# NORTH SHORE EQUITABLE BUILDING

PITTSBURGH, PA

STEPHAN NORTHROP - STRUCTURAL OPTION

## TECHNICAL REPORT #1

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OCTOBER 4, 2010

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# EXECUTIVE SUMMARY

In Technical Report 1, an analysis was performed on the existing conditions of the North Shore Equitable Building in Pittsburgh Pennsylvania. This analysis includes a brief look at the architectural and functional features of the building and an in depth look at the structural systems of the building.

The structural system of the North Shore Equitable Building is a steel frame of beams and girders combined with braced frames and moment frames surrounding the core of the building on all levels to resist lateral loads. The floor system is a composite floor slab with a metal floor deck and the roof system consists of a galvanized roof deck supported by K-series joists and steel girders. The foundation, which is designed to accommodate a future subgrade light rail transit line extension, incorporates a unique combination of auger cast piles and steel H piles. These structural features will be touched upon in greater detail in later sections of this report.

An overview of the foundation, the gravity system and the lateral systems is included. An overview of all loads the building was designed for is also provided. In addition to these overviews, wind load calculations and seismic load calculations have been performed as well to gain a better understanding of the design factors that went into the design of this structure. Finally, spot checks of a typical beam, girder and column were hand calculated to check and compare the sizes chosen by the structural engineer who designed the North Shore Equitable Building.

To supplement the analysis of the building structure, a list of codes used in the design is provided, along with a table of material strengths. Also, appendices with detailed calculations and additional plans and elevations have been provided to further supplement the results of the analysis. The results of this analysis have been discussed throughout the text.

From the analysis that follows, it can be concluded that the building as designed is capable of withstanding all loads and forces. Furthermore, it can be seen from the wind analysis and the seismic analysis that the wind is the controlling force in the design of this building. It can also be added that the wind controls in the north/south direction with a base shear of 385.45 kips and an overturning moment of 54609 Ft-K.

# 1. INTRODUCTION

The North Shore Equitable Building is a 6 story, 180,000 square foot low rise commercial office building located on Pittsburgh's North Shore. Completed in 2004, this building is part of the North Shore development project between Heinz Field and PNC Park. Of the building's 180,000 square foot area, 150,000 square feet consists of office space on floors 2 to 5 and the remaining 30,000 square feet is retail space on the ground level. In addition to the 6 above grade levels, one sublevel of parking is also provided, which accommodates 80 vehicles. The North Shore Equitable Building offers its tenants amenities such as an employee fitness center, a test kitchen for product development and the North Shore Riverfront Park (shown in figure 1) which offers access to riverside trails and beautiful views of the Pittsburgh skyline across the Allegheny River.

*Figure 1: View of the North Shore Equitable building looking from the southwest with the North Shore Riverfront Park in the foreground*

Among the Equitable building's notable architectural features are what is referred to as a turret, located at the southwest corner of the building and two towers located at the northwest and southeast corners of the building respectively (also shown in figure 1). The majority of the building's façade consists of cast stone masonry units up to the third level and a combination of composite metal paneling and face brick from the third level up to the roof level. Two skylights can be found on the roof as well with the architectural designs including a location for a proposed third skylight which was never built.

## 2. STRUCTURAL SYSTEMS OVERVIEW

As mentioned in the introduction, the structural system of the North Shore Equitable Building consists of steel beams and girders to resist gravity loads and a combination of braced frames and moment frames to resist lateral loads. These components of the building's structural design, along with all other structural design components, will be described in further detail below.

### Foundation

The foundation consists of a slab on grade supported by concrete grade beams and a combination of 18" auger cast piles and steel H-piles. The slab on grade is a 5 ½" concrete slab reinforced with 6x6 W2.9xW2.9 welded wire fabric. Interior grade beams (figure 2) are typically 2' wide and range from 2' to 3' deep. The exterior grade beams (figure 2) range from 3'4" to 4' wide and from 2' to 3'4" deep. All grade beam reinforcing is continuous through the pile caps and piers. The walls of the parking garage, which are reinforced concrete retaining walls, extend from the top of the grade beams to the first floor framing. These walls are restrained at the top by the first floor framing.

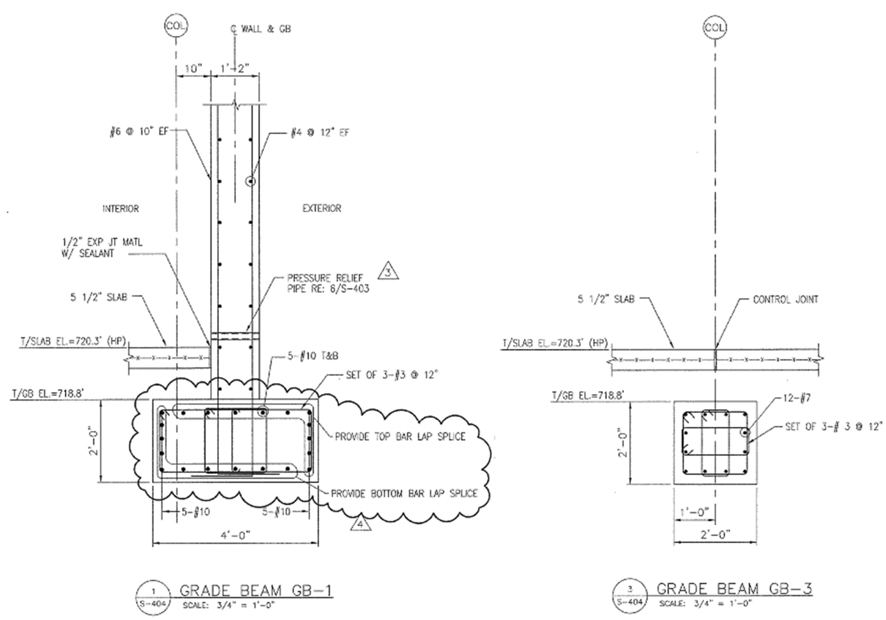



***Figure 2: Typical exterior grade beam (left) and typical interior grade beam (right)***

The piles for the Equitable Building pose a unique set of design requirements. The Allegheny Port Authority is currently undertaking a project that involves extending the light rail transit system from downtown Pittsburgh to an underground stop on the North Shore. This connection consists of two parallel tunnels running from the Gateway Plaza station below Stanwix Street to the North Shore. These two tunnels are designed to pass directly below the Equitable Building as seen in figure 3. As a result of this, the foundation is designed as a combination of two types of foundations; driven Steel H-piles (figure 4 on the right) to withstand pressures and settlement resulting from tunneling under the building and 18” auger cast piles (figure 4 on the left) for the remainder of the foundation. The steel H-piles are designed to resist a maximum uniform pressure of 4.25 ksf greater than the existing soil pressure.

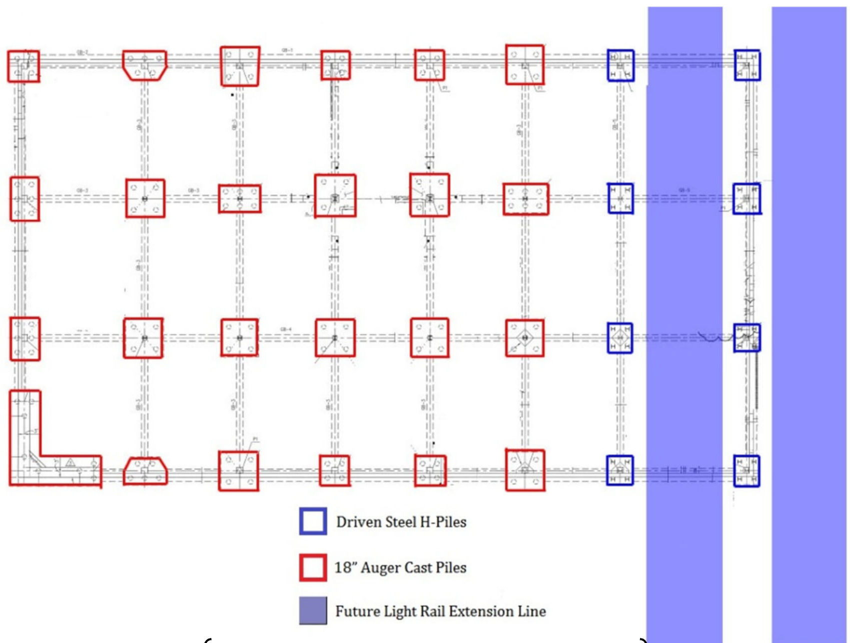
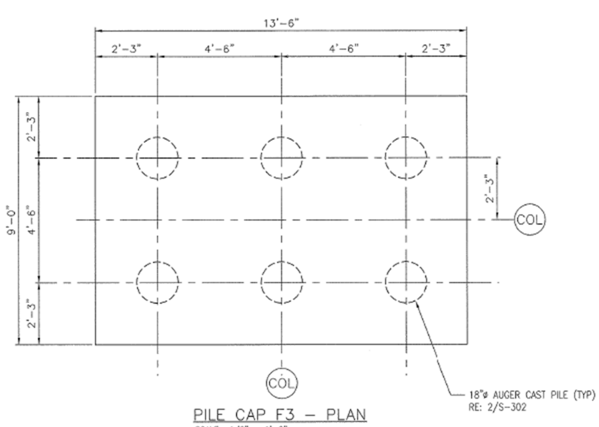



***Figure 3: Foundation plan with future transit line extension***



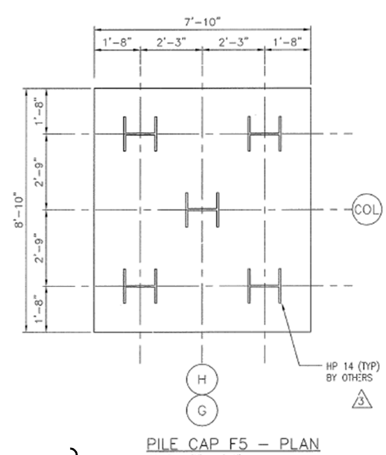




***Figure 4: Typical 18” auger cast pile cap (left) and typical steel H pile cap (right)***

## General Floor Framing

Due to the relatively rectangular shape of the North Shore Equitable building, the framing follows a simple grid pattern (128' wide by 228' long). Framing consists of a lightweight concrete slab supported by steel beams girders and columns. The slab has a total depth of 5 ½" consisting of 3 ½" lightweight concrete over 2" 18 gage composite galvanized metal floor deck reinforced with 6x6 W2.2xW2.1 welded wire fabric. The floor is supported by steel beams, typically W18x40's in exterior bays and W21x44's in interior bays, framing into girders ranging in size from W24x62 to W30x116. There are 7 bays on each level (approximately 30' x 42' or 40' x 42' for exterior bays and 30' x 44' or 40' x 44' for interior bays). The beams span 44' in the interior bays and 42' in the exterior bays and are spaced no more than 10' apart. The girders typically span either 30 or 40 feet. Shear studs (4 ½" length, ¾" diameter) are used to create composite action between the deck and the steel beams. The deck spans in the longitudinal direction perpendicular to the beams. The framing plan for level 2, which is nearly identical to floors 1-5, can be seen below in figure 5. The 6th floor framing plan differs slightly from floors 1-5 in that it includes some thicker beams and a few more transfer beams in order to accommodate a balcony spanning the 3 most interior bays on the south face of the building (figure 6).

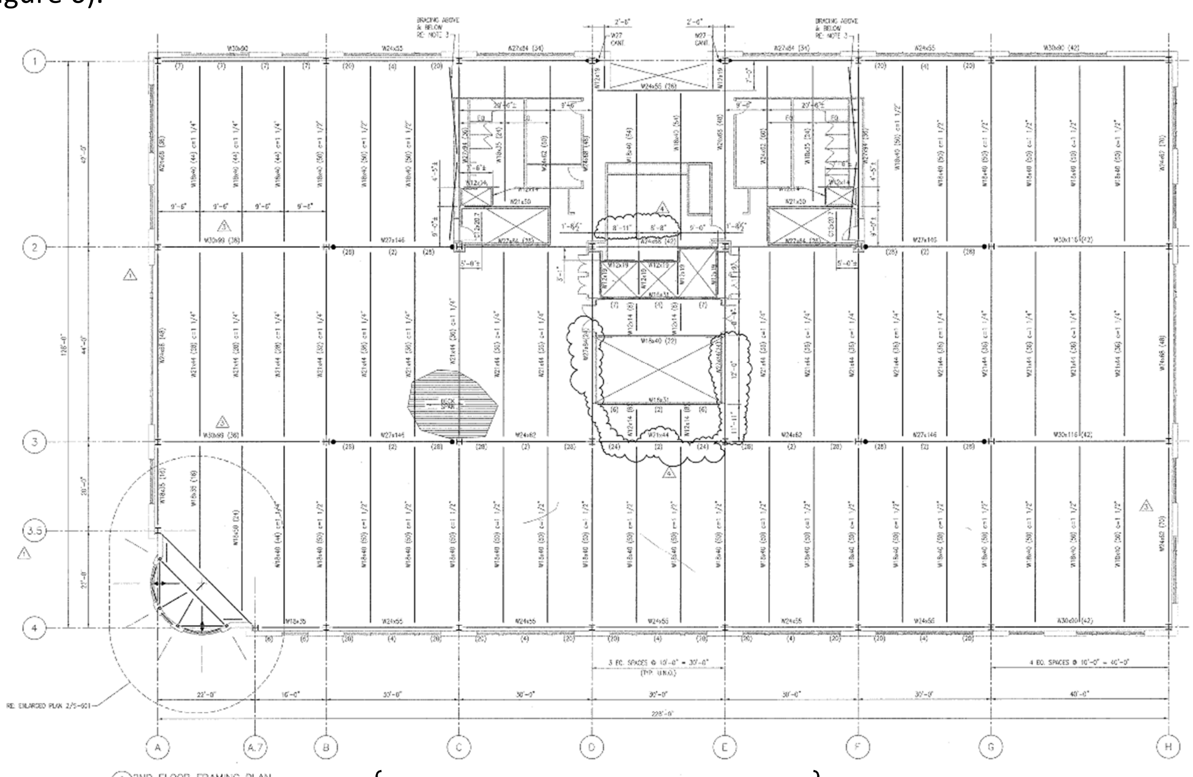



***Figure 5: Typical floor framing plan***

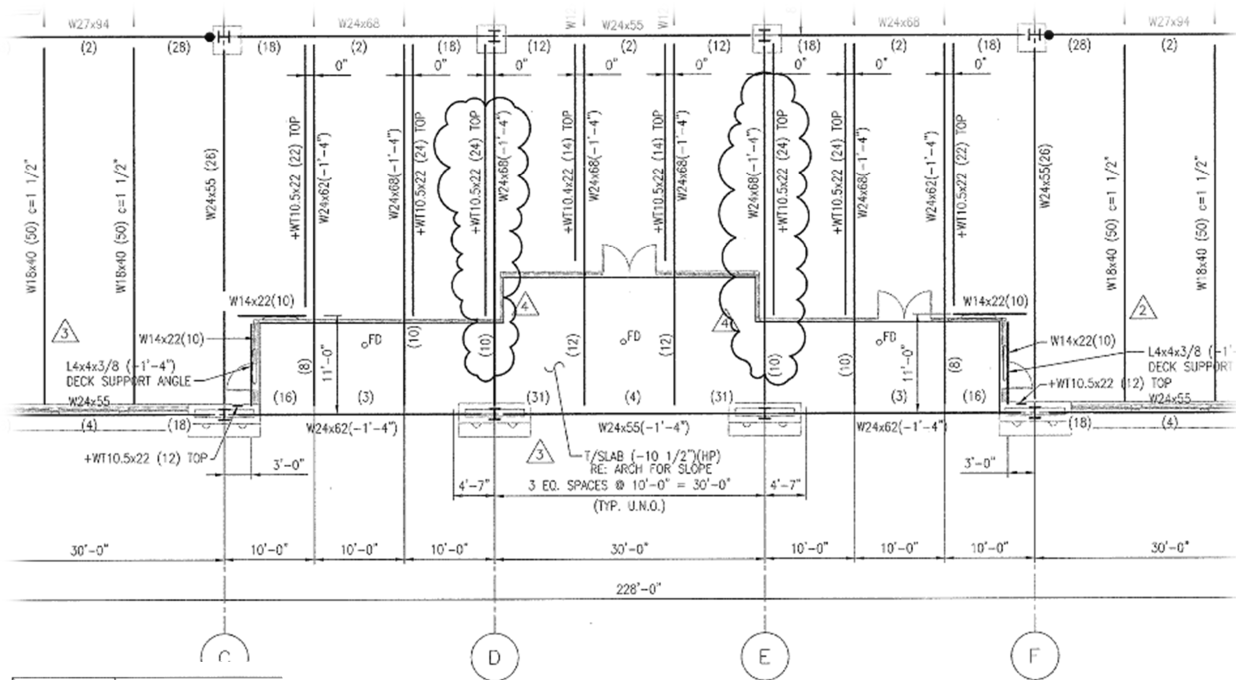


*Figure 6: Framing plan for 6th floor balcony*

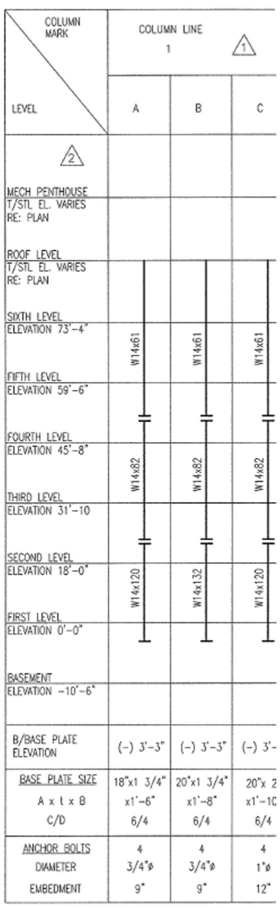



*Figure 7: Sample of column chart showing column splice locations*

Columns for the Equitable Building are all W14 wide flange columns ranging in weight from W14x311 on the first level to W14x48 extending up to the roof level. Columns are spliced at two locations along the vertical length of any given column line as shown in figure 7 to the left. These splices are generally located 4' above the floor level indicated, have minimum flange tension strength of 60 kips and a web shear strength of 20 kips. A typical column splice detail is shown to the right in figure 8.

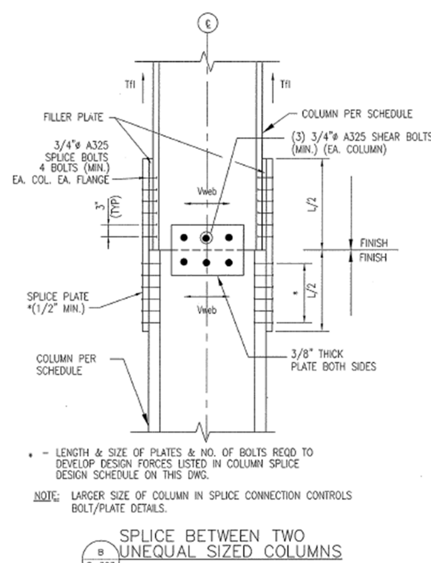



*Figure 8: Typical column splice detail*

## Turret Framing Plan

For the turret at the southwest corner of the building, members of varying sizes are used as seen below in figure 9. The columns for the turret are HSS columns ranging in size from HSS 6x6x 1/2 (on the first level) to HSS 6x6x 3/16 extending up to the roof level. These HSS columns are spliced at three locations along the column line.

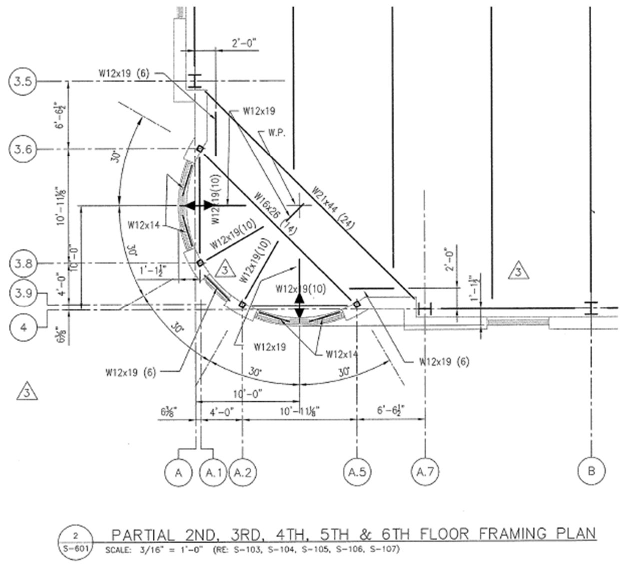



*Figure 9: Turret framing plan*

## Roof Framing Plan

The roof framing system, like the floor framing system, is laid out in a simple rectangular grid. It consists of a 1 ½" 20 gage type B galvanized roof deck supported by open-web K-series joists (figure 10) which frame into wide flange girders. The roof deck spans longitudinally which is perpendicular to the joist span direction. The K-series joists are generally either 28" or 30" deep and span either 44' (in interior bays) or 42' (in exterior bays). These joists are spaced no further apart than 5' typically. A handful of shallower joists are used for smaller spans at areas of irregularity in the plan such as at the turret, tower and stairwell locations.

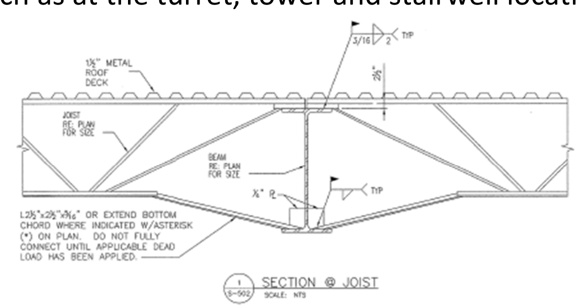



*Figure 10: Section at joist*

The girders in the roof plan vary greatly in both size and span length. This is because some girders spanning between column lines (in both the longitudinal and transverse directions) are carrying a typical roof load and some girders (spanning much shorter distances) are designed to carry a much larger load caused by mechanical equipment located on the roof above the core of the building. Girders carrying the typical roof load vary in size from W18x35's to W30x116's (spanning anywhere from 16' to 44'). The roof girders above the core of the building supporting mechanical equipment are mainly W12x19's and W24's with a few W14's and W18's used as well. 10" and 30" deep KCS-Type open-web K-series joists are also used to help support this equipment. A framing plan of the bays supporting mechanical equipment can be seen below in figure 11.

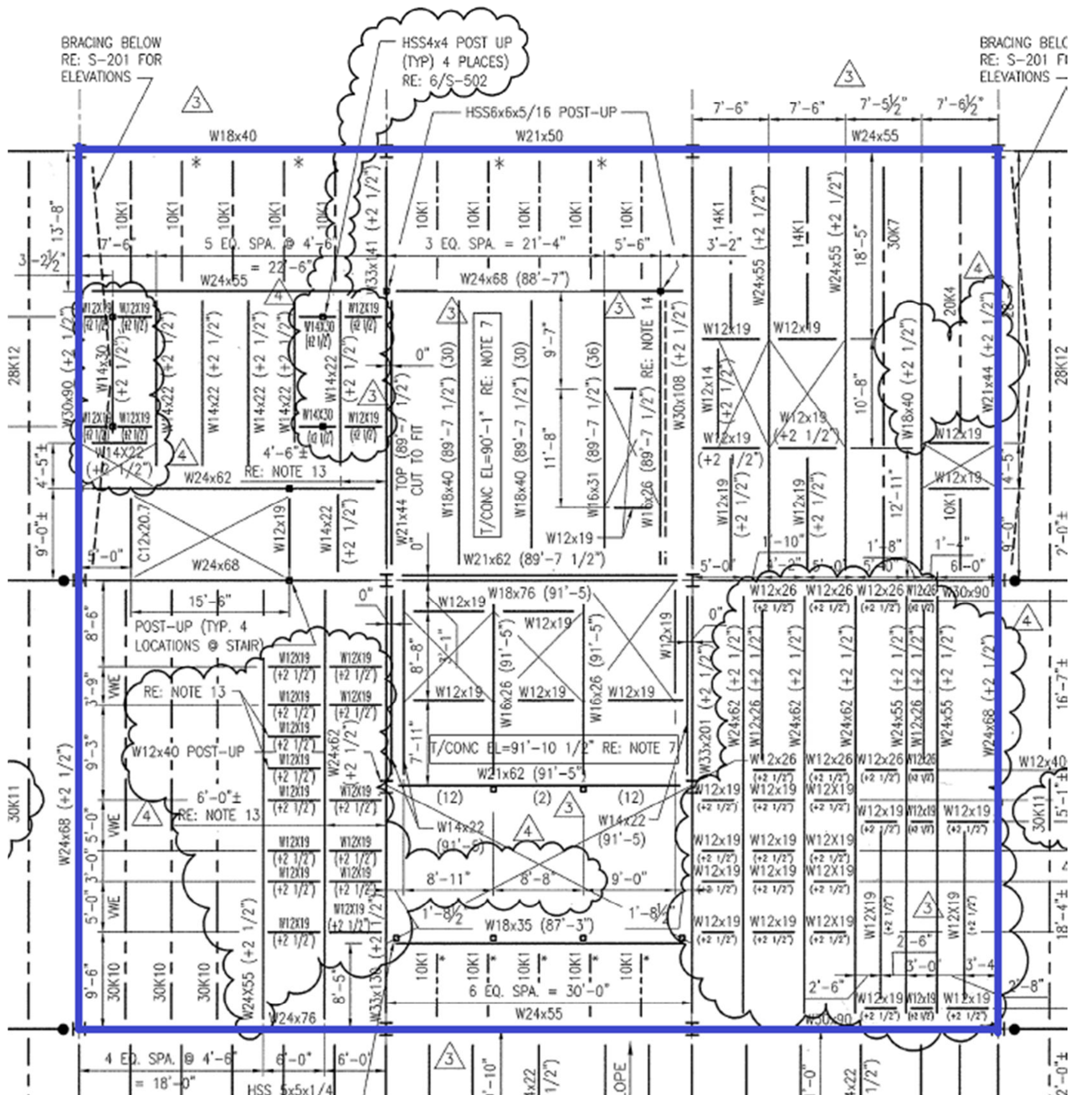



*Figure 11: Enlarged roof framing plan showing bays which support mechanical equipment*

The framing of the tower roofs consists of C10x20's, W10x22's and L2 ½ x 2 ½ x ¼ horizontal bridging, as seen in figure 12. The framing of the turret roof consists of curved C6x13 members and wide flange members of varying lengths as seen in figure 13.

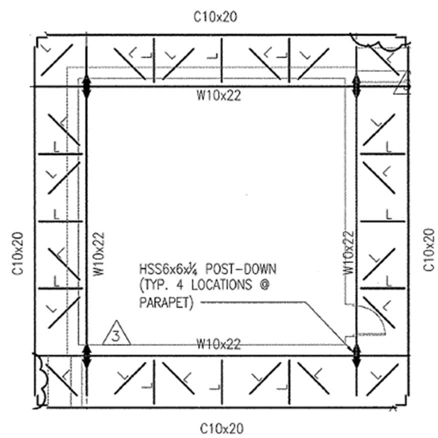



*Figure 12: Tower roof framing plan*

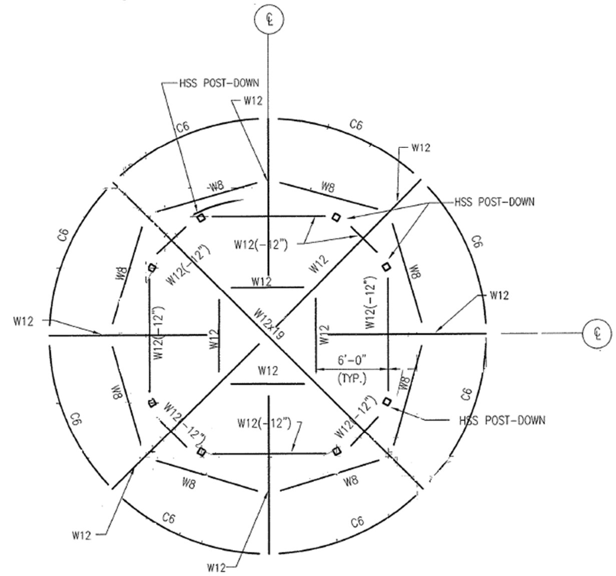



*Figure 13: Turret roof framing plan*

## Lateral Resisting System

Lateral stability in the North Shore Equitable Building is achieved through the use of a combination of braced frames and moment frames. Braced frames run in the transverse direction and moment frames run in the longitudinal direction as seen in figures 14 and 15 below. The floor and roof decks, which act as horizontal diaphragms, transfer lateral forces to the frames. Elevation views of these frames can be seen in figures 16 and 17. The connections in the moment frames are semi rigid connections. Details of a typical braced frame connection and a moment frame connection are shown in figures 18 and 19 respectively.


*Figure 14: Lateral Resisting elements at level 1*

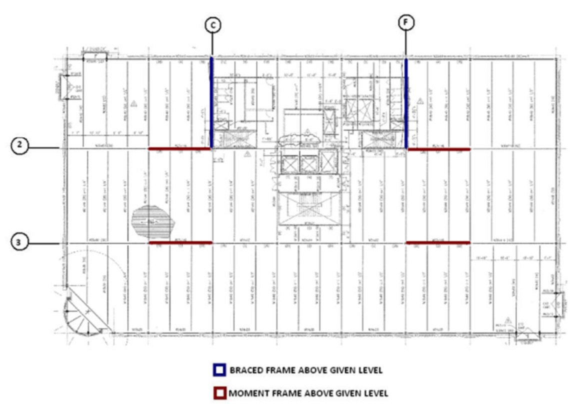



*Figure 15: Lateral Resisting elements at levels 2-6*

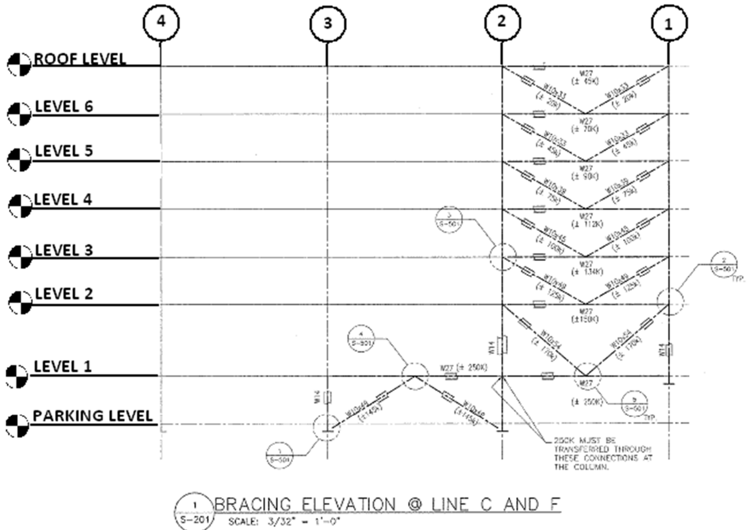



*Figure 16: Braced frame elevation*

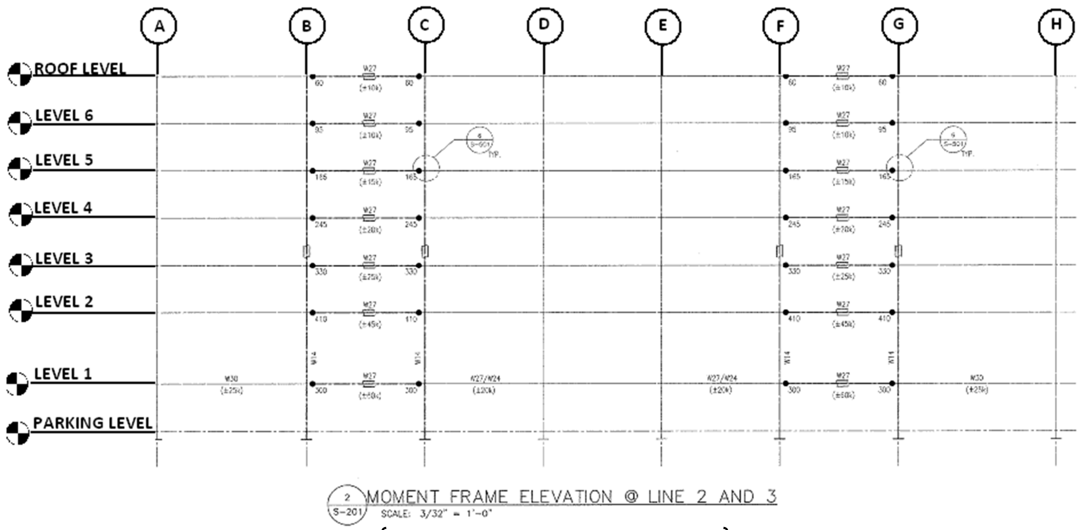



*Figure 17: Moment frame elevation*

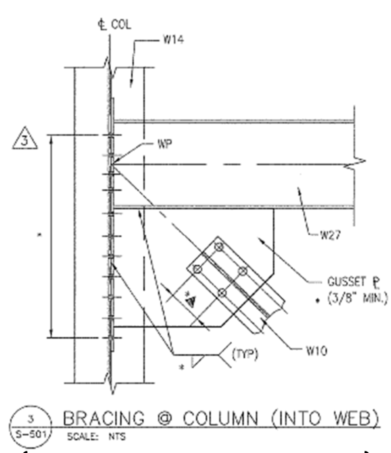



*Figure 18: Braced frame connection*

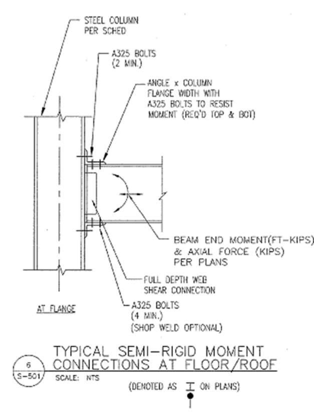



*Figure 19: Moment frame connection*

# 3. MATERIALS USED

**TABLE 3.1 - Concrete Materials Schedule**

| Structural Element | Weight (pcf) | Strength (f'c) |
|---|---|---|
| Footings | 150 | 4000 |
| Drilled Piers | 150 | 4000 |
| Grade Beams | 150 | 4000 |
| Slab On Grade | 150 | 4000 |
| Elevated Floor Slabs | 110 | 4000 |
| Auger Cast Piles | 150 | 4000 |
| All Other Concrete | 150 | 4000 |

**TABLE 3.2 - Masonry Materials Schedule**

| Structural Element | Compressive Strength |
|---|---|
| Concrete Masonry | 1500 PSI |

**TABLE 3.3 - Steel Materials Schedule**

| Structural Element | Yield Strength (ksi) | ASTM Designation |
|---|---|---|
| Steel Roof Deck | 33 (minimum) | A446 |
| Beams And Columns | 50 | A992 |
| Rectangular Tube Steel | 46 | A500 Grade B |
| Bracing | 36 | A36 |
| Connections, Plates And All Others | 36 | A36 |
| Anchor Rods | 36 | A36 |
| Pipes | 35 | A53 Grade B |
| Round Tube Steel | 42 | A500 Grade B |
| Light Gage Metal Studs | 50 | A653 |
| Structural Steel Bolts | 92 | A325 |

**Column Splice Design Schedule**

| Splice Mark | Flange Tension (K) | Web Shear (K) |
|---|---|---|
| CS1 | 60 | 20 |
| CS2 | 85 | 20 |

# 4. APPLICABLE CODES

## Codes Used In the Original Design

- The BOCA National Building Code, 1999
- City of Pittsburgh Amendments to The Boca National Building Code
- ASCE 7-95, Minimum Design Loads for Buildings
- ACI 301, Specifications for Structural Concrete for Buildings
- ACI 318, Building Code Requirements for Reinforced Concrete
- ACI 530, Building Code Requirements for Masonry Structures
- AISC/ASD-89, Manual of Steel Construction, 9th Edition
- AISC/LRFD-2001, Manual of Steel Construction, 3rd Edition
- SJI-41st Edition, Standard Specifications and Load Tables for Steel Joists and Joist Girders

## Codes Used In Tech 1 Analysis

- ASCE 7-05, Minimum Design Loads for Buildings
- ASCE 7-10, Minimum Design Loads for Buildings (Chapter 26.9)
- AISC Manual of Steel Construction, 13th Edition
- ACI 318, Building Code Requirements for Reinforced Concrete

# 5. DESIGN LOADS

## Gravity Loads

**TABLE 5.1 - Live Loads**

| Load Type | As Designed (psf) | Per ASCE 7-05 (psf) |
|---|---|---|
| **Floor Live Loads** | | |
| Office | **100** | **50** |
| Corridors | **100** | **100 (first level)**<br>**80 (upper levels)** |
| Mechanical | **150** | **(not provided)** |
| Stairs | **100** | **100** |
| Retail | **100** | **100** |
| **Garage Live Load** | **50** | **40** |
| **Roof Live Load** | **20 (min)** | **20** |

**TABLE 5.2 - Dead Loads**

| Load Type | As Designed (psf) |
|---|---|
| Superstructure Weight | 5 |
| Roofing, Ceiling, Misc. | 8 |
| Collateral Load (MEP) | 7 |
| **Total Roof Dead Load** | **20** |
| 5 ½" Light Weight Conc. Slab | 45 |
| Steel/Joist Framing | 10 |
| Ceiling, Misc. | 5 |
| MEP | 5 |
| **Total Floor Dead Load** | **65** |
| 6" Metal Studs + Insul + GWB<br>4" Brick | 10<br>40 |
| **Total Exterior Wall Load** | **50** |
| Stairs | 30 |
| Stair Landings | 40 |

**TABLE 5.3 - Snow Loads**

| Load Type | As Designed (psf) | Per ASCE 7-05 (psf) |
|---|---|---|
| **Ground Snow Load** | **30** | **25** |
| **Roof Snow Load** | **21 + Drifting** | **20** |
| $C_e = 0.9$ | $C_t = 1.0$ | $I = 1.0$ |

## Wind Loads

Wind loads were calculated using the ASCE 7-05 Main Wind-Force Resisting System analytical procedure method 2. Before calculating wind loads, ASCE 7-10 chapter 26.9 was referenced to determine if the building was a rigid or flexible structure. Using this chapter, the approximate frequencies for both moment frames and braced frames were calculated. Both these frequencies were less than one, classifying the building as a flexible structure. The larger frequency value of the two was used in the following calculations to be conservative. Using the Main Wind-Force Resisting System guidelines for flexible structures, the wind loads were calculated and it was found that the North South Direction controlled based on the fact that a larger building face was exposed to the wind in this direction. Below are the results of the calculations. Detailed hand calculations can be found in Appendix B.

**TABLE 5.4 - Wind Analysis Design Criteria**

| | |
|---|---|
| Basic Wind Speed | 90 mph |
| Building Classification | II |
| Importance Factor (I) | 1.0 |
| Exposure Category | C |
| Mean Height (h) | 87.08 Ft. |
| Building Length (L) | 128 Ft. for N/S |
| Building Base (B) | 228 Ft. for N/S |
| Ridges or Escarpments? | None |
| Structure Type | Flexible |

**TABLE 5.5 - Wind Pressures In The East/West Direction**

| Level | Height | $K_z$ | $q_z$ | External | Internal | Net Pressures (psf) | |
|---|---|---|---|---|---|---|---|
| | (Ft.) | | (psf) | Pressure | Pressure | + $GC_{pi}$ | + $GC_{pi}$ |
| Level 1 | 0.00 | 0.00 | 0.00 | 11.55 | -3.90 | 7.65 | 15.45 |
| Level 2 | 18.00 | 0.88 | 15.55 | 11.55 | -3.90 | 7.65 | 15.45 |
| Level 3 | 31.83 | 0.99 | 17.53 | 13.03 | -3.90 | 9.13 | 16.93 |
| Level 4 | 45.67 | 1.07 | 18.91 | 14.06 | -3.90 | 10.16 | 17.96 |
| Level 5 | 59.50 | 1.13 | 20.00 | 14.86 | -3.90 | 10.96 | 18.76 |
| Level 6 | 73.33 | 1.19 | 20.90 | 15.53 | -3.90 | 11.63 | 19.43 |
| Roof | 87.08 | 1.23 | 21.67 | 16.10 | -3.90 | 12.20 | 20.00 |
| Tower | 99.33 | 1.26 | 22.28 | 16.56 | -3.90 | 12.66 | 20.46 |
| Turret | 108.33 | 1.29 | 22.69 | 16.86 | -3.90 | 12.96 | 20.76 |

**TABLE 5.6 - Wind Pressures In The North/South Direction**

| Level | Height | $K_z$ | $q_z$ | External | Internal | Net Pressures (psf) | |
|---|---|---|---|---|---|---|---|
| | (Ft.) | | (psf) | Pressure | Pressure | + $GC_{pi}$ | + $GC_{pi}$ |
| **Level 1** | 0.00 | 0.00 | 0.00 | 11.36 | -3.90 | 7.46 | 15.26 |
| **Level 2** | 18.00 | 0.88 | 15.55 | 11.36 | -3.90 | 7.46 | 15.26 |
| **Level 3** | 31.83 | 0.99 | 17.53 | 12.80 | -3.90 | 8.90 | 16.70 |
| **Level 4** | 45.67 | 1.07 | 18.91 | 13.82 | -3.90 | 9.92 | 17.72 |
| **Level 5** | 59.50 | 1.13 | 20.00 | 14.61 | -3.90 | 10.71 | 18.51 |
| **Level 6** | 73.33 | 1.19 | 20.90 | 15.26 | -3.90 | 11.36 | 19.16 |
| **Roof** | 87.08 | 1.23 | 21.67 | 15.83 | -3.90 | 11.93 | 19.73 |
| **Tower** | 99.33 | 1.26 | 22.28 | 16.27 | -3.90 | 12.37 | 20.17 |
| **Turret** | 108.33 | 1.29 | 22.69 | 16.57 | -3.90 | 12.67 | 20.47 |

**TABLE 5.7 - Wind Pressures Independent Of Height (East/West Direction)**

| Pressure | q value | $C_p$ value | G value | Pressure (psf) |
|---|---|---|---|---|
| Leeward | 21.67 | -0.34 | 0.929 | -6.93 |
| Sidewall | 21.67 | -0.70 | 0.929 | -14.09 |
| Roof from 0 to 87.08* | 21.67 | -0.90 | 0.929 | -18.12 |
| Roof from 87.08 to 174.16* | 21.67 | -0.50 | 0.929 | -10.07 |
| Roof from 174.16 to 228* | 21.67 | -0.30 | 0.929 | -6.04 |
| Dome at point A | 22.69 | -1.17 | 0.929 | -24.73 |
| Dome at point B | 22.69 | -1.10 | 0.929 | -23.19 |
| Dome at point C | 22.69 | -0.50 | 0.929 | -10.54 |

** Distances given are horizontal distances in feet from windward edge*

**TABLE 5.8 - Pressures Independent Of Height (North/South Direction)**

| | q value | Cp value | G value | Pressure |
|---|---|---|---|---|
| Leeward | 21.67 | -0.34 | 0.913 | -6.81 |
| Sidewall | 21.67 | -0.70 | 0.913 | -13.85 |
| Roof from 0 to 87.08 | 21.67 | -0.90 | 0.913 | -17.81 |
| Roof from 87.08 to 128 | 21.67 | -0.50 | 0.913 | -9.89 |
| Dome at point A | 22.69 | -1.17 | 0.913 | -24.30 |
| Dome at point B | 22.69 | -1.10 | 0.913 | -22.79 |
| Dome at point C | 22.69 | -0.50 | 0.913 | -10.36 |

** Distances given are horizontal distances in feet from windward edge*

**TABLE 5.9 - Story Wind Forces (East/West Direction)**

| Level | Height | Face Length | Elevation | Pressure | Story Force | Story Shear | Moment |
|---|---|---|---|---|---|---|---|
| | (Ft.) | (Ft.) | (Ft.) | (psf) | (K) | (K) | (Ft-K) |
| **Turret** | 8.13 | 22.67 | 103.33 | 20.76 | 3.83 | 3.83 | 395.37 |
| **Roof** | 15 | 128 | 87.07 | 20.00 | 38.41 | 42.23 | 3677.13 |
| **Level 6** | 13.79 | 128 | 73.32 | 19.43 | 34.30 | 76.53 | 5611.13 |
| **Level 5** | 13.83 | 128 | 59.49 | 18.76 | 33.21 | 109.74 | 6528.60 |
| **Level 4** | 13.83 | 128 | 45.66 | 17.96 | 31.79 | 141.53 | 6462.32 |
| **Level 3** | 13.83 | 128 | 31.83 | 16.93 | 29.97 | 171.50 | 5458.79 |
| **Level 2** | 15.92 | 128 | 18 | 15.45 | 31.49 | 202.99 | 3653.85 |
| **Level 1** | 9 | 128 | 0 | 15.45 | 17.80 | 220.79 | 0.00 |

**TABLE 5.10 – Story Wind Forces (North/South Direction)**

| Level | Height | Face Length | Elevation | Pressure | Story Force | Story Shear | Moment |
|---|---|---|---|---|---|---|---|
| | (Ft.) | (Ft.) | (Ft.) | (psf) | (K) | (K) | (Ft-K) |
| **Turret** | 8.13 | 22.67 | 103.33 | 20.47 | 3.77 | 3.77 | 389.84 |
| **Roof** | 15 | 228 | 87.07 | 19.73 | 67.46 | 71.23 | 6202.36 |
| **Level 6** | 13.79 | 228 | 73.32 | 19.16 | 60.25 | 131.49 | 9640.52 |
| **Level 5** | 13.83 | 228 | 59.49 | 18.51 | 58.35 | 189.84 | 11293.59 |
| **Level 4** | 13.83 | 228 | 45.66 | 17.72 | 55.86 | 245.70 | 11218.65 |
| **Level 3** | 13.83 | 228 | 31.83 | 16.70 | 52.67 | 298.37 | 9497.15 |
| **Level 2** | 15.92 | 228 | 18 | 15.26 | 55.38 | 353.75 | 6367.44 |
| **Level 1** | 9 | 228 | 0 | 15.45 | 31.70 | 385.45 | 0.00 |

**TABLE 5.11 – Base Shears and Overturning Moments**

| | E/W | N/S |
|---|---|---|
| Wind Base Shear (K) | **220.79** | **385.45** |
| Overturning Moment (Ft-K) | **31787.20** | **54609.55** |

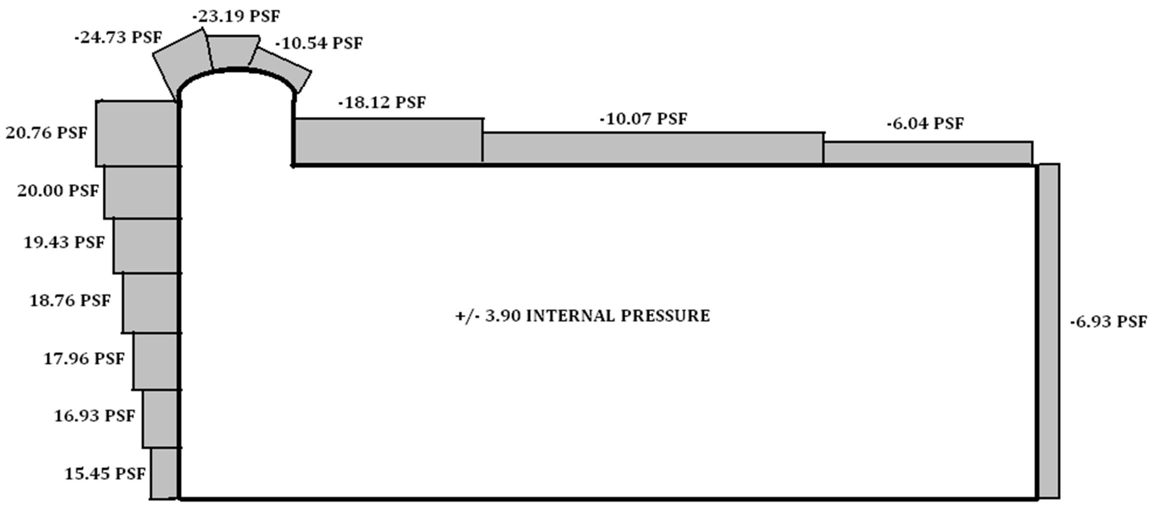



*Figure 20: EW Wind Pressure Elevation View*

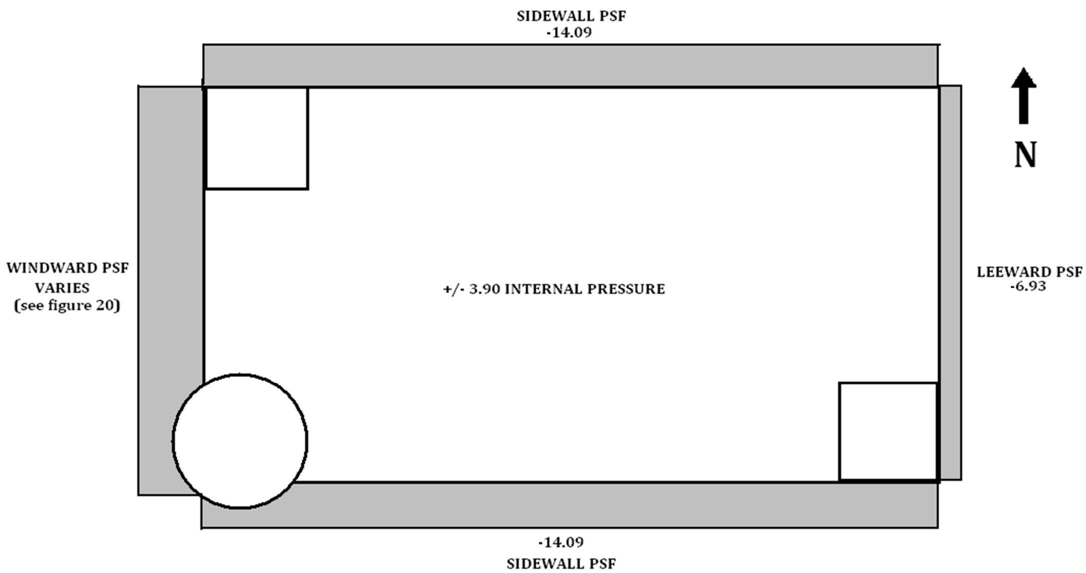



*Figure 21: EW Wind Pressure Plan View*

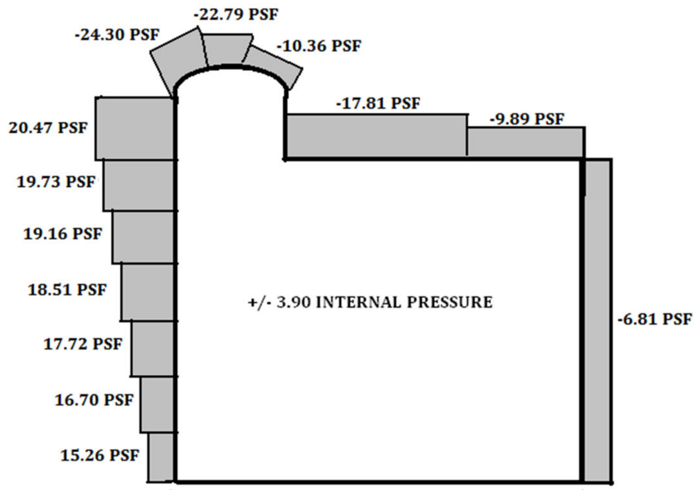



*Figure 22: NS Wind Pressure Elevation View*

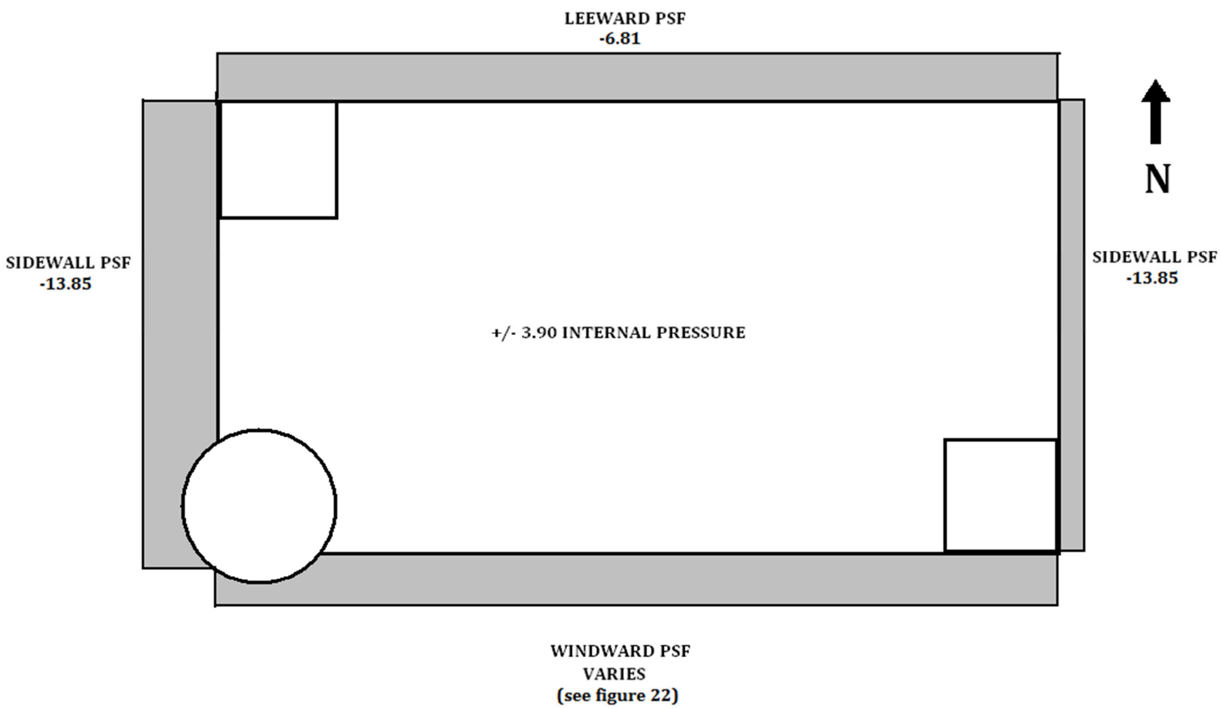



*Figure 23: NS Wind Pressure Plan View*

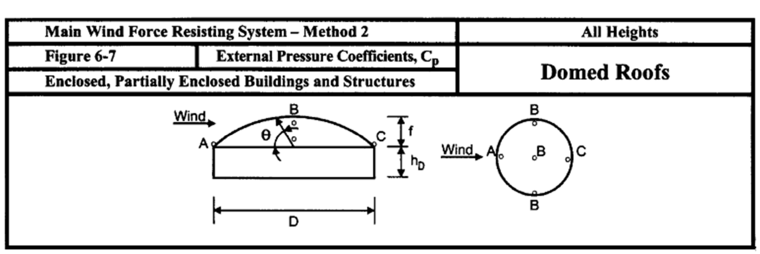



*Figure 24: ASCE 7-05 Domed Roof Excerpt*

## Seismic Loads

The seismic loads for the North Shore Equitable Building were calculated using ASCE 7-05's equivalent lateral force procedure. For the effective seismic weight, the first floor steel framing weight (excluding the turret framing) was calculated and found to be 10.26 psf. This calculation can be seen in table C.1. This value was rounded to 10.5 to account for the turret and to be conservative. For the upper levels, a steel framing unit weight of 10 psf was assumed (since the upper floor framing is somewhat lighter than the first floor). For simplicity, stairwell weights were excluded from the calculation, since assuming a continuous slab with no openings across the entire plan results in a heavier weight and thus is conservative. Below are the results of the seismic analysis.

**TABLE 5.12 - Story Seismic Forces**

| Level | Story Weight | Story Height | | | Story Force | Story Shear |
|---|---|---|---|---|---|---|
| | $w_x$ (K) | $h_x$ (Ft.) | $w_x h_x^k$ | $C_{vx}$ | $F_x$ (K) | $V_x$ (K) |
| **Level 1** | 2857.79 | 0.00 | 0.00 | 0.000 | 0.00 | 261.60 |
| **Level 2** | 2681.15 | 18.00 | 128939.59 | 0.049 | 12.76 | 261.60 |
| **level 3** | 2681.15 | 31.83 | 276772.04 | 0.105 | 27.39 | 248.84 |
| **Level 4** | 2681.15 | 45.66 | 448847.93 | 0.170 | 44.42 | 221.45 |
| **Level 5** | 2681.15 | 59.49 | 639846.84 | 0.242 | 63.32 | 177.03 |
| **Level 6** | 2678.30 | 73.32 | 845779.81 | 0.320 | 83.70 | 113.72 |
| **Roof** | 583.68 | 87.07 | 232059.13 | 0.088 | 22.96 | 30.02 |
| **Upper Roof** | 142.54 | 103.33 | 71285.33 | 0.027 | 7.05 | 7.05 |

**TABLE 5.13 - Seismic Design Criteria**

| Site Class: D | $S_s$=0.15 | $S_1$=0.04 | $F_a$=1.6 | $F_v$=2.4 | $C_t$=0.028 | X=0.8 |
|---|---|---|---|---|---|---|
| | $T_a$=1.188s | $T_o$=0.08 | $T_s$=0.4 | $T_l$=12 | R=3.5 | Cs=0.0154 |

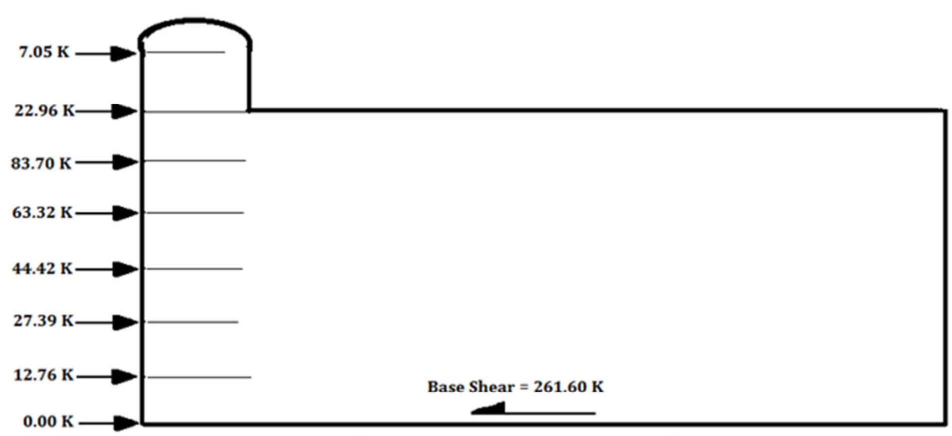



*Figure 25: Seismic Story Forces*

## 6. GRAVITY MEMBER SPOT CHECKS

### Typical Beam

W18x40's are the most common beam used in the framing of the North Shore Equitable Building, being used at over 170 locations throughout the building at a length of 42 feet and a span of 10 feet. As a result, a W18x40 was chosen for the typical beam spot check. The results of this check (on page 36) show that a W18x40 is just barely able to carry the applied loads. The deflection test fails by 0.11 inches however. The designer may have chosen to let this slide in order to maintain some degree of uniformity among the beam sizes.

### Typical Girder

Unlike the beams, the sizes for interior girders vary greatly throughout the building. W27x146's, appear at four locations on every level but are acting as part of the moment frames of the building so, a W24x62 was chosen for analysis since they act as gravity members only. The W24x62 girders are found on grid line C at two locations on all levels except level 6. They span 30 feet and carry point loads from the beams framing into them. The results of this check (on page 37) show that the girder both carries the applied loads and meets the deflection criteria.

### Typical Column

The column sizes, like the girder sizes, vary more than the beam sizes in this design. W14x311's appear at 8 locations on the first level, but are part of the moment frame system so, for this spot check, a W14x211 column on the first level at point 2D will be analyzed. The results of this check (on page 38) show that this column exhibits inelastic behavior and can carry the axial load both from a yielding and buckling standpoint. This analysis also shows that the column will buckle before it yields in the case of failure.

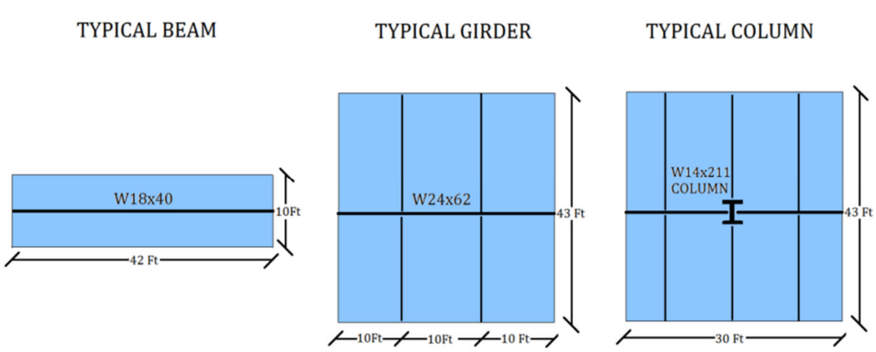
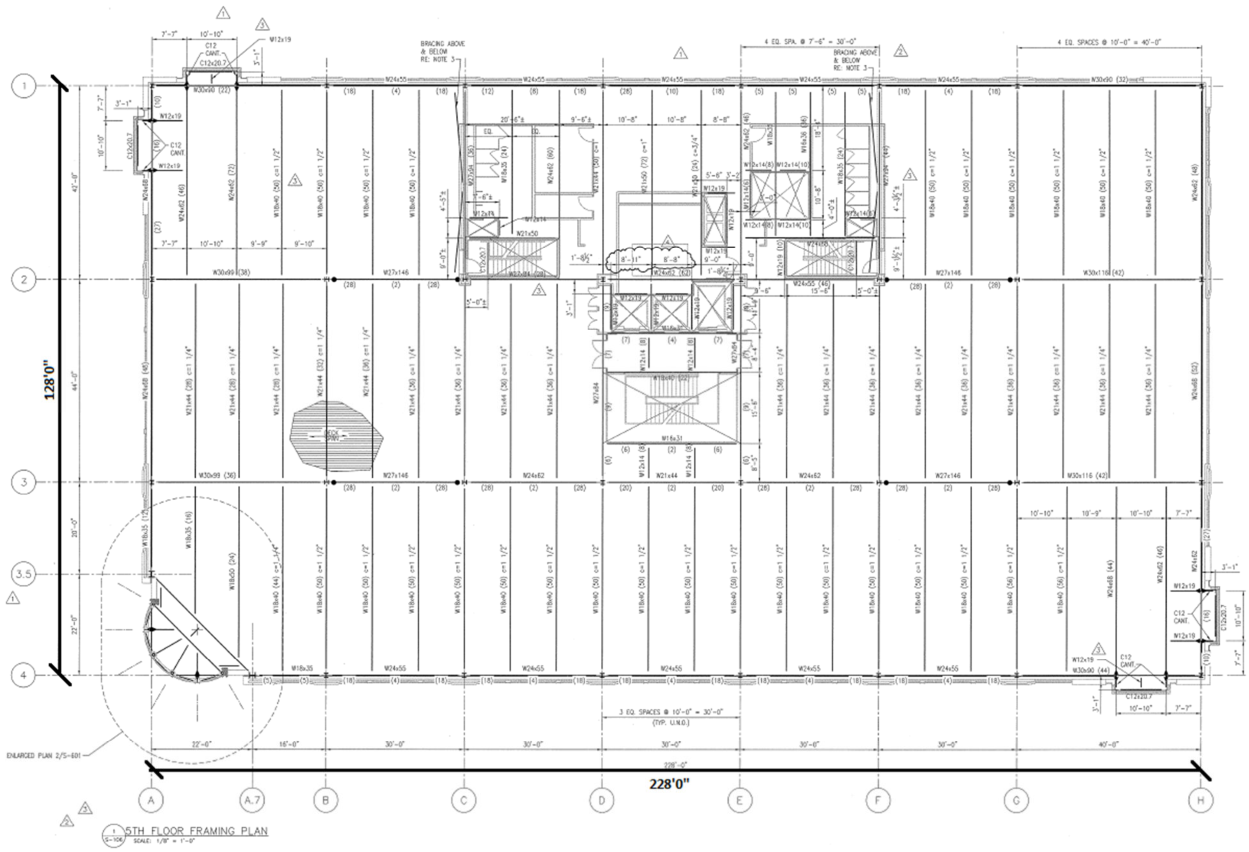
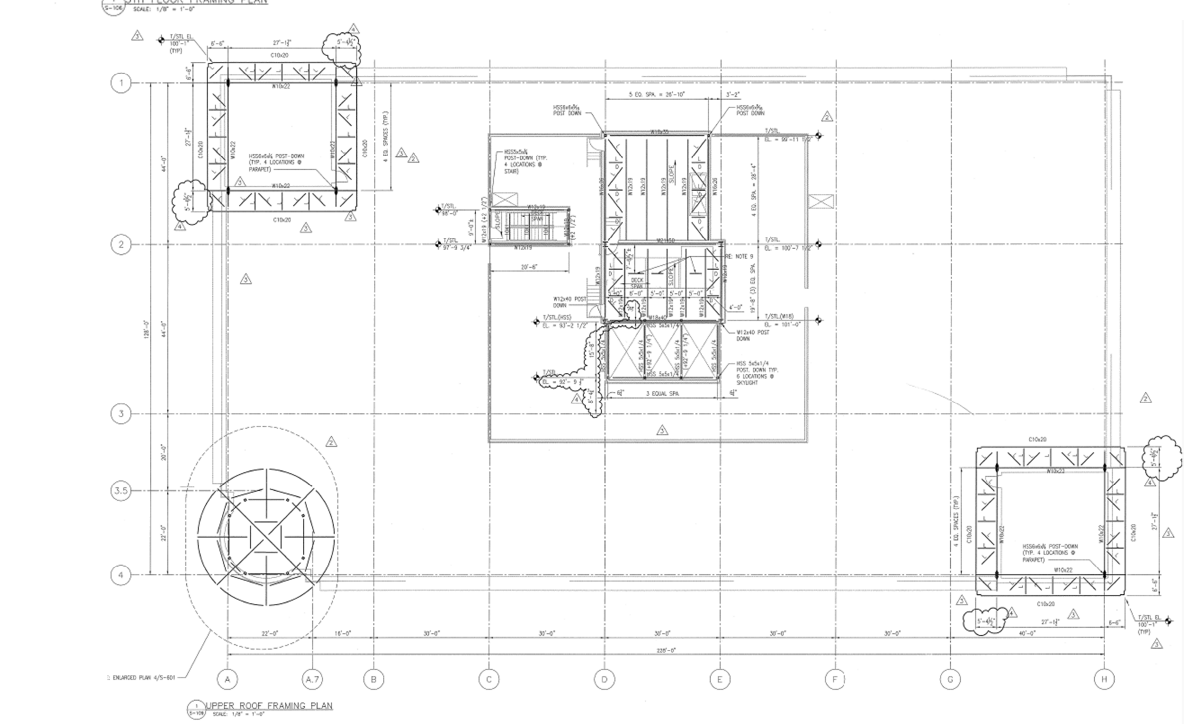


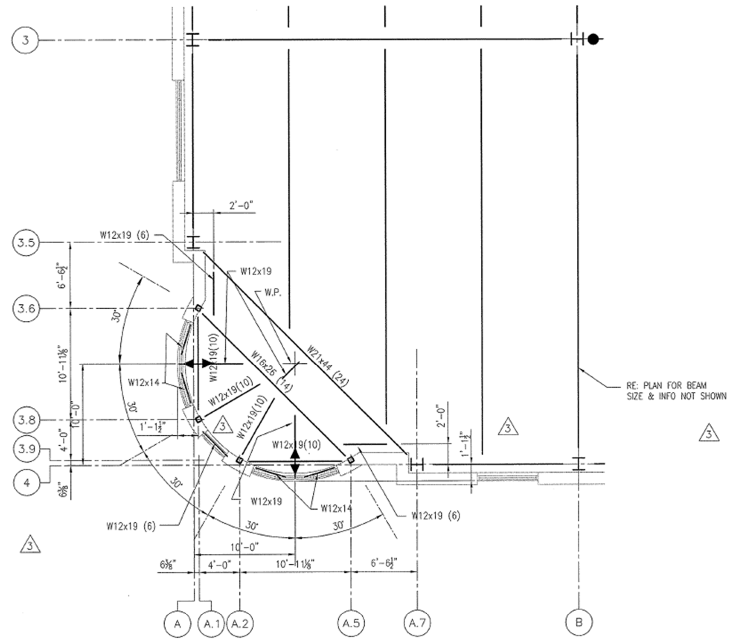

*Figure 26: Spot Check Diagrams*

# 7. APPENDICES

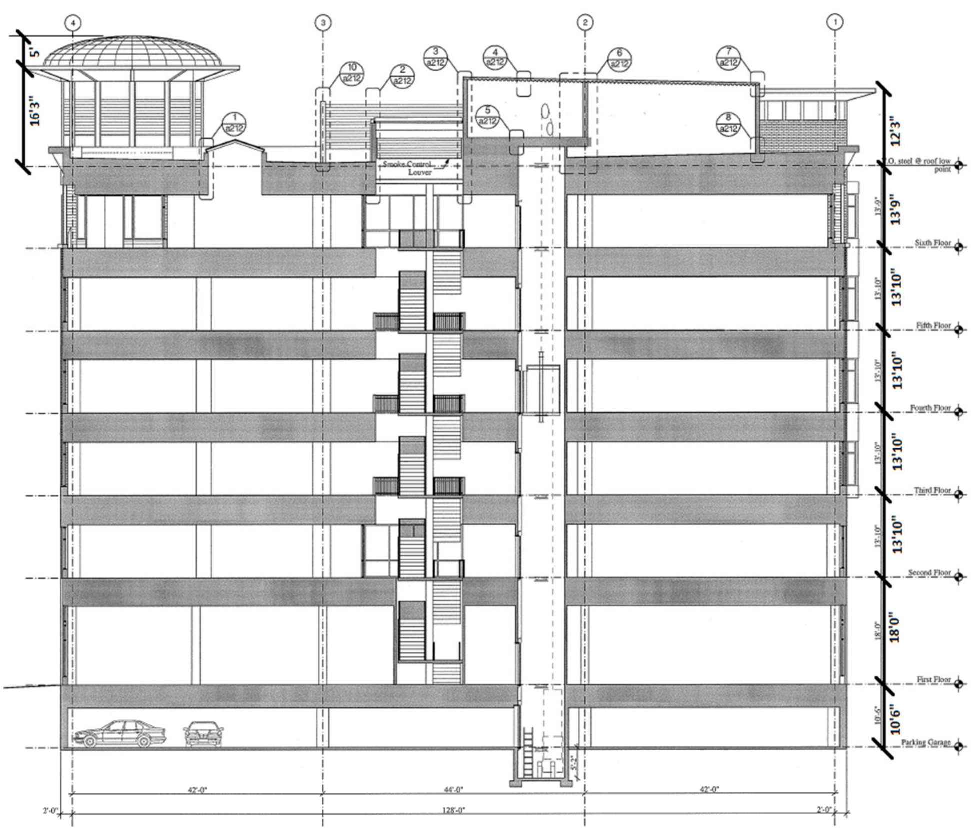
## APPENDIX A – PLANS & ELEVATIONS





PARTIAL 2ND, 3RD, 4TH, 5TH & 6TH FLOOR FRAMING PLAN
SCALE: 3/16" = 1'-0" (RE: S-103, S-104, S-105, S-106, S-107)



transverse building section
scale: 1/8" = 1'-0"

# APPENDIX B – WIND LOAD CALCULATIONS

**TABLE B.1 - Estimated Natural Frequency Check (E/W)**

| | | |
|---|---|---|
| **Effective Length (Ft.)** | 147.35 | |
| **26.9.2.1 Req't #1** | 87.08 < 300? | YES, OK |
| **26.9.2.1 Req't #2** | 87.08< 4*147.35? | YES, OK |
| **Moment Resisting Frame** | $n_a$ = .623 < 1 | Flexible Structure |
| **Steel Braced Frame** | $n_a$ = .861 < 1 | Flexible Structure |

* take $n_a$ as 0.861 to be conservative

**TABLE B.2 - Flexible Gust Effect Factor Calculation**

| Variable | East/West | North/South |
|---|---|---|
| $n_a$ | .861 | .861 |
| $g_q$ | 3.4 | 3.4 |
| $g_v$ | 3.4 | 3.4 |
| $g_r$ | 4.154 | 4.154 |
| $I_z$ | .1853 | .1853 |
| Q | .861 | .832 |
| R | .0322 | .0249 |
| $G_f$ | .929 | .913 |

**TABLE B.3 - Wind Force Variables**

| Variable | Symbol | E/W Value | N/S Value |
|---|---|---|---|
| Directionality Factor | $K_d$ | 0.85 | 0.85 |
| | $K_h$ | 1.23 | 1.23 |
| | α | 9.5 | 9.5 |
| | $Z_g$ | 900 | 900 |
| Topographic Factor | $K_{zt}$ | 1.0 | 1.0 |
| Flexible Gust Effect Factor | $G_f$ | .929 | .913 |
| Internal Pressure Coefficient | $GC_{pi}$ | +/- 0.18 | +/- 0.18 |
| Windward Wall Coefficient | $C_p$ | 0.8 | 0.8 |
| Leeward Wall Coefficient | $C_p$ | -0.34 | -0.5 |
| Side Wall Coefficient | $C_p$ | -0.7 | -0.7 |
| Roof Coefficient (0 to 87.08) | $C_p$ | -0.9 | -0.9 |
| Roof Coefficient (87.08 to 174.16) | $C_p$ | -0.5 | -0.5 |
| Roof Coefficient (174.16 to 228) | $C_p$ | -0.3 | -0.3 |
| Roof Coefficient Pt. A | $C_{pa}$ | -1.173 | -1.173 |
| Roof Coefficient Pt. B | $C_{pb}$ | -1.1 | -1.1 |
| Roof Coefficient Pt. C | $C_{pc}$ | -0.5 | -0.5 |

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ASCE7-05 chapter 6.5 - Method 2 - MWFRS Analytical procedure

Direction: East/west direction (controls)

Basic wind speed: 90 mph (Fig 6-1)

Building classification: II (Table 1-1)

Importance factor: I = 1.0 (Table 6-1)

Exposure category: C

Directionality factor: $K_d = 0.85$

$h = 87.08$ ft ⇒ interpolate $K_h$

| height | $K_h$ |
|---|---|
| 80 | 1.21 |
| 87.08 | |
| 90 | 1.24 |

⇒ $\frac{90-80}{1.24-1.21} = \frac{87.08-80}{K_h-1.21}$

$333.33(K_h - 1.21) = 7.08$

$K_h = 1.23$

$K_z = 2.01(z/z_g)^{2/\alpha}$ ⇒ see attached spreadsheet for $K_z$ values

$\alpha = 9.5$ (table 2)

$z_g = 900$ (table 2)

Topographic factor: $K_{zt} = (1 + k_1 k_2 k_3)^2$

No ridges or escarpments are present on site ⇒ $K_{zt} = 1.0$

Gust effect factor:

Used ASCE 7-10 to estimate natural frequency

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = \frac{h_1L_1 + h_2L_2 + h_3L_3 + h_4L_4 + h_5L_5 + h_6L_6 + h_7L_7}{h_1 + h_2 + h_3 + h_4 + h_5 + h_6 + h_7}$$

$$L_{eff} = \frac{228(18 + 31.8 + 45.7 + 59.5 + 73.3 + 87.1) + (27.2)(99.3) + (22.67)(108.33)}{18 + 31.8 + 45.7 + 59.5 + 73.3 + 87.1 + 99.3 + 108.33}$$

$$L_{eff} = \frac{77068}{523.03} = 147.35 \text{ ft}$$

Sect. 26.9.2.1 ⇒ Bldg height = 87.08 < 300 ✓ ok

87.08 < 4(147.35) ✓ ok

∴ natural frequency can be approximated

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For steel moment resisting frame: $n_a = \frac{22.2}{(h^{0.8})} = \frac{22.2}{(87.08^{0.8})} = .623\ hz$

$n_a = .623 < 1 \Rightarrow$ Flexible structure

For steel braced frame: $n_a = 75/h = \frac{75}{87.08} = .861\ hz$

$n_a = .861 < 1 \Rightarrow$ Flexible structure

$$G_f = 0.925\left[\frac{1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right]$$

$g_Q = g_v = 3.4$ , $g_r = \sqrt{2 L_n (3600 n_1)} + \frac{0.577}{\sqrt{2 L_n (3600 n_1)}}$

Use $n_1 = .861\ hz$ to be conservative

$$g_r = \sqrt{2 L_n (3600(.861))} + \frac{0.577}{\sqrt{2 L_n (3600(.861))}} = 4.154$$

To calculate R: $z_{min} = 15$, $\alpha = 9.5$ , $\ell(ft) = 500$ , $\bar{\epsilon} = 0.2$

$\bar{z} = 0.6h = 0.6(87.08) = 52.25 > 15$ ✓ ok

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{52.25}{33}\right)^{0.2} = 548.13$$

$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$ where $\bar{b} = 0.65$, $\bar{\alpha} = 1/6.5$

$$\bar{V}_{\bar{z}} = 0.65\left(\frac{52.25}{33}\right)^{.154} (90)\left(\frac{88}{60}\right) = 92.09$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{.861(548.13)}{92.09} = 5.125$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(5.125)}{(1 + 10.3(5.125))^{5/3}} = .05$$

For $R_h \Rightarrow \eta = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(.861)(87.08)}{(92.09)} = 3.745$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) = \frac{1}{3.745} - \frac{1}{2(3.745)^2}(1 - e^{-2(3.745)})$$

$$R_h = \frac{1}{3.745} - \left(\frac{1}{28.05}\right)(.999) = .231$$

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For $R_B \Rightarrow \eta = \dfrac{4.6 n_1 E B}{\bar{V}_{\bar{z}}}$ where $E = 1$ $\quad B = 128$ ft

$$\eta = \frac{4.6(.861)(128)}{92.09} = 5.51 \qquad R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta})$$

$$R_B = \frac{1}{5.51} - \frac{1}{2(5.51)^2}(1 - e^{-2(5.51)}) = .165$$

For $R_L \Rightarrow \eta = \dfrac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \dfrac{15.4(.861)(228)}{92.09} = 32.83$

$$R_L = \frac{1}{32.83} - \frac{1}{2(32.83)^2}(1 - e^{-2(32.83)}) = 0.03$$

Resonant Response Factor: $R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53 + 0.47 R_L)}$

$$R = \sqrt{(\tfrac{1}{1})(.05)(.231)(.165)(.53 + 0.47(.03))} = 0.0322$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{.167} \quad \text{where } c = 0.2 \text{ (table 6-2)}$$

$$I_{\bar{z}} = .2\left(\frac{33}{52.25}\right)^{.167} = 0.1853$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{128 + 87.08}{548.13}\right)^{0.63}}}$$

$$Q = \sqrt{\frac{1}{1.349}} = .861$$

Flexible Gust Effect Factor: $G_f = 0.925\left(\dfrac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$

$$G_f = 0.925\left(\frac{1 + 1.7(.1853)\sqrt{(3.4)^2(.861)^2 + (4.154)^2(.0322)^2}}{1 + 1.7(3.4)(.1853)}\right)$$

$$\boxed{G_f = 0.929}$$

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Enclosure classification: Enclosed

Internal pressure coefficient: $GC_{pi} = \pm 0.18$

External pressure coefficients: $C_p$

walls and roof:

windward wall ⇒ $C_p = 0.8$ (use with $q_z$)

Leeward wall ⇒ $L/B = 228/128 = 1.78$

interpolate: $\frac{2-1}{-.3+.5} = \frac{1.78-1}{C_p+.5} = 5$ $C_p = -.344$

Side wall ⇒ $C_p = -0.7$ (use with $q_h$)

Roof ⇒ $h/L = \frac{87.08}{228} = .381 < 0.5$

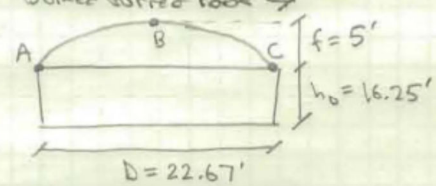
$\frac{h}{2} = \frac{87.08}{2} = 43.54'$ $2h = 174.16'$

from 0 to 87.08' from windward edge ⇒ $C_p = -0.9$

from 87.08' to 174.16' ⇒ $C_p = -0.5$

from 174.16' to 228' ⇒ $C_p = -0.3$

Domed turret roof ⇒




$\frac{h_D}{D} = \frac{16.25}{22.67} = .717$

$\frac{f}{D} = \frac{5}{22.67} = .221$

for $\frac{f}{D} = .221$ ⇒ $A(h_D/D = 0.25) = -0.55$

$A(h_D/D \geq 1) = -1.55$

interpolate: $\frac{.717-.25}{1-.25} = \frac{C_p+.55}{-1.55+.55} = .623 = \frac{C_p+.55}{-1}$

$C_{pA} = -1.173$

$B(h_D/D \geq 0.5) \Rightarrow -1.1$ $C_{pB} = -1.1$

$C(h_D/D \geq 0.5) \Rightarrow$ $C_{pC} = -0.5$

Velocity pressures: see addached spreadsheet

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

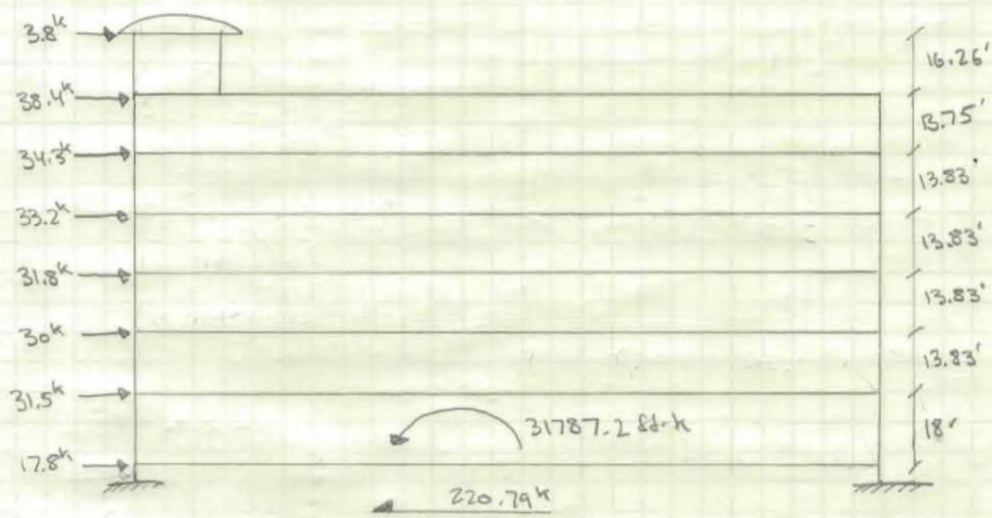
$K_{zt} = 1.0$, $K_d = 0.85$, $V = 90$, $I = 1.0$

Design wind loads:

For flexible buildings: $P = q G_f C_p - q_i (GC_{pi})$

See spreadsheet for design loads and pressures

Story forces diagram: (see spreadsheet for calcs)



Direction North/South

Gust Effect Factor:

$$L_{eff} = \frac{128(18+31.8+45.7+59.5+73.3+87.1)+27.2(99.3)+(22.67)(108.33)}{523.03}$$

$$L_{eff} = \frac{45528}{523.03} = 87.04 \text{ ft}$$

$87.08 < 4(87.04)$ ✓ ok ⇒ natural freq. can be approximated

$n_a = 0.861$

For $R_B$ ⇒ $\eta = \frac{4.6 n_1 EB}{\bar{V}_{\bar{z}}}$ where $E=1$ and $B = 228$ ft

$$\eta = \frac{4.6(.861)(228)}{92.09} = 9.806$$

$$R_B = \frac{1}{9.806} - \frac{1}{2(9.806)^2}\left(1-e^{-2(9.806)}\right) = 0.097$$

For $R_L$ ⇒ $\eta = \frac{15.4 n_1 L}{\bar{V}_z} = \frac{15.4(.861)(128)}{92.09} = 18.43$

$$R_L = \frac{1}{18.43} - \frac{1}{2(18.43)^2}\left(1-e^{-2(18.43)}\right) = .053$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53+0.47R_L)} = \sqrt{(\tfrac{1}{1})(.05)(.231)(.097)(.53+.47(053))}$$

$R = .0249$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{228+87.08}{548.13}\right)^{.63}}} = \sqrt{\frac{1}{1.444}}$$

$Q = 0.832$

Flexible Gust effect factor:

$$G_f = 0.925\left(\frac{1+1.7(.1853)\sqrt{(3.4)^2(.832)^2+(4.154)^2(.0249)^2}}{1+1.7(3.4)(.1853)}\right)$$

$G_f = 0.913$

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$GC_{pi} = \pm 0.18$

Cp values: windward wall: $C_p = 0.8$

leeward wall: $L/B = 128/228 = .5614 < 1$

$C_p = -0.5$

Side wall: $C_p = -0.7$

Roof: $h/L = 87.08/128 = .680 < 0.5$

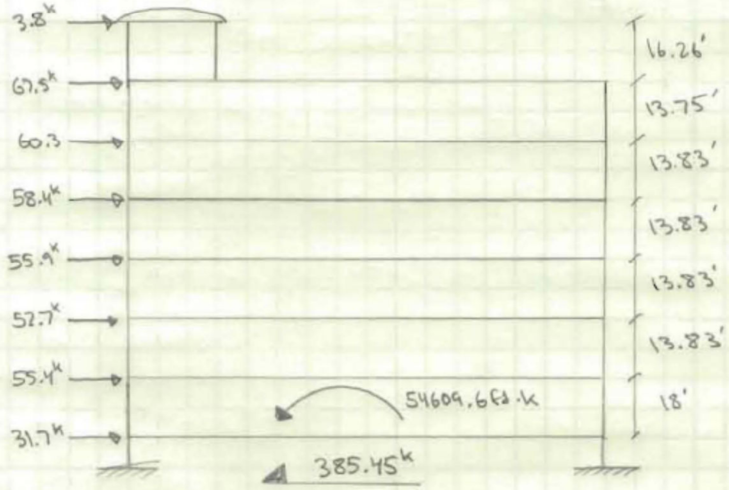
* Same as for E/W direction

Done Turned Roof

* Same as for E/W direction because of symmetry.

* See spreadsheet for story pressures

Story Forces Diagram: See spreadsheet for calcs



# APPENDIX C – SEISMIC LOAD CALCULATIONS

**TABLE C.1 - Level 1 Steel Framing Weight**

| **Beams** | | | | |
|---|---|---|---|---|
| **Designation** | **Unit Weight (lb/Ft.)** | **Quantity** | **Length (Ft.)** | **Total Weight (K)** |
| W18x40 | 40 | 36 | 42 | 60.48 |
| W27x94 | 94 | 2 | 42 | 7.90 |
| W24x62 | 62 | 3 | 42 | 7.81 |
| W24x55 | 55 | 2 | 42 | 4.62 |
| W24x76 | 76 | 2 | 42 | 6.38 |
| W18x35 | 35 | 2 | 42 | 2.94 |
| W18x35 | 35 | 1 | 15 | 0.53 |
| W21x44 | 44 | 15 | 44 | 29.04 |
| W27x94 | 94 | 2 | 44 | 8.27 |
| W30x99 | 99 | 2 | 44 | 8.71 |
| W24x68 | 68 | 3 | 32 | 6.53 |
| W24x55 | 55 | 6 | 7.5 | 2.48 |
| W12x19 | 19 | 4 | 12 | 0.91 |
| W12x19 | 19 | 2 | 9 | 0.34 |
| W27x94 | 94 | 1 | 30 | 2.82 |
| W30x99 | 99 | 2 | 38 | 7.52 |
| W27x146 | 146 | 4 | 30 | 17.52 |
| W27x84 | 84 | 2 | 30 | 5.04 |
| W24x62 | 62 | 2 | 30 | 3.72 |
| W21x44 | 44 | 1 | 30 | 1.32 |
| W30x90 | 90 | 1 | 30 | 2.70 |
| W30x116 | 116 | 2 | 40 | 9.28 |
| | | **Total Beam Weight =** | | **196.86** |
| **Columns** | | | | |
| **Type** | **Unit Weight (lb/Ft)** | **Quantity** | **Height (Ft.)** | **Total Weight (K)** |
| W14x120 | 120 | 4 | 18 | 8.64 |
| W14x132 | 132 | 4 | 18 | 9.50 |
| W14x145 | 145 | 5 | 18 | 13.05 |
| W14x99 | 99 | 6 | 18 | 10.69 |
| W14x159 | 159 | 2 | 18 | 5.72 |
| W14x311 | 311 | 8 | 18 | 44.78 |
| W14x211 | 211 | 2 | 18 | 7.60 |
| W14x68 | 68 | 2 | 18 | 2.45 |
| | | **Total Column Weight =** | | **102.44** |
| | | | | |
| | | **Total Framing Weight =** | | **299.3** |
| | | **Floor Square Footage =** | | **29184** |
| | | **Framing Unit Weight (psf)** | | **10.26** |

**TABLE C.2 - Estimated Building Weight**

| Level | Load Type | Design psf | Area ($Ft^2$) | Weight (K) |
|---|---|---|---|---|
| **Level 1** | 5 1/2" concrete slab | 45 | 29184 | 1313.28 |
| | Steel framing | 10.5 | 29184 | 306.43 |
| | Ceiling, Misc. | 5 | 29184 | 145.92 |
| | MEP | 5 | 29184 | 145.92 |
| | Exterior wall | 50 | 13088 | 654.40 |
| | partitions | 10 | 29184 | 291.84 |
| | | **Total floor weight =** | | **2857.792** |
| **Level 2-5** | 5 1/2" concrete slab | 45 | 29184 | 1313.28 |
| | Steel framing | 10 | 29184 | 291.84 |
| | Ceiling, Misc. | 5 | 29184 | 145.92 |
| | MEP | 5 | 29184 | 145.92 |
| | Exterior wall | 50 | 9847 | 492.35 |
| | partitions | 10 | 29184 | 291.84 |
| | | **Total floor weight =** | | **2681.15** |
| **Level 6** | 5 1/2" concrete slab | 45 | 29184 | 1313.28 |
| | Steel framing | 10 | 29184 | 291.84 |
| | Ceiling, Misc. | 5 | 29184 | 145.92 |
| | MEP | 5 | 29184 | 145.92 |
| | Exterior wall | 50 | 9790 | 489.50 |
| | partitions | 10 | 29184 | 291.84 |
| | | **Total floor weight =** | | **2678.30** |
| **Roof** | Superstructure Weight | 5 | 29184 | 145.92 |
| | Roofing, Ceiling, Misc. | 8 | 29184 | 233.47 |
| | Collateral Load (MEP) | 7 | 29184 | 204.29 |
| | | **Total roof weight =** | | **583.68** |
| **Upper Roof** | Turret framing | 10 | 381 | 3.81 |
| | Turret exterior wall | 50 | 1124 | 56.20 |
| | Tower Framing | 10 | 1513 | 15.13 |
| | Tower Exterior Wall | 50 | 1348 | 67.40 |
| | | **Total upper roof weight =** | | **142.54** |
| **TOTAL BUILDING WEIGHT** | | | | **16986.91** |

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ASCE7-05 - chapters 11 and 12

$S_s = 0.15$ Site class: D $F_a = 1.6$
$S_1 = 0.04$ $F_v = 2.4$

$S_{MS} = F_a S_s = 0.24$ $S_{M1} = S_1 F_v = .096$

Design spectral acceleration parameters:

$S_{DS} = \frac{2}{3} S_{MS} = 0.16$ $S_{D1} = 2/3\, S_{M1} = 0.064$

Determination of the period, T:

Lateral Force Resisting System: Ordinary steel moment frames
↳ per Table 12.8-2 ⇒ $C_t = 0.028$, $x = 0.8$
$h_n = 108.33$ ft

$T_a = C_t h_n^x = 0.028(108.33^{.8}) = 1.188$ s

Design spectral response acceleration:

$T_0 = 0.2 \frac{S_{D1}}{S_{DS}} = 0.2\left(\frac{.064}{0.16}\right) = .08$

$T_S = \frac{S_{D1}}{S_{DS}} = \frac{.064}{.16} = .4$ $T_L = 12$ (fig 22.15)

$T_S < T < T_L \Rightarrow S_a = \frac{S_{D1}}{T} = \frac{0.064}{1.188} = 0.054$

Importance factor: $I = 1.0$

Seismic Design category: A

$F_x = 0.01 w_x$ $R = 3.5$ (ordinary steel moment frame)

12.8 - Equivalent lateral Force Procedure

Building weight estimate: See spreadsheets

* round steel framing on first floor up from 10.25 to 10.5 psf to include turret framing. Estimate framing at all other levels to be 10 psf.

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Seismic Base shear:

$V = C_s W \Rightarrow W = 16987^k$

$C_s = \frac{S_{DS}}{(R/I)} \leq \frac{S_{D1}}{(TR/I)}$ for $T < T_L$

$= \frac{0.16}{(3.5/1)} = .0457$ $\qquad \frac{S_{D1} I}{TR} = \frac{.064(1)}{1.188(3.5)} = .0154$

$.0457 > .0154$ so take $\boxed{C_s = .0154}$

$.0154 > .01$ ✓ ok

$V = C_s W = .0154(16987) = 261.6^k$

Find k through linear interpolation: $\frac{2.5-.5}{2-1} = \frac{1.188-.5}{k-1}$

$k = 1.34$

$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ $\qquad F_x = C_{vx} V$

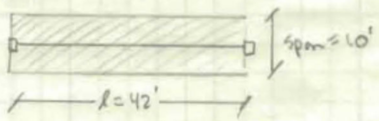
# APPENDIX D – SPOT CHECK CALCULATIONS

Typical Beam spot check | Tech Report #1 | Page 1 of 1

Typical beam (LRFD)

W18x40 (A992 steel)

$w_D = 55 + \text{selfweight} = 59$
$w_L = 50$ psf




$\text{LL reduction} = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$

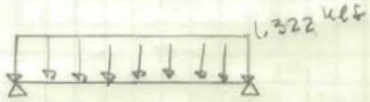
$A_T = 10(42) = 420\ ft^2$ $\quad K_{LL} = 2$ (interior beams)

$L = 50\left(0.25 + \frac{15}{\sqrt{2(420)}}\right) = 50(.768) = 38.377$

$\sqrt{2(420)} < 400 \Rightarrow$ reduction is ok

$w_u = 1.2w_D + 1.6w_L = 1.2(59) + 1.6(38.38) = 132.21$ psf

$132.21\,(10\ ft) = 1322$ plf $= 1.32$ klf




$V_u = \frac{w_u L}{2} = \frac{1.322(42)}{2} = 27.76^k$

$M_u = \frac{w_u L^2}{8} = \frac{1.322(42)^2}{8} = 291.5$ ft-k

$Z_{req} = \frac{M_u}{\phi F_y} = \frac{291.51(12)}{.9(50)} = \boxed{77.73\ in^3}$

manual Table 3.2 $\Rightarrow$ $Z_x$ for a W18x40 beam = 78.4

$\boxed{78.4 > 77.73 \Rightarrow \text{Beam size is ok}}$

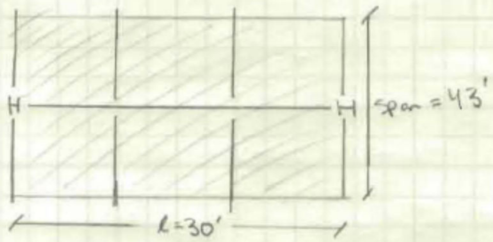
$\Delta_{LL} \le \frac{L}{360} = \frac{42(12)}{360} = 1.4$ in max

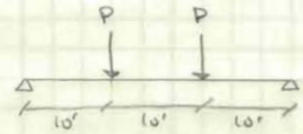
$\Delta_{LL} = \frac{5w_{LL}L^4}{384EI_x} = \frac{5(.384)(42)^4(1728\ in^3)}{384(29000)(612)} = \boxed{1.51\ in > 1.4 \Rightarrow \text{Not ok}}$

Typical Girder spot check | Tech Report #1 | Page 1 of 1

Typical girder (LRFD)

W24x62 A992




* To find $P_L$ must consider W18x40's and W21x44's framing into girder

W18x40 = $27.76^k$

For the W21x44: $w_L = 50$ psf, $w_D = 59.4$ psf

$A_{TT} = 44 \times 10 = 440$ psf

$L = 50\left(0.25 + \frac{15}{\sqrt{2(440)}}\right) = 37.78$

$w_u = 1.2(59.4) + 1.6(37.78) = 131.73$ psf

$\Rightarrow 131.73(10) = 1317.28$ plf $= 1.317$ klf

$V = \frac{w_u L}{2} = \frac{1.317(44)}{2} = 28.98^k$

$P = 27.76 + 28.98 = 56.74^k$ → $M_u = 56.74(10) = 567.4$ ft-k, $V_u = 56.74^k$

$Z_{req} = \frac{M_u}{\phi F_y} = \frac{567.4(12)}{0.9(50)} = 151.31\ in^3$

Manual table 3.2 ⇒ $Z_x$ for a W24x62 beam = 153 $in^3$

151.31 < 153 ⇒ Girder size is ok ✓

$\Delta_{LL} = \frac{L}{360} = \frac{30(12)}{360} = 1.00$ in

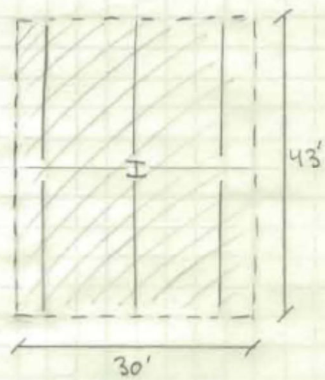
$P_{LL} = \frac{37.78(10)(44)}{2} + \frac{38.38(10)(42)}{2} = 8.31 + 8.06 = 16.37^k$

$\Delta P_{LL} = \frac{PL^3}{28EI} = \frac{16.37(30)^3(12)^3}{28(29000)(1550)} = 0.607$ in

0.607 in < 1.00 in ⇒ Deflection ok ✓

Typical column spot check | Tech Report #1 | Page 1 of 1

Analyze W14x211 at first level (A992 steel, LRFD)




$A_{TT} = 43(30) = 1290\ ft^2$

$k = 4$ (interior column)

$LL_{red} = 0.25 + \frac{15}{\sqrt{kA_{TT}}} = 0.25 + \frac{15}{\sqrt{(4)(5)(1290)}}$

$LL_{red} = 0.343 < 0.4 \Rightarrow$ use 0.4

Floor DL = 65 psf
Roof DL = 20 psf
Roof Snow = 21 psf
Floor LL = 100 psf (corridors)

$P_L = 0.4(100)(5)(1290) = 258^k$

$P_S = 21(1290) = 27.09^k$

$P_D = 20(1290) + 65(5)(1290) = 445.05^k$

$P_u = 1.2P_D + 1.6P_L + 0.5P_S = 1.2(445.05) + 1.6(258) + 0.5(27.09)$

$P_u = 960.405^k$

$A_s = 62\ in^2$
$I_x = 2660\ in^4$
$r_x = 6.55\ in$
$I_y = 1030\ in^4$
$r_y = 4.07$

Yielding: $P_y = F_y A_s = 50(62) = 3100^k$

$\phi P_y = 0.9(3100) = 2790$

| 960.4 < 2790 ✓ ok |

Buckling: y axis controls

$\frac{kL}{r_y} = \frac{(12)(18)}{4.07} = 53.07$

$4.71\sqrt{E/F_y} = 4.71\sqrt{\frac{29000}{50}} = 113.43 > 53.07 \Rightarrow$ inelastic buckling

$F_e = \frac{\pi^2 E}{(\frac{kL}{r})^2} = \frac{\pi^2(29000)}{(53.07)^2} = 101.62\ ksi$

$F_{cr} = \left[0.658^{F_y/F_e}\right]F_y = \left[0.658^{(50/101.62)}\right]50 = 40.69\ ksi$

$P_n = 40.69(62) = 2523.04^k \Rightarrow \phi P_n = 0.9(2523.04) = 2271^k$

| 960.405 < 2271 ✓ ok |