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.

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.
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.

According to the geotechnical report, the water table is approximately 10 feet below the first floor elevation, therefore a subdrainage 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.
Building Code Summary –

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

Material Strength Summary –

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<tr>
<th>Material</th>
<th>Weight (lbs/ft³)</th>
<th>Strength (psi)</th>
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<tr>
<td>Footings</td>
<td>145</td>
<td>4000</td>
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<td>Pile Caps</td>
<td>145</td>
<td>4000</td>
</tr>
<tr>
<td>Caissons</td>
<td>145</td>
<td>4000</td>
</tr>
<tr>
<td>Grade Beams</td>
<td>145</td>
<td>4000</td>
</tr>
<tr>
<td>Slab-on-grade</td>
<td>145</td>
<td>3500</td>
</tr>
<tr>
<td>Slabs/beams</td>
<td>145</td>
<td>5000</td>
</tr>
<tr>
<td>Slab on metal deck</td>
<td>115</td>
<td>3500</td>
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<td>Columns</td>
<td>145</td>
<td>Vary-see schedule</td>
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<tr>
<td>Shearwalls</td>
<td>145</td>
<td>Vary-see schedule</td>
</tr>
</tbody>
</table>

Steel

<table>
<thead>
<tr>
<th>Shape</th>
<th>Grade</th>
<th>Yield Strength (ksi)</th>
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</thead>
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<tr>
<td>W Shapes</td>
<td>A992</td>
<td>50</td>
</tr>
<tr>
<td>S, M and HP Shapes</td>
<td>A36</td>
<td>36</td>
</tr>
<tr>
<td>HSS</td>
<td>A500-GR.B</td>
<td>42</td>
</tr>
<tr>
<td>Channels, Tees, Angles, Bars, Plates</td>
<td>A36</td>
<td>36</td>
</tr>
<tr>
<td>Reinforcing Steel</td>
<td>GR. 60</td>
<td>60</td>
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</table>

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.

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.

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

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.

<table>
<thead>
<tr>
<th>Ground snow load</th>
<th>25 psf</th>
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<tbody>
<tr>
<td>Exposure factor</td>
<td>.9</td>
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<tr>
<td>Thermal factor</td>
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</tr>
<tr>
<td>Snow importance</td>
<td>1.0</td>
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<tr>
<td>Flat roof snow load</td>
<td>16 psf</td>
</tr>
<tr>
<td>Drift Height</td>
<td>3.9’</td>
</tr>
<tr>
<td>Max Snow load</td>
<td>83 psf</td>
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</table>

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 is provided

![Figure 9: loading diagram E-W direction](image)
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.

<table>
<thead>
<tr>
<th>Criteria</th>
<th>E-W Direction</th>
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<tr>
<td><strong>Tall Tower</strong></td>
<td><strong>Floor</strong></td>
</tr>
<tr>
<td>Gi</td>
<td>Penthouse</td>
</tr>
<tr>
<td>Cp (Windward)</td>
<td>Roof</td>
</tr>
<tr>
<td>Cp (Leeward)</td>
<td>20</td>
</tr>
<tr>
<td>Gcpi</td>
<td>19</td>
</tr>
<tr>
<td><strong>Floor</strong></td>
<td><strong>Height (ft)</strong></td>
</tr>
<tr>
<td>Lower Tower</td>
<td>18</td>
</tr>
<tr>
<td>Gi</td>
<td>17</td>
</tr>
<tr>
<td>Cp (Windward)</td>
<td>16</td>
</tr>
<tr>
<td>Cp (Leeward)</td>
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<td>Gcpi</td>
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<tr>
<td></td>
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Table 5 - chart used for loading diagram
### E-W Direction

<table>
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<tr>
<th>Floor</th>
<th>Height (ft)</th>
<th>Height Below (ft)</th>
<th>Heigh Above (ft)</th>
<th>Trib Area (ft²)</th>
<th>Story Force (K)</th>
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</thead>
<tbody>
<tr>
<td>Penthouse</td>
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<td>15.2</td>
<td>0</td>
<td>509.2</td>
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Table 6 - Chart used to calculate base shear E-W direction

### N-S Direction

<table>
<thead>
<tr>
<th>Floor</th>
<th>Height (ft)</th>
<th>Height Below (ft)</th>
<th>Heigh Above (ft)</th>
<th>Trib Area (ft²)</th>
<th>Story Force (K)</th>
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<tbody>
<tr>
<td>Penthouse</td>
<td>208.42</td>
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Base Shear (K) | 592
Overturning moment (k ft) | 60968
### Table 8 - Chart Used to Calculate Loading Diagram

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Floor</th>
<th>Height (ft)</th>
<th>Kz</th>
<th>qz (psf)</th>
<th>p(Windward)(psf)</th>
<th>p(Leeward)(psf)</th>
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<tr>
<td><strong>Tall Tower</strong></td>
<td></td>
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<tr>
<td>Gr</td>
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<td>Penthouse</td>
<td>208.42</td>
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<td>Roof</td>
<td>194.25</td>
<td>1.19</td>
<td>20.974</td>
<td>18.12</td>
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<td>Cp (Leeward)</td>
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<td>183.9</td>
<td>1.17</td>
<td>20.622</td>
<td>17.82</td>
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<td>19</td>
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<td>1.15</td>
<td>20.269</td>
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<td>1</td>
<td>0.7</td>
<td>12.338</td>
<td>10.81</td>
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</table>

**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.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Height (ft)</th>
<th>Weight (k)</th>
<th>$C_{vx}$</th>
<th>$F_x (K)$</th>
<th>Overturning Moment (k ft)</th>
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<td>Sum</td>
<td>39326.4</td>
<td>5606263584.25</td>
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<td></td>
<td>Base Overturning moment (k ft) 179588</td>
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</table>

Table 9 Seismic Table used to calculate base shear
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
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

<table>
<thead>
<tr>
<th>Brad Oliver</th>
<th>AE 481</th>
<th>Load Calculations</th>
</tr>
</thead>
</table>

### Dead Loads

- **Typical Floor**
  \[ \frac{8}{12} \times 150 \text{ psf} = 100 \text{ psf} \]

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

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

- **9th Floor**
  \[ \frac{9}{12} \times 150 \text{ psf} = 125 \text{ psf} \]

- **High roof**
  \[ \frac{9}{12} \times 150 \text{ psf} = 12.5 \text{ psf} \]

- **1st Floor for Mech equip**
  \[ \frac{4}{12} \times 150 \text{ psf} = 50 \text{ psf} \]

### Snow Loads - ASCE7-05 Ch 7

- **Flat roof**
  \[ P_s = \frac{1}{64} \times 0.75 \times 0.9 \times 1.5 \times 1.5 \]

- **From Fig 7-1**
  \[ P_s = 2.5 \text{ psf} \]

- **From table 7-3**
  \[ C_t = 1.0 \text{ (All other structures)} \]

- **Occupancy category II** from table 7-1
  - \[ I = 1.0 \text{ from table 7-1} \]

- **Site Class C from geotechnical report**
  - **Fully Exposed roof**
  - \[ C_e = 0.9 \text{ from table 7-2} \]

- **From table 7-2**
  \[ P_s = \frac{1}{64} \times 0.75 \times 0.9 \times 1.5 \times 1.5 \times 1.5 \times 1.5 \]

- **L = 16.3'**

- **h_d = 2.9'**

- **L_H = 16.8'**, **h_d = 2.9'**

- **W_H = \frac{4.3}{17.25} \times 15.6'**

- **P_d = \frac{3.9' (17.25 \text{ psf})}{15.6' \text{ psf}} = 67 \text{ psf}**

- **Max snow load = 67 + 16 = 83 \text{ psf}**
Appendix B – Wind Loads

\[ V = 90 \text{ mph} \]  
\[ I = 1.0 \text{ from Table 6-3 Category II Occupancy} \]  
\[ \text{Exposure B, level urban and suburban area, houses and buildings near by} \]  
\[ \text{Using Table 6-3, } K_z \times K_h = 1.21 \text{ Case 1 to 2} \]  
\[ \frac{200 - 200}{1.23} \times 1.2 = \frac{208 - 200}{1.2} \times 1.2 \]  
\[ \text{interp ok} \]  
\[ x = 1.21 \]  
\[ K_{pa} = 1.0 \text{ no wind speed up effects} \]  
\[ \text{Check if frame is flexible or rigid. } \]  
\[ \text{N-S direction (Coll. Tower)} \]  
\[ T_a = C_4 h_n \]  
\[ T_a = 0.02 \times (208,417)^{1/2} \]  
\[ = 1.05 \text{ sec} \]  
\[ f_{res} = \frac{1}{2} f_a = 0.912 f z \leq 1 f z ; \text{ Flexible} \]  
\[ g_s = \sqrt{21.4(3600)} + \frac{3.4}{\sqrt{21.4(3600)}} = 4.16 \]  
\[ n_t = \frac{1}{2} f_a f z = 0.912 f z \]  
\[ R = \frac{1}{2} N_h \times R_h \times (5.5 + 4.2) \]  
\[ N_h = 4.6 \times (1 - 0.03) \]  
\[ N_h = 4.6 \times (1 - 0.03) \]  
\[ R_h = 10.53 - \frac{1}{2} 0.035 (1 - 0.03) \]  
\[ = 0.09 \]  
\[ R_h = 10.53 - \frac{1}{2} 0.035 (1 - 0.03) \]  
\[ = 0.09 \]  
\[ N_h = 4.6 \times (1 - 0.03) \]  
\[ R_h = 10.53 - \frac{1}{2} 0.035 (1 - 0.03) \]  
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\[ N_h = 4.6 \times (1 - 0.03) \]  
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\[ R_h = 10.53 - \frac{1}{2} 0.035 (1 - 0.03) \]  
\[ = 0.09 \]  
\[ N_h = 4.6 \times (1 - 0.03) \]  
\[ R_h = 10.53 - \frac{1}{2} 0.035 (1 - 0.03) \]  
\[ = 0.09 \]
Damping Ratio $\beta = 1.5\%$ for concrete system.

$$R = \sqrt{\frac{1}{2.0}(0.014\times 0.09)(25.5 + 24.4(0.03))}$$

$$= 1.19$$

$$I_2 = e^{(\frac{33}{12})^{1/2}} = e^{(\frac{33}{12})^{1/2}} = 2.14$$

$$Q = \sqrt{\frac{1}{1.43(3.8)^{1/2}}} = \sqrt{\frac{1}{1.43(3.8)^{1/2}}} = 0.8354$$

$$G = \frac{0.925(1 + 1.7(0.2)^{1/2})}{1 + 1.7(0.2)^{1/2}} = 0.855$$

$$K_d = 0.85 \text{ MWFRS at } C_{K_d} \text{ from table 6-1}$$

$$q_i = 0.00256 \frac{K_d}{K_{d1}} = 0.00256(1.21)(1.1)(255)(20)(1)$$

$$= 21.3 \text{ psi}$$

Internal Pressure Coefficients - Fig 6-5

$$G_{C_m} = 0.18$$

Net MWFRS

$$L/B = 167.4^1/2 = 2.43$$

Windward Wall $C_p = 0.8$

Leeward Wall $C_p = 0.28$ from interpolation

$$q_i$$

Enclosed Fleming Building MWFRS

$$P = 2.5(C_{K_d} = 0.05)$$

$$= 21.5(0.05) = 21.5(0.18)$$

$$= 3.8 \text{ psi Windward}$$

$$P = 21.3(0.05) = 21.3(0.18)$$

$$= 3.8 \text{ psi Leeward}$$

N-S direction Top Tower
N=5 direction lower tower

Gust Factor

\[ L_2 = 3.20 \left( \frac{\sigma_{10,50,10}}{\sigma_{10,50,10}} \right)^{0.5} = 37.79 \]

\[ \sigma_2 = 1.45 \left( \frac{A_s}{A_{eq}} \right)^{0.5} \left( \frac{R_s}{R_{eq}} \right) = 67.6 \]

\[ N_s = 0.9125(1 + 0.05) = 5.1 \]

\[ R_s = \frac{7.415(5.1)}{(1 + 0.05)(5.1)} = 0.05 \]

For \( R_s \) \( n = 4.6 \times 0.9125(0.58/0.76) = 5.62 \)

\[ R_s = \frac{4}{5.62} \left( 1 - e^{-2(5/6.2)} \right) = 0.16 \]

For \( R_b \) \( n = 4.6 \times 0.9125(0.77/0.76) = 4.16 \)

\[ R_b = \frac{4}{4.16} - \frac{4}{4.16} \left( 1 - e^{-2(4/4.16)} \right) = 2.11 \]

For \( R_L \) \( n = 15.4 \times 0.9125(0.77/0.76) = 55.6 \)

\[ R_L = \frac{4}{55.6} - \frac{4}{55.6} \left( 1 - e^{-2(55/55.6)} \right) = 0.08 \]

\[ R = \frac{1}{55.6} \left( 0.52 \times 1.02 \times 0.52 \times 0.53 \right) = 0.005 \]

\[ I_2 = 0.3 \times \frac{0.52}{0.52} \times 0.52 = 0.27 \]

\[ \phi = \sqrt{0.63 + 0.05} = 0.857 \]

\[ R = 0.857 \left( \frac{1.02 + 0.52}{0.52} \right) = 9.25 \]

\[ \sigma_{10,50,10} = 9.25 \left( 1 + 0.7(0.52) + 0.52^2(2.4^2) \right) = 0.87 \]
B. 

E-W Direction. Upper Tower

Guess Factor

\[ \begin{align*}
  & L = 498.9 \\
  & W = 82.9 \\
  & N = 5.14 \\
  & R_B = 0.09 \\

  \text{For } R_B: & \frac{N}{W} \left(1 + \frac{2}{3} \frac{L}{W} \right) = 8.23 \\
  & R_B = \frac{1}{8.23} - \frac{1}{2} (8.23) \left(1 - e^{-8.23^2} \right) = 0.09 \\

  \text{For } R_L: & \frac{N}{W} \left(1 + \frac{2}{3} \frac{L}{W} \right) = 11.25 \\
  & R_L = \frac{1}{11.25} - \frac{1}{2} (11.25) \left(1 - e^{-11.25^2} \right) = 0.084 \\

  R &= \sqrt{0.09 \cdot (0.084 \cdot (1.14) \cdot 153 + 0.57 \cdot (0.09))} = 0.157 \\
  J_2 &= 0.24 \\

  Q &= \sqrt{ \frac{1}{1.63 (162.7 + 208.7)^2} } = 0.81 \\

  G_F &= 0.25 \left( \frac{1 + 1.7(0.24)}{1 + 1.7(0.24)} \right) \frac{3 \sqrt{N} \left(1 + \frac{1.7}{1.44} \right)}{1 + 1.7(0.24)} = 0.83 \\

  L_B &= \frac{6^2 (162.7)}{141} \Rightarrow C_p \text{ Leeward} = -0.5
\end{align*} \]
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E-W Direction Lower Tower
Gust Factor

\[ V_2 = 6.7 \text{ \text{mph}} \]
\[ L_B = 317.3 \text{ \text{ft}} \]
\[ N = 5 \text{ \text{mph}} \]
\[ R_n = 1.05 \]
\[ R_b = 1.42 \]

For \( R_b \) \( n = \frac{1.41(1.91)(2-1.03)}{6.7^6} = 16.84 \)
\[ R_b = \frac{\sqrt{2.84} - \frac{1}{2}(1.05)}{(1-e^{-2.84})} = 0.05 \]

For \( R_L \) \( n = \frac{1.41(1.91)(6.7)^6}{6.7^6} = 13.92 \)
\[ R_L = \frac{1}{13.92} - \frac{1}{2}(13.92)(1-e^{-13.92}) = 0.09 \]

\[ R = \sqrt{\frac{1}{1.04} \left(1.04 \times 1.05 \times 0.53 \times 1.04 \right)} = 1.33 \]
\[ I_2 = 2.75 \]

\[ Q = \sqrt{\frac{1}{1.03} \left(2/1.03 + 0.53 \times 0.53 \right)} = 1.75 \]

\[ C = 0.925 \left(1 + 1.7(1.7)^2 \times (1.8^2 + 1.4^2(2.7)^2) \right) = 1.84 \]

\[ \frac{1}{13} = \frac{1}{271.3} = 0.004 \quad \text{\text{mph}} \]

\[ \zeta_\text{leeward} = -0.5 \]
Appendix C – Seismic Loads

From geo-tech report:
- $S_a = 1.71\%\text{g}$
- $S_m = 5.1\%\text{g}$

Side class C based on geo-tech report.

Using IIA.3:
- $S_{s} = F_a S_0 = 1.2(1.17) = 2.058$
- $S_{m} = F_n S_m = 1.17(0.0867) = 0.103$

$S_{d0} = \frac{2}{3} \times S_{s} = \frac{2}{3} (2.058) = 1.372$

$S_{d1} = \frac{2}{3} \times S_{m} = \frac{2}{3} (0.0867) = 0.0578$

Importance factor: $1.0$ based on II occupancy.

Table IIa.1:
- $S_{d0} = 1.372 < 1.67$: Category A

Table IIa.2:
- $S_{d1} = 0.0578 < 0.067$: Category A

To be more conservative and accurate, will use equivalent lateral method.

$V = L_{w} W$

$c_3 = \frac{S_{mp}}{\mu} = \frac{1.372}{4/1} = 0.342$

$c_4 = 2.01$

$c_5 = 1.7$ (dry soil)

$S_{d1} = 0.0578 < 1.0$ from table IIb.1

$T_a = c_4 h_{a} = 0.0578(208.4)^{0.78} = 1.1$ seconds from table IIb.2

All other cases:

$c_w = \frac{100}{A_{b}} \frac{n}{1 + (n)} \left( \frac{A_i}{A_{b}} \right)$

Calculation available in spreadsheet upon request.

$T_a = \frac{c_{0019}}{V_{C_w}} h_{a}$

$20\% h_{a} = 1.75 \text{ sec}$ N-S direction

$K = 1.16$ through interp.

$2.5 \div 1 = \frac{12.5}{x} \div 1$

$T_a = \frac{c_{0019}}{1.18^{2}} 20\% h_{a} = 0.57 \text{ sec}$ E-W direction

$K = 1.033$ from interp.

N-S direction continues...
Weight Calculation

Penthouse Roof
Decking: 2 psf
Shear: 8 psf
Membrane: 1 psf
Roofing: 6 psf
Insulation: 6 psf

\[ 23 \text{ psf} \times 3392 \text{ ft}^2 = 79026 \text{ lbs} \]

High Roof

Slab: \( \frac{9}{12} \times (\frac{12}{12}) = 9.375 \text{ psf} \)
Superimposed: \( \frac{6}{12} \times (\frac{12}{12}) = 6 \text{ psf} \)

Green Roof: 3 psf

\[ 12.25 \text{ psf} \times 19.845 \text{ ft}^2 = 245.761 \text{ lbs} \]

HSS col: \( 1\frac{1}{4} \times \frac{1}{4} \times \frac{1}{4} = 0.313 \text{ lbs} \)
HSS col: \( 6\frac{1}{8} \times \frac{1}{4} \times \frac{1}{4} = 2.687 \text{ lbs} \)

Floor 20-

Slab: 6 psf

\[ 10.75 \text{ psf} \times 1\frac{1}{2} \text{ ft}^2 = 16.1125 \text{ lbs} \]

Columns: \( \frac{2}{12} \times \frac{2}{12} \times 10.533 \times \frac{1}{2} \text{ psf} = 6.458 \text{ lbs} \times \frac{1}{2} = 3.229 \text{ lbs} \)

Shear Wall #9: \( 1\frac{1}{2} \times (10.333 \times 10.333 \times 10.333) \times \frac{1}{2} \text{ psf} = 34.333 \text{ lbs} \)

Shear Wall #10: \( \frac{1}{2} \times (10.333 \times 10.333 \times 10.333) \times \frac{1}{2} \text{ psf} = 9.833 \text{ lbs} \)

\[ \frac{9}{12} \times (\frac{12}{12}) = 7.5 \text{ lbs} \]

\[ \frac{9}{12} \times (\frac{12}{12}) = 7.5 \text{ lbs} \]

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\[ \frac{9}{12} \times (\frac{12}{12}) = 7.5 \text{ lbs} \]
Floor 19-19

Slab = $3/4''(180) = 1000$ psi

SDH = $\frac{1}{8}$ in

\[ \text{Col} = \frac{0.08' \times 10,805 \times 52}{1000} = 1166.94 \text{ lb} \]

Columns used excel spreadsheet
Same dim different legs

Short walls used excel spreadsheet

Floor 19-12

Slab = 1000 psi

SDH = $\frac{1}{8}$ in

\[ \text{Col} = \frac{0.08' \times 10.849}{1000} = 117.1476 \text{ lb} \]

Floor 11

\[ \text{Col} = \frac{68.831}{1000} = 68.831 \text{ lb} \]

Shear walls = 314.111 lb

Floor 10

\[ \text{Col} = \frac{68.831}{1000} = 68.831 \text{ lb} \]

Floor 9

\[ \text{Col} = \frac{68.831}{1000} = 68.831 \text{ lb} \]

Floor 8

\[ \text{Col} = \frac{68.831}{1000} = 68.831 \text{ lb} \]

Floor 7-5

\[ \text{Col} = \frac{68.831}{1000} = 68.831 \text{ lb} \]
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Floor 4-2
Slab + SDL = 1020.5 + 1200(17,842) = 1926.936 lb
Col = 1142.99 lb
Shear walls = 5241.68 lb

Floor 1
Slab 8 1/4' (150) = 4875.74 lb
SDL = 8.155
705.75 x 1704.5 = 1263244 lb
Col = 171520.16 lb
Shear walls = 784443 lb
Appendix D – Column Spot Check

Brad Oliver

Col. 114 E

2nd Floor

\( P_{a} = 8 \text{kPa} \)

\( k_f = 1 \text{ kPA/m} \)

\( (3) = 3 \text{ Cycles} \)

\( A_u = (6.5' + 11.5') \times (25') = 450 \text{ ft}^2 \)

Dead Loads:

- Slab: 8" x (50') = 100 psf
- SDF: \( (\frac{30}{12})' \times (50)' \times (18') = 50 \text{ psf} \)
- SDL: 113 kips
- Live Load: 40 psf Residential
- 120 psf Restrooms
- 50 psf

Snow Load: 16 psf

Floor dead load: 91/2" x (50') = 125 psf

Floor live load: 30 psf

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

Typical Floor + Roof + Snow + Self Weight

\( P_{r} = (100 + 8) \times 450 \text{ psf} + (125)\times 450 + 113000 \text{ kips} \)

\( = 1090 \text{ kips} \)

\( P_{l} = 60 \text{ psf} \times 450 \text{ ft}^2 \times 1.5 \text{ Cycles} \)

\( = 5572 \text{ kips} \)

\( P_{s} = 16 \text{ psf} \times 450 \text{ ft}^2 = 72 \text{ k} \)

\( P_{a} = 1.2(1090) + 1.6(5572) + .5(72) \)

\( = 2153 \text{ kips} \)
Column will be checked for factored axial load \( P_{ax} \).

It is an interior column sufficiently far from shearwalls

and will not be participating in lateral resistance.

\[
\begin{align*}
P_{ax} &= 0.85 \times f_{c} \times A + A_{s} f_{y} \\
&= 0.85 \times (3,000 \times 600) + (60,000 \times 1.27) \times 65 \\
&= 314.7 \text{ kips}
\end{align*}
\]

Factored compression = 0.8f_{c} = 0.8(3,047) = 2,438 kips

\[
\begin{align*}
A_{s} &= 20(20) = 600 \text{ in}^{2} \\
A &= 16(1.25) = 20 \text{ in}^{2} \\
f_{y} &= 60,000 \text{ psi}
\end{align*}
\]

\[
\begin{align*}
\phi(\beta) &= 0.65 \text{ ksi, not spalled reduced} \\
\beta &= \frac{N}{f_{y} A} = \frac{2517}{60000} = 0.042 \text{ ksi, not spalled reduced}
\end{align*}
\]

Ties are at least \( \pm 3 \) for \( \pm 10 \) bar √

Spaced 18” OC

May spacing = 16 x 10 in. bar φ = \( \phi(1.25) = 0.042 \text{ ksi} \)

= 4 in. x tie x bar φ = \( \phi(3.25) = 0.05 \text{ ksi} \)

= least d in oc = 0.1 in.
Appendix E – Post Tensioning Spot Check

Preliminary

\[ h_1 = 1.5 \]

\[ h_2 = \frac{h_1}{1.45} = \frac{1.5}{1.45} = 1 \text{"} \]

Dead Load - $(3/12)(50 \text{ psi}) = 1000 \text{ psf}$

\[ \frac{5 \times 1}{1000 \text{ psf}} = 0.5 \text{"} \]

Add. 55 gal

11" - 55 gal

Assume targeted load balancing of 75% of DL

\[ 0.75(10\%) = 8 \text{ lb} \]

+ residual

int support

int midspan

\[ \text{Weight to balance} \]

\[ 81 \text{ psf} \times (9' + 3') = 1461 \text{ lb} = 136 \text{ kips} \]

\[ P = \frac{wL^2}{8a} \]

\[ = \frac{136(25)^2}{81} \]

\[ = 384 \text{ kips} \]

Tendon properties

- Wire strand A-414
- 1/2" diameter tendon area = 153 in²
- Ultimate strength = 270 ksi
Estimate loss @ 15 ksi:

\[ f_{es} = 0.7(270) - 15 = 194 \text{ ksi}. \]

\[ P_{es} = 0.53(194) = 26.6 \text{ kips/tendon} \approx 27 \text{ kips/tendon designed} \]

15 tendons to balance load

\[ 399/26.6 = 14.76 \approx 15 \text{ tendons} \]

\[ P_{max} = 15(26.6) = 399 \text{ kips} \]

This could be due to smaller self weight, balance, or the fact there is another 200 kips tendon running // is a way to make up for this difference.

Balanced load adjustment

\[ \frac{399}{1.66} = 237 \text{ kips} \]

\[ \frac{P_{max}}{A} = \frac{399}{(9.24+2.32)(3')} \geq \sigma_{fc} \text{ psi} \geq 125 \text{ psi} \text{ min. V} \]

\[ \leq 300 \text{ psi max} \]
Check slab stresses

$w_{dc} = 158 \text{ psi} \left(94 + \frac{24}{3}ight) = 2.21 \frac{154}{125} \frac{51}{55}$
$M_{o} = 2.21(\frac{24}{3}) = 132.1k$

$w_{u} = 53 \text{ psi} \left(94 + \frac{24}{3}ight) = 1.13 \frac{47}{47} M_{u} = 28.8k$

$w_{b} = -1.7 \frac{47}{47} M_{o} = 133.1k$

Stresses induced by After jacketing

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

$f_{cub} = (M_{du} - M_{bn})/s = 71/4$

$= 26.2 \text{ kips}$

$f_{cub} = (M_{du} - M_{bn})/s = 71/4$

$= 26.2 \text{ kips}$

Support

$f_{top} = (-129 + 100)(1200)/2624 = 2.03$

$f_{top} = (-129 + 100)(1200)/2624 = 2.03$

$f_{b} = (-129 + 100)(1200)/2624 = 2.03$

$f_{b} = (-129 + 100)(1200)/2624 = 2.03$
Stresses & Service Load

\[ f_{10} = \left( \frac{M_1 - M_2 + M_9}{b} \right) G1' \]

\[ = \left( \frac{-55089 + 113000}{2624} \right) \frac{1}{G1'} \]

\[ = -20.55 \text{ psi} < -45 \text{ psi} \]

\[ f_{90} = \left( \frac{M_9 - M_{10}}{b} \right) G1' \]

\[ = \left( \frac{55089 - 113000}{2624} \right) \frac{1}{G1'} \]

\[ = -20.55 \text{ psi} < -45 \text{ psi} \]

Support

\[ f_{20} = \left( \frac{-129 - 66 + 0}{2624} \right) \frac{1}{G1'} \]

\[ = -20.55 \text{ psi} < -45 \text{ psi} \]

\[ f_{29} = \left( \frac{129 + 66 - 0}{2624} \right) \frac{1}{G1'} \]

\[ = -20.55 \text{ psi} < -45 \text{ psi} \]
Appendix F – Supplemental Drawings

Figure 13 - typical lower floor layout

Figure 14 - typical upper floor layout
Figure 15 - column schedule
Figure 16 - 22 floor shear wall

Figure 17 - typical 9 floor shear shear wall
Figure 179 - West Elevation

Figure 20 - South Elevation