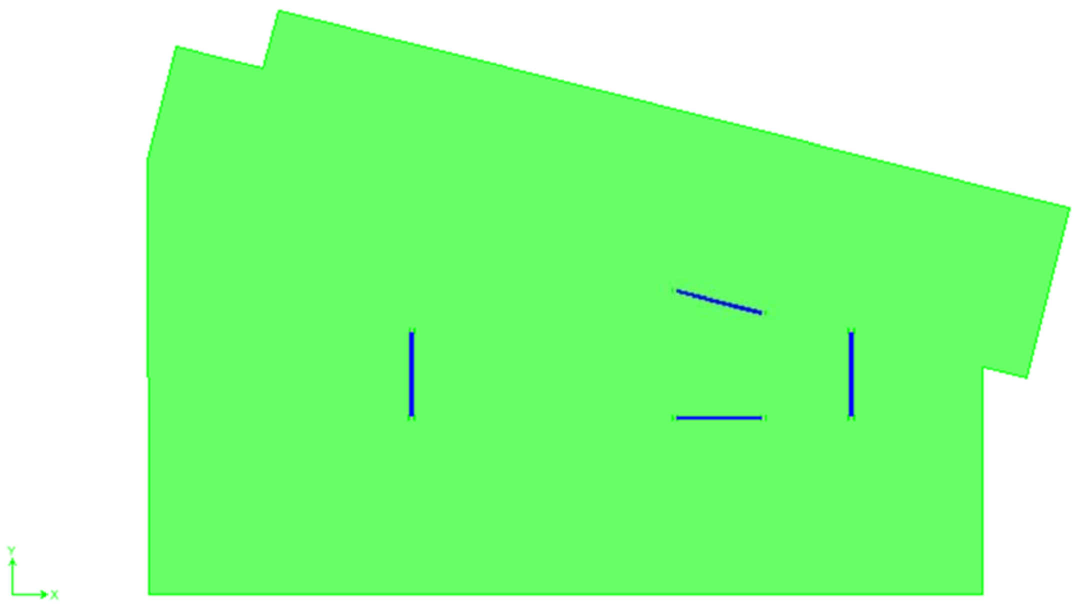

Figure 18: ETABS Model Typical Floor Plan

Load Combinations

The ASCE 7-10 section 2.3 strength design load combinations were considered for this technical report. It is important to note that the load combinations have changed in the new ASCE 7-10 compared to ASCE 7-05. These load cases include both gravity and lateral loads. The load combinations considered in this analysis are shown below.

1. 1.4D
2. 1.2D + 1.6L + 0.5Lr
3. 1.2D + 1.6Lr + 0.5W
4. 1.2D + 1.0W + 1.0L + 0.5Lr
5. 1.2D + 1.0E + 1.0L
6. 0.9D + 1.0W
7. 0.9D + 1.0E

All of these load combinations were put into the ETABS model. The displacements, member forces and reactions were analyzed to determine the governing load cases. Different load cases govern the design depending on the member that is being analyzed. For this reason, all load cases were considered. Load cases 6 and 7 generally govern for overturning, which is address later in this report.

Drift and Displacement

Story drift and the total lateral displacement of the building were checked for this report. According to ASCE 7-10, the allowable seismic story drift for a building in the occupancy category II is $0.020h_{sx}$. The accepted standard for total building displacement for wind loads is L/400. The ETABS building model was utilized to determine the story drifts and displacements. The unfactored loads were used to determine the seismic story drift, and the factored loads were used to determine the wind drift. The tables below show the story drifts for the wind and seismic loads versus the allowable drifts. As seen, the actual drifts are within the limits of the code and accepted standards.

Seismic Story Drift E-W Direction				
Floor	**Displacement (in)**	**Story Drift (in)**	**Allowable Story Drift (in)**	**Okay?**
PH Roof	2.234730	0.359764	3.84	Yes
Roof	1.874966	0.443783	3.24	Yes
Fifth	1.431183	0.431621	3.24	Yes
Fourth	0.999562	0.393187	3.24	Yes
Third	0.606375	0.344709	3.24	Yes
Second	0.261666	0.256500	3.6	Yes
Plaza	0.005166	0.004079	2.4	Yes
Parking	0.001087	0.001087	2.4	Yes

Seismic Story Drift N-S Direction				
Floor	**Displacement (in)**	**Story Drift (in)**	**Allowable Story Drift (in)**	**Okay?**
PH Roof	2.337196	0.349581	3.84	Yes
Roof	1.987615	0.421611	3.24	Yes
Fifth	1.566004	0.459493	3.24	Yes
Fourth	1.106511	0.433214	3.24	Yes
Third	0.673297	0.349156	3.24	Yes
Second	0.324141	0.317231	3.6	Yes
Plaza	0.006910	0.004778	2.4	Yes
Parking	0.002132	0.002132	2.4	Yes

Wind Story Displacement E-W Direction			
Floor	**Displacement (in)**	**Allowable Displacement (in)**	**Okay?**
PH Roof	0.841926	3.150	Yes
Roof	0.714409	2.670	Yes
Fifth	0.576085	2.265	Yes
Fourth	0.425502	1.860	Yes
Third	0.274697	1.455	Yes
Second	0.126942	1.050	Yes
Plaza	0.002472	0.600	Yes
Parking	0.000536	0.300	Yes

Wind Story Displacement N-S Direction			
Floor	**Displacement (in)**	**Allowable Displacement (in)**	**Okay?**
PH Roof	1.776954	3.150	Yes
Roof	1.531884	2.670	Yes
Fifth	1.269819	2.265	Yes
Fourth	0.952589	1.860	Yes
Third	0.620307	1.455	Yes
Second	0.320803	1.050	Yes
Plaza	0.008982	0.600	Yes
Parking	0.002742	0.300	Yes

Building Torsion

The ASHA National Office building will experience torsion due to lateral loads because the center of mass of the building is not at the same location as the center of rigidity. Due to the irregular shape of the building, the ETABS model was used to obtain both the center of mass and the center of rigidity of each floor. The eccentricity was then multiplied by the force applied to each floor diaphragm to determine the total moment cause by torsion. The ETABS model automatically accounts for this inherent torsion in the building. The seismic loads are also applied at an eccentricity of 5% of the building length. This accounts for the accidental torsion that occurs in the building. The table below shows the building torsion calculations due to the seismic loads in both the East-West and North-South directions. The calculations include torsion due to inherent torsion and accidental torsion.

Building Torsion E-W Direction - Seismic Loading							
Floor	**Story Force (kip)**	**Location of COR**	**Location of COM**	**e_x (ft)**	**M_t (ft-k)**	**M_a (ft-k)**	**M_{total} (ft-k)**
Roof	19.6	55.35	57.93	2.58	50.5	98.0	148.5
Fifth	36.3	55.87	57.93	2.06	74.6	181.5	256.1
Fourth	56.8	55.77	57.93	2.15	122.3	284.0	406.3
Third	77.7	55.78	57.93	2.15	167.0	388.5	555.5
Second	97.9	55.15	57.93	2.78	272.2	489.5	761.7
						Total	**2128.1**

Building Torsion N-S Direction - Seismic Loading							
Floor	**Story Force (kip)**	**Location of COR**	**Location of COM**	**e_x (ft)**	**M_t (ft-k)**	**M_a (ft-k)**	**M_{total} (ft-k)**
Roof	19.6	137.20	124.10	13.10	256.8	205.8	462.6
Fifth	36.3	137.10	124.10	13.00	472.0	381.2	853.1
Fourth	56.8	137.25	124.10	13.15	746.7	596.4	1343.1
Third	77.7	136.03	124.10	11.93	927.0	815.9	1742.8
Second	97.9	138.30	124.10	14.20	1390.6	1028.0	2418.5
						Total	**6820.2**

Overturning and Foundation Considerations

When a building is subjected to lateral loads, often times the effects of overturning moments can become an issue at the foundations of the building. Load cases 6 and 7 control for overturning. The table below shows the overturning moment calculations for the building. The overturning moments were determined for the seismic loads and the wind loads in both directions. As seen in the table, the overturning moment due to the seismic loads is the greatest. The effects of the overturning moment need to be considered for the foundations. According to the geotechnical report, the foundations can be designed for a maximum allowable soil bearing capacity of 6,000 psf. The foundations must be designed as to not to exceed this soil capacity, so that a soil failure does not occur.

Oveturning Moments							
		Seismic		E-W Wind		N-S Wind	
Floor	**Height (ft)**	**Lateral Force (k)**	**Moment (ft-k)**	**Lateral Force (k)**	**Moment (ft-k)**	**Lateral Force (k)**	**Moment (ft-k)**
Roof	79	97.9	7734.1	19.1	1508.9	38.2	3017.8
Fifth	65.5	77.7	5089.4	37.2	2436.6	74.6	4886.3
Fourth	52	56.8	2953.6	35.6	1851.2	71.3	3707.6
Third	38.5	36.3	1397.6	33.6	1293.6	67.1	2583.4
Second	25	19.6	490.0	30.5	762.5	64.4	1610.0
	Total Overturning Moment		**17664.6**		**7852.8**		**15805.1**

Member Checks

Spot checks on various members were done to verify that the members are adequate for the gravity and lateral loads. The ETABS model was used to obtain the loads. The controlling load case depended upon which member was being considered. A cross bracing HSS member in braced frame #1 was checked for adequate tension and compression strength. A W-flange column in braced frame #3 was checked for combined axial and bending forces. A concrete column that supports braced frame #2 was checked for the axial force and moment that it is subjected to. The concrete column was checked using spColumn. All of the members checked were found to be adequate for the loads applied to them. The members that were checked are highlighted below in Figures 19, 20 and 21. Detailed calculations for the member checks are shown in Appendix B.

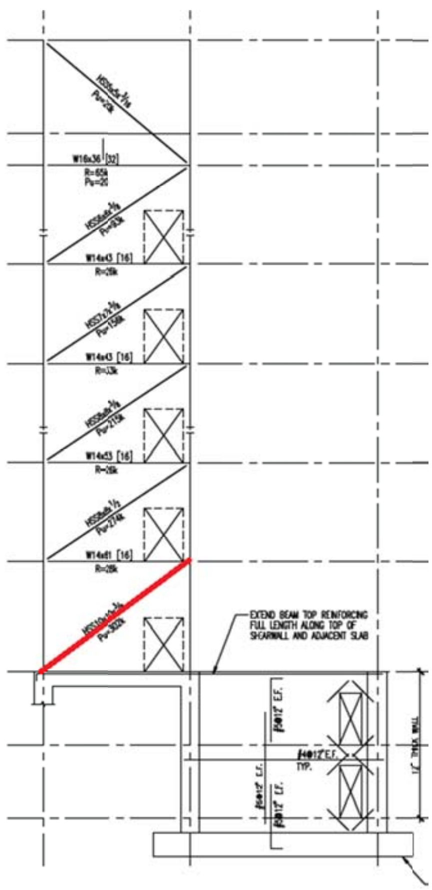

Figure 19: HSS Cross Bracing Member

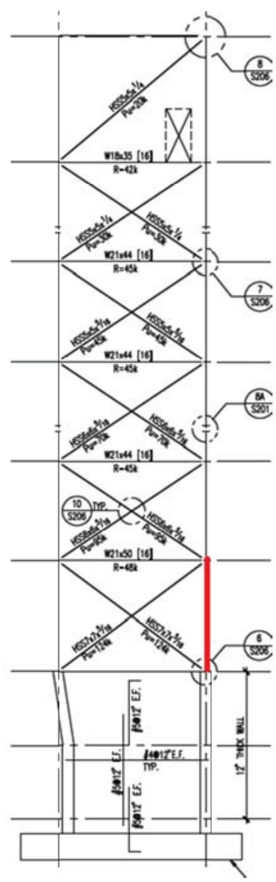

Figure 20: W-Flange Column

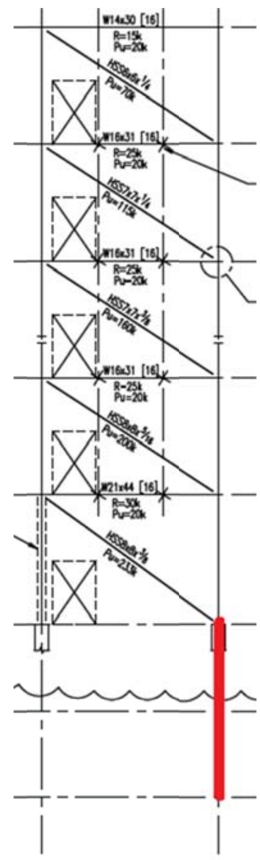

Figure 21: Concrete Column

Conclusions

This report explores the lateral system for the ASHA National Office building. The braced frame system with subgrade shear walls was analyzed using hand calculations and an ETABS computer model. The computer model was used due to the complex shape of the building plan, and allowed for a more accurate method of determining the load on each of the braced frames and members within those braced frames. Building torsion and overturning were considered along with the impact of the lateral loads on the foundations.

The story drifts and displacements were analyzed for the wind and seismic loads, and members were spot checked using the ASCE 7-10 load cases. The building was found to meet the drift and displacement serviceability requirements and standards. The actual story drift and displacements were well under the allowable values. The three members that were spot checked included a cross bracing member, a steel W-Flange column and a concrete column. All three were found to meet strength requirements. Because the strength and serviceability requirements were all met, it was determined that the lateral system of the building is adequate for the loads.

Appendix A: Typical Framing Plans

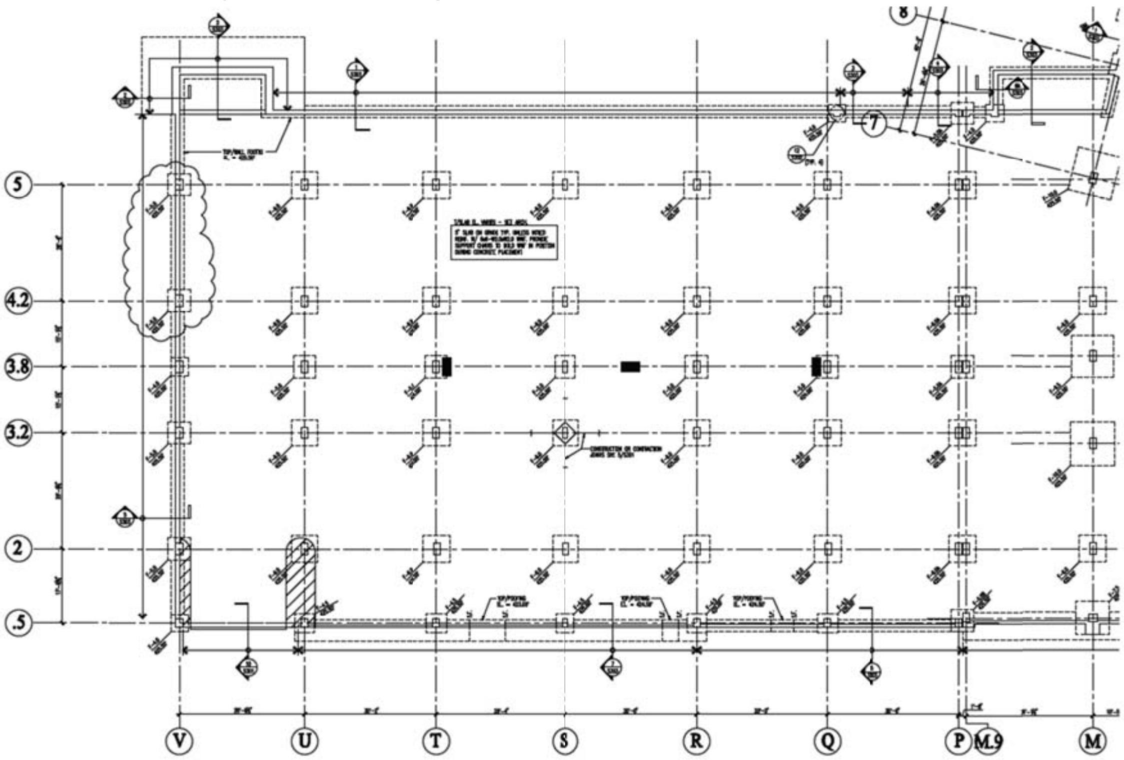

Figure 22: Foundation Plan Part A

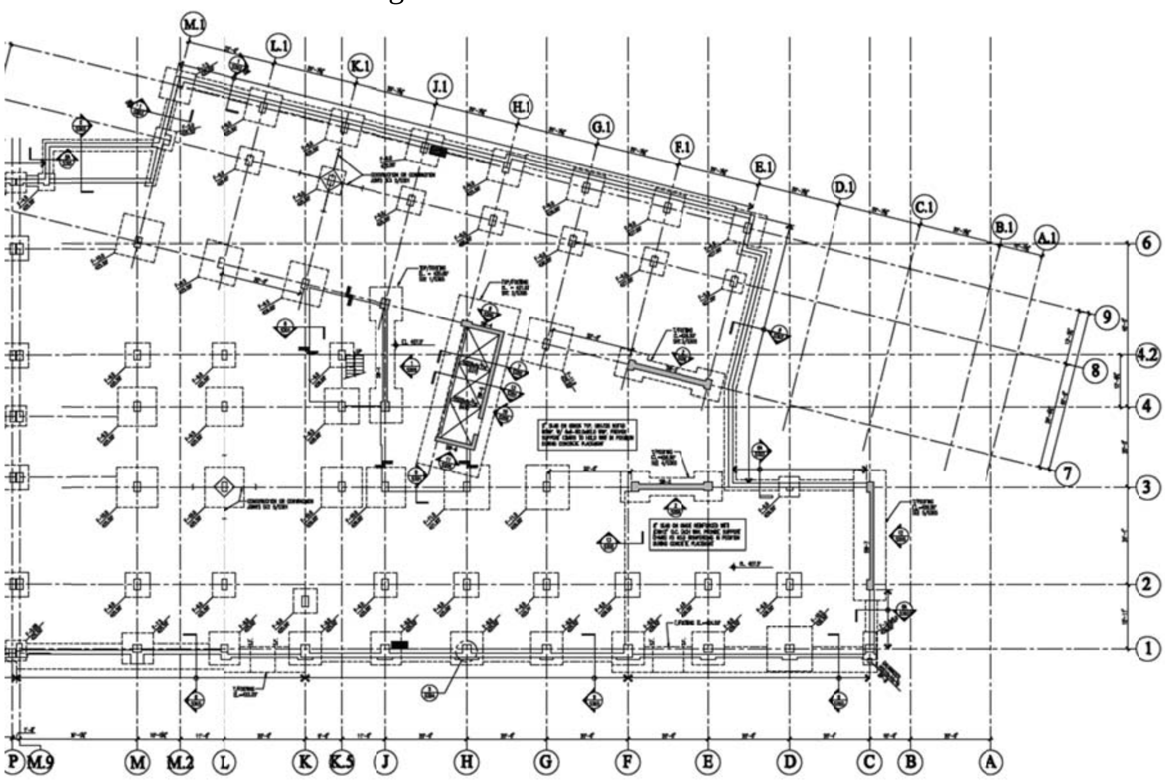

Figure 23: Foundation Plan Part B

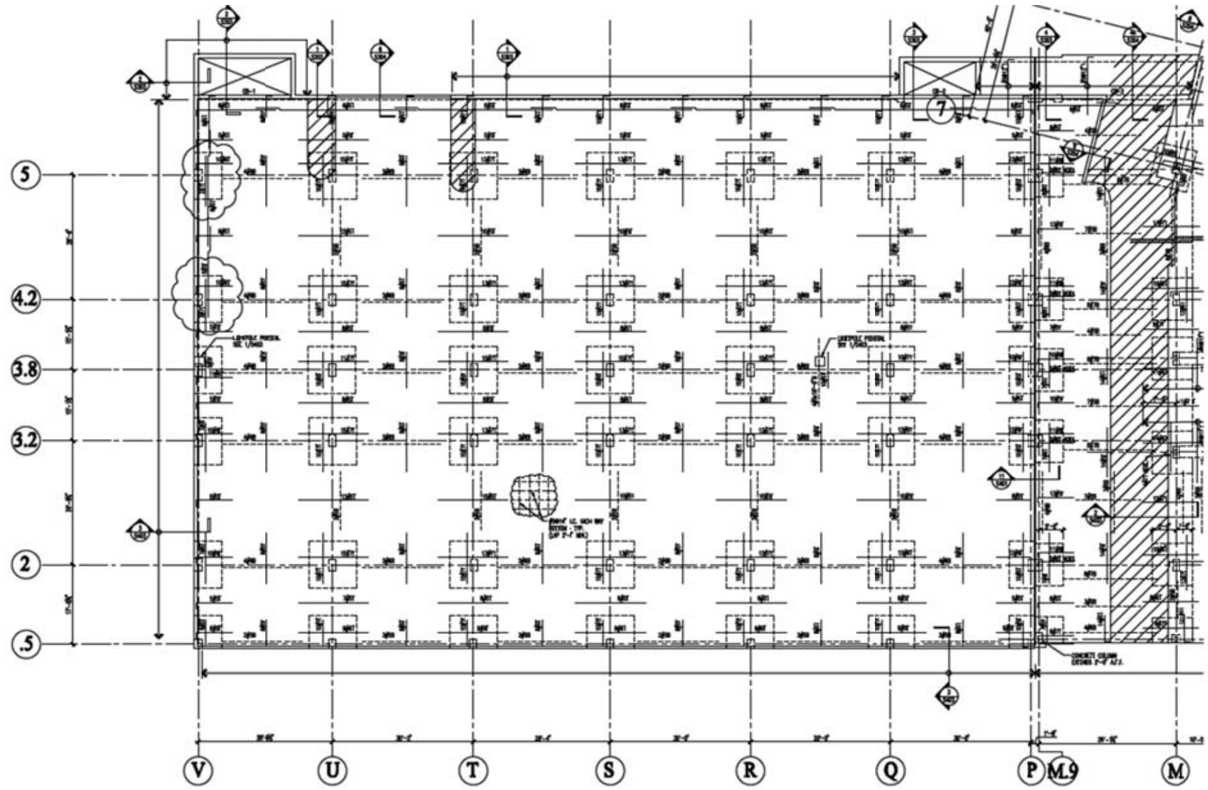

Figure 24: Plaza Level Framing Plan Part A

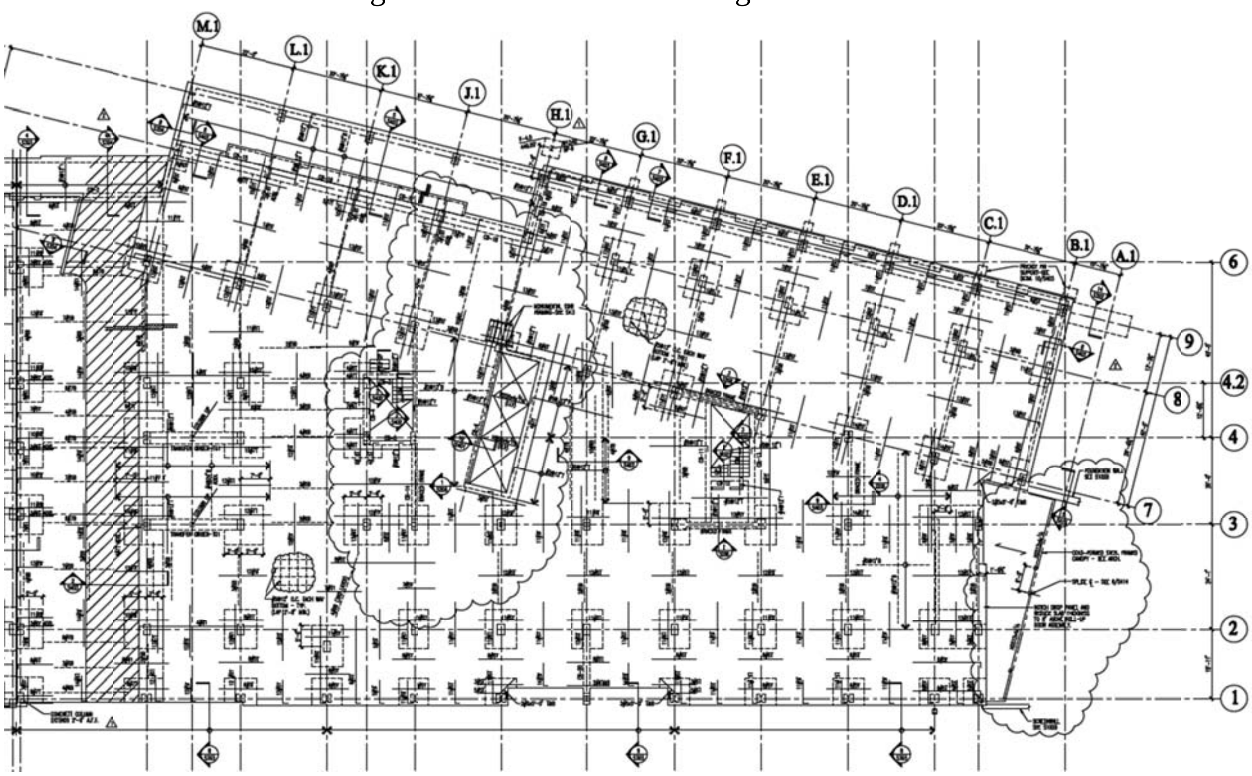

Figure 25: Plaza Level Framing Plan Part B

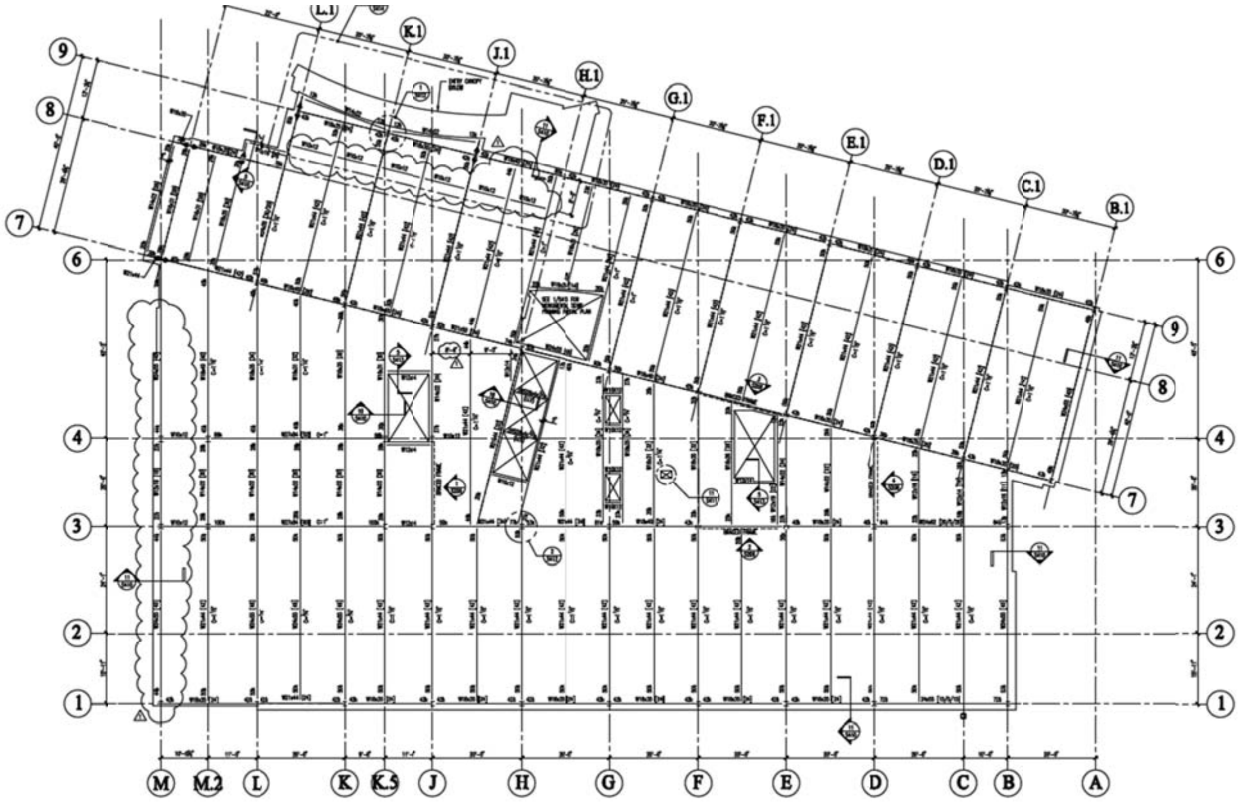

Figure 26: Second Floor Framing Plan

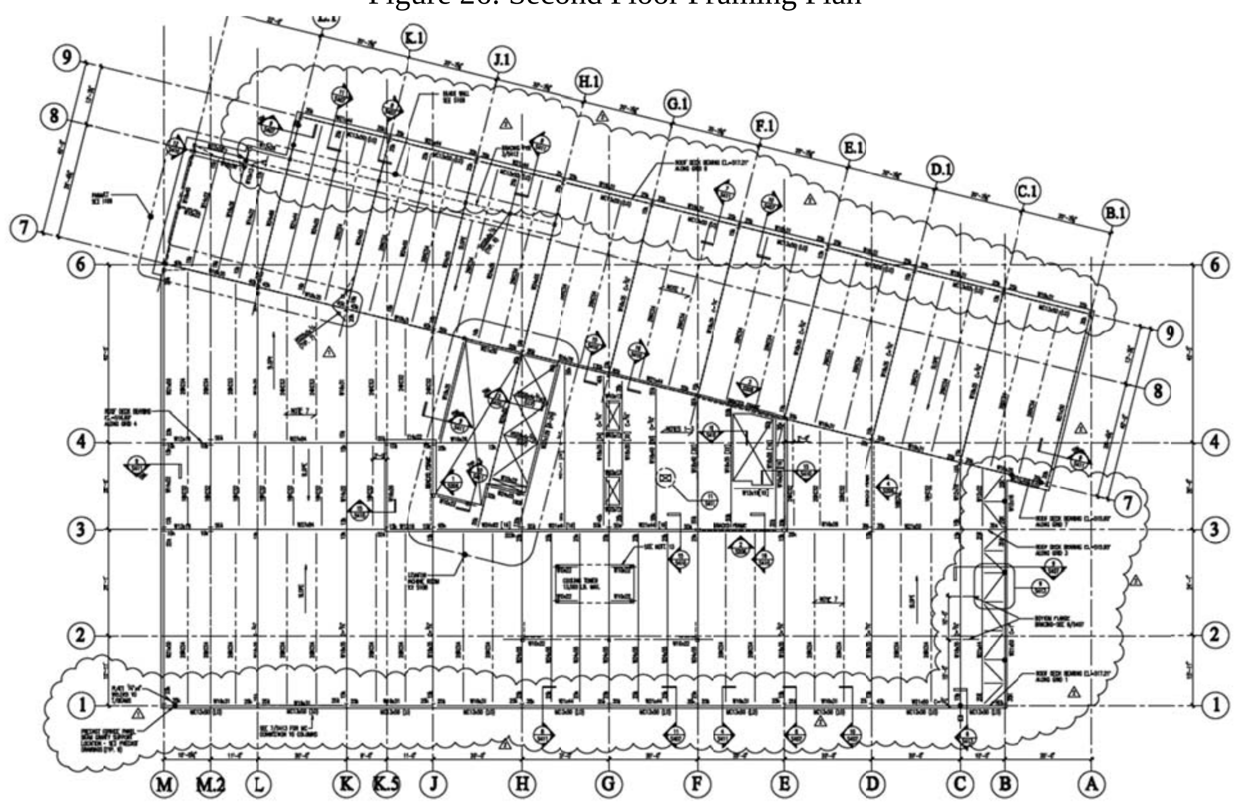

Figure 27: Roof Framing Plan

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Appendix B: Calculations

Braced Frame Stiffness Calculations

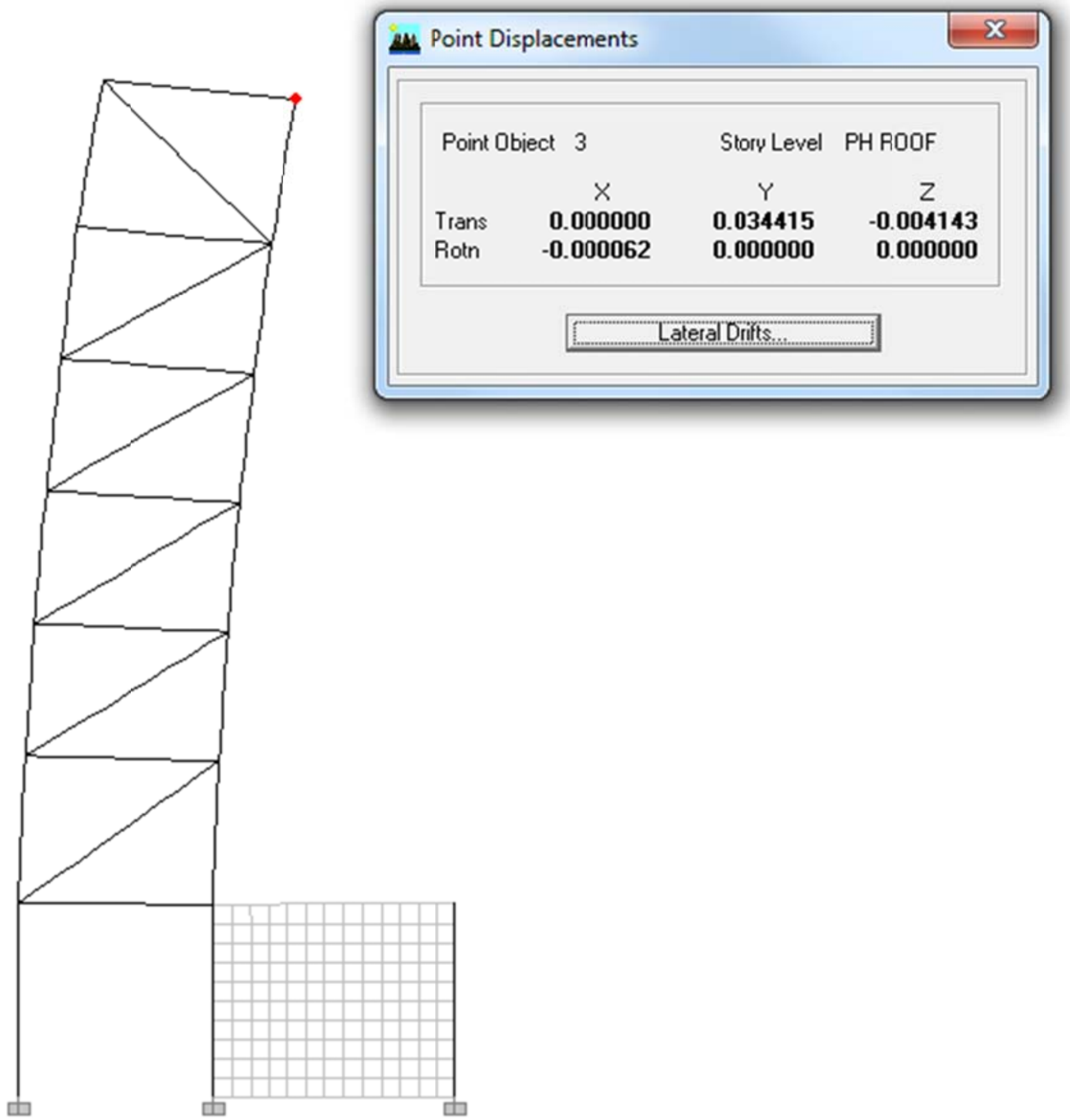


Figure 28: Braced Frame #1

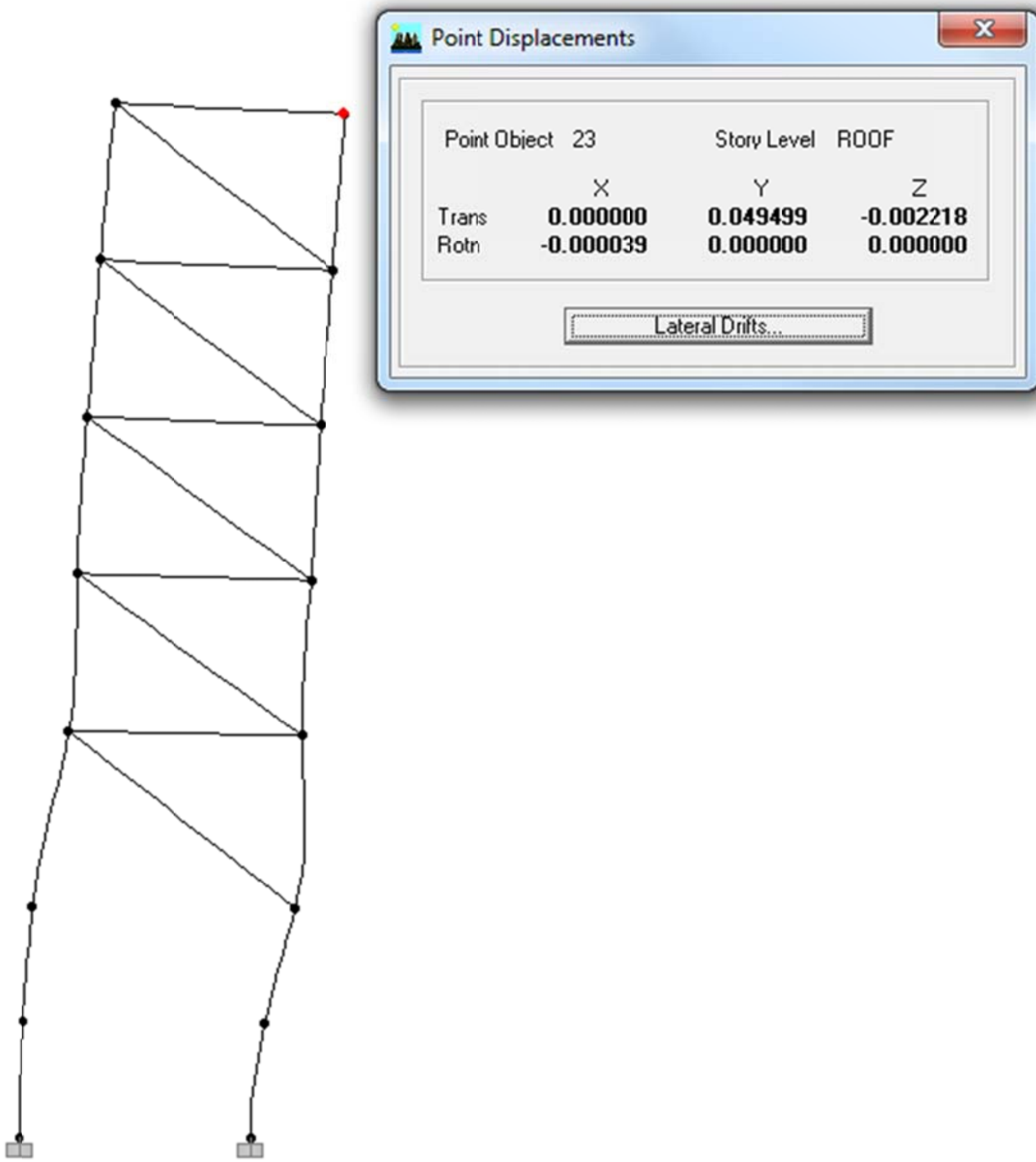


Figure 29: Braced Frame #2

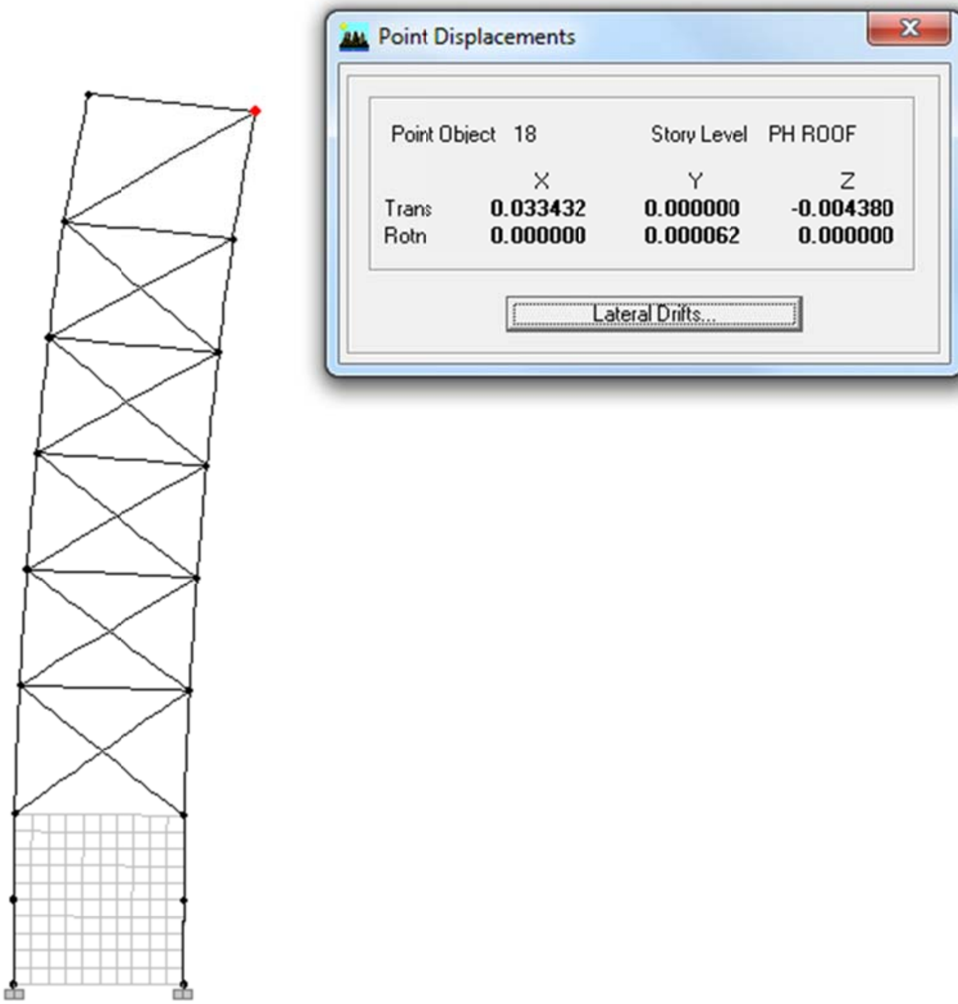


Figure 30: Braced Frame #3

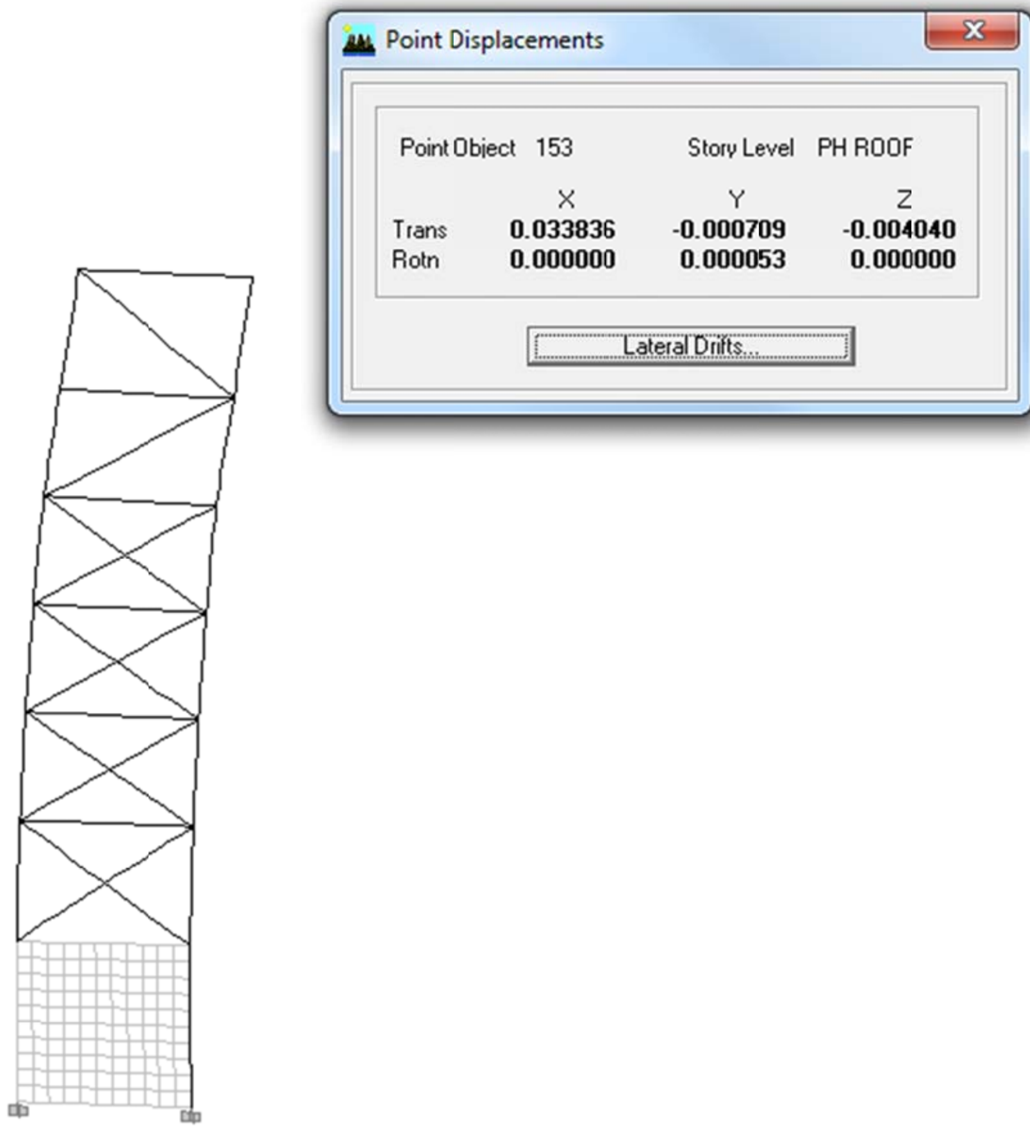
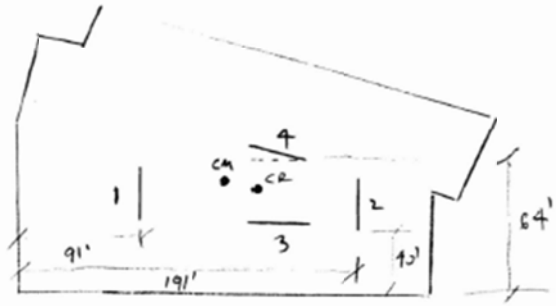


Figure 31: Braced Frame #4

Tech 3 Report | Distribution of Lateral Forces | 11/18/10



$K_1 = 29.06$ k/in

$K_2 = 20.20$ k/in

$K_3 = 29.91$ k/in

$K_4 = 29.55$ k/in

CENTER OF RIGIDITY

$\bar{x}_r = 137.5'$
$\bar{y}_r = 55.5'$ } FROM ETABS

CENTER OF MASS

$\bar{x}_M = 124.1'$

$\bar{y}_M = 57.9'$

DIRECT LOADS E-W

BRACED FRAMES 3 & 4 TAKE E-W LOADS

$$F_{D3} = \frac{29.91}{29.91 + 29.55} P = 0.503\, P_x \leftarrow$$

$$F_{D4} = \frac{29.55}{29.91 + 29.55} P = 0.497\, P_x \leftarrow$$

DIRECT LOADS N-S

$$F_{D1} = \frac{29.06}{29.06 + 20.20} P = 0.590\, P_Y \downarrow$$

$$F_{D2} = \frac{20.20}{29.06 + 20.20} P = 0.410\, P_Y \downarrow$$

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TORSIONAL LOADS N-S $e = 137.5 - 124.1 = 13.4'$

$$F_{T1} = \frac{k_1 d_1 P e}{\Sigma k_j d_j^2} = \frac{29.06 \cdot 46.5 \cdot P \cdot 13.4}{29.06 \cdot 46.5^2 + 20.20 \cdot 53.5^2 + 29.91 \cdot 15.5^2 + 29.55 \cdot 8.5^2} = 0.139 P_y \downarrow$$

$$F_{T2} = \frac{20.20 \cdot 53.5 \; P \cdot 13.4}{129973.3} = 0.111 P_y \uparrow$$

$$F_{T3} = \frac{29.91 \cdot 15.5 \; P \cdot 13.4}{129973.3} = 0.048 P_y \rightarrow$$

$$F_{T4} = \frac{29.55 \cdot 8.5 \; P \cdot 13.4}{129973.3} = 0.026 P_y \leftarrow$$

Torsional Loads E-W $e = 57.9 - 55.5 = 2.4'$

$$F_{T1} = \frac{2.4}{13.4} \cdot 0.139 P = 0.025 P_x \downarrow$$

$$F_{T2} = \frac{2.4}{13.4} \cdot 0.111 P = 0.020 P_x \uparrow$$

$$F_{T3} = \frac{2.4}{13.4} \; 0.048 P = 0.0086 P_x \longrightarrow$$

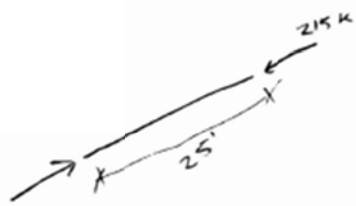
$$F_{T4} = \frac{2.4}{13.4} \cdot 0.026 P = 0.0047 P_x \leftarrow$$

Tech 3 Report | Member Checks | 11/18/10

Check of HSS cross bracing member in braced Frame # 1

MAXIMUM AXIAL LOAD = 215 k (ETABS OUTPUT)

LOAD CASE 6 0.9D + 1.0W GOVERNS



TENSION: $\phi P_n = \phi F_y A_g = 0.9 \cdot 46 \cdot 13.2$

$\phi P_n = 546\,k > P_u = 215\,k$ ∴ OKAY

COMPRESSION $I = 202\,in^4$

$P_{cr} = \dfrac{\pi^2 E I}{l^2} = \dfrac{\pi^2 \cdot 29000 \cdot 202}{300^2} = 642\,k$

$\phi P_n = 0.9 \cdot 642 = 578\,k > P_u = 215\,k$ ∴ OKAY

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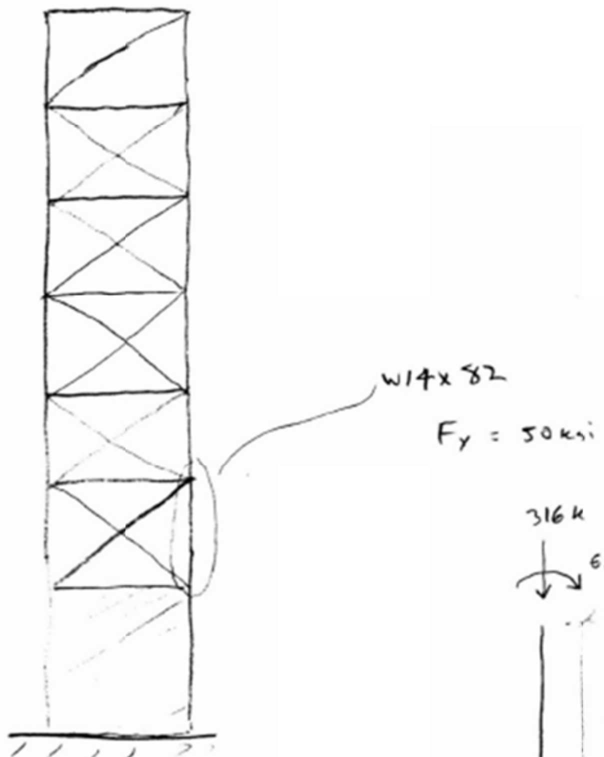
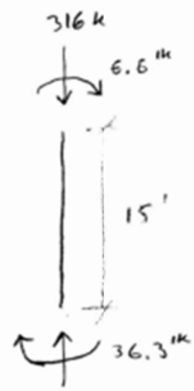
Tech 3 Report	MEMBER CHECKS	11/18/10

Check of W-Flange column in braced frame # 3





LOAD CASE 5 CONTROLS
1.2D + 1.0E + 1.0L

TABLE 6-1 AISC MANUAL

$p = 1.36 \times 10^{-3}$

$b_x = 1.89 \times 10^{-3}$

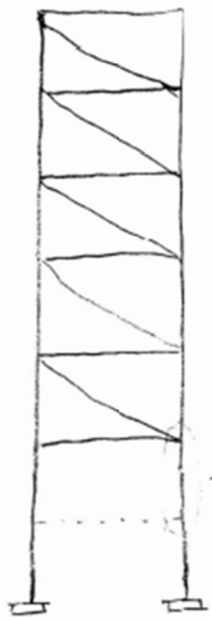
$pP_r = 1.36\times10^{-3} \cdot 316 = 0.430 > 0.2$

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

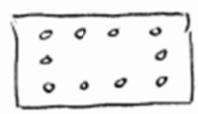
$0.430 + 1.89\times10^{-3} \cdot 36.3 = 0.499 \leq 1.0 \quad \therefore$ OK

Tech 3 Report | Member Checks | 11/18/10

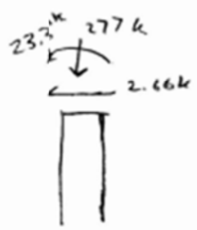
Check of concrete column in braced frame #2



18" x 30" concrete column



10 - #9 BARS

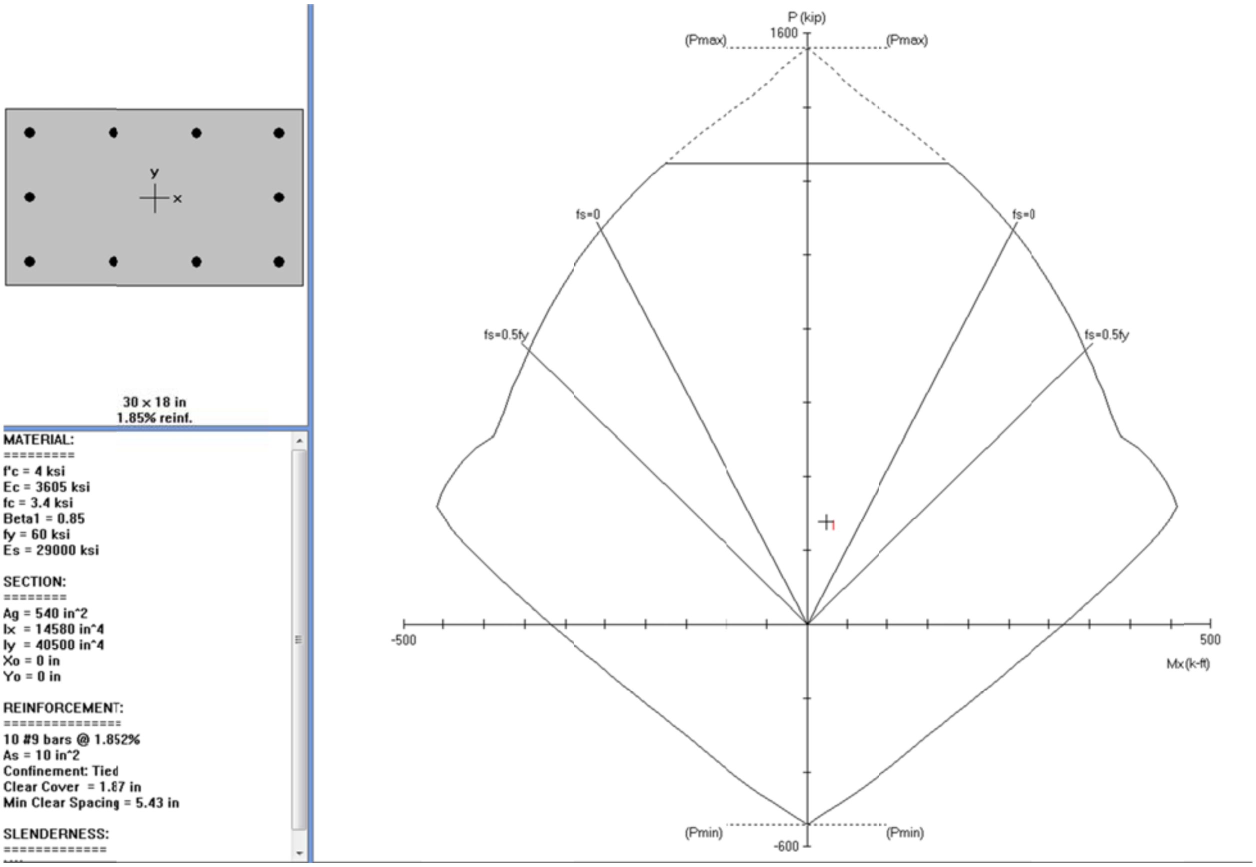


LOAD CASE
0.9D + 1.0W CONTROLS

It can be assumed that the column can handle the 2.66 k Shear load

SP Column was used to determine if the column is adequate for the applied loads.

As seen from the output below, the column works


y
x
30 x 18 in
1.85% reinf.
MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi
SECTION:
========
Ag = 540 in^2
Ix = 14580 in^4
Iy = 40500 in^4
Xo = 0 in
Yo = 0 in
REINFORCEMENT:
===============
10 #9 bars @ 1.852%
As = 10 in^2
Confinement: Tied
Clear Cover = 1.87 in
Min Clear Spacing = 5.43 in
SLENDERNESS:
=============
P (kip)
1600
(Pmax)
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
-500
500
Mx (k-ft)
(Pmin)
(Pmin)
-600
1