victoria interval

MTOB[pennsylvania]

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17 september 2012

MTOB

existing conditions [I]

contents

executive summary

The technical report to follow offers a structural synopsis of the Multi-Tenant Office Building (MTOB), providing knowledge and understanding. This report contains descriptions, diagrams and tables to achieve the summary. Atlantic Engineering Services (AES) designed the structure of the building, including the foundations. AES has provided all structural drawings.

Both gravity loads and lateral loads were calculated and compared to the loads used in the original design. Gravity spot checks were performed to further verify that these values were accurate. It should be noted that the original design is done using ASCE 7-05, while the results in this technical report were calculated using the updated ASCE 7-10. The updated code did not make a difference with the gravity calculations, but the wind calculations were affected.

The appendices include all hand and excel calculations for snow, wind, and seismic loads as well as some drawings that may be useful in understanding the building.

building introduction

The Multi-Tenant Office Building is currently being constructed in Pennsylvania and is expected to be done in July 2013. MTOB is designed as a 5-story, 152,000 square foot office building to be leased into different office spaces for multiple tenants. It is designed to hold high-end office spaces and sits in a luxury office park created by a developer. The architecture plays off of the existing buildings in the office park, which is mostly new construction. Over-sized windows allow natural light to penetrate deep into the spaces without being uncomfortable or distracting. It is expected to have full tenant lease agreements before the completion of the building, which will ensure a successful venture.

structural overview

MTOB is a 5-story steel structure with eccentrically braced frames sitting on drilled concrete caissons. The floors are concrete slab on grade and concrete slab on deck. All calculations are based on Occupancy Category II, for office buildings.

Included in this section:

- **building materials**
- **foundation system**
- **floor system**
- **lateral system**
- **framing system**
- **roof system**

building materials

Although the building exterior has some brick masonry work, the steel frame, CMU walls, and concrete floors and foundations are the only structural aspects of this building. The materials used in this building can be found in Figures 1-3. These were found on AES's sheet S001.

steel	
shape/type	**grade**
structural W shape	ASTM A992
structural M, S, C, MC, L	ASTM A36
HSS steel tube	ASTM A500, grade B
round HSS steel pipe	ASTM A500, grade B
plates and bars	ASTM A36

Figure 1: *(left)*
Structural steel shapes and standards for the project

masonry	
shape/type	**strength [psi]**
8" CMU wall	1500
12" CMU wall	1500
18" CMU wall	1500

Figure 2: *(left)*
Masonry wall sizes and standards for the project

concrete		
Usage	**weight [pcf]**	**strength [psi]**
footings, grade beams, caisson caps	> 144	3000
caissons [drilled piers]	> 144	4000
Walls	> 144	4000
slabs on grade	> 144	4000
elevated floor slabs	> 144	4000
balconies, with 2 ½ gallons of corrosion inhibitor per CY	> 144	5000

Figure 3: *(above)*
Concrete usage and standards for the project

foundation system

The foundation system of MTOB was designed by AES after reviewing the recommendations of the geotechnical reports from the geotechnical engineer, Professional Service Industries, Inc.

preliminary geotechnical recommendation

Professional Service Industries, Inc. (PSI) submitted a preliminary geotechnical recommendation report in December, 2011 based on geotechnical information from existing geotechnical reports and drawings from various geotechnical firms. From the existing reports, PSI noted 14 boring logs of interest to the project. From these borings, it was interpolated that rock can be expected between the approximate elevations of 1020-1030 ft, NGVD. PSI agreed with AES's proposed foundation system of drilled piers with grade beams. Initial design values were given as follows:

25ksf net end bearing pressure
2ksf preliminary slide friction

geotechnical report

A new geotechnical survey was conducted by PSI in February, 2012. The geotechnical engineering firm executed a total of 12 additional borings: 6 in the proposed footprint of the building and 6 in the parking lot areas surrounding the building footprint (see Figure 4). From borings B-1 through B-6, PSI recommends the drilled pier foundations extend to the limestone/sandstone bedrock (found between 9 and 27 feet below the finished floor elevation).

For adequate ground water control, sump pumps shall be used to keep water a minimum of two feet below the subgrade elevation.

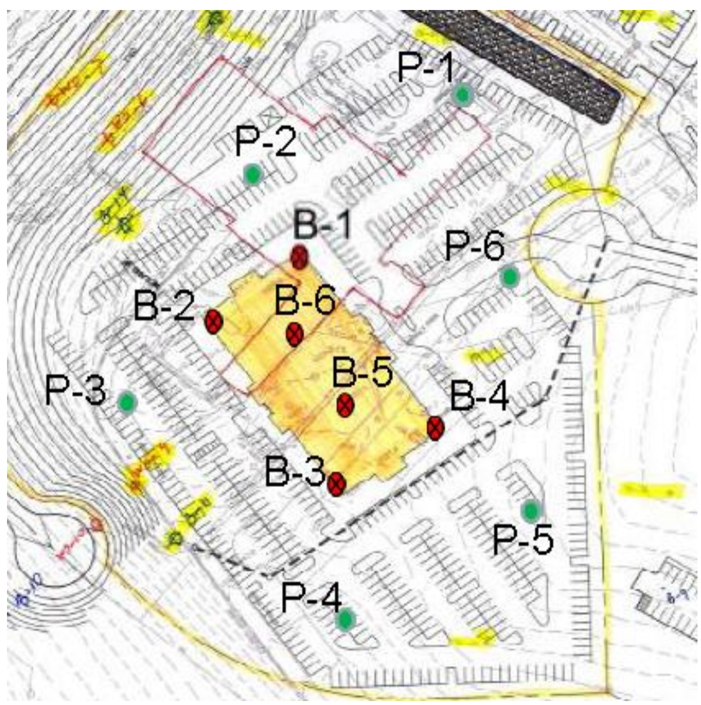


Figure 4: *(above)*
Locations of PSI test borings. Image from PSI geotechnical report

structural overview

foundation design

The MTOB foundation is designed as drilled piers and grade beams along the exterior walls. The concrete grade beams range in sizes from 12”-24” wide and 36”-68” deep. Reinforcement varies, but generally the grade beams are reinforced with #7 bars on top and bottom and #5 bars on the sides. The caissons are designed as 30” diameter concrete with reinforcing and caisson caps depending on the corresponding framing. A plan of the caissons and grade beams can be seen in Figure 5. Note that the grade beams have been highlighted in green and the caissons in pink.

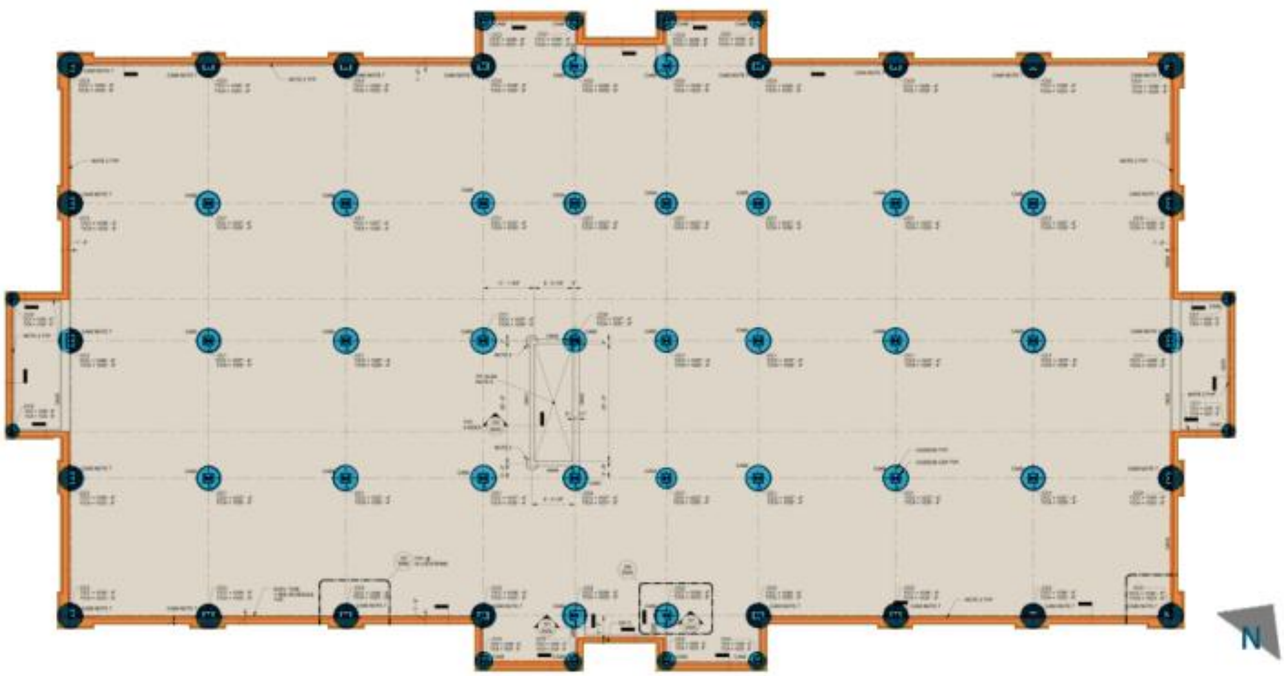


Figure 5: *(above)*
Modified AES foundation plan with caissons highlighted in blue and grade beams highlighted in orange.

floor system

The rectangular building shape is mirrored with regularly spaced bay sizes. Figure 7 shows a typical floor plan with the two typical bay sizes.

Level 1 floor is a typical slab on grade, and levels 2-5 floors are slab on composite deck. Specifically, 3 ½" normal weight concrete on 2" 20 gauge deck for a total thickness of 5 ½". Because of the building's regularity, this is the only type of floor system. See Figure 6 to see the typical floor system on beams.

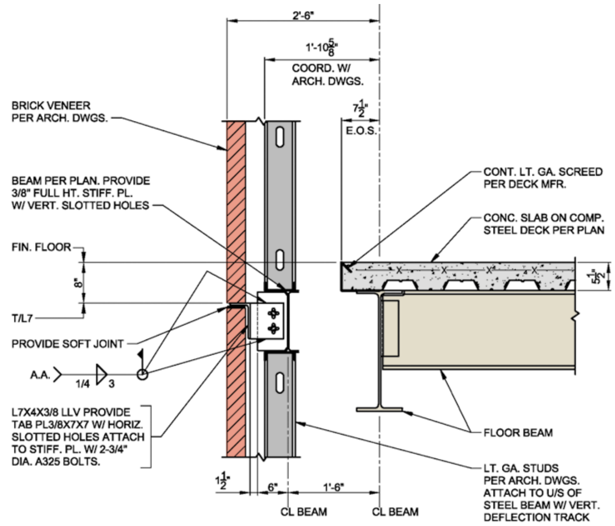
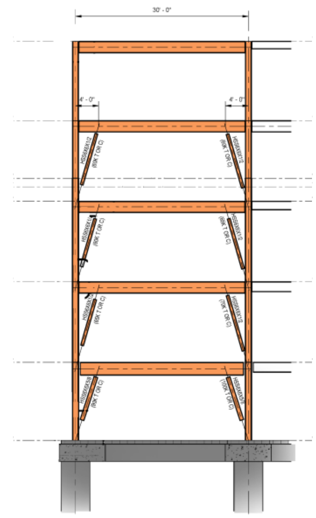


Figure 6: *(above)*
Modified AES section 201 showing a typical floor and exterior wall section.
Figure 7: *(below)*
Typical floor plan with typical bay sizes called out

lateral system

Braced frames resist lateral loads in the MTOB. There are a total of 8 braced frames throughout the building, with three different (though all eccentric) configurations. The frames are eccentric so that none of the bracing crosses behind the large windows that line the exterior walls at every level. See Figure 8 for the typical elevation of MTOB's braced frames. The layout of the braced frames is spaced so that the lateral forces will be adequately acknowledged no matter which direction they approach from. Figure 9 shows the location of each of the 8 braced frames in the building. A components and cladding check has not been included with this technical report, but will be explored in a later report to check that the lateral forces are adequately reaching the braced frames.



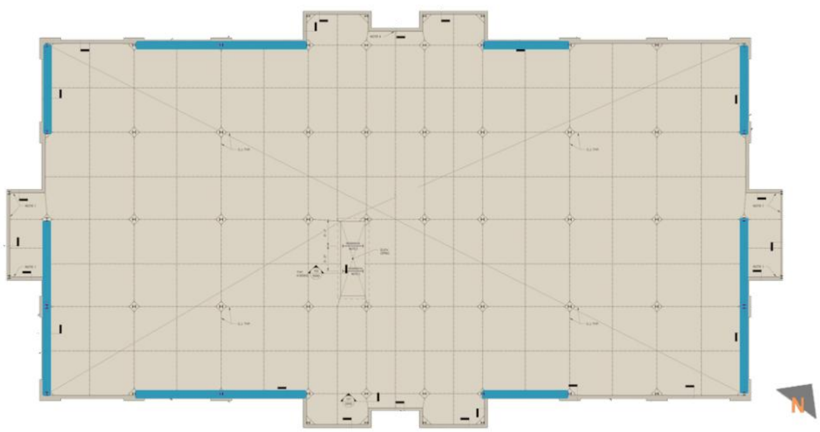


Figure 8: *(above)*
Modified AES braced frame elevation

Figure 9: *(left)*
Modified AES floor plan with locations of braced frames highlighted in pink

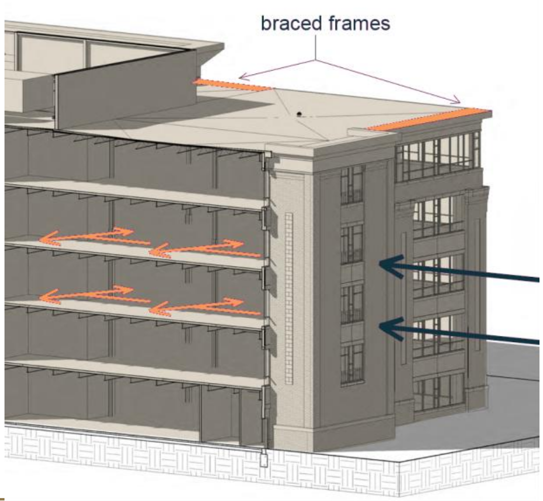


Figure 10: *(above)*
Modified Kernick Architecture building section showing lateral load path

As lateral forces are applied to the building exterior (specifically the components and cladding), bearing connections transfer the loads to the composite floor system. The load travels parallel to the original force. From there, the loads then travel perpendicularly to the braced frames at that particular level through the beams or girders. A lateral load path can be seen in Figure 10.

framing system

MTOB framing consists of five stories of steel columns. Column splices occur on level four at varying heights so that stability is not jeopardized. The majority of columns range from W12x40 to W12x78, but they reach W12x152 in the areas supporting heavier loads under the mechanical penthouse. The beams and girders are composite construction, supporting composite deck.

roof system

The roof of MTOB is an unassuming, simple structure because it does not play an architectural role for the building. The structure consists of 1 ½" galvanized roof deck on supporting beams. Like most steel construction buildings with concrete slabs on deck floor systems, the roof deck does not have any concrete because it is not structurally necessary and the extra weight would cause inefficiencies in the structure. The roof is finished with white TPO Membrane Roof (fully adhered) as the weather resistant covering on top of sloped structure and tapered 20CI insulation. White roofing is becoming more and more popular because of its reflective properties that allow it to minimize heat gain. In an office building, people are often a large contributor to mechanical load and so they have to be cooled most of the year, even in cooler climates like Pennsylvania.

design codes

original codes MTOB was designed using:

- 2009 International Building Code (IBC 2009)
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Building Code Requirements for Structural Concrete (ACI 318-08)
- AISC Manual of Steel Construction, Allowable Stress Design (ASD)

codes used to complete the analysis in this technical report:

- 2009 International Building Code (IBC 2009)
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)
- Building Code Requirements for Structural Concrete (ACI 318-11)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)

gravity load summary

Gravity loads for live, dead, flat roof snow, and drift snow are found using both codes and estimations. Tables are included tabulating the values of the load in each corresponding category.

Included in this section:

- **dead load**
- **live load**
- **snow load**
- **gravity spot checks**

dead load

superimposed dead loads	
description	**load**
level 1 ceiling + misc. mechanical	10 [psf]
levels 2-5 ceiling + misc. mechanical	15 [psf]
roofing	20 [psf]
mechanical spaces	80 [psf]
brick veneer (4" thick)	60 [psf]

Figure 11: *(above)*
Dead loads used in design and in technical report

live load

The design live loads of the building are found using ASCE 7-05. In comparing these with ASCE 7-10, the loads are found to be the same. The mechanical floor allowance is not higher because no expansion is expected for MTOB.

live loads		
description	**design load ASCE 7-05 [psf]**	**ASCE 7-10 [psf]**
public areas	100	100
office lobbies	100	100
office first floor corridors	100	100
office corridors above first floor	80	80
offices	50	50
partitions	15	15
mechanical	100	100
stairs	100	100

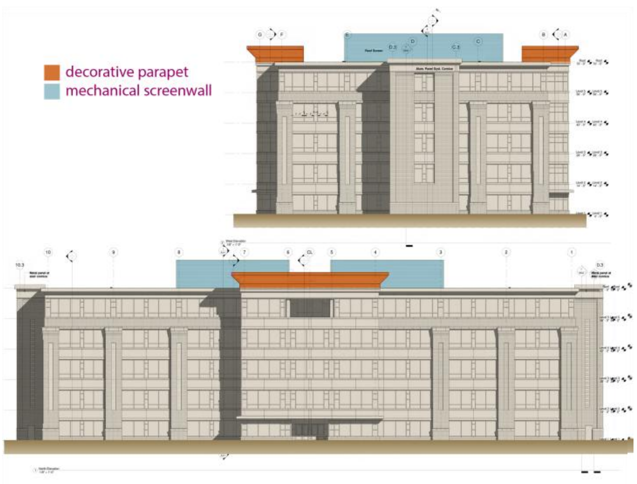
Figure 12: *(above)*
Live loads used in design and in technical report

snow load

Flat roof snow load was calculated using ASCE 7-10. A summary of the factors used and the results can be found in Figure 13 below. Although the maps from ASCE 7-10 chapter 7 (Figure 7-1) indicate a design ground snow load of 25 psf, local code governs with a 30 psf design limit for the area.

flat roof snow load	
description	**value**
exposure factor, C_e	1.0
temperature factor, C_t	1.0
importance factor, I_s	1.0
ground snow load, p_g [psf]	30
flat roof snow load, p_f [psf]	21

Figure 13: *(above)*
Dead loads used in design and in technical report



There were two types of areas on the roof that can cause snow drift. Since the mechanical penthouse stands 14' higher than the main roof, snow drift may accumulate around its walls. The penthouse is centered on the roof and is in the same rectangular shape of the MTOB footprint. Also, along the South and North facing facades, a small portion of the roof has a tall parapet as an architectural feature. See Figure 14, highlighting the areas that will cause snow drift.

Figure 14: *(above)*
Modified Kernick Architecture elevations showing the parapet and screenwall that cause snow drift

To simplify drift load, the worst case drift was calculated (using the longer rectangle dimension of the mechanical screenwall) for use along the exterior perimeter of the mechanical penthouse and along the decorative parapet. Figure 15 shows a summary sketch of the results. Full snow load/drift load calculations can be found in Appendix A.

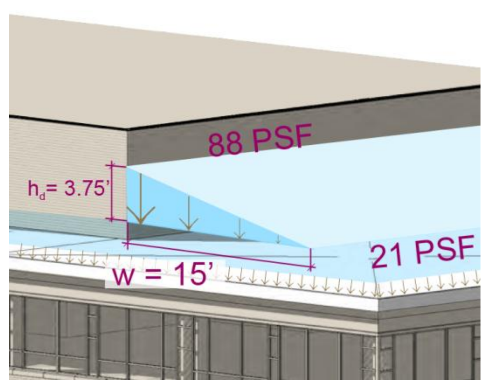


Figure 15: *(above)*
Drift load sketch

gravity spot checks

Gravity spot checks were calculated for the beam "B1," girder "G1," and column "E3." In addition to these calculations, the elevated floor slabs were also verified using the Vulcraft steel deck manual. See Figure 16 for the partial plan and appendix B for hand calculations.

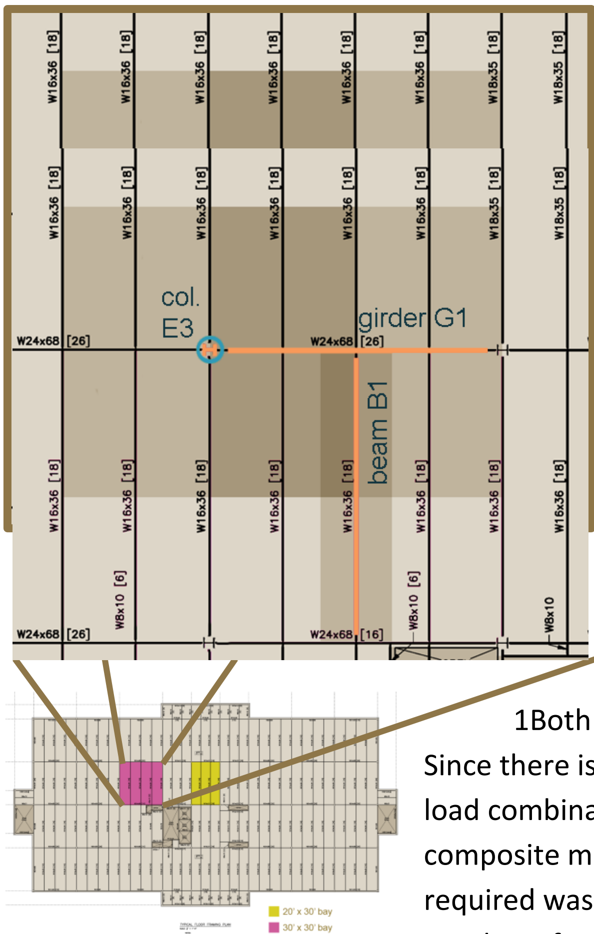


Figure 16: *(above)*
Blown up floor plan showing the beam, girder, and column analyzed in the gravity spot check

steel deck

As stated previously in this technical report, the elevated floor slabs are 3 ½" normal weight concrete on 2" 20 gauge steel deck. The unshored span for a three-span continuous system was verified since the allowable is 9'-9" and the typical bay spaces its beams every 7'-6". Also, the concrete thickness of 3 ½" with the deck provides 1 hour of fire rating, which is appropriate for this type of building.

beam B1 and girder G1

1Both the beam and girder were analyzed using ASD. Since there is no lateral analysis in this technical report, the load combination D+L controlled. Both B1 and G1 are composite members, and the number of studs calculated as required was comparable to the design number. The design number of studs was ~2 higher, but this can be explained away by the use of live load reduction being used for this report, or simply rounding up in the design to add redundancy. Deflection was found significantly under the maximum allowed (only around 50%).

column E3

The column was analyzed similarly to the beam and the girder. The difference was the thought of load combinations since the roof load is included on the column, and part of the roof load is snow. The load combinations that were looked at to control were D+L and D+0.75L+0.75S. In the end, D+L was found to control for two reasons:

1. The roof live load is almost the same as the flat roof snow load (20psf vs. 21psf)
2. Even if the snow load was higher, the reduced live loads of floors 2 to 5 would make a net loss in load, which would be less conservative.

With the load combination D+L chosen again to control, the compressive force was found on the column at its base. The values found for the compressive force were near the compressive strength of the column, but far enough removed that it was a reasonable size for the column loading. The column was also checked for buckling on the weak axis for the unbraced length of 14', which is the floor-to-floor height. This was also found to be adequate.

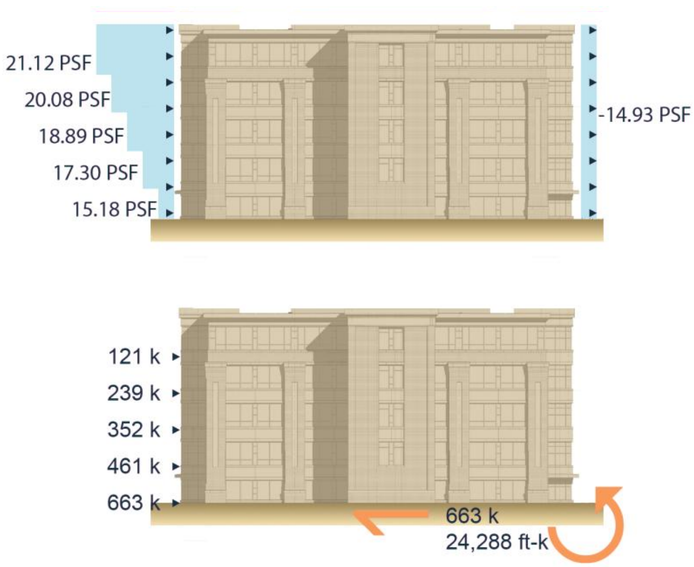
lateral load summary

Included in this section:

- **wind loading**
- **seismic loading**

wind load

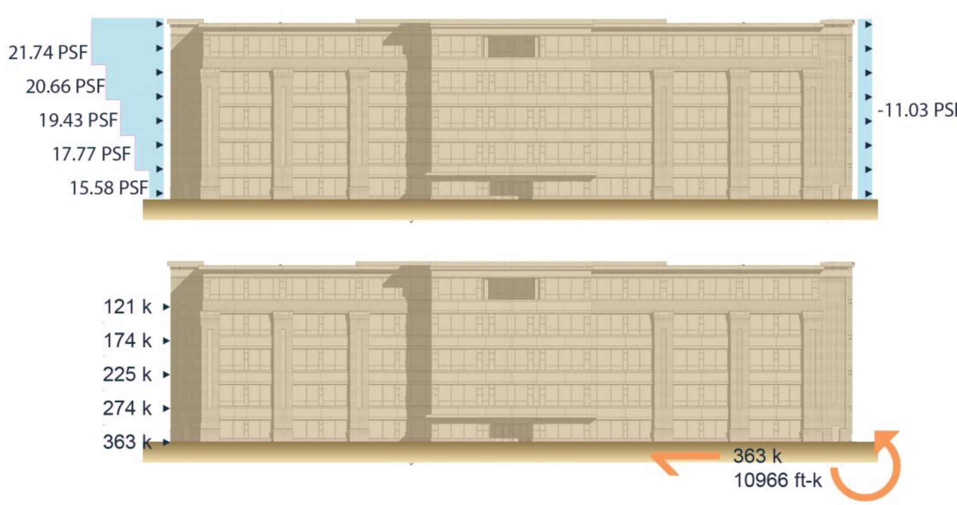
While the original MTOB design pressures were found using ASCE 7-05, the pressures in this technical report were calculated using the updated code, ASCE 7-10. All hand calculations following chapter 26 and 27 of ASCE 7-10 can be found in Appendix C. The design criterion for these calculations matches the design criteria of the original design, except for the main wind velocity. As part of the ASCE 7-10 update, the maps found in chapter 26 contain slightly higher values than the previous maps found in ASCE 7-05, chapter 6. With the changes in both procedure and criteria values, the pressures calculated in this report are slightly higher than the design values on the drawings.



The building is considered rigid since its fundamental frequency is less than 1 hz (see Appendix B for calculations). Using this, the gust factor was calculated for both the N|S and E|W wind directions. Since this is an office building, it is not necessary to withstand more than the basic code recommended values for wind velocity. For the purpose of simplifying, the roofline was assumed straight at 70'. The footprint of MTOB is already mostly rectangular in nature, so no extreme simplifications were necessary for calculations.

Figure 17: *(above)*
N|S wind pressure diagram and story forces with base shear and overturning moment diagram
Figure 18: *(below)*
E|W wind pressure diagram and story forces with base shear and overturning moment diagram

The wind pressures, story shear, base shear, and overturning moments can be seen in Figures 17 and 18 for the N|S and E|W wind directions, respectively. The excel spreadsheet calculations of these values can be found in appendix C with the hand calculations.



lateral load summary

seismic load

The area MTOB is located is not high in seismic activity. From the comparison between the base shear and overturning moment contributed by seismic forces vs. those contributed by wind forces, it is only about a quarter of the magnitude. The summary of seismic findings is tabulated in Figure 19, and full hand calculations can be found in appendix D.

seismic						
level	h_x [ft]	h_x^k	w_x [k]	c_{vx}	F_v [k]	overturning moment [ft-k]
1	0	0	1849	0.0	0.0	0
2	14	18.86429	2603.5	0.0779	10.424	146
3	28	40.80251	2603.5	0.1684	22.547	631
4	42	64.07321	2603.5	0.2645	35.406	1487
5	56	88.25377	2603.5	0.3643	48.767	2731
roof	70	113.1343	697	0.1250	16.736	1172
		$\Sigma w_i h_i^k$:	630780.4	base shear [k]:		134
		total overturning moment [ft-k]:				6167

Figure 19: *(above)*
Summary of seismic forces

lateral load summary

conclusion

Through this technical report of MTOB, a better understanding is gained of the structural system. The caisson foundations, steel W-shape column and beam framing system, eccentrically braced frame lateral system, and simple deck roof system are all summarized with appropriate descriptions and diagrams. The regularity of this 5-story composite steel structure made it relatively straight forward to analyze while still offering an educational challenge.

Both gravity and lateral loads were calculated using ASCE 7-10 and referencing the structural drawings. The wind pressures were a bit higher than the design values calculated by AES, but this can be explained by the changes in the code. The maps were updated, requiring higher wind speeds to design for. It is clear why braced frames were chosen for this building, since the wind load (controlling over seismic in both base shear and overturning moment) was found to have 660 k of base shear and 24,000 ft-k of overturning moment. The load path was described and diagramed, but not analyzed in this technical report. Lateral load checks will be included in a later report.

The gravity loads were used to calculate the overall building weight as well as gravity load spot checks. The spot checks were done on a beam, girder, and column in a typical bay. These checks revealed that the members are all reasonably over-designed to add redundancy into the system, but not too much so that the structure would be notably inefficient. The redundancy is desired in case the occupancy of the building changes throughout its lifespan.

appendices

Included in this section:

- **appendix A: snow load calculations**
- **appendix B: gravity spot checks**
- **appendix C: wind load calculations**
- **appendix D: seismic load calculations**
- **appendix E: additional drawings**

appendix A: snow load calculations

SNOW LOADS (ASCE 7-10) P 1/1

FLAT ROOF SNOW LOAD

$p_f = 0.7\, C_e C_t I_s p_g$

$C_e = 1.0$ T7-2, TERRAIN CATEGORY C, PARTIALLY EXPOSED

$C_t = 1.0$ T7-3

$I_s = 1.0$ T1.5-2, RISK CATEGORY II.

$p_g = 30$ PSF

$p_f = (0.7)(30) = 21$ PSF

DRIFT LOAD (ALONG MECHANICAL PENTHOUSE)

$l_{u,ww} = 60' \Rightarrow h_D = \frac{3}{4}(2.75') = 2.06'$ (LOWER ROOF, WW)

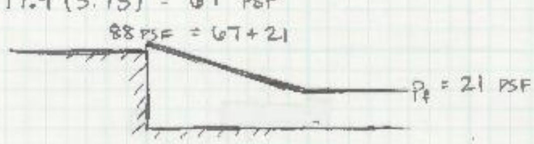
$l_{u,lw} = 120' \Rightarrow h_D = \underline{\underline{3.75'}}$ CONTROLS (UPPER ROOF, LW)

$h_c = 14' \geq 3.75'$

$\Rightarrow w = 4h_D = 4(3.75') = 15'$

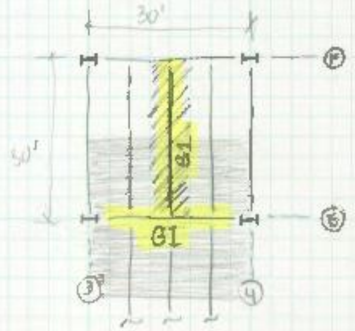
$\gamma = 0.13\, p_g + 14 = 0.13(30) + 14 = 17.9 \leq 30$ pcf

$p_d = \gamma h_D = 17.9(3.75) = 67$ PSF



appendix B: gravity spot checks

GRAVITY SPOT CHECKS 1/5



B1 : W16×36 [18] ← # STUDS

G1 : W24×68 [26]

- TYP LOADING:

SLAB	57 PSF
SDL	15 PSF
SELF WT	5 PSF
LL	80 PSF

- ASD LOAD COMBO § 2.4

D

D + L

D + (L_R OR S OR R)

D + 0.75 L + 0.75 (L_R OR S OR R)

SINCE NOT ON ROOF, S AND L_R NOT APPLICABLE. D + L CONTROLS

LATERAL ANALYSIS NOT INCLUDED

D + (0.6W OR 0.7E)

D + 0.75 L + 0.75 (0.6 W) + 0.75 (L_R OR S OR R)

D + 0.75 L + 0.75 (0.7E) + 0.75 S

0.6 D + 0.6 W

0.6 D + 0.7 E

appendix B: gravity spot checks

GRAVITY SPOT CHECKS 2/5

- BEAM B1

LL REDUCTION (ASCE 7-10, CH 4)

$A_T = (30/4)(30) = 225$ SF

$K_{LL} = 2$ T4-2, INT BEAM

$LL = 80\left[0.25 + \frac{15}{\sqrt{(7.5)(2)}}\right] = 77$ PSF

$W = D + L = (57 + 15 + 5) + (77) = 154$ PSF

154 PSF × 7.5' = 1155 PLF

= 1.155 KLF

$M = \frac{wl^2}{8} = \frac{(1.155)(30)^2}{8} = 129.94$ K·FT

(COMPOSITE BEAM)

$b_{eff} = \min\left|\begin{array}{l} span/8 \\ \tfrac{1}{2}\ d \text{ to adj BEAM} \end{array}\right. + \min\left|\begin{array}{l} span/8 \\ \tfrac{1}{2}\ d \text{ to adj BM} \end{array}\right.$

$= \min\left|\begin{array}{l} 30'/8 = 3.75 \\ \tfrac{1}{2}(7.5) = 3.75 \end{array}\right. + 3.75$

$= 7.5' = 90''$

ASSUME $a = 1.0$, $Y_2 = 6.5 - \frac{1.0}{2} = 6''$

$Q_n : \min\left|\begin{array}{ll} 0.5 A_{sc}\sqrt{f'_c E_c} & = 26.1\text{ K} \\ R_g R_p A_{sc} F_u & = 17.2\text{ K} \end{array}\right.$ (ONE STUD)

$A_{sc} = 3/4\ \Phi = \pi(\tfrac{3}{8})^2 = 0.4418$

$F_u = 65$ ksi

$R_p = 0.6$

$R_g = 1$ FOR 1 STUD

$f'_c = 4000$ PSI

$E_c = 145^{1.5}\sqrt{4} = 3.492$ ksi

CHECK W16×36 [18] M = 238

$\Sigma Q_n = 133$

$133/17.2 = 7.7 = 8 \Rightarrow 16$ STUDS

appendix B: gravity spot checks

GRAVITY SPOT CHECKS 3/5

- BEAM B1 (CONT)

CHECK $a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{133}{0.85(4)(90")} = 0.435 < 1.0$ GOOD

- check LL DEFL

$I_{LB} = 862 \text{ in}^4$

$w_{LL} = 77 \text{ PSF} (7.5') = 577.5 \text{ PLF}$

$y_2 = 6"$

$\Sigma Q_n = 133 \text{ k}$

$\Delta_{LL} = \frac{5(0.5775)(30)^4(12^3)}{384(29000)(862)} = 0.421"$

$\Delta_{LL\,max} = \frac{L}{360} = \frac{30'}{360} = 1" > 0.421"$ GOOD

- CHECK TOTAL LOAD DEFLECTION

$w_{TL} = 1.155 \text{ KLF (P.2)}$

$\Delta_{TL} = \frac{5(1.155)(30)^4 \times 12^3}{384(29000)(862)} = 0.842"$

$\Delta_{TL\,MAX} = \frac{L}{240} = \frac{30'}{240} = 1.5" > 0.842"$ GOOD

$\therefore$ W16×36 w/ 16 STUDS OK $\Rightarrow$ W16×36 w/ 18 STUDS OK

appendix B: gravity spot checks

GRAVITY SPOT CHECK 4/5

• GIRDER G1 → w/A TOTAL OF "6" B1-EQUIVALENT BEAMS FRAMING INTO IT. SEE P.1 FOR DIAGRAM

W24x68 [26]

$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} = 26.1\ k$

3/4" φ

$n = \frac{26}{2} = 13$

$\Sigma Q_n = n \times Q_n = 13(26.1) = 339.3 \rightarrow 251$ AISC T.3-19

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}}$

$b_{eff} = 2 \times \min \left| \begin{matrix} 30/8 = 3.75 \\ 1/2(30/8) = 1.875 \end{matrix} \right. = 45"$

$\frac{251}{0.85(4)(45)} = 1.64"$

$y_2 = 6.5 - \frac{1.64}{2} = 5.67 \Rightarrow$ TRY 5.5"

$M_c = 620\ k\text{-}ft$

ASSUME DISTRIBUTED LOAD:

$w = 68\ PLF + (154\ PSF)(30') = 4.688\ KLF$

$M_R = \frac{wl^2}{8} = \frac{4.688\,(30)^2}{8} = 527.4\ k\text{-}FT < 620\ kFT$ GOOD

• CHECK LL DEFL

$\Delta_{LL} = \frac{5(2.310)(30)^4 \times 12^3}{384(29000)(3450)} = 0.4207 < 1"$ GOOD

I_{LB}

$w = 77\ PSF(30') = 2.31\ KLF$

SAME AS B1 DEFLECTIONS SINCE SAME SPAN

• CHECK TL DEFL

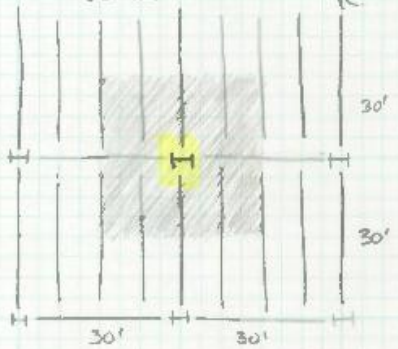
$\Delta_{TL} = \frac{5(4.688)(30)^4 \times 12^3}{384(29000)(3450)} = 0.854" < 1.5"$ GOOD

appendix B: gravity spot checks

GRAVITY SPOT CHECK 5/5

• COLUMN CHECK (@ E-3, BASE LEVEL)

W12×79



• LL REDUCTION

$A_T = 30 \times 30 = 900$ SF
$K_{LL} = 4$ T4-2
$\}$ >400 SF GOOD

$L = L_o\left[0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right] = 80\left[0.25 + \frac{15}{\sqrt{900 \times 4}}\right] = 40 \geq 0.4L_o$ GOOD

• CARRIED LOAD * SINCE THERE IS SNOW LOAD ON ROOF, NEED TO CHECK WHICH L.C. CONTROLS. SINCE THERE IS ONLY ONE LEVEL W/SNOW, AND $S = L_R$ AND THERE ARE 4 LEVELS BEING SUPPORTED W/O SNOW, D+L STILL CONTROLS B/L D+0.75L+0.75S REDUCES THE LOAD CONSIDERABLY MORE

	D + L
ROOF	20 + 20
5	77 + 40
4	77 + 40
3	77 + 40
2	77 + 40

(328 PSF + 180 PSF)(30'×30')

295.2K + 162K COMPRESSION

⇒ 457.2K

UNBRACED LENGTH 14' ⇒ $\frac{P_n}{\Omega} = 556K > 457.2K$ GOOD

T4-1

appendix C: wind calculations

wind pressures [N|S direction]										
level	q_h [psf]	z	k_z	q_z [psf]	windward [psf]	leeward [psf]	trib area [sf]	force [k]	story shear [k]	overturning moment [ft-k]
1	25.61	0	0.57	16.40	15.18	-14.93	3360	101	663	0
2	25.61	14	0.57	16.40	15.18	-14.93	3360	101	562	1417
3	25.61	28	0.684	19.68	17.30	-14.93	3360	108	461	3032
4	25.61	42	0.77	22.16	18.89	-14.93	3360	114	352	4773
5	25.61	56	0.834	24.00	20.08	-14.93	3360	118	239	6588
roof	25.61	70	0.89	25.61	21.12	-14.93	3360	121	121	8479
									base shear [k]:	**663**
									total overturning moment [ft-k]:	**24288**

wind pressures [E|W direction]										
level	q_h [psf]	z	k_z	q_z [psf]	windward [psf]	leeward [psf]	trib area [sf]	force [k]	story shear [k]	overturning moment [ft-k]
1	25.61	0	0.57	16.40	15.58	-11.03	1680	45	363	0
2	25.61	14	0.57	16.40	15.58	-11.03	1680	45	319	626
3	25.61	28	0.684	19.68	17.77	-11.03	1680	48	274	1355
4	25.61	42	0.77	22.16	19.43	-11.03	1680	51	225	2149
5	25.61	56	0.834	24.00	20.66	-11.03	1680	53	174	2982
roof	25.61	70	0.89	25.61	21.74	-11.03	1680	55	121	3854
									base shear [k]:	**363**
									total overturning moment [ft-k]:	**10966**

appendix C: wind calculations

WIND LOADS (ASCE 7-10) p 1/3

BASIC WIND SPEED 115 MPH (FIG 26.5-A)
IMPORTANCE FACTOR 1.0
OCCUPANCY CRITERIA II
EXPOSURE CATEGORY B
ENCLOSED

GC_{pi}	± 0.18	T 26.11-1
C_p (WW)	0.8	FIG 27.4-1

C_p (LW) (N/S) $\frac{L}{B} = \frac{120}{240} = \frac{1}{2} \Rightarrow -0.5$
E/W $\frac{L}{B} = \frac{240}{120} = 2 \Rightarrow -0.3$ } FIG 27.4-1

k_d	0.85	T 26.6-1
k_{zt}	1.0	
k_z	VARIES W/HEIGHT	T 27.3-1

GUST EFFECT FACTOR, G

CHECK IF BLDG IS RIGID: ($f > 1$ Hz) §12.8.2.1

$T_a = C_t h_n^x$

$C_t = 0.03$
$x = 0.75$ } T 12.8-2
$h = 70$ FT

$T_a = (0.03)(70)^{0.75} = 0.726$

$f = \frac{1}{T_a} = \frac{1}{0.726} = 1.377 > 1$ Hz $\therefore$ BLDG IS RIGID

CALCULATE G USING §26.9.4 FOR RIGID STRUCTURES
(SEE PG 2 CALCS)

appendix C: wind calculations

WIND LOADS (ASCE 7-10)   P 2/3

GUST EFFECT FACTOR, G   §26.9.4 (RIGID)

$$G = 0.925\left[\frac{1 + 1.7 g_Q I_z Q}{1 + 1.7 g_v I_z}\right]$$

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

$c = 0.3$ (T 26.9-1)

$z_{min} = 30\text{ FT}$ (T 26.9-1)

$z = 0.6h = 0.6(70) = 42\text{ FT} > 30\text{ FT (OK)}$

$$I_z = 0.3\left(\frac{33}{42}\right)^{1/6} = 0.288$$

$$Q = \sqrt{1/\left(1 + 0.63\left(\frac{B+h}{L_z}\right)^{0.63}\right)}$$

$B = 240$ FOR N/S, 120 FOR E/W

$h = 70\text{ FT}$

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

$\ell = 320\text{ FT}$ (T 26.9-1)

$\bar{\epsilon} = 1/3$ (T 26.9-1)

$$L_z = 320\left(\frac{42\text{FT}}{33}\right)^{1/3} = 346.8$$

N/S $$Q = \sqrt{1/\left(1 + 0.63\left(\frac{240+70}{346.8}\right)^{0.63}\right)} = 0.7938$$

E/W $$Q = \sqrt{1/\left(1 + 0.63\left(\frac{120+70}{346.8}\right)^{0.63}\right)} = 0.8359$$

$g_v = g_Q = 3.4$

N/S $$G = 0.925\left[\frac{1 + 1.7(3.4)(0.288)(0.7938)}{1 + 1.7(3.4)(0.288)}\right] = 0.8058$$

E/W $$G = 0.925\left[\frac{1 + 1.7(3.4)(0.288)(0.8359)}{1 + 1.7(3.4)(0.288)}\right] = 0.8302$$

appendix C: wind calculations

WIND LOADS (ASCE 7-10) p 3/3

$$q_h = 0.00256 K_z K_{zt} K_d V^2 I$$
$$= 0.00256(0.89)(1.0)(0.85)(115)^2(1.0)$$
$$= 25.61 \text{ PSF}$$

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

$$P_{WW} = q_z GC_p - q_i(GC_{pi}) = q_z(\overset{(N/S)}{0.8058})(0.8) - q_i(\pm 0.18)$$

$$P_{LW} = q_h GC_p - q_i(GC_{pi}) = 25.61(\overset{(E/W)}{0.8359})(\overset{(E/W)}{-0.3}) - q_i(\pm 0.18)$$

* ALL PRESSURES CALCULATED IN EXCEL SPREADSHEET

appendices

appendix D: seismic calculations

SEISMIC LOAD (ASCE 7-10) 1/2

BLDG OCCUPANCY CATEGORY		II
IMPORTANCE FACTOR		1.0
SITE CLASS		C
S_s	0.108 g	geohazards.usgs.gov/designmaps/us
S_1	0.053 g	
S_{MS}	0.129 g	
S_{M1}	0.090 g	
S_{DS}	0.086 g	
S_{D1}	0.060 g	

$T = C_t h_n^x = 0.726\,s$ (SEE WIND CALCS, p.1)

CHECK SPECTRAL RESPONSE ACCELERATION PARAMETERS

$S_{MS} = F_a S_s$ EQN 11.4-1

$F_a = 1.2$ T 11.4-1

$S_s = 0.108g < 0.25$

$= 1.2(0.108) = 0.1296 \sim 0.129$ OK

$S_{M1} = F_v S_1$ EQN 11.4-2

$F_v = 1.7$ T 11.4-2

$S_1 = 0.053g$

$= (1.7)(0.053) = 0.0901 \sim 0.090$ OK

$S_{DS} = \frac{2}{3} S_{MS} = 0.0864$ OK EQN 11.4-3

$S_{D1} = \frac{2}{3} S_{M1} = 0.06007$ OK EQN 11.4-4

WILL USE USGS VALUES

$T_L = 12\,s$ FIG 22-12

$T_0 = 0.2 \frac{S_{D1}}{S_{DS}} = 0.1395$

$T_S = \frac{S_{D1}}{S_{DS}} = 0.698$

$\rho = 1.0$

$\Omega = 2$ T 12.2-1

$C_d = 4$ T 12.2-1

$R = 8$ T 12.2-1

appendix D: seismic calculations

SEISMIC LOAD (ASCE 7-10) 2/2

$0.726s = T < T_L = 12s \Rightarrow C_s = \frac{S_{D1}}{T(\frac{R}{I})}$, $C_s = \frac{S_{DS}}{R/I}$ EQN 12.8-2, EQN 12.8-3

$C_s = \frac{(0.06)}{(0.726)(\frac{8}{1.0})} = 0.0103$

$C_s = \frac{0.086}{8} = 0.01075$

$\therefore$ 0.0103 CONTROLS C_s

TOTAL BLDG WT: 12960 k (SEE BLDG WT CALCS)

$V = C_s W = (0.0103)(12960k) = 133.88\ k$ EQN 12.8-1

$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$ EQN 12.8-12

k:

T	k
0.5	1
0.726	1.113
2.5	2

$\leftarrow = \left(\frac{2-1}{2.5-0.5}\right)(0.726-0.5)+1$

$C_{vx}@ROOF = \frac{(697)(70)^{1.113}}{(697(70)^{1.113} + 2603.5(56)^{1.113} + 2603(42)^{1.113} + 2603(28)^{1.113} + 2603(14)^{1.113})}$

$\Sigma w_i h_i^k = 630780.4$

$= 0.125$

$F_x @ ROOF = C_{vx} V = (0.125)(133.86) = 16.7\ k$

* OTHER LEVELS C_{vx} AND F_x IN EXCEL SPREADSHEET

appendix D: seismic calculations

FLOOR WEIGHTS (ASD) 1/2

TYP FLOOR SELF WT:

CONC ON DECK ——— 57 PSF (VULCRAFT, P 52)

STEEL BEAMS ——— 5.4 PLF

30'/4 = 7.5'

20'/3 = 6.67 ⇐ CONTROLS SPACING TO STAY CONSERVATIVE

W16×36 @ 6.67'

$\Rightarrow \frac{36 \text{ PLF}}{6.67'} = 5.4 \text{ PLF}$

STEEL GIRDERS ——— 2.27 PSF

30' SPACES

W24×68

$\Rightarrow \frac{68 \text{ PLF}}{30'} = 2.27 \text{ PSF}$

EXT WALL ——— 1209.6 K (TOTAL FOR ENTIRE BLDG)

ASSUME ~40% EXT

(120)(70)(2) + (240)(70)(2) = 50400 SF SURFACE AREA

50400 SF (0.4) × 60 PSF = 1209.6 K / 5 FLOORS = 241.9 K PER FLOOR

STEEL COLUMNS ——— 2.4 PSF

ASSUME W12×79 AS TYP MIDDLE SIZE

~62 COL/FLOOR

14' HEIGHT

$79 \times 14 \times 62 = \frac{68572 \text{ \# PER FLOOR}}{120 \times 240} = 2.4 \text{ PSF}$

▸ TOTAL SELF WT PER TYP FLOOR:

57 + 5.4 + 2.27 + 2.4 = 67 PSF SELF WT

appendix D: seismic calculations

FLOOR WTS 2/2

• ROOF

20 PSF ROOFING × 240×120 = 576 k

½ HEIGHT EXT WALL = 241.9/2 = 120.96 k

696.96 K

• FLOORS 2-5

(67 PSF + 15 PSF [mech + misc])(120)(240) + 241.9 [EXT WALL] = 2603.5 k

• FLOOR 1

SOG 4" NWC

$150 \text{ PCF} \times \frac{4''}{12} = 50 \text{ PSF}$

(50 PSF + 10 PSF [misc])(240)(120) = 1728 K + 241.9/2 = 1849 k

TOTAL BLDG WT:

ROOF + 1st + (2 to 5)

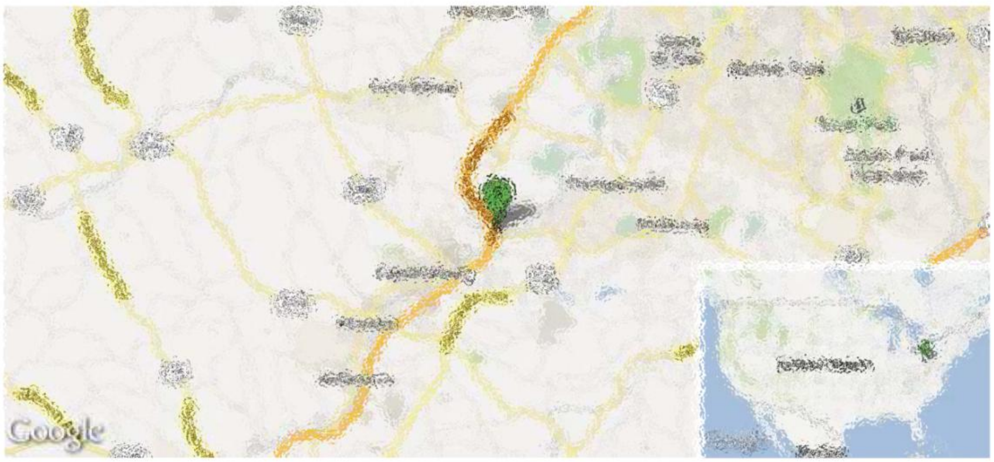
697 + 1849 + (4)(2603.5) = 12960 K

appendix D: seismic calculations

USGS Design Maps Summary Report

User–Specified Input

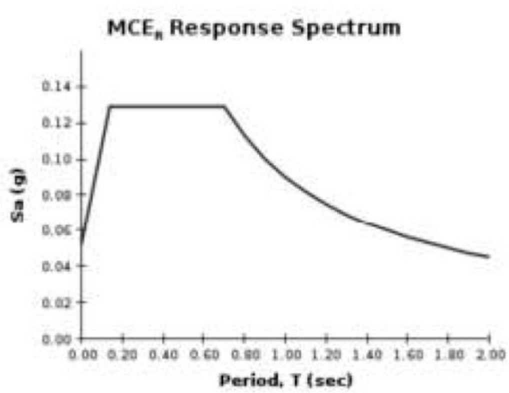
Report Title	Multi-Tenant Office Building Fri September 14, 2012 21:28:40 UTC
Building Code Reference Document	ASCE 7-10 Standard (which makes use of 2008 USGS hazard data)
Site Coordinates	[illegible]
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

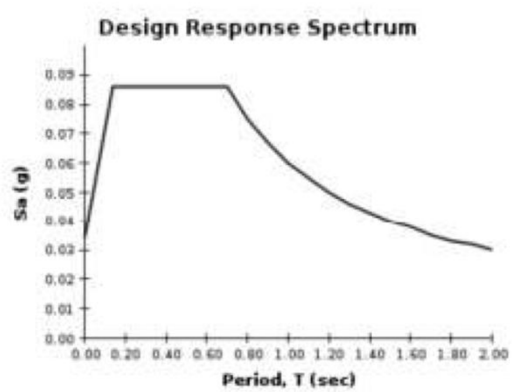


USGS–Provided Output

$S_S =$ 0.108 g	$S_{MS} =$ 0.129 g	$S_{DS} =$ 0.086 g
$S_1 =$ 0.053 g	$S_{M1} =$ 0.090 g	$S_{D1} =$ 0.060 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

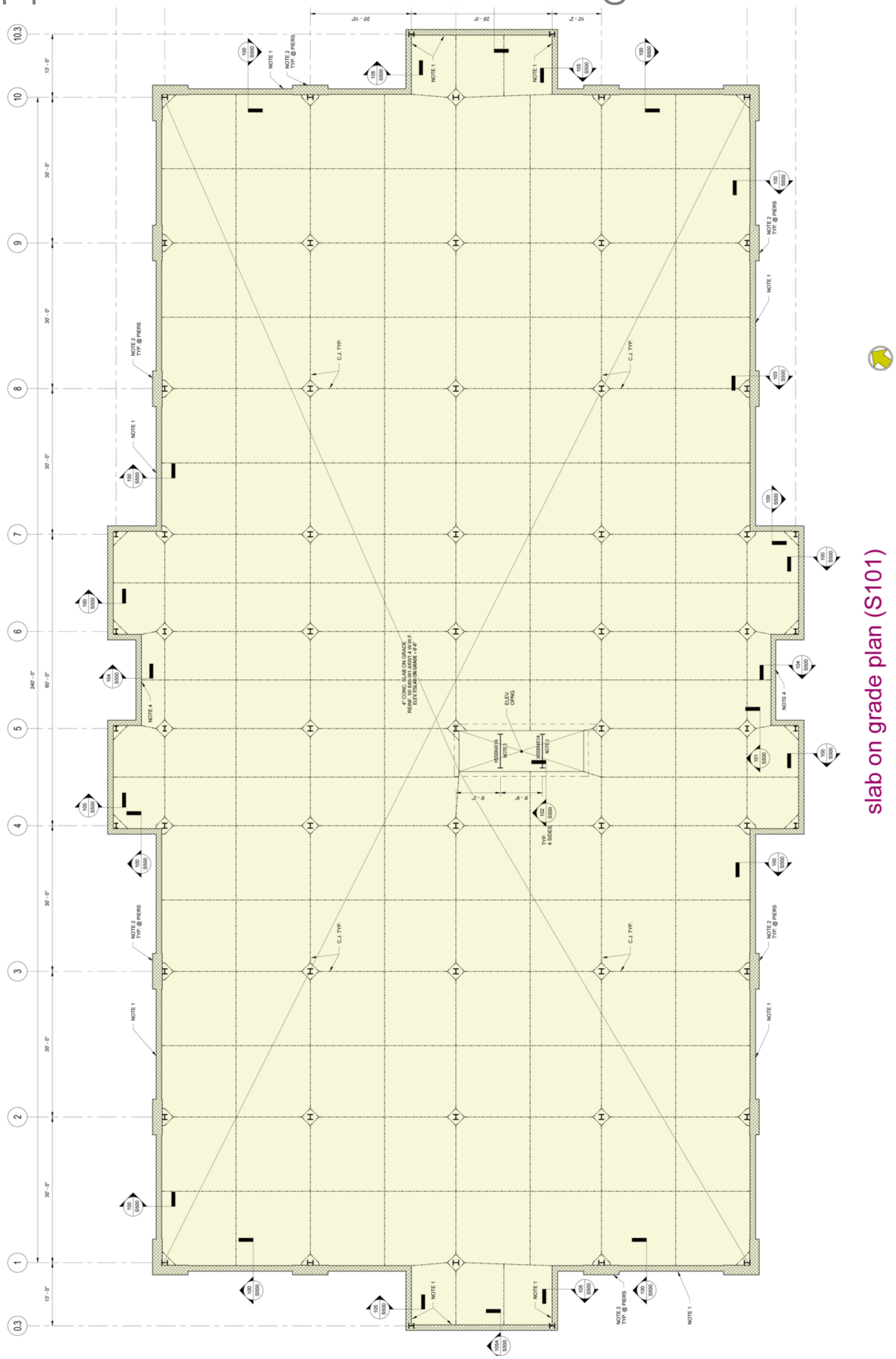




For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

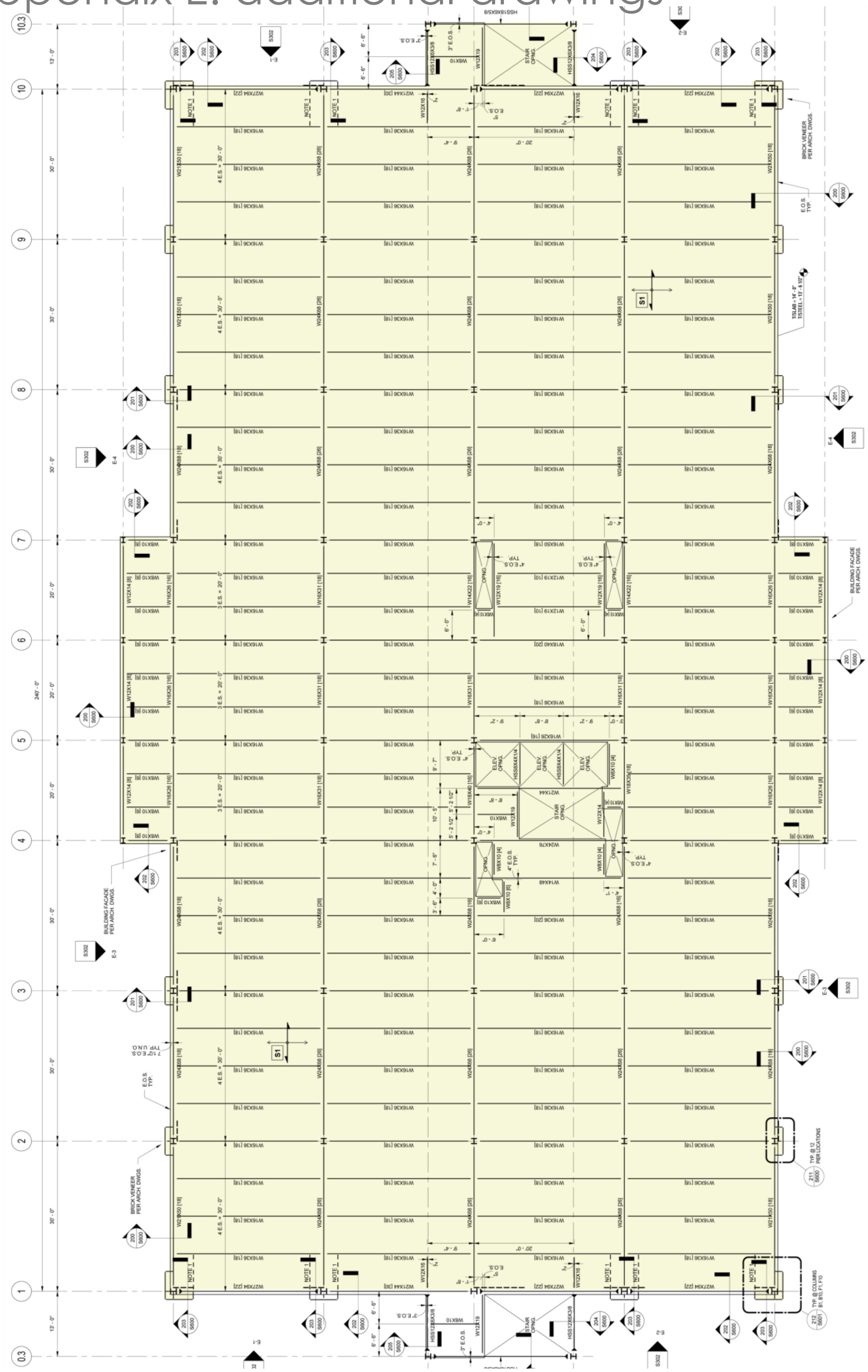
appendices

appendix E: additional drawings



slab on grade plan (S101)

appendix E: additional drawings



typical floor plan for floors 2-5 (S200)