Hiro McNulty – Structural Option
Faculty Advisor – Walt Schneider
Hyatt Regency – Hotel and Conference Center
Pittsburgh International Airport, PA
October 5, 2005
Technical Assignment 1

## EXECUTIVE SUMMARY

The Hyatt Regency – Hotel and Conference Center at the Pittsburgh International Airport, PA, is a 275,000 square foot multi-use building located directly adjacent to the airport's landside terminal. The building consists of an 11-story tower and 1-story conference center with an additional partial level below grade.

The structural systems of the low-rise conference center implements steel framing on spread footings and grade beams. The high-rise tower implements a cast-in-place moment resisting frame system with one-way slabs on piles/pile-caps as its primary structural system.

The original design used the BOCA 1996 building code for its design loads. As newer codes are now being used, the design loads that were examined have been updated to conform to the IBC 2003. Wind loadings have been determined from ASCE 7-02 and make fewer approximations than the original design. The original design did not incorporate seismic loading, so load values have been determined using the ASCE 7-02 Equivalent Lateral Force System.

Spot checks were performed on an interior column for gravity loading at the base of the high-rise tower and on a long-span joists that spans over the large ballroom in the conference center. The checks both used average tributary areas for members. The checks showed that the members had the appropriate strength to support the loadings as analyzed. However, some load assumptions may have been different than those of the original engineers.

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## OVERALL STRUCTURAL SYSTEM

The Hyatt Regency at the Pittsburgh International Airport is a 275,000 square foot hotel and conference center. The building consists of a 11-story tower with a 1-story attached conference center; there is an additional partial level below grade. The building has a combination of structural steel and cast-in-place concrete framing. The conference center is primarily structural steel framing while the hotel tower is cast-in-place concrete.

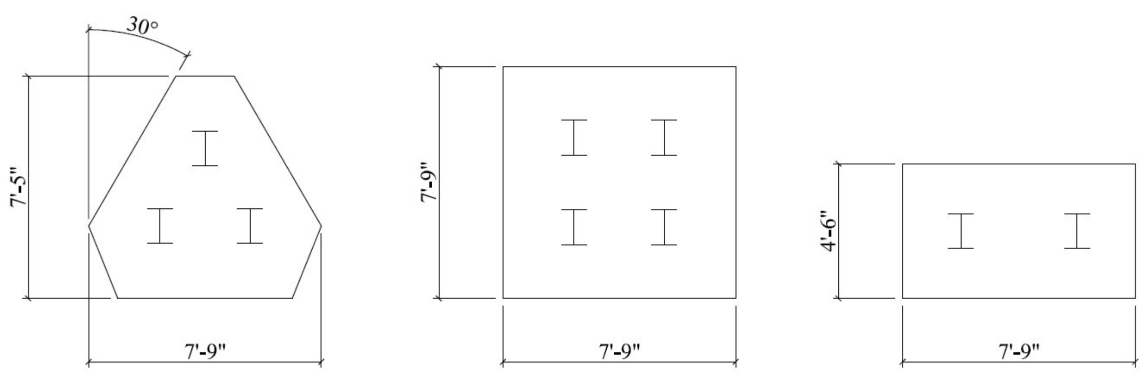
**Foundation (Spread Footings, Pile Caps, and Grade Beams)**

*Spread Footings*

- Spread footings are used under the conference center.
- Bottom of footings is located -3'-6" below grade.
- 15 different sizes of spread footings
  - 11 different square footings range from:
    5'-0"x5'-0" to 14'-0"x14'-0"
    Footings are from 12" to 27" deep with rebar sizes from #4 to #8.
  - 4 different rectangular footings ranging from:
    10'-0"x 14'-0" to 12'-0"x 26'-0"
    Footings are from 23" to 28" deep with rebar sized from #7 to #9.

*Pile Caps*

- Piles and pile caps are used under the main hotel tower.
- 3 sizes of pile caps are used, as follows:
  - Exterior pile caps are roughly triangular, see Pile Cap 1. They are 48" deep and have #11 rebar in 3 directions.
  - Interior pile caps are square, see Pile Cap 2. They are 43" deep and have #8 rebar in both directions.
  - Pile caps at stair wells are rectangular, see Pile Cap 3. They are 40" deep and have #11 rebar in the long direction and #4 rebar in the short.




PILE CAP 1 PILE CAP 2 PILE CAP 3

*Grade Beams*

- Grade beams are used between spread footings around the exterior of the structure.
- Top of grade beams is at an elevation of -0'-6" which allows the 6" slab-on-grade to extend over the grade beams.
- 10 different sizes of grade beams are used, ranging in size from 18"x36" to 26"x40".
- Grade beams have reinforcement from #6 to #9 rebar with #4 stirrups.

**Conference Center**

The conference center portion of the building is framed with structural steel; approximately 17,000 square feet on the ground floor (partially below grade) and approximately 50,000 square feet on the first floor. Steel connections are made with standard A325 or A490, ¾" bolts and welds are specified on the structural drawings as being no smaller than ¼". At moment connections, the connections are designed for the full moment capacity of the beams.

- <u>Ground Level:</u> The ground level framing supports a composite steel deck and concrete floor slab. A continuous 14" concrete foundation wall contains embedded plates to attach to the steel framing. The wall also acts as a retaining wall for soils around the section that is below grade. The steel framing for the first floor is typically W12X19 for 12'-16' spans, W14X22 for 25' spans, and W21X44 for 35' spans. Column sizes range from W10X33 to W10X49.

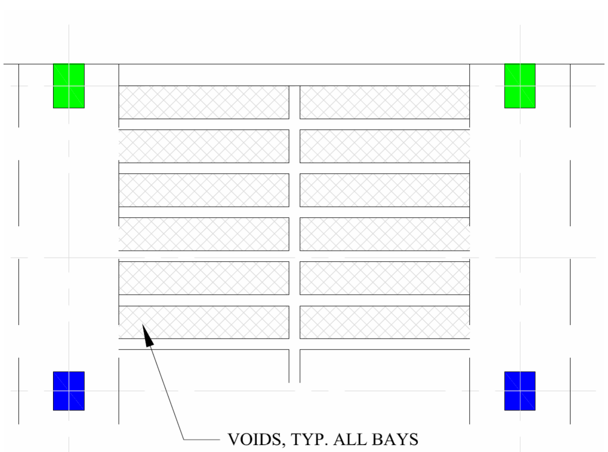
- <u>First Floor:</u> The first floor is the top level for the conference center. The framing for the roof consists of both W-sections and joists. There is a large size difference in all steel beams, ranging from W12X14 to W21X50 and girders ranging from W24X55 to W27X94. Long-span joists are used over the large span of the main conference center located in the middle of the conference center. 68DLH17(s) frame over the 73' span, with diagonal bracing between joists for stability. Framing supports 3"-18 gage steel roof deck.

**Tower (Guest Rooms)**

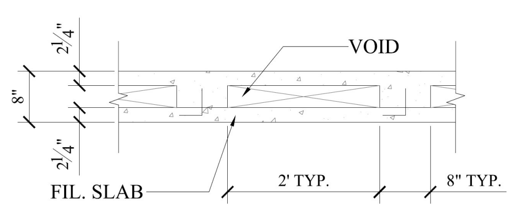
The tower is framed as cast-in place concrete. The concrete moment frames act as a lateral resisting system for the building as well as providing the primary gravity system. Each floor of the tower is approximately 17,000 square feet.

The tower is a system of concrete columns and a one-way slab system. There are 44 columns in the typical tower floor plan, 22”x28” or 22”x32”, with 4 smaller columns, 12”x18” or 16”x24” columns around each of the two stair towers. Typical bay sizes are: 27’-0”x18’-0” and 27’-0”x23’-0”. *(See plans, next page.)*

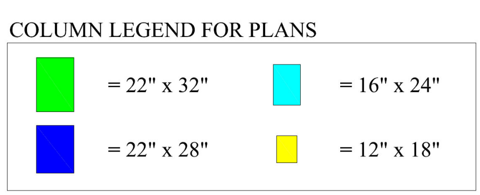
6’ wide, 8” deep column strips are oriented N-S on the typical tower plans. The floor slab consists of an 8” thick slab with polystyrene voids in a typical layout between column strips (see plan and section views below).

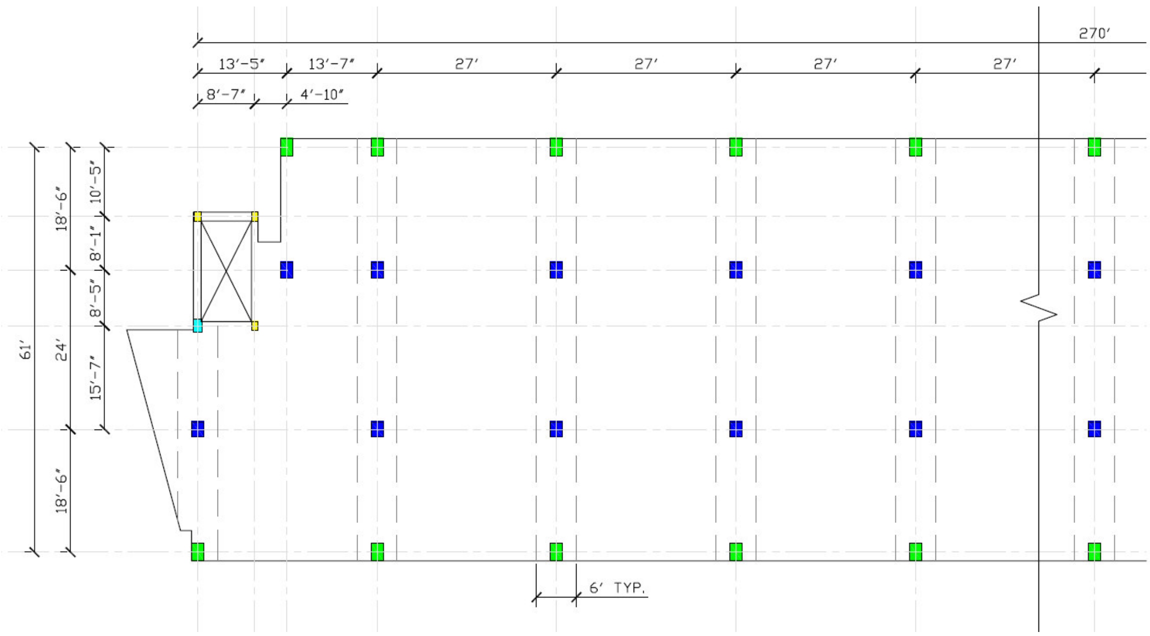



**TYPICAL BAY VOID LAYOUT**

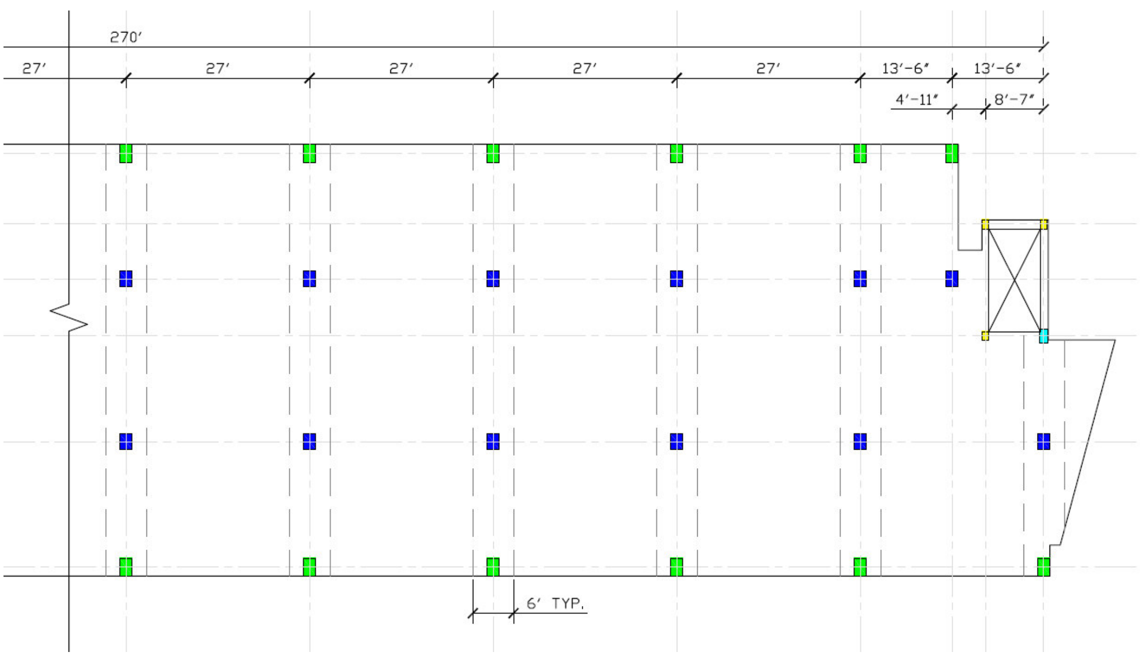



**SECTION THROUGH VOIDS**






**TOWER FRAMING PLAN – WEST END**




**TOWER FRAMING PLAN – EAST END**

## CODES AND REQUIREMENTS

- BOCA 1996 – adopted by the Township of Findlay, PA.
- AISC 1989 – "Specification for Structural Steel Buildings – Allowable Stress Design and Plastic Design" (Note: new load checks performed use LRFD design)
- ACI 318-89 – "Building Code Requirements for Reinforced Concrete"

## MATERIAL STRENGTHS

**Structural Steel**

| | |
|---|---|
| • Rolled Shapes | ASTM A572, Grade 50 |
| • Plates, Angles, Channels, Connection Materials | ASTM A36 |
| • Tube Sections | ASTM A500, Grade B |
| • Pipe Sections | ASTM A53, Grade B |
| • Anchor Bolts | ASTM A307 |

**Cast-in-place Concrete (Normal weight)**

| | |
|---|---|
| • Pile Caps | 3000 psi |
| • Columns | 5000 psi |
| • Basement Slab-on-Grade | 3000 psi |
| • Walls, Grade Beams, Structural Slab-on-Grade | 4000 psi |
| • Slabs on Metal Deck | 3500 psi |
| • Tower Slabs and Beams | 4500 psi |

**Reinforcement**

| | |
|---|---|
| • Deformed Rebar | ASTM A615, Grade 60 |
| • Welded Wire Fabric | ASTM A185, Grade 60 |

**Bolts and Welds**

| | |
|---|---|
| • Welding Electrodes | E70XX Low-Hydrogen |
| • Bolting Materials | ASTM A325 or A490 |

## GRAVITY LOADS

Design loads with updates from ASCE 7-02 – Minimum Design Loads for Buildings and Other Structures.

*Live Loads*

| | |
|---|---|
| • Basement – Slab-on-grade | 75 psf |
| • First Floor – Structural Slab | 100 psf |
| • Lobby | 100 psf |
| • Conference Center Roof | 12 psf |
| • Tower Roof | 20 psf |
| • Guest Rooms | 40 psf |
| • Tower Corridors | 100 psf |

*Dead Loads*

| | |
|---|---|
| • Basement – Slab-on-grade | 75 psf |
| • First Floor – Structural Slab | 125 psf |
| • Lobby | 60 psf |
| • Conference Center Roof | 30 psf |
| • Tower Roof | 80 psf |
| • Guest Rooms | 80 psf |
| • Tower Corridors | 80 psf |

*Superimposed Dead Loads*

| | |
|---|---|
| • Basement – Slab-on-grade | 20 psf |
| • First Floor – Structural Slab | 20 psf |
| • Lobby | 40 psf |
| • Conference Center Roof | 30 psf |
| • Tower Roof | 20 psf |
| • Guest Rooms | 20 psf |
| • Tower Corridors | 20 psf |

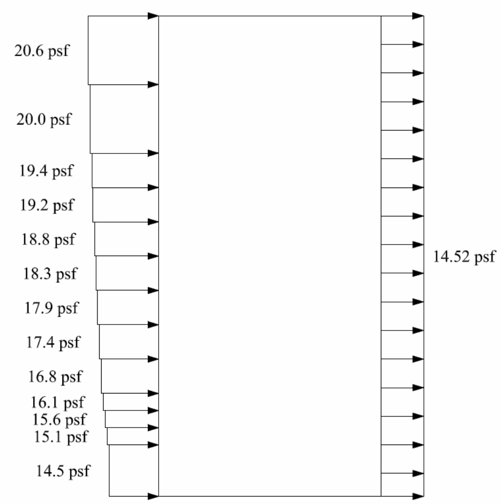
*Snow Load*

| | |
|---|---|
| • Roof Snow Load (See Appendix A) | 25 psf |

## WIND LOADS

The design wind loads have been determined in accordance to IBC 2003 and ASCE 7-02. Wind loads have been calculated based on the 11-story, 140-foot tower of the building. The main building factors for determining the wind loads are the basic wind speed of 90mph, exposure C, importance category II. The calculations assume that the building behaves as a rigid, rectangular structure. There is some variation between the calculated loads and those in the design documents; however, this is most likely due to code changes, and the values are not significantly different. Wind loading calculations are detailed in Appendix b.


**E/W Wind Loading**

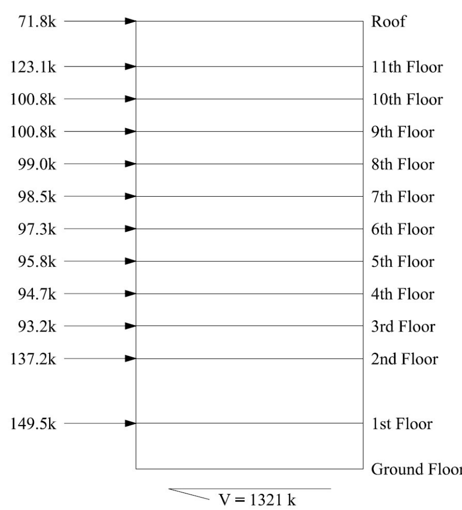



**N/S Wind Loading**

## STORY SHEARS

Story shears have been determined from the tributary area to each story. Refer to Appendix C for area and shear calculations.

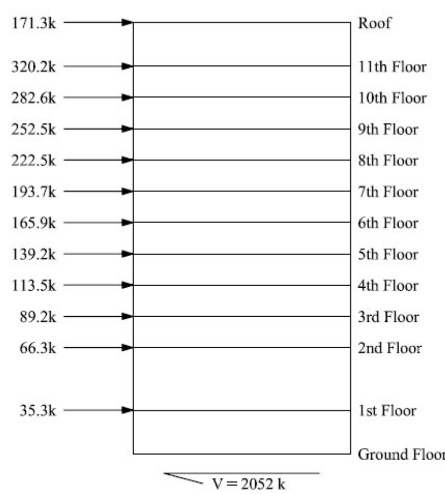

**E/W Story Shears**




**N/S Story Shears**

## SEISMIC LOADS

Seismic calculations have been calculated using IBC 2003 and ASCE 7-02. The loads have been calculated based on the tower, similar to wind loading. The original design of the building did not include seismic requirements, so these loadings were most likely not considered during the design of the concrete moment framing that serves as the lateral resisting system for the building. The calculations were made using the Equivalent Lateral Force Procedure. The building weight was approximated for the calculations based on a typical tower floor plan, so the value may vary slightly from the actual weight, but this should not change the loading significantly. Seismic loads are the same from each direction. Detailed calculations can be found in Appendix C.

## SPOT CHECKS
**(see calculations in Appendix D)**

Interior Tower Column – A spot check was performed at ground level of an interior 22"x28" concrete column for gravity loads only. The 5000 psi column has 14-#9 bars for reinforcement, with $f_y$ = 60 ksi. The loads stated previously were used for the analysis; in addition, the weight of the slabs and the column self-weight were considered. The column has a tributary area on each floor of 27'x21'3", with an influence area 4 times that size. A live load reduction was used with the limitations noted in ASCE 7-02. An axial load of 1690 kips was found from the above floors. The column had a design axial strength of 1770 kips, which is suitable for the load case checked. One possible problem with the loading is that the column may also have a moment on it, reducing its acceptable axial capacity.

Long-span Joist – A spot check was performed on one of the long span joists in the conference center. The joist spans 73' and has a tributary width of 9'; this width is based on an approximation from all joists. The joist is a 68DLH17, with full bracing detailed in the structural drawings. The maximum allowable loads were taken from the Vulcraft 2003 Joist catalog, and were found to be 820 lb/ft for dead loads and 394 lb/ft for live loads over a 73' span. The actual loading calculated was 505 lb for dead load and 333 lb for live load. These values are within the acceptable range for that selection of joist.

# APPENDIX

## APPENDIX A

**<u>Snow Load Calculations:</u>**

$p_g$ = 25 psf
$C_e$ = 0.9 (fully exposed, exposure C)
$C_t$ = 1.0
I = 1.0

$p_f = 0.7 C_e C_t I p_g \leq I p_g$

$p_f = 0.7*(0.9)*(1.0)*(1.0)*(25psf) = 15.75 \text{ psf} \leq 25 \text{ psf}$

**$p_f$ = 25 psf**

# APPENDIX B

**Wind Loading Calculations:**
(using ASCE 7-02 Method 2 – Analytical Procedure)

V = 90 mph
Exposure C
I = 1.0
$K_{zt}$ = 1.0 (no topographic features)
$K_d$ = 0.85 (main lateral system)
G = 0.85 (for rigid structures - assumed)
$GC_{pi}$ = ±0.18 (for enclosed buildings)

**Velocity Pressure, $q_z$**

| z (ft) | $K_z$ | $q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2\, I$ *(lb/ft²)* |
|---|---|---|
| 15 | 0.85 | 15.0 |
| 20 | 0.90 | 15.9 |
| 25 | 0.94 | 16.6 |
| 30 | 0.98 | 17.3 |
| 40 | 1.04 | 18.3 |
| 50 | 1.09 | 19.2 |
| 60 | 1.13 | 19.9 |
| 70 | 1.17 | 20.6 |
| 80 | 1.21 | 21.3 |
| 90 | 1.24 | 21.9 |
| 100 | 1.26 | 22.2 |
| 120 | 1.31 | 23.1 |
| 140 | 1.36 | 24.0 |

$q_h = 24.0\ \text{lb/ft}^2$

**Wall Pressure Coefficients, $C_p$**

| *Surface* | *Direction* | *L (ft)* | *B (ft)* | *L/B* | $C_p$ |
|---|---|---|---|---|---|
| *LEEWARD* | N/S | 65 | 273 | 0.2 | -0.5 |
| | E/W | 273 | 65 | 4.2 | -0.2 |
| *WINDWARD* | N/S, E/W | All Values | | | 0.8 |

**WIND PRESSURE CALCULATIONS:**

$p = qGC_p - q_i(GC_{pi})$ *(lb/ft²)*

**WINDWARD WIND PRESSURES:**

$p_{0\text{-}15}$ = 14.5 psf
$p_{20}$ = 15.1 psf
$p_{25}$ = 15.6 psf
$p_{30}$ = 16.1 psf
$p_{40}$ = 16.8 psf
$p_{50}$ = 17.4 psf
$p_{60}$ = 17.9 psf
$p_{70}$ = 18.3 psf
$p_{80}$ = 18.8 psf
$p_{90}$ = 19.2 psf
$p_{100}$ = 19.4 psf
$p_{120}$ = 20.0 psf
$p_{140}$ = 20.6 psf

**LEEWARD WIND PRESSURES:**

$p_{N/S}$ = -14.5 psf
$p_{E/W}$ = -8.4 psf

**STORY SHEARS AND BASE SHEAR CALCULATIONS:**

**East-West Story Shears**

| Story | Actual Elevation (ft) | Adjusted Elevation (ft) | Lower 'h' (ft) | Upper 'h' (ft) | Tributary Height (ft) | Tributary Width (ft) | $p_W$ (psf) | $p_L$ (psf) | Story Shear (k) |
|---|---|---|---|---|---|---|---|---|---|
| Ground | 1117 | 0 | 0 | 7 | 7 | 73 | 14.5 | 8.4 | 11.7 |
| 1 | 1131 | 14 | 7 | 24 | 17 | 73 | 15.6 | 8.4 | 29.8 |
| 2 | 1151 | 34 | 24 | 39 | 15 | 73 | 16.8 | 8.4 | 27.6 |
| 3 | 1161 | 44 | 39 | 49 | 10 | 73 | 17.4 | 8.4 | 18.8 |
| 4 | 1171 | 54 | 49 | 59 | 10 | 73 | 17.9 | 8.4 | 19.2 |
| 5 | 1181 | 64 | 59 | 69 | 10 | 73 | 18.3 | 8.4 | 19.5 |
| 6 | 1191 | 74 | 69 | 79 | 10 | 73 | 18.8 | 8.4 | 19.9 |
| 7 | 1201 | 84 | 79 | 89 | 10 | 73 | 19.2 | 8.4 | 20.1 |
| 8 | 1211 | 94 | 89 | 99 | 10 | 73 | 19.4 | 8.4 | 20.3 |
| 9 | 1221 | 104 | 99 | 109 | 10 | 73 | 20.0 | 8.4 | 20.7 |
| 10 | 1231 | 114 | 109 | 119 | 10 | 73 | 20.0 | 8.4 | 20.7 |
| 11 | 1241 | 124 | 119 | 131 | 12 | 73 | 20.6 | 8.4 | 25.4 |
| Roof | 1255 | 138 | 131 | 138 | 7 | 73 | 20.6 | 8.4 | 14.8 |

E-W Base Shear = 287 kips

**North-South Story Shears**

| Story | Actual Elevation (ft) | Adjusted Elevation (ft) | Lower 'h' (ft) | Upper 'h' (ft) | Tributary Height (ft) | Tributary Width (ft) | $p_W$ (psf) | $p_L$ (psf) | Story Shear (k) |
|---|---|---|---|---|---|---|---|---|---|
| Ground | 1117 | 0 | 0 | 7 | 7 | 292 | 14.5 | 14.52 | 59.3 |
| 1 | 1131 | 14 | 7 | 24 | 17 | 292 | 15.6 | 14.52 | 149.5 |
| 2 | 1151 | 34 | 24 | 39 | 15 | 292 | 16.8 | 14.52 | 137.2 |
| 3 | 1161 | 44 | 39 | 49 | 10 | 292 | 17.4 | 14.52 | 93.2 |
| 4 | 1171 | 54 | 49 | 59 | 10 | 292 | 17.9 | 14.52 | 94.7 |
| 5 | 1181 | 64 | 59 | 69 | 10 | 292 | 18.3 | 14.52 | 95.8 |
| 6 | 1191 | 74 | 69 | 79 | 10 | 292 | 18.8 | 14.52 | 97.3 |
| 7 | 1201 | 84 | 79 | 89 | 10 | 292 | 19.2 | 14.52 | 98.5 |
| 8 | 1211 | 94 | 89 | 99 | 10 | 292 | 19.4 | 14.52 | 99.0 |
| 9 | 1221 | 104 | 99 | 109 | 10 | 292 | 20.0 | 14.52 | 100.8 |
| 10 | 1231 | 114 | 109 | 119 | 10 | 292 | 20.0 | 14.52 | 100.8 |
| 11 | 1241 | 124 | 119 | 131 | 12 | 292 | 20.6 | 14.52 | 123.1 |
| Roof | 1255 | 138 | 131 | 138 | 7 | 292 | 20.6 | 14.52 | 71.8 |

N-S Base Shear = 1321 kips

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## APPENDIX C

**Seismic Loading Calculations:**
(using ASCE 7-02 Equivalent Lateral Force System)

For Pittsburgh, PA

$S_s = 0.127g$
$S_1 = 0.054g$

Occupancy II
Seismic Use Group I
$I_E = 1.0$

Site Class: D *(without sufficient detail to determine a Site Class, class D shall be used. As found in the geotechnical report prepared by L. Robert Kimball & Associates, the samples have a plasticity index (PI) ranging from 8-20. Site Class E is not used since the PI indicates that it is not a soft clay (PI>20)*

Based on site class, $S_s$, and $S_1$,

$F_a = 1.6$ $F_v = 2.4$

$S_{DS} = {}^2/_3 S_{MS} = {}^2/_3 F_a Ss = {}^2/_3(1.6)(0.127) = 0.135$
$S_{D1} = {}^2/_3 S_{M1} = {}^2/_3 F_v S_1 = {}^2/_3(2.4)(0.054) = 0.086$

$S_{DS}$, $S_{D1}$, and Seismic Use Group I, yields:

Seismic Design Category B

Based on this Seismic Design Category, the Equivalent Lateral Force System is permissible.

**Base Shear Calculation:**

$V_{BASE} = C_S W$

$C_S = S_{DS}/(R/I_E) \geq \mathit{0.044S_{DS}I_E}$

R = 3.0 (for ordinary reinforced concrete moment frames)

$C_S = 0.135/(3.0/1.0) \geq \mathit{0.044(0.135)(1.0)}$
$= 0.045 \geq \mathit{0.006}$ *(OK)*

W (Total weight is calculated from the typical floor plan for the tower. It is assumed for simplification that each floor has the same total weight, although some minor differences will occur.)

**Weight of Each Floor**

| Columns | | | | | | | |
|---|---|---|---|---|---|---|---|
| | *L* | *W* | *H* | *lb/ft3* | *#/floor* | | *Wt.* |
| | 22" | 32" | 10' | 150 pcf | 44/floor | = | **322.7k** |
| | 12" | 18" | 10' | 150 pcf | 8/floor | = | **13.5k** |
| Col. Strip | | | | | | | |
| | 61' | 72" | 8" | 150 pcf | 11/floor | = | **402.6k** |
| Slab | | | | | | | |
| | *t* | *SF* | | *lb/ft³* | | | |
| | 8" | 17000 sf | | 150 pcf | | = | **1700k** |
| Dead Load | | | | | | | |
| | | *SF* | | *lb/ft²* | | | |
| | | 17000 sf | | 80 | | = | **1360k** |

$W_{TOT} = 12 * \Sigma W_{FLOOR} = 12 * (3800k) = 45600$ k

$V_{BASE} = 0.045(45600k) = 2052$ k

**Distribution to Floors:**

The building does not exceed 12 stories, the lateral resisting system is entirely concrete, and the story height is at least 10ft, therefore the following assumption is valid:

$T_a = 0.1N = 0.1(11) = 1.1$ sec

$k = 1.3$ *(linear interpolation between 1 and 2 for a value of $T_a = 1.1$ sec)*

$F_x = C_{vx}V$ *(force at story x)*

$C_{vx} = w_x h_x^k / (\Sigma w_i h_i^k)$

**Story Forces**

| Story | $w_x$ | $h_x$ | $w_x h_x^k$ | $C_{vx}$ | Story Force |
|---|---|---|---|---|---|
| 1 | 3800 | 24 | 236624 | 0.017 | 35.3 |
| 2 | 3800 | 39 | 444803 | 0.032 | 66.3 |
| 3 | 3800 | 49 | 598465 | 0.043 | 89.2 |
| 4 | 3800 | 59 | 761888 | 0.055 | 113.5 |
| 5 | 3800 | 69 | 933872 | 0.068 | 139.2 |
| 6 | 3800 | 79 | 1113522 | 0.081 | 165.9 |
| 7 | 3800 | 89 | 1300141 | 0.094 | 193.7 |
| 8 | 3800 | 99 | 1493170 | 0.108 | 222.5 |
| 9 | 3800 | 109 | 1692147 | 0.123 | 252.2 |
| 10 | 3800 | 119 | 1896683 | 0.138 | 282.6 |
| 11 | 3800 | 131 | 2148999 | 0.156 | 320.2 |
| Roof | 1900 | 138 | 1149731 | 0.083 | 171.3 |
| | | Σ | 13770045 | 1 | 2052 |

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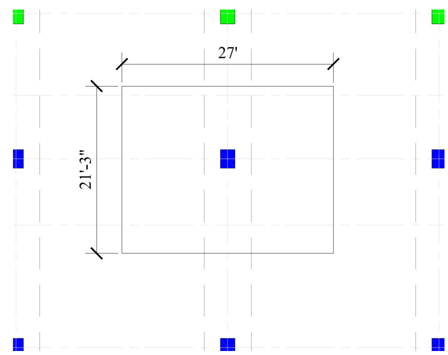
## APPENDIX D

**SPOT CHECK OF INTERIOR TOWER COLUMN – GRAVITY ONLY:**




$f_y$ = 60 ksi
$f'_c$ = 5000 psi
$A_s$ = 14 - #9 = 14(1) = 14 in$^2$
$A_c$ = 22" x 28"
$A_{TRIB}$ = 27'x21'-3" = 574 ft$^2$
#4 ties
$k_{LL}$ = 4 *(live load reduction factor)*

Assuming ½ loading is from room, ½ is from corridor, for a general approximation.

Number conversions not shown for simplicity.

$w_d$ = 574 ft$^2$ [20lb/ft$^2$ + 10(40 lb/ft$^2$/2+100 lb/ft$^2$/2)] + 150lb/ft$^3$[(22"x28"x138')+11(8"x574 ft$^2$)]
= (574 ft2 x 720 lb/ft$^2$) + (4800 ft$^3$ x 150lb/ft$^3$)
= 1133.3 k

$w_l$ = 574 ft$^2$ [11x(80 lb/ft$^2$)] (0.4) *(including live load reduction of 60%)*
= 202 k

$w_s$ = 25 lb/ft$^2$ x 574 ft$^2$
= 14.35 k

Load combinations

1.4 $w_d$ = 1.4 (1133.3) = 1587 k
1.2 $w_d$ + 1.6 $w_l$ + 0.5 $w_s$ = 1.2 (1133.3) + 1.6 (202) + 0.5 (14.35) = **1690 k**
1.2 $w_d$ + 1.6 $w_s$ + $w_l$ = 1.2 (1133.3) + 1.6 (14.35) + 202 = 1585 k

$P_o = 0.85 f'_c (A_c - A_s) + A_s F_y$ = 0.85(5)[(22)(28) – (14)] + 60(14) = 3400 k

$P_u$ = 1690 k ≤ 0.8 (0.65) Po = 1768 k **OK**

**SPOT CHECK OF LONG-SPAN JOIST OVER BALLROOM – GRAVITY:**

The joists over the ballroom must support the roof structure.
They consist of 68DLH17 long-span joists spanning 73'.
This checks a typical joist with tributary width of 9'

$A_{TRIB}$ = 73' x 9' = 657 $ft^2$
$w_{SELF}$ = 55 lb/ft

$w_d$ = 50 lb/$ft^2$ (657 $ft^2$) + 55lb/ft (73 ft) = 36,900 lbs

$w_l$ = 12 lb/$ft^2$ (657 $ft^2$) = 7,884 lbs

$w_s$ = 25 lb/$ft^2$ (657 $ft^2$) = 16,425 lbs

SAFE DL LOAD for 70-99 ft span = 60400 lbs / (73+0.67) = 820 lb/ft
SAFE LL LOAD for 70-99 ft span = 275*(105+0.67) / (73 + 0.67) = 394 lb/ft

Dead Load = 36,900lb/73ft = 505 lb/ft **OK**
Live Load (including snow) = (7884lb + 16425lb)/73ft = 333 lb/ft **OK**