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Executive summary

According to present design, the lateral force resisting system of Schenley Place consists of eccentrically and concentrically braced frames located at the building’s core. The braces are hollow steel shapes designed to transfer the lateral loads from the diaphragm to the wide-flange steel columns of the system. These wide-flange steel columns then transfer the loads to the cast-in-place concrete columns of the garage levels, where the load then distributes to the supporting foundation system and terminates in the surrounding soils. The following technical report contains a detailed analysis of this system.

Throughout the ensuing report, an ETABS model verifies the results of hand calculations and determines design values that are otherwise difficult to obtain. This model represents an analysis of the existing lateral members only. Diaphragms were modeled as rigid area elements with applied area masses. The ETABS model was used to determine the Fundamental Period of the building, in both the x and y-directions. Based on these presumably more accurate findings, previous seismic load calculations—performed in Technical Report 1: Structural Concepts and Existing Conditions—were revised. Wind loads too were recalculated to reflect the findings of this report.

Prior to applying the lateral loads to the ETABS model, an arbitrary force was exerted on each level in both the x and y-directions to determine the relative stiffness of each lateral braced frame. Distributions of lateral loads as well the center of rigidity of each level were determined in accordance with known relative stiffness. The results of this analysis were later compared to the effects of direct and torsional shear. Upon applying lateral loads to the ETABS model, the basic load combinations of ASCE 7-05 were evaluated in order to determine the controlling load case. This analysis concluded that wind loads control in the west-east direction, where as seismic loads control in the north-south direction. Once these governing load combinations were determined, an extensive analysis of the lateral force-resisting system was performed.

The ensuing analysis further contains an evaluation of the frames for direct and torsional shear, as well as an assessment of the building’s torsion produced by eccentricity. Furthermore, story drifts computed in ETABS were compared to the allowable wind drift industry standard and allowable drift due to seismic loads as dictated by code. Additionally, several spot checks on critical members as well a preliminary evaluation of overturning were performed.
Introduction

Schenley Place is a new office located at 4420 Bayard Street, in central Oakland, an inner-city neighborhood of Pittsburgh, PA. The building contains a total of 7 levels above grade and 3.5 levels of parking garage mostly below grade. On the fourth and remaining levels of the building’s north elevation, there is a substantial step-back in the building’s original footprint, dictated by the specialty zoning constraints placed upon the building site. The site is fully landscaped and contains a small pocket park along the building’s east elevation that is shared by the neighboring First Baptist Church of Pittsburgh.

Schenley Place has been designed to accommodate a variety of office type tenants. The first floor opens to a finished main building lobby with remaining unfinished space available for tenant occupancy. The remaining floors have open, unfinished office space to accommodate tenants. Supported on the roof of the building are HVAC units, an emergency generator, as well as a penthouse that accommodates both the electrical and elevator control room.

Due to its proximity to Schenley Farms (a historic residential district) and the First Baptist Church of Pittsburgh (a designated historic structure) various zoning and design constraints bear upon Schenley Place. Therefore, historic protection zoning ordinances controlled the building’s design. The zoning limit can be seen in Figure 1, defined by the thick black, broken line.

In compliance with the historic protection zoning ordinance, the building must be set-back 30 feet from the center-line of Bayard Street, and cannot exceed 40 feet in elevation. At 50 feet from the center-line of Bayard Street, the building height cannot exceed 50 feet in elevation. There is no longer a limitation in building height at 100 feet from the center-line of Bayard Street. As seen by Figure 1, the step-back at the fourth level is a design feature dictated by the zoning ordinances. Originally designed to be a 10-story building, the final 7-story design was based on the decision that the actual height of the neighboring First Baptist Church was governed by its roof ridge, rather than the height of its spire. Because the owner was concerned with maximizing the available rentable space, the structural design of...
Schenley Place results in heavier, shallower steel members in order maintain attractive ceiling heights without losing an additional rentable story.

The architectural design of Schenley Place also reflects the various constraints dictated by the location of the building site: For example, the three-story façade facing Bayard Street is mainly Indiana limestone to mimic the neighboring First Baptist Church. Moreover, the seven-story façade facing Ruskin Avenue is primarily buff-colored brick to compliment Ruskin Hall, a dormitory belonging to the University of Pittsburgh. Within the details of the building’s façade are punched aluminum windows, cast-stone cornices, sills, and headers, brick details, and aluminum curtain walls. Masonry parapets occur at the step-back on the fourth level and the roof level. The roof top penthouse is clad in metal panels and the HVAC units are disguised by metal screenwall.
Existing structural system summary

Foundation system summary

The foundation of Schenley Place incorporates a cast-in-place concrete perimeter caisson wall designed to act as the shoring system, and drilled cast-in-place caissons coupled with grade beams designed to support wall loads.

The perimeter caisson installer will design the cast-in-place perimeter caisson walls. The cast-in-place drilled caissons have a compressive strength of 4000 psi and are designed with an end bearing capacity of 25 tons per square foot (tsf). Additionally, they vary from 2'-6" to 4'-6" in diameter. At the first floor, the drilled caissons terminate and are tied to 2'-0"x2'-0" (24"x24") caisson caps with #6 dowels embedded at least two feet into the drilled caisson. The grade beams have a compressive strength of 4000 psi and range from 2'-0" to 3'-0" in width and 3'-0" to 3'-8" in depth. Each grade beam has top and bottom reinforcement with bar schedules that vary according to the size of the beam. The slab on-grade has a compressive strength of 4000 psi and is reinforced with 2x2-W1.4xW1.4 welded wire fabric, with a minimum thickness of 4".

Floor system summary

The floor system of the below grade parking levels of Schenley Place, as well as the first floor\(^1\), employs a two-way concrete flat slab with a minimum thickness of 0'-11" and a compressive strength of 5000 psi. Reinforcement is primarily #5 and #7 top and bottom bars, with depths based upon a slab thickness of 0'-11". The spacing of reinforcement

---

\(^1\) The first floor is on grade at the north elevation, and 9'-0" above grade at the south elevation.
depends on location. Additional reinforcement is placed at the cast-in-place concrete walls where necessary.

The typical floor system of the above grade office spaces are designed as 3 ½” normal weight cast-in-place concrete slabs, reinforced with 6x6-W1.4xW1.4 welded wire fabric. Slabs are cast on 3”-20 gauge composite steel decking that is supported by composite steel beams. Steel beams are connected to concrete slabs through ¾” diameter by 5½” long headed shear studs. Interior and exterior spans are on average 30’-0” and 18’-0”. Interior beams have a typical nominal depth of 1’-6” (18") and exterior beams have a typical nominal depth of 1’-0” (12"). The beams are generally spaced at 9’-0” center-to-center.

**Superstructure system summary**

The below grade gravity system of Schenley Place is a combination of cast-in-place concrete columns and load bearing walls. The cast-in-place concrete columns have a compressive strength of 7000 psi and are sized at 24”x28” or 18”x30”. Concrete columns typically span the height of the parking garage and terminate at the first floor. Additionally, they are reinforced with both #9 and #11 vertical bars and #3 and #4 horizontal ties. Concrete columns are spaced on average at 27’-0” in the north-south direction and 30’-0” in the west-east direction. The cast-in-place concrete load bearing walls have a compressive strength of 5000 psi and a thickness of 0’-8” or 1’-0”. These walls are reinforced with #5 or #6 bars at 12 or 16 inches on center. In addition, the location of load bearing walls correspond with the location of grade beams.

The typical above grade gravity system is designed as a steel frame, consisting of wide-flange steel shapes with yield strengths of 50 ksi. Steel beams span the west-east direction. Typical beam spacing, spans, and nominal depths are as stated above in the discussion of the floor system. Steel girders span the exterior north-south direction at 22’-9” or 21’-0” and the interior north-south direction at 27’-0”. Generally, the girders have a nominal depth of 18” and are typically spaced at 18’-0” or 30’-0” center-to-center. Steel columns span up to three stories before a splice is required. Column sizes depend on the gravity loads that each column is required to carry.

**Roof system summary**

The roof system of Schenley Place at both the fourth level and main roof employs 1½”-20 gage wide-rib, galvanized steel roof decking, supported by non-composite steel beams. Steel beams have average interior and exterior spans of 30’-0” and 18’-0” and are commonly spaced at 5’-0”. Where the main roof houses the rooftop mechanical

---

2 Typical story heights are 13’-4”.
3 The "main roof" refers to the roof level above the seventh floor.
units and penthouse, additional steel beams are designed to support the increased loads.

**Lateral force resisting system summary**

The following terms apply throughout this report: A *braced frame* is essentially a vertical truss, or its equivalent, provided in a building frame or dual system to resist lateral forces. *Concentrically braced frames* (CBF) have members that are primarily subjected to axial forces. *Eccentrically braced frames* (EBF) are diagonally braced frames in which at least one end of each brace frames into a beam a short distance from a beam-column or another diagonal brace.

The lateral force resisting system, located at the building’s core, is designed of both eccentrically and concentrically braced frames. Beginning on the first floor, the braced frames continue to the main roof level. The eccentrically braced frames span the west-east direction at 30’-0” between column lines C and D, and the concentrically braced frames span the north-south direction at 27’-0” between column lines 4 and 5.1 (Figure 3). The braces are designed as hollow steel shapes with yield strengths of 46 ksi. Refer to Appendix A for detailed lateral bracing elevations that include the column, beam, and HSS-brace sizes of each individual braced frame. In order to connect the braces to the column and/or beam, gusset plates with a minimum thickness to match the respective beam web are welded to the beam and/or connected by a double angle to the column. The HSS-braces are slotted as required and bolted to the gusset plate.
### Codes and design standards

*The following codes were used by the design professionals, Atlantic Engineering Services (AES). In the analysis of the lateral force resisting system the same codes were used.*

#### Relevant codes

- International Building Code (IBC), 2006  
  *(As amended by the city of Pittsburgh)*
- Minimum Design Loads for Buildings and Other Structures (2005), American Society for Civil Engineers (ASCE 7-05)
- Building Code Requirements for Structural Concrete, American Concrete Institute (ACI 318-08)
- Specification for Structural Steel Buildings, American Institute of Steel Construction (AISC 360-05)

*The following material strengths were used by the design professionals, AES. While analyzing the lateral force resisting system, these material strengths were also used, unless otherwise noted.*

#### Material strength requirement summary

##### Cast-in-place concrete

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Strength Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shallow foundations</td>
<td>f′c = 3000 psi</td>
</tr>
<tr>
<td>Caissons, grade beams, slabs on grade, and elevated floor slabs on deck</td>
<td>f′c = 4000 psi</td>
</tr>
<tr>
<td>Walls, beams, and formed elevated slabs</td>
<td>f′c = 5000 psi</td>
</tr>
<tr>
<td>Columns</td>
<td>f′c = 7000 psi</td>
</tr>
</tbody>
</table>

##### Reinforcement

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Strength Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deformed bars</td>
<td>Fy = 60 ksi</td>
</tr>
<tr>
<td>Welded wire fabric</td>
<td>Fy = 65 ksi</td>
</tr>
</tbody>
</table>

##### Structural steel

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Strength Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural W-shapes and channels</td>
<td>Fy = 50 ksi</td>
</tr>
<tr>
<td>Steel tubes (HSS shapes)</td>
<td>Fy = 46 ksi</td>
</tr>
<tr>
<td>Angles and plates</td>
<td>Fy = 36 ksi</td>
</tr>
<tr>
<td>¾“ bolts</td>
<td>ASTM A325</td>
</tr>
<tr>
<td>Composite steel deck (a minimum of 3&quot;-20 gage)</td>
<td>Fy ≥ 33 ksi</td>
</tr>
<tr>
<td>Steel roof deck (a minimum of 1½&quot;-20 gage)</td>
<td>Fy ≥ 33 ksi</td>
</tr>
</tbody>
</table>

##### Masonry

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Strength Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength</td>
<td>f′c = 1500 psi</td>
</tr>
</tbody>
</table>
Summary of design loads

Gravity design load summary

The following (Figure 4) is a comparative summary of the design gravity loads implemented by Atlantic Engineering Services (AES), as outlined in ASCE 7-05, and as applied in this report. Although the live loads vary according to the designed area, the owner requested that floor assemblies be designed with a 100 psf live load. In accordance with the owner’s request, this report uses a 100 psf live load where applicable. Note that AES does not apply live load reductions in their designs. To achieve comparable results, this report does not implement live load reductions.

<table>
<thead>
<tr>
<th>AREA</th>
<th>AES DESIGN LOAD (psf)</th>
<th>ASCE 7-05 DESIGN LOAD (psf)</th>
<th>DESIGN LOAD USED (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Public areas</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Office lobbies</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Office (first floor)</td>
<td>80</td>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td>Office corridors above first floor</td>
<td>80</td>
<td>80</td>
<td>100</td>
</tr>
<tr>
<td>Offices above first floor</td>
<td>60</td>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td>Partitions</td>
<td>20</td>
<td>≥15</td>
<td>20</td>
</tr>
<tr>
<td>Parking garage</td>
<td>40</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>Stairs</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Roof</td>
<td>20</td>
<td>20</td>
<td>20</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>AES DESIGN LOAD (psf)</th>
<th>ASCE 7-05 DESIGN LOAD (psf)</th>
<th>DESIGN LOAD USED (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3½” masonry façade</td>
<td>63</td>
<td>Section 3.1.1</td>
<td>63*</td>
</tr>
<tr>
<td>3½” n.w.c. slab on 3½”-20 GA composite steel deck</td>
<td>Unknown</td>
<td></td>
<td>10*</td>
</tr>
<tr>
<td>1½”-20 GA wide rib roof deck</td>
<td>Unknown</td>
<td></td>
<td>34</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>AREA</th>
<th>AES DESIGN LOAD (psf)</th>
<th>ASCE 7-05 DESIGN LOAD (psf)</th>
<th>DESIGN LOAD USED (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor</td>
<td>5</td>
<td>---</td>
<td>10</td>
</tr>
<tr>
<td>Roof</td>
<td>Unknown</td>
<td>---</td>
<td>10</td>
</tr>
</tbody>
</table>

Figure 4: Summary of gravity design loads

*The composite steel floor and roof deck manufacturers were not cited within the bid specifications for Schenley Place. As assumed in Technical Report 1, the manufacturer and deck types are as follows:

3½” n.w.c. slab on 3½”-20 G composite floor decking: Vulcraft, 3 VLI20

1½”-20 G wide rib roof decking: Vulcraft, 1.5B20

**Superimposed dead loads take into account the weight of finishes and/or MEP equipment.
Lateral design load summary: Wind

Wind loads were determined in accordance with Method 2 - Analytical Procedure, found in Chapter 6 of ASCE 7-05. The professionals at Atlantic Engineering Services (AES) used this code in their design of Schenley Place. Figure 5 is a summary of the wind design factors used in calculating the wind pressures. Figures 18 and 19 in Appendix B contain complete wind pressure data in both the north-south and west-east directions.

<table>
<thead>
<tr>
<th>VARIABLE</th>
<th>VALUE</th>
<th>ASCE 7-05 REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>V (mph)</td>
<td>90</td>
<td>Figure 6-1</td>
</tr>
<tr>
<td>K_d</td>
<td>0.85</td>
<td>Table 6-4</td>
</tr>
<tr>
<td>I</td>
<td>1.15</td>
<td>Table 6-1</td>
</tr>
<tr>
<td>EXPOSURE</td>
<td>B</td>
<td>Section 6.5.6</td>
</tr>
<tr>
<td>K_n</td>
<td>1.0</td>
<td>Section 6.5.7.1</td>
</tr>
<tr>
<td>ENCLOSURE</td>
<td>Fully enclosed</td>
<td>Section 6.5.9</td>
</tr>
<tr>
<td>n_1</td>
<td>1.44</td>
<td>(Rigid) Equation C6-19</td>
</tr>
<tr>
<td>G</td>
<td>0.85</td>
<td>Section 6.5.8.1</td>
</tr>
<tr>
<td>G_Cpn</td>
<td>Windward</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Leeward</td>
<td>-1.0</td>
</tr>
<tr>
<td>G_Cp</td>
<td>Enclosed building</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-0.18</td>
</tr>
<tr>
<td>C_p</td>
<td>Windard</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>Leeward</td>
<td>-0.47</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-0.50</td>
</tr>
</tbody>
</table>

Figure 5: Summary of wind design factors

The building was initially designed for a single tenant who intended to use the building as a healthcare facility. Therefore, AES evaluated the wind loading with an Occupancy Category of I. As a result, AES applied the corresponding Importance Factor (I) of 1.15. To be consistent, the same Importance Factor was maintained in the following wind analysis. This yielded higher velocity pressures and ultimately, higher design wind pressures. In the future, the owner may benefit from a reevaluation of the wind loads with an Occupancy Category of II.
The following tables summarize the force of total pressure, story shear, and moment at each level due to wind loads acting in both the north-south (Figure 6) and west-east (Figure 7) directions.

### STORIES FORCES, SHEARS, AND MOMENTS DUE TO WIND: NORTH-SOUTH DIRECTION

<table>
<thead>
<tr>
<th>FLOOR</th>
<th>HEIGHT ABOVE GROUND (ft)</th>
<th>FLOOR HEIGHT (ft)</th>
<th>FORCE OF TOTAL PRESSURE, F (kip)</th>
<th>STORY SHEAR, V (kip)</th>
<th>MOMENT, M (ft-k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROOF</td>
<td>94.99</td>
<td>0</td>
<td>19.48</td>
<td>19.48</td>
<td>1850.41</td>
</tr>
<tr>
<td>7</td>
<td>81.32</td>
<td>13.67</td>
<td>39.12</td>
<td>58.6</td>
<td>3181.24</td>
</tr>
<tr>
<td>6</td>
<td>67.99</td>
<td>13.33</td>
<td>36.46</td>
<td>95.06</td>
<td>2478.92</td>
</tr>
<tr>
<td>5</td>
<td>54.66</td>
<td>13.33</td>
<td>35.08</td>
<td>130.14</td>
<td>1917.47</td>
</tr>
<tr>
<td>4</td>
<td>41.33</td>
<td>13.33</td>
<td>33.64</td>
<td>163.78</td>
<td>1390.34</td>
</tr>
<tr>
<td>3</td>
<td>28</td>
<td>13.33</td>
<td>31.64</td>
<td>195.42</td>
<td>885.92</td>
</tr>
<tr>
<td>2</td>
<td>14.67</td>
<td>13.33</td>
<td>30.29</td>
<td>225.71</td>
<td>444.35</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>14.67</td>
<td>15.5</td>
<td>241.21</td>
<td>0.00</td>
</tr>
</tbody>
</table>

\[ \sum F = 241 \quad \sum M = 12149 \]

Figure 6: Summary of story forces, shears, and moments due to wind acting in the north-south direction

### STORIES FORCES, SHEARS, AND MOMENTS DUE TO WIND: WEST-EAST DIRECTION

<table>
<thead>
<tr>
<th>FLOOR</th>
<th>HEIGHT ABOVE GROUND (ft)</th>
<th>FLOOR HEIGHT (ft)</th>
<th>FORCE OF TOTAL PRESSURE, F (kip)</th>
<th>STORY SHEAR, V (kip)</th>
<th>MOMENT, M (ft-k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROOF</td>
<td>94.99</td>
<td>0</td>
<td>15.6</td>
<td>15.6</td>
<td>1481.84</td>
</tr>
<tr>
<td>7</td>
<td>81.32</td>
<td>13.67</td>
<td>30.47</td>
<td>46.07</td>
<td>2477.82</td>
</tr>
<tr>
<td>6</td>
<td>67.99</td>
<td>13.33</td>
<td>29.24</td>
<td>75.31</td>
<td>1988.03</td>
</tr>
<tr>
<td>5</td>
<td>54.66</td>
<td>13.33</td>
<td>28.16</td>
<td>103.47</td>
<td>1539.23</td>
</tr>
<tr>
<td>4</td>
<td>41.33</td>
<td>13.33</td>
<td>33.37</td>
<td>136.84</td>
<td>1379.18</td>
</tr>
<tr>
<td>3</td>
<td>28</td>
<td>13.33</td>
<td>37.59</td>
<td>174.43</td>
<td>1052.52</td>
</tr>
<tr>
<td>2</td>
<td>14.67</td>
<td>13.33</td>
<td>36.06</td>
<td>210.49</td>
<td>529.00</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>14.67</td>
<td>19.3</td>
<td>229.79</td>
<td>0.00</td>
</tr>
</tbody>
</table>

\[ \sum F = 230 \quad \sum M = 10448 \]

Figure 7: Summary of story forces, shears, and moments due to wind acting in the west-east direction

Forces of total pressure were calculated at each level by multiplying the level’s tributary façade areas by wind pressures (Appendix B) corresponding to the level’s height above ground. The base shear or total lateral force is a summation of the forces of total pressure at each floor. This calculation produced a wind design base shear of 230 kips in the west-east direction and a controlling wind design base shear of 241 kips in the north-south direction. In comparison, AES reported a 245 kip wind design base shear. Because both wind analyses were performed in accordance with ASCE 7-05, the minor discrepancy between the reported wind design base shears was expected.

### Lateral design load summary: Seismic

Seismic loads were determined in accordance with the Equivalent Lateral Force procedure of Chapter 11, ASCE 7-05. The professionals at Atlantic Engineering Services (AES) used this code in their design of Schenley Place. Figure 8 summarizes...
The building was initially designed for a single tenant who intended to use the building as a healthcare facility. Therefore, AES evaluated the seismic loading with an Occupancy Category of III. As a result, AES applied the corresponding Importance Factor (I) of 1.25. To be consistent, the same Importance Factor was maintained in the following seismic analysis. This yielded higher Seismic Response Coefficients ($C_s$) and ultimately, higher design base shears in a given direction. In the future, the owner may benefit from a reevaluation of the seismic loads with an Occupancy Category of II.

Seismic base shears ($V$) or the total design lateral forces in both the north-south and west-east directions were determined according to the following equation:

$$V = C_sW$$  \hspace{0.5cm} (Equation 12.8-1)

Figure 31 of Appendix C is a summary of the Effective Seismic Weight ($W$). Seismic Response Coefficients were calculated according to the following equation:
\[ C_s = \frac{S_{ts}}{T^2} \]  
\text{(Equation 12.8-3)}

Fundamental Periods of the building \((T_x \text{ and } T_y)\) produced by the ETABS model, were used to calculate the relative \(C_s\) rather than the Approximate Fundamental Period \((T_a)\) of the building. Figure 32 of Appendix C also summarizes the area masses applied to each level of the ETABS computer model. Area masses depend on the estimated \(W\) and relative area of each floor.

The following tables summarize the force of total pressure, story shear, and moment at each level due to seismic loads acting in both the north-south (Figure 9) and west-east (Figure 10) directions.

### STORY FORCES, SHEARS, AND MOMENTS DUE TO SEISMIC: NORTH-SOUTH DIRECTION

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>STORY WEIGHT (w_s) (kips)</th>
<th>HEIGHT (h_s) (ft)</th>
<th>(w_s h_s^k)</th>
<th>LATERAL FORCE (F_x) (kips)</th>
<th>(C_{vx})</th>
<th>STORY SHEAR (V_x) (kips)</th>
<th>MOMENT (M_x) (ft-k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROOF</td>
<td>419</td>
<td>95</td>
<td>103564</td>
<td>35.1</td>
<td>0.097</td>
<td>35.1</td>
<td>3337.9</td>
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<tr>
<td>7</td>
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<td>81.33</td>
<td>272421</td>
<td>92.4</td>
<td>0.256</td>
<td>127.6</td>
<td>7516.9</td>
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<tr>
<td>6</td>
<td>1336</td>
<td>68</td>
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<td>0.207</td>
<td>202.3</td>
<td>5082.1</td>
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<td>54.67</td>
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<td>57.4</td>
<td>0.159</td>
<td>259.7</td>
<td>3137.8</td>
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<tr>
<td>4</td>
<td>1524</td>
<td>41.33</td>
<td>137594</td>
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<td>0.129</td>
<td>306.4</td>
<td>1929.4</td>
</tr>
<tr>
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<td>1960</td>
<td>28</td>
<td>110472</td>
<td>37.5</td>
<td>0.104</td>
<td>343.9</td>
<td>1049.4</td>
</tr>
<tr>
<td>2</td>
<td>1960</td>
<td>14.67</td>
<td>50533</td>
<td>17.1</td>
<td>0.047</td>
<td>361.0</td>
<td>251.5</td>
</tr>
<tr>
<td>1</td>
<td>293</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.000</td>
<td>361.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

\[ \sum F = 361 \text{ kips} \quad \sum M = 22305 \text{ ft-k} \]

Figure 9: Summary of story forces, shears, and moments due to seismic acting in the north-south direction.

### STORY FORCES, SHEARS, AND MOMENTS DUE TO SEISMIC: WEST-EAST DIRECTION

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>STORY WEIGHT (w_s) (kips)</th>
<th>HEIGHT (h_s) (ft)</th>
<th>(w_s h_s^k)</th>
<th>LATERAL FORCE (F_x) (kips)</th>
<th>(C_{vx})</th>
<th>STORY SHEAR (V_x) (kips)</th>
<th>MOMENT (M_x) (ft-k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROOF</td>
<td>419</td>
<td>95</td>
<td>405999</td>
<td>24.9</td>
<td>0.113</td>
<td>24.9</td>
<td>2366.5</td>
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<td>7</td>
<td>1330</td>
<td>81.33</td>
<td>1019334</td>
<td>62.5</td>
<td>0.283</td>
<td>87.5</td>
<td>5086.6</td>
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<td>68</td>
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<td>0.217</td>
<td>135.4</td>
<td>3259.2</td>
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<tr>
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<td>1336</td>
<td>54.67</td>
<td>561897</td>
<td>34.5</td>
<td>0.156</td>
<td>169.9</td>
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<td>41.33</td>
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<td>195.6</td>
<td>1065.6</td>
</tr>
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<td>1960</td>
<td>28</td>
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</tr>
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<td>2</td>
<td>1960</td>
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<td>6.9</td>
<td>0.031</td>
<td>221.0</td>
<td>101.8</td>
</tr>
<tr>
<td>1</td>
<td>293</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.000</td>
<td>221.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

\[ \sum F = 221 \text{ kips} \quad \sum M = 14280 \text{ ft-k} \]

Figure 10: Summary of story forces, shears, and moments due to seismic acting in the west-east direction.

Equation 12.8-1 (defined above) produced a design base shear of 221 kips in the west-east direction and a controlling seismic design base shear of 361 kips in the north-south direction. In comparison, AES reported a 365 kip seismic design base shear, resulting in a difference of 4 kips. The minor discrepancy could be expected because both seismic analyses were performed in accordance with ASCE 7-05.
ETABS model

ETABS—a structural computer modeling and analysis program—was used in the analysis of the lateral force resisting system of Schenley Place. In addition to verifying the results of hand calculations, an ETABS computer model effectively determines the following: The relative stiffness of the concentrically and eccentrically braced frames, center of mass (COM) and center of rigidity (COR) of each level, the controlling ASCE 7-05 load combinations, story displacements, story drifts, and the effects of torsion. The ETABS model was simplified to represent only lateral members and diaphragms. Lateral members were modeled as sized within the structural documents. Diaphragms were assumed rigid and modeled as area elements, and gravity loads were applied as additional area masses to the diaphragms. User defined lateral loads are discussed in more detail in the following sections that implement the ETABS model.

Figure 11: Image of ETABS model created to analyze the lateral force resisting system of Schenley Place
Load path and distribution

Lateral force resisting systems, whether braced frames, moment frames, shear walls or any such combination, transfers the lateral loads (wind and/or seismic) to the building’s foundation where the loads dissipate. In the case of Schenley Place, the composite floor or roof decking acts as the diaphragm that transfers the lateral loads to the lateral force resisting system located at the building’s core. As previously discussed, the building’s lateral force resisting system consists of both concentrically and eccentrically braced frames. The HSS-braces transfer the lateral loads from the diaphragm to the wide-flange steel columns of the lateral system. The wide-flange steel columns then transfer the lateral loads to the cast-in-place concrete columns of the below grade parking garage. Sized at 1'-6"x2'-6" and reinforced with 10-#11 bars and #4 ties at 18 inches on center, the cast-in-place concrete columns transfer the lateral loads to the grade beams. Finally, the grade beams transfer the lateral loads to the supporting drilled caissons where the loads dissipate to the soil.

Figure 12 is an image of the sixth floor framing plan with the lateral braced frames labeled as referenced throughout this report. Additionally, Figure 12 illustrates the location of the braced frames.

Figure 12: Sixth floor framing plan with frames labeled as referenced throughout Technical Report 3 and location of concentrically braced frames (blue) and eccentrically braced frames (red)

4 See, “Lateral force resisting system summary” above.
The distribution of lateral loads is dependent on the relative stiffness of each braced frame. Elements with higher stiffness resist more lateral load than elements with lower stiffness. In determining the distribution of lateral loads, an arbitrary force (P) of 1000 kips was applied to the ETABS model in both the x-direction (to evaluate Frame 4 and Frame 5) and the y-direction (to evaluate Frame C and Frame D). Story displacements (Δp) were determined from the ETABS output, and were applied to calculate story stiffnesses (Ki). After determining each total story stiffness, the relative story stiffness (Ri) was calculated for each frame in order to determine the distribution of the lateral loads. Figure 13 and 14 summarize these findings.

### RELATIVE STORY STIFFNESS, Rix

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>WEST-EAST FRAMES (X-DIRECTION)</th>
<th>ARBITRARY UNIT LOAD, P (kips)</th>
<th>STORY STIFFNESS, Ki = P/Δp</th>
<th>TOTAL STORY STIFFNESS Ki,total</th>
<th>RELATIVE STORY STIFFNESS, Ri = Ki/Ki,total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FRAME 4</td>
<td>FRAME 5.1</td>
<td>FRAME 4</td>
<td>FRAME 5.1</td>
<td>FRAME 4</td>
</tr>
<tr>
<td>ROOF</td>
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</tr>
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<td>1000</td>
<td>1363.88</td>
<td>2186.27</td>
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</tbody>
</table>

**Figure 13: Relative story stiffness of eccentrically braced Frame 4 and Frame 5.1**

### RELATIVE STORY STIFFNESS, Riy

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>NORTH-SOUTH FRAMES (Y-DIRECTION)</th>
<th>ARBITRARY UNIT LOAD, P (kips)</th>
<th>STORY STIFFNESS, Ki = P/Δp</th>
<th>TOTAL STORY STIFFNESS Ki,total</th>
<th>RELATIVE STORY STIFFNESS, Ri = Ki/Ki,total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FRAME C</td>
<td>FRAME D</td>
<td>FRAME C</td>
<td>FRAME D</td>
<td>FRAME C</td>
</tr>
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<td>ROOF</td>
<td>3.4565</td>
<td>3.4478</td>
<td>1000</td>
<td>289.31</td>
<td>290.04</td>
</tr>
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</tr>
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<td>0.1588</td>
<td>0.1586</td>
<td>1000</td>
<td>6297.23</td>
<td>6305.17</td>
</tr>
</tbody>
</table>

**Figure 14: Relative story stiffness of concentrically braced Frame C and Frame D**

Upon calculating the relative story stiffness of each frame, the COR was determined for each level and compared to the output of the ETABS model. Because ETABS considers the stiffness of the diaphragms and frames when determining rigidity, where as the calculated COR values reflect only the stiffness of the frames, a notable difference exists between the ETABS output and the values calculated. Figures 33 and 34 of Appendix D summarize the calculation performed in determining the location of each level’s COR. Throughout this report, the calculated COR values were used in calculations and discussions where applicable. Figure 35 of Appendix D is a summary of the x and y-coordinates that correspond to the COR of each level. In addition, Figure 36 of Appendix D summarizes the COM of each level. Figure 15, found below, is a graphical representation of the location of both the COR and COM.
Figure 15: Graphical representation of the location of the COR and COM at each level

As Figure 13 demonstrates, Frame 5.1 resists an average of 62% of the lateral loads acting in the west-east direction until the fifth level. At the fifth level and above, Frame 5.1 only resists 45% of the lateral loads acting in the west-east direction, whereas Frame 4 resists 55% of the lateral loads. Figure 15 clearly shows that the location of the COR changes between level 4 and level 5. The COR is in closer proximity to Frame 5.1 on level 2 through 4. However, at level 5 the COR is located nearer to Frame 4. This is consistent with the shift in lateral distribution found in Figure 13. Lateral loads are equally distributed to Frame C and Frame D in the north-south direction as shown by Figure 14. As illustrated in Figure 15, Frame C and D are
located symmetrical about the COR at all levels, which verifies the equal distribution of the lateral loads to these frames.

**Load combinations**

In accordance with ASCE 7-05, Section 2.3.2, the following basic load combinations were applied to the ETABS model to determine the controlling load case:

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r or S or R)$
3. $1.2D + 1.6(L_r or S or R) + (L or 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r or S or R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

The wind load cases defined in Figure 6-9 of ASCE 7-05 (located in Appendix E of this report) were evaluated to account for torsion in these load combinations. This report assumes that the ETABS analysis of seismic load cases in the above load combinations accounted for inherent and accidental torsion. These effects are examined in greater detail later in this report.

After evaluating the base shears and overturning moments computed by ETABS for each of the above load combinations, and accounting for torsion, it was concluded that $1.2D + 1.0E + L + 0.2S$ controls in the north-south direction, where as $1.2D + 1.6W + L + 0.5(L_r or S or R)$ controls in the west-east direction. This analysis fails to consider second-order effects; an extensive evaluation of which will be performed in the future.

**Shear**

**Direct shear**

Upon determining both the relative story stiffness of the lateral force resisting frames as well as the governing load combination in the north-south and west-east directions, direct forces ($F_{direct}$) were computed. The force resisted by each frame is a function of its displacement and stiffness in a single axis. In calculating the direct shear in each frame, its relative stiffness at a given level is multiplied by the lateral force acting at that respective level. Figures 16 and 17 summarize the forces distributed to each lateral force resisting frame, dependent on the frame’s axis of stiffness and the controlling load case.
WEST-EAST DIRECT SHEAR DUE TO WIND
CONTROLLING LOAD CASE: 1.2D+1.6W+L+0.5(L, or S or R)

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>FORCE (k)</th>
<th>FACTORED FORCE (k)</th>
<th>RELATIVE STORY STIFFNESS</th>
<th>DISTRIBUTED FORCE (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Distributed Force</td>
<td>Frame 4</td>
</tr>
<tr>
<td>ROOF</td>
<td>16</td>
<td>25</td>
<td>0.5536</td>
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</tr>
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</tr>
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<td>58</td>
<td>0.3842</td>
<td>0.6158</td>
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</table>

Figure 16: Summary of direct forces in west-east direction due to wind

NORTH-SOUTH DIRECT SHEAR DUE TO SEISMIC
CONTROLLING LOAD CASE: 1.2D+1.0E+L+0.2S

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>FORCE (k)</th>
<th>FACTORED FORCE (k)</th>
<th>RELATIVE STORY STIFFNESS</th>
<th>DISTRIBUTED FORCE (k)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Distributed Force</td>
<td>Frame C</td>
</tr>
<tr>
<td>ROOF</td>
<td>35</td>
<td>35</td>
<td>0.4994</td>
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<td>92</td>
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<td>75</td>
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<td>0.5004</td>
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<td>57</td>
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<td>0.5003</td>
</tr>
<tr>
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<td>47</td>
<td>0.4997</td>
<td>0.5003</td>
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<tr>
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<tr>
<td>2</td>
<td>17</td>
<td>17</td>
<td>0.4997</td>
<td>0.5003</td>
</tr>
</tbody>
</table>

Figure 17: Summary of direct forces in north-south direction due to seismic

**Torsional shear**

In addition to direct shear, this report discusses the effects of torsional shear forces ($F_{\text{torsional}}$), produced by load eccentricities. Lateral loads are typically offset from the center of rigidity, resulting in a torsional moment. This moment produces an additional torsional shear force. Depending on the location of each lateral force resisting frame with respect to the center of rigidity, the following equation dictates the total shear ($F_{\text{total}}$) that is resisted by each frame:

$$F_{\text{total}} = F_{\text{direct}} \pm F_{\text{torsional}}$$

Figure 18 is a comparison of the total shear ETABS output to the direct shear, calculated above, at level 5. From this figure, it is obvious that shear induced by torsion significantly affects the total forces resisted by the lateral system. Although a detailed analysis of the torsional effects is beyond the scope of this Technical 3 Report, the impact of torsion has been realized. In order to fully understand the extent to which torsion affects the behavior of the force resisting system of Schenley Place, a far more extensive analysis is required.
Torsion

Torsion is present when lateral loads act on a structure with eccentricities. However, eccentricity is defined differently for both wind and seismic load cases. The following is a discussion of the torsional moments produced due to wind and seismic loads.

Because the diaphragm in the ETABS model was defined as “rigid,” both inherent and accidental torsion are considered in accordance to ASCE 7-05, Section 12.8.1 and 12.8.2. An inherent torsional moment ($M_t$) results from an eccentricity between the locations of the center of mass (COM) and center of rigidity (COR). Dependent solely on the displacement of the COM, accidental torsion ($M_{ta}$) accounts for an assumed displacement of the COM, a distance of 5% of the dimension of the structure perpendicular to the direction of the applied lateral seismic forces. Because Schenley Place is assigned to Seismic Design Category B, the analysis does not consider amplification of accidental torsional moments, in accordance to ASCE 7-05, Section 12.8.4.3. Figure 17 is a summary of the moments produced due to inherent and accidental torsion evaluated in both the north-south and west-east direction. Figure 39 of Appendix E are the calculations performed in determining inherent and accidental torsion. This report assumes that these torsional moments were accounted for in the ETABS analysis of the controlling load combinations.

SUMMARY OF SEISMIC TORSIONAL EFFECTS

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>FACTORED STORY FORCE (kips)</th>
<th>WEST-EAST (X-DIRECTION)</th>
<th>NORTH-SOUTH (Y-DIRECTION)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_t$ (ft-kips) $M_{ta}$ (ft-kips) $M_{total}$ (ft-kips)</td>
<td>$M_t$ (ft-kips) $M_{ta}$ (ft-kips) $M_{total}$ (ft-kips)</td>
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</tr>
</tbody>
</table>

Figure 19: Summary of seismic torsional effects

Where wind loads have been determined according to provisions of Section 6.5.12.2.1 of ASCE 7-05, it is necessary to analyze the wind load cases as defined by Figure 6-9 of ASCE 7-05 (Appendix E, Figure 37). Eccentricities, with respect to wind loads, are 15% of the dimension of the structure perpendicular to the direction of the applied lateral wind forces. Not every wind case defined by Figure 6-9 accounts for torsional affects. Figure 38 of Appendix E is a
summary of the wind cases evaluated and the resulting torsional and lateral force values resulting from each wind case analyzed.

Acceptable code and industry story drift standards

Drift is a serviceability issue often considered to ensure the comfort of its occupants. Drift limitations are dependent on the lateral load case considered. For instance, allowable drift due to wind is typically limited to 1/400th of the level's height above ground, an adopted industry standard. In comparison, allowable story drift due to seismic loads is summarized in Table 12.12-1 of the ASCE 7-05, and is dictated by the building's Occupancy Category.

Wind drifts computed by ETABS were evaluated against the allowable industry standard of h/400. A comparable summary of controlling wind drifts in the x and y-directions can be found in Figures 20 and 21. As seen in Figure 20, drift at the roof level exceeds the industry standard. Because wind drift is only a serviceability consideration, the designer is entitled to use judgment in allowing drift to exceed industry standard, so long as the lateral force resisting system meets necessary design strength requirements.

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>HEIGHT ABOVE GROUND, h (in)</th>
<th>ALLOWABLE DRIFT $\Delta_{\text{allowable}} = h/400$</th>
<th>TOTAL DRIFT (ETABS)</th>
<th>ACCEPTABLE?</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROOF</td>
<td>1140</td>
<td>2.85</td>
<td>2.87</td>
<td>NO</td>
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<tr>
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<td>6</td>
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<td>5</td>
<td>656.04</td>
<td>1.6401</td>
<td>0.71</td>
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</tr>
<tr>
<td>4</td>
<td>495.96</td>
<td>1.2399</td>
<td>0.22</td>
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</tr>
<tr>
<td>3</td>
<td>336</td>
<td>0.84</td>
<td>0.00</td>
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<td>2</td>
<td>176.04</td>
<td>0.4401</td>
<td>-0.77</td>
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Figure 20: Computed versus industry standard wind drift in the west-east direction

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>HEIGHT ABOVE GROUND, h (in)</th>
<th>ALLOWABLE DRIFT $\Delta_{\text{allowable}} = h/400$</th>
<th>TOTAL DRIFT (ETABS)</th>
<th>ACCEPTABLE?</th>
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</thead>
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<td>2</td>
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<td>0.4401</td>
<td>0.00</td>
<td>YES</td>
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</table>

Figure 21: Computed versus industry standard wind drift in the north-south direction

Seismic drifts computed by ETABS were evaluated against the ASCE 7-05 allowable story drifts found in Table 12.12-1. Because Schenley Place was designed with an Occupancy Category III, a drift of $0.015h_{sx}$ is allowed, where $h_{sx}$ is the story height below the level under consideration. Figures 22 and 23 summarize the results of the ETABS analysis and compare
the output to allowable story drifts. As seen below, the computed seismic story drifts do not exceed allowable drifts.

![CONTROLLING SEISMIC DRIFT VALUES (X-DIRECTION)](#)

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>HEIGHT OF STORY, $h_{sx}$ (in)</th>
<th>STORY DRIFT (ETABS)</th>
<th>ALLOWABLE STORY DRIFT $\Delta_{allowable} = 0.015h_{sx}$</th>
<th>ACCEPTABLE?</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROOF</td>
<td>13.67</td>
<td>0.092</td>
<td>0.205</td>
<td>YES</td>
</tr>
<tr>
<td>7</td>
<td>13.33</td>
<td>0.174</td>
<td>0.200</td>
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<td>0.254</td>
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<td>14.67</td>
<td>0.125</td>
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Figure 22: Computed versus allowable seismic drift in the west-east direction

![CONTROLLING SEISMIC DRIFT VALUES (Y-DIRECTION)](#)

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<tr>
<th>LEVEL</th>
<th>HEIGHT OF STORY, $h_{sy}$ (in)</th>
<th>STORY DRIFT (ETABS)</th>
<th>ALLOWABLE STORY DRIFT $\Delta_{allowable} = 0.015h_{sy}$</th>
<th>ACCEPTABLE?</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROOF</td>
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<td>0.085</td>
<td>0.205</td>
<td>YES</td>
</tr>
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<td>13.33</td>
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<tr>
<td>6</td>
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<td>0.200</td>
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<tr>
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<td>13.33</td>
<td>0.092</td>
<td>0.200</td>
<td>YES</td>
</tr>
<tr>
<td>2</td>
<td>14.67</td>
<td>0.068</td>
<td>0.220</td>
<td>YES</td>
</tr>
</tbody>
</table>

Figure 23: Computed versus allowable seismic drift in the north-south direction

**Overturning**

Overturning moments result from existing lateral forces. These moments are a concern due to the impact that they could potentially have on the foundation system. A preliminary analysis investigating overturning was performed on the lateral force resisting system located at the building’s core. Theoretically, if the axial forces resulting from the overturning moment in the end columns of the braced frames are less than the axial forces produced in the column due to dead loads, overturning should not be an issue. Figure 24 is a summary of the base shears and overturning moments of each lateral frame. Additionally, both the axial forces due to the overturning moment and the forces due to dead loads are summarized in Figure 24. Dead loads were estimated based upon previous calculations performed for the effective weight of the building.
According to this preliminary analysis, overturning may be an issue. Nevertheless, a far more extensive evaluation of the overturning moments and the effect that they have on the foundation system must be conducted before a final conclusion is reached.

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>EAST-WEST FRAMES (X-DIRECTION)</th>
<th>NORTH-SOUTH FRAMES (Y-DIRECTION)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FRAME 4</td>
<td>FRAME 5.1</td>
</tr>
<tr>
<td>ROOF</td>
<td>25</td>
<td>25</td>
</tr>
<tr>
<td>7</td>
<td>49</td>
<td>49</td>
</tr>
<tr>
<td>6</td>
<td>47</td>
<td>47</td>
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<td>58</td>
<td>58</td>
</tr>
<tr>
<td>BASE</td>
<td>337</td>
<td>337</td>
</tr>
</tbody>
</table>

| FORCE @ EDGE COLUMN (k) | 495 | 495 | 495 | 495 |

| base dimension (ft)     | 30  | 30  | 27  | 27  |
| base dimension (ft)     | 30  | 30  | 27  | 27  |

| OVERTURNING MOMENT (ft-k) | 16716 | 16716 | 22305 | 22305 |

Figure 24: Summary of base shears and design overturning moments
Member spot checks

Several spot-checks were performed on lateral force resisting members of Frame 5.1. Figure 25 illustrates those members that were checked for strength. ETABS output was utilized in determining the forces distributed to the members evaluated. Because gravity loads were not accounted for in ETABS, a summation of these loads was performed for the checked column.

Figure 25: Location of lateral force resisting members spot-checked
**Braced frame**

HSS10X10X5/8, $F_y=46$ksi

WEST-EAST DIRECTION: WIND CONTROLS

FACTORED AXIAL FORCE, $P_u$ DUE TO WIND (kips): 128.55

$L_b$ (ft) = 14.62

**TABLE 4-1 (AISC 360-05)**

| $\phi P_n$ (kips) | 749 |

$P_u/\phi P_n = 0.171629 < 1.0 \therefore \text{OK}$

This braced member is sufficiently adequate in terms of strength requirements. It is likely that the member was sized based on drift or displacement requirements, rather than strength.

**Column**

W14X193 @ LEVEL 7

TRIBUTARY AREA (psf): 850.5

ANALYZED IN THE WEST-EAST DIRECTION: WIND LOAD CONTROLS

<table>
<thead>
<tr>
<th>FACTORED WIND LOADS FROM ETABS</th>
<th>P (k)</th>
<th>M (ft-k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>18.2</td>
<td></td>
<td>20.5</td>
</tr>
</tbody>
</table>

| UNFACTORED DEAD LOAD | 113 | 0 |

| UNFACTORED LIVE LOAD | 102 | 0 |

**LOAD COMBINATIONS**

<table>
<thead>
<tr>
<th></th>
<th>P (k)</th>
<th>M (ft-k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4D</td>
<td>158.2</td>
<td>0</td>
</tr>
<tr>
<td>1.2D+1.6L</td>
<td>298.8</td>
<td>0</td>
</tr>
<tr>
<td>1.2D+1.6W+L</td>
<td>255.8</td>
<td>20.5</td>
</tr>
<tr>
<td>0.9D+1.6W</td>
<td>119.9</td>
<td>20.5</td>
</tr>
</tbody>
</table>

$L_b$ (ft): 13.33 or conservatively use 14

$px10^3$ (kips$^{-1}$): 0.444

$bx10^3$ (kips$^{-1}$): 0.668

$(219.8k)(.44E-3)+(20.5ft-k)(.668E-3)=0.127269 < 1.0 \therefore \text{OK}$
This column is adequately designed for strength. There are several explanations for the large remaining available capacity. Though second-order effects were not accounted for, it is not likely they would control the design. This particular column is affected by both the controlling wind loads in the west-east direction, as well as the seismic loads in the north-south direction. Because this member was only checked in the west-east direction, it is possible that the north-south loadings control the design. Furthermore, drift or displacement may have ultimately controlled the design of this member. Additionally, the location of column splices could account for the excessive remaining capacity.
Conclusions

After modeling the lateral force resisting system of Schenley Place in ETABS and completing a thorough analysis of the system, the following conclusions were reached:

- Upon evaluating the basic load combinations as defined in ASCE 7-05, it was determined that load case 1.2D+1.6W+L+0.5(L, or S or R) controls in the west-east direction where as 0.9D+1.0E+1.6H controls in the north-south direction.
- Before evaluating the load combinations, it was necessary to revise both the wind and seismic load analyses performed in Technical Report 1. As a result of these changes, it was found that wind base shear controls in the west-east direction, while seismic base shear controls in the north-south direction. In comparison, Technical Report 1 reported that seismic base shear controlled in both directions. This discrepancy is attributable to two significant revisions: (1) An error in the Fundamental Period initially used to calculate seismic base shears was corrected in this report; and (2) linear wind loads acting on the building’s parapets, accounted for in Technical Report 1, were omitted here.
- After evaluating both wind and seismic drifts, the ETABS output was compared to industry and code standards that dictate allowable drifts. It was found that wind drift at the roof level in the west-east direction slightly exceeded the industry standard. Because wind is only a serviceability consideration, this result was not alarming. All seismic drifts met the allowable drift limits defined by ASCE 7-05.
- Spot checks of critical members confirmed that these members met required strength design standards. The large remaining strength capacities of checked members suggested that they were probably designed for drift or displacement, rather strength.
- The results of a preliminary overturning check suggest that overturning may be a design issue. However, before a final conclusion may be reached, the prevailing theory regarding over turning moments requires further examination and a more extensive analysis than was performed here is desirable.
- An inspection of the animated ETABS model determined that torsion has an obvious material effect on the lateral system of Schenley Place. After calculating direct shears in the lateral force resisting members and comparing these values to the total shear reported by ETABS, this report concludes that torsional shear significantly impacts the distribution of lateral forces.
Appendices

The appendices are a compilation of ETABS output and hand calculations. The information found within the appendices was discussed and/or referenced within the applicable sections of this report.
Appendix A: Lateral bracing elevations and details

Figure 26: Elevation of Frame 4 (left) and Frame D (right)
Figure 27: Elevation of Frame 5.1 (left) and Frame C (right)
Appendix B: Wind load data

VELOCITY PRESSURE EXPOSURE
AND PRESSURE COEFFICIENTS

<table>
<thead>
<tr>
<th>HEIGHT ABOVE GROUND LEVEL z (ft)</th>
<th>K_z</th>
<th>q_z</th>
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<tbody>
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<td>15</td>
<td>0.57</td>
<td>11.55</td>
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<td>20</td>
<td>0.62</td>
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<tr>
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Figure 28: Summary of velocity pressure exposure and pressure coefficients

NORTH-SOUTH DESIGN WIND PRESSURES

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<th>LOCATION</th>
<th>HEIGHT ABOVE GROUND LEVEL, z (ft)</th>
<th>q (psf)</th>
<th>EXTERNAL PRESSURE</th>
<th>INTERNAL PRESSURE</th>
<th>NET PRESSURE, p (psf)</th>
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<td>19.66</td>
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Figure 29: Summary of north-south design wind pressures
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<thead>
<tr>
<th>LOCATION</th>
<th>HEIGHT ABOVE GROUND LEVEL, z (ft)</th>
<th>q (psf)</th>
<th>qGCp (psf)</th>
<th>q(GCpi) (psf)</th>
<th>NET PRESSURE, p (psf)</th>
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<tbody>
<tr>
<td>WINDWARD</td>
<td>95</td>
<td>19.66</td>
<td>13.37</td>
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<td>-11.36</td>
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</table>

Figure 30: Summary of west-east design wind pressures
### Appendix C: Seismic load data

#### EFFECTIVE SEISMIC WEIGHT, W

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<th>LEVEL</th>
<th>ELEMENT</th>
<th>WEIGHT (kips)</th>
</tr>
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<td>ROOF</td>
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<td>Composite decking</td>
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<tr>
<td></td>
<td>Brick façade</td>
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<td>Superimposed dead load</td>
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<td>Composite decking</td>
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#### SUMMARY OF EFFECTIVE SEISMIC WEIGHT AND AREA MASSES

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∑ WEIGHT= 10156

Figure 32: Summary of the effective seismic weight per level and area masses applied to the ETABS model

Figure 31: Breakdown of the effective seismic weight by element, per level
### Appendix D: COR and COM calculations and summary

The following equations were used to calculate the x (XCR) and y-coordinates (YCR) that correspond to the center of rigidity of each level, summarized in Figure 33 and 34:

\[
XCR = \frac{\sum k_{iy}x_i}{\sum K_{iy,total}}
\]

\[
YCR = \frac{\sum k_{ix}y_i}{\sum K_{ix,total}}
\]

**CALCULATED CENTER OF RIGIDITY, XCR**

<table>
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<tr>
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<th>FRAME D</th>
<th>FRAME C</th>
<th>FRAME D</th>
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**CALCULATED CENTER OF RIGIDITY, YCR**

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<th>FRAME 4</th>
<th>FRAME 5.1</th>
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**CENTER OF RIGIDITY**

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<th>DIFFERENCE IN VALUES</th>
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<td>YCR (in)</td>
<td>XCR (in)</td>
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<td>790.108</td>
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**Figure 33:** Calculated center of rigidities, x-coordinates

**Figure 34:** Calculated center of rigidities, y-coordinates

**Figure 35:** Summary of center of rigidities for each level
<table>
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<th>DIFFERENCE IN VALUES</th>
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<td>790</td>
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<td>790</td>
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</table>

Figure 36: Summary of center of masses for each level
Appendix E: Load Combinations

The following image was borrowed from Chapter 6 of ASCE 7-05.

Figure 37: Figure 6-9 from Chapter 6 of ASCE 7-05
The following tables summarize the wind data analyzed in ETABS when considering ASCE 7-05 load combinations. Data was calculated based on the wind load cases defined in Figure 6-9 of ASCE 7-05 (pictured above). Calculated design wind loads were multiplied by the worst case wind load factor of 1.6, resulting in the factored loads shown below.

**CASE 1: DESIGN WIND LOAD CASE 1**

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>FACTORED P_X (kip)</th>
<th>P_Y (kip)</th>
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**CASE 2: DESIGN WIND LOAD CASE 2**

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<th>Bx (ft)</th>
<th>e_x (ft)</th>
<th>M_T (ft-kips)</th>
</tr>
</thead>
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<tr>
<td>ROOF</td>
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<td>19</td>
<td>102</td>
<td>15.3</td>
<td>29214</td>
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<tr>
<td>7</td>
<td>49</td>
<td>37</td>
<td>102</td>
<td>15.3</td>
<td>57062</td>
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**CASE 3: DESIGN WIND LOAD CASE 3**

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<th>0.75P_Y (kip)</th>
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<td>31</td>
<td>19</td>
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**CASE 4: DESIGN WIND LOAD CASE 4**

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<th>0.56P_X (kip)</th>
<th>0.56P_Y (kip)</th>
<th>Bx (ft)</th>
<th>e_x (ft)</th>
<th>B_y (ft)</th>
<th>e_y (ft)</th>
<th>M_T (ft-kips)</th>
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Figure 38: Summary of wind load data analyzed in ETABS when considering ASCE 7-05 load combinations
The following tables are a summary of the seismic data considering ASCE 7-05 wind load combinations. Data was calculated based on inherent and accidental torsion, as defined in Section 12.8.4.1 and 12.8.4.2 of ASCE 7-05. Calculated design seismic loads were multiplied by the worst case seismic load factor of 1.0, resulting in the factored loads shown below.

**CASEE: INHERENT TORSION, M_t (X-DIRECTION)**

<table>
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<tr>
<th>LEVEL</th>
<th>FACTORED STORY FORCE</th>
<th>COM (in)</th>
<th>COR (in)</th>
<th>DIFFERENCE BETWEEN COM AND COR (in)</th>
<th>MOMENT, M_t (ft-kips)</th>
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<td>-137</td>
<td>-548</td>
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<td>7</td>
<td>910.5</td>
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**CASEE: ACCIDENTAL TORSION, M_{ta} (X-DIRECTION)**

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<th>STORY FORCE (kips)</th>
<th>MOMENT, M_{ta} (ft-kips)</th>
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<td>26</td>
<td>3395</td>
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<td>6.5835</td>
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<td>2425</td>
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<td>6.5835</td>
<td>7</td>
<td>914</td>
</tr>
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</table>

**CASEEY: INHERENT TORSION, M_t (Y-DIRECTION)**

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>FACTORED STORY FORCE</th>
<th>COM (in)</th>
<th>COR (in)</th>
<th>DIFFERENCE BETWEEN COM AND COR (in)</th>
<th>MOMENT, M_t (ft-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROOF</td>
<td>35</td>
<td>790</td>
<td>790.227</td>
<td>-0.23</td>
<td>-1</td>
</tr>
<tr>
<td>7</td>
<td>92</td>
<td>790</td>
<td>790.151</td>
<td>-0.15</td>
<td>-1</td>
</tr>
<tr>
<td>6</td>
<td>75</td>
<td>790</td>
<td>790.140</td>
<td>-0.14</td>
<td>-1</td>
</tr>
<tr>
<td>5</td>
<td>57</td>
<td>790</td>
<td>790.119</td>
<td>-0.12</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>47</td>
<td>790</td>
<td>790.125</td>
<td>-0.12</td>
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</tr>
<tr>
<td>3</td>
<td>37</td>
<td>790</td>
<td>790.121</td>
<td>-0.12</td>
<td>0</td>
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<tr>
<td>2</td>
<td>17</td>
<td>790</td>
<td>790.113</td>
<td>-0.11</td>
<td>0</td>
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</tbody>
</table>

**CASEEY: ACCIDENTAL TORSION, M_{ta} (Y-DIRECTION)**

<table>
<thead>
<tr>
<th>LEVEL</th>
<th>STRUCTURAL WIDTH (ft)</th>
<th>5% OF WIDTH (ft)</th>
<th>STORY FORCE (kips)</th>
<th>MOMENT, M_{ta} (ft-kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROOF</td>
<td>102</td>
<td>5.1</td>
<td>35</td>
<td>3584</td>
</tr>
<tr>
<td>7</td>
<td>102</td>
<td>5.1</td>
<td>92</td>
<td>9427</td>
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<tr>
<td>6</td>
<td>102</td>
<td>5.1</td>
<td>75</td>
<td>7623</td>
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<tr>
<td>5</td>
<td>102</td>
<td>5.1</td>
<td>57</td>
<td>5854</td>
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<tr>
<td>4</td>
<td>152</td>
<td>7.5875</td>
<td>47</td>
<td>7084</td>
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<td>3</td>
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<td>7.5875</td>
<td>37</td>
<td>5688</td>
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<td>7.5875</td>
<td>17</td>
<td>2602</td>
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Figure 39: Summary of seismic load data analyzed in ETABS when considering ASCE 7-05 load combinations.
### Table: Summary of base shears and overturning moments produced by ETABS in the analysis of ASCE 7-05 load combinations

<table>
<thead>
<tr>
<th>Story</th>
<th>Point</th>
<th>Load</th>
<th>FX</th>
<th>FY</th>
<th>FZ</th>
<th>MX</th>
<th>MY</th>
<th>MZ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Summation 0, 0, Base</td>
<td>CASE1X</td>
<td>337</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>-200852</td>
<td>257287.5</td>
<td></td>
</tr>
<tr>
<td>Summation 0, 0, Base</td>
<td>CASE1Y</td>
<td>0</td>
<td>-361</td>
<td>0</td>
<td>233260</td>
<td>0</td>
<td>-285190</td>
<td></td>
</tr>
<tr>
<td>Summation 0, 0, Base</td>
<td>CASE2X</td>
<td>-253</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>-151164</td>
<td>-444329</td>
<td></td>
</tr>
<tr>
<td>Summation 0, 0, Base</td>
<td>CASE2Y</td>
<td>0</td>
<td>-270</td>
<td>0</td>
<td>174492</td>
<td>0</td>
<td>-917664</td>
<td></td>
</tr>
<tr>
<td>Summation 0, 0, Base</td>
<td>CASE3</td>
<td>-253</td>
<td>-270</td>
<td>0</td>
<td>174492</td>
<td>-151164</td>
<td>-20256</td>
<td></td>
</tr>
<tr>
<td>Summation 0, 0, Base</td>
<td>CASE4</td>
<td>-188</td>
<td>-204</td>
<td>0</td>
<td>131976</td>
<td>-111864</td>
<td>-1024646</td>
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</tr>
<tr>
<td>Summation 0, 0, Base</td>
<td>CASEEX</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>-171636</td>
<td>122387.5</td>
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</tr>
<tr>
<td>Summation 0, 0, Base</td>
<td>CASEEY</td>
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<td>-360</td>
<td>0</td>
<td>267020</td>
<td>0</td>
<td>-326256</td>
<td></td>
</tr>
</tbody>
</table>

Figure 40: Summary of base shears and overturning moments produced by ETABS in the analysis of ASCE 7-05 load combinations (controlling base shears and overturning moments have been highlighted)