Technical Report I
2011-2012 AE Senior Thesis

9/23/2011
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Executive Summary

The following technical report summarizes the existing conditions and design concepts present at the UPMC Hamot Women’s Hospital. Structural plans were provided by Atlantic Engineering Services. All other plans were provided by Rectenwald Architects Inc. The existing conditions were closely examined and then analyzed using the IBC 2006 building code, which is the design code enforced on the building at its time of construction.

Wind and Seismic loads were calculated using ASCE 7-05. Wind from the North was considered over wind from the South due to a 60’ tall 2-D Escarpment present at the base of the North wall. The base shears and overturning moments were determined based on the pressure values attained. A base shear and over turning moment of 1040.3 k and 40230.8 ft-k was determined for the wind from the North, while a base shear and overturning moment of 435.9 k and 18927.2 ft-k was determined for the East or West wind. The seismic load was determined and is the same for both directions due to the building utilizing the same lateral system in both directions. The seismic forces produced a base shear and overturning moment of 278.5 k and 16163.6 ft-k; thus leading to wind controlling the design of the lateral system for both directions. This is likely the case due to the Exposure D classification required by ASCE 7-05, which is due to the buildings close proximity to Lake Erie.

Spot calculations of the gravity structure were also done in order to determine the accuracy of the gravity loads and how conservative the Engineer of Record was in the building design. The calculations proved the members to be adequate for all strength and serviceability concerns, thus meaning that the loads calculated by the other were close to the loads used by the Engineer of Record. The largest concern arose when the calculation of the maximum column load was calculated and compared to the load the column should see. The column appeared to be over designed by 275%, after discussing this with the Engineer of Record and thought on how the hand calculations performed differ from those of the computer it was found that the Engineer of Record imposes a self-limit of 80% stress on all columns, as well as the hand calculations omitted live loading combinations for simplicity.

The intent of this process was to determine how the various structural components behaved as a structural system.
Introduction

Located on the bay the shoreline of Lake Erie, 201 State Street, which will be referred to as UPMC Hamot Women’s Hospital, is a 5 story, steel framed healthcare and hospital facility. This site is centrally located on the UPMC Hamot campus, directly between the UPMC Hamot Main Hospital and the UPMC Hamot Heart Institute.

The 163,616 sq. ft. Women’s Hospital was completed in early January of 2011. This structure has a very unique history, originally the hospital wanted a four story building, but only had the financing for two levels. Thus the structure was designed for four stories, but only the first two were constructed. Then the hospital decided that a five story structure more suited their needs, so the building was stripped down to the shell (structural steel and floor slabs), the current roof slab was then removed with the columns being truncated 4’-0” above the second story slab. The decision was made to reinforce the columns and beams below this point, as needed, and to build to the desired five stories above.

The city of Erie zoned the UPMC Hamot campus as Waterfront Commercial 2 (W-C2), which permits residential, commercial, recreational, and historical uses. This zoning is similar to Waterfront Commercial (W-C), except that this area permits Group Care Facilities. The maximum building height in this zoning district is 100 ft, with a building footprint not greater than 65% of the lot. The exterior lighting of the building must prevent glare to adjoining properties and the lot is required to have 1 parking space per 4 beds.

The five stories of the UPMC Hamot Women’s Hospital are topped with a mechanical penthouse that does not cover the entire building footprint. This penthouse houses three air handling units that supply conditioned air to all areas of the building. This is achieved via a large mechanical opening in each floor; this opening is located on the west side of the building and measures approximately 27'-0"± by 30'-0"±.

The UPMC Hamot Women’s Hospital was designed to match the Architectural style of the other buildings on the Hamot Medical Center campus. This includes a brick and glass façade that

Figure 1: North Façade, Showing 2-D Escarpment

Figure 2: Interior Water Wall
Justin L. Kovach – Structural Option
Dr. Boothby, Advisor
UPMC Hamot Women’s Hospital has an exterior façade of 4” nominal face brick, a 3” air space, 1” of rigid insulation, on 6” nominal metal studs w/ R-19 batt insulation filing the wall core. The wall is then closed with 5/8” gypsum wall board. Where applicable the wall system is double pane insulated glass windows. The roof system is EPDM roofing on protection board on polyisocyanurate insulation.
**Structural System**

- **Foundation**

The foundation is unique in that many of the existing foundations also had to increase in size when the building increased in height. The foundation system utilizes both strip and spread footings. The strip footings are typically 2'-0” wide and 1'-0” deep; reinforcement consists of 3-#5 longitudinally and #5 x 1'-6” @ 12” O.C. transverse. The spread footings are the most unique because many of the existing spread footings had to be increased a length, width, and depth. The minimum height of the footings below grade is 3'-6”. The typical foundation overbuild details can be found on sheet S403.

- **Floor Construction**

The beams are typically W shapes that tend to be framed with the girders spanning the short direction and the beams framing the long direction of the bay. The beams are typically W14x22 composite beams, where concrete slab on deck exists. In the shorter spans (12'-4”) the beams become W8x10, and when the tributary spacing is decreased they tend to become W12x19 composite beams. Elsewhere the beams are non-composite. The girders are also composite where applicable.

The elevated floor slabs have a total thickness of 6”, consisting of 4” of lightweight 4000 psi concrete on a 2” – 20 GA composite metal deck. These slabs are reinforced with 6x6 – W1.4xW1.4 welded wire fabric.

- **Lateral System**

The lateral system in the N-S direction consists of a 5 story (6 with penthouse), 49’ long braced frame along column line N. This is the alone full height braced frame in the building. The N-S direction also has a full height 42'-8” long moment frame along column line B. The E-W direction utilizes full height moment frames along column line 1 and 17, which are 161’ and 173'-4” long, respectively. The columns are spliced 4’-0” above the second floor, where the existing shell remained and was reinforced below. The columns are also spliced at above the 4th floor, at the same 4’-0” elevation. The unique construction sequence has led to the need to reinforce the base of these columns dramatically, especially in the moment frames. The details of these reinforcements can be seen on sheet S400. The column sizes vary from W8 sizes to W14 sizes. The lateral system of the mechanical penthouse is entirely braced frames.
Design Codes & Standards

2006 International Building Code (IBC 2006) with Local Amendments
2006 International Mechanical Code (IMC 2006) with Local Amendments
2006 International Electrical Code (IEC 2006) with Local Amendments
2006 International Fire Code (IFC 2006) with Local Amendments
Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
Building Code Requirements for Structural Concrete (ACI 318-08)
Building Code Requirements for Masonry Structures (ACI 530)
AISC Manual of Steel Construction, Allowable Stress Design (ASD)

Structural Materials

<table>
<thead>
<tr>
<th>Structural Steel</th>
<th>Standard</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-Shape Structural Steel</td>
<td>ASTM A572</td>
<td>50</td>
</tr>
<tr>
<td>Hollow Structural Sections (HSS)</td>
<td>ASTM A500</td>
<td>C</td>
</tr>
<tr>
<td>Bars, Plates and Angles</td>
<td>ASTM A36</td>
<td>N/A</td>
</tr>
<tr>
<td>Bolts, Washers, and Nuts</td>
<td>ASTM A325</td>
<td>N/A</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Weight</th>
<th>Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Footings</td>
<td>Normal</td>
<td>3000 psi</td>
</tr>
<tr>
<td>Slab-on-Grade</td>
<td>Normal</td>
<td>4000 psi</td>
</tr>
<tr>
<td>Concrete on Steel Deck</td>
<td>Lightweight</td>
<td>4000 psi</td>
</tr>
</tbody>
</table>
Building Loads

Part of this technical report will incorporate the calculation of both gravity and lateral loads. The gravity loads will consist of dead, live, and snow loads. The lateral loads will be analyzed through wind and seismic loading. The intent of this aspect of the report is to lay the groundwork for remainder of this thesis project, as well as begin to determine how conservative the primary designer may or may not have been.

- **Dead Load**

Dead loads were calculated using the most recent data available through the Vulcraft Corporation. Typical floor weight was found to be 59 psf, although allowing for some unknowns a superimposed dead load was decided to be used, which is conservative; thus leaving a typical floor dead load of 69 psf. The roof dead load was also calculated using the Vulcraft Corporation manuals, and the roof dead load was determined to be 15 psf. To be conservative a roof dead load of 20 psf will be used, allowing for future roof coverings to be laid on the initial roof. Appendix A includes the appropriate figures from the Vulcraft Manuals used, as well as detailed calculations for the typical floor and roof dead load.

- **Live Load**

Live Loads were calculated in accordance with IBC 2006 using ASCE 7-05 (Minimum Design Loads for Buildings and Other Structures). The relevant loads derived are tabulated in Table 1 and in Appendix A.

<table>
<thead>
<tr>
<th>Space</th>
<th>Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lobbies</td>
<td>100</td>
</tr>
<tr>
<td>First Floor Corridors</td>
<td>100</td>
</tr>
<tr>
<td>Offices</td>
<td>50</td>
</tr>
<tr>
<td>Stairs</td>
<td>100</td>
</tr>
<tr>
<td>Mechanical</td>
<td>150</td>
</tr>
<tr>
<td>Roof</td>
<td>20</td>
</tr>
<tr>
<td>Hospitals</td>
<td></td>
</tr>
<tr>
<td>Operating Rooms/Labs</td>
<td>60</td>
</tr>
<tr>
<td>Patient Rooms</td>
<td>40</td>
</tr>
<tr>
<td>Corridors, above First Floor</td>
<td>80</td>
</tr>
</tbody>
</table>

Table 1: ASCE 7-05 Live Loads

- **Snow Load**

Snow loads were calculated using the procedure outlined in ASCE 7-05 Chapter 7. The city of Erie, PA falls into an area requiring a Case Study (CS) of the ground snow load. A call to the Erie Building Code Official yielded a local requirement for designers to use a ground snow load of 40 psf. The Snow Load Calculations are summarized in Table 2 and detailed calculations are available in Appendix B. Several
locations were determined to be potential drift locations, located around the Mechanical Penthouse and the Stair Pop-out. The Mechanical Penthouse yielded a peak drift load of 106.2 psf with a width of 17'-0". The Stair Pop-Out yielded a peak drift load of 58.2 psf with a width of 7'-0". A roof floor plan with mark-ups of the applicable snow drift areas is available in Appendix B.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground Snow Load, $p_g$ (psf)</td>
<td>40</td>
</tr>
<tr>
<td>Temperature Factor, $C_t$</td>
<td>1.0</td>
</tr>
<tr>
<td>Exposure Factor, $C_e$</td>
<td>0.8</td>
</tr>
<tr>
<td>Importance Factor, $I_s$</td>
<td>1.1</td>
</tr>
<tr>
<td>Flat Roof Snow Load, $p_f$ (psf)</td>
<td>24.64</td>
</tr>
</tbody>
</table>

Table 2: ASCE 7-05 Snow Loads

- **Wind Load**

Wind loads were calculated in accordance with Chapter 6 of ASCE 7-05, Method 2 Main Wind Force Resisting System (MWFRS). In order to use this procedure a few minor simplifications had to be made, such as reducing the five different building heights to three. This was done by taking two of the minor pop-outs (< 5 ft) and simplifying them into the main roof.

The wind loading for this building is very unique and interesting. The building sits on the peak of a 60 ft tall 2-D escarpment, as described in ASCE 7-05. This produces an atypical wind loading pattern in the North-South Direction. This problem is compounded by the building being located on the bay of Lake Erie, this flat open body of water allows for wind velocities to increase rapidly. This leads to a very large wind load at the base of the North wall of the building.

Wind loads on the building are collected by the exterior façade and distributed to the slab, at which point the slab will distribute the forces to the MWFRS, based on the stiffness and location of the various structural elements.

The user should note that the internal pressures are not added to the external windward and leeward pressures. This is due to the fact that the internal pressures effectively cancel themselves out. This has been done in this report as is standard practice in structural engineering.

The wind pressures that engage the North-South lateral system was analyzed as a wind coming from the North. This is due to the large 2-D escarpment located on that side of the building. The wind pressures engage the East-West lateral system was analyzed as a wind coming from the East, although the wind coming from the West would be identical.

Details pertaining to the wind calculations can be found in Appendix C, while a summary of the final wind pressures can be found in Table 3 and Table 4, for a pictorial view of how these pressures are applied to the building see Figure 5 and Figure 6.
Table 3: ASCE 7-05 Wind Pressures in N-S Direction

<table>
<thead>
<tr>
<th>Type</th>
<th>Height</th>
<th>Wind Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward Walls</td>
<td>0'-15'</td>
<td>59.51</td>
</tr>
<tr>
<td></td>
<td>15'-20'</td>
<td>39.39</td>
</tr>
<tr>
<td></td>
<td>20'-25'</td>
<td>36.35</td>
</tr>
<tr>
<td></td>
<td>25'-30'</td>
<td>34.03</td>
</tr>
<tr>
<td></td>
<td>30'-40'</td>
<td>32.76</td>
</tr>
<tr>
<td></td>
<td>40'-50'</td>
<td>29.87</td>
</tr>
<tr>
<td></td>
<td>50'-60'</td>
<td>28.13</td>
</tr>
<tr>
<td></td>
<td>60'-70'</td>
<td>26.98</td>
</tr>
<tr>
<td></td>
<td>70'-80'</td>
<td>26.40</td>
</tr>
<tr>
<td></td>
<td>80'-90'</td>
<td>26.03</td>
</tr>
<tr>
<td></td>
<td>90'-92'</td>
<td>25.71</td>
</tr>
<tr>
<td>Leeward Walls</td>
<td>Full Height</td>
<td>-15.55</td>
</tr>
</tbody>
</table>

Figure 5: Wind Pressures in N-S Direction, showing 2-D Escarpment
<table>
<thead>
<tr>
<th>Type</th>
<th>Height</th>
<th>Wind Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward Walls</td>
<td>0’-15’</td>
<td>19.20</td>
</tr>
<tr>
<td></td>
<td>15’-20’</td>
<td>19.88</td>
</tr>
<tr>
<td></td>
<td>20’-25’</td>
<td>20.43</td>
</tr>
<tr>
<td></td>
<td>25’-30’</td>
<td>20.99</td>
</tr>
<tr>
<td></td>
<td>30’-40’</td>
<td>21.82</td>
</tr>
<tr>
<td></td>
<td>40’-50’</td>
<td>22.50</td>
</tr>
<tr>
<td></td>
<td>50’-60’</td>
<td>23.05</td>
</tr>
<tr>
<td></td>
<td>60’-70’</td>
<td>23.47</td>
</tr>
<tr>
<td></td>
<td>70’-80’</td>
<td>24.16</td>
</tr>
<tr>
<td></td>
<td>80’-90’</td>
<td>24.44</td>
</tr>
<tr>
<td></td>
<td>90’-92’</td>
<td>24.58</td>
</tr>
<tr>
<td>Leeward Walls</td>
<td>Full Height</td>
<td>-14.13</td>
</tr>
</tbody>
</table>

Table 4: ASCE 7-05 Wind Pressures in E-W Direction

Figure 6: Wind Pressures in E-W Direction
• **Seismic Load**

Seismic loads were calculated as required by ASCE 7-05, Chapter 11 and 12. This section requires the use of the Equivalent Lateral Force Procedure. For this analysis an R-Factor of 3 was chosen, meaning the building is “not specifically detailed for seismic loads”.

Seismic loads tend to be very complicated in nature, due to the fact that no two earthquakes are ever the same. This leads to many engineering simplifications within the code to allow us to analyze the structure quickly and efficiently. Wind loads are easier to quantify because it acts as a pressure on the building. Earthquake loads are more difficult to quantify because the loading comes through the motion of the ground. ASCE 7-05 assists the structural engineer by providing a procedure that allows for the complicated loading to be turned into forces applied at the various levels. The overall base shear of the building is controlled by many factors, although the inertial mass of the building can be singled out as one of the most important factors. The mass and height of each level leads to how much of the overall base shear we can apply to that respective level.

Several assumptions had to be made in order to use the Equivalent Force Method in ASCE 7-05. The first assumption is that the mass of each story is lumped at that story level. This is not an outrageous assumption because the majority of a stories mass is located in the slab and beams attributed to that story. The mass associated with columns spanning between levels were divided to the stories above and below based on tributary height between the levels, giving half of the columns mass to the level above and half to the level below. The other major assumption is that the building utilizes a rigid diaphragm. This is a reasonable assumption due to the relative rigidity of the slab compared to that of the lateral system. This is also reasonable due to the absence of shear walls, if shear walls were present as a lateral system in this structure the interaction between the slab and the walls would have to be carefully analyzed and detailed to transfer the large loads that the shear walls would take.

Details pertaining to the seismic calculations can be found in Appendix D, while a summary of the final seismic forces can be found in Table 5, for a pictorial view of the forces being applied at the various story levels see Figure 7.

<table>
<thead>
<tr>
<th>Level</th>
<th>Level Weight (kips)</th>
<th>Level Height</th>
<th>EQ Force (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Penthouse</td>
<td>315.4</td>
<td>92’-0”</td>
<td>17.24</td>
</tr>
<tr>
<td>Stair Roof</td>
<td>74.3</td>
<td>82’-0”</td>
<td>3.41</td>
</tr>
<tr>
<td>Roof</td>
<td>1616.0</td>
<td>72’-0”</td>
<td>60.77</td>
</tr>
<tr>
<td>5th Floor</td>
<td>2282.7</td>
<td>58’-0”</td>
<td>61.71</td>
</tr>
<tr>
<td>4th Floor</td>
<td>2348.6</td>
<td>44’-0”</td>
<td>41.64</td>
</tr>
<tr>
<td>3rd Floor</td>
<td>2401.9</td>
<td>28’-0”</td>
<td>21.36</td>
</tr>
<tr>
<td>2nd Floor</td>
<td>2567.1</td>
<td>12’-0”</td>
<td>6.26</td>
</tr>
<tr>
<td>Ground Floor</td>
<td>N/A</td>
<td>0’-0”</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 5: ASCE 7-05 Seismic Calculations
Earthquake Forces

Figure 7: Earthquake Forces at Various Levels
Gravity Load Spot Checks

As part of this technical report spot checks of the existing structure were performed. The purpose of these spot checks is two-fold, to determine that the gravity loads calculated by the author of this report are similar to those used by the Engineer of Record, and to try and allow the author of this report to quantify how conservative the Engineer of Record was in their calculations. Columns M-7, M-5, L-7, and L-5 were chosen to represent the typical bay. This bay was chosen because of the uniformity of the bay mentioned and the adjacent bays in all directions. Complete hand calculations for the members being analyzed below are available in Appendix E; please refer to Appendix A for the calculations of the loading these members were subjected to.

- **Decking**

The typical floor slab at UPMC Hamot Women’s Hospital consists of a2” 20 gage steel deck with 4” of lightweight concrete. Using the Vulcraft Steel deck manual, enclosed in Appendix E, the determination of maximum capacity of the slab was determined. Then the maximum unshored clear span was checked versus the allowable. It was determined that the slab had 275% more capacity then was needed to carry the applied loads, and that the spacing was well within the maximum. Upon further investigation it was determined that the slab was chosen to be a 4” concrete on 2” deck due to the minimum fire rating as specified by code.

- **Beam and Girder**

The typical composite beam was chosen to be a W14x22[10], this loading on the beam was determined based on a tributary width analysis and the maximum factored shear and moment were determined. The capacity available was then determined based on AISC Steel Manual procedure. Finally the beam was checked to ensure that it passed live load deflection criteria, and unshored wet concrete deflections under construction loading. As is typical the controlling limits of the beam were the live load deflection and the unshored wet concrete deflection.

The typical composite girder was chosen to be a W16x26[18], this loading on the beam was determined based on a tributary width analysis and the maximum factored shear and moment were determined. The capacity available was then determined based on AISC Steel Manual procedure. Finally the beam was checked to ensure that it passed live load deflection criteria, and unshored wet concrete deflections under construction loading. As is typical the controlling limits of the beam were the live load deflection and the unshored wet concrete deflection.

The hand calculations for both the beam and the girder are available in Appendix E, both the beam and the girder were determined to work fine under the applied loading. It appears that the Engineer of Record was not overly conservative with this aspect of the design.

- **Column**
The typical column was chosen as the column along column L-5, the loading was determined based on a tributary area analysis, with live load reduction being done on all levels except the roof. The $K_{LL}$ value for the live load reduction was determined to be 4, by the methods described in the AISC Steel Manual. The column selected is spliced above the 2nd floor and the 4th floor; thus leading to the need to check the column for 3 different sizes, one of which was an existing 8WF67. The properties for this older beam were determined based on the AISC shape database CD (supplied with AISC Manual 13 ed.).

The column loading vs. column capacity was found to be drastically conservative. This can be explained through various sources. Primarily the Engineer of Record does not allow his columns to be stressed over 80%; thus leading to an already conservative column. Then due to simplifying for the use of hand calculations various live load patterns were not explored. Exploring these combinations would add some inherent moment into the column, thus taking away more of the available capacity. Exploring all of these combinations just is not possible without the use of a sophisticated structural analysis program.
Conclusion

Through this report and the exploration of the various structural systems employed by the Engineer of Record a greater understanding of the UPMC Women’s Hospital and the ASCE 7-05 code was developed. Through the various spot checks a greater understanding of the existing structural conditions was determined and a greater understanding of the various design guidelines the Engineer of Record used in design. These checks were determined to be adequate for both strength and serviceability.

The redesign phase of this thesis could include a comparison of the ASCE 7-05 building code and the ASCE 7-10 building code. The UPMC Hamot Women’s Hospital was subjected to the ASCE 7-05 code, but the newer ASCE 7-10 code changed the wind loading dramatically. Due to its unique location I feel this would be a great chance to examine the changes of the building code.
Appendix A: Gravity Load Calculations

A.1 – Dead Load Calculations

Dead Loads

Second Floor (Existing) Slab is 3 1/4" on 2" - 20 GA Composite Deck; Normal Weight or Lightweight Concrete → Unknown
   
   Use Self-Weight for all Slabs as 4" LW Conc. on 2" - 20 GA Composite Deck

   Total Slab Thickness = 6"
   Theoretical Concrete Volume = \(0.917 \frac{ft^3}{\text{cu ft}} \times 110 \frac{in}{ft} \times \frac{1\text{ ft}}{12\text{ in}} = 46 \frac{lb}{ft^2}\)
   
   Deck Weight = 2 psf

   Total Slab Weight = 48 psf
   MEP = 5 psf
   Ceiling/Lights/Flor = 6 psf
   
   Superimposed DL = 67 psf = Total Floor DL

Roof Weight

1/2" Galvanized Steel Roof Deck - 20 GA = 2 psf
   
   Roofing = 3 psf
   Insulation = 5 psf
   Ceiling / MEP = 5 psf
   
   Use 40 psf = total!
   
   Includes 5 psf Superimposed DL.
# A.2 – Vulcraft Manual Page for 2VLI Decks

---

**SLAB INFORMATION**

<table>
<thead>
<tr>
<th>Total Slab</th>
<th>Theor. Concrete Volume</th>
<th>Recommended Webbing Wire Gauge</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 1/2</td>
<td>1 1/2</td>
<td>6x8: WA4/7W1.4</td>
</tr>
<tr>
<td>5</td>
<td>1 1/2</td>
<td>6x8: WA4/7W1.4</td>
</tr>
<tr>
<td></td>
<td>5 1/4</td>
<td>6x8: WA4/7W1.4</td>
</tr>
<tr>
<td></td>
<td>5 3/4</td>
<td>6x8: WA4/7W1.4</td>
</tr>
<tr>
<td></td>
<td>3 1/2</td>
<td>6x8: WA4/7W1.4</td>
</tr>
</tbody>
</table>

---

**VULCFRAFT**

---

**L (N=14.15) LIGHTWEIGHT CONCRETE (11C PCF)**

<table>
<thead>
<tr>
<th>TOTAL Plank</th>
<th>SSI Max. Unshored Plank Loads</th>
<th>Unshored Live Loads, PSF</th>
<th>Compressive Strength NPSF</th>
</tr>
</thead>
<tbody>
<tr>
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</tr>
<tr>
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<td>10.9</td>
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<td>9&quot;</td>
<td>11.3</td>
<td>11.1</td>
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<tr>
<td>2022</td>
<td>10&quot;</td>
<td>11.7</td>
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<td>11&quot;</td>
<td>12.1</td>
<td>11.9</td>
</tr>
<tr>
<td>2022</td>
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<td>12.5</td>
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<tr>
<td>2022</td>
<td>13&quot;</td>
<td>12.9</td>
<td>12.7</td>
</tr>
<tr>
<td>2022</td>
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<td>13.1</td>
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<td>2022</td>
<td>15&quot;</td>
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<tr>
<td>2022</td>
<td>16&quot;</td>
<td>14.1</td>
<td>13.9</td>
</tr>
</tbody>
</table>

---

**COMPOSITE**

---

1. Minimum exterior bearing length required is 2.000 inches. Minimum interior bearing length is 4.000 inches. If these minimum lengths are not provided, web crippling must be checked.
2. Always contact Vulcraft when using loads in excess of 350 psf. Such loads can result from concentrated, dynamic, or long-term load cases for which reductions due to bond breakage, concrete creep, etc. should be evaluated.
3. All fire rated assemblies are subject to an upper load limit of 200 psf.

---

53
A.3 – Vulcraft Manual Page for 1.5B Roof Deck

### 1.5 B, BI, BA, BIA

Maximum Sheet Length 42'-0" Extra charge fo lengths under 6'-0"
ICC ER-3415
Factory Mutual Approved*

**Deck type & gauge — Max. deck span**
- 1.5B22, 1.5BI22: 6'-0"
- 1.5B16, 1.5BI16: 7'-5"

* Acoustical Deck is not approved by Factory Mutual.

**SECTION PROPERTIES**

<table>
<thead>
<tr>
<th>Deck type</th>
<th>Design thickness in.</th>
<th>V, lbf/ft²</th>
<th>f, psi</th>
<th>L₁, in.</th>
<th>L₂, in.</th>
<th>L₃, in.</th>
<th>Vₓ, lbf/ft²</th>
<th>Fₓ, kN/sq ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>B21</td>
<td>0.0299</td>
<td>1.16</td>
<td>54.89</td>
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<td>7.31</td>
<td>0.137</td>
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<td>14.16</td>
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<tr>
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<td>0.0299</td>
<td>1.18</td>
<td>54.89</td>
<td>21.17</td>
<td>7.31</td>
<td>0.137</td>
<td>40.14</td>
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<td>21.17</td>
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<td>2.32</td>
<td>54.89</td>
<td>21.17</td>
<td>7.31</td>
<td>0.137</td>
<td>40.14</td>
<td>14.16</td>
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**ACOUSTICAL INFORMATION**

<table>
<thead>
<tr>
<th>Deck Type</th>
<th>Absorption Coefficient</th>
<th>Sound Reduction Coefficient</th>
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<tr>
<td>1.5B, 1.5BA</td>
<td>1/2 0.18 66 1.0 0.6</td>
<td>0.33 0.00</td>
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</table>

1. Source: Riverbank Acoustical Laboratories

**VERTICAL LOADS FOR TYPE 1.5B**

<table>
<thead>
<tr>
<th>No. of Spans</th>
<th>Deck Type</th>
<th>Max. SD Cond. Span</th>
<th>3.5</th>
<th>5.0</th>
<th>6.5</th>
<th>7.5</th>
<th>8.0</th>
<th>9.0</th>
<th>10.5</th>
<th>12.0</th>
<th>15.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B21</td>
<td>90.3 99.1 60.0</td>
<td>174</td>
<td>180</td>
<td>215</td>
<td>240</td>
<td>280</td>
<td>320</td>
<td>370</td>
<td>420</td>
<td>510</td>
</tr>
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<td>90.3 99.1 60.0</td>
<td>174</td>
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<td>215</td>
<td>240</td>
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<td>370</td>
<td>420</td>
<td>510</td>
<td></td>
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<tr>
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<td>90.3 99.1 60.0</td>
<td>174</td>
<td>180</td>
<td>215</td>
<td>240</td>
<td>280</td>
<td>320</td>
<td>370</td>
<td>420</td>
<td>510</td>
<td></td>
</tr>
<tr>
<td>B19</td>
<td>90.3 99.1 60.0</td>
<td>174</td>
<td>180</td>
<td>215</td>
<td>240</td>
<td>280</td>
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<td>370</td>
<td>420</td>
<td>510</td>
<td></td>
</tr>
<tr>
<td>B16</td>
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<td>180</td>
<td>215</td>
<td>240</td>
<td>280</td>
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<td>370</td>
<td>420</td>
<td>510</td>
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<tr>
<td>2</td>
<td>B22</td>
<td>190.3 199.1 120.0</td>
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<td>240</td>
<td>280</td>
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<td>370</td>
<td>420</td>
<td>510</td>
</tr>
<tr>
<td>B20</td>
<td>190.3 199.1 120.0</td>
<td>174</td>
<td>180</td>
<td>215</td>
<td>240</td>
<td>280</td>
<td>320</td>
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<td>420</td>
<td>510</td>
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<tr>
<td>B19</td>
<td>190.3 199.1 120.0</td>
<td>174</td>
<td>180</td>
<td>215</td>
<td>240</td>
<td>280</td>
<td>320</td>
<td>370</td>
<td>420</td>
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<tr>
<td>B16</td>
<td>190.3 199.1 120.0</td>
<td>174</td>
<td>180</td>
<td>215</td>
<td>240</td>
<td>280</td>
<td>320</td>
<td>370</td>
<td>420</td>
<td>510</td>
<td></td>
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<tr>
<td>3</td>
<td>B22</td>
<td>219.3 229.1 180.0</td>
<td>174</td>
<td>180</td>
<td>215</td>
<td>240</td>
<td>280</td>
<td>320</td>
<td>370</td>
<td>420</td>
<td>510</td>
</tr>
<tr>
<td>B20</td>
<td>219.3 229.1 180.0</td>
<td>174</td>
<td>180</td>
<td>215</td>
<td>240</td>
<td>280</td>
<td>320</td>
<td>370</td>
<td>420</td>
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<td></td>
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<tr>
<td>B19</td>
<td>219.3 229.1 180.0</td>
<td>174</td>
<td>180</td>
<td>215</td>
<td>240</td>
<td>280</td>
<td>320</td>
<td>370</td>
<td>420</td>
<td>510</td>
<td></td>
</tr>
<tr>
<td>B16</td>
<td>219.3 229.1 180.0</td>
<td>174</td>
<td>180</td>
<td>215</td>
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<td>280</td>
<td>320</td>
<td>370</td>
<td>420</td>
<td>510</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1. Minimum exterior bearing length requires 1.50 feet. Minimum interior bearing length requires 2.00 feet.
### A.4 – Live Loads from ASCE 7-05

<table>
<thead>
<tr>
<th>Location</th>
<th>Live Loads (psf)</th>
<th>ASCE 7-05</th>
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<tbody>
<tr>
<td>Lobbies</td>
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</tr>
<tr>
<td>Hospitals</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Operating Rooms/Labs</td>
<td>60</td>
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<tr>
<td>Patient Rooms</td>
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<td>Corridors, above First Floor</td>
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<tr>
<td>First Floor Corridors</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Offices</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Stairs</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Mechanical</td>
<td>152</td>
<td></td>
</tr>
<tr>
<td>Roofs</td>
<td>20</td>
<td></td>
</tr>
</tbody>
</table>
Appendix B: Snow Load & Drift Calculations

B.1 - Snow Load and Drift Calculations

Snow Loads

The city of Erie, PA requires the use of 40 psf for the ground snow load. For these calculations, the use of 40 psf was required.

ASCE 7-05

Flat Roof Snow Load

\[ p_f = 0.7 C_o C_e I p_3 \]

\[ p_3 = 40 \text{ psf}, \text{ see note above} \]

\[ I = 1.1 \Rightarrow \text{Table 7-4 (ASCE 7-05)} \]

Occupancy Category II → No Emergency Facilities

\[ C_e = 1.0 \Rightarrow \text{Table 7-3 (ASCE 7-05)} \]

\[ C_e = 0.8 \Rightarrow \text{Table 7-2 (ASCE 7-05)} \]

- Fully Exposed
- Terrain Category D, on the lake

\[ p_f = 0.7 (0.8)(1.0)(1.1)(40 \text{ psf}) \]

\[ p_f = 33.64 \text{ psf!} \]
ASCE 7-05

Drift Snow Load (Penthouse Roof)

\[ y = 0.13 p_s + h = 0.13(16) + 11 = 19.2 \text{ psf} \]

N-S Drift

\[ l_u = 60' - 0'' \]
\[ h_c = 20' - 0'' \]
\[ h_d = 2.8' \]

E-W Drift

\[ l_u = 140' - 0'' \]
\[ h_c = 20' - 0'' \]
\[ h_d = 4.25' \]

\[ \therefore \text{Use } h_d = 4.25 \]

\[ w = 4 h_d = 17' - 0'' \]

\[ p_d = h_u y = (4.25 \times 19.2 \text{ psf}) + 24.6 = 106.2 \text{ psf} \]

\[ \therefore p_d = 106.2 \text{ psf} \]
B.3 - Snow Load and Drift Calculations (con’t)

ASCE 7-05

Drift Snow Load (Stair Pop-out)

\[ Y = 0.13 \rho_b + 14 = 5.13(40) + 14 = 19.2 \text{ pcf} \]

\[ h_a = 10' - 10" \]
\[ h_c = 10' - 0" \]
\[ \therefore h_d = 1.75' \]

\[ N = \text{Drift} \]
\[ E = \text{W Drift} \]

\[ h_a = 26' - 8" \]
\[ h_c = 10' - 0" \]
\[ \therefore h_d = 1.75' \]

\[ w = h_d = 4' \times 1.75' = 7' - 0" \]

\[ \rho_d = h_d \times Y = 1.75' \times (19.2 \text{ pcf}) + 24.6 = 58.2 \text{ pcf} \]

\[ \therefore \rho_d = 58.2 \text{ pcf} \]
B.4 - Drift Plan
Appendix C: Wind Load Calculations

C.1 – Wind Calculations

\textbf{Wind Loads}

\underline{ASCE 7-05}

\textit{Method 2 – Analytical Procedure}

\textit{Assume: Enclosed Building}

\textit{Rigid Building}

\textit{Wind From North}

\[ V = 90 \text{ mph} \rightarrow \text{Figure 6-1} \]

\[ K_d = 0.85 \rightarrow \text{Table 6-4} \]

\[ I = 1.15 \rightarrow \text{Table 81} \]

\textit{Occupancy Category = III \rightarrow Table 1-1}

\[ K_h \times K_z \rightarrow \text{Table 6-3} \rightarrow \text{Case 2} \]

\textit{Surface Roughness D \rightarrow Exposure D}

\begin{align*}
70' - 80' &= 1.38 \\
60' - 70' &= 1.34 \\
50' - 60' &= 1.31 \\
40' - 50' &= 1.27 \\
30' - 40' &= 1.22 \\
25' - 30' &= 1.16 \\
20' - 25' &= 1.13 \\
15' - 20' &= 1.08 \\
6' - 15' &= 1.03 \\
80' - 90' &= 1.40 \\
90' - 90' &= 1.41 \rightarrow \text{Interpolated Value}
\end{align*}
C.2 – Wind Calculations (con’t)

\[ K_{11} = F_g \cdot \sigma - 4 \]
\[ K_{11} = (1 + K_1 K_2 K_3)^2 \]

\[ K_1 = 0.75 (z) \]
\[ K_2 = \left(1 - \frac{1}{L_{11}} \right) \]
\[ = \left(1 - \frac{0}{y(60)} \right) \]
\[ = 1 \]

\[ K_3 = e^{-\frac{y^2}{L_{11}}} \]

\[ z = 80 \quad \begin{array}{l} 0.036 \\ 70 \quad 0.059 \\ 60 \quad 0.082 \\ 50 \quad 0.145 \\ 40 \quad 0.189 \\ 30 \quad 0.287 \\ 25 \quad 0.353 \\ 20 \quad 0.435 \\ 15 \quad 0.535 \\ 0 \quad 1.0 \end{array} \]

\[ z = 90 \quad 0.024 \]
C.3 – Wind Calculations (con’t)

\[
\begin{align*}
K_w &= 1.105 \\
K_{w0} &= 1.162 \\
K_{w15} &= 1.252 \\
K_{w30} &= 1.391 \\
K_{w45} &= 1.620 \\
K_{w60} &= 1.783 \\
K_{w75} &= 1.997 \\
K_{w90} &= 2.275 \\
K_{w105} &= 3.803
\end{align*}
\]

Gust Factor = Sec 6.5.8

\[ \frac{V}{U} = 0.85 \]

Enclosed Building ⇒ Figure 6-5

\[ G_C = 1/2 \times 0.18 \]

\[ C_p \text{ Values ⇒ Figure 6-6} \]

- \[ i_p = 0.8 \rightarrow \text{Windward Wall} \]
- \[ i_p = 0.5 \rightarrow \text{Leeward Wall} \]
- \[ i_p = -0.1 \rightarrow \text{Roof} \rightarrow 0° \rightarrow 39° \]
- \[ i_p = -0.9 \rightarrow \text{Roof} \rightarrow 39° \rightarrow 78° \]
- \[ i_p = -0.5 \rightarrow \text{Roof} \rightarrow 78° \rightarrow 145° \]
C.4 – Wind Calculations (con’t)

### Wind Loads (con’t)

\[ \bar{z} \text{ Values } \Rightarrow \text{Section 6.5.10} \]

<table>
<thead>
<tr>
<th>( \hat{z} )</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( z_{10} )</td>
<td>30.91</td>
</tr>
<tr>
<td>( z_{20} )</td>
<td>31.56</td>
</tr>
<tr>
<td>( z_{26} )</td>
<td>33.29</td>
</tr>
<tr>
<td>( z_{50} )</td>
<td>35.81</td>
</tr>
<tr>
<td>( z_{90} )</td>
<td>40.05</td>
</tr>
<tr>
<td>( z_{95} )</td>
<td>41.92</td>
</tr>
<tr>
<td>( z_{99} )</td>
<td>45.33</td>
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<tr>
<td>( \bar{z} )</td>
<td>49.80</td>
</tr>
<tr>
<td>( \hat{z} )</td>
<td>74.40</td>
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</table>

### Underside Wall Pressures \( \Rightarrow \text{Sec 6.5.12.4.2} \)

<table>
<thead>
<tr>
<th>Height (ft)</th>
<th>Wind Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>21.40</td>
</tr>
<tr>
<td>70</td>
<td>26.98</td>
</tr>
<tr>
<td>60</td>
<td>28.13</td>
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<tr>
<td>50</td>
<td>34.87</td>
</tr>
<tr>
<td>40</td>
<td>32.76</td>
</tr>
<tr>
<td>30</td>
<td>34.03</td>
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<tr>
<td>25</td>
<td>31.35</td>
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<td>20</td>
<td>34.39</td>
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<tr>
<td>15</td>
<td>51.51</td>
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</tbody>
</table>

### Leeward Wall Pressures \( \Rightarrow \text{Sec 6.5.12.4.2} \)

<table>
<thead>
<tr>
<th>Height (ft)</th>
<th>Wind Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-15.55</td>
</tr>
</tbody>
</table>
Wind loads (con't)

Wind from East or West

V = 90 mph ⇒ Figure 6-1

K_d = 2.85 ⇒ Table 6-1

I = 1.15 ⇒ Table 6-1

Occupancy Category = III ⇒ Table 1-1

K_n + K_z ⇒ Table 6-3 ⇒ Case 5

Surface Roughness B ⇒ Exposure D

<table>
<thead>
<tr>
<th>Surface Roughness</th>
<th>Exposure D</th>
</tr>
</thead>
<tbody>
<tr>
<td>70 - 80</td>
<td>1.38</td>
</tr>
<tr>
<td>60 - 70</td>
<td>1.34</td>
</tr>
<tr>
<td>50 - 60</td>
<td>1.31</td>
</tr>
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<td>40 - 50</td>
<td>1.27</td>
</tr>
<tr>
<td>30 - 40</td>
<td>1.22</td>
</tr>
<tr>
<td>25 - 30</td>
<td>1.16</td>
</tr>
<tr>
<td>20 - 25</td>
<td>1.12</td>
</tr>
<tr>
<td>15 - 20</td>
<td>1.08</td>
</tr>
<tr>
<td>0 - 15</td>
<td>1.03</td>
</tr>
</tbody>
</table>

K_{et} = 1.0 ⇒ No Ridge in this direction ⇒ Sec 6.5.7.2

Gust Factor ⇒ Sec 6.5.8

g = 0.85

Enclosed Building ⇒ Figure 6-5

G_{f} = \frac{1}{2} \times 0.18
C.6 – Wind Calculations (con’t)

Wind Loads (cont)

$C_p$ Values → Figure 6-2

- $C_p = 0.8 \rightarrow$ Windward Wall
- $C_p = -0.37 \rightarrow$ Lee wind wall (Interpolated)
- $C_p = 0.9 \rightarrow$ Roof $= 0^\circ$ to $31^\circ$ (Plan View)
- $C_p = -0.9 \rightarrow$ Roof $= 31^\circ$ to $78^\circ$
- $C_p = -0.5 \rightarrow$ Roof $= 78^\circ$ to $156^\circ$
- $C_p = -0.3 \rightarrow$ Roof $> 156^\circ$

$z_0$ Values → Section 6.5.10

- $z_0 = 27.97$
- $z_{170} = 27.16$
- $z_{310} = 26.55$
- $z_{510} = 25.74$
- $z_{740} = 24.73$
- $z_{1140} = 23.51$
- $z_{175} = 22.20$
- $z_{210} = 21.89$
- $z_{250} = 20.58$
C.7 – Wind Calculations (con’t)

Wind Loads (cont)

Windward Wall Pressures \( \Rightarrow \) Sec 6.5.12.4.2

\[
\begin{align*}
P_{20} &= 24.16 \\
P_{70} &= 23.47 \\
P_{60} &= 23.05 \\
P_{50} &= 22.50 \\
\vdots
\end{align*}
\]

Leeward Wall Pressures \( \Rightarrow \) Sec 6.5.12.4.2

\[
\rho = -14.13
\]
### C.8 – Wind Calculations (con't)

#### Basic Shear and Overturning Moment Calculator

**Description: Wind from North**

<table>
<thead>
<tr>
<th>Model Building</th>
<th>( F_{ref} )</th>
<th>( F_{ref} )</th>
<th>( V )</th>
<th>( M )</th>
</tr>
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<tbody>
<tr>
<td>30 ft</td>
<td>72 lb</td>
<td>60 lb</td>
<td>3.5 kip</td>
<td>1020.5 kip ft</td>
</tr>
<tr>
<td>50 ft</td>
<td>72 lb</td>
<td>70 lb</td>
<td>3.8 kip</td>
<td>1230.5 kip ft</td>
</tr>
<tr>
<td>50 ft</td>
<td>60 lb</td>
<td>60 lb</td>
<td>3.2 kip</td>
<td>860.5 kip ft</td>
</tr>
<tr>
<td>40 ft</td>
<td>60 lb</td>
<td>50 lb</td>
<td>2.8 kip</td>
<td>620.5 kip ft</td>
</tr>
<tr>
<td>30 ft</td>
<td>60 lb</td>
<td>40 lb</td>
<td>2.3 kip</td>
<td>460.5 kip ft</td>
</tr>
<tr>
<td>20 ft</td>
<td>40 lb</td>
<td>30 lb</td>
<td>1.7 kip</td>
<td>300.5 kip ft</td>
</tr>
<tr>
<td>20 ft</td>
<td>40 lb</td>
<td>20 lb</td>
<td>1.5 kip</td>
<td>200.5 kip ft</td>
</tr>
<tr>
<td>10 ft</td>
<td>20 lb</td>
<td>10 lb</td>
<td>1.5 kip</td>
<td>100.5 kip ft</td>
</tr>
</tbody>
</table>

#### Stair Pop-Out

\( F_{ref} \) = 30 ft, \( V \) = 3 kip, \( M \) = 150.5 kip ft

#### Mechanical Penthouse

\( F_{ref} \) = 30 ft, \( V \) = 5 kip, \( M \) = 200.5 kip ft

#### Section

\( F_{ref} \) = 30 ft, \( V \) = 5 kip, \( M \) = 200.5 kip ft

#### Total

\( V_{ref} \) = 40.9 kip, \( M_{ref} \) = 1330 kip ft
Appendix D: Seismic Calculations

D.1 – Seismic Calculations

EQ Load

**ASCE 7-05**

\[ R = 3 - \text{Not Specifically Detailed For Seismic} \Rightarrow \text{Table 12.2-1} \]
\[ I = 1.85 \Rightarrow \text{Table 12.8-1} \]

\[ T = C_u T_a \]
\[ T_L = 1.2 \Rightarrow \text{Fig 22-15} \]

\[ C_u = 1.7 \Rightarrow \text{Table 12.8-1} \]
\[ T_a = C_v \frac{l_a}{h_a} = 0.028 (92')^{0.8} = 1.043 \]

\[ T = 1.7(1.043) = 1.773 \]

\[ C_{ps} = 0.175 \]
\[ S_{dv} = 0.078 \]

From ULSAS

\[ C_e = \min \left( \frac{S_{dv}}{S_{dv} + T}, \frac{C_v}{T_a} \right) = 0.078 (0.12) = 0.1241 \]

\[ \therefore C_e = 0.0183 \]

\[ V = C_e W = 0.0183 (11,626) \]

\[ \therefore V = 210.39 \text{ kips} \]
D.2 – Seismic Calculations (con’t)

**EQ Loads (cont)**

\[
\begin{align*}
W_{pl} &= 815.4 \text{k} \\
W_{br} &= 74.3 \text{k} \\
W_{pl} &= 1616.6 \text{k} \\
W_{s} &= 2882.7 \text{k} \\
W_{y} &= 2348.6 \text{k} \\
W_{s} &= 2401.9 \text{k} \\
W_{m} &= 2567.1 \text{k}
\end{align*}
\]

\[
W_{ph} = 92' \\
W_{br} = 82' \\
W_{pl} = 72' \\
W_{s} = 58' \\
W_{y} = 44' \\
W_{s} = 28' \\
W_{m} = 12'
\]

\[
k = 1.5865 \Rightarrow \text{Interpolation}
\]

\[
\begin{align*}
PH & \quad W_{ph} \quad h_{ph} \quad k \quad = 313,750 \\
SR & \quad W_{br} \quad h_{br} \quad k \quad = 62,005 \\
P & \quad W_{pl} \quad h_{pl} \quad k \quad = 1,105,756 \\
J & \quad W_{s} \quad h_{s} \quad k \quad = 1,128,449 \\
4 & \quad W_{y} \quad h_{y} \quad k \quad = 757,774 \\
3 & \quad W_{s} \quad h_{s} \quad k \quad = 388,724 \\
2 & \quad W_{m} \quad h_{m} \quad k \quad = 113,976
\end{align*}
\]

\[
\frac{3,864,830}{3,864,830}
\]

\[
C_{vpl} = 0.08168 \\
C_{var} = 0.01604 \\
C_{va} = 0.28611 \\
C_{y} = 0.29053 \\
C_{s} = 0.19607 \\
C_{s} = 0.10058 \\
C_{m} = 0.08749
\]
D.3 – Seismic Calculations (con’t)

E & Loads (cont)

\[ F_{PL} = C_{VPL} V = 17.24 \text{ k} \]
\[ F_{SR} = C_{VSR} V = 3.41 \text{ k} \]
\[ F_{SK} = C_{VSK} V = 60.77 \text{ k} \]
\[ F_{S} = C_{VS} V = 61.71 \text{ k} \]
\[ F_{S1} = C_{VS1} V = 41.64 \text{ k} \]
\[ F_{S2} = C_{VS2} V = 81.36 \text{ k} \]
\[ F_{S3} = C_{VS3} V = 6.86 \text{ k} \]
Appendix E: Spot Checks

E.1 – Decking Check

\[
\begin{align*}
\text{Decking} \\
\text{Span} &= \frac{21\text{.4}''}{3} = 7' - 2'' \\
n &= 4'' \rightarrow \text{total thickness} = 6'' \\
\text{AVLI 20 Deck} \\
\text{Loads} \\
\text{Dead} &= 69 \text{ psf} \\
\text{Live} &= 80 \text{ psf} \\
\frac{199 \text{ psf}}{48 \text{ psf}} \Rightarrow \text{Deck 45lb SW} \\
\text{u} &= 101 \text{ psf} \leq 278 \text{ psf} \ (7' - 6'' \text{ clear}; \text{AVLI}) \quad \therefore \text{OK}
\end{align*}
\]

Max Unshared Clear Span

- 3 span condition met

\[
5 = 7' - 2'' < 10' - 9'' = S_{max} \\
\therefore \text{OK}
\]
E.2 – Beam Check

Beam

Composite Beam: W 14 x 22 [in]

\[ A_b = 6.49 \text{ in}^2 \]
\[ f_x = 199 \text{ ksi} \]
\[ F_y = 50 \text{ ksi} \]
\[ d = 13.7'' \]

\[ W_i = 64 \text{ psi} = 506 \text{ pfsf} \]
\[ W_e = 80 \text{ psi} = 586.67 \text{ pfsf} \]

\[ W_u = 1.2 W_i + 1.6 W_e = 1.2 (506) + 1.6 (587) \]

\[ W_u = 1.55 \text{ ksf} \]

\[ V_u = \frac{W_u L}{2} = \frac{1.55 (24.6)}{2} = 19.62 \text{ ksf} \]

\[ M_u = \frac{W_u L^2}{8} = \frac{1.55 (24.6)^2}{8} = 117.9 \text{ k-ft} \]

\[ b_{ess} = \frac{\text{Span}}{y} = \frac{6.17}{6.17'} = \text{controls} \]

\[ M \]

\[ b_{ess} = 74'' \]

\[ V_c = 0.85 F_c b_{ess} t_i = 0.85 (4) (74) (6.17') = 1594.6 \text{ k} \]

\[ V_s = F_y A_s = 50 (6.49) = 324.5 \text{ k} \]

\[ V_c' > V_s' \Rightarrow \text{NA in concrete} \]
E.3 – Beam Check (con’t)

\[
V_s' = 0.85 \frac{f_c'(b)}{b} (x) \quad \Rightarrow \quad \alpha = \frac{V_s'}{0.85 \frac{f_c'}{b} b_{ess}} = \frac{384.5}{0.85 (N/lb)}
\]

\[
\alpha = 1.29''
\]

\[
M_n = \frac{V_s (\frac{d}{2} + t - \frac{9d}{12})}{12} = \frac{384.5 \left( \frac{12d}{2} + 6 - \frac{12d}{12} \right)}{12} = 330.0 \text{ ft-lb}
\]

\[
\phi M_n = 0.9 M_n = 0.9 (330.0 \text{ ft-lb})
\]

\[
\phi M_n = 307.0 \text{ ft-lb}
\]

\[
\phi M_n > M_n \quad \Rightarrow \quad \text{OK}
\]

\[
\phi V_s = 945 \text{ ft-lb} > V_s = 19.12 \text{ ft-lb}
\]

\[
\Leftrightarrow \quad \text{Table 3-2}
\]

\[
\Delta_l = \frac{360}{360} = \frac{2422 (12)}{360} = 0.8222 \text{ in}
\]

\[
\epsilon_{0r} = \frac{A_s f_y}{A} = \frac{324.5}{360} \Rightarrow \text{Concrete}
\]

\[
\epsilon = \frac{A_s f_y}{A_s + \frac{\epsilon_{0r} f_y}{f_y}} = \frac{6.49 \left( \frac{157}{2} \right) + 324.5 \left( \frac{157 + 5.36}{2} \right)}{6.49 + 324.5} = 0.455
\]

\[
\therefore \quad \gamma = 0.555''
\]
E.4 – Beam Check (con’t)

\[ L_6 = \frac{I_{x_{6b}} + A_y (y - \frac{h}{2})^2 + \frac{2}{5} A_z (l_y + 2l_z - 2y)^2}{E' I_{x_{6b}}} = 199 + 6.49 (12.975 - 12.76)^2 + \frac{2}{5} \times \frac{5.36}{20} (13.7 + 5.36 - 12.975)^2 \]

\[ I_{6b} = 682.8 \text{ in}^4 \]

\[ \Delta_{6b} = \frac{5E \Delta}{381 E I_{x_{6b}}} = \frac{5 \times (587)(24.67)^4}{381 \times (20000)(682.8)} = 0.247 \text{ in} \]

\[ \Delta_{6b} = 0.247'' < 0.8222'' \quad \therefore \text{OK} \]

Wet Concrete Deflection

\[ \Delta_{max} = \frac{4}{3} f_c \frac{L}{x_{4b}} = \frac{4 \times 24,650 (12)}{24.65} = 1.733'' \]

\[ W = \frac{69 (7.167) + 22 \rho f}{0.547 \text{ kif}} = 0.317 \text{ kif} \]

\[ I_{rec} = \frac{5E \Delta_{max}^2}{381 E x_{4b} \Delta_{max}} = \frac{5 \times (0.577)(24.67)^{12.88}}{381 \times (20000)(1.233)} \]

\[ I_{rec} = 120.3 \text{ in}^4 < 199 \text{ in}^4 \quad \therefore \text{OK} \]

\[ 10W H \times 22 \text{ Works} \]
E.5 – Girder Check

Composite: W16 x 26 [18]

$F_y = 36,000$ psi
$A_w = 7.68$ in$^2$
$F_y = 50$ ksi
$d = 15.7$ in

$P_a = 2(17.12k) = 34.24k$

$V_u = 38.47k$

$M_u = 373.2k \cdot \frac{d}{2}$

$b_{ef} = \min \left( \frac{Sp_{ey}}{2}, \frac{Sp_{ey}}{2} \cdot \frac{d}{4} \right) = \text{controls}$

$V'_c = 0.85 \cdot \frac{F_y}{b_{ef}} \cdot b_{ef} +\frac{1}{2} g(13)(6)$

$V'_c = \frac{F_y}{A_s} = 50(2.68)$

$\therefore V'_c = 805.6k$

$V'_s = 334k$

$V'_c > V'_s \therefore \text{NA in concrete}$
E.6 – Girder Check (con’t)

\[ v_s = 0.85 \bar{f}_c \left( \frac{L_{eff}}{2} \right) \Rightarrow a = \frac{v_s}{0.85 \bar{f}_c \cdot L_{eff}} = \frac{384}{0.85 \cdot (15.7\pi)} \]

\[ a = 1.7^2'' \]

\[ M_n = \frac{v_s (A \cdot l + t - g/2)}{12} = \frac{384 (15.7\pi + 6 - 1.7\pi \cdot 2)}{12} = 95.0 \ k \cdot \text{f} \]

\[ \phi M_n = 0.9 M_n = 0.9 (95.0) \]

\[ \therefore \phi M_n = 373.5 \ \text{k-\text{f}} \]

\[ \phi M_n > M_n : \text{OK} \]

\[ V_n = 106 k > V_n = 38.5 k \]

\[ \therefore \text{OK} \]

\[ \Delta_L = \frac{L}{860} = 0.33 (12) / 860 = 0.711'' \]

\[ \delta a = 0.85 \bar{f}_c \cdot L_{eff} \Rightarrow 1305.6 \]

\[ \delta a = 384 \Rightarrow \text{controls} \]

\[ \gamma_0 = \frac{t \cdot d_{lab}}{a} = \frac{6 - 1.7\pi}{2} = 5.12'' \]

\[ \bar{y} = \frac{A (0.5) + \frac{A_f}{F_f} \cdot (0.6)}{A_f + \frac{A_{lab}}{F_f}} = \frac{768 (15.7\pi) + 384}{50 (15.7 + 5.12')} \]

\[ \therefore \bar{y} = 14.335'' \]
E.7 – Girder Check (cont)

\[ I_{xx} = I_{xx0} + A \left( y - \frac{h}{2} \right)^2 + \frac{8a}{h^2} (d + y_a - y)^2 \]
\[ = 301 + \frac{7.62 \left( 14.855 - 15.72 \right)^2}{2} + \frac{384}{30} \left( 15.7 + 5.12 - 19.35 \right)^2 \]
\[ I_{xx} = 947.0 \text{ in}^4 \]

\[ \Delta_{ul} = \frac{P_{u} (a)}{2EI_{xx}} \left( 3a^2 - 4a^2 \right) \]
\[ = 0.314'' \]
\[ \Delta_l = 0.314'' < \Delta_{ul, max} = 0.71'' \]
\[ \therefore \text{OK} \]

Wet Concrete Deflection

\[ \Delta_{max} = \frac{6P_{u}a}{24El_{40}} = \frac{a}{24} \left( \frac{a}{24} \right) = 1.07'' \]

\[ I_{eq} = \frac{P_{u}a}{2EI_{xx}} \left( 3a^2 - 4a^2 \right) \]
\[ = \frac{14.48 \times (85.33)}{24 \times 9600 \times (1.07)} \left( 3 \times 85.33^2 - 4 \times 85.33^2 \right) \]
\[ \therefore I_{eq} = 277.9 \text{ in}^4 < I_{xx} = 301 \text{ in}^4 \]
\[ \therefore \text{OK} \]

\[ \therefore W16 \times 26 \text{ Works} \]
E.8 – Column Check

Influence Area

\[ A_i = (19'-4" + 19'-4")(24'-8" + 24'-8"') \]
\[ \therefore A_i = 2006.2 \, \text{ft}^2 \]

Tributary Area

\[ A_t = \left( \frac{19'-4" + 21'-4"'}{2} \right) (24'-8") \]
\[ \therefore A_t = 501.6 \, \text{ft}^2 \]
\[ \therefore K_{ct} = 4 \]
E.9 – Column Check (con’t)

Column Loads

Below 5th

\[ P_c = 21.64 \text{psf} \left(501.6 \text{ ft}^2 \right) = 12.36 \text{k} \]
\[ P_L = 0.985(90)(501.6 \text{ ft}^2) = 43.47 \text{k} \]
\[ U_{\text{red}} = 0.25 + \frac{15}{4 \times 301.6} = 0.585 \]
\[ P_D = 20(501.6) + 69(501.6) = 44.62 \text{k} \]
\[ P_D = 1.2(14.62) + 1.6(23.47) + 0.5(12.36) \]
\[ \therefore P_{\text{tot}} = 97.28 \text{k} \]

Below 3rd

\[ P_c = 12.36 \text{k} \]
\[ P_D = 20(501.6) + 3.6(501.6) = 113.86 \text{k} \]
\[ P_L = 0.443(3)(90)(501.6) = 53.38 \text{k} \]
\[ U_{\text{red}} = 0.25 + \frac{15}{4 \times 301.6} = 0.443 \]
\[ P_D = 1.2(13.86) + 1.6(53.38) + 0.5(12.36) \]
\[ \therefore P_{\text{tot}} = 228.14 \text{k} \]
E.10 – Column Check (con’t)

Column (cont)

Column Loads

Below 2° (4 Floors of Reduable Area)

\[ P_a = 12.36 \, k \]
\[ P_b = 20(50.6) + 4(41)(50.6) = 48.47 \, k \]
\[ P_c = 0.97(4)(70)(50.6) = 26.95 \, k \]

\[ U_{red} = 0.25 \div \frac{15}{14 \times 4(50.6)} = 0.417 \]

\[ P_u = 1.2(48.47) + 1.6(26.95) + 0.5(12.34) \]

\[ \therefore P_{u2} = 291.45 \, k \]
E.11 – Column Check (con’t)

Max. Column Capacity (Assume k = 1.0 = conservative)

\[ A_2 = \frac{141.1}{12} \]
\[ \gamma_f = 2.08 \]

\[ F_c = \frac{\gamma_f^2 E_2}{(kE_2) \alpha} = \frac{2.08^2 (29000)}{(2.08(141.1))} = 43.87 \]

\[ F_{cr} = \left[ 0.658 \left( \frac{F_{cr}}{F_c} \right) \right] F_y = \left[ 0.658 \left( \frac{658}{437.56} \right) \right] 437.56 = 3103 \]

\[ P_n = F_c A_2 = 3103 \times 141.1 = 43756 \]
\[ \phi P_n = 0.9 P_n = 0.9 \times 43756 = 39379 > P_{uf} = 9728 \]

\[ \text{∴ } W8 \times 48 \text{ Works} \]
E.12 – Column Check (con’t)

Max Column Capacity (Assume $k=1.0$ ⇒ conservative)

\[ W_8 \times 67 \]

\[ A_g = 19.7 \text{ in}^2 \]
\[ f_x = 3.72 \]
\[ f_y = 2.12 \]

\[ F_e = \frac{\pi^2 E}{(k f_y)^2} \]
\[ = \frac{\pi^2 (29,000)}{(1.0 (2.12))^2} = 140.33 \]

\[ F_e = \frac{\pi^2 E}{(k f_y)^2} \]
\[ = \frac{\pi^2 (29,000)}{(1.0 (2.12))^2} = 34.89 \]

\[ F_{cr} = \frac{0.658 \left( \frac{f_y}{f_e} \right) f_y}{50} = 43.07 \]

\[ F_{cr} = \frac{0.658 \left( \frac{f_y}{f_e} \right) f_y}{50} = 27.45 \]

\[ P_h = F_{cr} A_g \]
\[ = 43.07 \text{ (19.7)} = 829.86 \text{ k} \]

\[ \phi P_h = 0.9 (829.86) \]
\[ \phi P_h = 756.62 \text{ k} \]

\[ \text{check Slenderness} \]
\[ \frac{k l}{r} \approx 200 \]
\[ \frac{k l}{r} = \frac{10 (2508)}{2.12} = 90.57 \leq 200 \]

\[ \therefore W_8 \times 67 \text{ OK} \]
Max Column Capacity (Assume $k=1.0$ = Conservative)

$A_g = 17.7 \text{ in}^2$
$F_y = 2.12$

$F_e = \frac{\pi^2 E}{(\frac{F_y}{A_y})^2} = \frac{7.15(29000)}{1.0(180.10)^2} = 67.04$

$F_{cr} = \left[0.658 \left(\frac{F_y}{F_e}\right)\right] F_y = \left[0.658 \left(\frac{2.12}{67.04}\right)\right] 58 = 35.68$

$P_n = F_{cr} A_g = 35.68 (17.7) = 702.96 \text{ k}$
$\phi P_n = 0.9 (702.96)$

$\therefore \phi P_n = 632.66 > 871.45 \text{ k} = P_{cr} \therefore \text{OK}$

Slenderness

$k_{fl} \leq 200 \quad \frac{k_{fl}}{F_y} = \frac{1.0 (180.10)}{2.12} = 67.92 \leq 200 \therefore \text{OK}$

::: EXW67 Works
Appendix F: Relevant Building Plans

F.1 – S200 - Second Floor Structural Plan
F.2 – S302 - Moment Frame Elevations
F.3 – S303 - Braced Frame Elevations
F.4 – S400 - Column Schedule and Typical Column Reinforcements

<table>
<thead>
<tr>
<th>Column Number</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Column A</td>
</tr>
<tr>
<td>2</td>
<td>Column B</td>
</tr>
<tr>
<td>3</td>
<td>Column C</td>
</tr>
</tbody>
</table>

- **Column Schedule**
- **Typical Column Reinforcements**
F.5 – S401 - Column Schedule and Typical Beam Reinforcements
F.6 – S402 – Column Schedule and Typical Beam Reinforcements
F.7 – S403 - Foundation Overbuilds