lateral system analysis [III]

victoria interval

MTOB[pennsylvania]

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12 november 2012

contents

building introduction

The Multi-Tenant Office Building [MTOB] 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.

executive summary

Technical report 3 analyzes the existing lateral system in more detail than was covered in technical report one. As part of the analysis, a computer model of MTOB is created using RAM Structural System. The results are then found using RAM and verified with hand calculation spot checks.

Different load cases for each type of seismic load are analyzed. Case 1 for wind is found to control overall, so this is used in both the RAM model and in hand calculation checks.

Stiffnesses of each braced frame are found by modeling each type of braced frame in STAAD, another computer modeling software. There are three different types of braced frames in MTOB, so three different stiffnesses are found. In reflecting upon these values, the stiffnesses are logical for each frame, with the double frames carrying larger values and shorter gaps between the braces at the center of beam span also carrying larger values.

Drift and displacement are found using RAM and analyzed against the code values. It should be noted that while previous technical reports used ASCE 7-10, this report uses ASCE 7-05 to take into account the program's available codes for modeling. The inter-story drift values are found to be all well within the allowable code drift values.

Distribution of lateral forces is also examined in this technical report. First, general lateral load paths are discussed. Second, these load paths are taken into further consideration with the relative stiffnesses of each frame, and third with the addition (or subtraction) of torsional shear to find the total shear on each frame.

Finally, a lateral spot check is done on one of the braced frames. The check analyzes a brace at the second story and the column that spans from story 1 to 2. Both of these members are found to adequately hold the required load.

Appendices can be found at the end of the report with more details in each of these areas, including RAM results output, hand calculations, and lateral frame elevations.

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 [ASCE7-10].

included in this section:

building materials
foundation system
framing system
floor system
lateral 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).

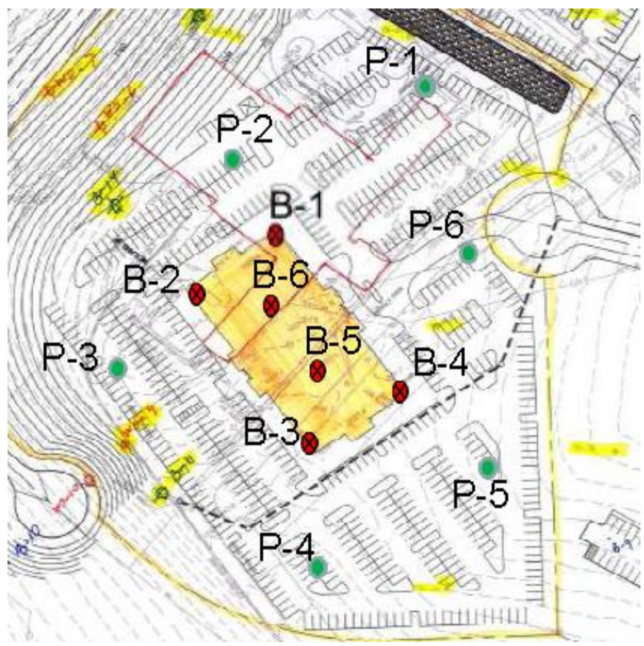


Figure 4: *(above)*
Locations of PSI test borings. Image taken from PSI geotechnical report

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

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.

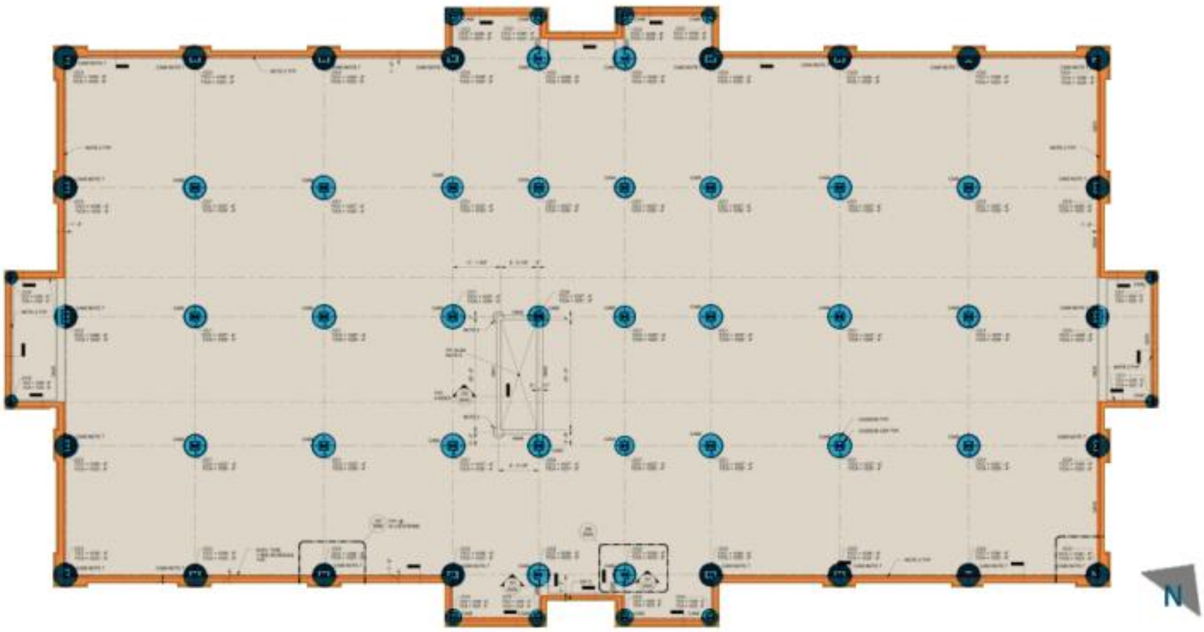


Figure 5: *(above)*
Modified AES foundation plan with caissons highlighted in blue and grade beams highlighted in orange.

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.

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.

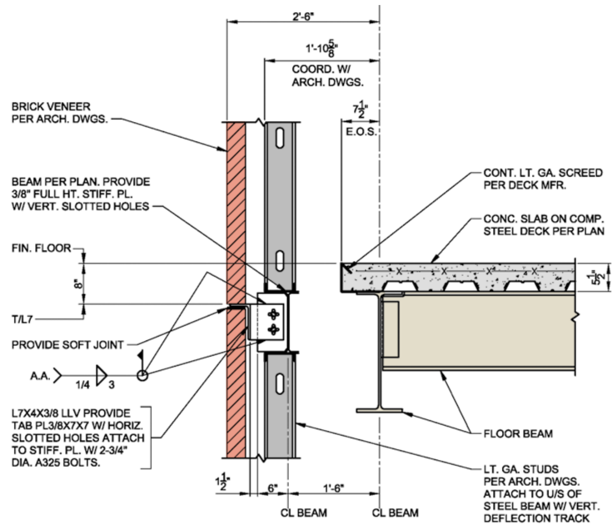
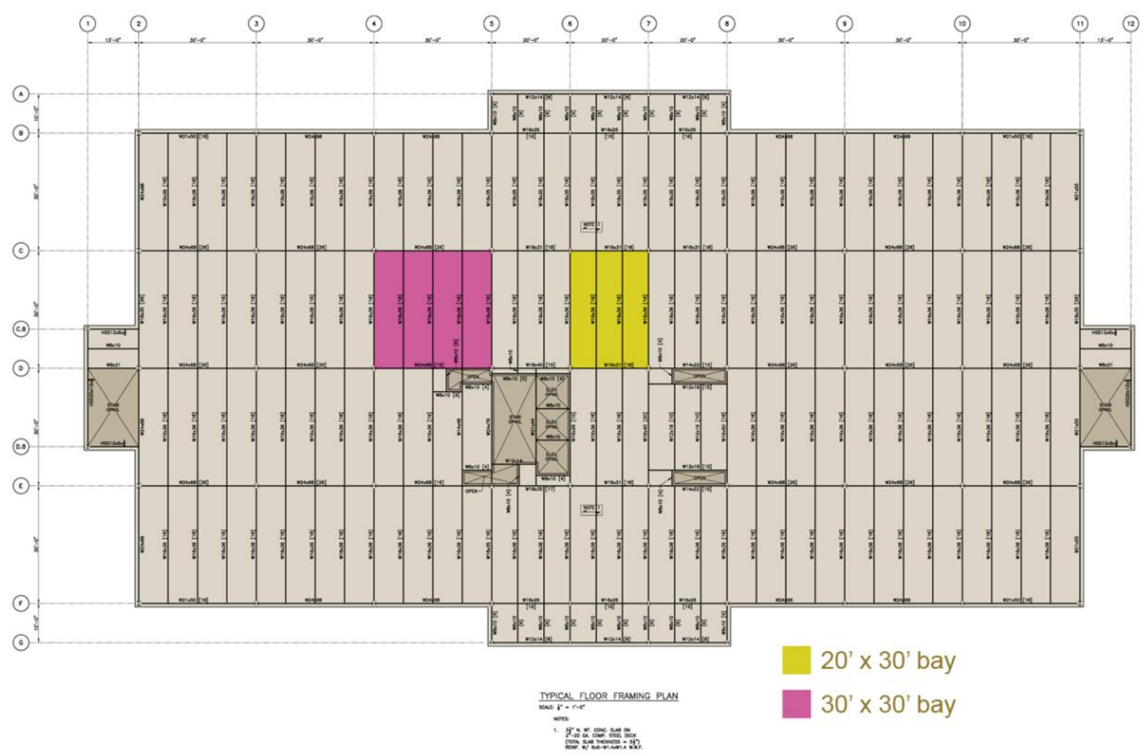


Figure 6: *(above)*
Modified AES section 201 showing a typical floor and exterior wall section.
Figure 7: *(below)*
Typical floor plan with typical bay sizes called out



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.

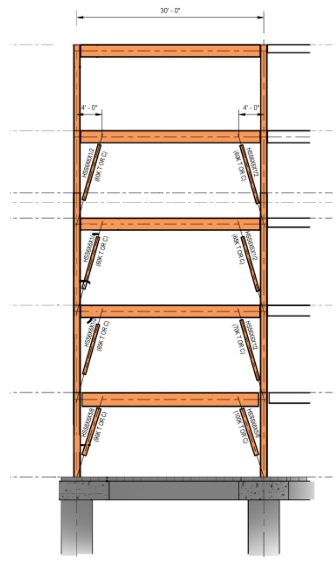


Figure 8: *(above)*
Modified AES braced frame elevation

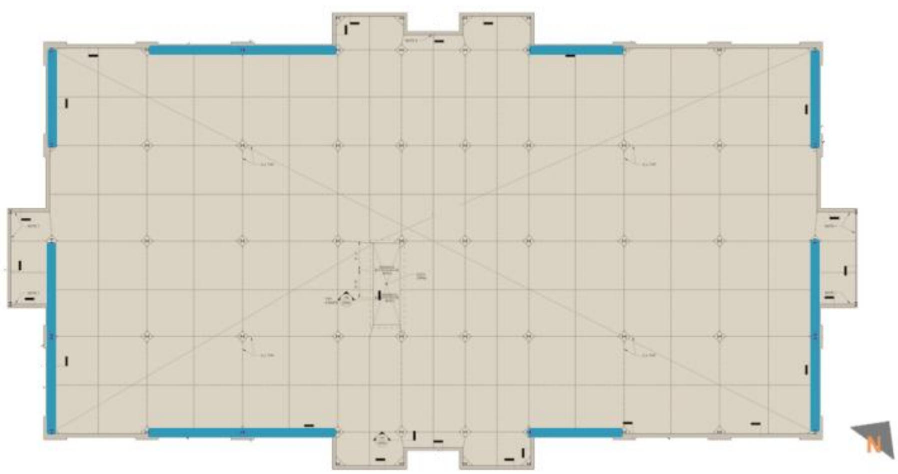


Figure 9: *(left)*
Modified AES floor plan with locations of braced frames highlighted in pink

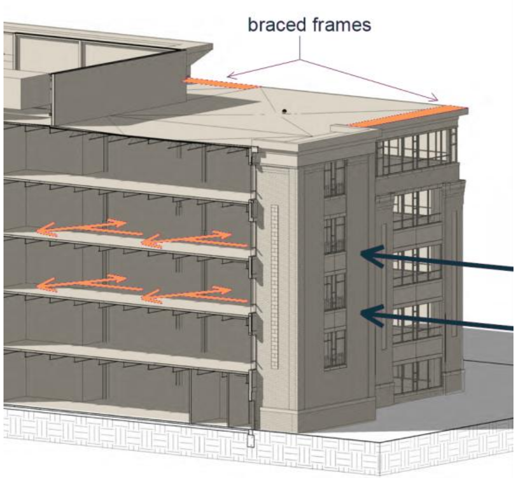


Figure 10: *(above)*
Modified Kernick Architecture building section showing lateral load path

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.

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)

load summary

Gravity loads for live, dead, flat roof snow, and drift snow are found using ASCE 7-10 code and estimations. Tables are included tabulating the values of the load in each corresponding category. Lateral loads are also calculated using ASCE 7-10.

included in this section:
dead load
live load
snow load
gravity spot checks
wind load
seismic load

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

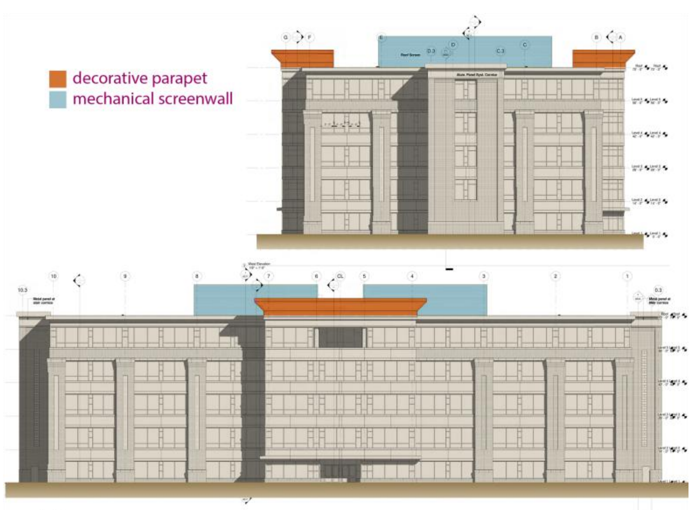


Figure 14: *(above)*
Modified Kernick Architecture elevations showing the parapet and screenwall that cause snow drift

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.

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.

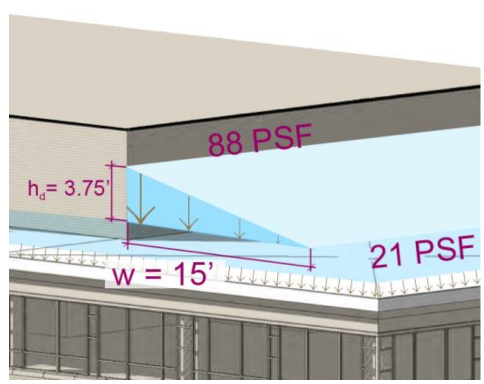


Figure 15: *(above)*
Drift load sketch

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 B. 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.

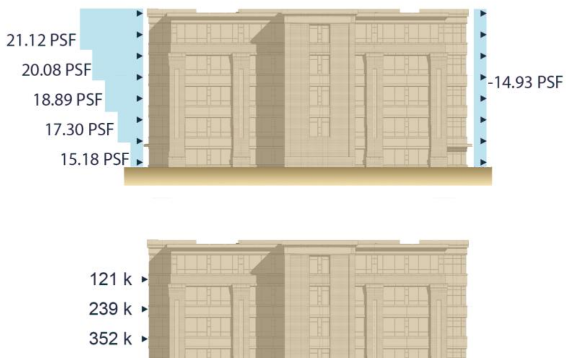
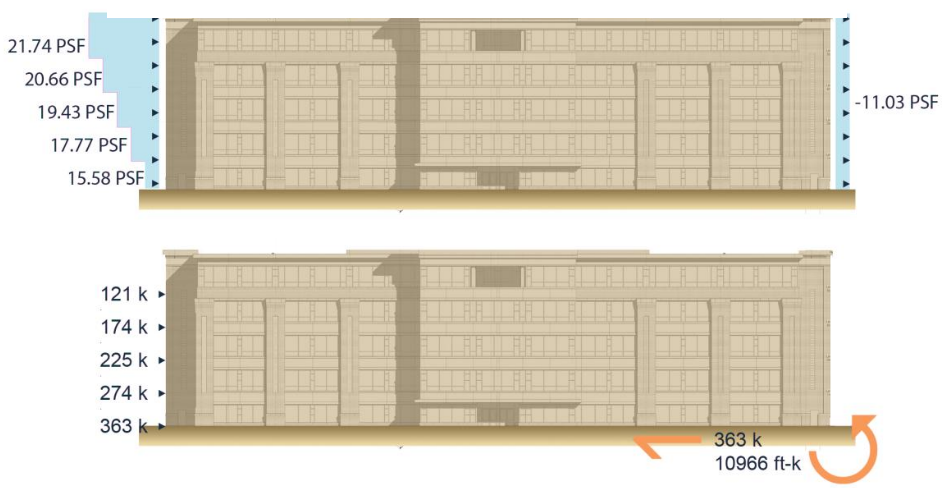


Figure 16: *(above)*
North-South wind load pressures, story shears, base shear, and overturning moment

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.

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.

Figure 17: *(below)*
East-West wind load pressures, story shears, base shear, and overturning moment



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 C.

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	13.895	195
3	28	40.80251	2603.5	0.1684	30.054	842
4	42	64.07321	2603.5	0.2645	47.195	1982
5	56	88.25377	2603.5	0.3643	65.006	3640
roof	70	113.1343	697	0.1250	22.309	1562
		$\Sigma w_i h_i^k$:	630780.4		base shear [k]:	178
					total overturning moment [ft-k]:	8220

Figure 18: *(above)*
Summary of seismic forces

RAM model

RAM Structural Systems is chosen as the structural modeling program for MTOB. The program was introduced at the end of the author's Computer Modeling course, and further studied at a summer internship. As mentioned previously in this report, the building is framed with structural steel and has no shear walls. Because of this, no wall meshing had to be considered. Instead, concentrically braced frames are placed in the appropriate locations. The offset distances of each brace was modified for each frame type to ensure accuracy.

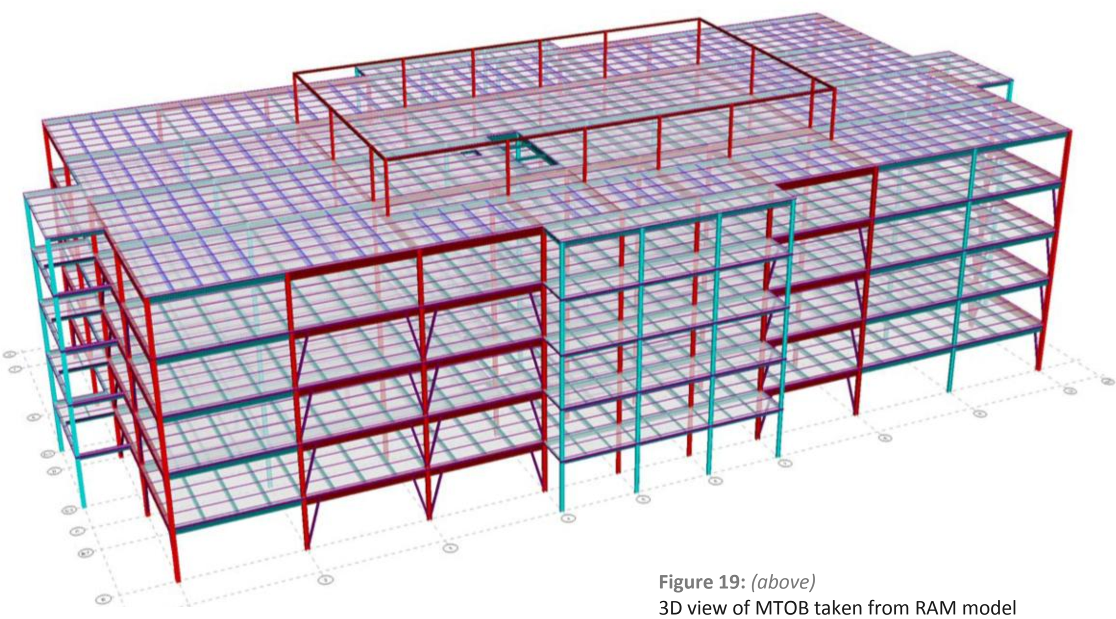
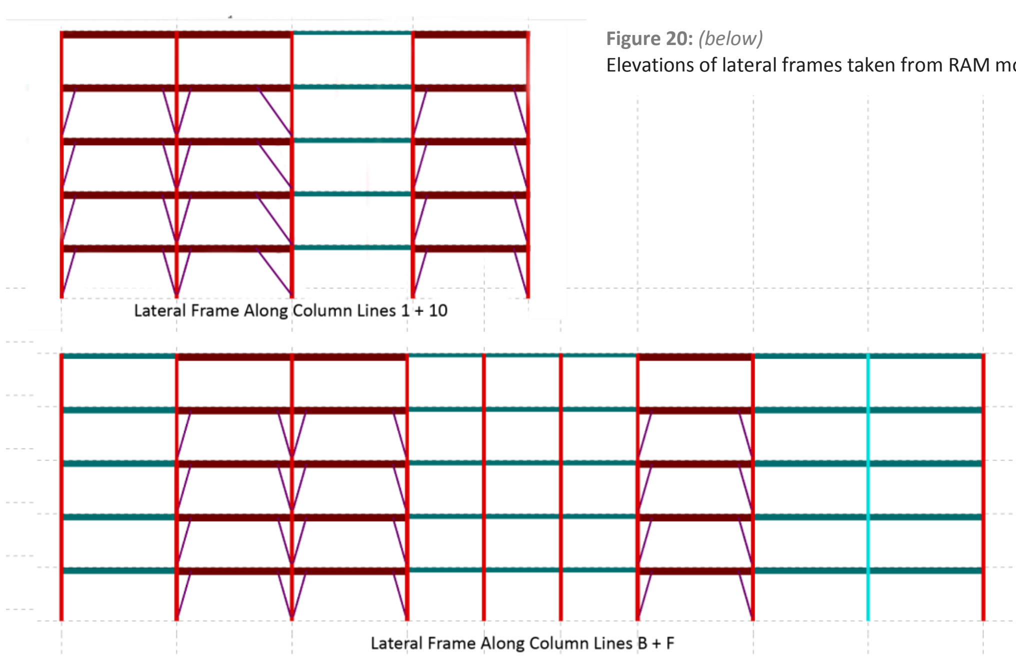
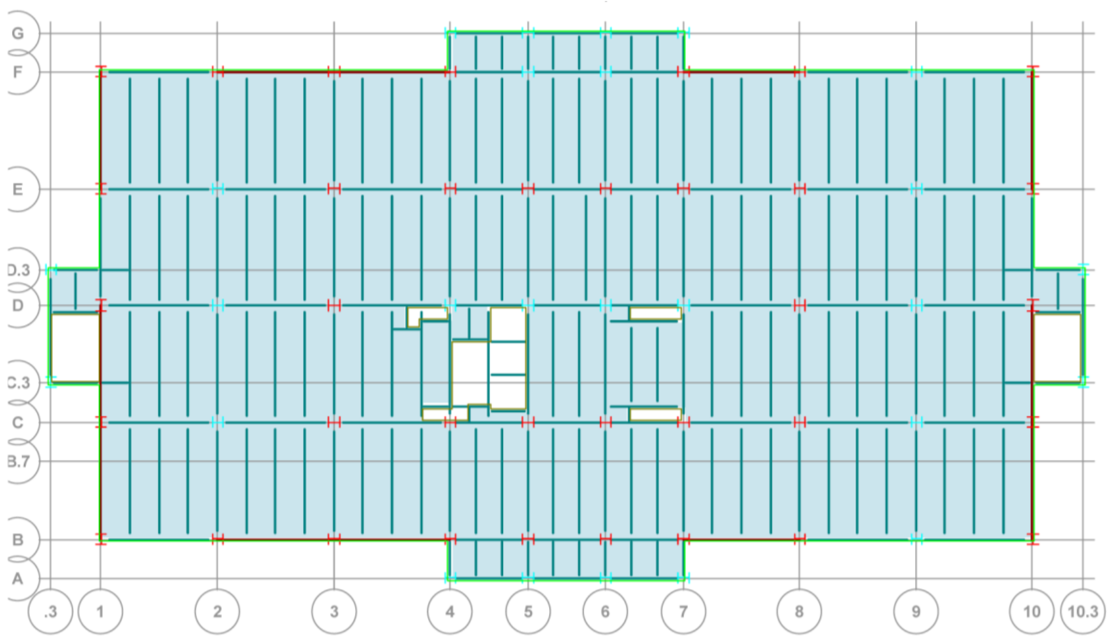


Figure 19: *(above)*
3D view of MTOB taken from RAM model

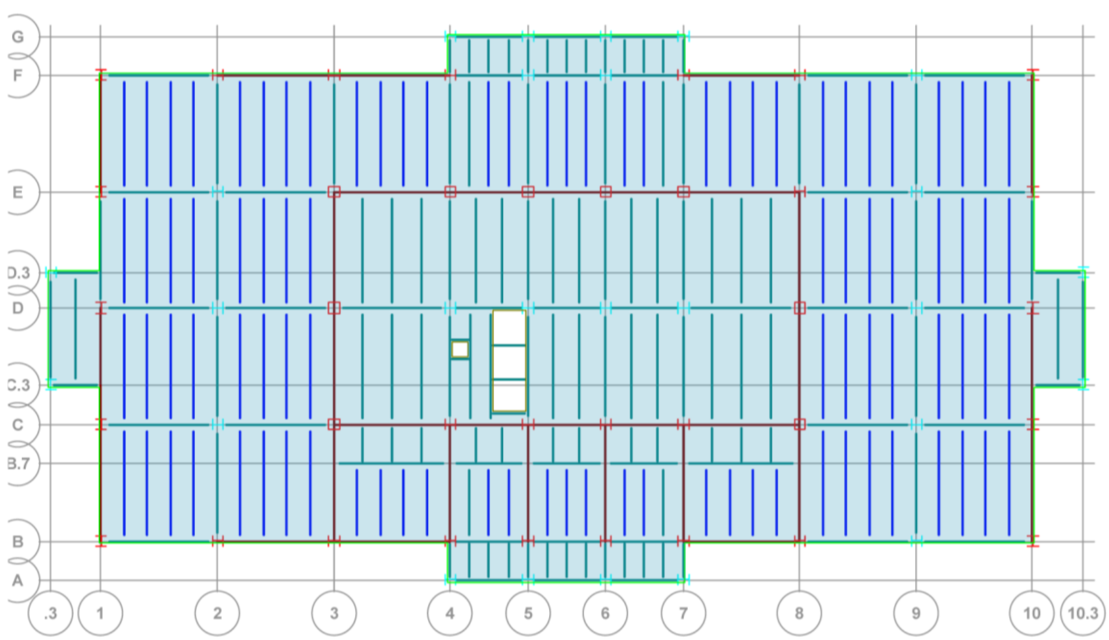
Figure 20: *(below)*
Elevations of lateral frames taken from RAM model





RAM Floor Plan, Levels 2 to 5 [steel beam framing]

Figure 21: *(above)*
Typical floor plan taken from RAM model



RAM Roof Plan [steel beam and joist framing]

Figure 22: *(above)*
Roof plan taken from RAM model

lateral system analysis

The lateral system analysis is completed using information gathered through the lateral load calculations and through the RAM structural model of MTOB.

included in this section:

load cases
building properties
+ stiffness
+ center of rigidity
+ center of mass
distribution of lateral forces

load cases

RAM Structural System generates its own load cases based on options selected. This model uses ASCE 7-05 with Allowable Stress Design. The following section is taken directly from ASCE 7-05 to display the load combinations:

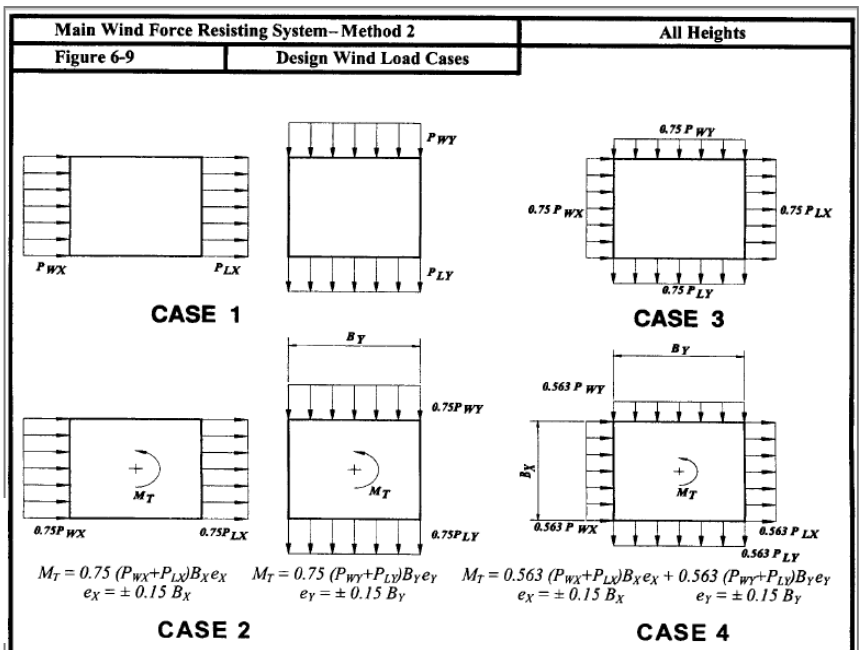
2.4.1 Basic Combinations. Loads listed herein shall be considered to act in the following combinations; whichever produces the most unfavorable effect in the building, foundation, or structural member being considered. Effects of one or more loads not acting shall be considered.

1. $D + F$
2. $D + H + F + L + T$
3. $D + H + F + (L_r \text{ or } S \text{ or } R)$
4. $D + H + F + 0.75(L + T) + 0.75(L_r \text{ or } S \text{ or } R)$
5. $D + H + F + (W \text{ or } 0.7E)$
6. $D + H + F + 0.75(W \text{ or } 0.7E) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
7. $0.6D + W + H$
8. $0.6D + 0.7E + H$

Figure 23: *(left)*
Load combinations taken from ASCE 7-05

Because of the symmetry of MTOB and its lateral system layout, several of the load combinations can be eliminated. In addition, the wind load was found to control over the seismic load in this region. This lets us eliminate seismic cases and just look at wind for the hand calculations portion.

In wind design, four load cases are considered from ASCE 7-05. In examining each case, it is found that case 1 controls for MTOB, so this is the case that modeled in RAM.



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

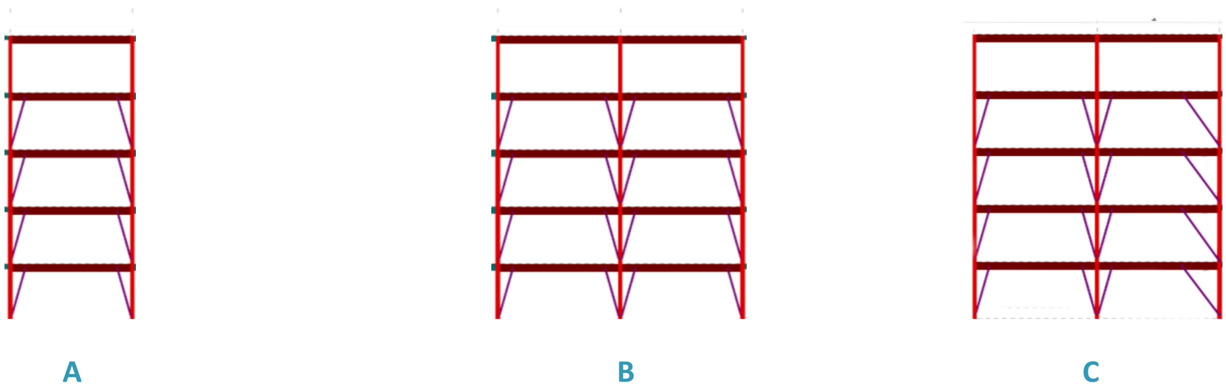
Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Figure 24: *(right)*
Wind loading cases, taken from ASCE 7-05

building properties

finding stiffness

Before the lateral analysis can continue, it is important that the respective stiffness for each frame is found. Stiffness is defined as the amount of force required to displace a member one unit length. To find the stiffness, the different types of braced frames were considered.



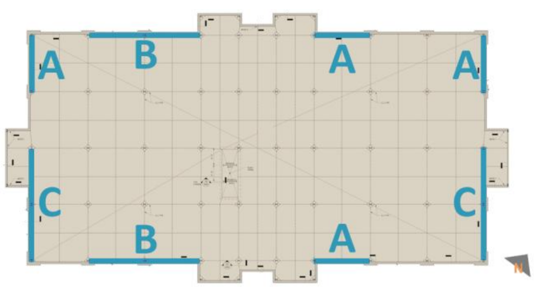


Figure 25: *(above)*
Elevations showing each type of lateral frame

Figure 26: *(left)*
Plan showing locations of lateral frame types

Notice that there are three different types of braced frames:

A. One-bay symmetrical (4' distance to brace)

B. Two-bay symmetrical (4' distance to brace)

C. Two-bay asymmetrical (4' and 10' distances to brace)

Each of these braces will have a different stiffness. To find the respective stiffness of each frame, they are all separately modeled in STAAD, including the member sizes and connections. Next, a unit load of one kip is applied at the top left corner of each frame. The three structures are then analyzed in STAAD to find the displacement at the tops of the frames. Since K = P/Δ, by taking the inverse of the displacement we can find the stiffness. The results came out that type "1" was the least stiff, at 20 k/in, followed by type "2" at 40 k/in. Type "3" has the largest stiffness at 52.6 k/in. These results are expected, since two bays are stiffer than one, and the smaller the "gap" in the center of the beam, the stiffer the frame becomes.

center of rigidity + center of mass

The centers of rigidity and mass are often very close together, but they represent different ideas. The center of rigidity represents the point at which forces may be applied that would cause no torsion. A building's center of mass is exactly as it seems; the central location of the mass of the building (in plan). Mass and plan layout can vary from level to level, so the center of mass on one floor may not necessarily be the same on an adjacent floor. In the case of MTOB, the building's uniform layout allows the centers of mass and rigidity to be in the same place on every level.

Because of the symmetrical layout of the braced frames (both in geometry and in stiffness), the center of rigidity is calculated as exactly in the center of the plan. In addition, since there are no shear walls or other massive features to unbalance the floor slabs and exterior wall weights, the center of mass is assumed to be in the center of the plan. These hand calculated values are compared with the computer model values found in RAM. The difference in center of mass can be explained through RAM's more precise calculation which includes beam and column weights. In looking at the actual values, they differ very slightly from the hand calculated values. The differences are negligible, which will be explained further in the distribution of lateral forces section. Figure 27 illustrates the slight differences found between the hand calculated values and the RAM models.

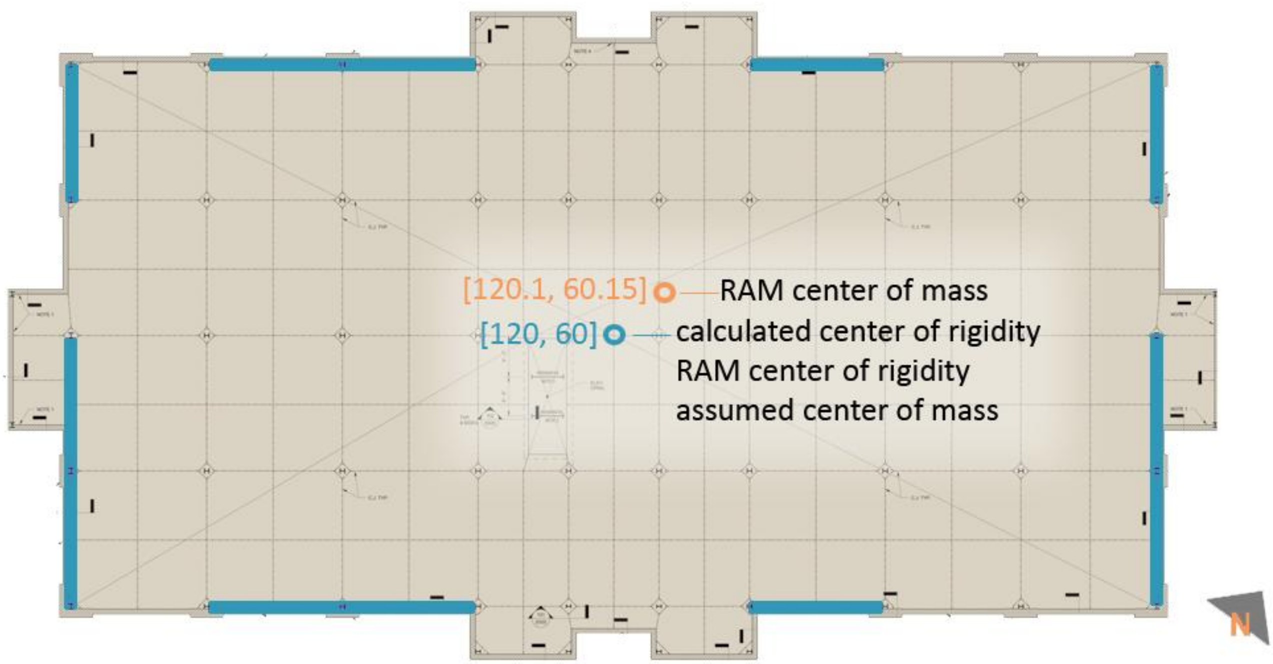


Figure 27: *(above)*
Plan showing locations of hand calculated and RAM calculated centers of mass and rigidity

lateral system analysis

distribution of lateral forces

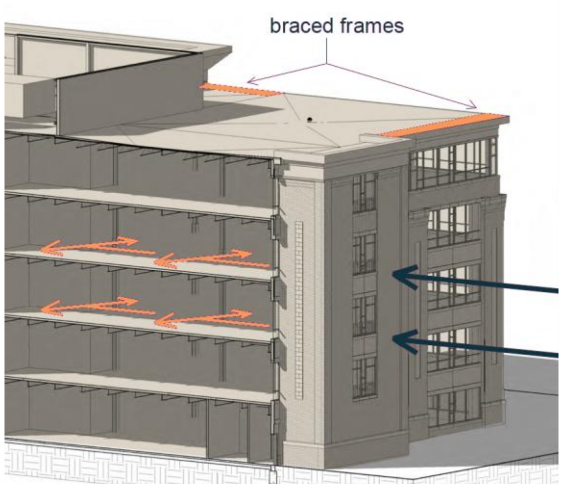


Figure 28: *(above)*
3D view showing lateral force distribution

Lateral forces are applied at the exterior components and cladding. The loads travel through the relative floor slabs, eventually finding one of the eight braced frames in MTOB.

The forces are distributed to the frames based on relative stiffnesses and the location of the frame relative to eccentricity. In a building with a large eccentricity, the torsional shear may add a significant amount of shear to the direct shear. It also may be a subtractive force in some of the frames, depending on the direction of the loading.

It was stated earlier that the torsional effects of the building may be neglected because the eccentricity was so small. The tables below display this idea. Direct shear and torsional shear are calculated for all frames. Notice that the torsional shear at most adds 0.04 k to any one direct shear. This does not change any end results, so it did not have to be calculated.

[wind case 1]	N | S load distribution									
frame	stiffness [k/in]	height story [ft]	lateral force [k]	e_x	e_y	d	kd^2	direct shear [k]	torsional shear [k]	total shear [k]
1 double	52.6	14	149.43	0.1	0.15	-120.1	758703	54.132	-0.0373	54.0950
1 single	20	14	149.43	0.1	0.15	-120.1	288480	20.583	-0.0142	20.5684
10 double	52.6	14	149.43	0.1	0.15	120.1	758703	54.132	0.0373	54.1697
10 single	20	14	149.43	0.1	0.15	120.1	288480	20.583	0.0142	20.5968
B double	40	14	149.43	0.1	0.15	-60.15	144721	0	-0.0142	-0.0142
B single	20	14	149.43	0.1	0.15	-60.15	72360	0	-0.0071	-0.0071
F double	40	14	149.43	0.1	0.15	60.15	144721	0	0.0142	0.0142
F single	20	14	149.43	0.1	0.15	60.15	72360	0	0.0071	0.0071
						$\Sigma K*d^2$	2528529			

[wind case 1]	E | W load distribution									
frame	stiffness [k/in]	height story [ft]	lateral force [k]	e_x	e_y	d	kd^2	direct shear [k]	torsional shear [k]	total shear [k]
1 double	52.6	14	71.4	0.1	0.15	-120.1	758703	0	-0.0268	-0.0268
1 single	20	14	71.4	0.1	0.15	-120.1	288480	0	-0.0102	-0.0102
10 double	52.6	14	71.4	0.1	0.15	120.1	758703	0	0.0268	0.0268
10 single	20	14	71.4	0.1	0.15	120.1	288480	0	0.0102	0.0102
B double	40	14	71.4	0.1	0.15	-60.15	144721	23.800	-0.0102	23.7898
B single	20	14	71.4	0.1	0.15	-60.15	72360	11.900	-0.0051	11.8949
F double	40	14	71.4	0.1	0.15	60.15	144721	23.800	0.0102	23.8102
F single	20	14	71.4	0.1	0.15	60.15	72360	11.900	0.0051	11.9051
						$\Sigma K*d^2$	2528529			

Figure 29-30: *(above)*
Tables showing the tablulated values of direct shear, torsional shear, and total shear for both N|S and E|W directional loading for wind case 1 loading

results

This section is to provide the results from the lateral analysis using both the computer generated solutions and hand calculated solutions.

included in this section:

torsional irregularity check
building period
lateral members spot check
drift + displacement
overturning + impact on foundations

torsional irregularity

Torsional irregularity of MTOB is checked and ruled out with some simple hand calculations. These can be viewed in Appendix G.

period

A building's period is not linked to the loads that are applied to it during its design or lifespan. Instead, the period depends on the materials, connections, height of the building, and the mode being analyzed. This report only looks at the first three modes, or the X, Y, and Z directional modes (where Z is torsion).

$T_1 = 2.322$ s

$T_2 = 1.755$ s

$T_3 = 0.861$ s

This is comparable to the structural engineering firm's calculations of $T_1 = 2.479$ s, $T_2 = 1.989$ s, and $T_3 = 1.209$ s. The small discrepancy can be explained by small differences in modeling.

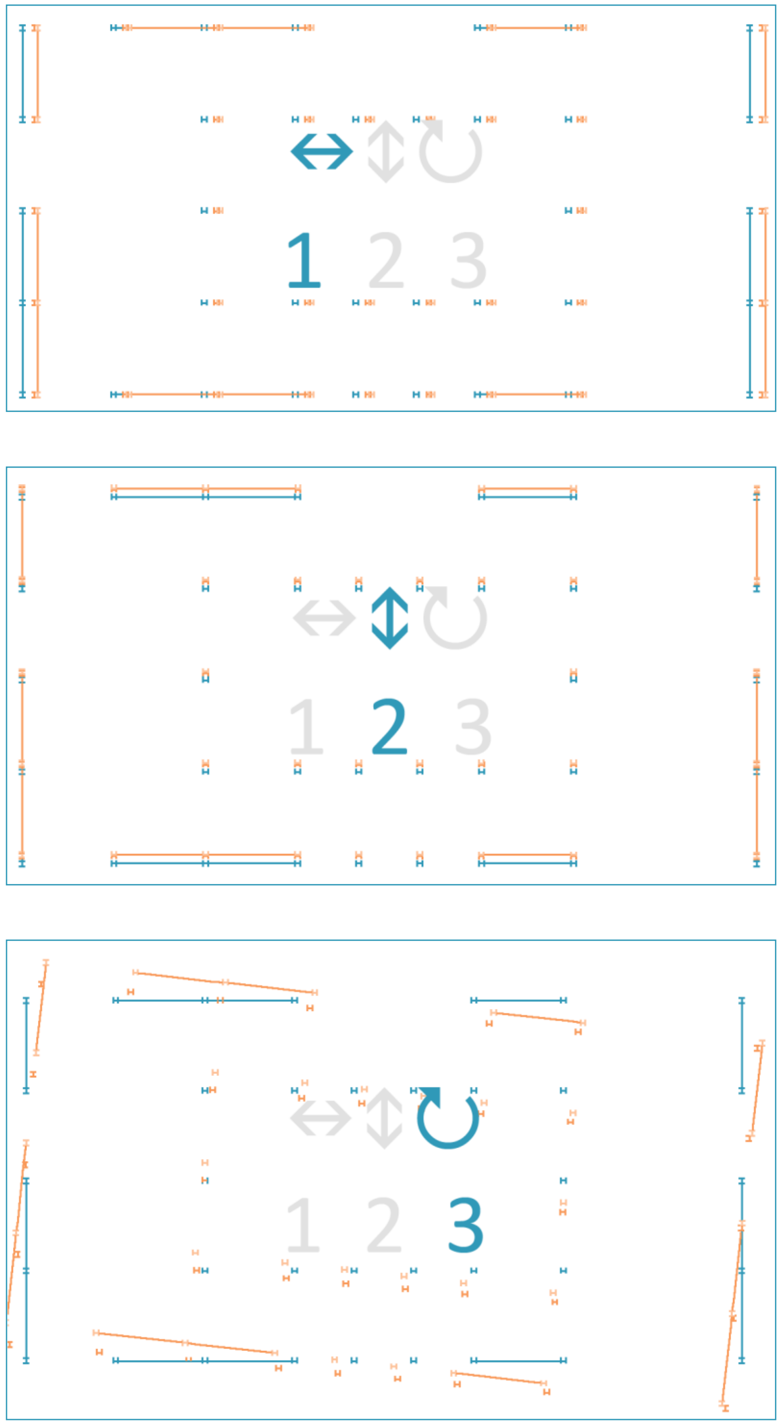


Figure 31: *(above)*
Plans showing movement for each of the three modes discussed

lateral spot check

A spot check is performed on one of the braced frames to confirm its adequacy for both gravity and lateral loads. A specific brace and column were chosen along column line B to check the adequacy. The brace was selected for its controlling axial load in relation to its neighboring braces. Actual forces and moments on the column and brace analyzed in this report are found using the RAM model created for this report. See figure 32 for the location of the actual member that is being analyzed. Full calculations can be found in Appendix F.

The brace is investigated for its axial load capacity, in both tension and compression. It is necessary to check both of these directions, even though the RAM model shows the member in tension. If the lateral load were to switch directions by 180°, the forces in braces would change from tension to compression, and vice versa. For tension checks, AISC (14th), table 5-5 is used to look at both yielding and rupture. Table 4-4 is used for compression checks. Brace B8 at story 2 is found to pass both tension and compression checks.

Column B8 is analyzed as part of the frame spot check. Because the column undergoes both gravity and lateral loading, it must be checked with both of these conditions applied. Therefore, AISC (14th), table 6-1 is used to check the column for combined flexural and axial force. M_1 and M_2 are obtained via the RAM model, using the worst case wind load (since seismic loading did not control in this area). Out-of-plane bending is excluded because it does not control in this lateral check. The check of column B8 showed that the size selected is both adequate and appropriate for the loading conditions.

Figure 32: *(above)*
Plan showing location of frame used in spot check

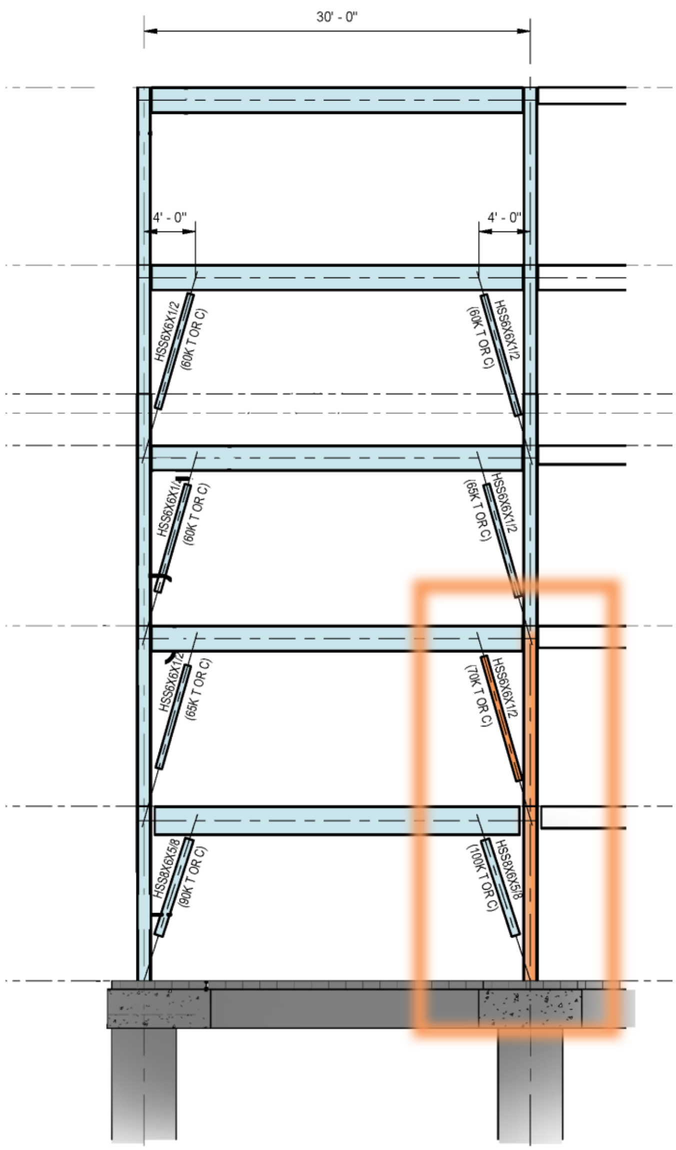


Figure 33: *(above)*
Elevation showing locations of brace and column analyzed in spot check

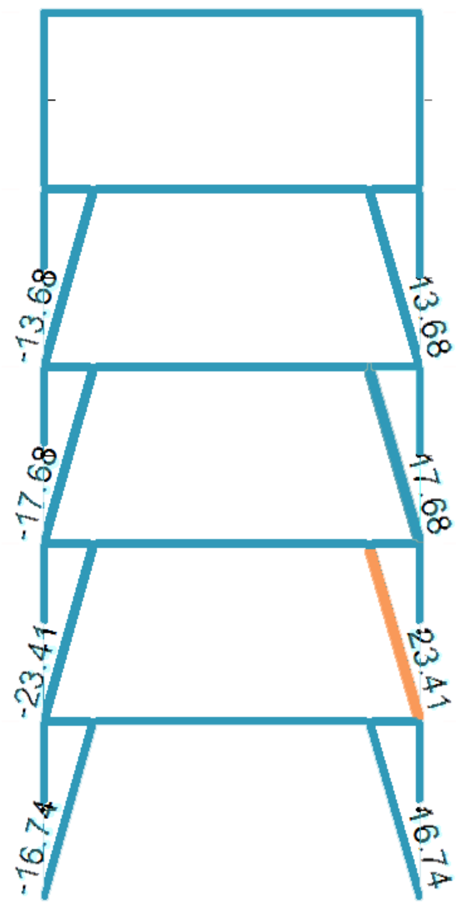


Figure 34: *(above)*
Elevation showing axial forces, taken from RAM model

drift + displacement

Inter-story drift and overall displacement are checked using the RAM model created for this technical report. Under ASCE 7-05, Table 12.12-1, allowable seismic story drift is $0.02h_{sx}$ for occupancy category II. For wind cases, allowable drift is taken as L/400. The tables below summarize the drift and displacement results for both wind and seismic found using the RAM computer model. All drift values are found to be within the code allowable values.

[wind]	N | S displacement + drift				
Story	Δx [in]	Δy [in]	drift x [in]	drift y [in]	allowable drift [in]
Story 5	0.0055	0.4369	0.0013	0.1055	0.72
Story 4	0.0042	0.3341	0.0016	0.0583	0.72
Story 3	0.0026	0.2758	0.0016	0.079	0.72
Story 2	0.001	0.1968	0.0004	0.1104	0.72
Story 1	0.0006	0.0864	0.0006	0.0864	0.72

[seismic]	N | S displacement + drift				
Story	Δx [in]	Δy [in]	drift x [in]	drift y [in]	allowable drift [in]
Story 5	0.0055	0.4369	0.0013	0.1055	3.36
Story 4	0.0042	0.3341	0.0016	0.0583	3.36
Story 3	0.0026	0.2758	0.0016	0.079	3.36
Story 2	0.001	0.1968	0.0004	0.1104	3.36
Story 1	0.0006	0.0864	0.0006	0.0864	3.36

[wind]	E | W displacement + drift				
Story	Δx [in]	Δy [in]	drift x [in]	drift y [in]	allowable drift [in]
Story 5	0.7779	0.0003	0.1676	0.0009	0.72
Story 4	0.6104	-0.0006	0.1227	0.0004	0.72
Story 3	0.4877	-0.001	0.1582	0.0001	0.72
Story 2	0.3295	-0.001	0.1951	-0.0003	0.72
Story 1	0.1344	-0.0007	0.1344	-0.0007	0.72

[seismic]	E | W displacement + drift				
Story	Δx [in]	Δy [in]	drift x [in]	drift y [in]	allowable drift [in]
Story 5	0.7779	0.0003	0.1676	0.0009	3.36
Story 4	0.6104	-0.0006	0.1227	0.0004	3.36
Story 3	0.4877	-0.001	0.1582	0.0001	3.36
Story 2	0.3295	-0.001	0.1951	-0.0003	3.36
Story 1	0.1344	-0.0007	0.1344	-0.0007	3.36

Figures 35-38: *(above)*
Tables showing summaries of inter-story drift

overturning + impact on foundations

Overturning moments need to be calculated in order to check for possible issues in uplift and foundations. The controlling case is used to determine possible overturning moment from lateral loads. As previously discussed in this technical report, case 1 for wind (in both N|S and E|W directions) controls over seismic. Resisting moments are found by multiplying the building weight (calculated in Appendix C) by half of the building length in the direction being analyzed. This value is then multiplied by 0.6 to match the controlling load combination. This reduces the resisting moment because although dead load is over estimated for strength purposes, the over-estimate becomes unconservative in this check.

It is found that overturning in both directions of case 1 wind are resisted by the building weight, so there is no expected impact on the foundations. A summary of these calculations can be seen in Figures 39 and 40 below.

wind case 1 [N|S direction]			
level	height [ft]	lateral force [k]	overturning moment [ft-k]
2	14	149.43	2092
3	28	149.43	4184
4	42	149.43	6276
5	56	149.43	8368
roof	70	149.43	10460
total overturning moment [ft-k]:			31,380
resisting moment N|S [ft-k]:			933,120

wind case 1 [E|W direction]			
level	height [ft]	lateral force [k]	overturning moment [ft-k]
2	14	71.40	1000
3	28	71.40	1999
4	42	71.40	2999
5	56	71.40	3998
roof	70	71.40	4998
total overturning moment [ft-k]:			14,994
resisting moment E|W [ft-k]:			466,560

Figures 39-40: *(above)*
Tables showing calculated overturning moments for both N|S and E|W lateral forces

conclusion

An in-depth lateral analysis is completed with the aid of computer modeling. Hand checks verify that the model is accurate to the structure. In the analysis of the lateral system, it is concluded that the existing braced frames configuration is adequate to resist code-specified seismic and wind loads with an appropriate margin for safety.

To aid in the lateral analysis, a computer model is created using RAM Structural System software from Bentley. The software is chosen because of the author's familiarity with it, both through graduate level course work and professional work experience.

Hand calculations are used for two main purposes: to verify the accuracy of the RAM model, and to do lateral member spot checks. The RAM model analysis corresponds with the hand calculated values, meaning that the model is true to the building structure. Two spot checks are completed: one on a lateral brace, and one on a lateral column. It is found that the lateral brace passes both compressive and tensile axial forces, and that the column passes the tests with combined flexural and axial forces applied.

Several other categories are presented and discussed in the results section of this report. Torsional irregularity is checked and ruled out. Drift and displacement is found to be within code limits. Overturning moments are found to have no impact on foundations with the controlling load cases.

Overall, the existing lateral system of MTOB is found to be both adequate and appropriate for the building type and location. Wind and seismic loads are accounted for in the designs with margins of safety.

appendices

The appendices are to provide further detail in all the hand calculations, computer model aided calculations, and building details.

included in this section:

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)

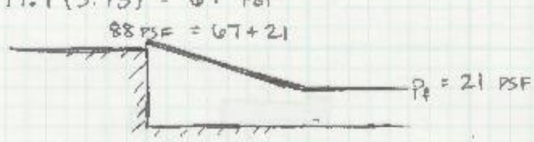
$l_{u,lw} = 120' \Rightarrow h_D = 3.75'$ CONTROLS (UPPER ROOF, LW)

$h_c = 14' \geq 3.75'$

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

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

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



appendices

appendix B: 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

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) $\frac{L}{B} = \frac{120}{240} = \frac{1}{2} \Rightarrow -0.5$ } FIG 27.4-1

E/W $\frac{L}{B} = \frac{240}{120} = 2 \Rightarrow -0.3$

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

$T_a = C_t h_n^x$

$C_t = 0.03$ } T 12.8-2

$x = 0.75$

$h = 70$ FT

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

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

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

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)

$h = 70\text{FT}$

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

$\ell = 320$ 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: 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
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
} geohazards.usgs.gov/designmaps/us

$T = C_t h_n^x = 0.726\,s$ (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
$S_s = 0.108\,g < 0.25$
$= 1.2(0.108) = 0.1296 \sim 0.129$ OK

$S_{M1} = F_v S_1$ EQN 11.4-2
$F_v = 1.7$ T 11.4-2
$S_1 = 0.053\,g$
$= (1.7)(0.053) = 0.0901 \sim 0.090$ OK

$S_{DS} = 2/3\, S_{MS} = 0.0864$ OK EQN 11.4-3
$S_{D1} = 2/3\, S_{M1} = 0.06007$ 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

appendices

SEISMIC LOAD (ASCE 7-10) 2/2

$0.726s = T < T_L = 12s \Rightarrow C_s = \frac{S_{D1}}{T(\frac{R}{I})}, \quad C_s = \frac{S_{DS}}{R/I}$ 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

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

k:

T	k
0.5	1
0.726	1.113
2.5	2

$1.113 \leftarrow = \left(\frac{2-1}{2.5-0.5}\right)(0.726-0.5)+1$

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

$\sum w_i h_i^k = 630780.4$

$= 0.125$

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

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

appendices

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'

⇒ 36 PLF / 6.67' = 5.4 PLF

STEEL GIRDERS — 2.27 PSF

30' SPACES

W24×68

⇒ 68 PLF / 30' = 2.27 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

79×14×62 = 68572 # PER FLOOR

68572 / (120×240) = 2.4 PSF

▶ TOTAL SELF WT PER TYP FLOOR:

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

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

mech + misc; EXT WALL
(67 PSF + 15 PSF)(120)(240) + 241.9 = 2603.5 K

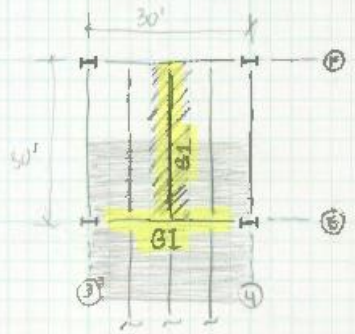
- FLOOR 1

SOG 4" NWC
150 PCF × 4"/12 = 50 PSF
(50 PSF + 10 PSF)(240)(120) = 1728 K + 241.9/2 = 1849 k
misc

TOTAL BLDG WT:
ROOF + 1st + (2 to 5)
697 + 1849 + (4)(2603.5) = 12960 K

appendix D: gravity spot checks

GRAVITY SPOT CHECKS 1/5



\#STUDS

B1 : W16x36 [18]

G1 : W24x68 [26]

- TYP LOADING:

SLAB	57PSF
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+0.75L+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+0.75L+0.75(0.6W)+0.75(L_R OR S OR R)
D+0.75L+0.75(0.7E)+0.75S
0.6D+0.6W
0.6D+0.7E

appendices

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)

$b_{eff} = \min \begin{vmatrix} span/8 \\ \tfrac{1}{2}\, d \text{ to adj BEAM} \end{vmatrix} + \min \begin{vmatrix} span/8 \\ \tfrac{1}{2}\, d \text{ to adj BM} \end{vmatrix}$

$= \min \begin{vmatrix} 30'/8 = 3.75 \\ \tfrac{1}{2}(7.5) = 3.75 \end{vmatrix} + 3.75$

$= 7.5' = 90''$

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

$Q_n : \min \begin{vmatrix} 0.5 A_{sc}\sqrt{f'_c E_c} & = 26.1\text{ K} \\ R_g R_p A_{sc} F_u & = 17.2\text{ K} \end{vmatrix}$ (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 ⇒ 16 STUDS

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

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

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

$y_2 = 6"$

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

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

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

- CHECK TOTAL LOAD DEFLECTION

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

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

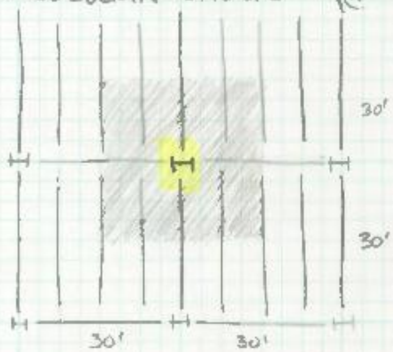
$$\Delta_{TL MAX} = \frac{L}{240} = \frac{30'}{240} = 1.5" > 0.842" \text{ GOOD}$$

∴ W16×36 w/16 STUDS OK ⇒ W16×36 w/18 STUDS OK

GRAVITY SPOT CHECK 5/5

• COLUMN CHECK (@ E-3, BASE LEVEL)

W12×79



• LL REDUCTION

$A_T = 30 \times 30 = 900$ SF } > 400 SF GOOD

$K_{LL} = 4$ T4-2

$L = L_o\left[0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right] = 80\left[0.25 + \frac{15}{\sqrt{900 \times 4}}\right] = 40 \geq 0.4 L_o$ 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/C 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' ⇒ P_n/Ω = 556K > 457.2K GOOD

↑ T4-1

appendices

appendix E: center of rigidity + mass

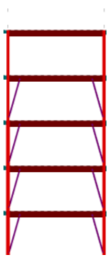
center of rigidity

each type of braced frame is modeled in STAAD with a unit load of 1 kip at the upper left hand corner. The displacement is found at the upper right hand corner. In taking the inverse of the displacement, stiffness is found for each type of frame.

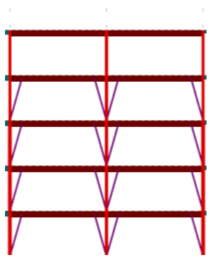
A) 1-bay symmetric $\Delta = 0.05''$ $K_A = 20$ k/in

B) 2-bay symmetric $\Delta = 0.025''$ $K_B = 40$ k/in

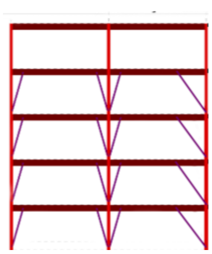
C) 2-bay asymmetric $\Delta = 0.019''$ $K_C = 52.6$ k/in



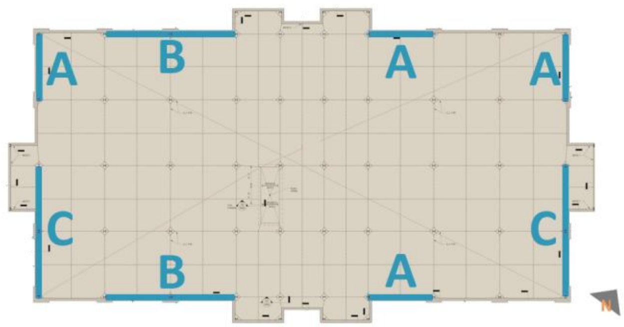
A



B



C



$x_r = \sum[kx_i/(\sum k_x)] = 0 + 0 + 20(240)/(20+20+52.6+52.6) + 52.6(240)/(20+20+52.6+52.6) = 120$ ft

$y_r = \sum[ky_j/(\sum k_y)] = 0 + 0 + 20(120)/(20+20+40+40) + 40(120)/(20+20+40+40) = 60$ ft

$C_R = (120 \text{ ft}, 60 \text{ ft})$

center of mass

Assumed in center due to symmetry of building shape, lateral framing layout, and material layout.

$C_M = (120 \text{ ft}, 60 \text{ ft})$

appendix F: lateral spot check

brace HSS6x6x1/2

L = 14.56’

From RAM model:

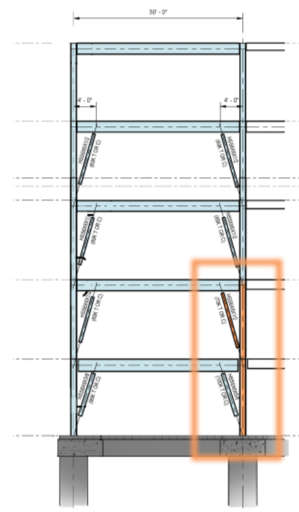
P = 23.6 k [N|S wind]

AISC, 14th Edition, T4-4, p. 4-58:

ϕP_n = 173 k @ KL = 15’

173 k > 23.6 k GOOD

∴ brace meets requirements for compression



AISC, 14th Edition, T5-5, p. 5-37:

$\phi P_{n,\ yield}$ = 268 k

$\phi P_{n,\ rupture}$ = 212 k

268 k > 23.6 k GOOD
212 k > 23.6 k GOOD

∴ brace meets requirements for tension

column B8: W12x152

L = 28’

Unbraced length: 14’

From RAM model:

P = -99.78 k

M_1 = 2.48 k-in [N|S wind]

M_{mid} = 0.67 k-in [N|S wind]

M_2 = 21.73 k-in [N|S wind]

AISC, 14th Edition, T6-1, p. 6-77:

$p \times 10^3 = .915$

$b_x \times 10^3 = .1.49$

$pP_r + b_xM_{rx} + b_yM_{ry} \leq 1.0$

$(0.915 \times 10^{-3})(99.78k) + (1.49 \times 10^{-3})(21.73k\text{-}in)(1/12\ ft/in) = 0.639$

0.639 ≤ 1.0 GOOD

∴ col B8 meets requirements for combined gravity and lateral loading

appendix G: torsional irregularity

δ_1 = displacement at point 1
δ_2 = displacement at point 2

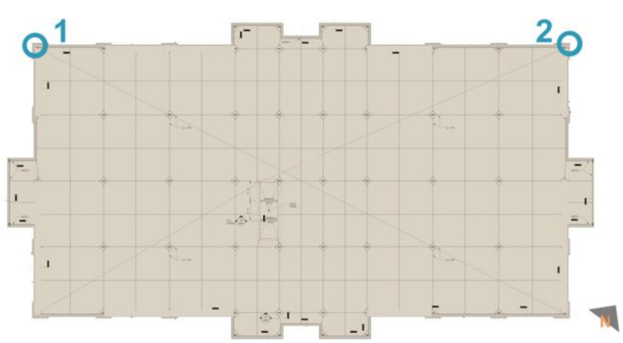
to avoid torsional irregularity:
$\delta_2 \leq 0.6(\delta_1 + \delta_2)$

N|S wind case 1

δ_1 = 0.4619"
δ_2 = 0.4369"
0.6(0.4619 + 0.4369) = 0.53928"
0.4369" ≤ 0.5393" GOOD
∴ NO torsional irregularity in this direction

E|W wind case 1

δ_1 = -0.0003"
δ_2 = 0.0003"
0.6(-0.0003 + 0.0003) = 0.0"
0.0003" ≤ 0.0" GOOD
∴ NO torsional irregularity in this direction



appendices

appendix H: RAM output

drift (at north east point)

Drift

RAM Frame v14.04.07.00
DataBase: MTOB 1.1 with frame analysis 11/07/12 17:36:28
Building Code: IBC Steel Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: No
Ground Level: Base

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
E1	Wind	EQ_IBC09_X_+E_F
E2	Wind	EQ_IBC09_X_-E_F
E3	Wind	EQ_IBC09_Y_+E_F
E4	Wind	EQ_IBC09_Y_-E_F
E5	Seismic	EQ_IBC09_X_+E_F
E6	Seismic	EQ_IBC09_X_-E_F
E7	Seismic	EQ_IBC09_Y_+E_F
E8	Seismic	EQ_IBC09_Y_-E_F

RESULTS:

Location (ft): (252.715, 129.447)

Story	LdC	Displacement		Story Drift		Drift Ratio	
		X	Y	X	Y	X	Y
		in	in	in	in		
TOP SCREEN	D	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Lp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Sp	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E1	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E2	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E3	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E4	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E5	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E6	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E7	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	E8	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
ROOF	D	-0.0007	-0.0165	-0.0002	-0.0050	0.0000	0.0000
	Lp	-0.0003	-0.0078	-0.0001	-0.0014	0.0000	0.0000
	Sp	-0.0004	-0.0008	-0.0001	-0.0019	0.0000	0.0000
	E1	0.7779	0.0003	0.1676	0.0009	0.0010	0.0000
	E2	0.7779	0.0003	0.1676	0.0009	0.0010	0.0000
	E3	0.0055	0.4396	0.0013	0.1055	0.0000	0.0006

Drift

RAM Frame v14.04.07.00 Page 2/3
DataBase: MTOB 1.1 with frame analysis 11/07/12 17:36:28
Building Code: IBC Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	E4	0.0055	0.4396	0.0013	0.1055	0.0000	0.0006
	E5	0.7779	0.0003	0.1676	0.0009	0.0010	0.0000
	E6	0.7779	0.0003	0.1676	0.0009	0.0010	0.0000
	E7	0.0055	0.4396	0.0013	0.1055	0.0000	0.0006
	E8	0.0055	0.4396	0.0013	0.1055	0.0000	0.0006
FLR 5	D	-0.0005	-0.0116	-0.0002	-0.0038	0.0000	0.0000
	Lp	-0.0002	-0.0064	-0.0001	-0.0020	0.0000	0.0000
	Sp	-0.0003	0.0012	-0.0001	0.0002	0.0000	0.0000
	E1	0.6104	-0.0006	0.1227	0.0004	0.0007	0.0000
	E2	0.6104	-0.0006	0.1227	0.0004	0.0007	0.0000
	E3	0.0042	0.3341	0.0016	0.0583	0.0000	0.0003
	E4	0.0042	0.3341	0.0016	0.0583	0.0000	0.0003
	E5	0.6104	-0.0006	0.1227	0.0004	0.0007	0.0000
	E6	0.6104	-0.0006	0.1227	0.0004	0.0007	0.0000
	E7	0.0042	0.3341	0.0016	0.0583	0.0000	0.0003
	E8	0.0042	0.3341	0.0016	0.0583	0.0000	0.0003
FLR 4	D	-0.0003	-0.0078	-0.0002	-0.0033	0.0000	0.0000
	Lp	-0.0001	-0.0044	-0.0001	-0.0019	0.0000	0.0000
	Sp	-0.0003	0.0010	-0.0001	0.0004	0.0000	0.0000
	E1	0.4877	-0.0010	0.1582	0.0001	0.0009	0.0000
	E2	0.4877	-0.0010	0.1582	0.0001	0.0009	0.0000
	E3	0.0026	0.2758	0.0016	0.0790	0.0000	0.0005
	E4	0.0026	0.2758	0.0016	0.0790	0.0000	0.0005
	E5	0.4877	-0.0010	0.1582	0.0001	0.0009	0.0000
	E6	0.4877	-0.0010	0.1582	0.0001	0.0009	0.0000
	E7	0.0026	0.2758	0.0016	0.0790	0.0000	0.0005
	E8	0.0026	0.2758	0.0016	0.0790	0.0000	0.0005
FLR 3	D	-0.0001	-0.0045	-0.0001	-0.0029	0.0000	0.0000
	Lp	-0.0001	-0.0025	-0.0000	-0.0016	0.0000	0.0000
	Sp	-0.0002	0.0005	-0.0001	0.0003	0.0000	0.0000
	E1	0.3295	-0.0010	0.1951	-0.0003	0.0012	0.0000
	E2	0.3295	-0.0010	0.1951	-0.0003	0.0012	0.0000
	E3	0.0010	0.1968	0.0004	0.1104	0.0000	0.0007
	E4	0.0010	0.1968	0.0004	0.1104	0.0000	0.0007
	E5	0.3295	-0.0010	0.1951	-0.0003	0.0012	0.0000
	E6	0.3295	-0.0010	0.1951	-0.0003	0.0012	0.0000
	E7	0.0010	0.1968	0.0004	0.1104	0.0000	0.0007
	E8	0.0010	0.1968	0.0004	0.1104	0.0000	0.0007
FLR 2	D	-0.0001	-0.0016	-0.0001	-0.0016	0.0000	0.0000
	Lp	-0.0000	-0.0009	-0.0000	-0.0009	0.0000	0.0000
	Sp	-0.0001	0.0002	-0.0001	0.0002	0.0000	0.0000

Drift

RAM Frame v14.04.07.00 Page 3/3
DataBase: MTOB 1.1 with frame analysis 11/07/12 17:36:28
Building Code: IBC Steel Code: IBC

Story	LdC	Displacement		Story Drift		Drift Ratio	
	E1	0.1344	-0.0007	0.1344	-0.0007	0.0008	0.0000
	E2	0.1344	-0.0007	0.1344	-0.0007	0.0008	0.0000
	E3	0.0006	0.0864	0.0006	0.0864	0.0000	0.0005
	E4	0.0006	0.0864	0.0006	0.0864	0.0000	0.0005
	E5	0.1344	-0.0007	0.1344	-0.0007	0.0008	0.0000
	E6	0.1344	-0.0007	0.1344	-0.0007	0.0008	0.0000
	E7	0.0006	0.0864	0.0006	0.0864	0.0000	0.0005
	E8	0.0006	0.0864	0.0006	0.0864	0.0000	0.0005

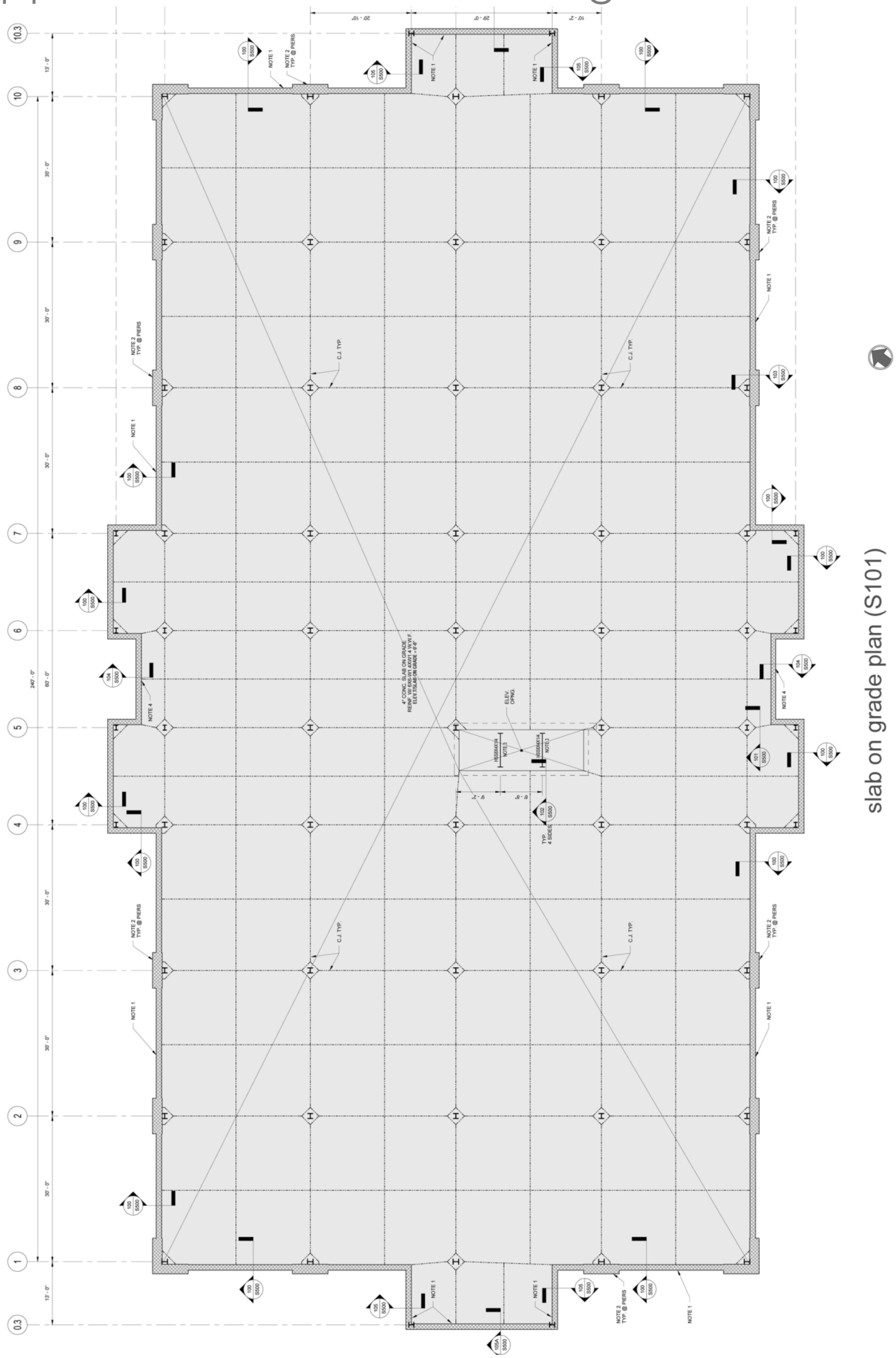
ASCE 7-05 Section 12.12.1

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a [a,b]

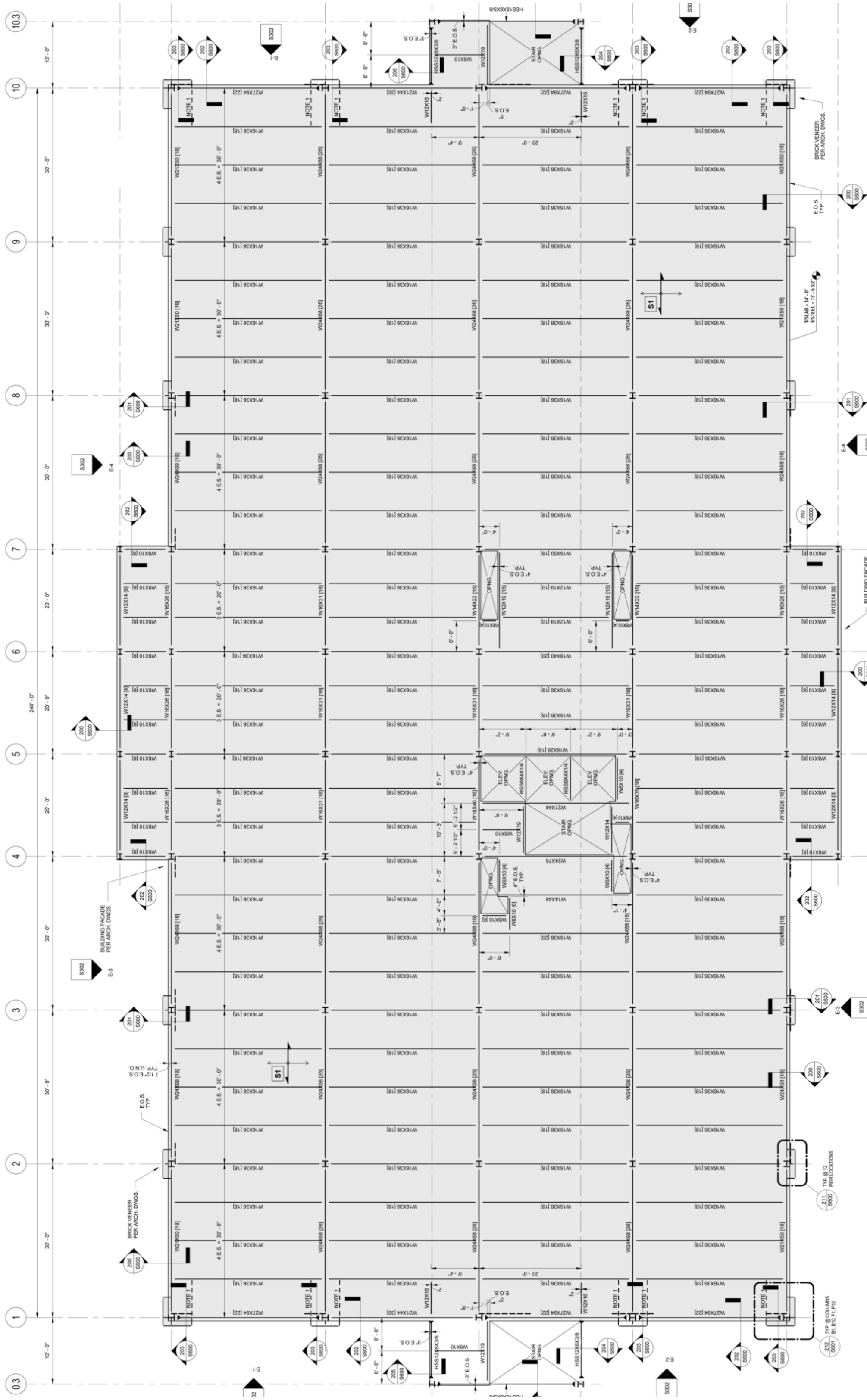
Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$ [c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

appendix I: additional drawings

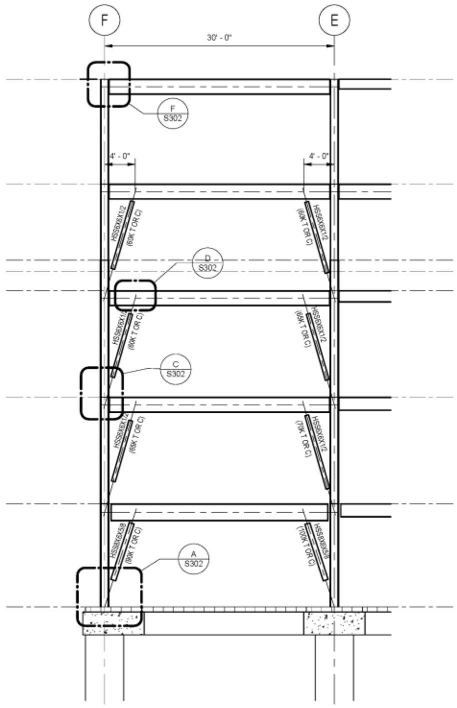


slab on grade plan (S101)



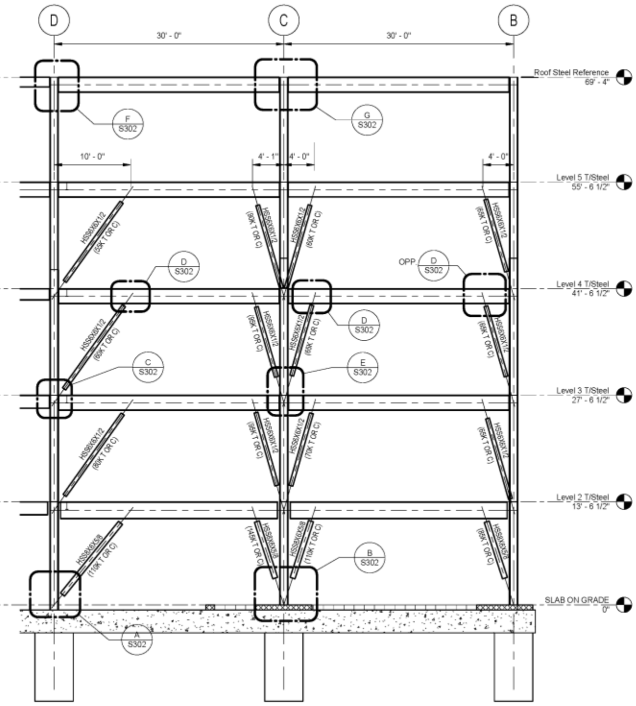
typical floor plan for floors 2-5 (S200)

appendices



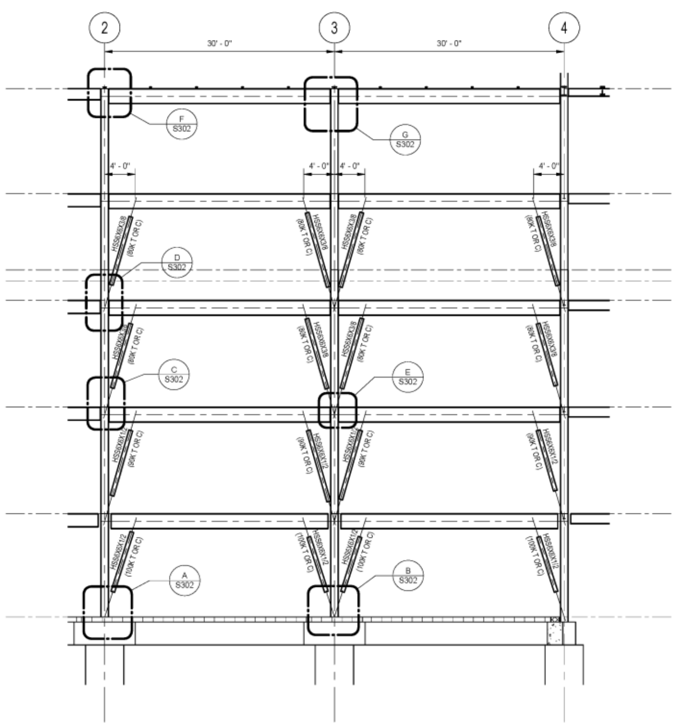
WIND BRACING ELEVATION
ALONG COLUMN LINES 1 &10
SCALE: 1/8" = 1'-0"
E-1
S302

REFER TO " BRACING REQUIREMENTS" PER GENERAL NOTES SHEET FOR DESIGN REQUIREMENTS



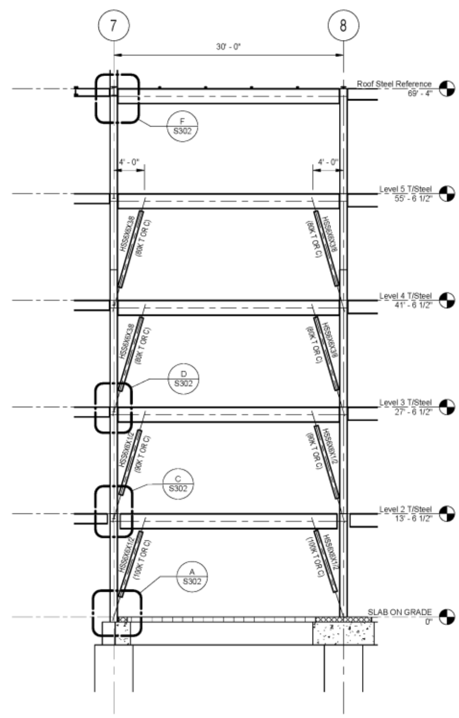
WIND BRACING ELEVATION
ALONG COLUMN LINES 1 & 10
SCALE: 1/8" = 1'-0"
E-2
S302

REFER TO " BRACING REQUIREMENTS" PER GENERAL NOTES SHEET FOR DESIGN REQUIREMENTS



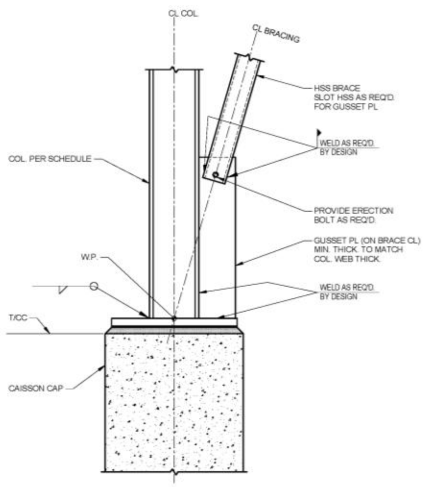
WIND BRACING ELEVATION
ALONG COLUMN LINES B & F
SCALE: 1/8" = 1'-0"
E-3
S302

REFER TO " BRACING REQUIREMENTS" PER GENERAL NOTES SHEET FOR DESIGN REQUIREMENTS

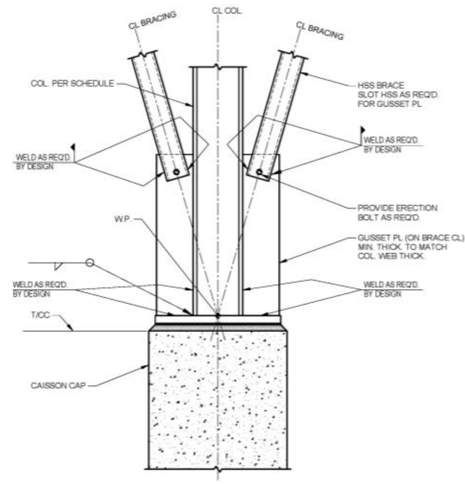


WIND BRACING ELEVATION
ALONG COLUMN LINES B &F
SCALE: 1/8" = 1'-0"
E-4
S302

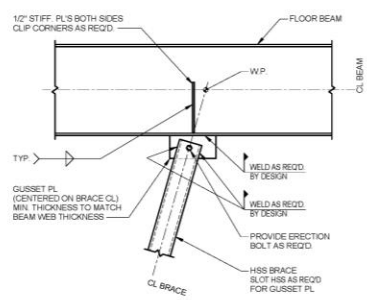
REFER TO " BRACING REQUIREMENTS" PER GENERAL NOTES SHEET FOR DESIGN REQUIREMENTS



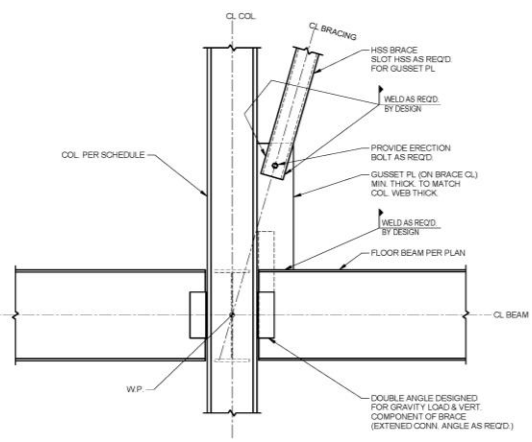
DETAIL A S302
SCALE: 3/4" = 1'-0"



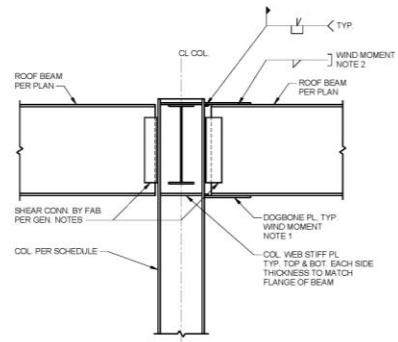
DETAIL B S302
SCALE: 3/4" = 1'-0"



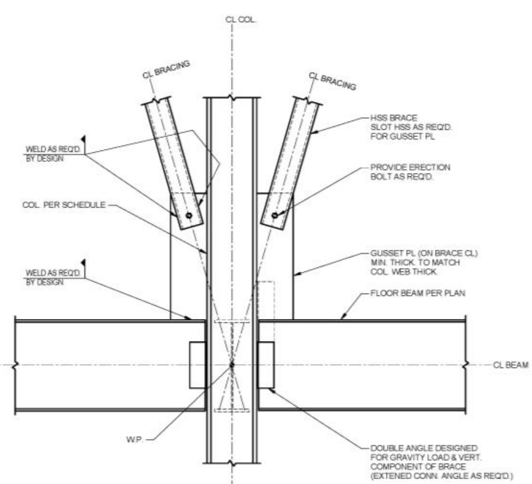
DETAIL D S302
SCALE: 3/4" = 1'-0"



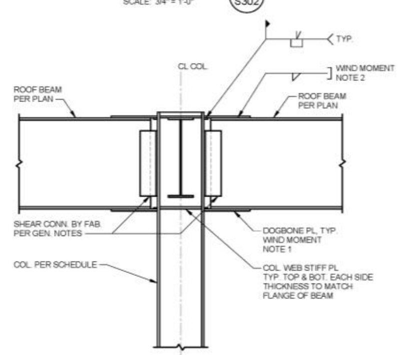
DETAIL C S302
SCALE: 3/4" = 1'-0"



DETAIL F S302
SCALE: 3/4" = 1'-0"



DETAIL E S302
SCALE: 3/4" = 1'-0"



DETAIL G S302
SCALE: 3/4" = 1'-0"

appendices