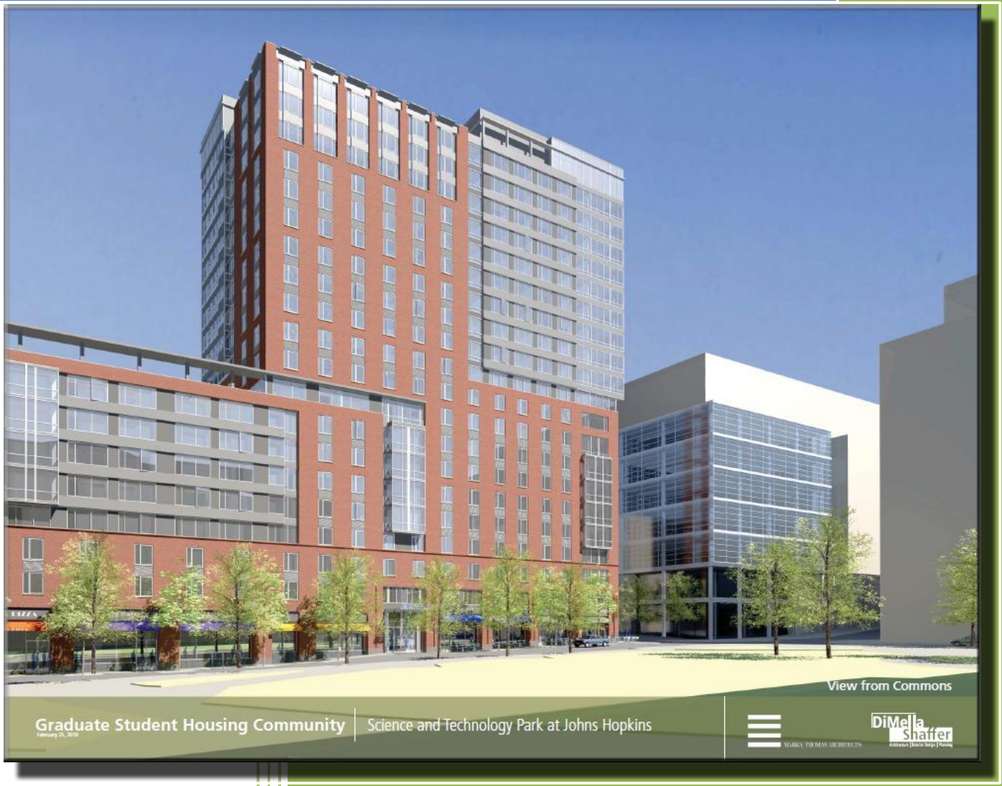
John Hopkins Graduate Student Housing

Technical Report 1



929 North Wolfe Street
Baltimore, Maryland

Brad Oliver – Structural

Advisor: Professor Memari

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Executive Summary -

Technical report 1 summarizes the structural systems found in John Hopkins Graduate Student Housing in Baltimore Maryland. All figures and photos found in this report are provided courtesy of Education Realty Trust and Marks Thomas Architects. This 20 story, primarily residential, building is supported through a concrete structure. Floors loads are supported with an 8" thick 5000 psi two way post tensioned slab. From there, gravity loads move to the varying strength columns typically 20"x 30". Columns take the load to the foundation which consists of 4000 psi caissons reaching depths of 91 feet. Gravity loads were calculated, summarized, and compared to the designed loads as well as building codes.

Lateral loads, wind and seismic, were calculated using ASCE7-05 standards. Wind loads were found to be larger in the East West direction producing a base shear of 600 kips and an overturning moment of 61,000 kip ft. Seismic loads will control this design though due to the weight of concrete construction. It was found that the North-South direction would control the design producing base shear of 1300 kips and an overturning moment of 179,600 kip ft. It was found that this is slightly higher than the designer's seismic calculation which could be due to conservative weight counting.

Spot checks were also performed on gravity members to ensure that they are adequate to carry the loads determined early on. A column with the largest tributary area and a post tensioned tendon with the longest span were chosen for analysis. Axial loads found in the column check on floor two, 2150 kips, were comparable to what the designer estimated at the foundation level, 2400 kips which verifies the weight and load calculations. Calculations show that the column and tendon were both adequate to carry loads and pass code limitations on stress.

Appendixes are also available at the end of this report providing hand calculations for every category. One appendix will also contain additional floor plans and elevations for context.

Introduction –

Located just outside the heart of Baltimore, 2 blocks from John Hopkins campus, is the site for the new John Hopkins Graduate Student Housing. This housing project is being constructed in the science and technology park of John Hopkins. A developing "neighborhood", the science and technology park is over 277,000 sq. ft. which is planned to host at least five more buildings dedicated to research for John Hopkins University. The site is also directly across from a 3 acre green space. This location is ideal because it places graduate students within walking distance of the schools hospitals, shopping, dining and relaxing.

Figure 1 - Showing glass and brick facade along with curtain wall

John Hopkins Graduate Student Housing project is a new building constructed with brick and glass facades for a modern look. Upon completion, the building's main function is predominantly for graduate residential use, providing 929 bedrooms over 20 floors. There are efficiencies, 1, 2, and 4 bedroom apartments available. Other features include a fitness room and rooftop terrace. A secondary function of the building is three separate commercial spaces located on the first floor. Retail spaces provide a mixed use floor, creating a welcoming environment and bringing in additional revenue. At the 10th floor, the typical floor size decreases, creating a low roof and a tower for the remaining ten floors. Glass curtain walls on two corners of the building also begin on the 10th floor and extend to the upper roof.

The façade of John Hopkins GSH is composed mainly of red brick and tempered glass with metal cladding. Large storefront windows will be located on the first floor and approximately 6' x 6' windows in the apartments. The curtain wall is to be constructed of glass and metal cladding that can withstand wind loads without damage. There is a mechanical shading system in the windows to assist in the LEED silver certification.

Figure 2 - an overhead showing the green roof and large green area across the street

John Hopkins GSH is striving to achieve LEED silver certification. Most of the points accumulated to achieve this level come from the sustainable sites category. A total of 20/26 points were picked up in this category due to a number of achievements such as; community connectivity, public transportation access, and storm water design and quality control. Indoor air quality is the next largest category where the building picks up an additional 11 points for the use of low emitting materials throughout construction. Several miscellaneous points are picked up for using local materials and recycling efforts as well. Shading mechanisms are also implemented throughout the design as well as an accessible green roof.

There are three different types of roofs on this project. Above the concrete slab on the green roof is a hot rubberized waterproofing followed by polystyrene insulation, a composite sheet drying system, and finally the shrubbery. The sections of roof containing pavers will be constructed using the same waterproofing, a separation sheet, the insulation and finally pavers placed on a shim system. The remaining portions of the roof will be constructed using a TPO membrane system.

Structural Systems –

Foundations:

A geotechnical report was created based on 7 soil test borings drilled from 80' to 115' deep. Four soil types were found during these tests: man placed fill from previous construction 7-13 feet deep, Potomac group deposits of silty sands at 40-75 feet, and competent bedrock at 80-105 feet. Soil tests showed a maximum unconfined compressive strength of 12.37 ksi. The expected compression loads from the structure were 2400k and 1100k for the 20 and 9 floor towers respectively. The foundation system will also have to support an expected uplift and shear force of 1400k per column and 180k per column. Based on preexisting soils and heavy axial loads it was determined that a shallow foundation system was neither suitable nor economical.

In order to reach the competent bedrock, John Hopkins GSH sits on deep caissons 71-91 feet deep. Caissons range in 36-54" in diameter and are composed of 4000psi concrete. Grade beams, 4000psi, sit on top of the caissons followed by the slab on grade. Slab on grade consists of 3500 psi reinforced with W2.9XW2.9 and rests on 6" of granular fill compacted to at least 95% of maximum dry density based on standard proctor.

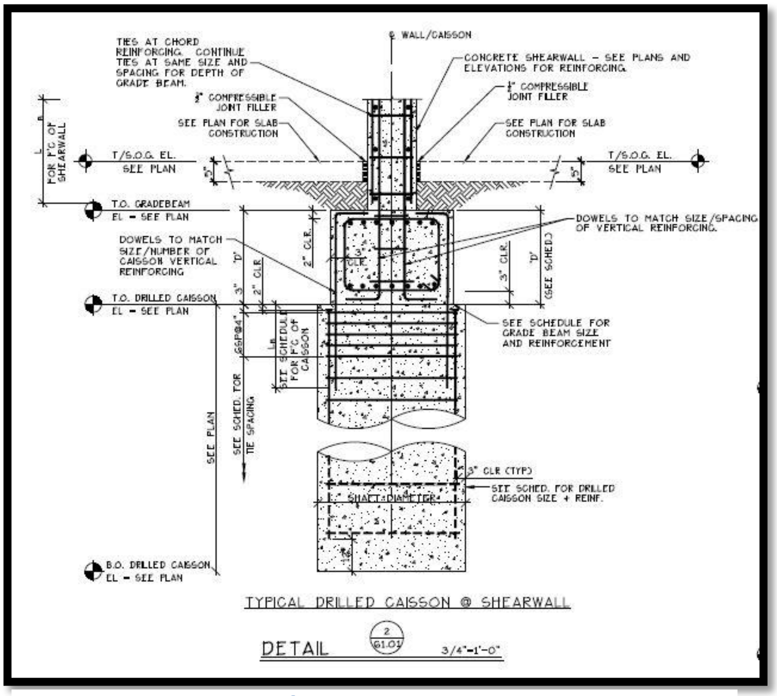


Figure 3 - a detail section of a caisson and column

According to the geotechnical report, the water table is approximately 10 feet below the first floor elevation, therefore a sub drainage system was not necessary.

Floor Framing:

Dead and live loads are supported in John Hopkins GSH through a 2 way post-tensioned slab. The slab is typically 8" thick normal weight 5000 psi concrete reinforced with #4 bars at 24" on center along the bottom in both directions. The tendons are low relaxation composed of a 7 wire strand according to ASTM A-416. Effective post tensioning forces vary throughout the floor, but the interior bands are typically 240k and 260k. This system is typical for every floor except for the 9th which supports a green roof and accessible terrace. Higher loads on this floor require a 10" thick 2 way post tensioned slab reaching a maximum effective strength of 415k. The bottom layer of reinforcing in this area is also increased to #5 bars spaced every 18". One bay on the 9th floor (grid lines 7-8) is constructed with a 10" cast in place slab. Plans of this floor can be found in appendix F.

Mechanical penthouses exist on the 9th and 20th roof constructed with a steel moment frame. Typical sizes for the 9th floor penthouse are W10's and W12's with 1.5" 20 gage "B" metal deck. As for the 20th floor penthouse, the typical beam size is W16x26. Equipment will be supported on concrete pads typically 4" thick. Two air handling units and cooling towers on the roof will require 6" pads.

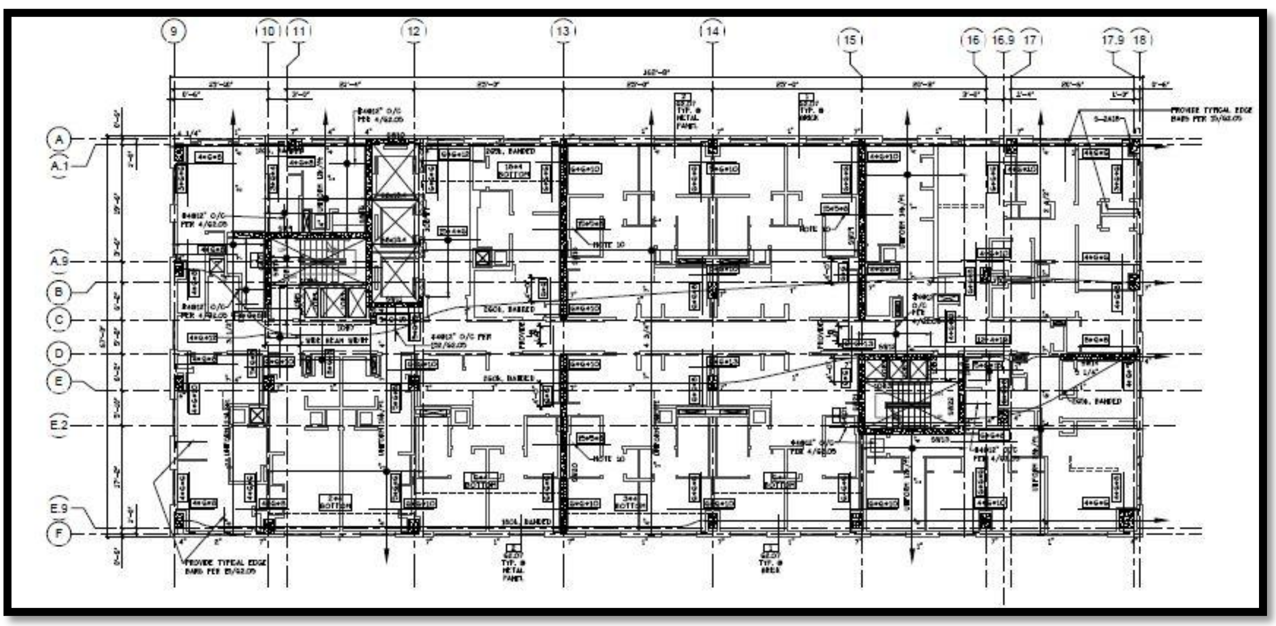


Figure 4 - Typical floor plan of upper tower

The loads will flow through the slab and reinforcement to the columns eventually making their way down to the foundation. To tie the slab and framing system into the columns, two tendons pass through the columns in each direction. To further tie the systems together, bottom bars have hooked bars at discontinuous edges. Dovetail inserts are installed every 2' on center to tie the brick façade in with the superstructure. Columns are typically 30"x20" and composed of 4ksi strength in the northern tower (9 floors), while columns in the southern tower vary from 8ksi at the bottom, and 4 ksi at the top.

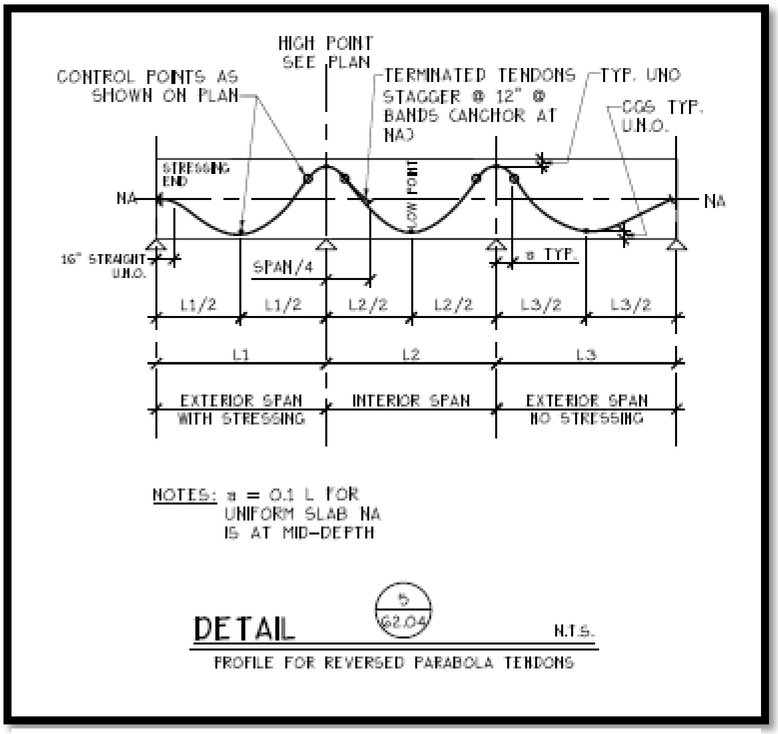


Figure 5- Typical detail for post tensioned tendon profile

Lateral System:

John Hopkins GSH is supported laterally through a cast in place reinforced concrete shear wall system. All of the shear walls are be 12” thick and are located throughout the building and around stairwells and elevator shafts. Shear walls in the 9 floor tower are poured with 4000psi strength concrete while shear walls in the 20 floor tower vary in three locations. From the foundation to 7th floor, 8ksi concrete was required, 6ksi from 7th to below 14th floor, and 4ksi for walls above the 14th floor. The shear walls are tied into the foundation system through bent vertical bars 1’ deep into the grade beam as shown in figure 6. Shear walls are shown below in the figure with N-S walls highlighted in blue and E-W walls red. Walls in the center of the building will support lateral stresses directly, while those on the end support the torsion effects caused by eccentric loads. Elevations of shear walls can be found in appendix F.

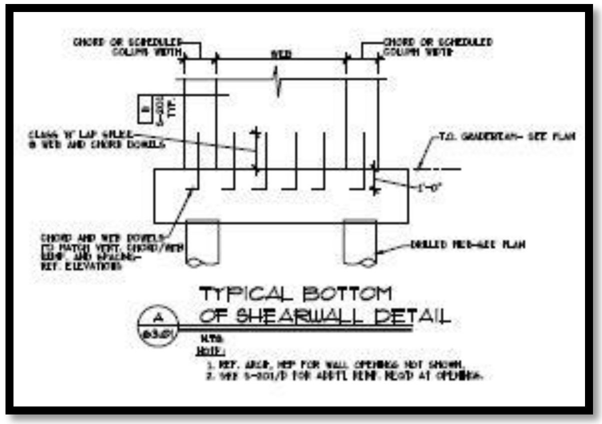


Figure 6 - detail tying shear wall into foundation

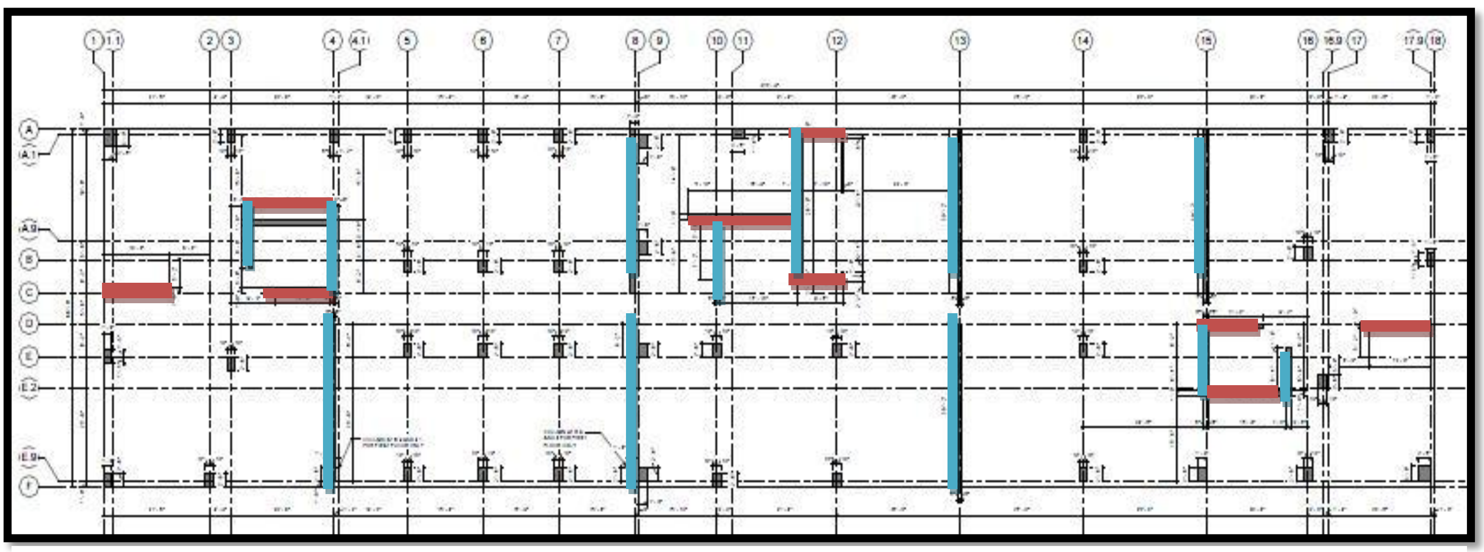


Figure 7 - Shear wall layout

Building Code Summary –

	John Hopkins GSH was designed to comply with:	**My Thesis analysis/design will be based on:**
General Building Code	IBC 2006	IBC 2006
Lateral Analysis	ASCE7	ASCE7-05
Concrete Specifications	ACI 301, 318, 315	ACI 318-08
Steel Specifications	AISC and AWS D1.1	AISC 2006
Masonry Specifications	ACI 530.1/ASCE 6	ACI 530.1-08/ASCE 6-08

Table 1- Building Code Comparison

Material Strength Summary –

Material Strengths		
Concrete		
Material	**Weight (lbs/ft^3)**	**Strength (psi)**
Footings	145	4000
Pile Caps	145	4000
Caissons	145	4000
Grade Beams	145	4000
Slab-on-grade	145	3500
Slabs/beams	145	5000
Slab on metal deck	115	3500
Columns	145	Vary-see schedule
Shearwalls	145	Vary-see schedule
Steel		
Shape	**Grade**	**Yield Strength** (ksi)
W Shapes	A992	50
S, M and HP Shapes	A36	36
HSS	A500-GR.B	42
Channels, Tees, Angles, Bars, Plates	A36	36
Reinforcing Steel	GR. 60	60

Table 2 - Material Strength Summary

Load Calculations

Dead Loads-

The dead loads calculated in appendix A have confirmed the dead loads that were provided in the loading schedule as seen in table 3. It appears that the designer used ASD in their analysis because the total load does not have any factors applied to it. The analysis in this tech report will be LRFD which typically results in a more aggressive design.

LOADING SCHEDULE (PSF)

LOADING \ LOCATION	TYPICAL FLOOR	9TH FLOOR TERRACE	HIGH ROOF	PENTHOUSE ROOF	EXTERIOR MECHANICAL AREAS (9TH + 22ND)	9TH FLR. PLANTER AREAS
CONCRETE SLAB	100	125	115	-	100-115	125
METAL DECK	-	-	-	2	-	-
M/E/C/L	6	6	6	6	6	6
MEMBRANE	-	-	-	1	-	-
ROOFING	-	-	-	6	-	-
INSULATION	-	-	-	8	-	27
PARTITION (LIVE LOAD)	15	-	-	-	-	-
GREEN ROOF	-	30	30	-	-	85
4" TOPPING SLAB	-	50	50	-	50	50
TOTAL DEAD LOAD	108	213	201	23	156-171	295
LIVE LOAD	55	100	30	30	150	30
TOTAL LOAD	163	313	231	53	306-321	325

NOTES:
1. ALL LIVE LOADS ARE IN ACCORDANCE WITH INTERNATIONAL BUILDING CODE 2006 EDITION.
2. NO LIVE LOAD REDUCTION HAS BEEN TAKEN INTO ACCOUNT.
3. TOTAL DEAD LOADS DO NOT INCLUDE WEIGHT OF STEEL OR PRIMARY FRAMING MEMBERS.
4. LOADS IN SCHEDULE DO NOT INCLUDE WEIGHTS OF ROOF TOP MECHANICAL UNITS. THE PROVISION FOR THE SUPPORT OF THESE UNITS HAVE BEEN MADE ON AN INDIVIDUAL BASIS. ANY CHANGE FROM SPECIFIED MECHANICAL UNIT SIZE, WEIGHT AND LOCATIONS SHALL BE BROUGHT TO THE ATTENTION OF THE STRUCTURAL ENGINEER.
5. SEE PLANS FOR LOCALIZED CONCENTRATED LOADS.
6. DRIFTED AND SLIDING SNOW LOADS SHALL BE CALCULATED BY TRUSS MANUFACTURER BASED ON ROOF/BLDG. GEOMETRY AND 'DESIGN CRITERIA' ABOVE.

Figure 8 - Summary of loads used by designer

Live Loads -

It seems John Hopkins used loads very similar to the ASCE7-05 standards. Exterior mechanical loads were not specified in the standard, but I am assuming the equipment can cause significant loads while operating. The 30psf on non-assembly roof areas is most likely a judgment call to account for the maintenance that would be required for a green roof. Although not specified on the table, the 100psf required in the corridor and stairwells are most likely balanced by the large banded post tensioned tendons running parallel to the corridor and around the stairwells.

Area	Designed for – (psf)	ASCE7-05 (psf)
Typical Floor	55 (includes partitions)	40 (residential) + 15 (partitions)
Corridors	N/A	100
Stairs	N/A	100
Assembly	N/A	100
First story retail	N/A	100
Roof used for garden/assembly	100	100
Exterior Mechanical areas	150	N/A
High Roof	30	N/A
Penthouse Roof	30	N/A
Planter Areas	30	N/A

Table 3 - Live Load Comparison

Snow Loads –

Snow loads calculated in this report confirmed those performed by the structural engineer. Drift was calculated on the 9th floor where the floor plan steps back and creates a low roof. A summary is provided below, and more detailed calculations can be found in appendix A.

Ground snow load	25 psf
Exposure factor	.9
Thermal factor	1.0
Snow importance	1.0
Flat roof snow load	16 psf
Drift Height	3.9'
Max Snow load	83 psf

Table 4 - Snow Load Summary

Wind Loads –

Wind loads for this analysis will comply with ASCE7-05 and used a simplified model of the structure. John Hopkins GSH did not comply with ASCE standards for a rigid building so gust factors had to be hand calculated. To make a more accurate model of the building it was split into two sections. This is seen below in the wind distribution where the blue section represents the taller tower while the red represents the lower building. The lateral wind load is resisted through the 12" thick concrete shear walls discussed earlier. A summary of results in provided

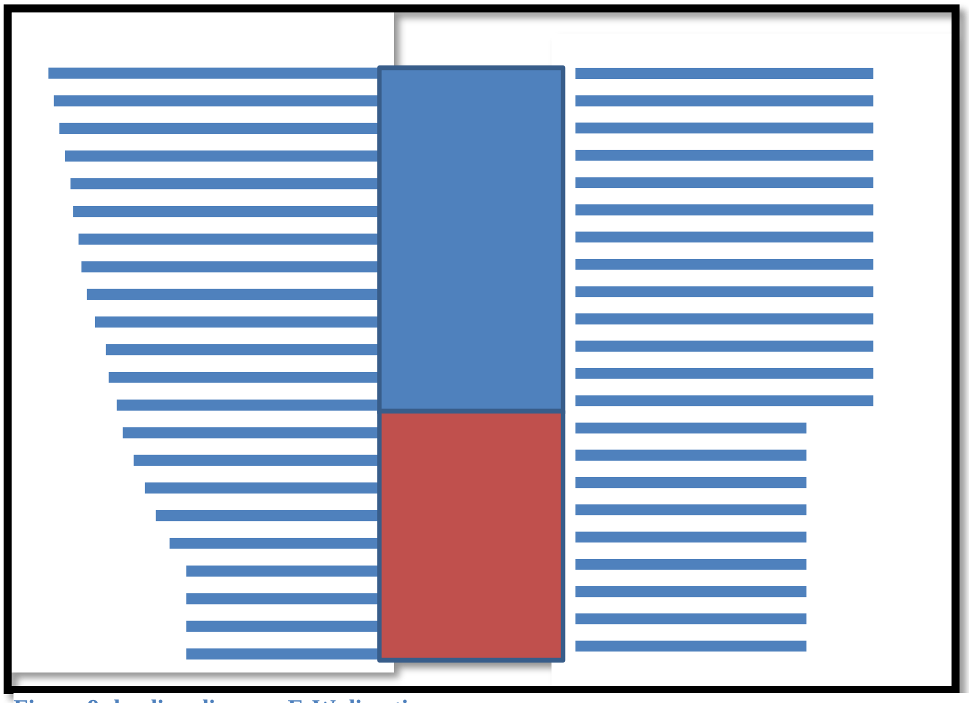


Figure 9- loading diagram E-W direction

in the spreadsheets below while detailed calculations can be found in appendix B. Calculations show that wind in the East-West direction were larger with a base shear of almost 600 kips and an overturning moment of 61,000 kip ft. This makes sense when you look at the geometry of the building. The East-West direction has large amounts of area of façade which produce a larger story force. The highest base shear calculated in this building was 592 K producing an overturning moment of 60968 K ft in the East – West direction. This was to be expected due to the buildings extreme height and large façade facing the East – West direction.

Criteria		E-W Direction					
Tall Tower		**Floor**	**Height (ft)**	**K_z**	**q_z**	**p (windward) (psf)**	**p(leeward) (psf)**
G_f	0.83	Penthouse	208.42	1.21	21.327	18.00	-12.69
C_p (Windward)	0.8	Roof	194.25	1.19	20.974	17.70	-12.69
C_p (Leeward)	-0.5	20	183.9	1.17	20.622	17.40	-12.69
Gc_{pi}	0.18	19	174.6	1.15	20.269	17.11	-12.69
Lower Tower		18	165.3	1.13	19.917	16.81	-12.69
G_f	0.84	17	155.9	1.12	19.741	16.66	-12.69
C_p (Windward)	0.8	16	146.6	1.1	19.388	16.36	-12.69
C_p (Leeward)	-0.5	15	137.2	1.09	19.212	16.21	-12.69
Gc_{pi}	0.18	14	127.9	1.07	18.859	15.92	-12.69
		13	118.6	1.04	18.331	15.47	-12.69
		12	109.3	1	17.626	14.88	-12.69
		11	99.9	0.99	17.449	14.73	-12.69
		10	90.6	0.96	16.921	14.28	-12.69
		9	81.3	0.93	16.392	13.97	-9.84
		8	71	0.89	15.687	13.37	-9.84
		7	61.7	0.85	14.982	12.76	-9.84
		6	52.3	0.81	14.277	12.16	-9.84
		5	43	0.76	13.395	11.41	-9.84
		4	33.7	0.7	12.338	10.51	-9.84
		3	24.3	0.7	12.338	10.51	-9.84
		2	15	0.7	12.338	10.51	-9.84
		1	1	0.7	12.338	10.51	-9.84

Table 5 - chart used for loading diagram

E-W Direction					
Floor	**Height (ft)**	**Height Below (ft)**	**Heigh Above (ft)**	**Trib Area (ft2)**	**Story Force (K)**
Penthouse	208.42	15.2	0	1236.52	22.26
Roof	194.25	10.33	15.2	2076.87	36.77
20	183.9	9.33	10.33	1599.34	27.84
19	174.6	9.33	9.33	1517.99	25.97
18	165.3	9.33	9.33	1517.99	25.52
17	155.9	9.33	9.33	1517.99	25.29
16	146.6	9.33	9.33	1517.99	24.84
15	137.2	9.33	9.33	1517.99	24.61
14	127.9	9.33	9.33	1517.99	24.16
13	118.6	9.33	9.33	1517.99	23.48
12	109.3	9.33	9.33	1517.99	22.58
11	99.9	9.33	9.33	1517.99	22.36
10	90.6	9.33	9.33	1517.99	21.68
9	81.3	10.25	9.33	2099.45	29.32
8	71	9.33	10.25	2656.03	35.50
7	61.7	9.33	9.33	2531.23	32.31
6	52.3	9.33	9.33	2531.23	30.79
5	43	9.33	9.33	2531.23	28.89
4	33.7	9.33	9.33	2531.23	26.61
3	24.3	9.33	9.33	2531.23	26.61
2	15	14	9.33	3164.71	33.27
1	1	1	14	2034.75	21.39
				Base Shear (K)	**592**
				Overturning moment (k ft)	**60968**

Table 6 - Chart used to calculate base shear E-W direction

N-S Direction							
Floor	**Height (ft)**	**Height Below (ft)**	**Heigh Above (ft)**	**Trib Area (ft2)**	**Story Force (K)**		
Penthouse	208.42	15.2	0	509.2	9.38		
Roof	194.25	10.33	15.2	855.255	15.50		
20	183.9	9.33	10.33	658.61	11.73		
19	174.6	9.33	9.33	625.11	10.95		
18	165.3	9.33	9.33	625.11	10.76		
17	155.9	9.33	9.33	625.11	10.66		
16	146.6	9.33	9.33	625.11	10.47		
15	137.2	9.33	9.33	625.11	10.38		
14	127.9	9.33	9.33	625.11	10.19		
13	118.6	9.33	9.33	625.11	9.90		
12	109.3	9.33	9.33	625.11	9.52		
11	99.9	9.33	9.33	625.11	9.42		
10	90.6	9.33	9.33	625.11	9.14		
9	81.3	10.25	9.33	655.93	9.42		
8	71	9.33	10.25	655.93	9.01		
7	61.7	9.33	9.33	625.11	8.20		
6	52.3	9.33	9.33	625.11	7.82		
5	43	9.33	9.33	625.11	7.34		
4	33.7	9.33	9.33	625.11	6.76		
3	24.3	9.33	9.33	625.11	6.76		
2	15	14	9.33	781.555	8.45	**Base Shear (K)**	**207**
1	1	1	14	502.5	5.43	**Overturning moment (k ft)**	**23882**

Criteria		N-S Direction					
Tall Tower		**Floor**	**Height (ft)**	**K_z**	**q_z (psf)**	**p (windward) (psf)**	**p(leeward) (psf)**
G_f	0.855	Penthouse	208.42	1.21	21.327	18.43	-8.94
C_p (Windward)	0.8	Roof	194.25	1.19	20.974	18.12	-8.94
C_p (Leeward)	-0.28	20	183.9	1.17	20.622	17.82	-8.94
GC_{pi}	0.18	19	174.6	1.15	20.269	17.51	-8.94
Lower Tower		18	165.3	1.13	19.917	17.21	-8.94
G_f	0.87	17	155.9	1.12	19.741	17.06	-8.94
C_p (Windward)	0.8	16	146.6	1.1	19.388	16.75	-8.94
C_p (Leeward)	-0.2	15	137.2	1.09	19.212	16.60	-8.94
GC_{pi}	0.18	14	127.9	1.07	18.859	16.29	-8.94
		13	118.6	1.04	18.331	15.84	-8.94
		12	109.3	1	17.626	15.23	-8.94
		11	99.9	0.99	17.449	15.08	-8.94
		10	90.6	0.96	16.921	14.62	-8.94
		9	81.3	0.93	16.392	14.36	-5.80
		8	71	0.89	15.687	13.74	-5.80
		7	61.7	0.85	14.982	13.12	-5.80
		6	52.3	0.81	14.277	12.51	-5.80
		5	43	0.76	13.395	11.73	-5.80
		4	33.7	0.7	12.338	10.81	-5.80
		3	24.3	0.7	12.338	10.81	-5.80
		2	15	0.7	12.338	10.81	-5.80
		1	1	0.7	12.338	10.81	-5.80

Table 8 - chart used to calculate loading diagram

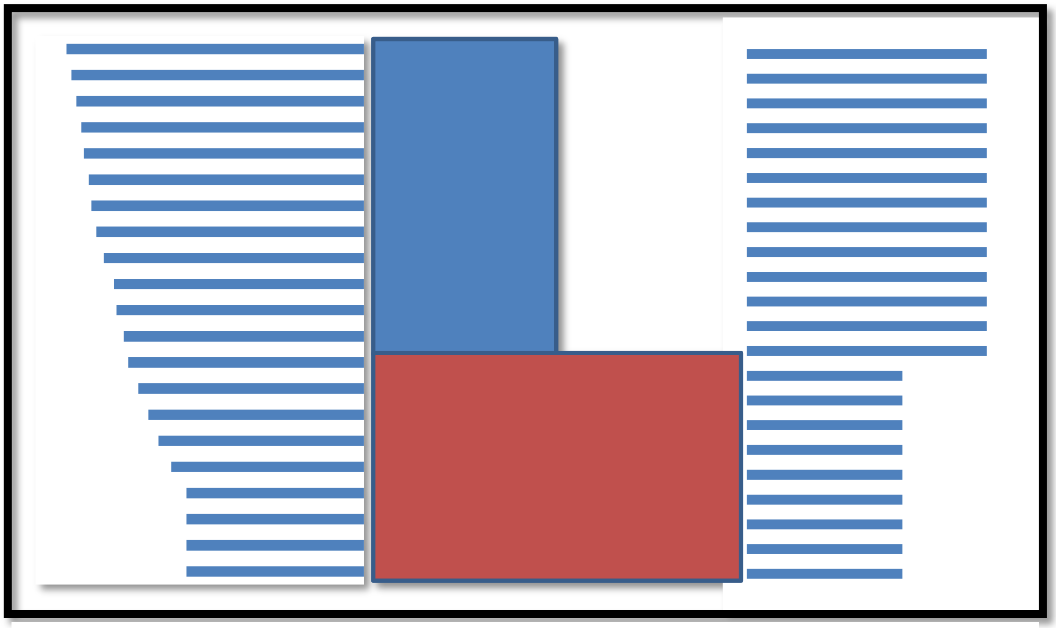


Figure 10- N-S loading diagram

Seismic Loads –

Seismic loads were calculated following ASCE7-05 provisions. Figures in chapter 22 were used to calculate the S_s and S_1 values of 16% and 5% respectively. However the geotechnical report found values of 17% and 5.1% for the site which was used in this calculation since it is most likely more accurate. It was also found that the North South direction controlled for seismic based on C_w values calculated in both directions. After a few calculations, John Hopkins GSH site was classified as seismic category A which would allow a basic procedure described in 11.7. In this calculation though a more precise answer was desired so equivalent lateral force method was used. The weight was calculated per floor and then distributed to each floor based on C_{vx} The total base shear was added up and found to 1345 k which is significantly higher than the 900k found by the structural engineer. Sources of error for this area could be due to double counting of concrete at intersections. Members are monolithically poured which would decrease the weight of concrete calculated. Also the areas of opening in the slab and shear walls were not subtracted out of the concrete count to be conservative. Detailed weight calculations can be found in Appendix C.

Seismic Force Distribution						
Floor	**Height (ft)**	**Weight (k)**	**$(w_x h_x)^k$**	**C_{vx}**	**F_x (K)**	**Overturning Moment (k ft)**
Penthouse	208.42	78.026	5468723.63	0.001	1.31	273.44
Roof	194.25	1545.463	580592502.50	0.104	139.29	27056.39
20	183.9	1590.959	557166693.58	0.099	133.67	24581.26
19	174.6	1549.912	491770018.95	0.088	117.98	20598.88
18	165.3	1549.912	450534269.61	0.080	108.08	17866.44
17	155.9	1554.448	412169358.01	0.074	98.88	15415.55
16	146.6	1554.448	373539273.67	0.067	89.61	13137.34
15	137.2	1554.448	335960714.09	0.060	80.60	11058.09
14	127.9	1554.448	300271966.82	0.054	72.04	9213.46
13	118.6	1554.448	266107686.89	0.047	63.84	7571.46
12	109.3	1554.448	233514673.95	0.042	56.02	6123.10
11	99.9	1529.824	197120200.84	0.035	47.29	4724.26
10	90.6	1534.792	169466694.76	0.030	40.66	3683.41
9	81.3	2557.037	322495821.95	0.058	77.37	6290.02
8	71	2643.302	273809311.53	0.049	65.69	4663.84
7	61.7	2574.841	209728247.52	0.037	50.31	3104.41
6	52.3	2574.841	160991656.00	0.029	38.62	2019.96
5	43	2574.841	117692881.09	0.021	28.23	1214.10
4	33.7	2565.337	79220744.86	0.014	19.01	640.48
3	24.3	2565.337	46946272.77	0.008	11.26	273.68
2	15	2565.337	21695871.23	0.004	5.20	78.07
	Sum	39326.4	5606263584.25		**Base Shear (K)**	**1345**
					Base Overturning moment (k ft)	**179588**

Table 9 Seismic Table used to calculate base shear

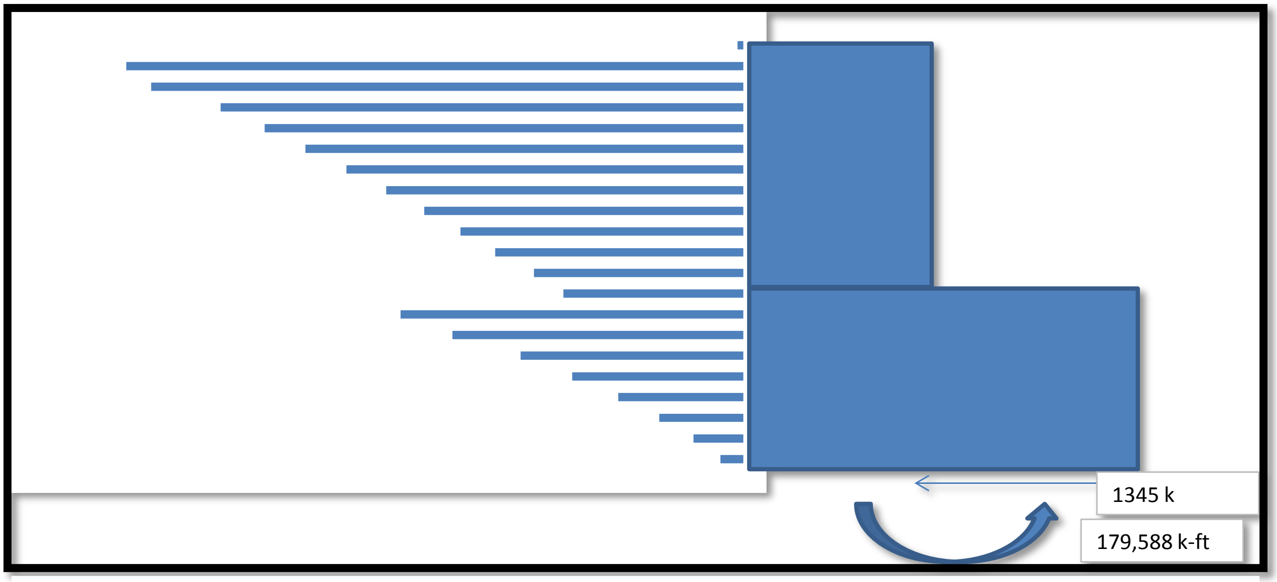


Figure 11- loading diagram seismic

Spot Checks –

Gravity spot checks were performed to test the axial strength of columns and the strength of post tensioned tendons. These checks were performed using Load and Resistance Factored Design (LRFD). In the geo technical reports, compression loads produced by the buildings were expected to be 2400 kips. Column E 14 was analyzed at the second floor and found axial loads to be almost 2150 kips which means the load calculations were accurate. It was found that the column was sufficient to support the loads assuming pure compression and

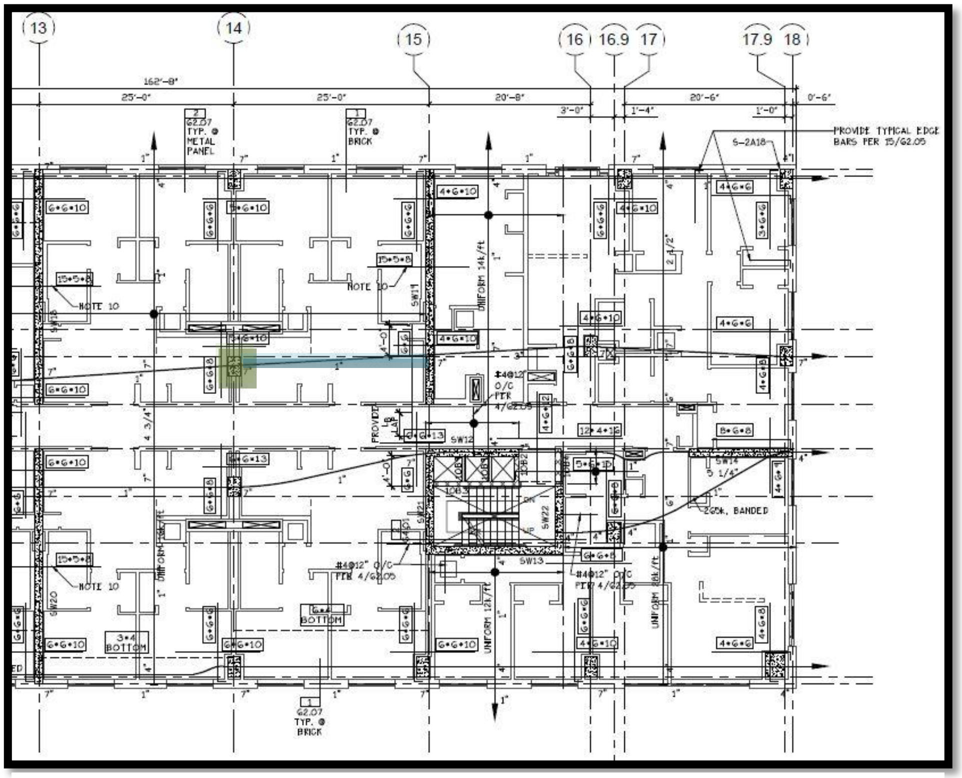


Figure 12 - Area used for spot checks

minimum eccentricity. This column was chosen because it has the largest tributary area in the building and near the bottom of the building, supporting the most weight.

A spot check was also performed for the post tensioning along grid B between 14 and 15 as seen in figure 12. This tendon was chosen for analysis because it has the longest span between supports and is also located near the main corridor supporting the largest loads in the structure. The post tensioning was found to be adequate in design and complied with all codes regarding applied stresses to the slab. A major assumption made during this analysis is that the tendon does not curve. This would require advanced analysis techniques that could be implemented in later tech reports.

Conclusions –

The main goal for technical report 1 is to gain an understanding of the existing structural system and understand how the loads will be resisted. John Hopkins GSH housing is primarily a concrete structure. Vertically, loads travel from the post tensioned slab to reinforced columns and down to the foundation of deep drilled caissons. Laterally, loads will be supported through high strength reinforced concrete shear walls in the four cardinal directions. Dead and live loads were calculated by hand or found in codes, and then compared to the structural engineers design with discrepancies being addressed along the way. It appears that the designer of this structure used ASD analysis, but this tech report and future reports will be based on LRFD calculations which could result in less conservative results.

Lateral loads were calculated using ASCE7-05 standards. Winds were calculated using method 2 as addressed in chapter 6, while seismic loads were determined using chapters 11 and 12. Wind loads were largest in the East-West direction which makes sense due to the large amount of façade exposed in this direction. Seismic loads were found to control the overall design however with a base shear of 1345 kips in the North-South direction. It makes sense that seismic would control the lateral design of this building due to its weight. A pure concrete structure is extremely heavy and creates large inertial forces when ground motion occurs.

Vertically, spot checks were performed in load critical areas on a column and post tensioned tendon. Loads supported by the columns were found to be close to those estimated in the geotechnical reports at 2150 kips. Calculations showed that in pure axial compression, the column strength is 2500 kips at the second floor and adequate to support the load. This takes into account phi factors and a coefficient for minimum eccentricity. The post tensioned tendon was checked for strength per tendon, balanced self weight and eventually stresses in the slab were compared to those allowed by code. Both spot checks proved that the current systems in place are adequate to support Johns Hopkins Graduate Student Housing.

Appendix A – Gravity Loads

Brad Oliver | AE 481 | Load Calculations

Dead Loads

Typical Floor

$\frac{8"}{12'} \times 150\ pcf = 100\ psf$

Superimposed DL - 8 psf (Mech, Elec, Ceiling, lighting etc)

Through online research... (www.bae.ncsu.edu)
green roof typ 30-35 psf

9th floor

$\frac{10"}{12'} \times 150\ pcf = 125\ psf$

High roof - $\frac{9"}{12'} \times 150\ pcf = 112.5\ psf$

4" slabs for Mech equip - $\frac{4"}{12'} \times 150\ pcf = 50\ psf$

Snow Loads - ASCE 7-05 Ch 7 - Flat roof - $p_f = .7 C_e C_t I p_g$

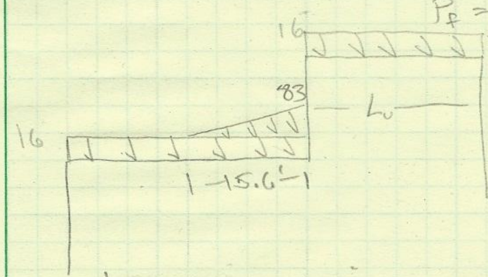
From Fig 7-1 $p_g = 25\ psf$

From table 7-3 $C_t = 1.0$ (All other structures)

Occupancy category II from table 1-1
$\therefore I = 1.0$ from table 7-4

Site Class C from geotechnical report
Fully Exposed roof
$\therefore C_e = .9$ from table 7-2

$p_f = .7(.9)(1.0)(1.0)(25)$
$= 15.75\ psf \cong 16\ psf$



$L_u = 163'$

$h_d = 3.9'$

$h_b = 16\ psf \times \frac{ft^3}{17.25\ lbs} = .93' < 3.9'$

$\gamma = .13(25) + 14 < 30$
$= 17.25 < 30\ pcf$

Leeward
$L_u = 163'$ $h_d = 3.9'$

Wind Word
$L_u = 241' - 163' = 108'$
$h_d = 3.2' \times .75 = 1.8'$ Leeward controls

$W_d = 4(3.9') = 15.6'$

$P_d = 3.9'(17.25\ pcf) = 67\ psf$
Max snow Load $= 67 + 16$
$= 83\ psf$

Appendix B – Wind Loads

Brad Oliver | AE 481 | WIND LOADS | 1

$V = 90$ mph (Fig 6-1c)

$I = 1.0$ from 6-1 Table & category II occupancy

Exposure B b/c urban & suburban area, houses & bldgs near by.

Using table 6-3 K_z & $K_h = 1.21$ case 1 & 2

$$\frac{250-200}{1.28-1.2} = \frac{208-200}{x-1.2} \quad \text{interp is OK}$$
$$x = 1.21$$

$K_{zt} = 1.0$ b/c no Wind speed up effects.

Check if frame is flexible or rigid N-S direction (Tall Tower)

$$T_a = C_t h_n^x \quad \text{Other struc Systems } C_t = .02 \quad x = .75$$
$$T_a = .02(208.4)^{.75}$$
$$= 1.09 \text{ sec}$$

$$freq = 1/1.09 = .912 \text{ Hz} < 1 \text{ Hz} \therefore \text{Flexible}$$

$$G_F = .925\left(\frac{1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_r^2 R^2}}{1 + 1.7 g_v I_z}\right)$$

$$g_Q = g_v = 3.4$$
$$g_r = \sqrt{2\ln(3600 \cdot n_1)} + \frac{.577}{\sqrt{2\ln(3600 n_1)}} \qquad n_1 = .912 \text{ Hz}$$

$$g_r = \sqrt{2\ln(3600 \cdot .912)} + \frac{.577}{\sqrt{2\ln(3600 \cdot .912)}} = 4.167$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_b (.53 + .47 R_L)} \qquad R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(5.49)}{(1+10.3(5.49))^{5/3}} = .048$$

$$R_\ell = \frac{1}{n} - \frac{1}{2n^2}(1 - e^{-2n}) \qquad N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(.912)(498.9)}{82.9} = 5.49$$

For R_h $n = 4.6 n_1 h / \bar{V}_z = 4.6(.912)(208.42)/82.9$
$$= 10.55$$
$$R_h = \frac{1}{10.55} - \frac{1}{2(10.55^2)}(1 - e^{-2(10.55)})$$
$$= .09$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{.6(208.42)}{33}\right)^{1/3} = 498.9$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = .45\left(\frac{125}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 82.9$$

For R_b $n = 4.6 n_1 B / \bar{V}_z = 4.6(.912)(67')/82.9 = 3.39$
$$R_b = 1/3.39 - \frac{1}{2(3.39^2)}(1 - e^{-2(3.39)})$$
$$= .252$$

For R_L $n = 15.4 n_1 L / \bar{V}_z = 15.4(.912)(162.67)/82.9 = 27.56$
$$R_L = 1/27.56 - \frac{1}{2(27.56)^2}(1 - e^{-2(27.56)})$$
$$= .036$$

Brad Oliver | AE 481 | Wind Loads cont. 2

Damping Ratio $\beta = 1.5\%$ for concrete system. C6.5 ISO [REF 6-55]

$$R = \sqrt{\frac{1}{.015}(.048)(.04)(.25)(.53+.47(.036))}$$
$$= .198 \quad .199$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .3\left(\frac{33}{125}\right)^{1/6} = .24$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}} = \sqrt{\frac{1}{1+.63\left(\frac{67+208}{498.9}\right)^{.63}}} = .8354$$

$$G_f = .925\left(\frac{1+1.7(.24)\sqrt{3.4^2(.8354)^2+(4.167)^2(.198)^2}}{1+1.7(3.4)(.24)}\right) = .855$$

$K_d = .85$ MWFRS & C&C from table 6-4

$$q_z = .00256 K_z K_{zt} K_d V^2 I$$
$$= .00256(1.21)(1)(.85)(90^2)(1)$$
$$= 21.3 \text{ psf}$$

Internal Pressure Coefficients - Fig 6-5
$GC_{pi} = \pm .18$

Ext MWFRS $L/B = 162.67/67 = 2.43$

Fig 6-6			use w/
	Windward Wall	$C_p = .8$	q_z
	Leeward Wall	$C_p = -.28$ from interpolation	q_h
	Sidewall	$C_p = -.7$	q_h

Enclosed Flexible Buildings MWFRS

$$P = qG_fC_p - q_i(GC_{pi})$$
$$= 21.3(.855)(.8) - 21.3(\pm .18)$$
$$= 18.4 \text{ psf} \quad \text{Windward}$$

$$P = 21.3(.855)(-.28) - 21.3(\pm .18)$$
$$= -8.9 \text{ psf} \quad \text{Leeward}$$

N-S direction Top Tower

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N-S direction lower tower

Gust Factor

$$L_{\bar{z}} = 320\left(\frac{90.58(.6)}{33}\right)^{1/3} = 377.9$$

$$V_{\bar{z}} = .45\left(\frac{55.35}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 67.6$$

$$N_1 = \frac{.912(377.9)}{67.6} = 5.1$$

$$R_n = \frac{7.47(5.1)}{(1+10.3(5.1))^{5/3}} = .05$$

For R_h $n = 4.6(.912)(90.58/67.6) = 5.62$

$$R_h = 1/5.62 - \frac{1}{2(5.62^2)}\left(1-e^{-2(5.62)}\right) = .162$$

For R_b $n = 4.6(.912)(67)/67.6 = 4.16$

$$R_b = 1/4.16 - 1/2(4.16^2)\left(1-e^{-2(4.16)}\right) = .211$$

For R_L $n = 15.4(.912)(271.3)/67.6 = 56.4$

$$R_L = 1/56.4 - 1/2(56.4^2)\left(1-e^{-2(56.4)}\right) = .018$$

$$R = \sqrt{\tfrac{1}{.015}(.05)(.162)(.211)(.53+.47(.018))} = .248$$

$$I_{\bar{z}} = .3\left(\frac{33}{55.35}\right)^{1/6} = .275$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{67+90.58}{377.9}\right)^{.63}}} = .857$$

$$G_F = .925\left(\frac{1+1.7(.275)\sqrt{3.4^2(.857)+4.167^2(.248^2)}}{1+1.7(3.4)(.275)}\right) = .87$$

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E-W Direction Upper Tower

Gust Factor

$$\left.\begin{array}{l} L_{\bar{z}} = 498.9 \\ \bar{V}_{\bar{z}} = 82.9 \\ N_1 = 5.49 \end{array}\right\} R_N = .048$$ Same as previous upper tower

$$R_h = .09$$

For R_b $n = 4.6(.912)(162.7)/82.9 = 8.23$

$$R_b = 1/8.23 - 1/2(8.23)^2\left(1 - e^{-2(8.23)}\right) = .114$$

For R_L $n = 15.4(.912)(67)/82.9 = 11.35$

$$R_L = 1/11.35 - 1/2(11.35^2)\left(1 - e^{-2(11.35)}\right) = .084$$

$$R = \sqrt{\tfrac{1}{.015}(.048)(.09)(.114)(.53 + .47(.084))} = .137$$

$$I_{\bar{z}} = .24$$

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{162.7 + 208}{498.9}\right)^{.63}}} = .81$$

$$G_F = .925\left(\frac{1 + 1.7(.24)\sqrt{3.4^2(.81^2) + 4.167^2(.137^2)}}{1 + 1.7(3.4)(.24)}\right) = .83$$

$L/B = 67/162.7 = .41 \therefore C_p$ leeward $= -.5$

TOPS.

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E-W Direction Lower Tower
Gust Factor

$V_{\bar{z}} = 67.6$

$L_{\bar{z}} = 377.9$

$N_1 = 5.1$

$R_n = .05$

$R_h = .162$

For R_b $n = 4.6(.412)(271.3)/67.6 = 16.84$

$R_b = 1/16.84 - 1/2(16.84^2)(1 - e^{-2(16.84)}) = .058$

For R_L $n = 15.4(.412)(67)/67.6 = 13.42$

$R_L = 1/13.42 - 1/2(13.42^2)(1 - e^{-2(13.42)}) = .069$

$$R = \sqrt{\frac{1}{.05}(.05)(.162)(.058)(.53 + .47(.069))} = .133$$

$I_{\bar{z}} = .275$

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{271.3 + 90.58}{377.9}\right)^{.63}}} = .787$$

$$G_f = .925\left(\frac{1 + 1.7(.275)\sqrt{3.4^2(.787^2) + 4.167^2(.275)^2}}{(1 + 1.7(3.4)(.275))}\right) = .84$$

$L/B = 67/271.3 = .24$ $\therefore$ C_p leeward $= -.5$

TOPS

Appendix C – Seismic Loads

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From geo-technical report $\left\{ \begin{array}{l} S_s = 17.1\%g \\ S_1 \;\; 5.1\%g \end{array} \right.$

Site class C based on geotechnical report

Using 11.4.3 $S_{ms} = F_a S_s = 1.2(.171) = .2052$

$S_{M1} = F_v S_1 = 1.7(.051) = .0867$

$S_{ds} = 2/3\, S_{ms} = 2/3\,(.2052) = .1368$

$S_{d1} = 2/3\, S_{m1} = 2/3\,(.0867) = .0578$

Importance = 1.0 based on II occupancy

Table 11.6-1
$S_{ds} = .1368 < .167 \therefore$ Category A ✓

Table 11.6-2
$S_{D1} = .0578 < .067 \therefore$ Category A ✓

To be more conservative & accurate will use equivalent lateral method

$V = C_s W$

$C_s = \frac{S_{DS}}{\frac{R}{I}} = \frac{.1368}{4/1} = .0342$

R=4 from table 12.2-1
Bearing wall system
ordinary reinforced shear wall

$C_s > .01$ ✓

$C_u = 1.7$ b/c $S_{d1} = .0578 < .1$ table 12.8-1

$T_a = C_t h_n^x = .02(208.4)^{.75} = 1.1$ seconds table 12.8-2
All other struc systems

$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left(1 + .83\left(\frac{h_i}{D_i}\right)^2\right)}$ Calculation available in spreadsheet upon request

$T_a = \frac{.0019}{\sqrt{C_w}} h_n = \frac{.0019}{\sqrt{.0035}} 208.4 = 1.7$ sec N-S direction

$\therefore K = 1.6$ through interp

$\frac{2.5 - .5}{2 - 1} = \frac{1.7 - .5}{x - 1}$

$T_a = \frac{.0019}{\sqrt{.487}} 208.4 = .57$ sec E-W direction

$\therefore K = 1.035$ from interp

N-S direction controls

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Weight Calculation

Penthouse Roof
decking - 2 psf
Super imposed - 8 psf
membrane - 1 psf
roofing - 6 psf
insulation - 6 psf
23 psf × 3392 ft² = 78,026 lbs

High Roof -
Slab 9"/12' (150) = 112.5 psf
Superimposed = 8 psf
120.2 psf × 10,805 ft² = 1,298,761 lbs

Green Roof = 30 psf × (10,805 - 3392) = 222,390 lbs
toppings (4"/12')(150) = 50 psf × 461 ft² = 23,050 lbs

HSS col 4×4×1/4 - 12.21 lbs/ft × 15' = 183 lbs × 4 col = 733 lbs
HSS col 6×6×1/2 - 35.24 lbs/ft × 15' = 529 lbs

Floor 20 -
Slab - 8"/12' (150) = 100 psf
Super imposed = 8 psf
108 psf × 10,805 ft² = 1,166,940 lbs

Columns - 30"/12' × 20"/12' × 10.33' × 150 pcf = 6,458 lbs × 10 = 64581 lbs
36"/12" × 30"/12' × 10.33' × 150 pcf = 11,625 lbs

Shear Wall #9 - 1' (22.17)(10.33') × 150 pcf = 34352 lbs
Shear Wall #10 - 8"/12' (9'6) (10.33') × 150 pcf = 9,816 lbs
#11 - = 9,816 lbs
#12 - 1' (11') (10.33') × 150 pcf = 17,044 lbs
#13 - 1' (16.7')(10.33') × 150 pcf = 25,877 lbs
#14 1' (12.25')(10.33')(150) = 18,981 lbs
#15 = 17,044 lbs
#16 1' (26.67')(10.33')(150) = 41,325 lbs
#18 1' (30.17)(10.33)(150) = 46,748 lbs
#19 = 46,748 lbs
#20 = 46,748 lbs
#21 1' (13.5')(10.33)(150) = 20,918 lbs
#22 1' (8')(10.33)(150) = 12,396 lbs

TOPS.

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Floor 19-18

Slab - $8''/12'(150) = 100$ psf
SDL = 8 psf
108 psf × 10805 ft² = 1166940 lbs

Columns: used excel spread sheet 68,831 lbs
same dim different length

Shear Walls used excel spreadsheet 314141 lbs

Floor 17-12

Slab - 100 psf
SDL - 8 psf
108 psf × 10,844 = 1171476 lbs

Col - Same as Floor 18-19 68,831 lbs
Shear Walls " " 314,141 lbs

Floor 11

Slab & SDL = 108 psf × 10,619 = 1146852 lbs

Col = 68,831 lbs
Shear wall = 314,141 lbs

Floor 10

Slab & SDL = 108 psf × 10,665 = 115 1820 lbs
Col = 68,831 lbs
Shear = 314,141 lbs

Floor 9

SLAB & SDL grid 9-18 = 108 psf × 10,665 = 1,151,820 lbs
Col = 68,831 lbs
Shear Wall = 314,141 lbs

Slab grid 7-8 = $10''/12'(15.67)(67)(150) = 131,236$ lbs
Slab grid 1-7 = $10''/12'(42.33')(67')(150) = 773264$ lbs
green roof - 30 psf × (26.3×43 + 14×42.3) = 86,538 lbs
Planter Areas - 50 psf × 300 ft² = 15,000 lbs
4" Pads - 50 psf × 313 ft² = 15,650 lbs
HSS 4x4x1/4 12.21 lb/ft (9.33) = 114 lbs × 2 = 228 lbs
6x6x1/2 35.24 lb/ft (9.33) = 329 lbs

Floor 8

Slab & SDL - 108 psf × 17,930 = 1936440 lbs
Col = 126574 lbs
Shear walls = 580283 lbs

Floor 7-5

Slab & SDL - 108 psf × 17,930 = 1936440 lbs
Col = 114293 lbs
Shear walls = 524108 lbs

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Floor 4-2

Slab + SDL = 108 psf (17,842) = 1926936 lbs
Col = 114293 lbs
Shear walls = 524108 lbs

Floor 1

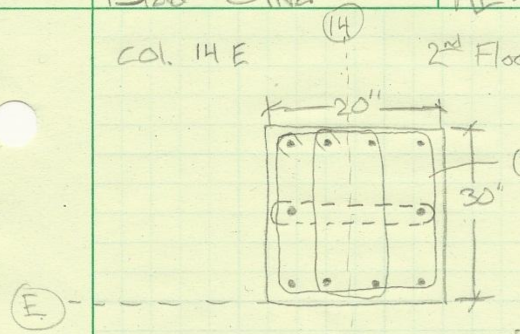
Slab 5"/12' (150) = 62.5 psf
SDL = 8 psf
70.5 psf × 17068 ft² = 1203294 lbs

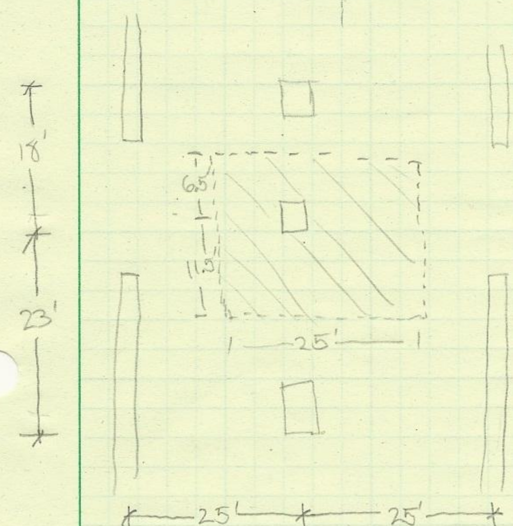
Col = 171500 lbs
Shear walls = 786443 lbs

Appendix D – Column Spot Check

Brad Oliver | AE 481 | Column Spot check 1

Col. 14 E 2nd Floor $F'_c = 8$ ksi (10) #10's





$A_t = (6.5' + 11.5') \times (25') = 450\ ft^2$

Dead Loads –

Slab – $8"/12' (150) = 100$ psf

Self – $\left(\frac{30"}{12'}\right)\left(\frac{20"}{12}\right)(150)(180.25') =$ 113 Kips

SDL – 8 psf

Live Load – 40 psf residential
+20 psf partitions
60 psf

Snow load – 16 psf

roof dead load – $9"/12' \times (150) = 112.5$ psf
Roof Live Load – 30 psf

Using LRFD load case $1.2D + 1.6L + .5S$

19 typical floors + roof + snow + self weight

*I did not use LL reduction to remain conservative in my spot check.

$P_d = \underbrace{(100+8) \times 450 ft^2 \times 19}_{typical} + \underbrace{(112+8) \times 450}_{roof} + \underbrace{113{,}000}_{self}$

$= 1090$ Kips

$P_L = \underbrace{60 psf \times 450 ft^2 \times 19}_{typical} + 30 psf \times 450 ft^2$

$= 526$ Kips

$P_s = 16 psf \times 450 ft^2 = 7.2K$

$P_u = 1.2(1090) + 1.6(526) + .5(7.2)$

$= 2153$ Kips.

Brad Oliver | AE 481 | Column spot check 2

Column will be checked for pure Axial load b/c
it is an interior column sufficiently far from shearwalls
& will not be participating in lateral resistance.

$\Phi P_o = .85 f'_c A_c + A_s f_y$

$= [.85(8000)(600)+(60{,}000)(12.7)].65$

$= 3147$ Kips

$A_c = 20(30) = 600\ in^2$

$A_s = (10)(1.27) = 12.7\ in^2$

$f_y = 60{,}000$ psi

$\Phi = .65$ b/c not spiral reinforced

Pure compression $= .8\Phi P_o$

$.8(3147) = 2517K > 2153K$ ∴ Okay

Ties are at least #3 for #10 bar ✓

Spaced 18" OC

Max spacing = 16 × long bar Φ = 16(1.27) = 20" ✓
= 48 × tie bar Φ = 48(3×3/8) = 54" ✓
= least dim of col = 20" ✓

Appendix E – Post Tensioning Spot Check

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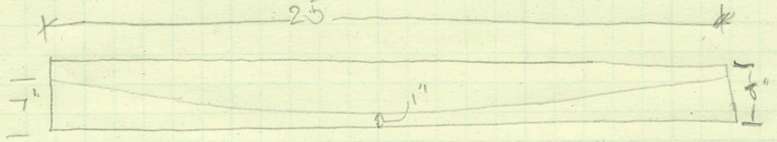

8" thick
$f'_c = 5000$ psi
$f'_{ci} = 3500$ psi
Additional #4 @ 24" both ways

Preliminary Thickness

$L/H = 45 \qquad H = L/45 = (25 \times 12)/45 = 7''$

designed was 8"
close enough.

Dead Load - $(8''/12'')(150\text{ pcf}) = 100$ psf
SDL - 8 psf
108 psf

LL - 55 psf

Assume targeted load balancing of 75% of DL

$.75(108) = 81$ psf

tendon ordinate	Tendon CG Location	
int support	7"	
int midspan	3"	$a = 7 - 3 = 4''$

Weight to balance

$81\text{ psf}\left(4' + \frac{23'}{2}\right) = 1661\text{ lb/ft} = 1.66\text{ K/ft}$

Force req in tendon

$P = w_d L^2 / 8a$

$= 1.66(25^2)/8/\left(\frac{4}{12}\right)$

$= 389$ K

Tendon properties

7 Wire strand A-416
1/2" diameter tendon Area - .153 in²
Ultimate Strength - 270 Ksi

Brad Oliver | AE 481 | P.T. Check (2)

Estimate losses @ 15 ksi

$f_{se} = .7(270) - 15 = 174$ ksi

$P_{eff} = .153(174) = 26.6$ Kips/tendon $\approx$ 27K/tendon designed for.

# of tendons to balance load

$389/26.6 = 14.6 \approx 15$ tendons

$P_{actual} = 15(26.6) = 399^{K}$

399 >> 260K designed.

This could be due to smaller self weight balance, or the fact there is another 260 K tendon running // 18' away to make up for this difference.

Balanced load adjustment

$\frac{399}{389}(1.66) = 1.7$ K/ft

$\frac{P_{Act}}{A} = \frac{399^{K}}{(9\times12+\frac{23}{2}\times12)(8")} = 203$ psi > 125 psi min ✓

< 300 psi max ✓

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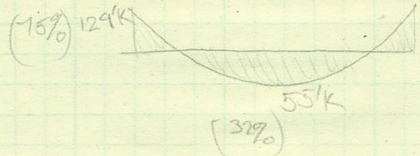
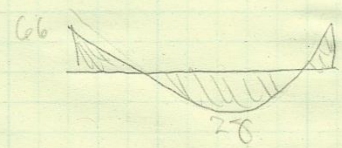
Check slab stresses

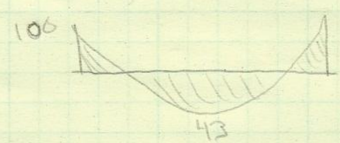
$w_{dL} = 108 \text{ psf } (9+\frac{23}{2}) = 2.21 \text{ k/ft}$ $\quad M_o = \frac{2.21(25^2)}{8} = 173 \text{ 'k}$



$w_{LL} = 55 \text{ psf } (9+\frac{23}{2}) = 1.13 \text{ k/ft}$ $\quad M_o = 88 \text{ 'k}$



$w_b = -1.7 \text{ k/ft}$ $\quad M_o = 133 \text{ 'k}$



Stresses immediately After jacking

Mid span

$f_{top} = (-M_{dL} + M_{bal})/S - P/A$

$(-55+43)(12000)/2624 - 203$

$= -256 \text{ psi comp} < .6(3500) = 2100 \text{ psi}$ ✓

$f'_{ci} = 3500 \text{ psi}$

$S = bh^2/6 = (108+138)(8^2)/6 = 2624 \text{ in}^3$

$f_{bot} \quad (M_{dL} - M_{bal})/S - P/A$

$(55 - 43)(12000)/2624 - 203$

$= -148 \text{ psi comp} < 2100 \text{ psi}$ ✓

Support

$f_{top} = (-129+100)(12000)/2624 - 203$

$= -132 \text{ comp} < 2100 \text{ psi}$ ✓

$f_{bot} = (129-100)(12000)/2624 - 203$

$= -70 \text{ psi comp} < 2100 \text{ psi}$ ✓

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Stresses @ service load

Midspan

$f_{top} = (-M_{DL} - M_{LL} + M_{BAL})/S - P/A$

$= (-55 - 28 + 43)(12000)/S - P/A$

$= -386$ psi $< .45(5000) = 2250$ psi ✓

$f_{bot} = (M_{DL} + M_{LL} - M_{bal})/S - P/A$

$= (55 + 28 - 43)(12000)/2624 - 203$

$= -20$ psi < 2250 psi ✓

Support

$f_{top} = (-129 - 66 + 100)(12000)/2624 - 203$

$= -637$ psi < 2250 psi ✓

$f_{bot} = (129 + 66 - 100)(12000)/2624 - 203$

$= 221$ psi tension $< 6\sqrt{5000} = 424$ psi ✓

Appendix F – Supplemental Drawings

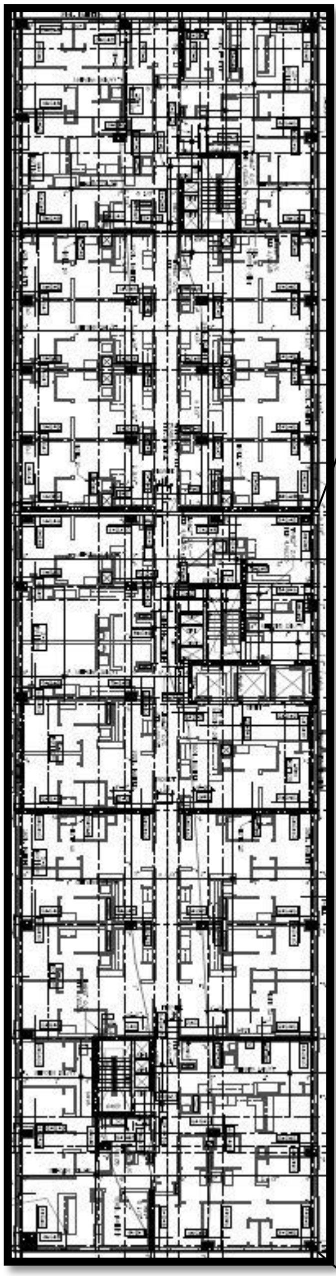

Figure 13 - typical lower floor

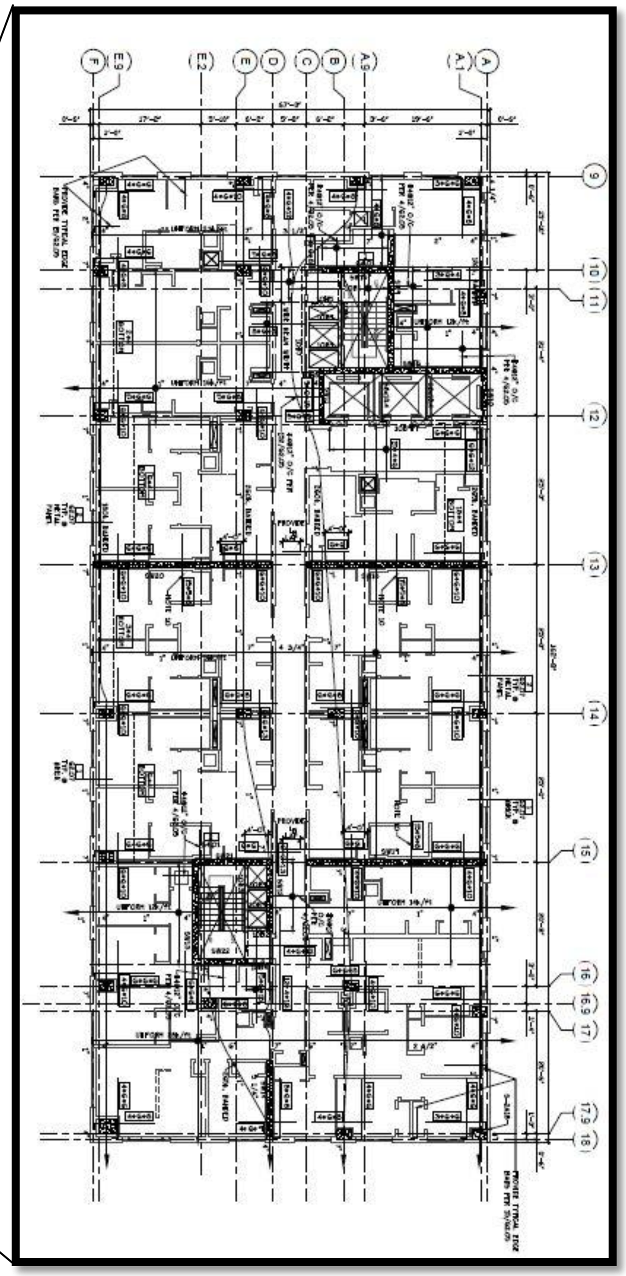

Figure 14 - typical upper floor layout

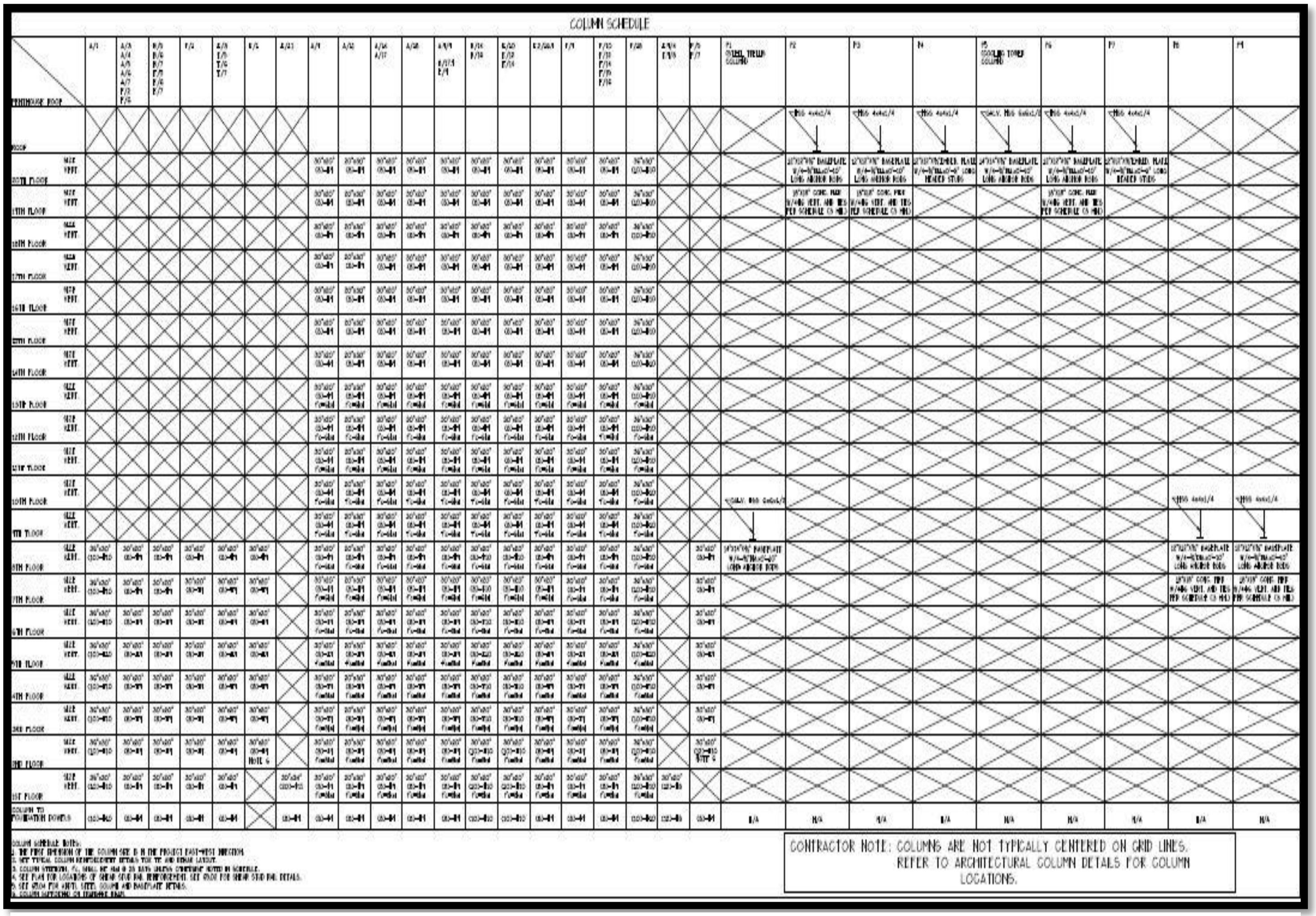

Figure 15 - column schedule

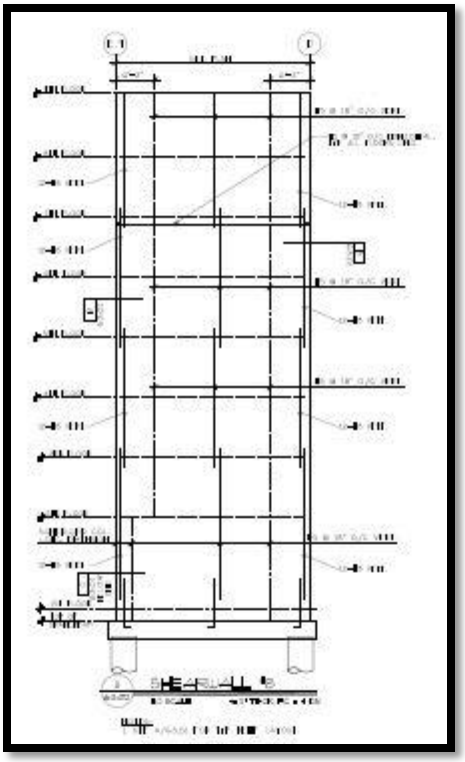

Figure 17 - typical 9 floor shear shear wall

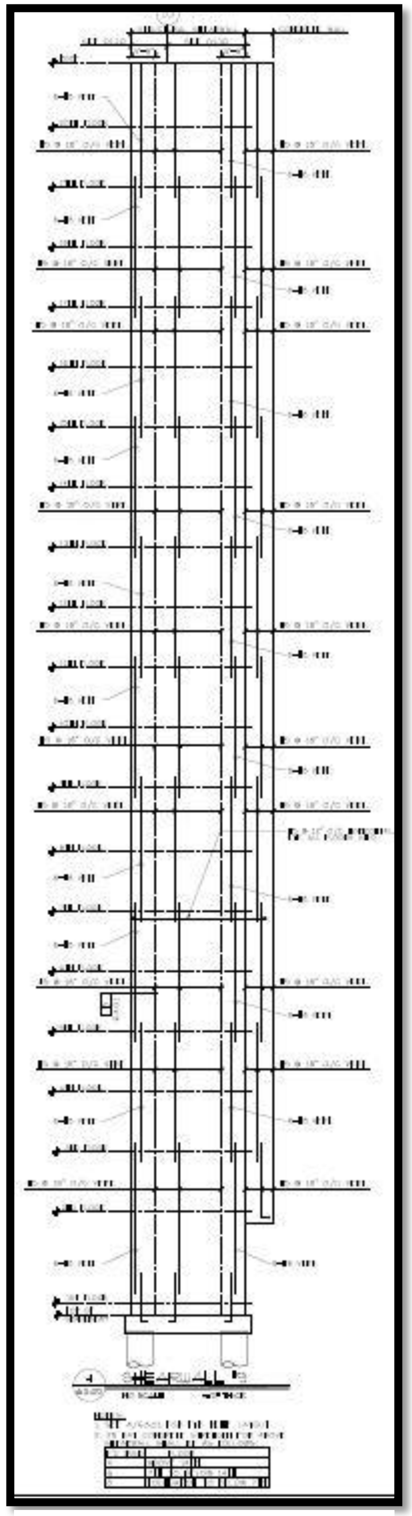

Figure 16 - 22 floor shear wall

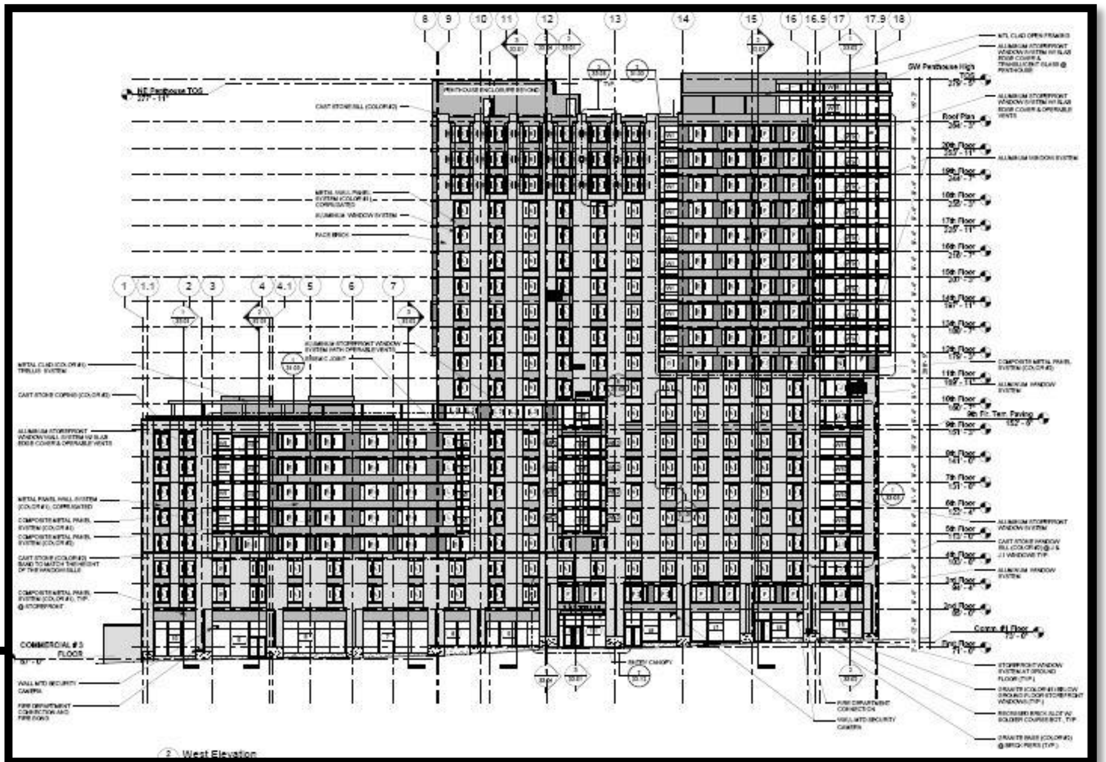

Figure 179 - West Elevation

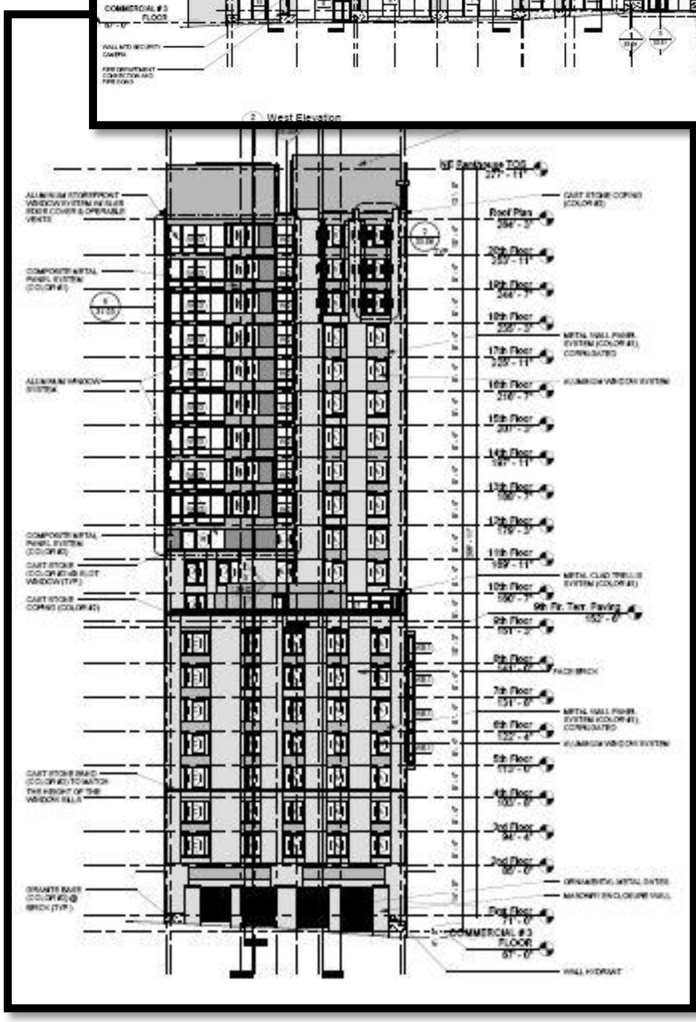

Figure 20 - South Elevation