Technical Report 1

Existing Conditions
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September 17, 2012

Office Building

Sayre, PA

Seth M. Moyer | Structural

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Executive Summary

The purpose of Technical Report 1 is to investigate and present a thorough understanding of the existing structure of the Office Building through a comprehensive review and summary of its structural conditions and design concepts. The structural system will be summarized through descriptions and calculations for both gravity and lateral loading conditions. The relevant design codes and material specifications for construction will also be outlined.

In addition to calculating and comparing the gravity loads to design values, various spot checks were performed for typical floor framing elements. These elements included the composite deck floor slab (typical), a typical steel floor joist, an interior wide flange beam and an interior column at the first floor. All elements checked met or exceeded required minimum design values (any discrepancies in calculated design values will be addressed in that element's respective gravity check section of this report).

Lateral loads were calculated according to ASCE 7-10 for loading due to both wind and seismic. The base shear due to wind was found to control over seismic by a factor of about 1.7 for both N-S and E-W direction loading. Further in-depth lateral analysis will be performed in Technical Report 3.

Supplemental figures in the form of images and tables are provided and referenced throughout the report as well as appendices, following the conclusion, that contain further detailed hand calculations.

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 ample office and meeting space, as well as 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.

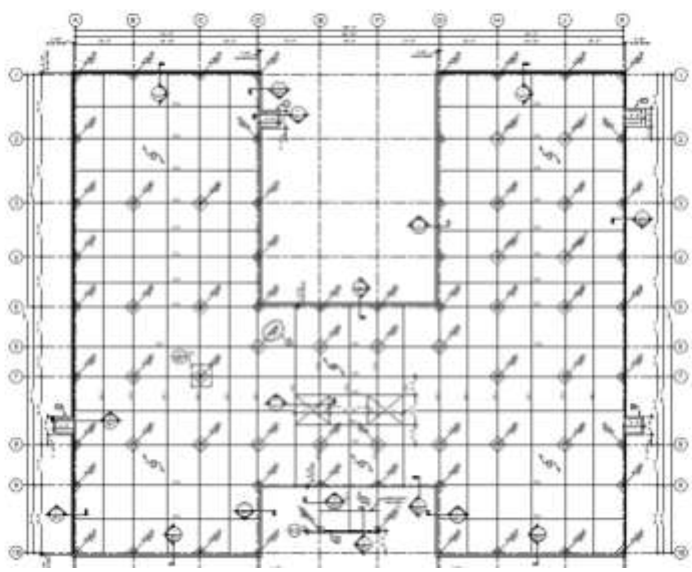


Figure 1: First Floor Slab Plan
(Image Credit: Larson Design Group)

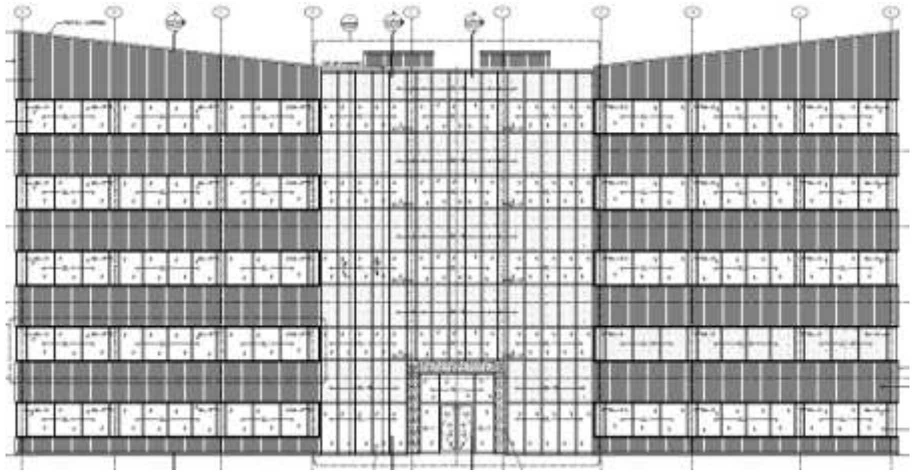


Figure 2: South Elevation
(Image Credit: Silling Associates, Inc.)

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. The joists frame into wide flange steel beams which transfer the loads to wide flange steel columns. The lateral system is comprised 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 utilize 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 detrimental 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 final cover, measured from the bottom of the footing to finish grade. 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.

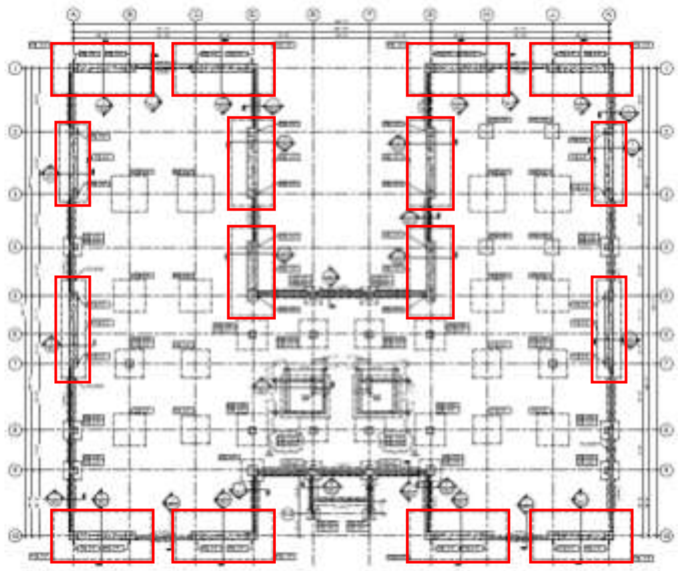


Figure 3: Braced Frames/Combined Footing Locations
(Image Credit: Larson Design Group)

Floor and Framing System

The first floor is a 4” thick slab-on-grade with WWR 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 WWR 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.

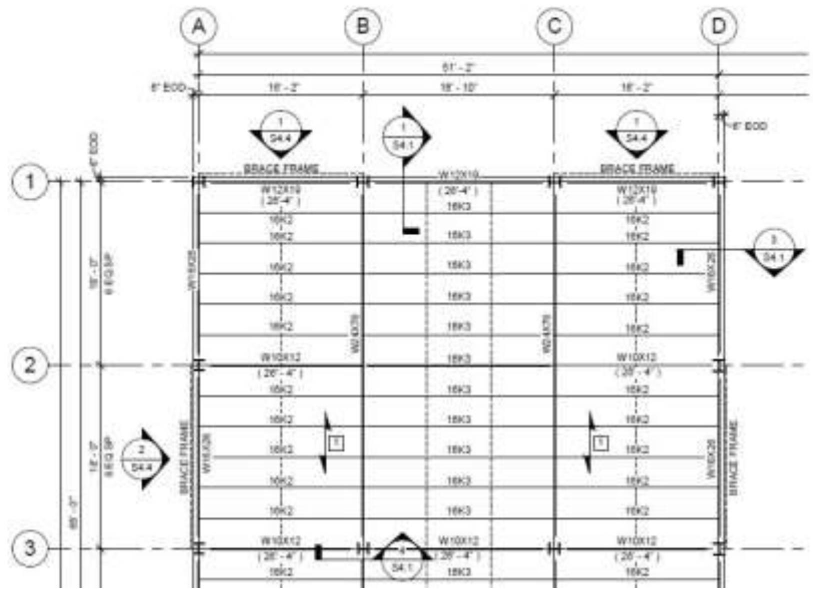


Figure 4: Typical Floor Framing Plan (Enlarged)
(Image Credit: Larson Design Group)

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.

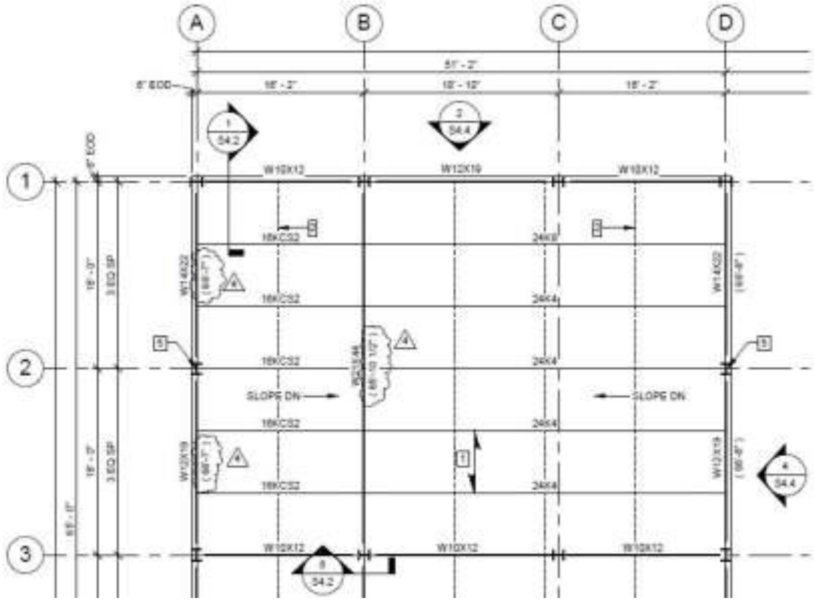


Figure 5: Typical Roof Framing Plan (Enlarged)
(Image Credit: Larson Design Group)

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) (See Figure 6 for typical bracing details).

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 based on their relative degrees of 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 buildings 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 travel 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.

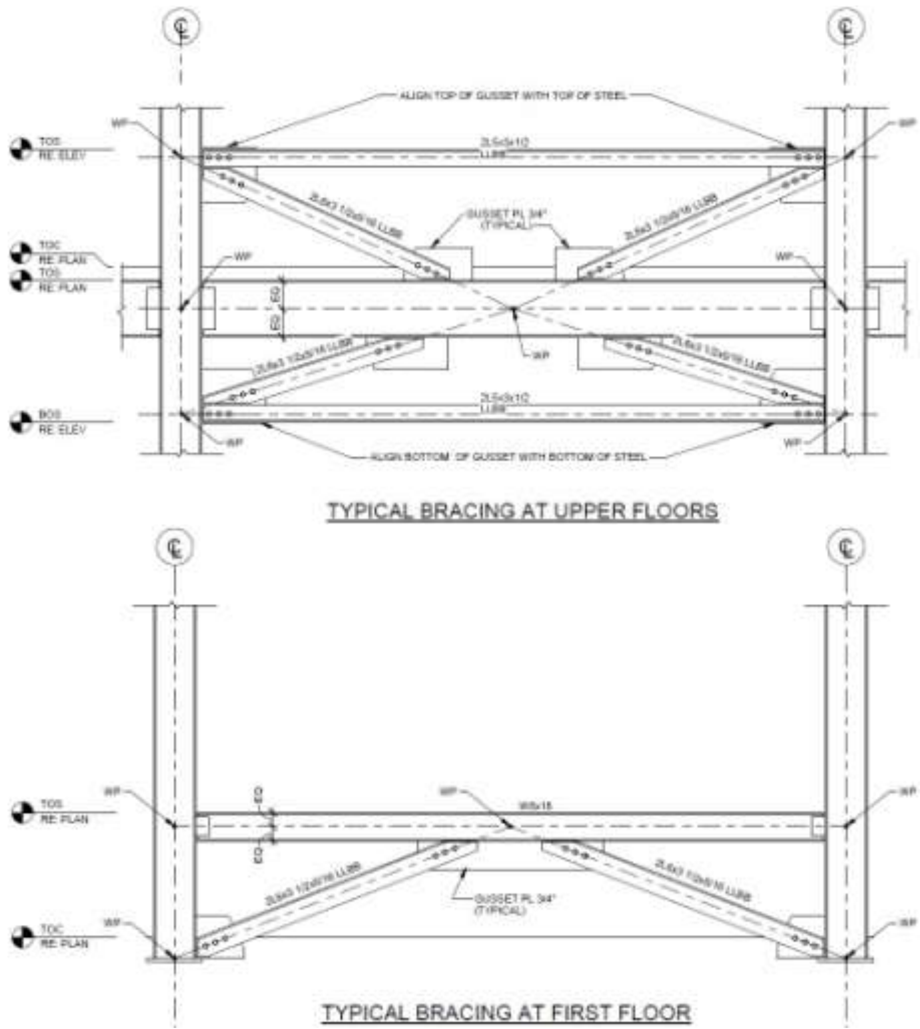


Figure 6: Typical Bracing Details
(Image Credit: Larson Design Group)

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 7.

Steel		
Type	ASTM Standard	Grade
W and WT Shapes	A992	50
Standard Shapes	A36	N/A
Angles, Channels and Plates	A37	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 7: 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 8 below). Refer to Appendix A for a detailed breakdown of the dead load calculations.

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

Figure 8: 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 9 below).

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

Figure 9: 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 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.

<u>Gravity Checks</u>

Refer to Appendix B for full hand calculations of the gravity spot checks

Typical Composite Deck Floor Slab Check:

The 4" thick composite floor deck was checked with the Vulcraft 1.5VL20 composite load tables. The SDI max unshored clear span of 8'-11" (3 span condition) far exceeded the unshored length of 3' that the deck needs to span. The superimposed live load allowed of 400 psf (even at a 5' clear span) is significantly more than the 110 psf that it needs to support. The slab system checked out but seems overdesigned, at least for strength at such a short span. It seems likely that this was designed this way purposely, perhaps to lessen the vibrations that can become a serviceability issue in light framed floor systems such as this. The extra mass should add considerable damping to the overall floor system.

Typical 16K2 Steel Floor Joist Check: (See Figure 4 for member locations)

The joist was checked using the SJI Standard LRFD Load Table for open web k-series steel joists. The member spans 16'-2" and sees a load of wu=703 plf. From the table, a 16K2 spanning 17' has a capacity of 768 plf > 703 => OK. Live load and total load deflections were checked against the appropriate table values and both checks passed easily.

W24x76 Interior Beam Check: (See Figure 4 for member locations)

This beam spans 36' between grid lines 1 and 3 along grid lines B,C,H and J on floors 2-5. The live load was reduced to 67.3 psf. Mu on the beam was calculated to be 510.3 ft-k. The beam was assumed to be laterally braced continuously by the composite deck slab system and phiMpx was found to be 750 ft-k > 510.5 ft-k => OK. Both live load and total load deflections passed fairly easily. Since the deflection checks passed so easily, it may be due to the fact that the designers did not use a reduced live load. When running a quick check with the full 100 psf LL, the deflections do come out to just under the limit.

Column B3 (W12x65) Check at Level 1: (See Figures 4 and/or 5 for member location)

The live load was reduced to 40.7 psf for the column based on its area of influence. Pu was determined to be 331.6 k. Using an effective length (KL) of 13.34’, phiPn was found to be 699.6 k. The column checks out for buckling but is only at about 50% capacity (low utilization). It looks like live load reduction may not have been employed here. It is also possible that the column could be seeing some incidental moment, maybe due to uneven spans and may not be in a state of pure compression.

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 10-15 and Appendix C for wind loading summary and calculations.

N-S Design Wind Pressures							
Surface	Level	Distance (ft)	Wind Pressure (psf)	Internal Pressure		Net Pressure	
				$(+)GC_{pi}$	$(-)GC_{pi}$	$(+)GC_{pi}$	$(-)GC_{pi}$
Windward Wall	1	0	16.63	6.01	-6.01	10.62	22.64
	2	13.33	16.63	6.01	-6.01	10.62	22.64
	3	26.67	18.59	6.01	-6.01	12.58	24.6
	4	40	20.35	6.01	-6.01	14.34	26.36
	5	53.33	21.53	6.01	-6.01	15.52	27.54
	Roof	66.67	22.70	6.01	-6.01	16.69	28.71
	Parapet	74.42	51.38	N/A	N/A	N/A	N/A
Leeward Wall	1-Roof	66.67	-14.19	6.01	-6.01	-20.2	-8.18
	Parapet	74.42	-34.25	N/A	N/A	N/A	N/A
Side Wall	All	N/A	-19.86	6.01	-6.01	-25.87	-13.85
Roof	N/A	0-67	-25.54	6.01	-6.01	-31.55	-19.53
	N/A	67-134	-14.19	6.01	-6.01	-20.2	-8.18
	N/A	>134	-8.51	6.01	-6.01	-14.52	-2.5

Figure 10: 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 11: N-S Wind Forces

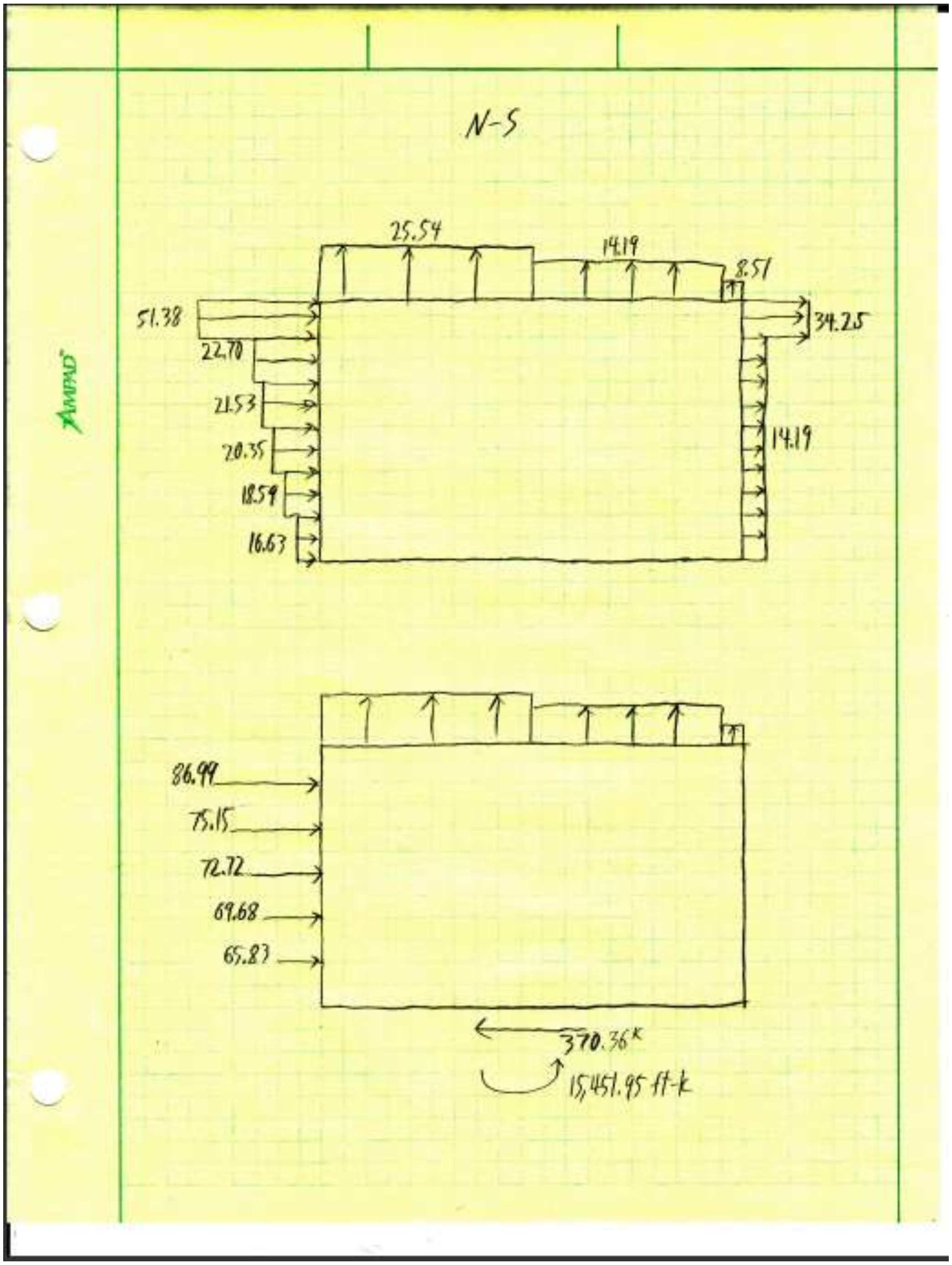


Figure 12: N-S Wind Loading Diagram

E-W Design Wind Pressures							
Surface	Level	Distance (ft)	Wind Pressure (psf)	Internal Pressure		Net Pressure	
				(+)GC_{pi}	(-)GC_{pi}	(+)GC_{pi}	(-)GC_{pi}
Windward Wall	1	0	16.63	6.01	-6.01	10.62	22.64
	2	13.33	16.63	6.01	-6.01	10.62	22.64
	3	26.67	18.59	6.01	-6.01	12.58	24.6
	4	40	20.35	6.01	-6.01	14.34	26.36
	5	53.33	21.53	6.01	-6.01	15.52	27.54
	Roof	66.67	22.70	6.01	-6.01	16.69	28.71
	Parapet	74.42	51.38	N/A	N/A	N/A	N/A
Leeward Wall	1-Roof	66.67	-13.34	6.01	-6.01	-19.35	-7.33
	Parapet	74.42	-34.25	N/A	N/A	N/A	N/A
Side Wall	All	N/A	-19.86	6.01	-6.01	-25.87	-13.85
Roof	N/A	0-67	-25.54	6.01	-6.01	-31.55	-19.53
	N/A	67-134	-14.19	6.01	-6.01	-20.2	-8.18
	N/A	>134	-8.51	6.01	-6.01	-14.52	-2.5

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

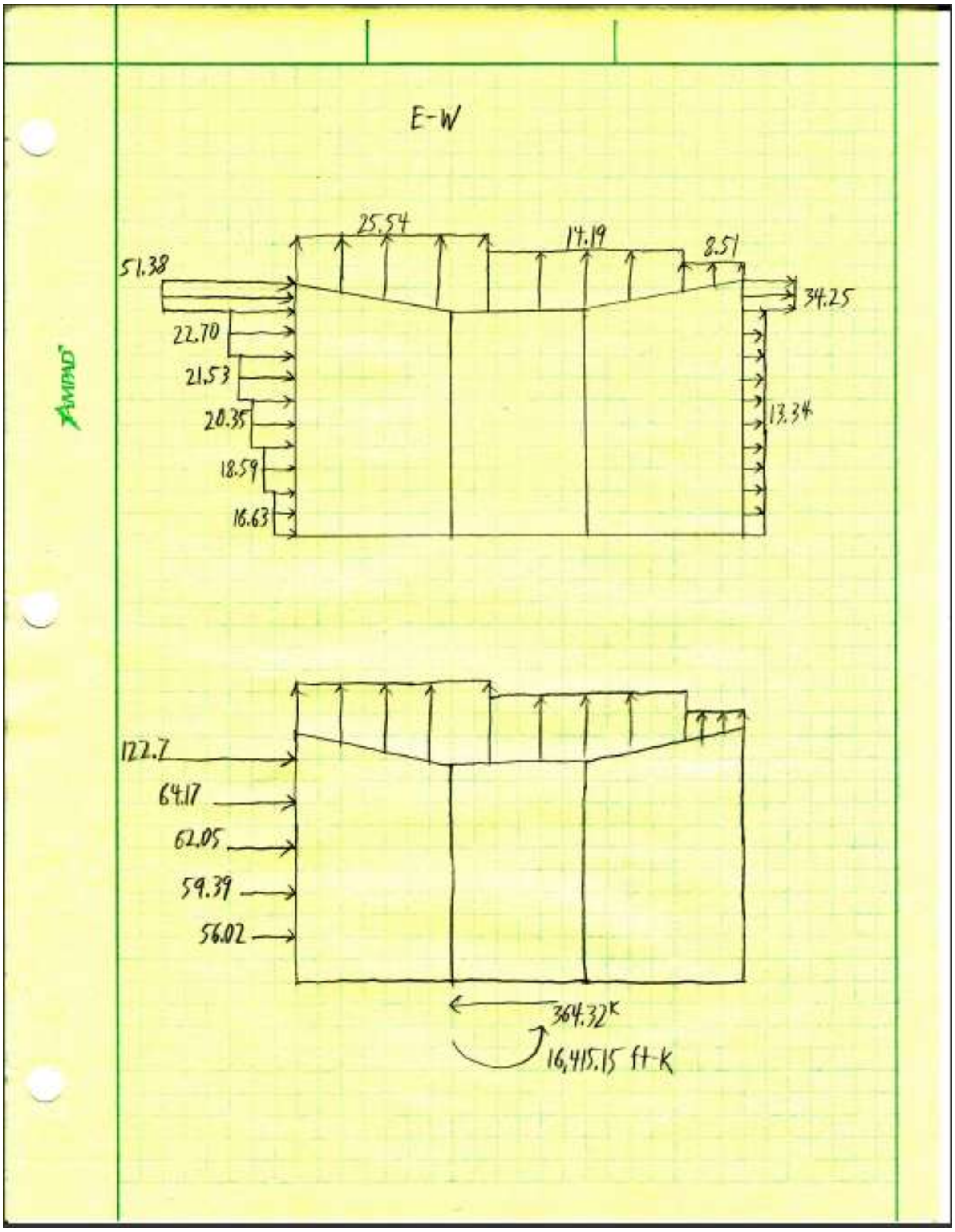


Figure 15: E-W Wind Loading Diagram

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 16-17 and Appendix D 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 16: Seismic Forces

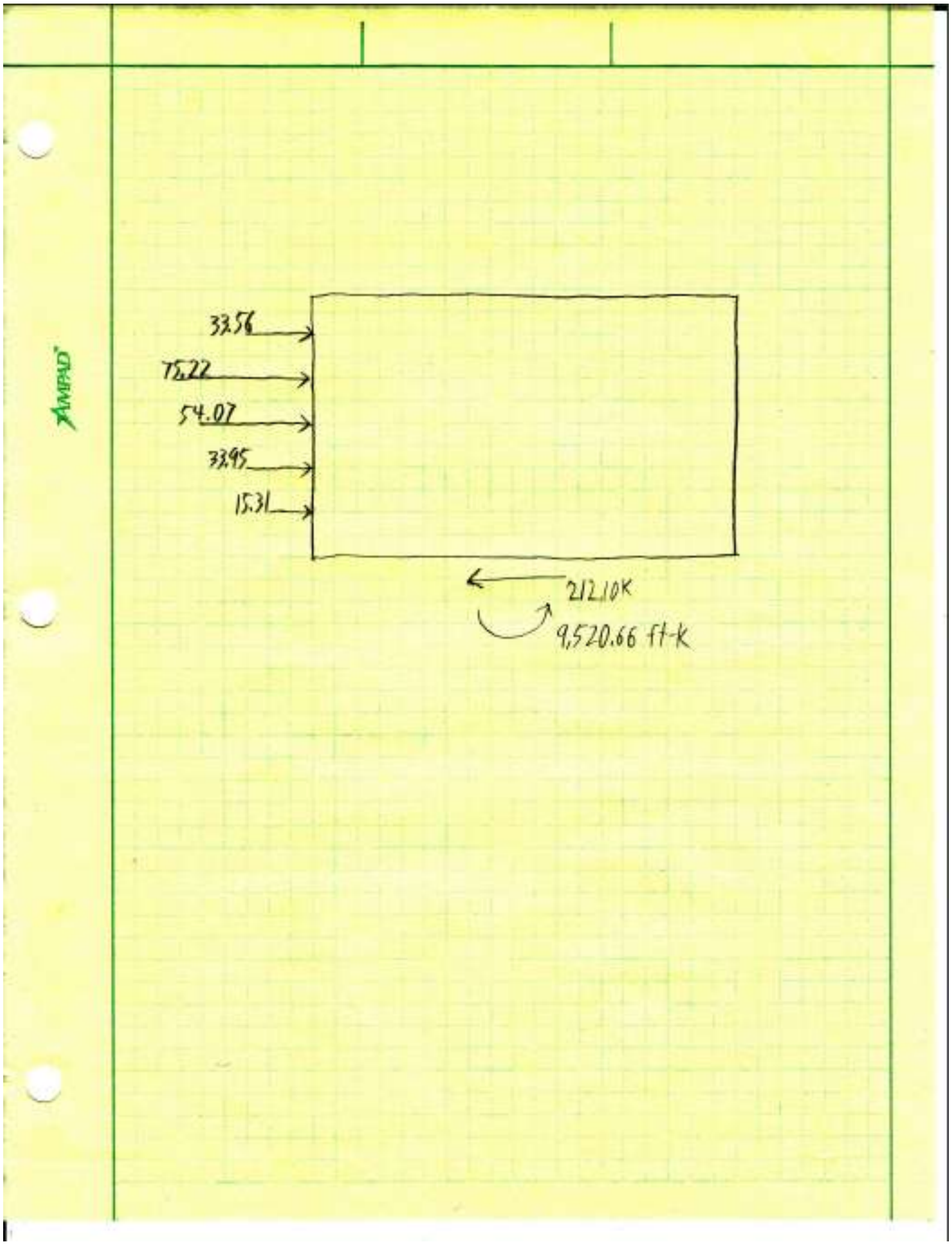


Figure 17: Seismic Loading Diagram

Conclusion

Technical Report 1 has provided an extensive overview of the existing structural conditions and design concepts of the Office Building. A summary of the major structural systems (gravity, lateral and foundations) has been brought together to gain a better understanding of the design concepts, construction methodologies and overall behavior of the structure.

Gravity loads were calculated and compared to the design values for the project. The dead and live loads were determined to match the design loads listed on the structural note sheets. In addition, snow loading was calculated and the discrepancy between it and the design value was addressed. Snow drift loading was also found for the worst case scenario possible on the roof (piled up against the leeward parapet). Spot checks were conducted for various gravity system elements, all of which checked out for strength and deflection criteria. Any discrepancies in loading or member utilization were discussed.

Lateral loads were also calculated for both wind and seismic design conditions. Story shear forces were distributed among the levels of the building and the overall base shears were determined, as well as the overturning moments caused by the story forces. Base shear due to wind loading was found to control over seismic by a factor of about 1.7 for both N-S and E-W wind loading directions.

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

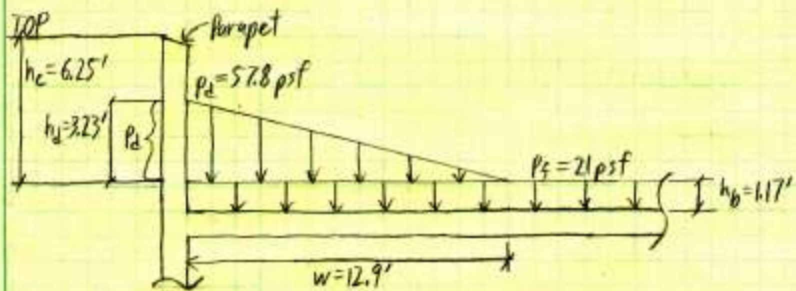
- Drift Loads: (§ 7.8)

Max Drift Case: $h_r = 155.33'$

$h_d = 0.75(0.43\sqrt[3]{155.33}\sqrt[4]{30+10} - 1.5) = 3.23'$ (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 | Tech1-Gravity Load Spot Checks | 1/2

- Typical Floor Slab w/ 1½" Composite Deck (Vulcraft 1.5VL20) (t = 2.5")

  SDI Max. Unshored Clr. Span = 8'-11" > 3'-0" ∴ OK

  Superimposed Live Load @ 5'-0" = 400 psf > 100 (LL) + 10 (SD) = 110 psf ∴ OK

- Typical 16K2 Steel Floor Joist b/w Grid Lines A-B, C-D, G-H, J-K (all 16'-2" spans)

  $W_u = 1.2(60) + 1.6(100) = 232$ psf

  $w_u = 232(3') = 696$ plf $+ 1.2(5.5) = 703$ plf

  From SJI Standard LRFD Load Table, a 16K2 w/ a 17' span has a capacity of 768 plf > 703 plf ∴ OK

  Deflection: $w_L = 100(3') = 300$ plf < 488 plf (load causing Δ of L/360 for 16K2 at 17' span) ∴ OK

  $w_T = (60+100)(3') = 480$ plf $< 1.5(488) = 732$ plf ∴ OK

- W24x76 Beam (Spans 36' b/w Grid Lines 1 & 3 along Grid Lines B, C, H, J on Floors 2-5)

  LL: $L = 100(0.25 + 15/\sqrt{36 \times 35'}) = 67.3$ psf

  $W_u = 1.2(60) + 1.6(67.3) = 180$ psf $w_u = \underbrace{(18 + 10/12 + 16 + 2/12)/2}_{17.5'} \times 180 = 3.15$ klf

  $M_u = 3.15(36)^2/8 = 510.3$ ft-k

  W24x76: $\phi_b M_{px} = 750$ ft-k > 510.3 ft-k ∴ OK (Braced continuously by comp. deck slab)

  $$\Delta_{LL} = \frac{5(67.3 \times 17.5')/1000(36)^4(1728)}{384(29{,}000)(2100)} = 0.73'' < \frac{36' \times 12}{360} = 1.2'' \therefore OK$$

  $$\Delta_{TL} = \frac{5(60+67.3)(17.5)/1000(36)^4(1728)}{384(29{,}000)(2100)} = 1.38'' < \frac{36' \times 12}{240} = 1.8'' \therefore OK$$

  Member self-wt. allowance: 76 plf/17.5' + 6.3 plf/3' = 6.44 psf < 10 psf assumed ∴ OK

Seth Moyer | Tech 1 - Gravity Load Spot Checks | 2/2

- Column B3, W12x65, at it's base at Level 1

Trib area per floor: $(14.5' + 18')(18 + 10/12 + 16 + 7/12)/2 = 568.75\,sf$

LL: $L = 100\left(0.25 + 15/\sqrt{4 \times 568.75 \times 4}\right) = 40.7\,psf$

$P_u = [1.2(60) + 1.6(40.7)](4)(568.75) + [1.2(20) + 0.5(21)](568.75) = 331.6^K$

$\frac{KL}{r} = \frac{1.0(13.34)(12)}{3.02} = 53.0 < 4.71\sqrt{29{,}000/50} = 113.4$

$F_e = \frac{\pi^2(29{,}000)}{[13.34(12)/3.02]^2} = 101.9\ ksi$ $\qquad F_{cr} = \left(0.658^{50/101.9}\right)50 = 40.7\ ksi$ (E3-2)

$\phi P_n = 0.9(40.7)(19.1) = 699.6^K > P_u = 331.6^K \therefore OK$ (E3-1)

AISC 14th, Spec Chapter E.3

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Appendix C

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 $\Rightarrow$ 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 => $135.75/155.33 = 0.87 \Rightarrow C_p = -0.5$

E-W => $155.33/135.75 = 1.14 \Rightarrow C_p = -0.47$ (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 => $p = 24.46(0.85)(0.8) - 33.38(\pm 0.18) = 16.63 \pm 6.01$

2 => $p = 16.63 \pm 6.01$

3 => $p = 27.34(0.85)(0.8) \pm 6.01 = 18.59 \pm 6.01$

4 => $p = 29.93(0.85)(0.8) \pm 6.01 = 20.35 \pm 6.01$

5 => $p = 31.66(0.85)(0.8) \pm 6.01 = 21.53 \pm 6.01$

R => $p = 33.38(0.85)(0.8) \pm 6.01 = 22.70 \pm 6.01$

TOP => $p_p = q_p(GC_{pn})$ (Eq 27.4-4) $\rightarrow p_p = 34.25(1.5) = 51.38$

Leeward Wall: N-S => $33.38(0.85)(-0.5) \pm 6.01 = -14.19 \pm 6.01$

E-W => $33.38(0.85)(-0.47) \pm 6.01 = -13.34 \pm 6.01$

TOP => $p_p = 34.25(-1.0) = -34.25$

Side Wall: $p = 33.38(0.85)(-0.7) \pm 6.01 = -19.86 \pm 6.01$

Roof: Horiz Dist (ft)

0-67 => $p = 33.38(0.85)(-0.9) \pm 6.01 = -25.54 \pm 6.01$

67-134 => $p = 33.38(0.85)(-0.5) \pm 6.01 = -14.19 \pm 6.01$

>134 => $p = 33.38(0.85)(-0.3) \pm 6.01 = -8.51 \pm 6.01$

Appendix D

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
    - $S_{DS} = 2/3\, S_{MS} = 2/3(0.194) = 0.129g$ (Eq 11.4-3)
    - $S_{D1} = 2/3\, S_{M1} = 2/3(0.130) = 0.087g$ (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)
- $C_s = \min \begin{cases} S_{DS}/(R/I_e) = 0.129/(3/1) = 0.043 \\ S_{D1}/(T \cdot R/I_e) = 0.087/(0.796 \cdot 3/1) = 0.0364 \quad \geq 0.044(0.129)(1.0) \geq 0.01 = 0.00568 \therefore ok \\ S_{D1} \cdot T_L/(T^2 \cdot R/I_e) = 0.087(6)/(0.796^2 \cdot 3/1) = 0.275 \end{cases}$

$\Rightarrow C_s = 0.0364$

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

- Effective Seismic Wt

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

  Roof: $20psf\,(17{,}015\ sf)+15psf\,(6.67\ ft+7.75\ ft)(135.75)(2)+15psf\,(6.67\ ft+3.1\ ft)(53.33\ ft)(4)+$

  $15psf\,(6.67\ ft+1.54\ ft)(48.67\times2+18.67\times2+65\times2)=463^{k}$

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

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