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Structural Option

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33 Harry Agganis Way

Boston, Mass

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Tech Report 1

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Executive Summary

The following thesis technical report summarizes the existing conditions and design concepts of 33 Harry Agganis Way. Structural Plans were provided by Weidlinger Associates Inc. All other plans, schedules and photos were provided by Cannon Design. The existing conditions were closely examined, and then analyzed using the most recent national codes and standards. This examination and analysis of each individual system was done to determine how they work together as one structural system to support the required loads.

ASCE7-10 was used to determine the loads on 33 Harry Agganis Way. A simplified building shape was used to determine the wind and seismic loads on the structure to allow for certain procedures from ASCE7-10 to be applied for this analysis. The wind analysis was done in both directions and produced base shear values of 2348 k and 5400 k in the North-South direction and East-West direction respectively. Overturning moments were found to be 329,600 ft-k and 784,200 ft in the NS and EW directions. The seismic forces on the structure were calculated to produce a base shear of 5381 k and an overturning moment 1,015,900 ft-k. The wind forces produce a higher base shear while the seismic forces produce a higher overturning moment. These two load cases do not control over one another due to the fact that they are two different scenarios and therefore both must be designed for.

Spot calculations of the existing structure were performed to check the adequacy of typical members in typical bays. Checks were done for the composite slab with metal deck, a composite beam and a girder all located on floor five. A typical base column was checked for adequate compressive axial strength at a given unbraced length. These spot checks confirmed that the structural components are adequate to carry the required loads.

Introduction

Located on the Boston University Campus, 33 Harry Agganis Way, which will be referred to as Res Tower II, is a 27 story, steel framed dormitory. It is located on the northwest corner of the John Hancock Student Village, bordered by the Charles River and Commonwealth Ave. Because two more dormitories are planned for the JH Student Village and the cost of developing in Boston is so high, the footprint of Res Tower II had to be as limited as possible, thus forcing the structure to be tall in nature.

The south tower is 19 stories tall with a fan room and mechanical penthouse above. A student activity space, with large windows and a terracotta walkout space, occupies the 27th story of the north tower. The roof of the north tower supports a fan room, large air handling units and other large service equipment. Floors 3 through 26, aside from the spaces mentioned above, are all private residential areas with some study rooms and computer labs mixed in. The first two levels of Res Tower II serve as the public and service offices for the rest of the building.

The façade of Res Tower II is a panelized skin comprised of terracotta and a metal panel rainscreen. This façade is a curtain wall system with its gravity load being supported by the floor above it; which can be assumed to be a continuous load due the small spacing of hung supports.

Res Tower II utilizes four main roof systems, all of which include gypsum under-laminate board, a vapor retarder and an adhered roofing membrane; the prior three aspects will be referred to as the typical roof assembly. Where mechanical equipment is being supported the typical roof assembly is placed on concrete deck while on the outer edges of the building, a metal deck is used. On the 26th story, to support the walkout space mentioned above, precast terracotta pavers on concrete deck are combined with the typical roof assembly to create an inviting yet durable roof system.

Structural Systems

Foundation

Haley & Aldrich performed the geotechnical studies for the JH Student Village area and provided the report in which H&A explain site and below-grade conditions along with recommendations for the structure. A net allowable soil bearing pressure of 6 kips per square foot (ksf) was recommended for the design of foundations on the naturally, undisturbed glacial deposits below the site. A recommended design groundwater level was also given which is on average 10-12’ below the bottom of the existing foundation.

Res Tower II utilizes a mat foundation system with two main thicknesses, 4’-3”and 3’-9”. Logically, the taller tower is supported using the deeper mat foundation to resist the higher loads transferred by the braced frames. The foundation step occurs between grid lines 9 and 10. The typical reinforcement in the east-west direction is #10’s spaced at 10” on center top and bottom while in the north-south direction, the reinforcement is #9’s spaced at 10” on center top and bottom. Additional reinforcing cages are placed under the braced frame columns with the anchor bolts of these columns being tied to the bottom of the cage to increase the resistance to uplift. A detail of this connection is shown below in figure 1.

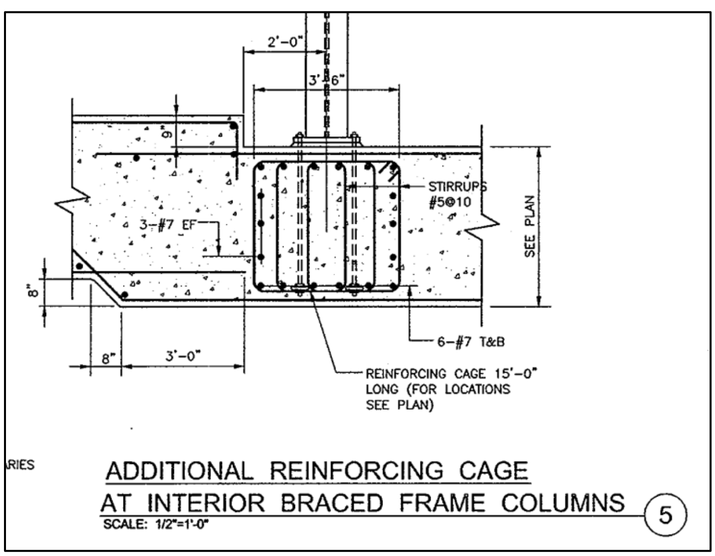


Figure 1: Additional foundation reinforcing

A 9" deep trench runs along the center of each towers foundation, parallel to the length of the building. This trench is filled in with 4000 psi concrete and reinforced with WWF after the erection of the interior columns in this area. In figure 2 below, the trench is shaded and outlined in red with the lateral columns marked in blue.

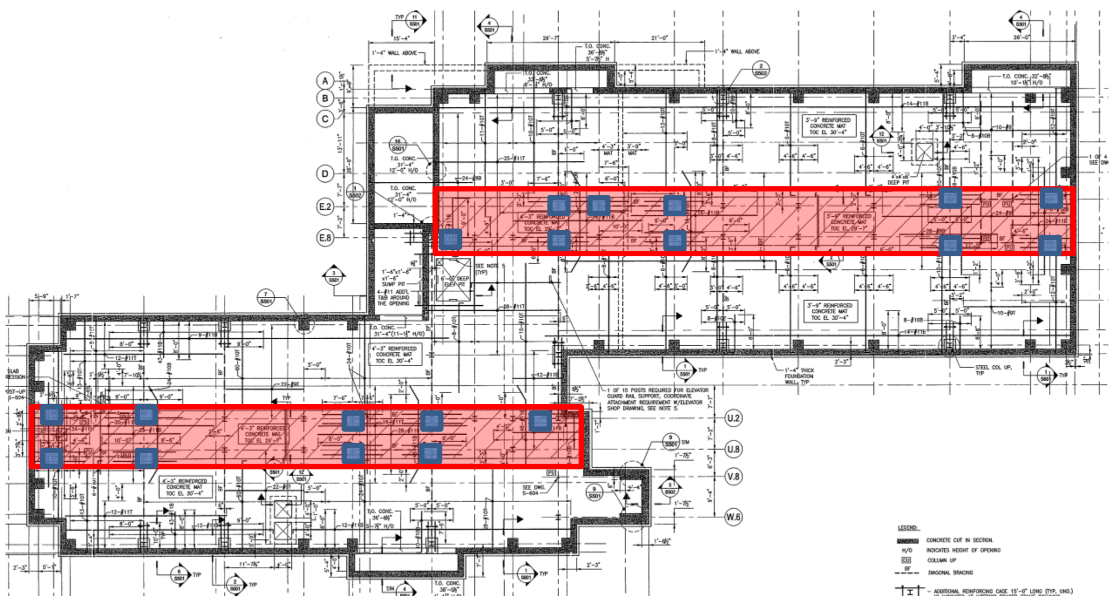


Figure 2: Foundation Trench

Floor Construction

The typical floor construction for Res Tower II is comprised of 3" 18 gage galvanized steel deck with 3 ¼" lightweight concrete topping, a total thickness of 6 ¼", and 6x6 WWF reinforcement. This is used everywhere except the loading dock and trash compactor area on the first floor. The floor system for these areas is comprised of 3" 16 gage steel deck with 6" normal weight concrete topping, a total thickness of 9", and epoxy coated reinforcement of #7's spaced at 12" on center in the bottom of the flutes and #5's spaced at 12" on center in the top running each way. All deck acts compositely.

The decking typically spans about 8'-9" supported by beams ranging in size from W14's to W18's. These composite beams then span roughly 23 feet to girders or columns. The girders have the same range in sizes as the beams mentioned previously. These spans create a typical bay size of 17-18' x 24-23'. The actual bay sizes vary but never too far from the typical dimensions. Figure 3 shows a typical floor plan for floors 3-18.

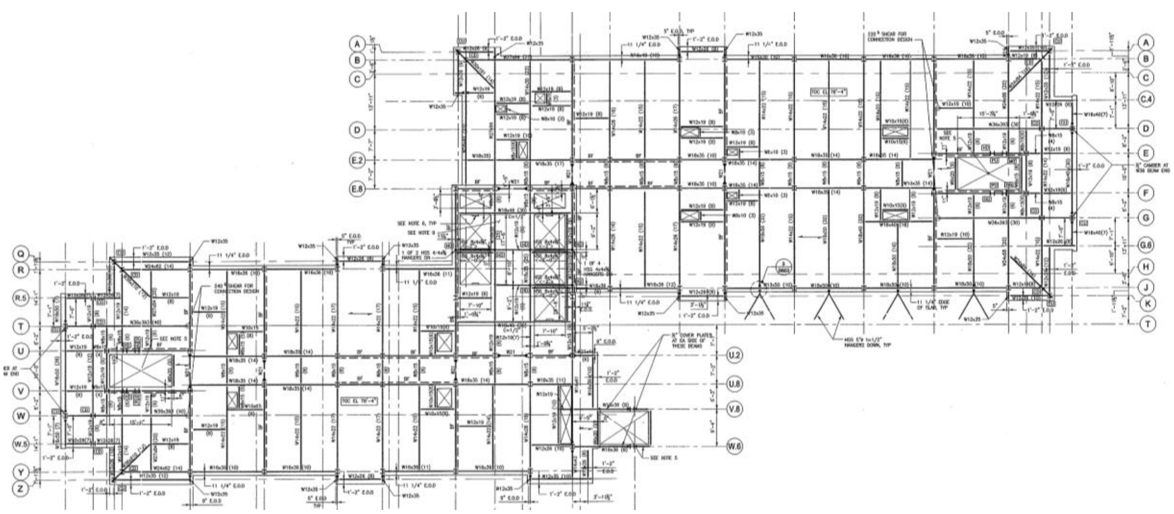


Figure 3: Typical floor plan

Lateral System

Steel braced frames are used to resist the lateral loads placed on the structure. At the termination of these columns, extra reinforcement is added to better tie the columns to the foundation and resist overturning forces. All columns in these braced frames are W14's ranging in size from W14x61 near the top of the structure to W14x398 for the bottom columns. The diagonal bracing members are W12's ranging in size from W12x152 to W12x45. This braced frame construction is categorized as a concentrically braced frame in ASCE7-10 which has an R value of 3.25. To allow for corridors to pass through the center of these braced frames, moment connections were made. Figure 4 shows an elevation of a braced frame with the moment connections clearly shown. The braced framed locations are highlighted in figure 5.

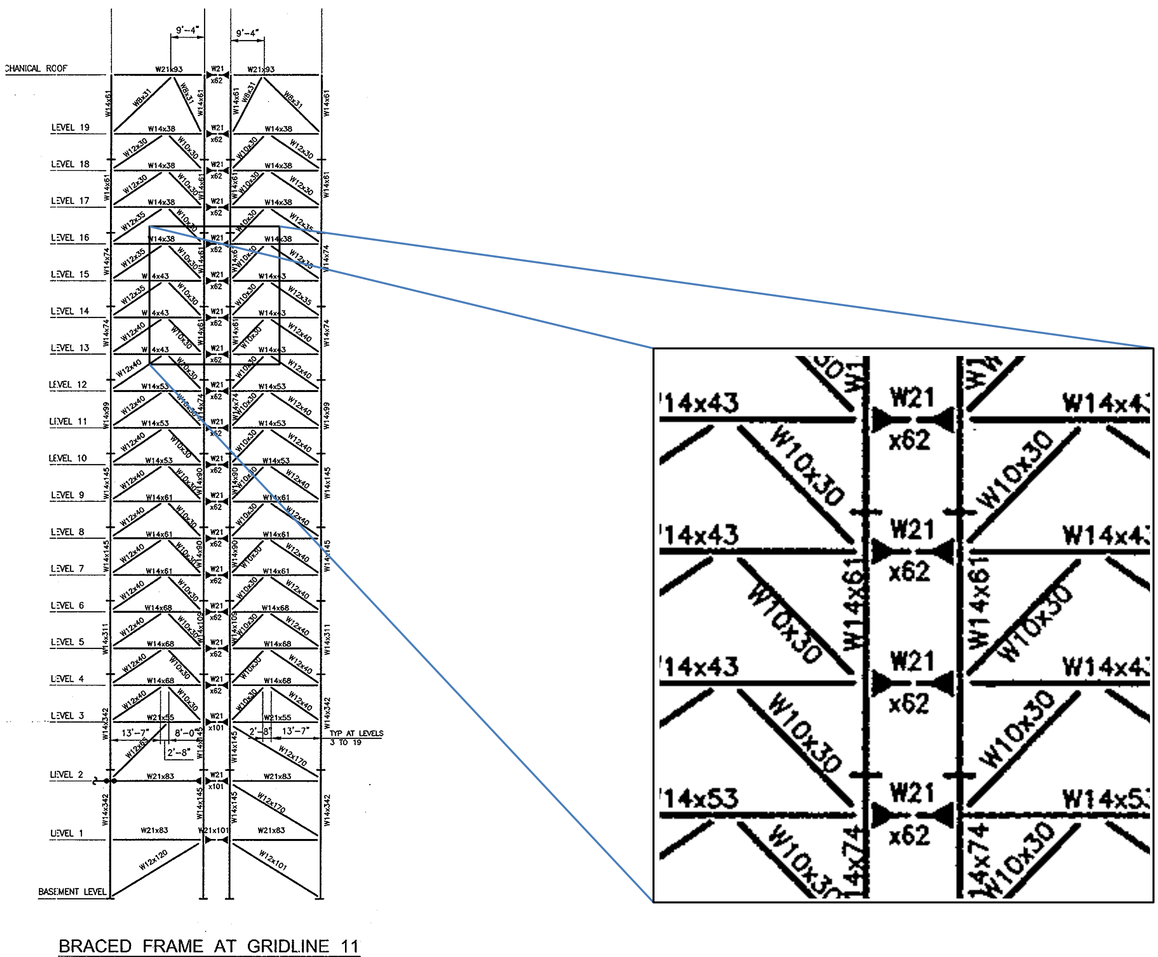


Figure 4: Braced frame elevation with moment connection

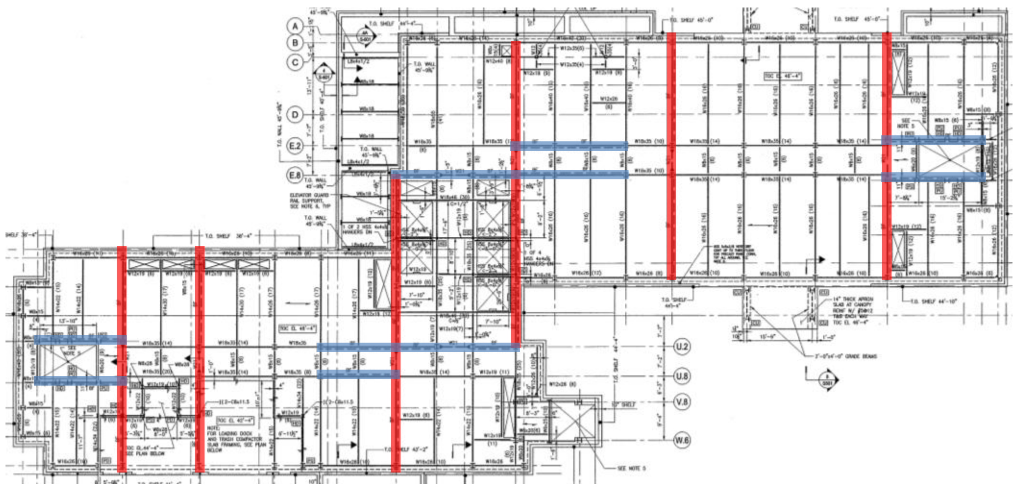


Figure 5: Foundation plan with braced frame locations highlighted

Due to the slender shape of the building in the short direction, the braced frames in this direction (highlighted in red) have wider bases than the braced frames in the longer direction (shown in blue). The wider base provides a more effective geometry for transferring lateral loads to the foundation in the form of vertical loads.

Figure 6: Connection construction photo

Some of the braced frames in perpendicular directions utilize the same columns making for very complicated connection details and erection processes. To successfully portray these connections, 3 dimensional models had to be built, presented and given to the contractors. Because of this, the design phase of the schedule had to be extended and more risk was taken by the connection designer. A construction photo of these connections is shown in figure 6.

Figure 7 shows one of the further issues encountered due to the connections of the braced frames. Where the columns terminate, some of the foundation had to be cut away to allow for the columns to be placed due to the large connections for the diagonal bracing members. A last minute adjustment of this type is both unnecessary and disruptive. This issue also pushed the steel erection schedule and caused delays in the overall construction schedule.

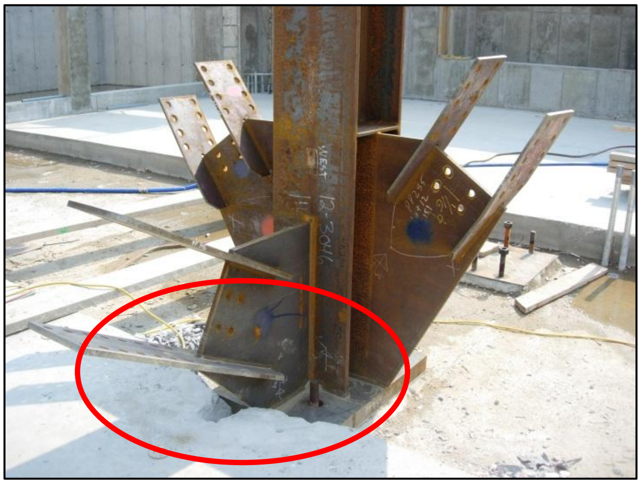


Figure 7:Foundation braced frame connection issues

Design Codes & Standards

Original Design	Thesis Design
Massachusetts Building Code 6th Edition	2009 International Building Code
1993 BOCA National Building Code	American Society of Civil Engineers (ASCE7-10)
American Institute of Steel Construction (2005 Manual)	2005 AISC Steel Manual

Table 1: Design codes vs. Thesis codes

Structural Materials

The materials listed in the chart below are specified in the structural drawings via the General Notes page of the structural drawings (S000) or general notes on the individual framing plans.

Material Properties		
Material		Strength
Steel	**Grade**	**fy = ksi**
Structural Shapes	A992	50
Plates	A36	36
Angles	A36	36
Structural Tubes	A500, B	46
Structural Pipes	A53, B or A501	30
Column Base Plates	A572, 50	50
Concrete	**Weight (lb/ft^3)**	**f'_c = psi**
Mat Foundation	145	4000
Slabs (Dock & Trash)	145	4000
Walls	145	4000
Typ. Slabs	115	3000
Reinforcing Steel		**fy = 60 ksi**
Welding Electrodes	E70 XX	**70 ksi**

Table 2: Material properties

Building Loads

In the tables that follow, the dead and live loads that were used by the designers and that were used for this thesis are listed. The dead loads were looked up in literature, assumed or calculated depending on the type of material they consist of; while the live loads were designated as specified by the codes listed in the tables.

Dead Load

Dead Loads	
Material	Load (psf)
Slab	
-Roof Deck	56
-Floor Deck	46
Façade	18
Superimposed	30

Table 3: Dead loads

Live Load

Live Loads		
	Design Load (psf)	Thesis Load (psf)
Occupancy Type	Mass. State Building Code	IBC 2009 & ASCE7-10
Public Area	100	100
Corridor	80	100
Dwelling Unit	40	40
Loading Dock	250	250
Mechanical Penthouse	150	125
Roof	30	20

Table 4: Live loads

Snow Load

The snow load for Res Tower II was determined using section 7.3 of ASCE7-10 (flat roof snow loads). Following the procedure and using ground snow load maps, the snow load for areas without drifting was calculated to be 27.72 psf.

Above floor 21, the building steps back. This geometric change will cause snow to accumulate against the taller tower forming a snow drift with the dimensions calculated by ASCE7 as depicted in figure 8 below (not drawn to scale). The mechanical penthouse would cause similar drifts but to a smaller scale. Full snow load and drift calculations can be found in appendix A.

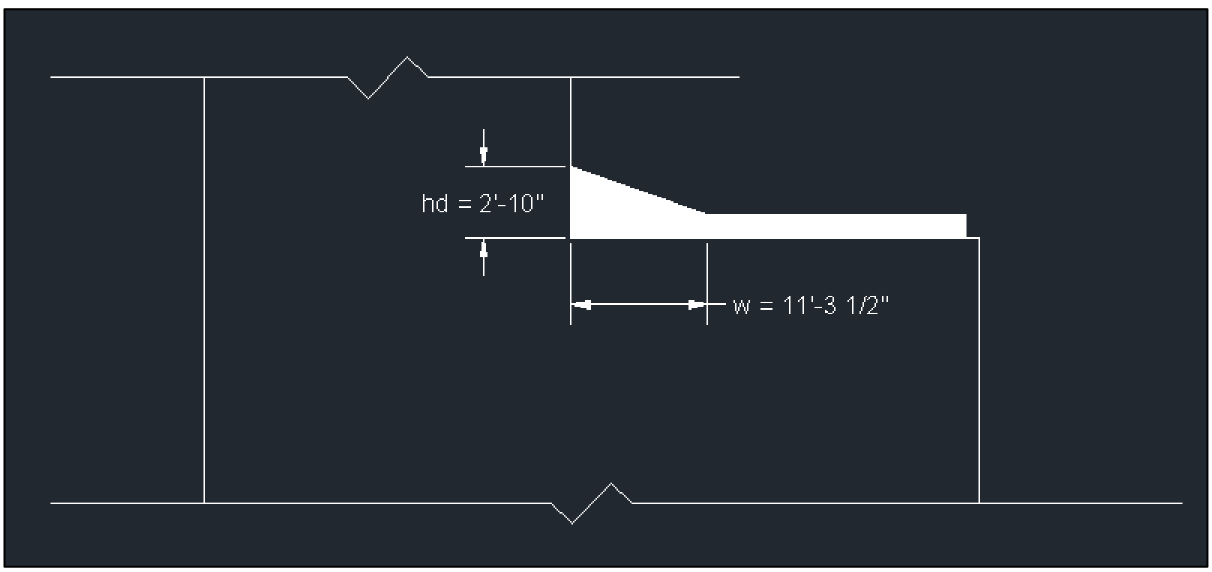


Figure 8: Geometry of snow drift

Wind Load

ASCE7-10 was used to determine the wind pressures on Res Tower II in both the North-South direction and the East-West direction and thus the forces transferred to the Main Wind-Force Resisting System (MWFRS). During the process of calculating these forces, assumptions had to be made.

The structure had to be assumed flexible as opposed to rigid due to the slender nature of the building. Because of this assumption the method of determining a structures approximate natural frequency (ASCE 26.9.2.1) could not be used. Instead of modeling the structure, the natural frequency was calculated using equations given in the seismic design section (ASCE 12.8.2.1). Inverting equation 12.8-7 (ASCE), $T_a = C_t h_n^x$, provided a natural frequency equal to 0.701 Hz. The code specifies that any natural frequency less than 1.0 Hz implies that the structure is flexible; because 0.701 Hz is less than 1.0 Hz, the assumption of a flexible building was correct.

Assumptions were also made to the geometry of the building. The building shape was simplified to compensate for setbacks and the vertical geometry was broken into two pieces to take advantage of similar floor plans. The lower section of the building was adjusted from the original shape to the red outline shown in figure 9. The upper section of the building was adjusted to the green outline, also shown in figure 9.

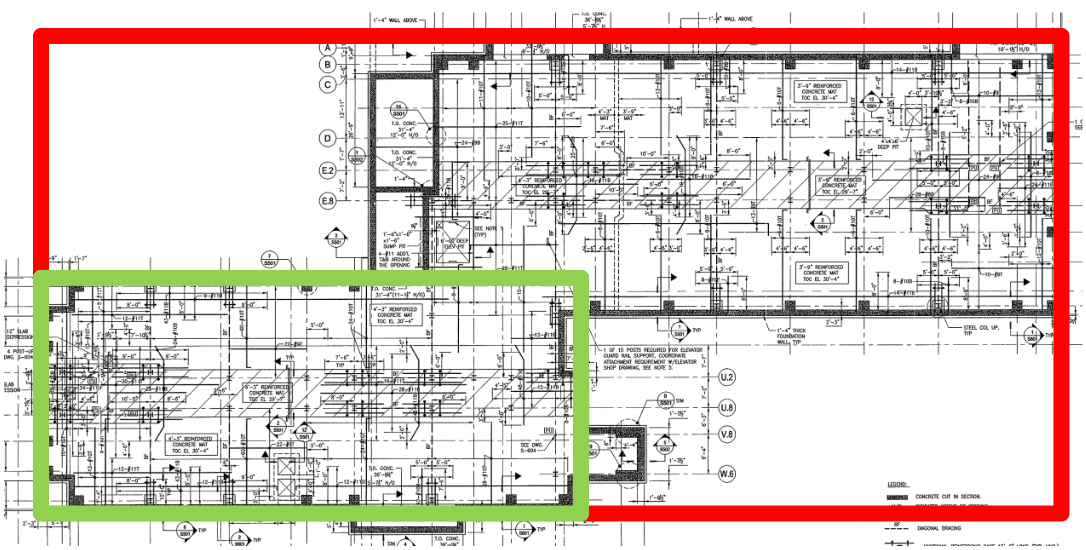


Figure 9: Simplified building plan for wind calculations

Figure 10 shows a rough Google SketchUp model of how the vertical geometries of the building were broken up. Using these two separate pieces allowed for more specific Gust Factors (26.9.5 ASCE). This alteration also allowed for a better estimation of the distribution of wind pressures (psf) to each floor (plf) and then accordingly to generalized story forces (k). A sample hand calculation of the wind pressures is provided in appendix B. After a firm understanding of the calculations necessary, excel spreadsheets were used to find the pressures in other directions and on the other piece of the building.

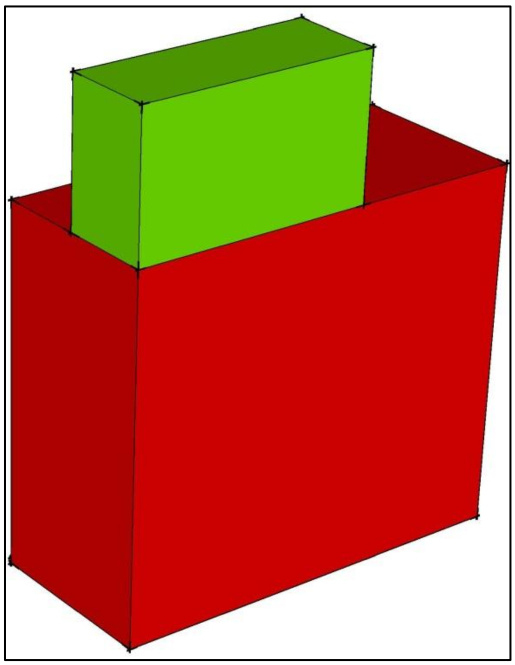


Figure 10: Simplified building geometry wind designated peices for wind calculations

The penthouse on the roof of Res Tower II was modeled as a continuation of the rest of the façade and therefore the forces on the roof were calculated with no parapet. This assumption is reasonable because the area outside of the penthouse is relatively small compared to the entire roof area and instead of having a conventional parapet; the architect provided a kick-edge with perforated steel along the edge of the building. This perforated steel allows for wind penetration and therefore has a negligible addition to the wind pressure on the MWFRS.

The final base shear and overturning moment were calculated using an excel spreadsheet which is shown in the following table. In the image following the table, a schematic depiction shows how the wind pressure is distributed along the height of the building. For wind pressures on the windward and leeward side in both directions, see appendix B.1.

North South			
Floor	Force (k)	Height (ft)	Moment (ft k)
1	59.90	0	0.00
2	120.06	16	1920.99
3	100.01	32	3200.21
4	80.66	42	3387.85
5	82.46	52	4287.86
6	83.95	62	5204.89
7	85.31	72	6141.97
8	86.63	82	7103.38
9	87.74	92	8072.51
10	88.57	102	9034.63
11	89.35	112	10007.66
12	90.20	122	11004.54
13	91.05	132	12018.37
14	91.86	142	13044.33
15	92.51	152	14060.79
16	93.03	162	15070.93
17	93.64	172	16106.13
18	94.30	182	17162.79
19	94.88	192	18216.39
20	95.38	202	19267.14
21	97.65	212	20700.92
22	76.42	222	16966.28
23	53.32	232	12369.81
24	53.57	242	12964.89
25	64.62	252	16283.25
26	81.17	266	21591.47
27	81.63	282	23020.03
ROOF	38.44	296	11378.36
BASE	2348.31		329588.37

East West			
Floor	Force (k)	Height (ft)	Moment (ft k)
1	132.31	0	0.00
2	265.21	16	4243.37
3	220.82	32	7066.38
4	177.99	42	7475.38
5	181.89	52	9458.26
6	185.13	62	11478.12
7	188.08	72	13541.60
8	190.95	82	15657.96
9	193.38	92	17791.12
10	195.19	102	19909.03
11	196.88	112	22050.67
12	198.72	122	24244.16
13	200.56	132	26474.49
14	202.33	142	28731.19
15	203.73	152	30967.26
16	204.87	162	33189.56
17	206.20	172	35466.37
18	207.64	182	37789.82
19	208.89	192	40106.63
20	209.99	202	42417.24
21	214.68	212	45511.42
22	183.09	222	40646.29
23	147.68	232	34261.47
24	148.31	242	35891.94
25	178.79	252	45054.94
26	224.48	266	59712.22
27	225.63	282	63626.29
ROOF	106.15	296	31420.50
BASE	5399.57		784183.67

Table 5: Wind forces

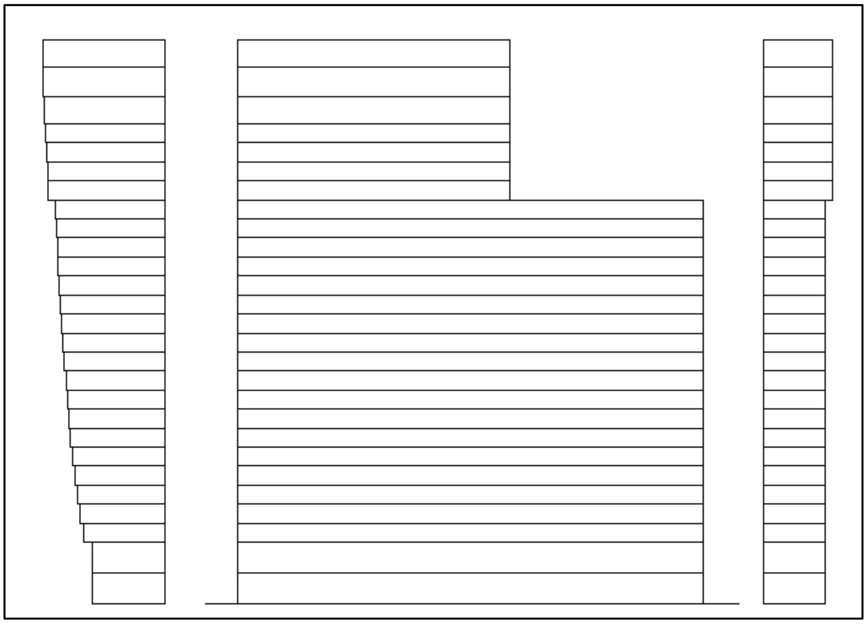


Figure 11: Wind pressure vertical distribution, North-South direction

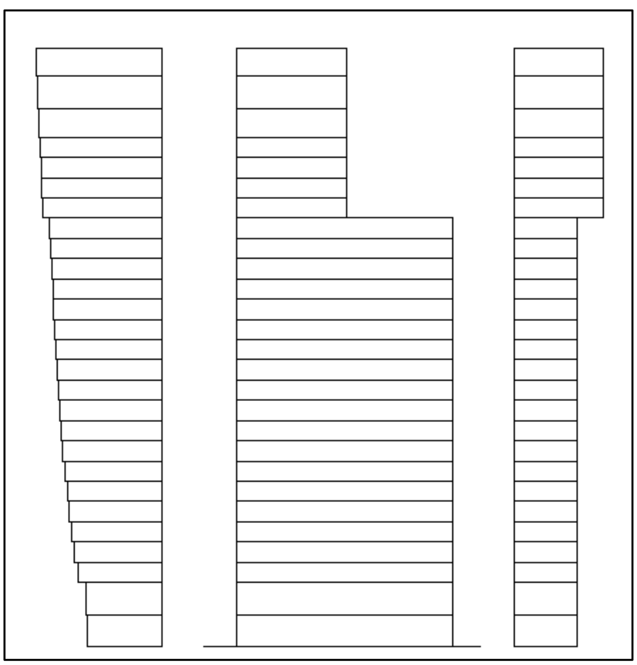


Figure 12: Wind pressure vertical distribution, East-West direction

Seismic Load

The seismic design for Res Tower II followed the procedure and criteria specified in ASCE7-10 chapters 11 and 12. Due to the geotechnical report being completed relevant to the Massachusetts Building Code, comparisons had to be made between that and ASCE7-10. In the geotechnical report, H&A give the soil a category rating of S3 from the Massachusetts Building Code, which compared relatively close to both site class C and D from ASCE7-10. Taking the more conservative class meant categorizing the soil as class D.

The equivalent lateral force procedure which is specified in section 12.8 (ASCE), was used to determine the base shear and overturning moment. To proceed with the specified calculations, the total building weight had to be calculated. This was done by counting beams and columns, then multiplying their respective lengths by the unit weight of the particular shape. Using the Vulcraft Metal Decking catalog, weights were found for the specified floor systems. A superimposed dead load of 30 psf was used to account for MEP systems, ceiling systems and fixtures, partitions and the different types of floor finishes including tile, wood and carpet. The façade system was specified to weigh 18 psf with 2 ft thick exterior walls which lead to 36 lbs per linear foot of exterior wall. These weights are shown below in tabulated form.

Material	Weight (k)
Non-lateral Columns	385
Lateral Columns	2137.5
Concrete: Slab and Deck	18210
Beams	1574.7
Façade	620.5
Superimposed	11383.3
Total Self Weight	34311

Table 6: Tabulation of building self weight

For the repetitive calculations, an excel spreadsheet (from AE 597A) was used to determine the load on each floor, the base shear and the overturning moment. This table is shown below.

Level	Height (ft)	Weight (k)	$w*h^k$	C_{VX}	Fi (k)	V_i (k)	M (ft-k)
roof	296	437.26	1809152	0.065	350	350	103452.97
27	282	407.65	1571184	0.056	304	653	85595.75
26	266	400.43	1416905	0.051	274	927	72811.27
25	252	400.43	1309110	0.047	253	1180	63731.32
24	242	400.43	1233788	0.044	238	1418	57680.88
23	232	400.43	1159894	0.042	224	1642	51985.52
22	222	400.43	1087462	0.039	210	1852	46638.36
21	212	417.19	1059074	0.038	205	2057	43374.89
20	202	838.48	1983236	0.071	383	2440	77393.04
19	192	838.48	1841213	0.066	356	2796	68293.82
18	182	838.48	1702579	0.061	329	3125	59862.47
17	172	838.48	1567432	0.056	303	3427	52082.66
16	162	838.48	1435879	0.052	277	3705	44937.51
15	152	838.48	1308039	0.047	253	3957	38409.66
14	142	838.48	1184041	0.043	229	4186	32481.13
13	132	838.48	1064027	0.038	206	4392	27133.30
12	122	838.48	948157	0.034	183	4575	22346.85
11	112	838.48	836610	0.030	162	4736	18101.62
10	102	838.48	729590	0.026	141	4877	14376.57
9	92	838.48	627330	0.023	121	4999	11149.62
8	82	838.48	530102	0.019	102	5101	8397.48
7	72	838.48	438227	0.016	85	5186	6095.47
6	62	838.48	352093	0.013	68	5254	4217.21
5	52	838.48	272185	0.010	53	5306	2734.28
4	42	851.3	202166	0.007	39	5345	1640.34
3	32	852.29	135948	0.005	26	5372	840.43
2	16	839.06	48531	0.002	9	5381	150.01
				Base:	5381		1015914

Table 7: Seismic story forces, base shear and overturning moment

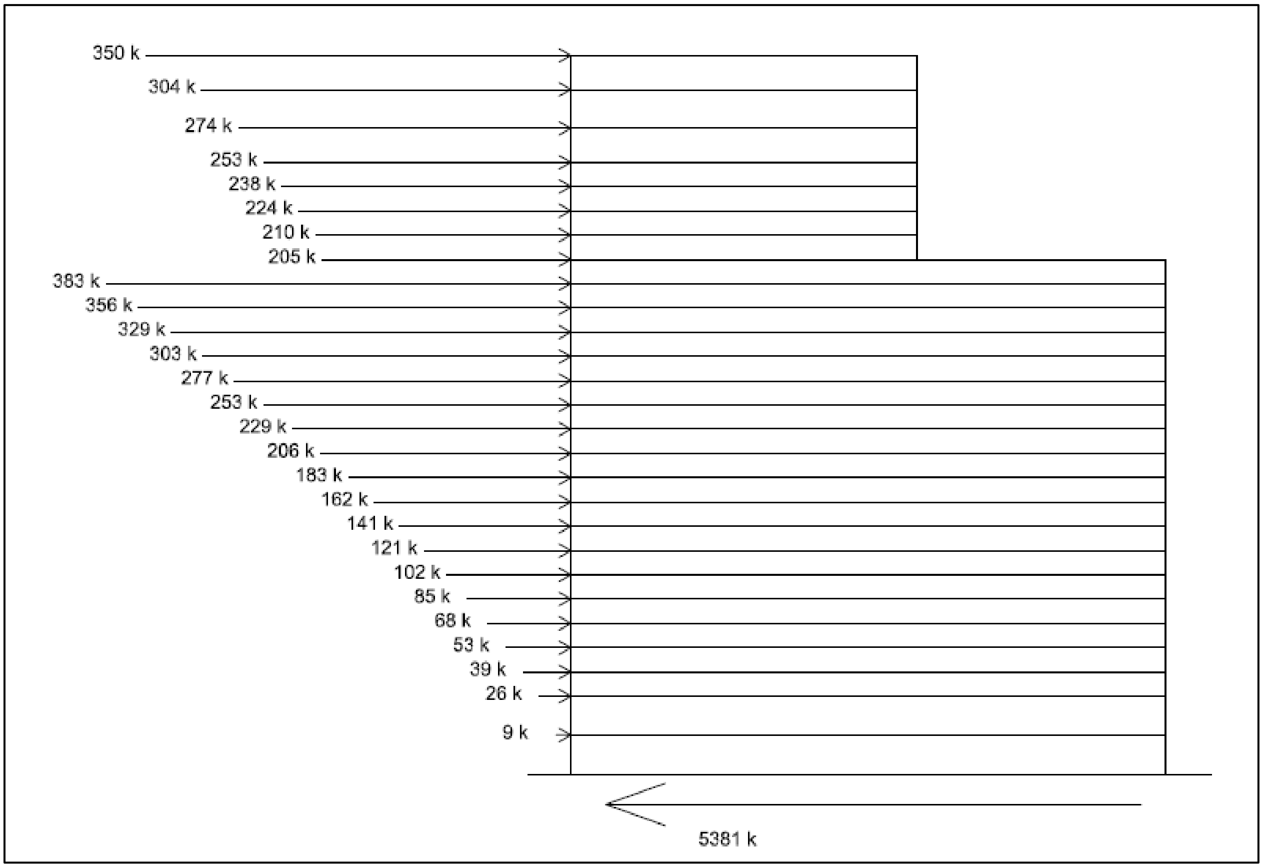


Figure 13: Seismic story forces

Gravity Load Spot Checks

Spot checks were performed on a typical bay of a typical floor. Columns F-12, F-13, J-12 and F-13 make up the corners of the bay on floor 5 that was used for these spot checks. Complete hand calculated spot checks can be found in appendix D.

Decking

The typical floor construction of Res Tower II utilizes a 3” 18 gage steel deck with 3 ¼” light weight concrete. Using the Vulcraft Steel deck catalog, deck type 3VLI18 matches these characteristics. The 3VLI18 works for unshored length and has almost 4 times the required strength to handle the required load. This extra strength was due to the 2 hour fire rating requirement; a slab of light weight concrete must be 3 ¼” thick to receive a 2 hour rating. Hand calculations for decking can be found in appendix D.1.

Beam & Girder

Strength and deflection checks for both the construction and post-construction phases were performed on a typical beam and girder. The members appear to be slightly over designed but the repetitive nature of the design may be the reason. The original design may have had simplicity of construction as an emphasis so that the designers may have chosen to repeat members and allow them to be stronger than necessary. This extra strength may also have been designed to allow for variation of use; such that areas could be utilized differently over time and still have sufficient strength. Hand calculations for a typical beam and girder can be found in appendices D.2 and D.3 respectively.

Column

Column 13-E.8 was chosen to analyze for the column spot check because it supports four different area types, including lobby, corridor, dwelling and roof areas. The column at the base of 13-E.8 is a W14x109 which has a max axial load equal to 1190 kips at an unbraced length of 16 ft. After summing all the above columns with their tributary weights and self-weights, an equivalent vertical load was added for uneven loading on floors three through nineteen. These columns support a corridor (80 psf) on one side and dwelling areas (40 psf) on the other. This is loosely depicted in appendix D.4. The extra equivalent proved to be very small, almost negligible. The load in the base column culminated to 959 k using unreduced live loads; this is less than the maximum allowable load and therefore this column has sufficient strength. Calculations for the equivalent load are shown in appendix D.4 and the tabulated contributions from each floor are shown below. The reduced values are also included below. A reduction factor of 0.481 was used for reducing the load in columns on floors one through 18 but for floor 19 and the roof, 0.5 was used because these columns are not supporting two or more floors.

Tributary Area of Column =	270.36	ft2
Corridor =	62.97	ft2
Dwelling =	207.39	ft2

Table 8: Tributary areas

						Unreduced	Reduced		
Floor	DL (psf)	LL 1 (psf)	Reduced (psf)	LL 2 (psf)	Reduced (psf)	Pu (lb)	Pu (lb)	W14x	Splice Length (ft)
1	77	100	not			72424.46	72424.46	109	32
2	77	100	not			49076.03	49076.03		
3	77	80	38.49	40	19.25	48692.30	37623.22	99	20
4	77	80	38.49	40	19.25	46316.30	35247.22		
5	77	80	38.49	40	19.25	48092.30	37023.22	74	20
6	77	80	38.49	40	19.25	46316.30	35247.22		
7	77	80	38.49	40	19.25	47948.30	36879.22	68	20
8	77	80	38.49	40	19.25	46316.30	35247.22		
9	77	80	38.49	40	19.25	47780.30	36711.22	61	20
10	77	80	38.49	40	19.25	46316.30	35247.22		
11	77	80	38.49	40	19.25	47588.30	36519.22	53	20
12	77	80	38.49	40	19.25	46316.30	35247.22		
13	77	80	38.49	40	19.25	47468.30	36399.22	48	20
14	77	80	38.49	40	19.25	46316.30	35247.22		
15	77	80	38.49	40	19.25	47348.30	36279.22	43	20
16	77	80	38.49	40	19.25	46316.30	35247.22		
17	77	80	38.49	40	19.25	47348.30	36279.22	43	20
18	77	80	38.49	40	19.25	46316.30	35247.22		
19	77	80	40	40	20	47348.30	36681.74	43	20
roof	60	20	not			37110.70	37110.70		
	snow load (psf):								
	27.72				**Total Load (k):**	958.76	770.98		

Table 9: Column loads

Conclusion

By examining and analyzing each individual system, a greater understanding of the whole structural system was gained. Through spot checks and by verifying that proper loads were used, it was determined that the existing structural conditions are adequate in strength to carry the actual loads that will be required by the structure.

Complicated connections caused unnecessary problems in both the schematic and construction phases of the construction process. These connections could serve as a focus for the redesign phase of this thesis.

After calculating the seismic overturning moment using ASCE7-10 it is clear to see that the designers chose to place extra reinforcement in the mat foundation below the lateral columns to counteract the large uplift forces. Realizing that a seismic load will not control over a wind load, or vice-versa, both loads must be accounted for when determining the required strength of the lateral system. The vertical force distributions for both the wind and seismic forces have abrupt changes near the 21 story because of the relatively large change in building plan. Further investigation into how this abrupt change affects the design of the lateral system shall be done for tech report 3.

Spot checks verified that typical members were adequate for the required lateral loads and their deflections were well under the required deflection limits both for live load and wet concrete. Also in tech 3, it will have to be determined how these members handle lateral loads and how they distribute them to other structural systems.

Appendix A: Snow Load & Drift Calculations

TYLER MEEK | AE SENIOR THESIS | SNOW LOAD 1

(ASCE 7-10) 7.3 FLAT ROOF SNOW LOADS

$P_f = 0.7 C_e C_t I_s P_g$

C_e (TABLE 7-2): EXPOSURE C, FULLY EXPOSED ROOF $\Rightarrow C_e = 0.9$

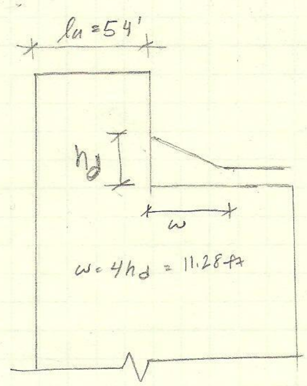
C_t (TABLE 7-3): $C_t = 1.0$

I_s (TABLE 1.5-2): $I_s = 1.10$

(FIG 7-1) $P_g = 40$

$P_f = (0.7)(0.9)(1.0)(1.10)(40) \Rightarrow \boxed{P_f = 27.72 \text{ lb/ft}^2}$

BUILDING STEP @ FLOOR 21 WILL CAUSE DRIFT



FROM FIG 7-9:

$h_d = 2.82 \text{ ft}$

(7.7.1)

$P_d = h_d \gamma$

$\gamma = 0.13 P_g + 14$

$\gamma = (0.13)(40) + 14$

$\gamma = 19.2$

LOAD FROM DRIFT

$P_d = (2.82)(19.2) = 54.14 \text{ lb/ft}^2$

Appendix B: Wind Load Calculations

B.1: Wind Pressures

North-South										
Floor	Elev	z	kz	q	windw	windward	windward	leeward ps	leeward plf	leeward k
1	46	0	0.85	36.25	38.35	4088.89	32.711	-31.87	-3398.02	-27.184
2	62	16	0.86	36.68	38.67	4122.78	65.693	-31.87	-3398.02	-54.368
3	78	32	0.99	42.31	42.86	4570.03	55.832	-31.87	-3398.02	-44.174
4	88	42	1.05	44.78	44.70	4766.55	46.683	-31.87	-3398.02	-33.980
5	98	52	1.10	46.83	46.23	4929.19	48.479	-31.87	-3398.02	-33.980
6	108	62	1.14	48.54	47.50	5064.72	49.970	-31.87	-3398.02	-33.980
7	118	72	1.18	50.24	48.77	5200.26	51.325	-31.87	-3398.02	-33.980
8	128	82	1.22	51.86	49.98	5329.01	52.646	-31.87	-3398.02	-33.980
9	138	92	1.24	53.06	50.87	5423.88	53.764	-31.87	-3398.02	-33.980
10	148	102	1.27	53.95	51.54	5495.04	54.595	-31.87	-3398.02	-33.980
11	158	112	1.29	55.02	52.33	5579.75	55.374	-31.87	-3398.02	-33.980
12	168	122	1.32	56.08	53.12	5664.45	56.221	-31.87	-3398.02	-33.980
13	178	132	1.34	57.15	53.92	5749.16	57.068	-31.87	-3398.02	-33.980
14	188	142	1.36	58.13	54.65	5827.09	57.881	-31.87	-3398.02	-33.980
15	198	152	1.38	58.77	55.13	5877.92	58.525	-31.87	-3398.02	-33.980
16	208	162	1.39	59.45	55.64	5932.13	59.050	-31.87	-3398.02	-33.980
17	218	172	1.41	60.31	56.27	5999.89	59.660	-31.87	-3398.02	-33.980
18	228	182	1.43	61.12	56.87	6064.27	60.321	-31.87	-3398.02	-33.980
19	238	192	1.45	61.76	57.35	6115.10	60.897	-31.87	-3398.02	-33.980
20	248	202	1.46	62.39	57.82	6165.24	61.402	-31.87	-3398.02	-33.980
21	258	212	1.48	62.98	61.60	6567.89	63.666	-31.87	-3398.02	-33.980
22	268	222	1.49	63.58	62.07	3387.78	49.778	-35.38	-1931.24	-26.646
23	278	232	1.50	64.18	62.54	3413.36	34.006	-35.38	-1931.24	-19.312
24	288	242	1.52	64.78	63.00	3438.94	34.261	-35.38	-1931.24	-19.312
25	298	252	1.53	65.36	63.46	3463.78	41.441	-35.38	-1931.24	-23.175
26	312	266	1.55	66.07	64.02	3494.47	52.202	-35.38	-1931.24	-28.969
27	328	282	1.57	66.89	64.66	3529.55	52.663	-35.38	-1931.24	-28.969
ROOF	342	296	1.59	67.61	65.23	3560.24	24.922	-35.38	-1931.24	-13.519

East-West										
Floor	Elev	z	kz	q	windward psf	windward plf	windward k	leeward psf	leeward plf	leeward k
1	46	0	0.85	36.25	37.07	9020.45	72.164	-30.90	-7518.37	-60.147
2	62	16	0.86	36.68	37.37	9094.12	144.917	-30.90	-7518.37	-120.294
3	78	32	0.99	42.31	41.37	10066.52	123.086	-30.90	-7518.37	-97.739
4	88	42	1.05	44.78	43.13	10493.79	102.802	-30.90	-7518.37	-75.184
5	98	52	1.10	46.83	44.58	10847.39	106.706	-30.90	-7518.37	-75.184
6	108	62	1.14	48.54	45.79	11142.06	109.947	-30.90	-7518.37	-75.184
7	118	72	1.18	50.24	47.00	11436.73	112.894	-30.90	-7518.37	-75.184
8	128	82	1.22	51.86	48.15	11716.67	115.767	-30.90	-7518.37	-75.184
9	138	92	1.24	53.06	49.00	11922.93	118.198	-30.90	-7518.37	-75.184
10	148	102	1.27	53.95	49.63	12077.64	120.003	-30.90	-7518.37	-75.184
11	158	112	1.29	55.02	50.39	12261.80	121.697	-30.90	-7518.37	-75.184
12	168	122	1.32	56.08	51.15	12445.97	123.539	-30.90	-7518.37	-75.184
13	178	132	1.34	57.15	51.90	12630.14	125.381	-30.90	-7518.37	-75.184
14	188	142	1.36	58.13	52.60	12799.57	127.149	-30.90	-7518.37	-75.184
15	198	152	1.38	58.77	53.06	12910.07	128.548	-30.90	-7518.37	-75.184
16	208	162	1.39	59.45	53.54	13027.94	129.690	-30.90	-7518.37	-75.184
17	218	172	1.41	60.31	54.14	13175.28	131.016	-30.90	-7518.37	-75.184
18	228	182	1.43	61.12	54.72	13315.24	132.453	-30.90	-7518.37	-75.184
19	238	192	1.45	61.76	55.17	13425.74	133.705	-30.90	-7518.37	-75.184
20	248	202	1.46	62.39	55.62	13534.77	134.803	-30.90	-7518.37	-75.184
21	258	212	1.48	62.98	59.03	14363.78	139.493	-30.90	-7518.37	-75.184
22	268	222	1.49	63.58	59.47	8502.21	114.330	-43.61	-6233.91	-68.761
23	278	232	1.50	64.18	59.92	8565.72	85.340	-43.61	-6233.91	-62.339
24	288	242	1.52	64.78	60.36	8629.22	85.975	-43.61	-6233.91	-62.339
25	298	252	1.53	65.36	60.79	8690.91	103.983	-43.61	-6233.91	-74.807
26	312	266	1.55	66.07	61.33	8767.12	130.973	-43.61	-6233.91	-93.509
27	328	282	1.57	66.89	61.94	8854.22	132.116	-43.61	-6233.91	-93.509
ROOF	342	296	1.59	67.61	62.47	8930.42	62.513	-43.61	-6233.91	-43.637

B.2: Hand Calculations

TYLER MEEK | AE SENIOR THESIS | WIND CALCS | 1

ASCE 7-10: DESIGN CRITERIA

RISK CATEGORY (TABLE 1.5.1) : III

BASIC WIND SPEED (FIG 26.5-1B) : V = 140 MPH

DIRECTIONALITY FACTOR (TABLE 26.6-1) : $K_d = 0.85$

EXPOSURE CATEGORY (SPECIFIED IN PLANS) : C

TOPOGRAPHIC FACTOR (SECTION 26.8) : $K_{zt} = 1.0$

GUST FACTOR: (CAN'T ASSUME RIGID) (26.9)

26.9.2.1 : BUILDING HEIGHT = 342' > 300' ∴ 26.9.3 CAN NOT BE USED

DUE TO SLENDER FLOOR PLAN } ASSUME FLEXIBLE STEEL FRAME SYSTEM

NORTH-SOUTH DIRECTION

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3{,}600\, n_1)} + \frac{0.577}{\sqrt{2\ln(3600\, n_1)}}$$

n_1 = NATURAL FREQUENCY

$n_1 = \frac{1}{T_a}$; T_a = NATURAL PERIOD

(P.90) FROM 12.8.2.1 (SEISMIC DESIGN)

$T_a = C_t h_n^{x}$ $\quad C_t = 0.02$ (CONCENTRICALLY BRACED FRAMES)

$x = 0.75$

$h_n = \cancel{342\ ft}$ 296

$$n_1 = \frac{1}{(0.02)(\cancel{342}\ 296)^{0.75}}$$

$n_1 = \cancel{0.6287}$ 0.701 Hz < 1.0 ∴ FLEXIBLE ASSUMPTION CORRECT

$\sqrt{2\ln[3600(\cancel{0.6287}\ 0.701)]} = \cancel{3.9305}$ 3.958

$$g_R = \cancel{3.9305}\ 3.958 + \frac{0.577}{\cancel{3.9305}\ 3.958} \Rightarrow g_R = \cancel{4.077}\ 4.104$$

TYLER MEEK | AE SENIOR THESIS | WIND CALCS | 2

(p. 255)

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

$\bar{z} = 0.6h$ $h = 342$

$\bar{z} = 205.2 \geq z_{min} = 15\text{ ft}$ ok

(TABLE 26.9-1)
$z_{min} = 15$
$c = 0.20$
$\ell = 500$
$\bar{\epsilon} = 1/5.0$

$$I_{\bar{z}} = (0.20)\left(\frac{33}{205.2}\right)^{1/6}$$

$I_{\bar{z}} = 0.1475$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

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

* B = HORIZONTAL LENGTH $\perp$ TO WIND DIRECTION $= 54.583''$

$$L_{\bar{z}} = 500\left(\frac{205.2}{33}\right)^{1/5}$$

$L_{\bar{z}} = 720.614$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{54.583 + 342}{720.614}\right)^{0.63}}}$$

$Q = 0.8355$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V$$

$\bar{b} = 0.65$
$\bar{\alpha} = 1/6.5$
$V = 140$

$$\bar{V}_{\bar{z}} = (0.65)\left(\frac{205.2}{33}\right)^{1/6.5}\left(\frac{88}{60}\right)(140)$$

$\bar{V}_{\bar{z}} = 176.797$

$$N_1 = \frac{(0.6281)(720.64)}{176.797} \Rightarrow N_1 = 2.563$$

$$R_n = \frac{7.47(2.563)}{[1 + (10.3)(2.563)]^{5/3}} \Rightarrow R_n = 0.07688$$

TYLER MEEK | AE SENIOR THESIS | WIND CALCS | 3

R_h: $\eta = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(0.6287)(342)/176.797 \Rightarrow \eta = 5.594$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = \frac{1}{5.594} - \frac{1}{2(5.594)^2}\left[1 - e^{-2(5.594)}\right]$$

$R_h = 0.16277$

R_B: $\eta = 4.6 n_1 B/\bar{V}_{\bar{z}} = 4.6(0.6287)(54.585)/176.797 \Rightarrow \eta_1 = 0.8929$

$$R_B = \frac{1}{0.8929} - \frac{1}{2(.8929)^2}\left(1 - e^{-2(0.8929)}\right) \Rightarrow R_B = 0.5980$$

R_L: $\eta = 15.4 n_1 L/\bar{V}_{\bar{z}}$ $\quad$ L = HORIZONTAL DIMENSION // TO WIND DIRECTION

$$\eta = \frac{15.4(0.6287)(142.958)}{176.797} \qquad L = 142.958'$$

$\eta = 7.8288$

$$R_L = \frac{1}{7.8288} - \frac{1}{2(7.8288)^2}\left(1 - e^{-2(7.8288)}\right) \Rightarrow R_L = 0.1196$$

DAMPING RATIO (β): 1.0 RECOMMENDED BY ISO (1997) ON PAGE 521 OF ASCE 7-10

$$R = \sqrt{\left(\frac{1}{0.01}\right)(0.07688)(0.16277)(0.5980)\left[0.53 + 0.47(0.1196)\right]}$$

$R = 0.6623$

$$G_f = 0.925\left[\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right]$$

$$G_f = 0.925\left[\frac{1 + 1.7(0.1475)\sqrt{(3.4^2)(0.8355^2) + (3.4^2)(0.6623)^2}}{1 + 1.7(3.4)(0.1475)}\right]$$

1.8526

$\boxed{G_f = 0.953}$ FOR NORTH-SOUTH, TALL TOWER

TYLER MEEK | AE SENIOR THESIS | WIND CALCS | 4

ENCLOSURE CLASSIFICATION ⇒ ENCLOSED ∴ $GC_{pi} = \pm 0.18$
↳ NO OPERABLE WINDOWS

C_p: WALL PRESSURE COEFFICIENTS

USED WITH

$L/B = 2.6$
• ONLY FOR TALL TOWER

WIND WARD → 0.8 — q_z

LEE WARD → " −0.35 — q_h

SIDE WALL → −0.7 — q_h

27.4.2 ENCLOSED FLEXIBLE BUILDINGS

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

$q = q_z$ FOR WINDWARD

$q = q_h$ FOR LEEWARD

$q_i = q_h$ FOR ENCLOSED

$$q = 0.00256\,K_z K_{zt} K_d V^2 \qquad K_h = 1.632$$

$$q_h = 69.6$$

WINDWARD:

$$p = q_z(0.953)(0.8) - 69.6(\pm 0.18)$$

$$p = (0.7624\,q_z + 12.53)\ \text{psf}$$

N-S DIRECTION

FOR TALL TOWER ONLY

LEEWARD:

$$p = 69.6(0.953)(-0.35) - 69.6(\pm 0.18)$$

$$p = -35.74\ \text{psf}$$

Appendix C: Seismic Calculations

TYLER MEEK | AE SENIOR THESIS | SEISMIC CALCS | 1

ASCE7-10

(p.65) 11.4: SEISMIC GROUND MOTION

GEOTECH REPORT SPECIFIES SITE CLASS S3 FROM MASSACHUSETTS BUILDING CODE

TABLE 1612.4.1

S3 ⇒ • UNDRAINED SHEAR STRENGTH ≥ 1000 psf
• CONTAINING 40 - 100 ft MEDIUM STIFF CLAY

ASSUMPTION SITE CLASS D: STIFF SOIL

$S_{MS} = F_a S_S$ FROM USGS DESIGN TOOLS: GROUND MOTION CALCULATOR

$S_{M1} = F_v S_1$ $S_S = 0.415$
$S_1 = 0.156$

(TABLE 11.4-1) $F_a = 1.468$

(11.4-2) $F_v = 2.176$

(11.4.3) $S_{MS} = F_a S_S = 0.60922$ $S_{M1} = F_v S_1 = 0.33946$

(11.4.4) $S_{DS} = \frac{2}{3} S_{MS} = 0.40615$

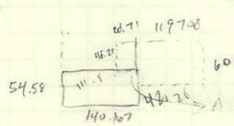
$S_{D1} = \frac{2}{3} S_{M1} = 0.22630$

(11.5.1) $I_e = 1.25$ (RISK CATEGORY III)

(TABLE 11.6-1) SEISMIC DESIGN CATEGORY ⇒ D (MORE SEVERE CASE)

USE EQUIVALENT LATERAL FORCE PROCEDURE (12.8)

$V = C_s W$



A = 16285 sf FOR 1-20

TYLER MEEK | AE SENIOR THESIS | SEISMIC CALCS 2

$V = C_s W$ $W = 34449^k$ (CALCULATED USING EXCEL SPREADSHEET)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

$R = 3.25$ PER TABLE 12.2-1
- CONCENTRICALLY BRACED FRAMES (ORDINARY)

$$C_s = \frac{0.40615}{\left(\frac{3.25}{1.25}\right)} \Rightarrow C_s = 0.1562$$

$\therefore V = (34449)(0.1562) \Rightarrow V = 5381^k$

(12.8.2.1) $T_a = C_t h_n^x \Rightarrow T_a = 1.427$ (WORKED OUT IN WIND CALCS)

(12.8.3) VERTICAL DISTRIBUTION OF SEISMIC FORCES

$F_x = C_{vx} V$

$k = 1.4635$ (12.8.3)

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i}$$

CALCULATIONS COMPLETED USING SPREAD SHEET

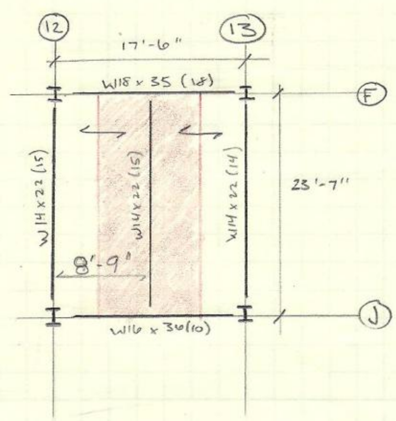
Appendix D: Spot Checks

D.1: Decking Check

Tyler Meek | AE Senior Thesis | Decking Spot Checks | 1

Spot checks performed on typical bay of typical floor

Floor 5



Floor Construction:

3" - 18g Steel Deck

3 1/4" LWT Concrete

Total t = 6 1/4"

f'_c = 3000 psi

Loads:

LL = 40 psf

SDL = 30 psf

Total = 70 psf

Vulcraft Decking Catalog

3VLI18: Check unshored length (3 span)

15' > 8'-9" ∴ ok for unshored length

Check superimposed live load

9'-0" clear span:

275 psf >> 70 psf ∴ ok for loading

The loading check shows that this floor construction is very over designed but this is not the controlling load factor

Fire resistance:

2HR rating ⇒ 3 1/4" LWT concrete

2HR rating ok

$\boxed{\text{Deck 3VLI18 checks out}}$

D.2: Beam Check

TYLER MEEK	AE SENIOR THESIS	BEAM SPOT CHECKS	2

COMPOSITE BEAM: W14 x 22 (15)

TRIBUTARY WIDTH = 8'-9"
SPAN = 23'-7"

W14 x 22:
$A_g = 6.49 \text{ in}^2$
$I_x = 199 \text{ in}^4$
$F_y = 50$ ksi

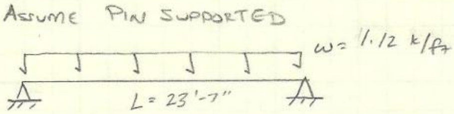
LL = 40 psf (DO NOT REDUCE)
SDL = 30 psf
DL = 47 psf (DECKING CATALOG + ALLOWANCE FOR REINFORCEMENT)
SELF WEIGHT = 22 plf

$w_u = 1.2D + 1.6L$

$D = (30 + 47)(8.75) + 22 = 463.25$ plf
$L = 40(8.75) = 350$ plf

$w_u = 1.2(463.25) + 1.6(350)$
$w_u = 1.12$ k/ft

ASSUME PIN SUPPORTED



$V_u = \frac{(1.12)(23.583)}{2} \Rightarrow V_u = 13.2^k$

$M_u = \frac{wl^2}{8} = \frac{(1.12)(23.583)^2}{8}$

$M_u = 77.86$ ft-k

b_{eff}: (EQUAL ON BOTH SIDES)

$b_{eff} = \min \begin{cases} \text{SPAN}/4 = 5.896 \leftarrow \text{CONTROLS} \\ \text{SPACING} = 8.75 \end{cases}$

(TABLE 3-19) STEEL MANUAL

W14 x 22, PNA = 7 $\Rightarrow \Sigma Q_n = 81.2$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{81.2}{0.85(3)(5.896)(12)} \Rightarrow a = 0.45 < 1.0$

USE $a = 1.0$

$Y_2 = t_{SLAB} - \frac{a}{2} = 6.25 - \frac{1}{2} = 5.75$

$\phi M_n = 190.5$ ft-k $> M_u = 77.86$ ∴ OK

$Q_n = \frac{81.2}{17.2} = 4.72 \sim 5$ ∴ 10 STUDS REQUIRED < 15 STUDS USED ∴ OK

(TABLE 3-2) $\phi V_n = 94.8^k < V_u = 13.2^k$ ∴ OK

TYLER MEEK | AE SENIOR THESIS | BEAM SPOT CHECKS | 3

CHECK Δ_{LL}:

$$\Delta_{LL} = \frac{l}{360} = \frac{(23.583)(12)}{360} = 0.7861 \text{ in} \quad \text{MAXIMUM DEFLECTION}$$

(TABLE 3-20) $I_{LB} = 406 \text{ in}^4$ $\qquad w_{LL} = (40 \text{ psf})(8.75)/1000$

$$\Delta_{LL} = \frac{5 w_{LL} l^4}{384 EI_{LB}} = \frac{(5)(0.35)(23.583)^4(1728)}{384(29000)(406)}$$

$\Delta_{LL} = 0.207 \text{ in} < \Delta_{LL_{max}} = 0.7861 \quad \therefore \underline{ok}$

CHECK BEAM DEFLECTIONS UNDER WET CONCRETE

$$\Delta_{max} = \frac{l}{240} = \frac{(23.583)(12)}{240} = 1.18 \text{ in}$$

$$I_{REQ} = \frac{5 w l^4}{384 E \Delta_{max}} \qquad w = [(47)(8.75) + 22]/1000$$

$w = 0.414$

$$I_{REQ} = \frac{(5)(0.414)(23.583)^4(1728)}{384(29000)(1.18)}$$

$I_{REQ} = 84.20 \text{ in}^4 < I_{BB} = 199 \text{ in}^4 \quad \therefore \underline{ok}$

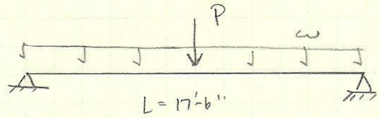
$\boxed{\text{W14 x 22 WORKS}}$

D.3: Girder Check

TYLER MEEK | AE SENIOR THESIS | GIRDER SPOT CHECKS | 4

$I_x = 510\ in^4$

COMPOSITE GIRDER: W18 x 35 (18)



ASSUMING PIN IS CONSERVATIVE FOR BEAM DESIGN

$P_u = 2(13.2^k) = 26.2^k$

$w_u = 35/1000 = 0.035\ k/ft$

$V_u = 26.2/2 + \frac{(0.035)(17.5)}{2} \Rightarrow V_u = 13.41^k$

$M_u = \frac{(0.035)(17.5)^2}{8} + \frac{(26.2)(17.5)}{4}$

$M_u = 116\ ft\text{-}k$

(TABLE 3-19) PNA=7

$\Sigma Q_n = 129$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{129}{(0.85)(3)(4.375)(12)}$

$b_{eff} = \min \begin{cases} SPAN/4 = 4.375 \leftarrow \text{CONTROLS} \\ SPACING = 23 \end{cases}$

$a = 0.96 < 1.0$ USE $a = 1.0$

$Y_2 = t_{SLAB} - \frac{a}{2} = 6.25 - \frac{1}{2} = 5.75$

(TABLE 3-19)

$\phi M_n = 369.5\ ft\text{-}k > M_u = 116\ ft\text{-}k \therefore$ OK

$\phi V_n = 159^k >> V_u = 13.41^k \therefore$ OK

STRENGTH GOOD

DEFLECTION

Δ_{LL}: $P_L = \frac{(350)(23.583)}{1000} = 8.25^k$

$\Delta_{LL} \le \frac{l}{360} = \frac{(17.5)(12)}{360} = 0.5833\ in$

(TABLE 3-20) $I_{LB} = 950\ in^4$

$\Delta_{LL} = \frac{Pl^3}{48EI} + \frac{5wl^4}{384EI} = \frac{(8.25)(17.5)^3(1728)}{48(29000)(950)} + \frac{5(0.035)(17.5)^4(1728)}{384(29000)(950)}$

$= 0.06\ in$

$\Delta_{LL} < 0.5833\ in \therefore$ OK

W/ CONCRETE:

$\Delta_{MAX} = \frac{l}{240} = 0.875\ in$ $P = 9.76^k$

$I_{REQ} = \frac{Pl^3}{48E\Delta} + \frac{5wl^4}{384E\Delta} = \frac{(9.76)(17.5)^3(1728)}{48(29000)(.875)} + \frac{5(0.035)(17.5)^4(1728)}{384(29000)(0.875)}$

$I_{REQ} = 77.12\ in^4 < 510\ in^4 \therefore$ OK

$\boxed{\text{W18 x 35 WORKS}}$

D.4: Column Check

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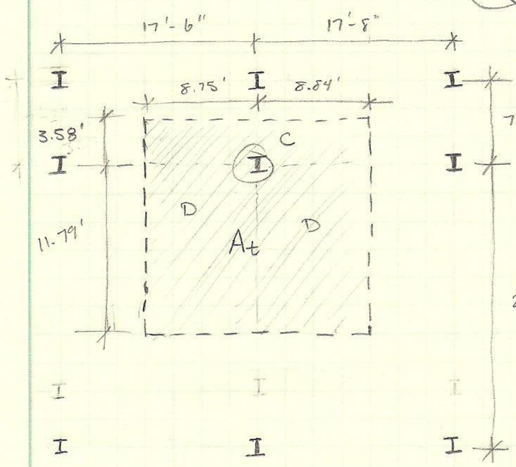
13-E8

Column to be checked: ~~F13~~ W14 x 109 at base



$A_t = (17.59')(15.37')$

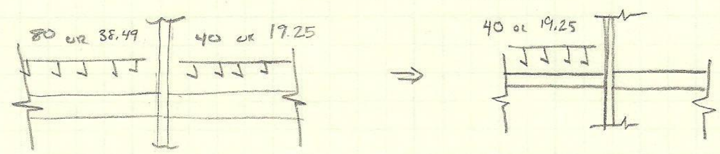
$A_t = 270.36\ ft^2$

Values tabulated using Excel

Live loads reduced where applicable

W14x109 $\phi P_n = 1190^k$ @ 16 ft

Unbalanced load for floors 3-19



Unbalanced moment causes an equivalent vertical load

$$M_{UN} = \frac{w_u l^2}{16} = \frac{[(40)(8.75)/1000](7.17)^2}{16} = 1.12 \text{ ft-k}$$

Additional $P_{eq} = \dfrac{24 M_u}{d} = \dfrac{24(1.12)}{14} = 1.92^k$