Technical Report 3

Lateral System Analysis
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November 12, 2012

Office Building

Sayre, PA

Seth M. Moyer | Structural

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Executive Summary

The Office Building is being constructed as part of an office complex development project located in Sayre, PA. The building is five stories tall (all above grade), extending up to 67'-0" at the mean roof height (top of parapet elevation = 74'-5"), and has 85,075 ft^2 of total floor area. The floor structure is made up of 4" thick concrete slabs on composite steel deck (4" total combined depth). The slab is carried by open web steel joists which are supported by wide flange steel beams. The beams carry the gravity loads to wide flange steel columns that distribute the loads down to the foundation. The lateral system of the Office Building consists of 16 double angle braced frames (8 in each the N-S and E-W directions).

The purpose of Technical Report 3 is to present a detailed lateral analysis of the Office Building. For this report, a thorough analysis was performed using fully developed lateral loading conditions and an ETABS model of the lateral force resisting system. The floors and roof were modeled as rigid diaphragms with additional mass and load assignments to account for the dead weight and superimposed loads on the structure. The roof diaphragm was treated rigidly for simplification and to be consistent with the constraints that were placed on the floors by their diaphragm definitions.

Hand calculations to determine the centers of mass of the floors and roof were compared to the ETABS output values. The centers of mass varied slightly between the two methods. This is likely due to the fact that in the model, the total weight of the floor and exterior walls at each level were added together and then distributed equally over the entire floor diaphragm area. By hand, the centers of mass of the floors and exterior walls were calculated separately and then the weighted average was found. The assumed theoretical centers of rigidity were also compared to the values from ETABS. It is likely that the centers, according to ETABS, were shifted upwards (north in plan) slightly based on the additional consideration of the stiffness/rigidity of the floor diaphragms instead of the braced frames alone.

All applicable loads and load cases had to be developed and input into the ETABS model. Individual calculated loads and ASCE 7-10 strength design load combinations were used to consider direct and torsional (inherent and accidental) effects on the structure. Due to the symmetric layout of the lateral system elements, the total number of load combinations used in the analysis was able to be reduced significantly to 19. For Seismic Design Category B, the redundancy factor was allowed to be taken as 1.0 and accidental torsional moment amplification was not required, so no amplifying modification of the calculated seismic loads was necessary.

The ETABS model was analyzed including P-Delta effects, which were set to "Non-iterative – Based on Mass." The maximum forces in each of the 16 braced frames and the maximum brace forces were determined from the analysis output as well as the specific load combinations that caused them. Drifts and displacements due to wind were compared to the industry accepted value of H/400, and those due to seismic forces were compared to the ASCE 7-10 allowable

story drift value of $0.020h_{sx}$. For the wind loading, several individual story drifts exceeded acceptable values, particularly at the roof (story 5). However, the total lateral displacements at the top of the building were within the typical industry limit. All seismic drift values were well within the code allowance.

The worst case overturning moment scenario was determined to be with the N-S wind loading condition. This direction provided the least resistance from the moment due to the dead weight of the building. However, the overturning moment was only about 7% of the available resisting moment. Finally, a strength spot check of an upper brace at the first floor, typical in frames P11 and P13, was performed. The double angle brace was sufficient to carry the maximum applied tensile and compressive axial loads.

Building Introduction

The Office Building is being constructed as part of a multi-phase office complex development project in Sayre, PA. Upon completion, currently slated for April 2013, the building will provide office and meeting space. It will also feature a fitness wing and locker rooms for employees on the second floor. With five stories (all above grade) extending up to 67'-0" at the mean roof height (top of parapet elevation = 74'-5"), the 85,075 sq ft Office Building has been designed for a total occupancy load of 1134.

The footprint of the Office Building is laid out in an off-centered "H" configuration (See Figure 1). The façade enclosing the east and west wings is primarily made up of insulated metal panels on 6" cold formed metal studs. 6' high horizontal glazing strips break up the exterior at each story. The portion of the building that connects the two wings is enclosed with a curtain wall glazing system. Figure 2 shows an elevation of the south-facing (main entrance) side of the building in which you can see both the wings and connecting portion. The parapet extends up past the roof to a maximum height of 74'-5" along both the east and west facades. It tapers down to a height of 68'-2 1/2" at the interior edge of the wings and continues at that elevation across the connecting segment.

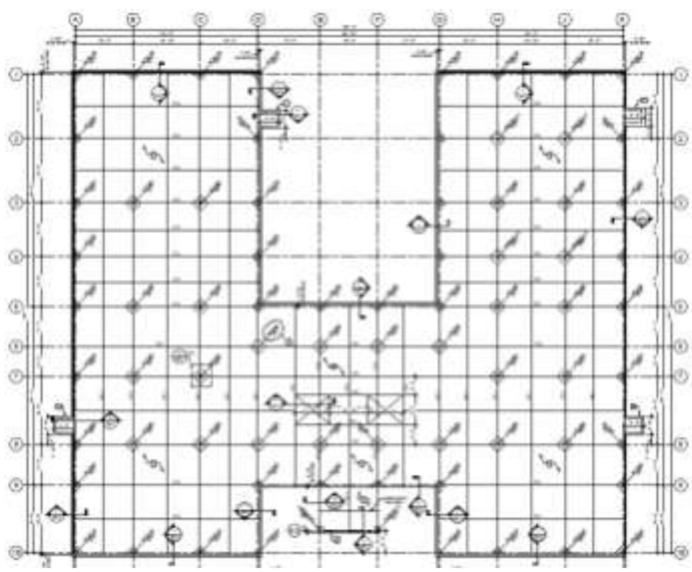


Figure 1: First Floor Slab Plan
(Image Credit: Larson Design Group)

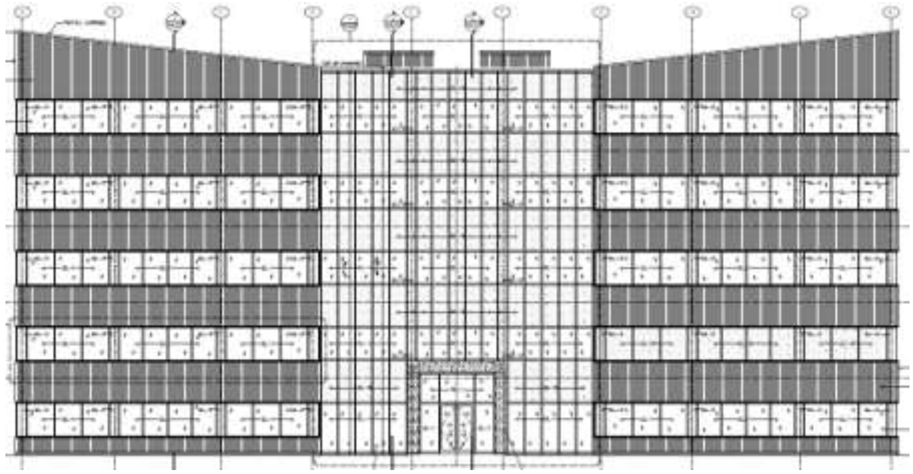


Figure 2: South Elevation
(Image Credit: Silling Associates, Inc.)

Structural Overview

The Office Building structure is founded on spread, combined and strip footings which support the concrete piers, pier walls, foundation walls and columns directly to transfer the loads from the superstructure to the soil they bear upon. The floor system is made up of 4" thick (total) composite deck floor slabs on open web steel joists (non-composite for joists/beams). The joists frame into wide flange steel beams which transfer the loads to wide flange steel columns. The lateral system consists of braced frames in both the N-S and E-W directions, which all extend up to the roof.

Foundations

The geotechnical report conducted by CME Associates, Inc. for the Office Building site subsurface conditions indicates that spread and continuous footing foundations may be designed for an allowable soil bearing pressure of 4,000 psf. The report also specifies that spread footings should not be less than 3'-3" square and continuous strip footings should not be less than 2'-3" wide to prevent excessive settlements.

Typical interior columns are supported directly by spread footings just under the slab-on-grade. Typical perimeter columns sit on concrete piers that extend down to the spread footings. To protect against frost heave, perimeter footings have a minimum of 4'-0" of soil above their bearing elevation, measured from the bottom of the footing to finish grade. Both 8" and 12" thick concrete foundation walls run continuously along the outside perimeter of the building footprint, centered on 2'-3" strip footings, between the perimeter piers and footings.

At the braced frame locations outlined in Figure 3, 28" thick pier walls extend between the individual column piers. Combined footings also extend from pier to pier. The combined footings help to resist the overturning moments that result from lateral loading along their longitudinal axis. They also help to prevent differential settlement of the individual columns that form the braced frame.

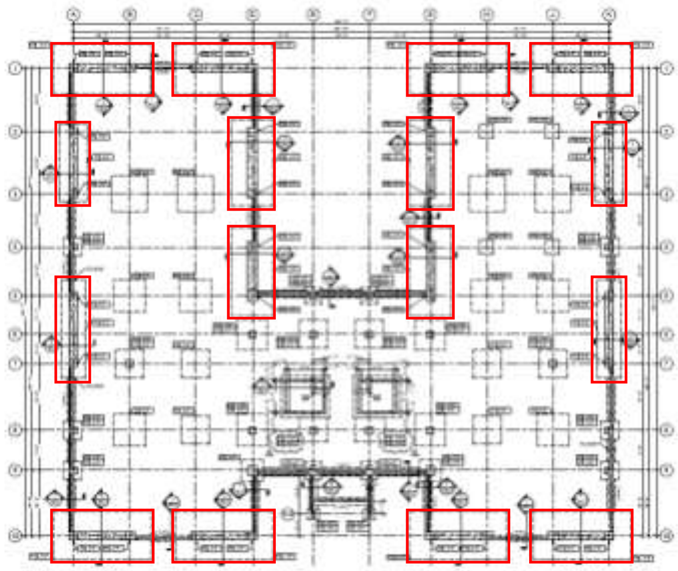


Figure 3: Braced Frames/Combined Footing Locations
(Image Credit: Larson Design Group)

Floor and Framing System

The first floor is a 4” thick slab-on-grade with WWF 6x6 – W2.9xW2.9 at mid-depth. Floors 2-5 consist of 2 1/2” thick normal weight concrete on 20 gauge 1 1/2” composite deck with WWF 6x6 – W4.0xW4.0 at mid-depth (4” total slab thickness). The composite deck slab is supported by open web steel joists (typically 16K2 up to 16K4) spaced at 3’-0” on center max. The floor joists distribute the gravity loads to the wide flange beams (interior beams are typically W24s and the exterior beams range from W12 to W16). The maximum beam span is 36’, between grid lines 1 and 3, for the W24x76 interior beams along grid lines B, C, H and J.

The beams carry the loads to wide flange columns to then be dispersed to the foundation. Typical column sizes include W12x53, W12x65, W12x79 and W12x106. All typical columns are spliced at 30’-8” above first floor (4’ above the third floor). Where the fitness room is located in the east wing on level 2, HSS6x6x1/4 columns run up to the bottom of the W24x55 and W24x76 beams at grid points H2, H4, J2 and J4. The primary purpose of these one story columns is to reduce vibrations in the bays supporting the fitness center activities, which might otherwise create a serviceability issue with the light system of framing being utilized.

An enlarged portion of the typical floor framing plan can be seen in Figure 4 below.

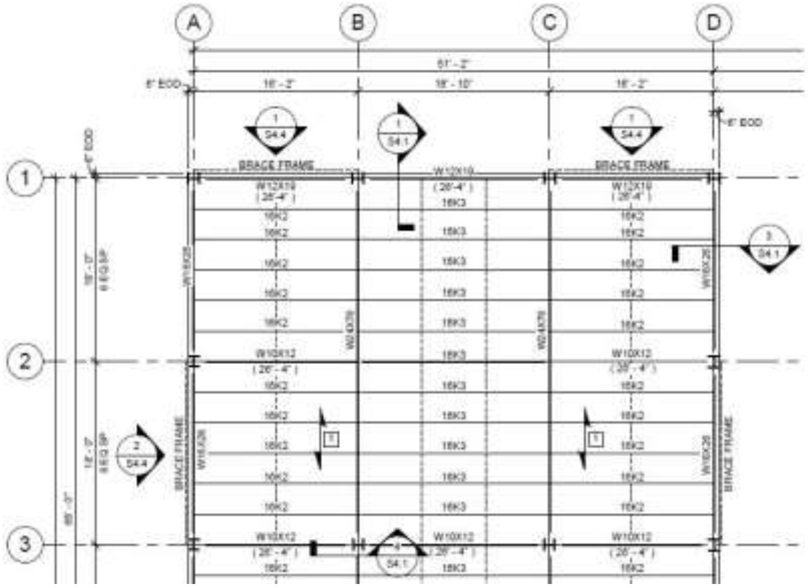


Figure 4: Typical Floor Framing Plan (Enlarged)
(Image Credit: Larson Design Group)

Roof and Framing System

The roof structure is made up of 1 1/2” Type B 20 gauge wide rib roof deck. A maximum thickness of 4” of rigid insulation is laid on top of the deck and is covered with fully adhered EPDM roof membrane. The deck is typically supported by 16KCS2 and 24K4 open web steel joists spaced at 6’-0” on center max. The joists then rest on W21x44 interior beams (towards which they slope down from the perimeter beams) and either W12x19 or W14x22 exterior beams. All gravity loads are then transferred to the wide flange columns.

An enlarged portion of the typical roof framing plan can be seen in Figure 5 below.

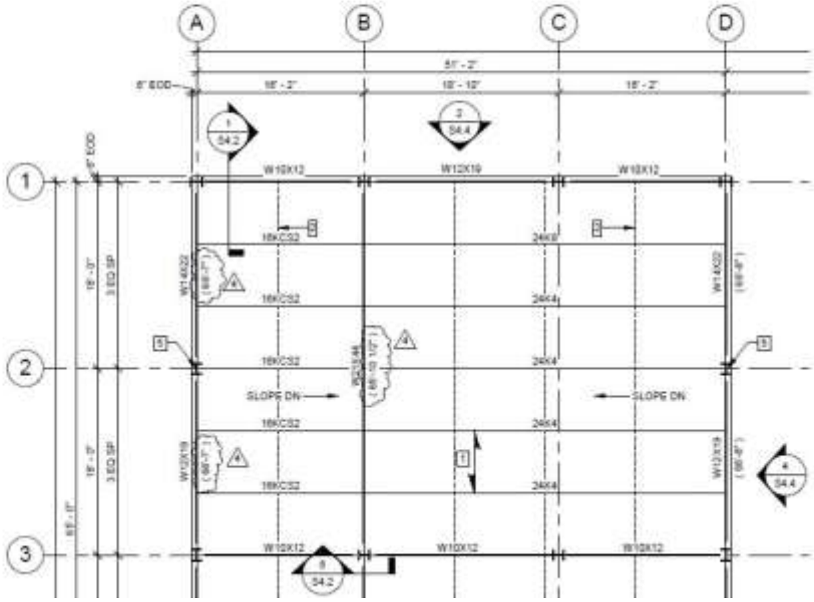


Figure 5: Typical Roof Framing Plan (Enlarged)
(Image Credit: Larson Design Group)

Lateral System

The lateral force resisting system of the Office Building is made up of 16 “K” braced frames (8 in each the N-S and E-W directions) (See Figure 3 for plan locations). The double angles brace the center work point of the perimeter beam at each floor down to the horizontal double angle-to-column intersection points above the windows of the floor below and up to the horizontal double angle-to-column intersection points below the windows of the floor above (double angles brace the base of the columns to the center work point of the horizontal wide flange beam below the windows at level 1).

Wind pressures on the exterior of the building are collected by the façade and the resultant forces are transferred into the floor/roof diaphragms. The diaphragms at each story act rigidly and transfer the story shear forces to the braced frames that run parallel to the direction of the loading (the roof diaphragm has been treated as rigid for simplification of modeling and analysis, although it will likely behave as flexible since it is constructed of untopped steel decking). The braced frames resist the lateral loads based on the proportion of their relative stiffness. These story forces accumulate at each floor, moving down through the building until the total base shear is transferred into the ground via the foundation.

Similarly, for seismic loads induced by the building’s response to ground motion/acceleration, the total base shear is distributed to the diaphragms at each story as a function of the respective heights and weights attributed to each level. Once distributed, the seismic forces are transmitted through the diaphragms and into the braced frames based on relative stiffness.

Similarly, the story forces accumulate and are eventually transferred down to the bearing soils through the foundation.

Design Codes

The major model and design codes and standards used in the design of the Office Building:

- Pennsylvania Uniform Construction Code (PAUCC)
- International Building Code 2009 (IBC 2009) (as adopted and modified by the PAUCC)
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Specification for Structural Concrete (ACI 301-05)
- Building Code Requirements for Structural Concrete (ACI 318-08)
- Specification for Structural Steel Buildings (AISC 360-05)
- Standard Specifications for Open Web Steel Joists, K-Series (SJI-K-1.1 05)
- Design Manual for Composite Decks, Form Decks, Roof Decks and Cellular Metal Floor Deck with Electrical Distribution, SDI Pub. No. 29

The same codes and standards are being referenced for use in this technical report with the following exceptions:

- ASCE 7-10
- AISC Steel Construction Manual, 14th Edition, LRFD
- Specification for Structural Steel Buildings (AISC 360-10)
- Building Code Requirements for Structural Concrete (ACI 318-11)

Materials Used

Materials were referenced from Sheets S0.1 and S0.2 and are summarized below in Figure 6.

Steel		
Type	ASTM Standard	Grade
W and WT Shapes	A992	50
Standard Shapes	A36	N/A
Angles, Channels and Plates	A36	N/A
HSS	A500	B
Pipe	A53, E or S	B
Anchor Rods	F1554	N/A
Shear/Anchor Studs	A108	N/A
Deformed Anchors	A496	N/A
Bolts (Plain)	A307	N/A
Bolts (High Strength)	A325	N/A
Nuts	A563	C
Hardened Washers	F436	N/A
Plate Washers	A36	N/A
Deformed and Plain Bars	A615	60
Welded Wire Reinforcement	A185	N/A
Steel Deck	A611	C,D,E
or Steel Deck	A653-94	33
Zinc Coated Steel Sheet	A1003	N/A
Hot Dipped, Galvanized Finish	A123	N/A
Load-Bearing Cold-Formed	C955-07	N/A
SS Pipes and Tubes	A312	N/A
SS Bars and Fittings	A582	N/A
Alum. Pipes and Tubes	B429	N/A
Alum. Bars and Fittings	B221	N/A
SS Fasteners	A240/A666	N/A

Concrete		
Usage	Weight	f'c (psi)
Foundation Walls	Normal	4500
Column Piers	Normal	4500
Combined Footings	Normal	4500
Exterior Slabs-on-Grade	Normal	4500
Specified Column Piers	Normal	5500
Elements Not Specified	Normal	3000

Miscellaneous	
Type	Standard
Grout (6000 psi)	ASTM C1107
Weld Electrodes	AWS Class E7018

Figure 6: Materials Summary

Gravity Loads

Dead, live and snow loads will be calculated and compared to the design loads used by the structural engineer. Spot checks of various typical framing members will then be made using the loads that were calculated.

Dead and Live Loads

Dead loads for the roof and floors were calculated using the actual weights of construction materials and additional allowances to account for superimposed loads due to MEP and ceiling materials as well as various structural framing. The calculated values of both the roof and floor dead loads matched the design values (See Figure 7 below). Refer to Appendix A for a detailed breakdown of the gravity load calculations.

Dead Loads (psf)		
	Design	Calculated
Roof	20	20
Floor	60	60

Figure 7: Dead Load Summary

Live loads for the roof and floors were determined from ASCE 7-10, Table 4-1 for office buildings and roofs. For optimal flexibility of the Office Building in years to come, 80 psf for corridors above the first floor was selected as well as an additional allowance of 20 psf for partitions. This total load of 100 psf for the floors will allow for a variety of configurations of the office space instead of just designing for the corridors where they fall in the current layout. The calculated values for both the roof (minimum live load from Table 4-1) and floors matched the design values (See Figure 8 below).

Live Loads (psf)		
	Design	Calculated
Roof	20	20
Floor	100	100

Figure 8: Live Load Summary

Snow and Drift Loads

The flat roof snow load was determined to be 21 psf from a ground snow load value of 30 psf (Refer to Appendix A for flat roof snow load calculation details). 21 psf is less than the design snow load of 24 psf. This is due to the fact that the design value was calculated using a thermal factor of 1.1 as opposed to the 1.0 used for the calculation in this report. It was assumed that the roof could be considered warm, since the structure is heated and the roof is not openly ventilated, and therefore Ct=1.0. However, using the thermal factor of 1.1 is conservative.

The maximum value of the snow drift load was calculated for the longest stretch of roof (lu=155.33') upwind of the full-height parapet. In this case, the drift snow load was found to be a maximum of 57.8 psf directly against the parapet at the east or west exterior walls. This value is superimposed onto the flat roof snow load and results in a maximum snow load value of 78.8 psf at the inside face of the parapet. Refer to Appendix A for the hand calculations of the drift load as well as a loading diagram at the parapet.

Lateral Loads

Wind Loads

Design wind pressures and loads were calculated for both N-S and E-W directions in accordance with ASCE 7-10, Chapter 27 (MWFRS – Directional Procedure). Design pressures were calculated by hand and were resolved into story forces using Excel. Refer to Figures 9-16 and Appendix B for wind loading summary and calculations.

N-S Design Wind Pressures					
Surface	Level	Distance (ft)	Wind Pressure (psf)	Internal Pressure	
				(+)GC_{pi}	(-)GC_{pi}
Windward Wall	1	0	16.63	6.01	-6.01
	2	13.33	16.63	6.01	-6.01
	3	26.67	18.59	6.01	-6.01
	4	40	20.35	6.01	-6.01
	5	53.33	21.53	6.01	-6.01
	Roof	66.67	22.70	6.01	-6.01
	Parapet	74.42	51.38	N/A	N/A
Leeward Wall	1-Roof	66.67	-14.19	6.01	-6.01
	Parapet	74.42	-34.25	N/A	N/A
Side Wall	All	N/A	-19.86	6.01	-6.01
Roof	N/A	0-67	-25.54	6.01	-6.01
	N/A	67-134	-14.19	6.01	-6.01
	N/A	>134	-8.51	6.01	-6.01

Figure 9: N-S Wind Pressures

N-S Wind Forces								
Level	Story Height	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (ft-k)
		Height (ft)	Area (sf)	Height (ft)	Area (sf)			
1	0	N/A	N/A	6.67	1035	0	370.36	0
2	13.33	6.67	1035	6.67	1035	65.83	370.36	877.46
3	26.67	6.67	1035	6.67	1035	69.68	304.54	1858.26
4	40	6.67	1035	6.67	1035	72.72	234.86	2908.76
5	53.33	6.67	1035	6.67	1035	75.15	162.14	4007.82
Roof	66.67	6.67	1035	Varies	570	86.99	86.99	5799.64
							Base Shear (k)	370.36
							Total Overturning Moment (ft-k)	15451.95

Figure 10: N-S Wind Forces

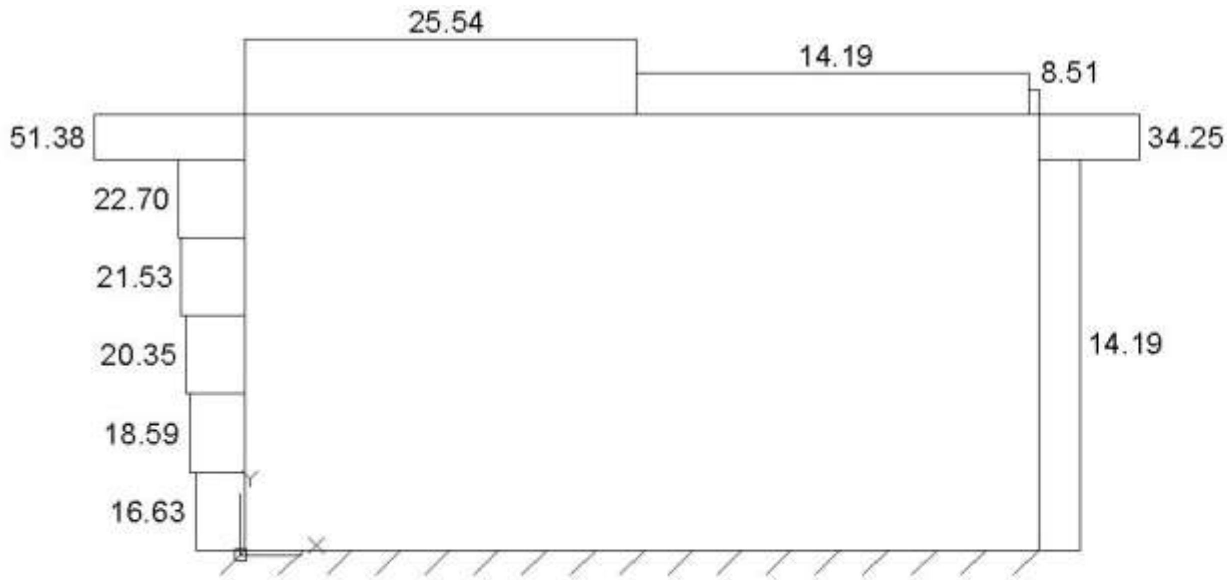


Figure 11: N-S Wind Pressure Diagram

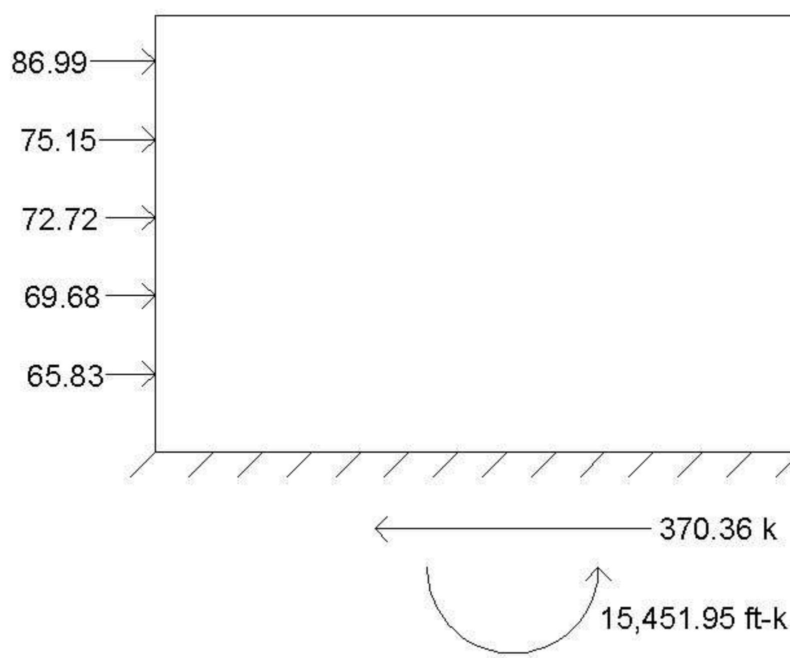


Figure 12: N-S Wind Force Diagram

E-W Design Wind Pressures					
Surface	Level	Distance (ft)	Wind Pressure (psf)	Internal Pressure	
				$(+)GC_{pi}$	$(-)GC_{pi}$
Windward Wall	1	0	16.63	6.01	-6.01
	2	13.33	16.63	6.01	-6.01
	3	26.67	18.59	6.01	-6.01
	4	40	20.35	6.01	-6.01
	5	53.33	21.53	6.01	-6.01
	Roof	66.67	22.70	6.01	-6.01
	Parapet	74.42	51.38	N/A	N/A
Leeward Wall	1-Roof	66.67	-13.34	6.01	-6.01
	Parapet	74.42	-34.25	N/A	N/A
Side Wall	All	N/A	-19.86	6.01	-6.01
Roof	N/A	0-67	-25.54	6.01	-6.01
	N/A	67-134	-14.19	6.01	-6.01
	N/A	>134	-8.51	6.01	-6.01

Figure 13: E-W Wind Pressures

E-W Wind Forces								
Level	Story Height	Trib. Below		Trib. Above		Story Force (k)	Story Shear (k)	Overturning Moment (ft-k)
		Height (ft)	Area (sf)	Height (ft)	Area (sf)			
1	0	N/A	N/A	6.67	905	0	364.32	0
2	13.33	6.67	905	6.67	905	56.02	364.32	746.74
3	26.67	6.67	905	6.67	905	59.39	308.31	1583.83
4	40	6.67	905	6.67	905	62.05	248.92	2481.87
5	53.33	6.67	905	6.67	905	64.17	186.87	3422.38
Roof	66.67	6.67	905	7.75	1052	122.70	122.70	8180.34
							Base Shear (k)	364.32
							Total Overturning Moment (ft-k)	16415.15

Figure 14: E-W Wind Forces

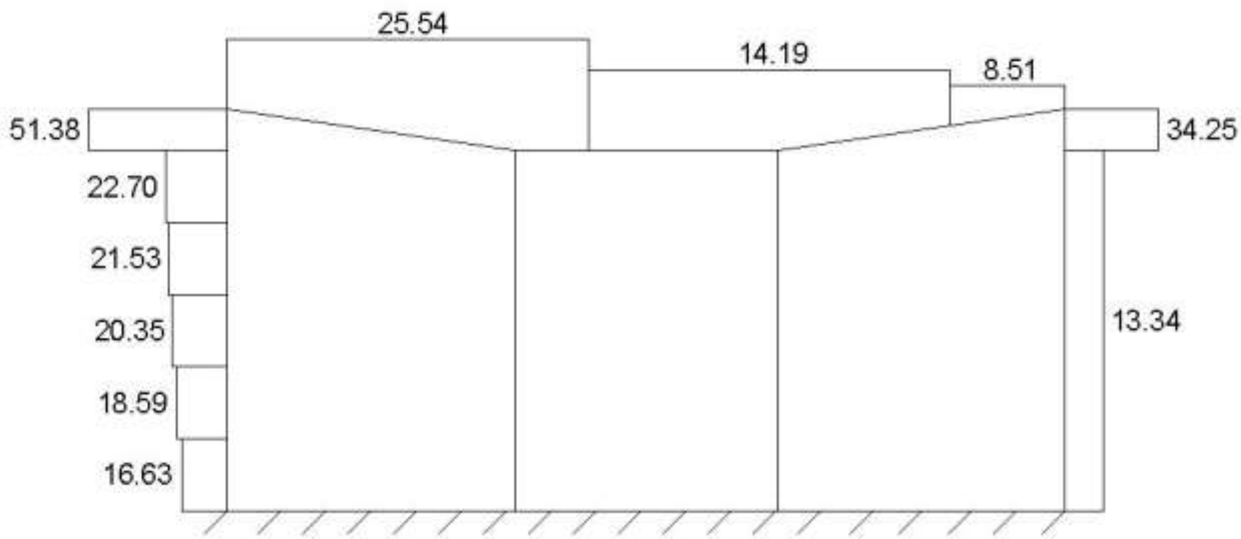


Figure 15: E-W Wind Pressure Diagram

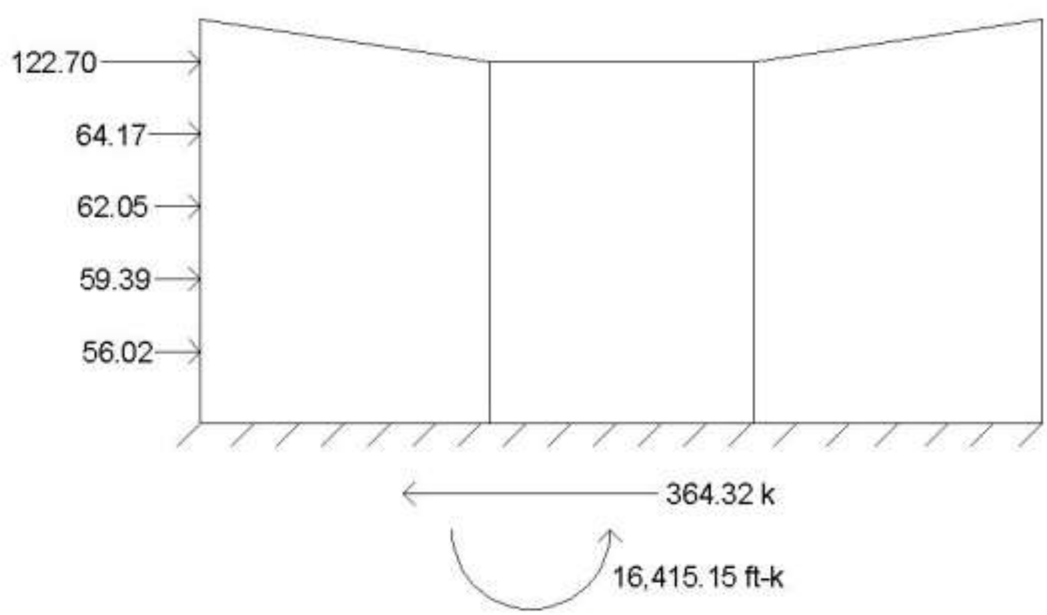


Figure 16: E-W Wind Force Diagram

Seismic Loads

Design seismic loads were calculated for the Office Building in accordance with ASCE 7-10, Chapters 11 and 12 (and in particular, section 12.8 – Equivalent Lateral Force Procedure). The design seismic base shear was calculated by hand and was resolved into story forces using Excel. Refer to Figures 17-18 and Appendix C for seismic loading summary and calculations.

Seismic Forces							
Level	Story Height, h_x (ft)	Story Weight, w_x (k)	$w_x h_x^k$	C_{vx}	Story Force (k)	Story Shear (k)	Overturning Moment (ft-k)
1	0	N/A	0	0	0	212.10	0
2	13.33	1341	26226.10	0.0722	15.31	212.10	204.12
3	26.67	1341	58143.77	0.1601	33.95	196.79	905.42
4	40	1341	92596.30	0.2549	54.07	162.84	2162.60
5	53.33	1341	128822.63	0.3546	75.22	108.77	4011.31
Roof	66.67	463	57471.58	0.1582	33.56	33.56	2237.21
						Base Shear (k)	212.10
						Total Overturning Moment (ft-k)	9520.66

Figure 17: Seismic Forces

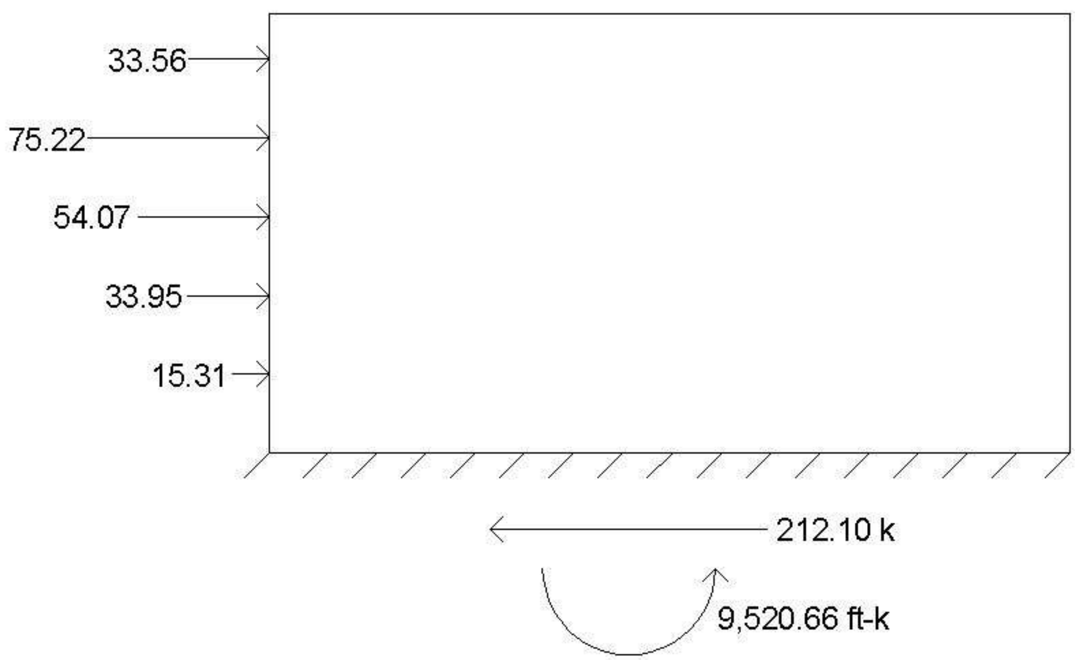
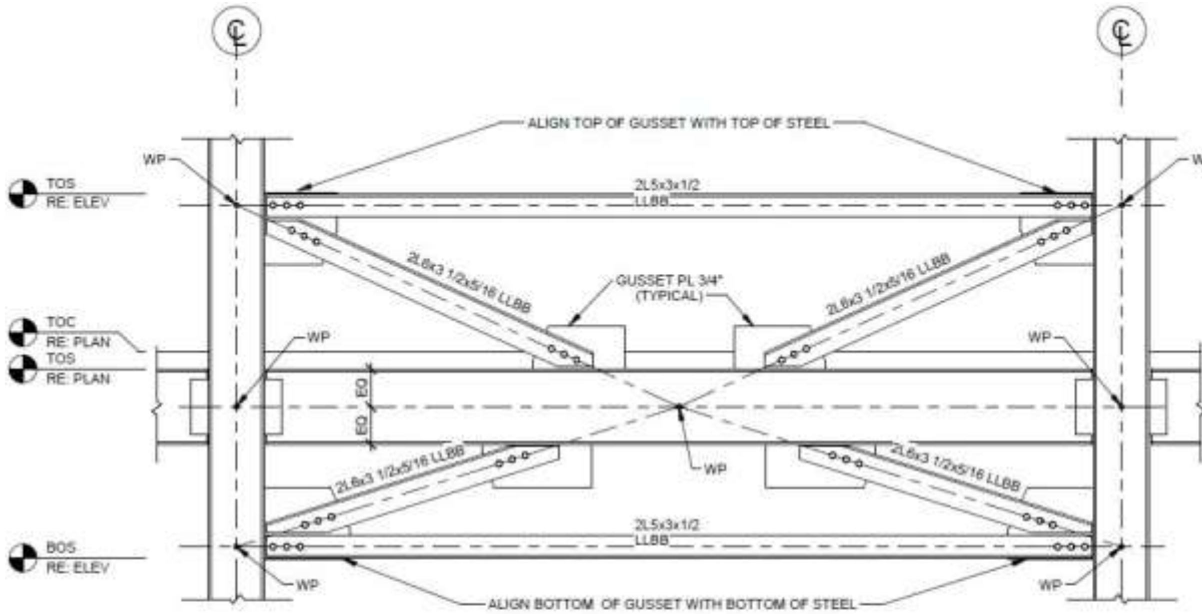


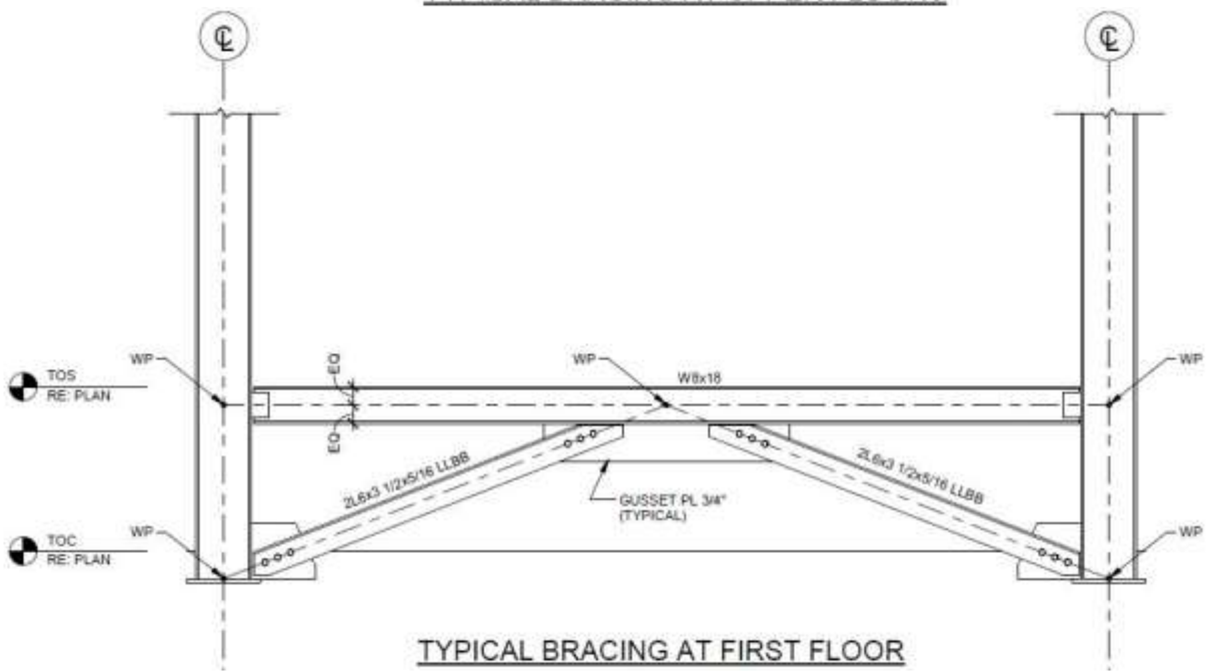
Figure 18: Seismic Force Diagram

Lateral Load Distribution

Lateral loads are resisted by 16 braced frames that make up the lateral system of the Office Building (8 in each the N-S and E-W directions). Double angles (2L6x3 1/2x5/16 LLBB) brace the frames above and below the windows at stories 1 through 4. At story 5, the frames are braced below the windows only. The double angle braces connect the columns and the perimeter floor beams at each floor. Figure 19 below shows typical bracing details and a typical braced frame.



TYPICAL BRACING AT UPPER FLOORS



TYPICAL BRACING AT FIRST FLOOR

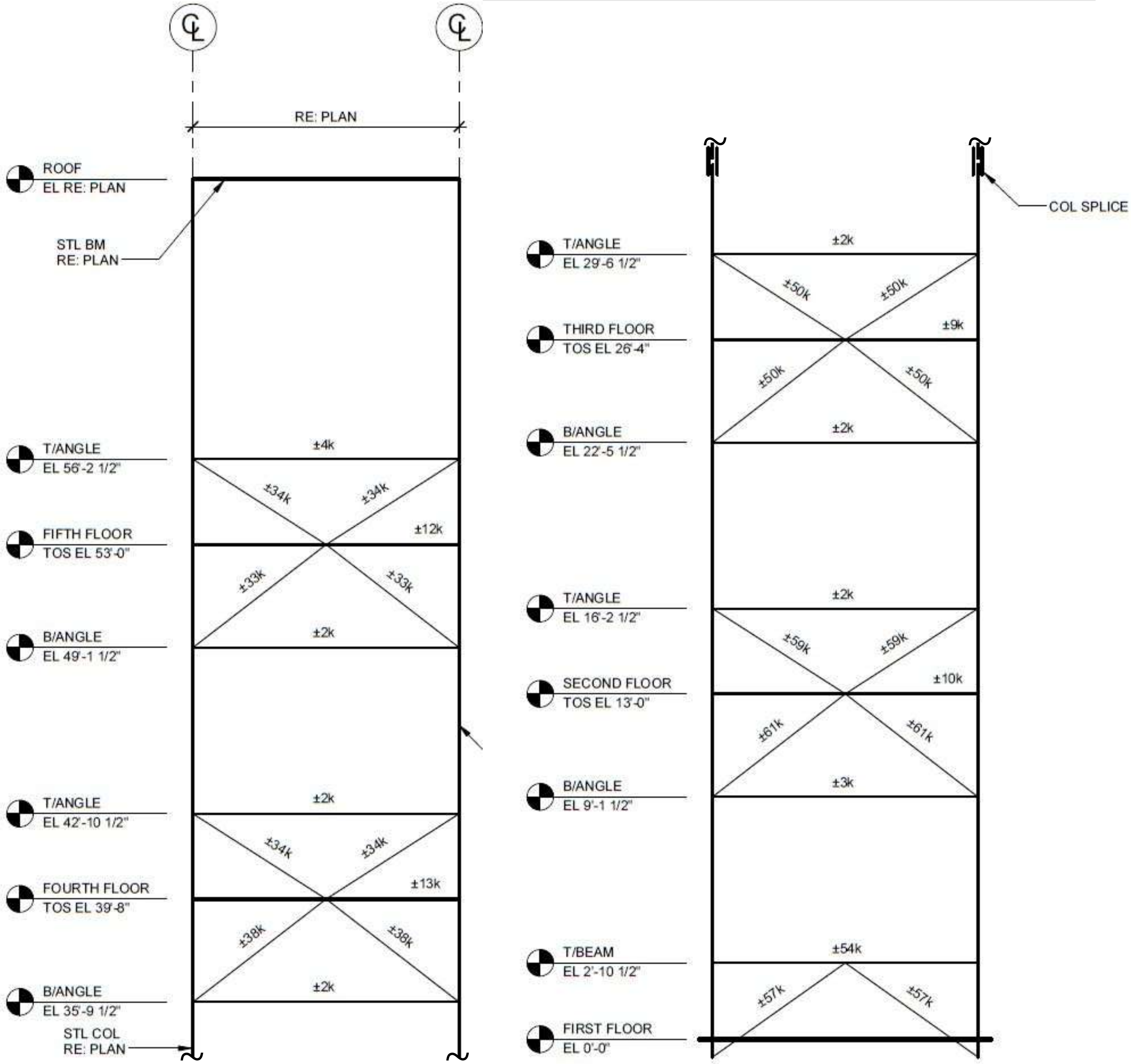


Figure 19: Typical Braced Frame and Bracing Details
(Image Credit: Larson Design Group)

As previously discussed in the lateral system overview, wind pressures on the exterior of the building are collected by the façade and the resultant forces are transferred into the floor/roof diaphragms. The diaphragms at each story act rigidly and transfer the story shear forces to the braced frames that act in the direction of the loading (the roof diaphragm has been treated as rigid for the purposes of this report, although it will likely behave flexibly). The braced frames resist the lateral loads based on the proportion of their relative stiffness. When lateral loads cause the frames to deflect or sway, the forces are transmitted into the braces as axial forces while resisting this sway based on the geometry of the bracing configuration. The story forces accumulate at each successive floor, down through the building until the total base shear is transferred into the ground via the foundation.

Seismic loads follow a similar path that wind loads do except that they are induced by the building's response to ground motion/acceleration and act through the center of mass rather than being collected by the façade. The total base shear is distributed to the diaphragms at each story based on the respective heights and weights of each level. Once distributed, the seismic forces are transmitted through the diaphragms into the braced frames, based on relative stiffness, and then into the individual braces. Similarly, the story forces accumulate and are eventually transferred down to the bearing soils through the foundation.

ETABS Lateral Analysis

Model

The floors and roof were modeled as rigid diaphragms in order to effectively constrain the displacements of all the points making up each floor. With rigid diaphragms, the lateral loads are distributed based on the relative stiffness of the resisting elements. To account for the dead weight of the Office Building, the weights of the floors and exterior walls were calculated and converted into masses. The masses were assigned to the diaphragms as additional area mass. Live and snow loads were also added to each diaphragm and were assigned to a point located at the center of mass for each level. Figure 20 below shows a typical floor diaphragm in plan.

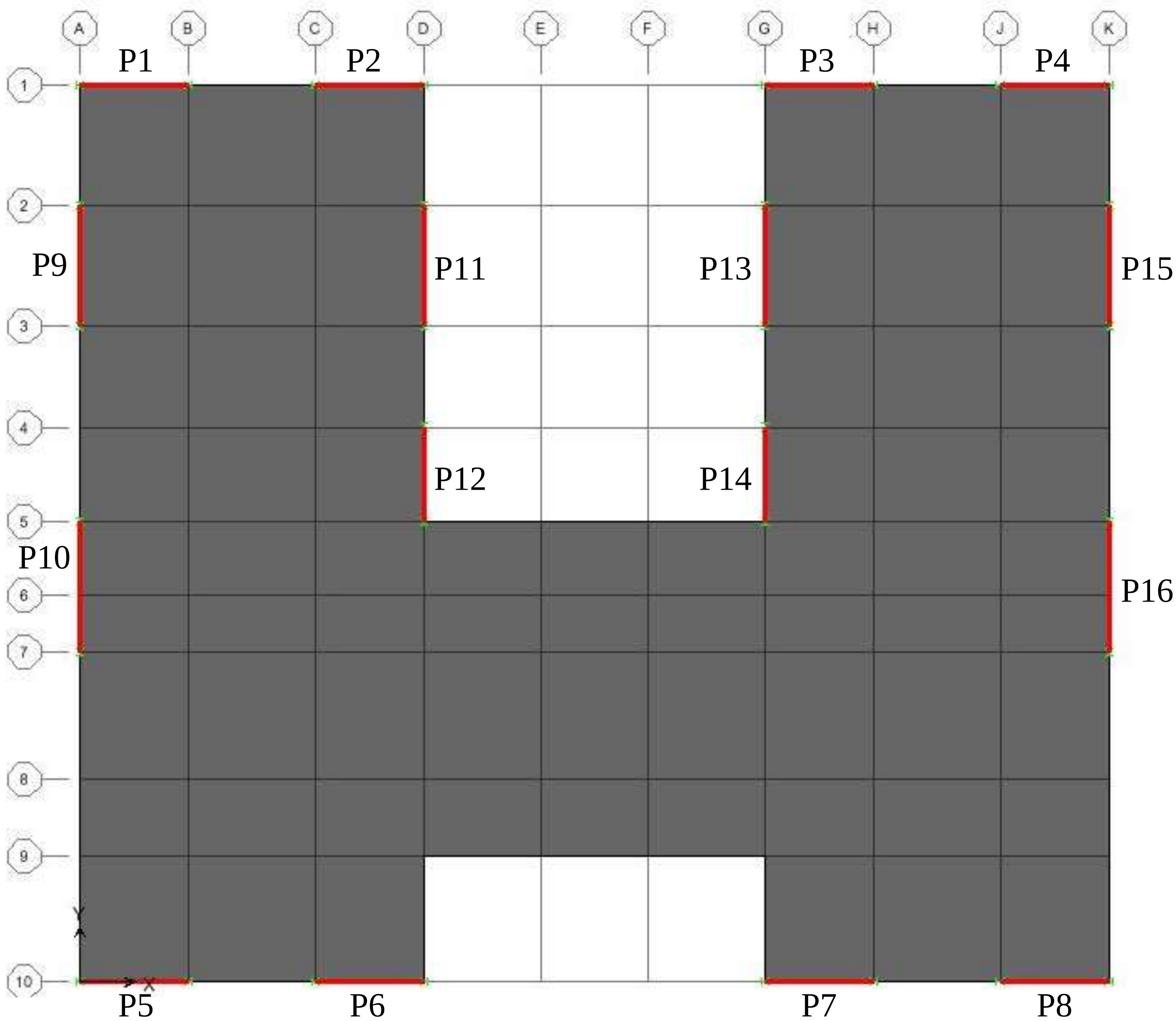


Figure 20: Typical Floor Diaphragm

The layout of the braced frames can be seen in Figure 20 above, with the frames labeled P1-P16. All elements that make up an individual frame were assigned a pier label so that the lateral forces being resisted by each frame could easily be determined from the ETABS output data. Figures 21-23 show elevations of the braced frames. In the figures below, columns are blue, beams are red and the double angle braces are gray.

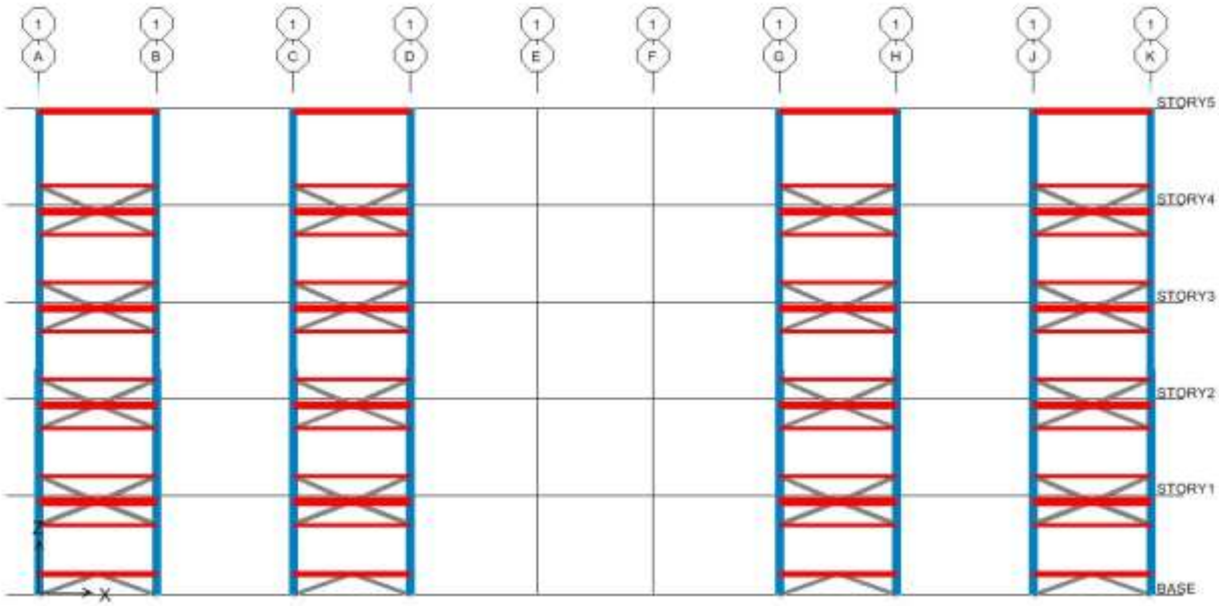


Figure 21: Braced Frames at Grids 1 and 10

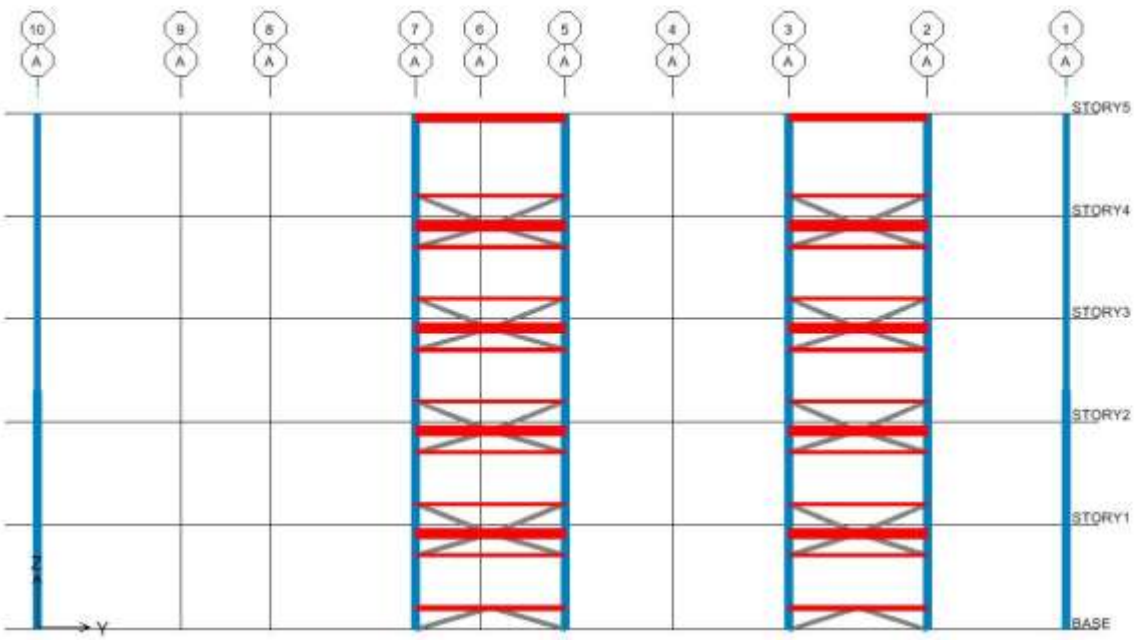


Figure 22: Braced Frames at Grids A and K

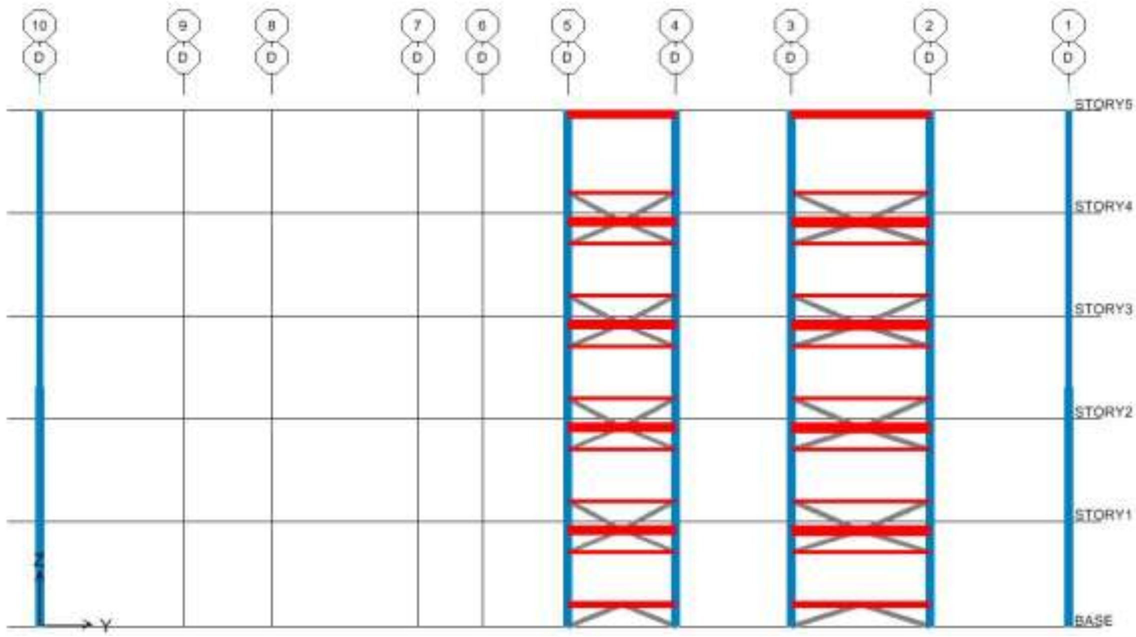


Figure 23: Braced Frames at Grids D and G

All columns are spliced at 4'-0" above Floor 3 (Story 2 in ETABS) and reduced down to lighter sections. The columns were modeled with pin supports at their bases, 4" below the finish floor elevation to account for the 4" thick slab-on-grade. They were also modeled so that the strong axis (flexural) would be parallel to the direction of lateral resistance of the overall frame.

The perimeter floor beams were located in the model with offsets from the defined story levels to account for the floor and joist seat depths. The elevations of the diaphragms coincide with the top of finish floor elevations at each floor level and with the top of the steel decking at the roof. The first floor (Base in ETABS) is the exception where the entire level grid was offset down 4" to represent the effective base of the frames. All beams and braces were modeled as pinned-pinned by providing moment releases at both ends of the members.

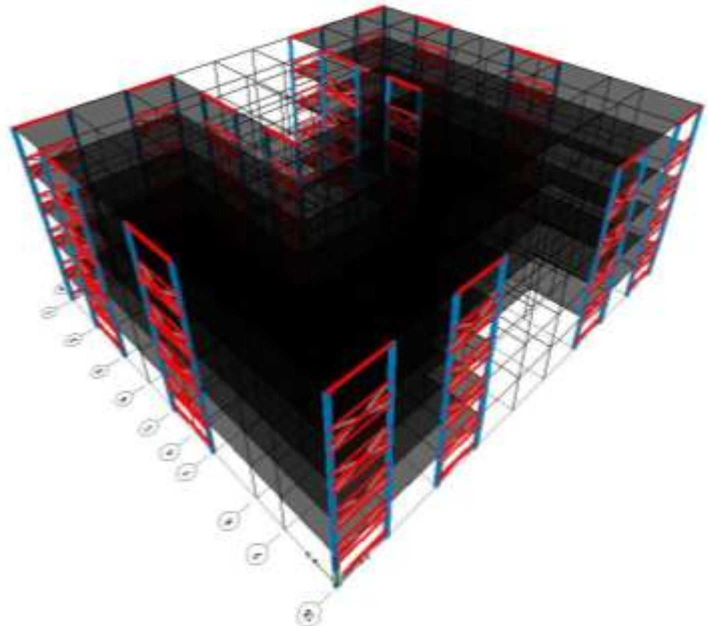


Figure 24: 3D View of ETABS Model

Center of Mass and Center of Rigidity

Because the building is symmetrical in the E-W direction, the center of mass in the X direction is half of the plan dimension in that direction, or 76.583’ from Grid A (and Grid K). The centers of mass in the N-S (Y) direction of the floors and roof were calculated by hand to be 63.658’ from Grid 10 and 64.213’ from Grid 10, respectively. From the ETABS building output, the Y direction center of mass for the floors was 63.727’ and 63.699’ for the roof (measured from Grid 10). The values came out to be very close, justifying use of the ETABS automatic feature for computing the torsional effects from the seismic loads applied at the center of mass within the model instead of explicitly specifying the moments acting on the diaphragms in the load case definitions. Refer to Appendices D-E for the mass definition and center of mass hand calculations.

There were no hand calculations necessary to determine the center of rigidity for the Office Building. The braced frames are laid out symmetrically in each direction, theoretically placing the center of rigidity at the center point of the building’s plan dimensions. This places the center of rigidity at 76.583’ from Grid A (and Grid K) in the X direction and 66.792’ from Grid 10 (and Grid 1) in the Y direction. The output from ETABS has the center of rigidity at the center point in the X direction and at an average of 67.201’ from Grid 10 for all levels. This small shift upwards as calculated by ETABS is likely due to the fact that the diaphragms at each level are taken into account in determining the overall stiffness/rigidity of the floors as opposed to the braced frames alone.

Due to the central location of the centers of rigidity and the symmetry of the braced frames in both directions, positive and negative eccentricities for wind and seismic did not both have to be considered separately. This allowed for the total number of load cases/combinations being considered to be cut down significantly. The forces were applied with positive forces/eccentricities only. Then, the worst case loading for the frames on either side of the building was found and taken as the maximum force in the similar frame that was on the opposite side. Since the torsional moments and direct forces in the frames are additive on one side and they subtract on the opposite side, the maximum force obtained in any frame was also taken as the maximum in the equal and opposite frame on the other side of the building's line of symmetry.

Loads and Load Cases

Design wind loads for the Office Building were previously calculated for Technical Report 1 using the ASCE 7-10 MWFRS Directional Procedure. The pressures and resultant forces can be found in the "Lateral Loads" section of this report. The four directional load cases from ASCE 7-10 were used to consider the potential effects of the basic wind loads. Since the center of rigidity was considered to be at the exact center of the building's plan dimensions (in both X and Y directions), the wind load acts at the center of pressure without any inherent or additional eccentricity for Case 1 and Case 3. Case 3 was therefore able to be eliminated as a controlling condition by inspection since the resultant loads are reduced by 25%. Because there is no torsional moment produced in Case 1 or Case 3, the fact that the X and Y direction loads act simultaneously in the latter case results only in a smaller direct load. The effects of Case 1 (X and Y), Case 2 (X and Y) and Case 4 were all analyzed in ETABS through five different wind loading scenarios for each load combination involving wind. The wind load values for direct and torsional effects for each load case are shown in the tables of Figure 25.

ASCE 7-10 Wind Load Case 1						
Level	F_x (k)	F_y (k)	e_y (ft)	e_x (ft)	M_x (ft-k)	M_y (ft-k)
1	0	0	0	0	0	0
2	56.02	65.83	0	0	0	0
3	59.39	69.68	0	0	0	0
4	62.05	72.72	0	0	0	0
5	64.17	75.15	0	0	0	0
Roof	122.70	86.99	0	0	0	0

ASCE 7-10 Wind Load Case 2						
Level	F_x (k)	F_y (k)	e_y (ft)	e_x (ft)	M_x (ft-k)	M_y (ft-k)
1	0	0	20.04	22.98	0	0
2	42.01	49.37	20.04	22.98	841.87	1134.26
3	44.54	52.26	20.04	22.98	892.46	1200.61
4	46.54	54.54	20.04	22.98	932.45	1253.04
5	48.13	56.36	20.04	22.98	964.41	1294.95
Roof	92.02	65.24	20.04	22.98	1843.94	1498.95

ASCE 7-10 Wind Load Case 3							
Level	F_x (k)	F_y (k)	e_y (ft)	e_x (ft)	M_x (ft-k)	M_y (ft-k)	M_{total} (ft-k)
1	0	0	0	0	0	0	0
2	42.01	49.37	0	0	0	0	0
3	44.54	52.26	0	0	0	0	0
4	46.54	54.54	0	0	0	0	0
5	48.13	56.36	0	0	0	0	0
Roof	92.02	65.24	0	0	0	0	0

ASCE 7-10 Wind Load Case 4							
Level	F_x (k)	F_y (k)	e_y (ft)	e_x (ft)	M_x (ft-k)	M_y (ft-k)	M_{total} (ft-k)
1	0	0	20.04	22.98	0	0	0
2	31.54	37.06	20.04	22.98	631.96	851.45	1483.42
3	33.43	39.23	20.04	22.98	669.94	901.26	1571.20
4	34.93	40.94	20.04	22.98	699.96	940.62	1640.57
5	36.13	42.31	20.04	22.98	723.95	972.08	1696.03
Roof	69.08	48.98	20.04	22.98	1384.18	1125.21	2509.39

Figure 25: ASCE 7-10 Wind Case Loads

Design seismic loads were also previously calculated for Technical Report 1 using the ASCE 7-10 Equivalent Lateral Force Procedure and can be found in the “Lateral Loads” section of this report. The loads induced by seismic activity act through the center of mass at each story. Since the center of mass in the N-S direction does not coincide with the center of rigidity, there is an inherent torsional moment caused by the seismic forces that act in the E-W direction. In the E-W direction, the building plan is symmetrical and the center of mass and center of rigidity are aligned. Thus, there is no inherent torsion caused by the seismic forces that act N-S direction. In both directions, an accidental torsional moment was also applied to the model to account for the assumed displacement of the center of mass by a distance of 5% of the plan dimension perpendicular to the direction of loading, as outlined in ASCE 7-10. For Seismic Design Category B, amplification of the accidental torsional moment is not required and the redundancy factor, ρ, is permitted to equal 1.0 so that the horizontal seismic load effects for the Office Building are not amplified. The calculated seismic load effects for the Office Building are outlined in Figure 26.

N-S Seismic Forces								
Level	Story Force (k)	Story Shear (k)	e (ft)	e_{acc} (ft)	M_t (ft-k)	M_{ta} (ft-k)	M_{total} (ft-k)	Story M (ft-k)
1	0	207.30	N/A	N/A	0	0	0	1587.57
2	14.96	207.30	0	7.658	0	114.55	114.55	1587.57
3	33.16	192.34	0	7.658	0	253.96	253.96	1473.02
4	52.81	159.18	0	7.658	0	404.44	404.44	1219.07
5	73.47	106.37	0	7.658	0	562.66	562.66	814.63
Roof	32.90	32.90	0	7.658	0	251.97	251.97	251.97

E-W Seismic Forces								
Level	Story Force (k)	Story Shear (k)	e (ft)	e_{acc} (ft)	M_t (ft-k)	M_{ta} (ft-k)	M_{total} (ft-k)	Story M (ft-k)
1	0	207.30	N/A	N/A	0	0	0	2015.94
2	14.96	207.30	3.134	6.679	46.87	99.90	146.77	2015.94
3	33.16	192.34	3.134	6.679	103.92	221.49	325.40	1869.17
4	52.81	159.18	3.134	6.679	165.49	352.73	518.22	1543.76
5	73.47	106.37	3.134	6.679	230.23	490.72	720.96	1025.55
Roof	32.90	32.90	2.579	6.679	84.84	219.75	304.59	304.59

Figure 26: Seismic Load Effects

The torsional effects from the seismic loads outlined above were not entered into ETABS directly, as they were for the wind loading. Instead, only the story forces were entered and were applied at the center of mass for each story. The effects of the inherent eccentricity to the center of rigidity as well as the accidental torsional moments were taken into account within the ETABS model.

The following ASCE 7-10 strength design load combinations were considered in the analysis:

- 1.2D + 1.6S + 0.5W
- 1.2D + 1.0W + 1.0L + 0.5S
- 1.2D + 1.0E + 1.0L + 0.2S
- 0.9D + 1.0W
- 0.9D + 1.0E

Based on these load combinations, 19 total load cases were developed for the ETABS model to consider all applicable lateral loading conditions. The following cases were input into the model:

- COMB1: 1.2D + 1.6S + 0.5WINDC1X
- COMB2: 1.2D + 1.6S + 0.5WINDC1Y
- COMB3: 1.2D + 1.6S + 0.5WINDC2X
- COMB4: 1.2D + 1.6S + 0.5WINDC2Y
- COMB5: 1.2D + 1.6S + 0.5WINDC4
- COMB6: 1.2D + 1.0WINDC1X + 1.0L + 0.5S
- COMB7: 1.2D + 1.0WINDC1Y + 1.0L + 0.5S
- COMB8: 1.2D + 1.0WINDC2X + 1.0L + 0.5S
- COMB9: 1.2D + 1.0WINDC2Y + 1.0L + 0.5S
- COMB10: 1.2D + 1.0WINDC4 + 1.0L + 0.5S
- COMB11: 1.2D + 1.0QUAKEX + 1.0L + 0.2S
- COMB12: 1.2D + 1.0QUAKEY + 1.0L + 0.2S
- COMB13: 0.9D + 1.0WINDC1X
- COMB14: 0.9D + 1.0WINDC1Y
- COMB15: 0.9D + 1.0WINDC2X
- COMB16: 0.9D + 1.0WINDC2Y
- COMB17: 0.9D + 1.0WINDC4

- COMB18: 0.9D + 1.0QUAKEX
- COMB19: 0.9D + 1.0QUAKEY

=>where C1, C2 and C4 indicate wind load Case 1, 2 and 4, respectively and X and Y indicate the direction of loading.

ETABS Output

The critical forces and displacements from the ETABS analysis output are summarized in the tables that follow. Figure 27 shows the maximum shear forces that occur in each of the 16 frames and specifies the specific load combinations that cause those forces.

Frame Forces		
Frame	Max Shear (k)	Load Combo
P1	46.65	COMB6 & COMB13
P2	46.65	COMB6 & COMB13
P3	46.65	COMB6 & COMB13
P4	46.65	COMB6 & COMB13
P5	46.65	COMB6 & COMB13
P6	46.65	COMB6 & COMB13
P7	46.65	COMB6 & COMB13
P8	46.65	COMB6 & COMB13
P9	46.46	COMB7 & COMB14
P10	44.47	COMB7 & COMB14
P11	50.29	COMB7 & COMB14
P12	52.56	COMB7 & COMB14
P13	50.29	COMB7 & COMB14
P14	52.56	COMB7 & COMB14
P15	46.46	COMB7 & COMB14
P16	44.47	COMB7 & COMB14

Figure 27: Frame Forces

In Figure 28, the maximum tensile and compressive forces that occur in the braces at each frame are reported. The braces that are loaded with these maximum forces are those at the first story that extend from the columns at the top of the first floor windows to the center of the second story perimeter floor beams.

Maximum Brace Forces				
Frame	Tension (k)	Load Combo	Compression (k)	Load Combo
P1	51.59	COMB13	52.57	COMB6
P2	51.59	COMB13	52.57	COMB6
P3	51.59	COMB13	52.57	COMB6
P4	51.59	COMB13	52.57	COMB6
P5	51.59	COMB13	52.57	COMB6
P6	51.59	COMB13	52.57	COMB6
P7	51.59	COMB13	52.57	COMB6
P8	51.59	COMB13	52.57	COMB6
P9	48.20	COMB14	49.32	COMB7
P10	46.65	COMB14	47.85	COMB7
P11	55.86	COMB14	56.91	COMB7
P12	53.10	COMB14	54.06	COMB7
P13	55.86	COMB14	56.91	COMB7
P14	53.10	COMB14	54.06	COMB7
P15	48.20	COMB14	49.32	COMB7
P16	46.65	COMB14	47.85	COMB7

Figure 28: Maximum Brace Forces

The maximum drifts and displacements due to wind loading are shown below in Figure 29. The allowable drift due to wind of 0.400” per story comes from H/400 (13.33’*12/400=0.400”). The total allowable lateral displacement at the top of the building is 2”. While the total displacements at the roof are considered to be ok in both directions, floor 2 (story 1) just barely exceeds the story drift limit in the Y direction and the roof (story 5) exceeds the limit significantly in both directions. Concerning the story drift at the roof, the fact that there are no braces above the windows on the fifth floor is likely why the story drift is so much greater at that level.

Wind Drifts and Displacements						
Level	Elevation (ft)	E-W (X) Displ (in)	E-W (X) Story Drift (in)	N-S (Y) Displ (in)	N-S (Y) Story Drift (in)	Allow Drift (in)
1	0	0	0	0	0	0
2	13.33	0.351	0.351	0.406	0.406	0.400
3	26.67	0.614	0.263	0.704	0.298	0.400
4	40.00	0.885	0.271	0.988	0.284	0.400
5	53.33	1.157	0.272	1.248	0.26	0.400
Roof	66.67	1.898	0.741	1.837	0.589	0.400

Figure 29: Wind Drifts and Displacements

The maximum drifts and displacements due to the seismic loads on the Office Building are summarized in Figure 30. The allowable story drift is 3.200". This limit comes from ASCE 7-10 Table 12.12-1, where Δ_a=0.020h_{sx} for Risk Category II. The total allowable lateral displacement at the top of the building is 16".

Seismic Drifts and Displacements						
Level	Elevation (ft)	E-W (X) Displ (in)	E-W (X) Story Drift (in)	N-S (Y) Displ (in)	N-S (Y) Story Drift (in)	Allow Drift (in)
1	0	0	0	0	0	0
2	13.33	0.2	0.2	0.229	0.229	3.200
3	26.67	0.36	0.16	0.416	0.187	3.200
4	40.00	0.529	0.169	0.604	0.188	3.200
5	53.33	0.678	0.149	0.768	0.164	3.200
Roof	66.67	0.901	0.223	1.012	0.244	3.200

Figure 30: Seismic Drifts and Displacements

Figure 31 reports the first three modes (periods of vibration) of the Office Building. Modes 1 and 2 both have greater periods than the calculated fundamental period of 0.796 seconds used to determine the seismic base shear and seismic lateral forces in Technical Report 1. Since a longer period induces smaller seismic forces in the structure, the values previously calculated using 0.796 seconds may be considered conservative.

Office Building Modes		
Mode	Period (s)	Direction
1	1.2289	N-S (Y)
2	1.1552	E-W (X)
3	0.8341	Rotation (Z)

Figure 31: Office Building Modes

Overturning Check

The worst lateral load case concerning overturning is from the N-S wind loading, Case 1. The total overturning moment caused by the resultant wind forces is 15,451.95 ft-k. While the overturning moment caused by the E-W wind loading for Case 1 is actually a little greater at 16,415.15 ft-k, there is also a greater resisting moment from the dead load acting with a greater moment arm. The resisting moment due to the dead weight of the building acting through its center of mass is calculated in Figure 32.

Resisting Moment				
Bldg Dead Wt (k)	0.9D (k)	Dist to CM (ft)	Resist Mom (ft-k)	(2/3)*Resist Mom (ft-k)
5694	5124.60	63.658	326221.79	217481.19

Figure 32: Resisting Moment

The overturning moment at the base of the structure should not exceed two thirds the resisting moment due to the building dead load. The controlling load combination for overturning is 0.9D + 1.0W, or COMB14: 0.9D + 1.0WINDC1Y more specifically. The resisting moment is much greater than overturning, as the overturning moment is only about 7% of the resisting value.

Brace Check

- Check the strength of the upper braces at the first floor (just below floor 2) at frames P11 and P13 where the maximum brace forces occur.

-All braces are 2L6x3 1/2x5/16 LLBB double angles connected to 3/4" thick gusset plates.

-Tension:

$\phi R_n = \phi F_y A_g = 0.9(36)(5.78) = 187.3$ k > 55.86 k => OK

-Compression:

3/4" gusset plate => 3/4" between LL of angles

$r_y = 1.50 > 1.37$ => steel manual value may be used conservatively

ϕP_n @ KL = 10' (> 9.42') = 91.0 k > 56.91 k => OK

Conclusion

Technical Report 3 has expanded on the overview of the lateral system and loading of the Office Building which was covered in Technical Report 1. A thorough lateral analysis was performed with fully developed loading conditions and an ETABS model of the lateral force resisting system. The floors and roof were modeled as rigid diaphragms with additional mass and load assignments to account for the dead weight and superimposed loads on the structure. Hand calculations for the centers of mass of the floors and roof were compared to the ETABS output values. The assumed theoretical centers of rigidity were also compared to the ETABS values. For both sets of values, the results were similar and the methods effectively reinforced one another.

Once the modeling was completed, the loads and load cases had to be developed for input into the model. These were based on the calculated individual load cases and the applicable ASCE 7-10 strength design load combinations. The loading included the direct and torsional (inherent and accidental) effects on the structure. After the proper loading was assigned, the model was analyzed. From the ETABS output, the maximum individual frame and brace forces were determined as well as the load combinations that caused those critical values. The drifts and displacements due to wind and seismic forces were compared with the industry and code allowable values for story drift and overall lateral displacement. For the wind loading, several individual story drifts exceeded allowable values, but the overall displacement at the top of the building was within the limit. The worst case overturning moment due to N-S wind loading was checked against the resisting moment due to the dead weight of the building. The overturning moment was only about 7% of the available resisting moment. Finally, a strength spot check of the upper braces at the first floor for frames P11 and P13 was performed. The double angle brace was sufficient to carry the applied tensile and compressive axial loads.

Appendix A

Seth Moyer | Tech 1 - Gravity Loads | 1/2

- Dead Loads:

Roof: 1½" Type B 20 ga. wide rib roof deck = 2.14 psf (Vulcraft Deck Cat.)

24K4 @ 6' OC = 8.4 plf / 6' = 1.4 psf (SJI)

4" Rigid Insulation = 6 psf

EPDM = 0.7 psf

MEP/Ceiling = 10 psf

Total = 2.14 + 1.4 + 6 + 0.7 + 10 = 20.24 => 20 psf

Floor: 2½" thk. conc. slab on 20 ga. 1½" composite deck = 39 psf (Vulcraft Deck Cat., 1.5VL20)

16K4 @ 3' OC = 7.0 plf / 3' = 2.33 psf (SJI)

Bms/Girders = 7 psf

MEP/Ceiling = 10 psf

Total = 39 + 2.33 + 7 + 10 = 58.33 => 60 psf

- Live Loads: (ASCE 7-10, Table 4-1)

Roof = 20 psf

Floor: Corridors above first floor = 80 psf

Partitions = 20 psf

Total = 80 + 20 = 100 psf

- Snow Loads:

- Ground Snow Load: p_g = 30 psf (ASCE 7-10, Figure 7-1)

- Exposure Factor: C_e = 1.0 (Partially Exposed) (Table 7-2)

- Thermal Factor: C_t = 1.0 (Table 7-3)

- Importance Factor: I_s = 1.0 (Table 1.5-2)

- Flat Roof Snow Load: p_f = 0.7(1.0)(1.0)(1.0)(30) = 21 psf (Eq. 7.3-1)

Seth Moyer | Tech 1 – Gravity Loads | 2/2

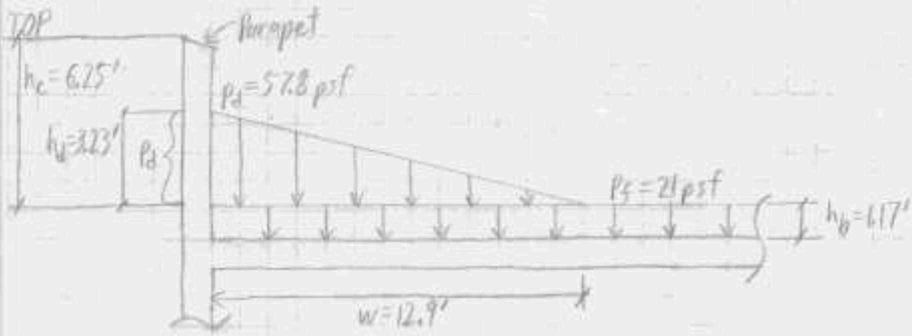
– Drift Loads: (§ 7.8)

Max Drift Case: $l_u = 155.33'$

$h_d = 0.75(0.43\sqrt[3]{155.33}\sqrt[4]{30+10} - 1.5) = 3.23'$ (Figure 7-9)

$p_s = p_f = 21 \Rightarrow \gamma = 0.13(30) + 14 = 17.9$ (Eq 7.7-1) $\Rightarrow h_b = 21/17.9 = 1.17'$

$h_c = 7.42' - 1.17' = 6.25' \Rightarrow w = 4(3.23) = 12.9' \Rightarrow p_d = h_d \gamma = 3.23'(17.9) = 57.8$ psf



Appendix B

Seth Moyer | Tech 1 - Wind Loads | 1/2

Wind Loads: (ASCE 7-10 Chapter 27: Wind Loads on Buildings - MWFRS: Directional Procedure)

- Risk Category: II (Table 1.5-1)
- Basic Wind Speed: V = 115 mph (Figure 26.5-1A)
- Wind Directionality Factor: $K_d = 0.85$ (Table 26.6-1)
- Exposure Category: C (§26.7.2 & §26.7.3)
- Topographic Factor: $K_{zt} = 1.0$ (§26.8.2)
- Gust Effect Factor:

  Check Approx. Natural Frequency Limitations (§26.9.2.1)

  $h = 67 \text{ ft} < 300 \text{ ft} \therefore$ OK

  $L_{eff} = 135.75 \text{ ft} \Rightarrow h = 67 \text{ ft} < 4(35.75) \therefore$ OK

  $n_a = 75/h = 75/67 = 1.12$ (Eq 26.9-4)

  $n_a = 1.12 > 1.0 \therefore$ Rigid $\Rightarrow G = 0.85$ (§26.9.1)

- Enclosure Classification: Enclosed ⇒ Internal Pressure Coeff.: $GC_{pi} = \pm 0.18$ (Table 26.11-1)
- Velocity Pressure Exposure Coefficients: (Table 27.3-1)

K_h, K_z	z (ft)
0.85	(0-15)
0.90	(20)
0.94	(25)
0.98	(30)
1.04	(40)
1.09	(50)
1.13	(60)
1.17	(70)
1.21	(80)

Flr.	z (ft)	K_h, K_z
1	0	0.85
2	13.33	0.85
3	26.67	0.95
4	40	1.04
5	53.33	1.10
R	66.67	1.16
TOP	74.42	1.19

- Velocity Pressure: $q_z = 0.00256 K_z K_{zt} K_d V^2$ (Eq 27.3-1)

Flr.	q_z (psf)
1	24.46
2	24.46
3	27.34
4	29.93
5	31.66
R	33.38
TOP	34.25

Seth Moyer | Tech 1 - Wind Loads | 2/2

- External Pressure Coefficients: (Figure 27.4-1)

Windward Wall: C_p 0.8

Leeward Wall: N-S => 135.75/155.33 = 0.87 => C_p = -0.5

E-W => 155.33/135.75 = 1.14 => C_p = -0.47 (from interp)

Side Wall: C_p = -0.7

Roof: N-S => h/L = 67/135.75 = 0.49 < 0.5 E-W => 67/155.33 = 0.43 < 0.5

Horiz dist (ft)	C_p
0-33.5	-0.9, -0.18
33.5-67	-0.9, -0.18
67-134	-0.5, -0.18
>134	-0.3, -0.18

- Design Wind Pressures: $p = qGC_p - q_i(GC_{pi})$ (Eq 27.4-1)

Windward Wall: Flr

1 => p = 24.46(0.85)(0.8) - 33.38(±0.18) = 16.63 ± 6.01
2 => p = 16.63 ± 6.01
3 => p = 27.34(0.85)(0.8) ± 6.01 = 18.59 ± 6.01
4 => p = 29.93(0.85)(0.8) ± 6.01 = 20.35 ± 6.01
5 => p = 31.66(0.85)(0.8) ± 6.01 = 21.53 ± 6.01
R => p = 33.38(0.85)(0.8) ± 6.01 = 22.70 ± 6.01
TOP => $p_p = q_p(GC_{pn})$ (Eq 27.4-4) => p_p = 34.25(1.5) = 51.38

Leeward Wall: N-S => 33.38(0.85)(-0.5) ± 6.01 = -14.19 ± 6.01
E-W => 33.38(0.85)(-0.47) ± 6.01 = -13.34 ± 6.01
TOP => p_p = 34.25(-1.0) = -34.25

Side Wall: p = 33.38(0.85)(-0.7) ± 6.01 = -19.86 ± 6.01

Roof: Horiz Dist (ft)
0-67 => p = 33.38(0.85)(-0.9) ± 6.01 = -25.54 ± 6.01
67-134 => p = 33.38(0.85)(-0.5) ± 6.01 = -14.19 ± 6.01
>134 => p = 33.38(0.85)(-0.3) ± 6.01 = -8.51 ± 6.01

Appendix C

Seth Moyer | Tech 1 – Seismic Loads | 1/2

- Acceleration Parameters: Bldg. Site @ 41°59'07"N −76°33'42"W
  - Site Class: D (See Geotech Report) => Use Equivalent Lateral Force Proced. (§12.8) – permitted by Table 12.6-1
  - Risk Category II (Table 1.5-1)
  - $S_s = 0.121g$ $S_1 = 0.054g$ (http://earthquake.usgs.gov/hazards/designmaps/)
- Spectral Response Acceleration Parameters
  - $S_{MS} = F_a S_s = 1.6(0.121) = 0.194g$ (Eq 11.4-1)
  - $S_{M1} = F_v S_1 = 2.4(0.054) = 0.130g$ (Eq 11.4-2)
- Design Spectral Acceleration Parameters
  - $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.194) = 0.129g$ (Eq 11.4-3)
  - $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.130) = 0.087g$ (Eq 11.4-4)
- Importance Factor: $I_e = 1.0$ (Table 1.5-2)
- Seismic Design Category: B (Table 11.6-2)
- Response Modification Coeff: $R = 3$ for Steel Systems not Specifically Detailed for Seismic Resistance (Table 12.2-1)
- $T_L = 6$
- Approx. Fund. Period: $T_a = C_t h_n^x = 0.02(67)^{0.75} = 0.468$ (Eq 12.8-7)
- $T = C_u T_a = 1.7(0.468) = 0.796$ (§12.8.2)

$$C_s = \begin{cases} S_{DS}/(R/I_e) = 0.129/(3/1) = 0.043 \\ S_{D1}/(T \cdot R/I_e) = 0.087/(0.796 \cdot 3/1) = 0.0364 \geq 0.044(0.129)(1.0) \geq 0.01 = 0.00568 \therefore OK \\ \text{min } S_{D1} T_L/(T^2 R/I_e) = 0.087(6)/(0.796^2 \cdot 3/1) = 0.275 \end{cases}$$

$\Rightarrow C_s = 0.0364$

Seth Moyer | Tech 1 - Seismic Loads | 2/2

-Effective Seismic Wt

Flr 2-5: (60 psf + 10 psf)(17,015 sf) + 15 psf (13.33 ft)(750 ft) = $1{,}341^k$

Roof: 20 psf (17,015 sf) + 15 psf (6.67 ft + 2.75 ft)(135.25)(2) + 15 psf (6.67 ft + 3.1 ft)(53.33 ft)(4) +

15 psf (6.67 ft + 1.54 ft)(48.67×2 + 18.67×2 + 65×2) = 463^k

$W = 4(1{,}341) + 463 = 5{,}827^k$

-Seismic Base Shear: $V = C_s W = 0.0364(5{,}827) = 212.1^k$ (Eq. 12.8-1)

Appendix D

Seth Moyer | Tech3 - Bldg Weight & Mass | 1/1

-Effective Seismic Wt.

Flrs: 60+10 = 70psf

Roof: 20psf

Ext. Walls: 15psf

Wts:

Floors 2-5: [2(7021)+2537] 70psf = 1,161^K

Ext Walls 2-5: (13.33')(745') 15psf = 149^K

Roof: [2(7021)+2537] 20psf = 332^K

Ext Walls @ Roof: [[(54+9/12)(6.67+2.75)(2)+(52+3/12)(6.67+3.1)(4)+[2(65)+2(18+8/12)+249+10/12][6.67+1.54]] 15psf =>

= 122^K

-Mass

Flrs 2-5: (1161+149)/16,579 = 0.079 ksf

Mass/Area: $0.079/32.2/12^3 = 1.42\times10^{-6}$

Roof: (332+122)/16,579 = 0.0274 ksf

Mass/Area: $0.0274/32.2/12^3 = 4.92\times10^{-7}$

Appendix E

Seth Moyer | Tech 3 - Center of Mass | 1/1

- CM(y):

Floors 2-5 + Roof: $\dfrac{2(7021)[(33+7/12)/2]+2537[(49+11/12)/2+(18+9/12)]}{2(7021)+2537} = 63.247'$ from Grid 10

Ext Walls (2-5): $[2(134+7/12)(33+7/12)/2+2(65)(65/2+50+11/12+18+2/12)+2(18+9/12)[(18+9/12)/2-0.5]+$
$2(52+3/12)(134+7/12-0.5)+2(52+3/12)(-0.5)+(49+10/12)(50+11/12+18+2/12)+(49+10/12)(18+2/12)]/$
$[2(134+7/12)+4(52+3/12)+2(65)+2(18+9/12)+2(49+10/12)] = 66.859'$ from Grid 10

Ext Walls (Roof): $[2(134+7/12)(6.67+2.75)(33+7/12)/2+2(65)(6.67+1.54)(65/2+50+11/12+18+2/12)+$
$2(18+9/12)(6.67+1.54)[(18+9/12)/2-0.5]+2(52+3/12)(6.67+3.1)(134+7/12-0.5)+$
$2(52+3/12)(6.67+3.1)(-0.5)+(49+10/12)(6.67+1.54)(50+11/12+18+2/12)+(49+10/12)(6.67+1.54)(18+2/12)]/$
$[2(134+7/12)(6.67+2.75)+2(65)(6.67+1.54)+2(18+9/12)(6.67+1.54)+4(52+3/12)(6.67+3.1)+$
$2(49+10/12)(6.67+1.54)] = 66.842'$ from Grid 10

- CM - Flr 2-5: $\dfrac{1161(63.247)+149(66.859)}{1161+149} = 63.658'$ from Grid 10

- CM - Roof: $\dfrac{332(63.247)+122(66.842)}{332+122} = 64.213'$ from Grid 10