# NORTH SHORE EQUITABLE BUILDING

## PITTSBURGH, PA

STEPHAN NORTHROP - STRUCTURAL OPTION

## TECHNICAL REPORT #3

FACULTY CONSULTANT: DR LINDA HANAGAN

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## TABLE OF CONTENTS

# EXECUTIVE SUMMARY

In Technical Report 3, an in depth analysis was performed on the lateral system of the North Shore Equitable Building in Pittsburgh Pennsylvania. The structural system of the North Shore Equitable Building is a composite steel frame 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.

To begin the analysis, a 3D computer model of the building was prepared using ETABS. All 7 ASCE 7-05 load combinations and 4 ASCE 7-05 wind load cases were applied to the model. Stiffness values were found for both the moment frames and braced frames and a hand calculated center of rigidity was compared to the ETABS center of rigidity to confirm the accuracy of the model. A hand analysis was then performed to determine the controlling wind load case. Hand calculations were also performed to check overturning and member strengths.

After reevaluating the wind and seismic analyses performed in Tech 1, it was discovered that the wind story forces are actually lower than previously believed. These lateral wind forces still control however, having a slightly higher base shear than the seismic base shear. After running an analysis of the 3D ETABS model, it was found that ASCE 7-05 load combination 7 (represented as load combination 12 on page 21 of this report) controls the design. It was also determined that wind load case 4 is the controlling wind load case.

The 3D computer analysis, along with hand calculations, confirmed that building deflections meet industry standards and an appropriate load path exists for the distribution of calculated loads. It was also concluded through hand calculated strength checks that overturning will not be an issue and all lateral framing members are appropriately sized to carry the applied loads.

# 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 which offers access to riverside trails and beautiful views of the Pittsburgh skyline across the Allegheny River.

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. 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.

***Figure 1-1: View of the North Shore Equitable building from Mazeroski Way***

# 2. STRUCTURAL SYSTEMS OVERVIEW

The structural system of the North Shore Equitable Building consists of composite 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 5 ½" slab on grade supported by concrete grade beams and a combination of 18" auger cast piles and steel H-piles. Reinforced concrete retaining walls in the parking garage 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.

The piles for the Equitable Building pose a unique set of design requirements. The Allegheny Port Authority is currently extending their light rail transit system under the Allegheny River to Pittsburgh's North Shore. This extension consists of two parallel tunnels which are designed to pass directly below the Equitable Building as seen in Figure 2-1. As a result, the foundation is designed as a combination of two types of foundations; driven Steel H-piles (Figure 2-2 on the right) to withstand pressures and settlement resulting from tunneling under the building and 18" auger cast piles (Figure 2-2 on the left) for the remainder of the foundation.

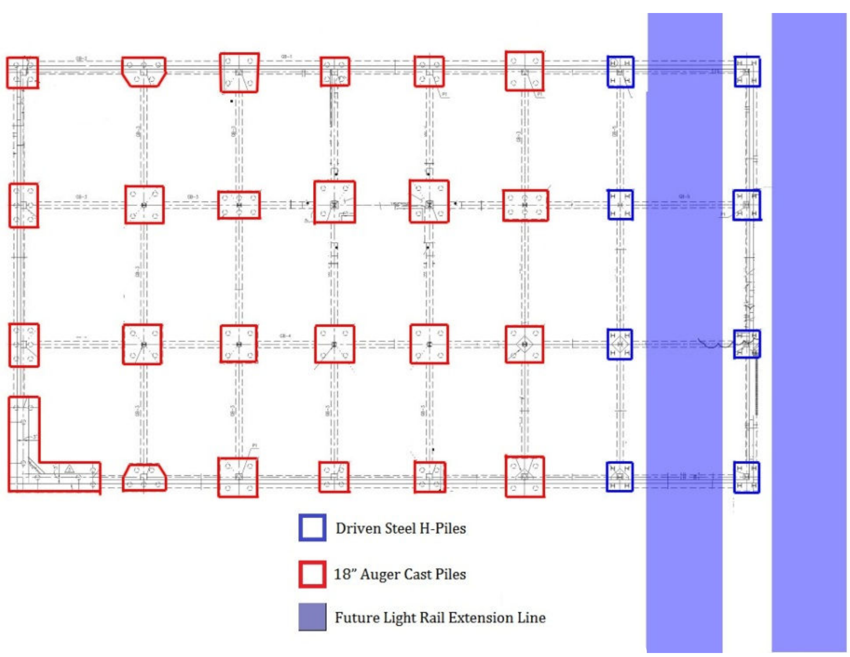



*Figure 2-1: Foundation plan with future transit line extension*

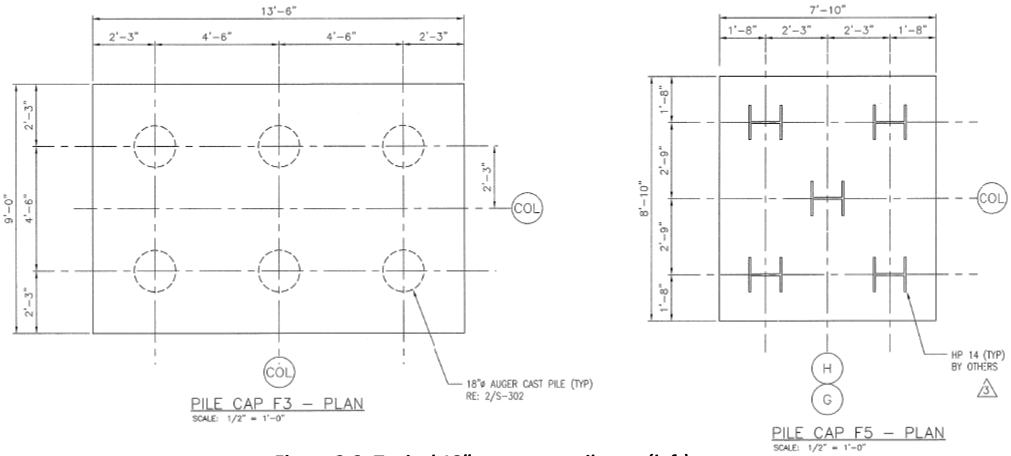



***Figure 2-2: Typical 18" auger cast pile cap (left) and typical steel H pile cap (right)***

## General Floor Framing

Due to the equitable building's rectangular shape, 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 a 2" 18 gage composite galvanized metal floor deck. 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. Figure 2-4 on the following page shows the typical floor plan for the existing structural system.

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 each column line at 4' above the floor level indicated. A typical column splice detail is shown to the right in Figure 2-3.

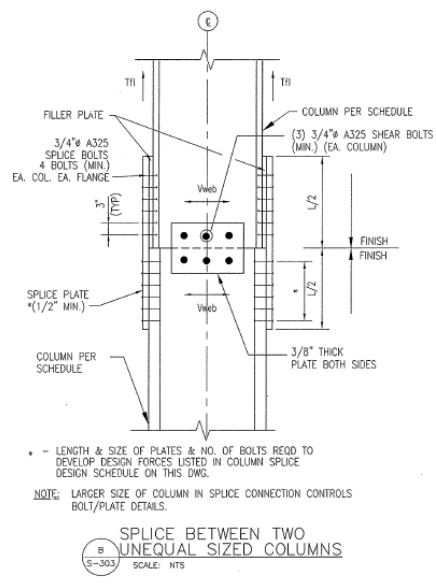



***Figure 2-3: Typical column splice detail***

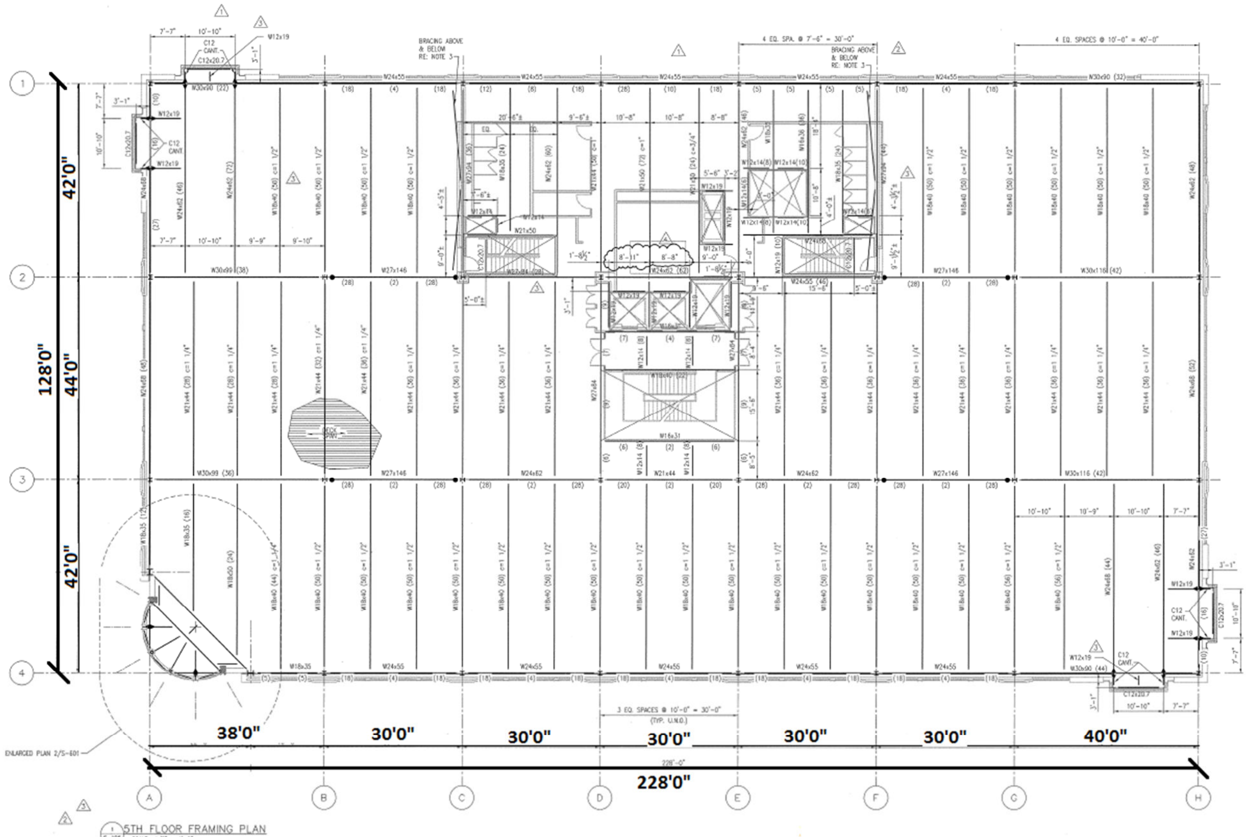



*Figure 2-4: Typical floor framing plan*

## Turret Framing Plan

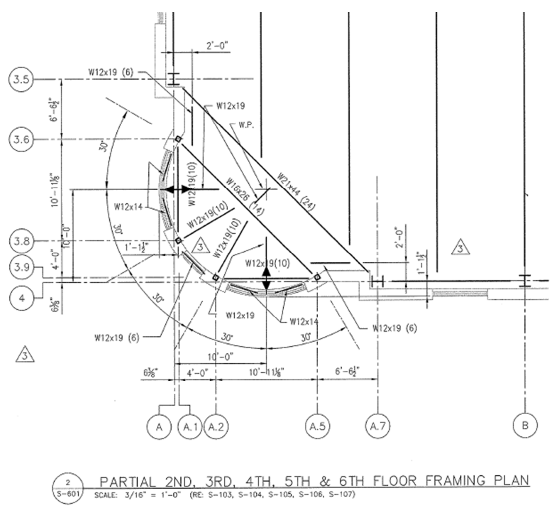



*Figure 2-5: Turret framing plan*

For the turret at the southwest corner of the building, members of varying sizes are used as seen to the right in Figure 2-5. 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.

## 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 2-6) 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.

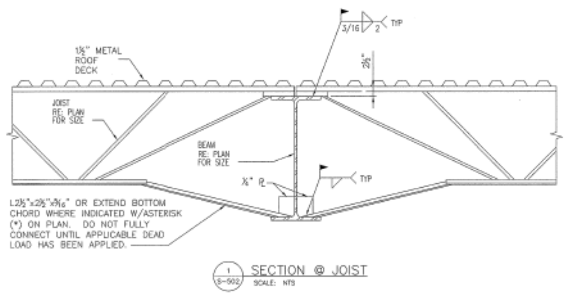



***Figure 2-6: Section at joist***

The girders in the roof plan vary greatly in both size and span length. 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.

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 2-7. The framing of the turret roof consists of curved C6x13 members and wide flange members of varying lengths as seen in Figure 2-8.

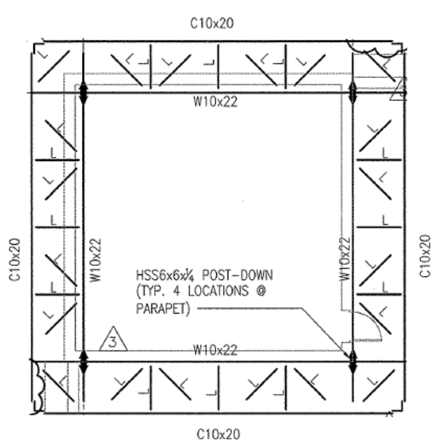



***Figure 2-7: Tower roof framing plan***

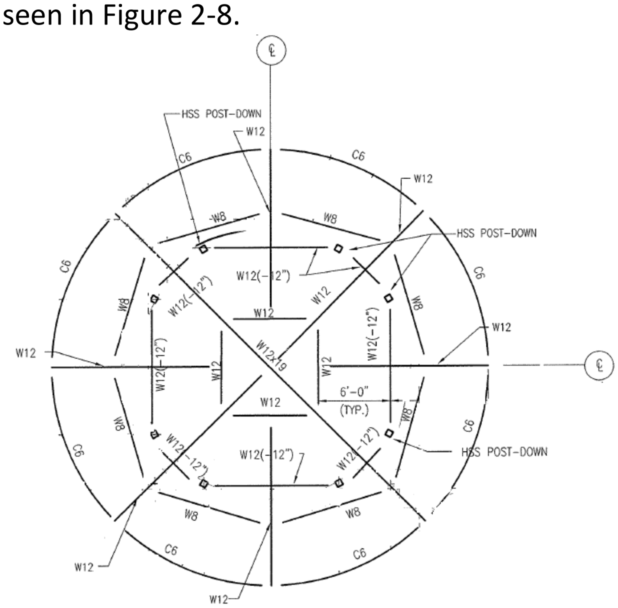



***Figure 2-8: Turret roof framing plan***

## 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 2-9 and 2-10 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 2-11 and 2-12. 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 2-13 and 2-14 respectively.

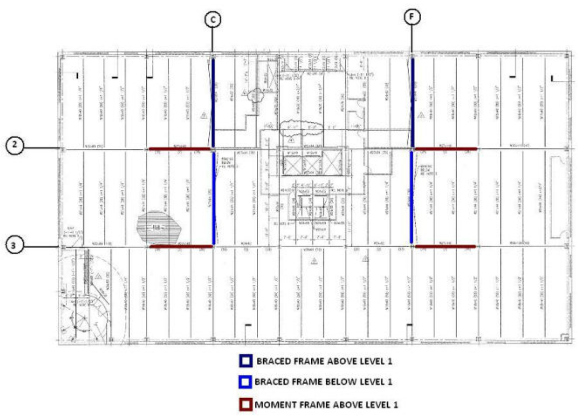



*Figure 2-9: Lateral Resisting elements at level 1*

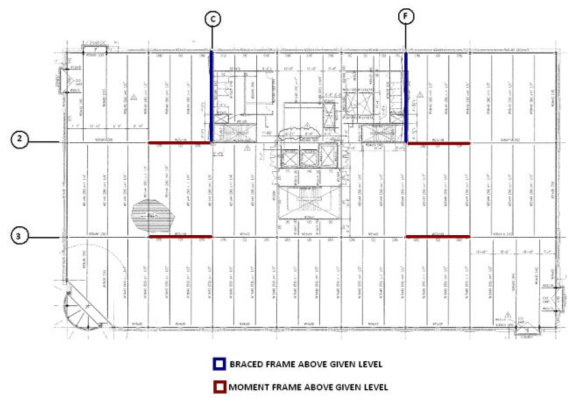



*Figure 2-10: Lateral Resisting elements at levels 2-6*

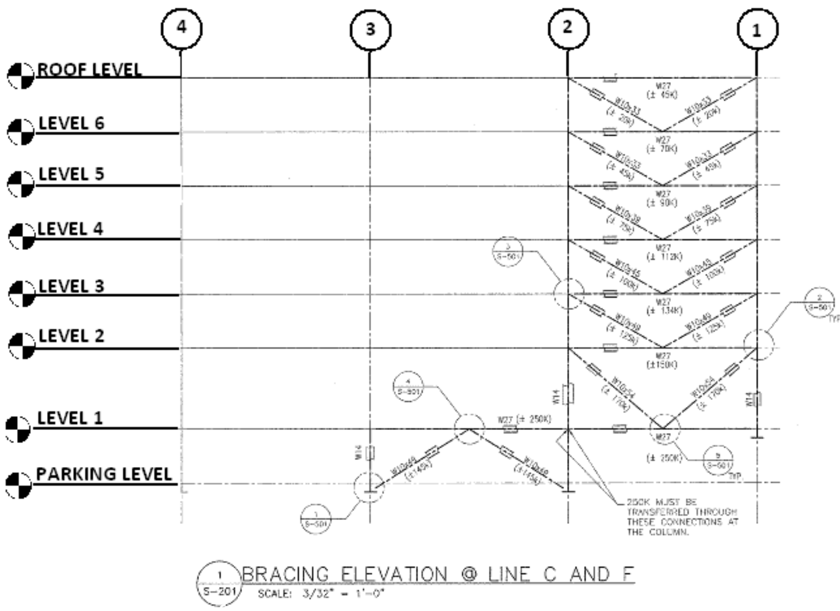



*Figure 2-11: Braced frame elevation*

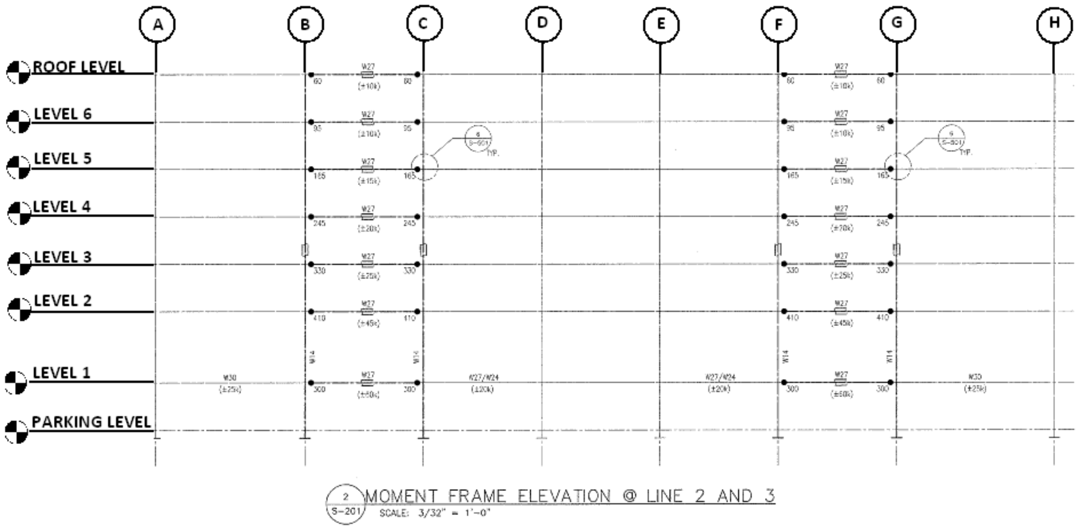



*Figure 2-12: Moment frame*

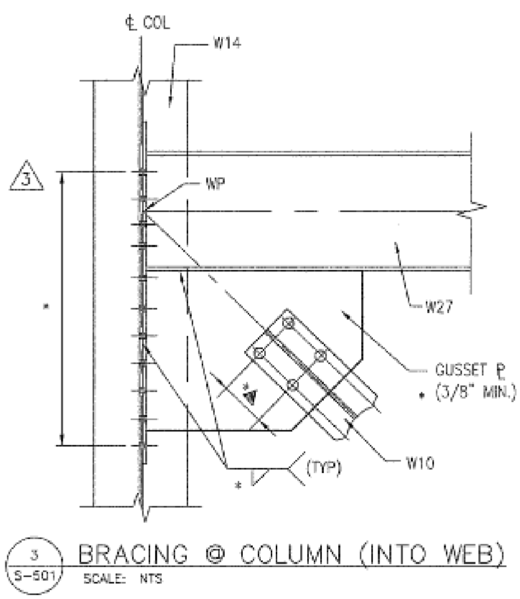



*Figure 2-13: Braced frame connection*

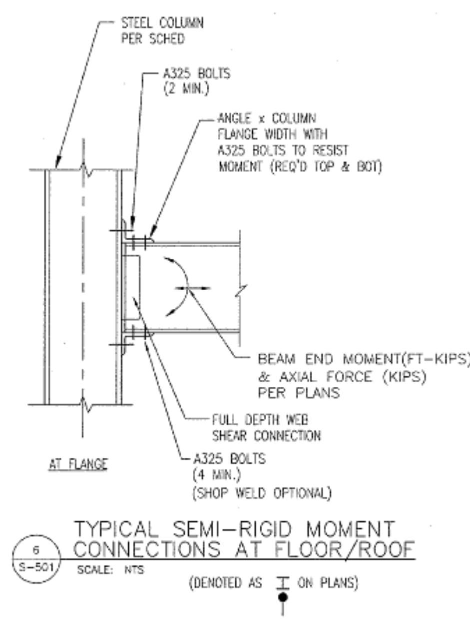



*Figure 2-14: Moment frame connection*

## 3. MATERIALS USED

Several different structural material types are used in the design of the North Shore Equitable building. Generally, standard material strengths are used throughout the building. Slabs, footings and grade beams all consist of normal weight concrete (with the exception of the elevated floor slabs). Steel is used for all framing and lateral members, with A992 steel being used for beams, girders and columns and A36 steel being used for all connecting elements (as is customary)

**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

Since the North Shore Equitable building was designed and built between 2003 and 2004, the codes used by the designers are a couple editions older than the codes used for this report. In addition the use of ASCE 7-05 in this report, the natural frequency of the building was approximated using ASCE 7-10 chapter 26. This was done due to the fact that ASCE 7-05 appears to offer no method of estimating the natural frequency. The codes used by the designers and in this report are given below.

## 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 318-95, Building Code Requirements for Reinforced Concrete
- ACI 530, Building Code Requirements for Masonry Structures
- AISC/ASD-89, Manual of Steel Construction, 9$^{th}$ Edition
- AISC/LRFD-2001, Manual of Steel Construction, 3$^{rd}$ Edition
- SJI-41$^{st}$ Edition, Standard Specifications and Load Tables for Steel Joists and Joist Girders

## Codes Used In Tech 2 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, 13$^{th}$ Edition
- ACI 318-08, Building Code Requirements for Reinforced Concrete

# 5. DESIGN LOADS

## Gravity Loads

For the design of this building, the structural engineers at Michael Baker chose to conservatively take the live load as 100 psf rather than the 50 psf recommended by ASCE 7-05. Having worked at Michael Baker as an intern this past summer, it is my understanding that the structural engineers use 100 psf live loads as a general rule of thumb when designing composite steel buildings. For the alternate system analyses in this report, an 80 psf live load is used rather than the ASCE prescribed 50 psf. This was done in an attempt to be conservative but also to try to avoid overdesigning the alternate systems.

**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 | 10 |
| 4" Brick | 40 |
| **Total Exterior Wall Load** | **50** |
| Stairs | 30 |
| Stair Landings | 40 |

## 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 ASCE 7-10 chapter 26.9, 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. The original calculations were performed for Tech Report 1 with some corrections being made for this report. These corrections resulted in smaller lateral forces than those given in Tech 1. Below are the results of the calculations. Detailed hand calculations can be found in Appendix A.

**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 |
| R value | 3.5 |

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

| Level | Height (Ft.) | $K_z$ | $q_z$ (psf) | Windward Pressure (psf) |
|---|---|---|---|---|
| Level 1 | 0.00 | 0.00 | 0.00 | 11.55 |
| Level 2 | 18.00 | 0.88 | 15.55 | 11.55 |
| Level 3 | 31.83 | 0.99 | 17.53 | 13.03 |
| Level 4 | 45.67 | 1.07 | 18.91 | 14.06 |
| Level 5 | 59.50 | 1.13 | 20.00 | 14.86 |
| Level 6 | 73.33 | 1.19 | 20.90 | 15.53 |
| Roof | 87.08 | 1.23 | 21.67 | 16.10 |
| Tower | 99.33 | 1.26 | 22.28 | 16.56 |
| Turret | 108.33 | 1.29 | 22.69 | 16.86 |

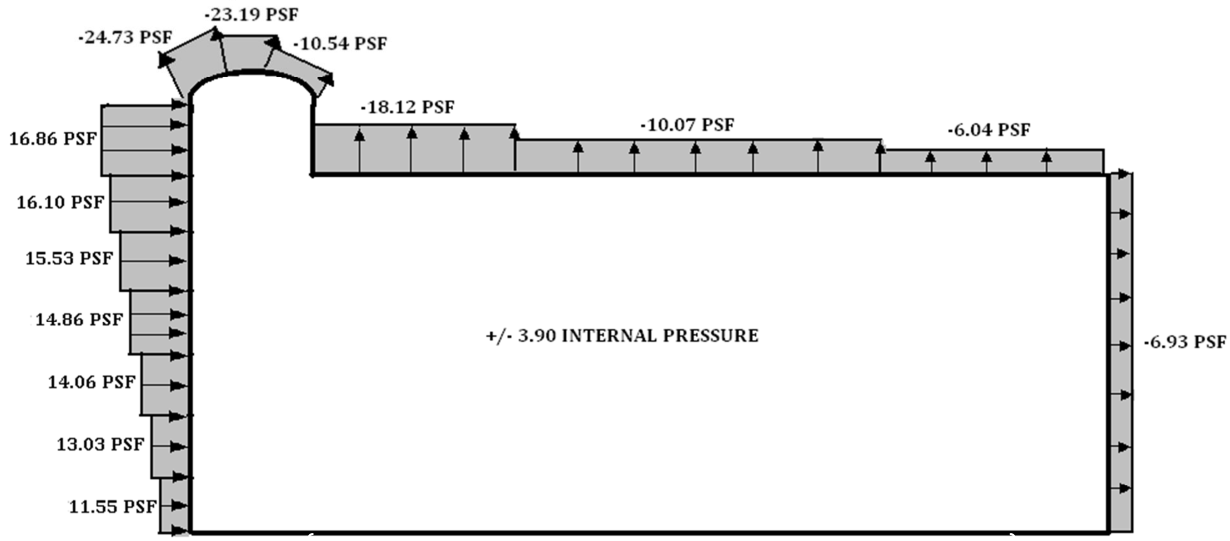



*Figure 5-1: East/West Wind Pressure Elevation View*

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

| Level | Height (Ft.) | $K_z$ | $q_z$ (psf) | Windward Pressure (psf) |
|---|---|---|---|---|
| **Level 1** | 0.00 | 0.00 | 0.00 | 11.36 |
| **Level 2** | 18.00 | 0.88 | 15.55 | 11.36 |
| **Level 3** | 31.83 | 0.99 | 17.53 | 12.80 |
| **Level 4** | 45.67 | 1.07 | 18.91 | 13.82 |
| **Level 5** | 59.50 | 1.13 | 20.00 | 14.61 |
| **Level 6** | 73.33 | 1.19 | 20.90 | 15.26 |
| **Roof** | 87.08 | 1.23 | 21.67 | 15.83 |
| **Tower** | 99.33 | 1.26 | 22.28 | 16.27 |
| **Turret** | 108.33 | 1.29 | 22.69 | 16.57 |

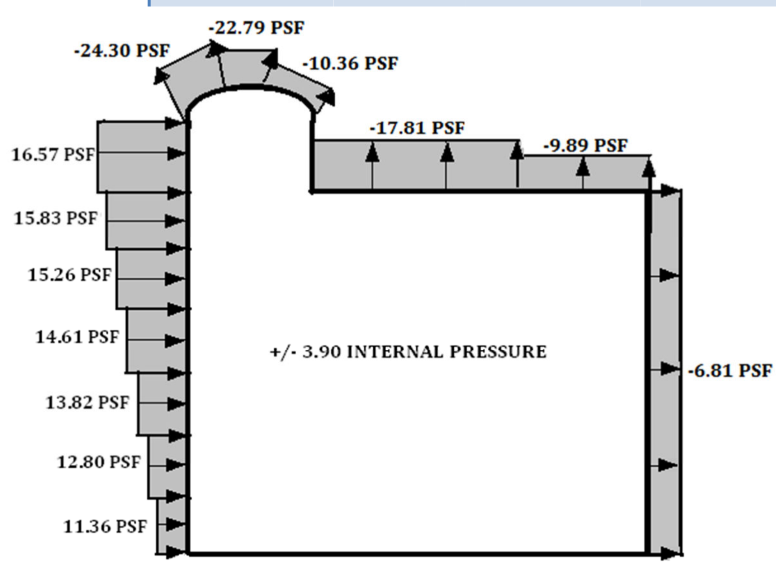



*Figure 5-2: North/South Wind Pressure Elevation View*

**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*

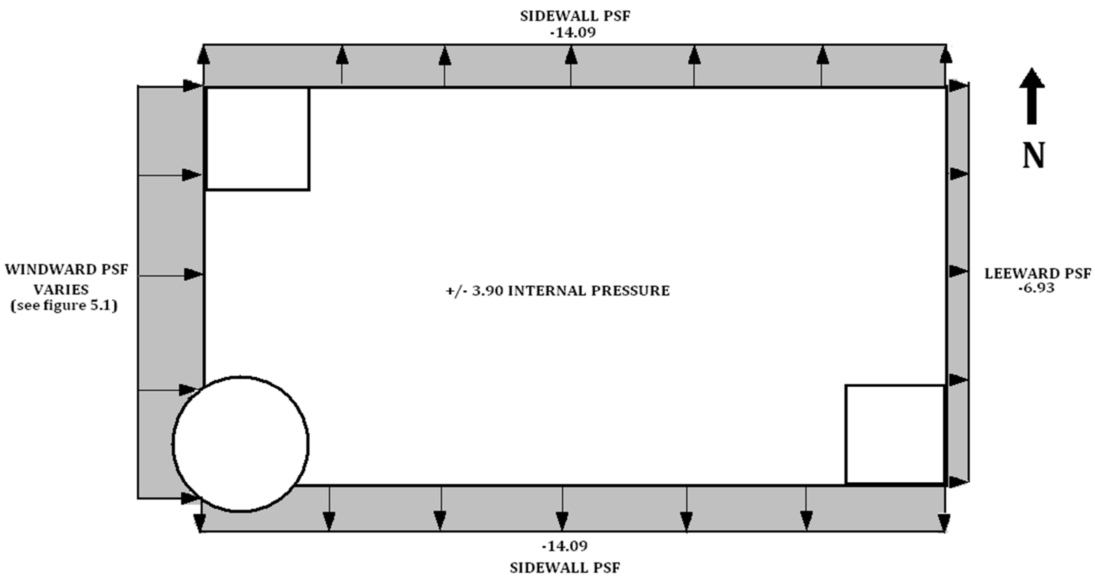



**TABLE 5.8 - Pressures Independent Of Height (North/South Direction)**

| | **q value** | **Cp value** | **G value** | **Pressure (psf)** |
|---|---|---|---|---|
| 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*

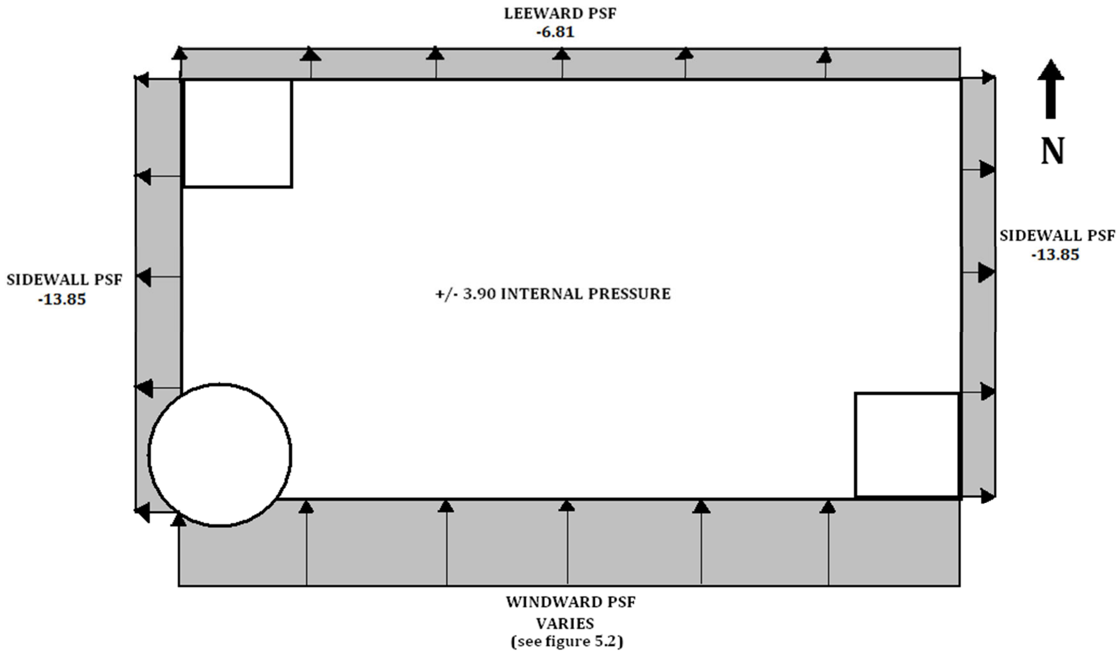



*Figure 5-4:North/South Wind Pressure Plan View*

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

| Level | Height (Ft.) | Face Length (Ft.) | Elevation (Ft.) | Pressure (psf) | Story Force (K) | Story Shear (K) |
|---|---|---|---|---|---|---|
| **Turret** | 8.13 | 22.67 | 103.33 | 16.86 | 3.11 | 3.11 |
| **Roof** | 15 | 128 | 87.07 | 16.10 | 14.18 | 17.28 |
| **Level 6** | 13.79 | 128 | 73.32 | 15.53 | 27.41 | 44.69 |
| **Level 5** | 13.83 | 128 | 59.49 | 14.86 | 26.31 | 71.00 |
| **Level 4** | 13.83 | 128 | 45.66 | 14.06 | 24.89 | 95.89 |
| **Level 3** | 13.83 | 128 | 31.83 | 13.03 | 23.07 | 118.96 |
| **Level 2** | 15.92 | 128 | 18 | 11.55 | 23.54 | 142.50 |
| **Level 1** | 9 | 128 | 0 | 11.55 | 13.31 | 155.81 |

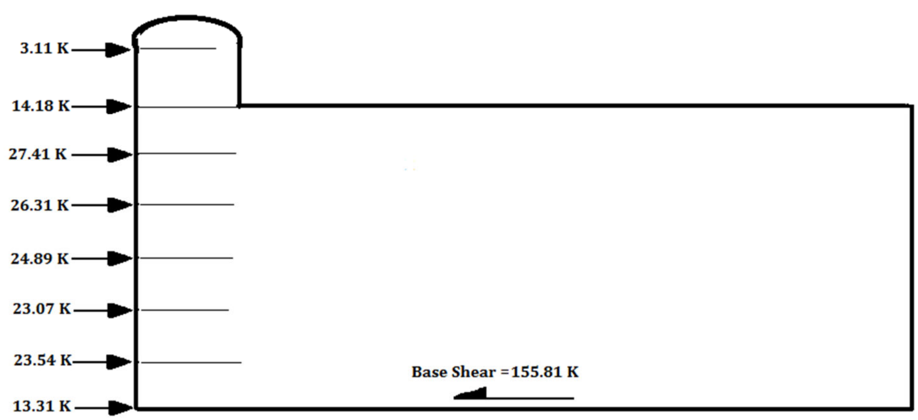



*Figure 5-5: East/West Wind Story Forces*

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

| Level | Height (Ft.) | Face Length (Ft.) | Elevation (Ft.) | Pressure (psf) | Story Force (K) | Story Shear (K) |
|---|---|---|---|---|---|---|
| **Turret** | 8.13 | 22.67 | 103.33 | 16.57 | 3.05 | 3.05 |
| **Roof** | 15 | 228 | 87.07 | 15.83 | 24.83 | 27.88 |
| **Level 6** | 13.79 | 228 | 73.32 | 15.26 | 47.98 | 75.86 |
| **Level 5** | 13.83 | 228 | 59.49 | 14.61 | 46.07 | 121.93 |
| **Level 4** | 13.83 | 228 | 45.66 | 13.82 | 43.58 | 165.51 |
| **Level 3** | 13.83 | 228 | 31.83 | 12.80 | 40.36 | 205.87 |
| **Level 2** | 15.92 | 228 | 18 | 11.36 | 41.23 | 247.10 |
| **Level 1** | 9 | 228 | 0 | 11.36 | 23.31 | 270.41 |

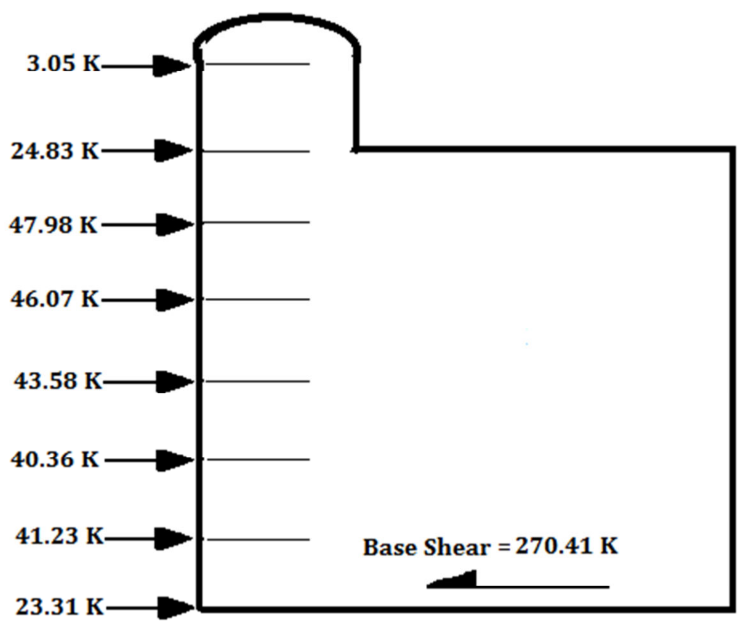



*Figure 5-6: North/South Wind Story Forces*

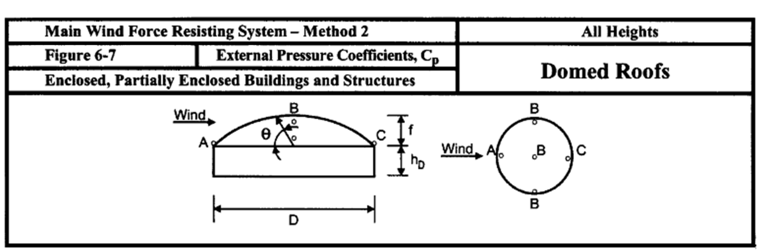



*Figure 5-7: ASCE 7-05 Domed Roof Excerpt*

## Seismic Forces

The seismic loads for the North Shore Equitable Building were calculated using ASCE 7-05's equivalent lateral force procedure. The calculations were performed in Tech Report 1 and reiterated here. 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 B.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 seismic analysis results.

**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 | 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 |

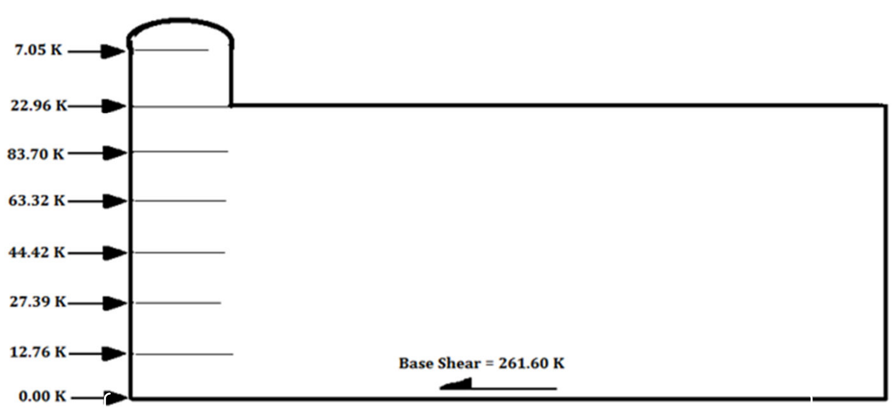



*Figure 5-8 Seismic Story Forces*

# 6. ETABS MODEL ANALYSIS

## Model Description

In order to analyze the lateral forces, a computer model of the North Shore Equitable building was constructed using ETABS. Included in this model are the lateral framing members only, as well as story diaphragms. For simplicity, the building was modeled as a rectangle, omitting the turret and tower details at each corner of the building. The diaphragms were set as rigid and a mass per area value was assigned to each diaphragm based on the total story weights calculated in tech 1 and found in Table B.2. Retaining walls were modeled on the parking sublevel as well since these walls will affect the base shear of the building. The concrete modifier for these walls was set to 0.7 to be conservative. Once added, the walls were auto meshed with a max mesh size of 24”. Since all mass was included in the story diaphragms, the material weights of the shear walls and steel members were set to 0 to avoid double counting the mass. Shown below in Figure 6.1 is an image of the ETABS model.

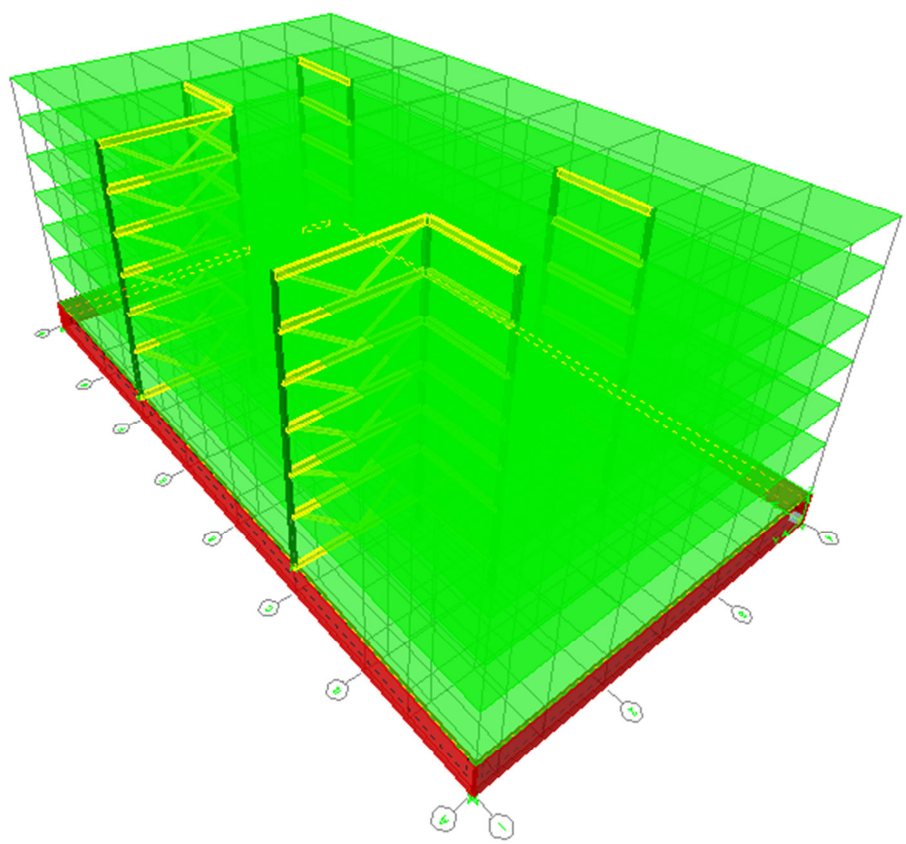


*Figure 6-1: ETABS Model of the North Shore Equitable Building*

Once the model was completed, the loads, load cases, and forces were added. A 100 psf live load was applied to each level as designed. The wind and seismic loads were set to "user defined" and the seismic and wind story forces from Tables 5-9, 5-10 and 5-12 of this report were input. The user designed seismic forces were applied at the center of mass. Output data based on all of these load combinations can be found in Appendix C.

A good indication that the model accurately represents the building design is whether or not a reasonable period can be obtained. The periods for this ETABS model are shown below, compared with an approximated period from ASCE 7-05; 12.8.2:

**TABLE 6.1 - Building Period Values (in seconds)**

| | | | |
|---|---|---|---|
| **ETABS Analysis Periods** | $T_x = 2.71$ s | $T_y = 1.96$ s | $T_z = 1.21$ s |
| **ASCE 7-05 Approximated Period** | $T_a = C_t h_n^x = 1.188$ s | | |

While the building periods calculated by the designers are not available for comparison, it can be seen that reasonable periods were obtained from this analysis. Normally, the ASCE approximated period is longer and more conservative than the computer model periods. Knowing this, it can be concluded that the ETABS model is fairly conservative, having larger periods than the ASCE approximated period.

## Load Combinations

From the 7 load cases defined in ASCE 7-05 chapter 2, all load combinations were defined in ETABS. Defining separate load combinations for N/S wind and E/W wind, along with seismic forces in the N/S and E/W directions resulted in 13 different load combinations entered into ETABS. These resulting 13 load combinations are shown below in Table 6.2. Four separate wind load cases were taken into account as well and will be discussed in greater detail in chapter 7 of this report. To simplify the model analysis, roof live load, snow load and rain load have been neglected.

**TABLE 6.2 – Load Combinations used in ETABS**

| Combo | Equation | Combo | Equation |
|---|---|---|---|
| 1 | 1.4D | 8 | $1.2D + 1.0\,E_X + L$ |
| 2 | 1.2D + 1.6L | 9 | $1.2D + 1.0\,E_Y + L$ |
| 3 | 1.2D + L | 10 | $0.9D + 1.6W_X$ |
| 4 | $1.2D + 0.8W_X$ | 11 | $0.9D + 1.6W_Y$ |
| 5 | $1.2D + 0.8W_Y$ | 12 | $0.9D + 1.0\,E_X$ |
| 6 | $1.2D + 1.6W_Y + L$ | 13 | $0.9D + 1.0\,E_Y$ |
| 7 | $1.2D + 1.6W_X + L$ | | |

## Relative Stiffness

Another way to check the accuracy of the model is to compare a hand calculation of the center of rigidity to the ETABS center of rigidity. In order to do this, the stiffness values of each lateral force resisting element must be known. Since the design of the North Shore Equitable building has two types of frame, moment and braced, two stiffness values must be calculated.

To find these stiffness values, the frames were isolated in the ETABS model and a 1 kip horizontal load was applied at the top right corner of each frame. The ETABS analysis was run and the resulting frame deflections were recorded. The relative stiffness values were then calculated and can be seen in Table 6.3 below.

**TABLE 6.3 - Frame Stiffness Values at Level 6**

| | Applied force (K) | Deflection (in) | Stiffness ($K = p_i/\Delta$) |
|---|---|---|---|
| **Braced Frame** | 1.0 | -.006181 | 162 (k/in) |
| **Moment Frame** | 1.0 | -.0602 | 16.6 (k/in) |

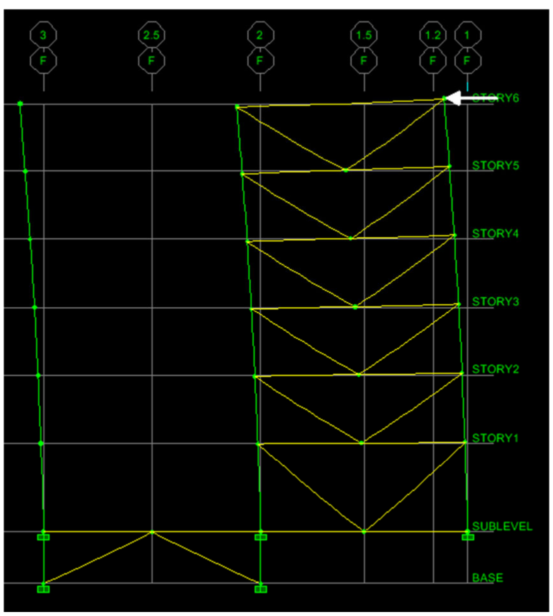



*Figure 6-2: Braced Frame Deflection*

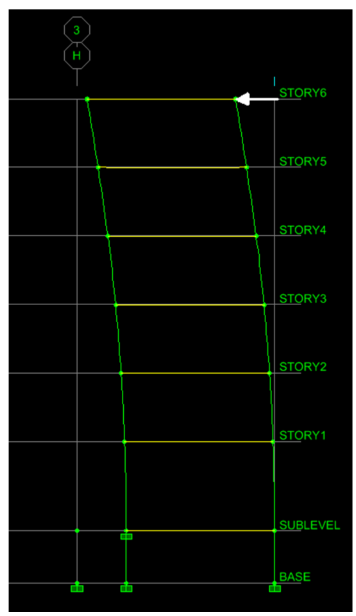



*Figure 6-3: Moment Frame Deflection*

## Center of Mass and Rigidity

To check the accuracy of the ETABS model, the center of rigidity was calculated by hand and compared to the computer results for center of rigidity. The center of mass was also found using ETABS. The results are given on the following page in Table 6.4.

**TABLE 6.4 – Center of Mass and Rigidity**

| | C.O.M. | | ETABS C.O.R. | | Hand | |
|---|---|---|---|---|---|---|
| **Level** | **X(ft)** | **Y(ft)** | **X(ft)** | **Y(ft)** | **X(ft)** | **Y(ft)** |
| **Sublevel** | 113.00 | 71.96 | | | | |
| **1** | 113.98 | 64.27 | 112.82 | 67.6 | 113 | 64 |
| **2** | 113.98 | 64.25 | 112.94 | 66.8 | 113 | 64 |
| **3** | 113.98 | 64.23 | 112.96 | 67 | 113 | 64 |
| **4** | 113.98 | 64.22 | 112.97 | 67.1 | 113 | 64 |
| **5** | 113.98 | 64.21 | 112.98 | 67.5 | 113 | 64 |
| **6** | 113.68 | 69.34 | 112.98 | 67.7 | 113 | 64 |

## Deflections

In order to assure that the design of this building achieves lateral stability, deflections must be checked and compared to acceptable industry values. Once an analysis was run of the ETABS model, the deflections of each building level were found for each load case. These values are compared to an industry acceptable value of $h_x/400$ for wind loads and 0.02 $h_{sx}$ for seismic loads. Shown below in Table 6.5 are the deflections for all load cases at level 6. Tables for levels 1 through 5 can be found in Appendix D.

**TABLE 6.5 - ETABS Deflections Output for Level 6**

| Load Combo | $\Delta_X$ (in) | $\Delta_Y$ (in) | $h_x/400$ | 0.02 $h_{sx}$ | Acceptable? |
|---|---|---|---|---|---|
| COMB1 | -0.0416 | -0.502 | 2.613 | 20.9 | Yes |
| COMB2 | -0.0357 | -0.4303 | 2.613 | 20.9 | Yes |
| COMB3 | -0.0357 | -0.4303 | 2.613 | 20.9 | Yes |
| COMB4 | 0.8472 | -0.43 | 2.613 | 20.9 | Yes |
| COMB5 | -0.0351 | -0.085 | 2.613 | 20.9 | Yes |
| COMB6 | -0.0346 | 0.2602 | 2.613 | 20.9 | Yes |
| COMB7 | 1.73 | -0.4297 | 2.613 | 20.9 | Yes |
| COMB8 | 2.3724 | -0.4295 | 2.613 | 20.9 | Yes |
| COMB9 | -0.0348 | 0.1086 | 2.613 | 20.9 | Yes |
| COMB10 | 1.739 | -0.3222 | 2.613 | 20.9 | Yes |
| COMB11 | -0.0257 | 0.3678 | 2.613 | 20.9 | Yes |
| COMB12 | 2.3813 | -0.322 | 2.613 | 20.9 | Yes |
| COMB13 | -0.0259 | 0.2161 | 2.613 | 20.9 | Yes |

According to the ETABS analysis, all load combinations produce deflections that are within the acceptable range defined by industry standards. Therefore, it can be concluded that this design is satisfactory as far as deflections are concerned.

# 7. WIND LOAD CASE ANALYSIS

The four wind loading combinations from ASCE 7-05 (shown below in figure 7-1) were taken into account as part of the analysis as well. Load Case 1 was performed by hand for level 6, and a spreadsheet was prepared for the remaining levels and load cases. Shown in Figure 7-2 below is the lateral framing plan with centers of mass and rigidity used for the wind load case calculations. The eccentricities shown are for load case 1. These values vary for the other load cases.

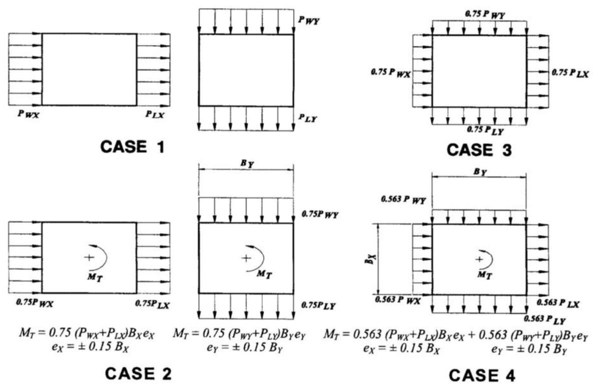



*Figure 7-1: ASCE 7-05 Wind Load Cases*

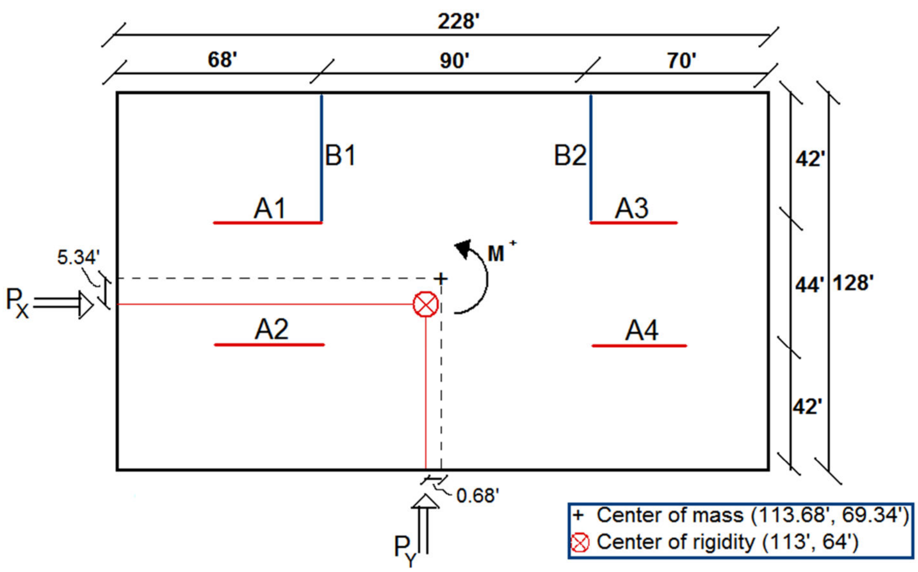



*Figure 7-2: Lateral Framing Plan Showing Centers of Mass and Rigidity*

## Torsional Shear

Because of the location of the lateral frames in the building, the center of rigidity is not equal to the center of mass. This will introduce some torsion into the building when wind loads are applied. The eccentricities causing the torsion and their resulting moments (for level 6 only) can be seen below in Tables 7.1 and 7.2. For level 1 through 5, please see Appendix E.

**TABLE 7.1 – Level 6 Eccentricities and Moments for Load Cases 1 and 2**

| | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- |
|---|---|---|---|---|---|---|
| **Py (k)** | 69.39 | 0.00 | 52.04 | 52.04 | 0.00 | 0.00 |
| **ex (ft)** | -0.68 | 0.00 | 33.52 | -34.88 | 0.00 | 0.00 |
| **Px (k)** | 0.00 | 39.64 | 0.00 | 0.00 | 29.73 | 29.73 |
| **ey (ft)** | 0.00 | -5.34 | 0.00 | 0.00 | 13.86 | -24.54 |
| **Mx (ft-k)** | -47.19 | -47.19 | 1744.46 | -1815.24 | 1744.46 | -1815.24 |
| **My (ft-k)** | 211.68 | 211.68 | 412.06 | -729.57 | 412.06 | -729.57 |

**TABLE 7.2 – Level 6 Eccentricities and Moments for Load Cases 3 and 4**

| | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|
| **Py (k)** | 52.04 | 39.07 | 39.07 | 39.07 | 39.07 |
| **ex (ft)** | -0.68 | 33.52 | 33.52 | -34.88 | -34.88 |
| **Px (k)** | 29.73 | 22.32 | 22.32 | 22.32 | 22.32 |
| **ey (ft)** | -5.34 | 13.86 | -24.54 | 13.86 | -24.54 |
| **M (ft-k)** | 123.37 | 1000.19 | 1857.18 | -1671.96 | -814.97 |

To find the torsional shear, a torsional coefficient was calculated using the equation $k_i d_i / \sum k_i d_i^2$. Multiplying this coefficient by the moment for each given load case results in the torsional shear. Given in Table 7.3 are the torsional shears for each frame at level 6. These torsional shear values are for the controlling load case only (case 4+-). The method for determining this controlling load case will be covered in the next section. The torsional shears for this load case for levels 1 through 5 can be found in Appendix E (Tables E.14 through E.19).

**TABLE 7.3 - Level 6 Torsional Shears for Load Case 4+-**

| Frame | Torsional Coefficient (1/Ft) | Moment (Ft-K) | Torsional Shear (K) |
|---|---|---|---|
| **B1** | -0.01111 | 1857.18 | -20.64 |
| **B2** | 0.01111 | 1857.18 | 20.64 |
| **A1** | -0.01136 | 1857.18 | -21.10 |
| **A2** | 0.01136 | 1857.18 | 21.10 |
| **A3** | -0.01136 | 1857.18 | -21.10 |
| **A4** | 0.01136 | 1857.18 | 21.10 |

## Total Shear

Once the direct and torsional shears were calculated, they were combined to find the total shear on each frame. Tables 7.4 and 7.5 below give the total shear values (or lateral forces) applied to each frame at level 6 for all 4 load cases. Spreadsheets for levels 1 through 5, along with the hand calculations, can be found in Appendix E.

**TABLE 7.4 – 6th Floor Lateral Forces for Load Cases 1 and 2 (Kips)**

| Frame | Direct x | Direct y | Torsional | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- |
|---|---|---|---|---|---|---|---|---|---|
| B1 | 0 | 0.5 | -0.01111 | 35.22 | -0.52 | 6.64 | 46.19 | -19.38 | 20.17 |
| B2 | 0 | 0.5 | 0.01111 | 34.17 | 0.52 | 45.40 | 5.85 | 19.38 | -20.17 |
| A1 | 0.25 | 0 | -0.01136 | -2.41 | 7.50 | -4.68 | 8.29 | 2.75 | 15.72 |
| A2 | 0.25 | 0 | 0.01136 | 2.41 | 12.32 | 4.68 | -8.29 | 12.11 | -0.86 |
| A3 | 0.25 | 0 | -0.01136 | -2.41 | 7.50 | -4.68 | 8.29 | 2.75 | 15.72 |
| A4 | 0.25 | 0 | 0.01136 | 2.41 | 12.32 | 4.68 | -8.29 | 12.11 | -0.86 |

**TABLE 7.5 – 6th Floor Lateral Forces for Load Cases 3 and 4 (Kips)**

| Frame | Direct x | Direct y | Torsional | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|
| B1 | 0 | 0.5 | -0.01111 | 24.65 | 8.42 | -1.10 | 38.11 | 28.59 |
| B2 | 0 | 0.5 | 0.01111 | 27.39 | 30.65 | 40.17 | 0.96 | 10.48 |
| A1 | 0.25 | 0 | -0.01136 | 6.03 | -5.79 | -15.52 | 24.58 | 14.84 |
| A2 | 0.25 | 0 | 0.01136 | 8.83 | 16.95 | 26.68 | -13.42 | -3.68 |
| A3 | 0.25 | 0 | -0.01136 | 6.03 | -5.79 | -15.52 | 24.58 | 14.84 |
| A4 | 0.25 | 0 | 0.01136 | 8.83 | 16.95 | 26.68 | -13.42 | -3.68 |

Since the stiffness for frame A is much lower than the stiffness for frame B, the controlling load case will be the load case that results in the largest forces applied to frames A1, A2, A3 and A4. This is because a lower stiffness will equate to a higher deflection. From Table 7.5 it can be seen that the controlling wind load case is load case 4 where a positive eccentricity is applied to the Y direction force and a negative eccentricity is applied to the X direction force (represented as "case 4+-" in Table 7.5). A diagram of this controlling load case can be seen to the right in figure 7-3.

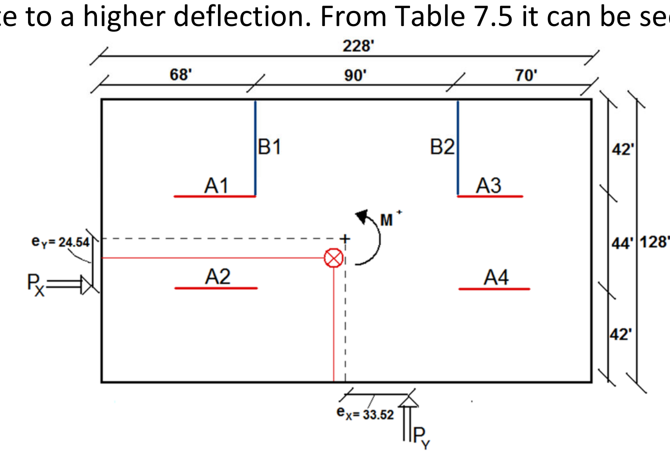



*Figure 7-3: Diagram of the controlling load case (case 4+-)*

## 8. OVERTURNING & STRENGTH CHECKS

To check the design's overturning moment, the moment caused by the controlling wind load case was taken at the base of column line C3. This value was then compared to the total dead load supported by column line C3. Column line C3 was chosen because it is part of the moment frame A2 (shown below in figure 8-1) which spans the shortest distance and is subject to the largest wind loads. The calculation shows that the dead load is sufficiently large to prevent overturning. These calculations can be found in Appendix F.

To perform a strength check on a typical beam and column within the moment frame, the portal method was used to find the maximum moments. Through this analysis, both the W27x146 beam and the W14x311 column (shown in figure 8-1 below) were found to be adequate to support the applied lateral loads and gravity loads. A deflection check of the beam also confirmed that the beam falls within the minimum deflection criteria. Strength calculations for both the beam and column can be found in Appendix F.

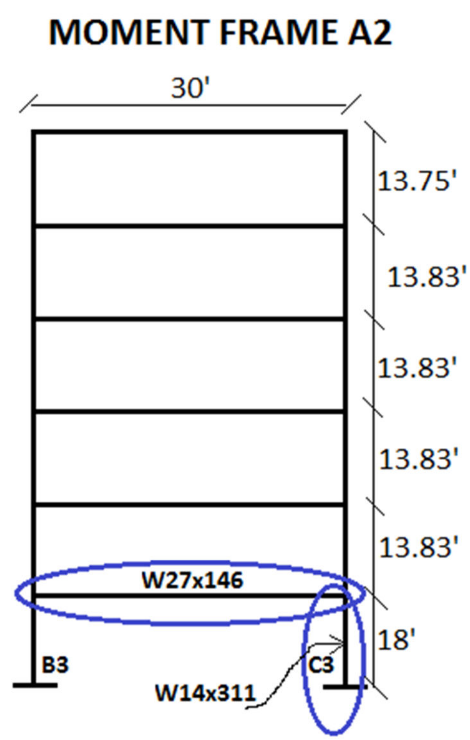



*Figure 8-1: Moment Frame A2 used for Overturning and Strength Checks*

# 9. CONCLUSION

After performing an analysis of the lateral system of the North Shore Equitable Building, several conclusions can be made. Once adjustments were made to the Tech 1 wind load analysis, the lateral wind forces were found to be smaller than originally calculated. Wind loading still controls however, having a base shear value of 270.41 kips (compared to a seismic base shear of 261.6 kips). The ETABS analysis results show that load combination 7 yields the largest deflections.

Through an analysis of ASCE 7-05's 4 wind load cases, it was determined that load case 4 is the controlling wind load case. From the wind load case analysis, it was also found that torsion is present in the design due to eccentricities caused by differences in the centers of mass and rigidity.

The analysis of the 3D ETABS model shows that all building deflections are within the acceptable range according to industry standards. It was also concluded that an appropriate load path exists for the distribution of lateral forces through the building.

Finally, hand calculations confirmed that overturning moments are not an issue in this design and that all lateral framing members are sized sufficiently to carry all applied lateral loads.

# 10. APPENDICES

## APPENDIX A – WIND LOAD CALCULATIONS

**TABLE A.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 A.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 A.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 |
| | $\alpha$ | 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 |

Wind Analysis | Tech Report # 1 | Page 1 of 7

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 $\Rightarrow$ interpolate $K_h$

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

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

$$333.33(K_h - 1.21) = 7.08$$

$K_h = 1.23$

$K_z = 2.01(z/z_g)^{2/\alpha}$ $\Rightarrow$ 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 $\Rightarrow K_{zt} = 1.0$

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 $\Rightarrow$ Bldg height = 87.08 < 300 ✓ ok

87.08 < 4(147.35) ✓ ok

∴ natural frequency can be approximated

Wind Analysis | Tech Report #1 | Page 2 of 7

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Wind Analysis | Tech Report # 1 | Page 3 of 7

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

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

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

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

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

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

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

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{.167}$ where $c = 0.2$ (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: $G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$

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

$\boxed{G_f = 0.929}$

Enclosure classification: Enclosed

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

External pressure coefficients: $C_p$

walls and roof:

windward wall $\Rightarrow$ $\boxed{C_p = 0.8}$ (use with $q_z$)

Leeward wall $\Rightarrow$ $L/B = 228/128 = 1.78$

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

Side wall $\Rightarrow$ $\boxed{C_p = -0.7}$ (use with $q_h$)

Roof $\Rightarrow$ $h/L = \dfrac{87.08}{228} = .381 < 0.5$

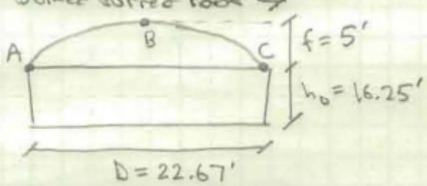
$\dfrac{h}{2} = \dfrac{87.08}{2} = 43.54'$ $\quad 2h = 174.16'$

from 0 to 87.08' from windward edge $\Rightarrow$ $\boxed{C_p = -0.9}$

from 87.08' to 174.16' $\Rightarrow$ $\boxed{C_p = -0.5}$

from 174.16' to 228' $\Rightarrow$ $\boxed{C_p = -0.3}$

Domed turret roof $\Rightarrow$




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

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

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

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

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

$\boxed{C_{pA} = -1.173}$

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

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

| Wind Analysis | Tech Report #1 | Page 5 of 7 |
|---|---|---|

Velocity pressures: see attached 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: $P = qG_fC_p - q_i(GC_{pi})$

See spreadsheet for design loads and pressures

Story forces diagram: (see spreadsheet for calcs)

Wind Analysis | Tech Report #1 | Page 6 of 7

Direction North/South

Gust Effect Factor:

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

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

$\boxed{87.08 < 4(87.04) \checkmark \text{ ok}}$ ⇒ natural freq. can be approximated

$n_a = 0.861$

For $R_B \Rightarrow \eta = \frac{4.6\, n_1 EB}{\bar{V}_{\bar{z}}}$ 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 $R_L \Rightarrow \eta = \frac{15.4\, n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4(.861)(128)}{92.09} = 18.43$

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

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

$R = .0249$

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

$Q = 0.832$

Flexible Gust effect factor:

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

$\boxed{G_f = 0.913}$

Wind Analysis | Tech Report # 1 | Page 7 of 7

$GC_{pi} = \pm 0.18$

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

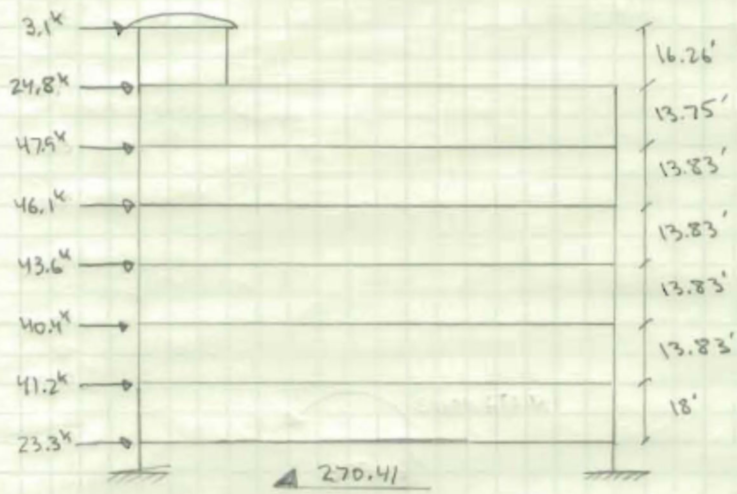
* Same as for E/W direction

Dome Turret Roof:

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

* See spreadsheet for story pressures

Story Forces Diagram: See spreadsheet for calcs



## APPENDIX B – SEISMIC LOAD CALCULATIONS

**TABLE B.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 B.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** |

Seismic Analysis | Tech Report #1 | Page 1 of 2

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.

Seismic Analysis | Tech Report #1 | Page 2 of 2

Seismic Base shear:

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

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

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

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

$.0154 > .01$ ✓ ok

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

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

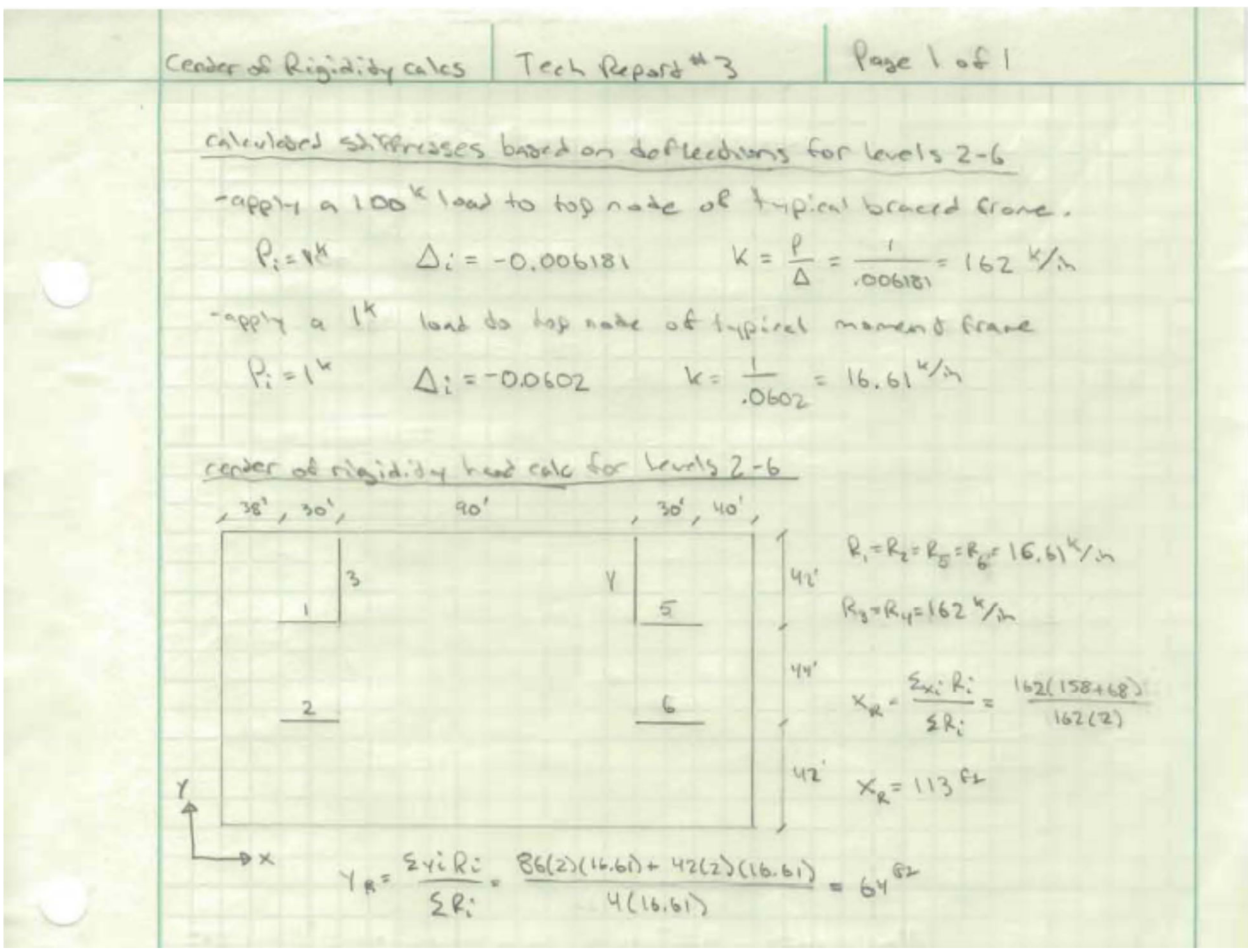
$k = 1.34$

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

## APPENDIX C – STIFFNESS AND RIGIDITY CALCULATIONS

**TABLE C.1 – Frame Stiffness Values At All Levels**

| Standard Moment Frame | | | | Standard Braced Frame | | | |
|---|---|---|---|---|---|---|---|
| **Level** | **defl (in)** | **load (k)** | **k (k/in)** | **Level** | **defl (in)** | **load (k)** | **k (k/in)** |
| **1** | 0.008767 | 1 | 114.0641 | **1** | 0.000403 | 1 | 2481.39 |
| **2** | 0.016903 | 1 | 59.1611 | **2** | 0.000195 | 1 | 5128.205 |
| **3** | 0.025161 | 1 | 39.74405 | **3** | 0.001463 | 1 | 683.527 |
| **4** | 0.036806 | 1 | 27.16948 | **4** | 0.002943 | 1 | 339.7893 |
| **5** | 0.048613 | 1 | 20.57063 | **5** | 0.00458 | 1 | 218.3406 |
| **6** | 0.060248 | 1 | 16.59806 | **6** | 0.006181 | 1 | 161.7861 |


Center of Rigidity calcs | Tech Report #3 | Page 1 of 1

calculated stiffnesses based on deflections for levels 2-6

-apply a 100k load to top node of typical braced frame.

$P_i = 1^k$ $\quad \Delta_i = -0.006181$ $\quad k = \frac{P}{\Delta} = \frac{1}{.006181} = 162\ ^k/_{in}$

-apply a $1^k$ load to top node of typical moment frame

$P_i = 1^k$ $\quad \Delta_i = -0.0602$ $\quad k = \frac{1}{.0602} = 16.61\ ^k/_{in}$

center of rigidity hand calc for levels 2-6



$R_1 = R_2 = R_5 = R_6 = 16.61\ ^k/_{in}$

$R_3 = R_4 = 162\ ^k/_{in}$

$X_R = \frac{\Sigma x_i R_i}{\Sigma R_i} = \frac{162(158+68)}{162(2)}$

$X_R = 113^{ft}$

$Y_R = \frac{\Sigma y_i R_i}{\Sigma R_i} = \frac{86(2)(16.61) + 42(2)(16.61)}{4(16.61)} = 64^{ft}$

# APPENDIX D – ETABS MODEL OUTPUTS

**TABLE D.1 – Centers of Mass and Rigidity**

| STORY | Diaphragm | MassX | MassY | XCM(in) | YCM(in) | CumMassX | CumMassY | XCCM(in) | YCCM(in) | XCR (in) | YCR(in) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| STORY6 | D1 | 0.22 | 0.22 | 1364.21 | 832.04 | 0.22 | 0.22 | 1364.21 | 832.04 | 1355.81 | 812.85 |
| STORY5 | D1 | 7.05 | 7.05 | 1367.82 | 770.56 | 7.27 | 7.27 | 1367.71 | 772.43 | 1355.75 | 809.43 |
| STORY4 | D1 | 7.05 | 7.05 | 1367.82 | 770.62 | 14.31 | 14.31 | 1367.76 | 771.54 | 1355.66 | 804.92 |
| STORY3 | D1 | 7.06 | 7.06 | 1367.79 | 770.81 | 21.38 | 21.38 | 1367.77 | 771.30 | 1355.50 | 803.90 |
| STORY2 | D1 | 7.08 | 7.08 | 1367.76 | 770.98 | 28.46 | 28.46 | 1367.77 | 771.22 | 1355.30 | 801.43 |
| STORY1 | D1 | 7.53 | 7.53 | 1367.72 | 771.19 | 35.99 | 35.99 | 1367.76 | 771.21 | 1353.86 | 811.24 |
| SUBLEVEL | D1 | 0.20 | 0.20 | 1356.00 | 863.47 | 36.18 | 36.18 | 1367.69 | 771.71 | | |

ETABS Model
Load Application | Tech Report # 3 | Page 1 of 1

Calculated building weight applied to model

Level 1: $\frac{2857.79^k}{(128)(228)}(1000) = 97.92\ psf \Rightarrow \frac{.09792}{(32.2)(12^3)} = 1.76\ E{-6}\ k\text{-}in$

Level 2-6: $\frac{2681.15^k}{(128)(228)}(1000) = 91.87\ psf \Rightarrow \frac{.09187}{(32.2)(12^3)} = 1.651\ E{-6}\ k\text{-}in$

Roof Level: $\frac{583.68^k}{(128)(228)}(1000) = 20\ psf \Rightarrow \frac{.02}{(32.2)(12^3)} = 3.594\ E{-7}\ k\text{-}in$

$\frac{lb \cdot s^2}{ft} = slug \quad \frac{kiloslug}{in^2}$

**TABLE D.2 - Load Combinations Used**

| Combo | Equation |
|---|---|
| 1 | 1.4D |
| 2 | 1.2D + 1.6L |
| 3 | 1.2D + L |
| 4 | 1.2D + 0.8WX |
| 5 | 1.2D + 0.8WY |
| 6 | 1.2D + 1.6WY + L |
| 7 | 1.2D + 1.6WX + L |
| 8 | 1.2D + 1.0 EX + L |
| 9 | 1.2D + 1.0 EY + L |
| 10 | 0.9D + 1.6WX |
| 11 | 0.9D + 1.6WY |
| 12 | 0.9D + 1.0 EX |
| 13 | 0.9D + 1.0 EY |

**TABLE D.3 – Level 1 ETABS Deflection Outputs**

| Load Combination | X-axis (in) | Y-axis (in) | $h_x$/400 (in) | 0.02 $h_{sx}$ (in) | Acceptable? |
|---|---|---|---|---|---|
| COMB1 | -0.0035 | -0.0589 | 0.54 | 4.32 | Yes |
| COMB2 | -0.003 | -0.0505 | 0.54 | 4.32 | Yes |
| COMB3 | -0.003 | -0.0505 | 0.54 | 4.32 | Yes |
| COMB4 | 0.1993 | -0.0504 | 0.54 | 4.32 | Yes |
| COMB5 | -0.0029 | 0.0208 | 0.54 | 4.32 | Yes |
| COMB6 | -0.0028 | 0.0921 | 0.54 | 4.32 | Yes |
| COMB7 | 0.4015 | -0.0503 | 0.54 | 4.32 | Yes |
| COMB8 | 0.4743 | -0.0503 | 0.54 | 4.32 | Yes |
| COMB9 | -0.0029 | 0.0456 | 0.54 | 4.32 | Yes |
| COMB10 | 0.4022 | -0.0377 | 0.54 | 4.32 | Yes |
| COMB11 | -0.0021 | 0.1047 | 0.54 | 4.32 | Yes |
| COMB12 | 0.475 | -0.0377 | 0.54 | 4.32 | Yes |
| COMB13 | -0.0021 | 0.0582 | 0.54 | 4.32 | Yes |

**TABLE D.4 – Level 2 ETABS Deflection Outputs**

| Load Combination | X-axis (in) | Y-axis (in) | $h_x$/400 (in) | 0.02 $h_{sx}$ (in) | Acceptable? |
|---|---|---|---|---|---|
| COMB1 | -0.0069 | -0.1205 | 0.955 | 7.64 | Yes |
| COMB2 | -0.0059 | -0.1033 | 0.955 | 7.64 | Yes |
| COMB3 | -0.0059 | -0.1033 | 0.955 | 7.64 | Yes |
| COMB4 | 0.3909 | -0.1032 | 0.955 | 7.64 | Yes |
| COMB5 | -0.0058 | 0.0291 | 0.955 | 7.64 | Yes |
| COMB6 | -0.0057 | 0.1616 | 0.955 | 7.64 | Yes |
| COMB7 | 0.7877 | -0.1031 | 0.955 | 7.64 | Yes |
| COMB8 | 0.9744 | -0.1031 | 0.955 | 7.64 | Yes |
| COMB9 | -0.0058 | 0.0851 | 0.955 | 7.64 | Yes |
| COMB10 | 0.7892 | -0.0773 | 0.955 | 7.64 | Yes |
| COMB11 | -0.0042 | 0.1874 | 0.955 | 7.64 | Yes |
| COMB12 | 0.9759 | -0.0773 | 0.955 | 7.64 | Yes |
| COMB13 | -0.0043 | 0.1109 | 0.955 | 7.64 | Yes |

**TABLE D.5 – Level 3 ETABS Deflection Outputs**

| Load Combination | X-axis (in) | Y-axis (in) | $h_x$/400 (in) | 0.02 $h_{sx}$ (in) | Acceptable? |
|---|---|---|---|---|---|
| COMB1 | -0.0123 | -0.1942 | 1.37 | 10.96 | Yes |
| COMB2 | -0.0106 | -0.1664 | 1.37 | 10.96 | Yes |
| COMB3 | -0.0106 | -0.1664 | 1.37 | 10.96 | Yes |
| COMB4 | 0.5489 | -0.1663 | 1.37 | 10.96 | Yes |
| COMB5 | -0.0104 | 0.0264 | 1.37 | 10.96 | Yes |
| COMB6 | -0.0102 | 0.2193 | 1.37 | 10.96 | Yes |
| COMB7 | 1.1084 | -0.1662 | 1.37 | 10.96 | Yes |
| COMB8 | 1.4312 | -0.1661 | 1.37 | 10.96 | Yes |
| COMB9 | -0.0103 | 0.1201 | 1.37 | 10.96 | Yes |
| COMB10 | 1.1111 | -0.1246 | 1.37 | 10.96 | Yes |
| COMB11 | -0.0075 | 0.2609 | 1.37 | 10.96 | Yes |
| COMB12 | 1.4339 | -0.1245 | 1.37 | 10.96 | Yes |
| COMB13 | -0.0076 | 0.1617 | 1.37 | 10.96 | Yes |

**TABLE D.6 – Level 4 ETABS Deflection Outputs**

| Load Combination | X-axis (in) | Y-axis (in) | $h_x/400$ (in) | 0.02 $h_{sx}$ (in) | Acceptable? |
|---|---|---|---|---|---|
| COMB1 | -0.0202 | -0.2855 | 1.785 | 14.28 | Yes |
| COMB2 | -0.0173 | -0.2447 | 1.785 | 14.28 | Yes |
| COMB3 | -0.0173 | -0.2447 | 1.785 | 14.28 | Yes |
| COMB4 | 0.708 | -0.2446 | 1.785 | 14.28 | Yes |
| COMB5 | -0.017 | 0.0079 | 1.785 | 14.28 | Yes |
| COMB6 | -0.0167 | 0.2605 | 1.785 | 14.28 | Yes |
| COMB7 | 1.4332 | -0.2444 | 1.785 | 14.28 | Yes |
| COMB8 | 1.9319 | -0.2443 | 1.785 | 14.28 | Yes |
| COMB9 | -0.0168 | 0.1434 | 1.785 | 14.28 | Yes |
| COMB10 | 1.4376 | -0.1832 | 1.785 | 14.28 | Yes |
| COMB11 | -0.0124 | 0.3216 | 1.785 | 14.28 | Yes |
| COMB12 | 1.9362 | -0.1831 | 1.785 | 14.28 | Yes |
| COMB13 | -0.0125 | 0.2046 | 1.785 | 14.28 | Yes |

**TABLE D.7 – Level 5 ETABS Deflection Outputs**

| Load Combination | X-axis (in) | Y-axis (in) | $h_x/400$ (in) | 0.02 $h_{sx}$ (in) | Acceptable? |
|---|---|---|---|---|---|
| COMB1 | -0.0304 | -0.3925 | 2.2 | 17.6 | Yes |
| COMB2 | -0.0261 | -0.3364 | 2.2 | 17.6 | Yes |
| COMB3 | -0.0261 | -0.3364 | 2.2 | 17.6 | Yes |
| COMB4 | 0.8058 | -0.3362 | 2.2 | 17.6 | Yes |
| COMB5 | -0.0257 | -0.0305 | 2.2 | 17.6 | Yes |
| COMB6 | -0.0252 | 0.2754 | 2.2 | 17.6 | Yes |
| COMB7 | 1.6378 | -0.336 | 2.2 | 17.6 | Yes |
| COMB8 | 2.2539 | -0.3358 | 2.2 | 17.6 | Yes |
| COMB9 | -0.0254 | 0.1424 | 2.2 | 17.6 | Yes |
| COMB10 | 1.6443 | -0.2519 | 2.2 | 17.6 | Yes |
| COMB11 | -0.0187 | 0.3595 | 2.2 | 17.6 | Yes |
| COMB12 | 2.2604 | -0.2517 | 2.2 | 17.6 | Yes |
| COMB13 | -0.0189 | 0.2265 | 2.2 | 17.6 | Yes |

**TABLE D.8 – Level 6 ETABS Deflection Outputs**

| Load Combination | X-axis (in) | Y-axis (in) | $h_x/400$ (in) | 0.02 $h_{sx}$ (in) | Acceptable? |
|---|---|---|---|---|---|
| COMB1 | -0.0416 | -0.502 | 2.613 | 20.9 | Yes |
| COMB2 | -0.0357 | -0.4303 | 2.613 | 20.9 | Yes |
| COMB3 | -0.0357 | -0.4303 | 2.613 | 20.9 | Yes |
| COMB4 | 0.8472 | -0.43 | 2.613 | 20.9 | Yes |
| COMB5 | -0.0351 | -0.085 | 2.613 | 20.9 | Yes |
| COMB6 | -0.0346 | 0.2602 | 2.613 | 20.9 | Yes |
| COMB7 | 1.73 | -0.4297 | 2.613 | 20.9 | Yes |
| COMB8 | 2.3724 | -0.4295 | 2.613 | 20.9 | Yes |
| COMB9 | -0.0348 | 0.1086 | 2.613 | 20.9 | Yes |
| COMB10 | 1.739 | -0.3222 | 2.613 | 20.9 | Yes |
| COMB11 | -0.0257 | 0.3678 | 2.613 | 20.9 | Yes |
| COMB12 | 2.3813 | -0.322 | 2.613 | 20.9 | Yes |
| COMB13 | -0.0259 | 0.2161 | 2.613 | 20.9 | Yes |

# APPENDIX E – WIND LOAD CASE ANALYSIS

**TABLE E.1 – Calculation of Direct and Torsional Coefficients**

| Frame | $K_{xi}$ ($^k/_{in}$) | $K_{yi}$ ($^k/_{in}$) | $X_i$ (ft) | $Y_i$ (ft) | $K_{yi}$*$X_i$ | $K_{xi}$*$Y_i$ | $d_i$ | $K_i d_i^2$ | Direct x | Direct y | Torsional |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **B1** | 0 | 161.8 | 68 | 0 | 11002.4 | 0 | -45 | 327645 | 0 | 0.5 | -0.01111 |
| **B2** | 0 | 161.8 | 158 | 0 | 25564.4 | 0 | 45 | 327645 | 0 | 0.5 | 0.011111 |
| **A1** | 16.6 | 0 | 0 | 86 | 0 | 1427.6 | 22 | 8034.4 | 0.25 | 0 | 0.011364 |
| **A2** | 16.6 | 0 | 0 | 42 | 0 | 697.2 | -22 | 8034.4 | 0.25 | 0 | -0.01136 |
| **A3** | 16.6 | 0 | 0 | 86 | 0 | 1427.6 | 22 | 8034.4 | 0.25 | 0 | 0.011364 |
| **A4** | 16.6 | 0 | 0 | 42 | 0 | 697.2 | -22 | 8034.4 | 0.25 | 0 | -0.01136 |
| | 66.4 | 323.6 | | | 36566.8 | 4249.6 | $\Sigma k_{ix} d_i^2$= | 655290 | 1 | 1 | |
| | | | | | | | $\Sigma k_{iy} d_i^2$= | 32137.6 | | | |

# Forces, Eccentricities and Loads for each Frame

**TABLE E.2 – Level 1 Resultant Forces and Eccentricities**

| | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $P_y$ (k) | 37.28 | 0.00 | 27.96 | 27.96 | 0.00 | 0.00 | 27.96 | 20.99 | 20.99 | 20.99 | 20.99 |
| $e_x$ (ft) | -0.98 | 0.00 | 33.22 | -35.18 | 0.00 | 0.00 | -0.98 | 33.22 | 33.22 | -35.18 | -35.18 |
| $P_x$ (k) | 0.00 | 21.29 | 0.00 | 0.00 | 15.97 | 15.97 | 15.97 | 11.99 | 11.99 | 11.99 | 11.99 |
| $e_y$ (ft) | 0.00 | -0.27 | 0.00 | 0.00 | 18.93 | -19.47 | -0.27 | 18.93 | -19.47 | 18.93 | -19.47 |
| $M_x$ (ft-k) | -36.54 | -36.54 | 928.95 | -983.76 | 928.95 | -983.76 | -23.09 | 470.44 | 930.69 | -965.37 | -505.11 |
| $M_y$ (ft-k) | 5.75 | 5.75 | 302.25 | -310.87 | 302.25 | -310.87 | - - | - - | - - | - - | - - |

**TABLE E.3 – Level 1 Lateral Loads on Each Frame (Kips)**

| Frame | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|---|---|---|
| B1 | 19.05 | -0.41 | 3.66 | 24.91 | -10.32 | 10.93 | 14.24 | 5.27 | 0.15 | 21.22 | 16.11 |
| B2 | 18.24 | 0.41 | 24.30 | 3.05 | 10.32 | -10.93 | 13.73 | 15.72 | 20.84 | -0.23 | 4.88 |
| A1 | -0.07 | 5.26 | -3.43 | 3.53 | 0.56 | 7.52 | 4.25 | -2.35 | -7.58 | 13.97 | 8.74 |
| A2 | 0.07 | 5.39 | 3.43 | -3.53 | 7.43 | 0.46 | 3.73 | 8.34 | 13.57 | -7.97 | -2.74 |
| A3 | -0.07 | 5.26 | -3.43 | 3.53 | 0.56 | 7.52 | 4.25 | -2.35 | -7.58 | 13.97 | 8.74 |
| A4 | 0.07 | 5.39 | 3.43 | -3.53 | 7.43 | 0.46 | 3.73 | 8.34 | 13.57 | -7.97 | -2.74 |

**TABLE E.4 – Level 2 Resultant Forces and Eccentricities**

| | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $P_y$ (k) | 65.95 | 0.00 | 49.46 | 49.46 | 0.00 | 0.00 | 49.46 | 37.13 | 37.13 | 37.13 | 37.13 |
| $e_x$ (ft) | -0.98 | 0.00 | 33.22 | -35.18 | 0.00 | 0.00 | -0.98 | 33.22 | 33.22 | -35.18 | -35.18 |
| $P_x$ (k) | 0.00 | 37.66 | 0.00 | 0.00 | 28.24 | 28.24 | 28.24 | 21.20 | 21.20 | 21.20 | 21.20 |
| $e_y$ (ft) | 0.00 | -0.25 | 0.00 | 0.00 | 18.95 | -19.45 | -0.25 | 18.95 | -19.45 | 18.95 | -19.45 |
| $M_x$ (ft-k) | -64.63 | -64.63 | 1643.21 | -1740.16 | 1643.21 | -1740.16 | -41.41 | 831.74 | 1645.87 | -1708.05 | -893.92 |
| $M_y$ (ft-k) | 9.41 | 9.41 | 535.21 | -549.33 | 535.21 | -549.33 | - - | - - | - - | - - | - - |

**TABLE E.5 – Level 2 Lateral Loads on Each Frame (Kips)**

| Frame | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|---|---|---|
| B1 | 33.69 | -0.72 | 6.47 | 44.07 | -18.26 | 19.34 | 25.19 | 9.32 | 0.28 | 37.54 | 28.50 |
| B2 | 32.26 | 0.72 | 42.99 | 5.40 | 18.26 | -19.34 | 24.27 | 27.81 | 36.85 | -0.41 | 8.63 |
| A1 | -0.11 | 9.31 | -6.08 | 6.24 | 0.98 | 13.30 | 7.53 | -4.15 | -13.40 | 24.71 | 15.46 |
| A2 | 0.11 | 9.52 | 6.08 | -6.24 | 13.14 | 0.82 | 6.59 | 14.75 | 24.00 | -14.11 | -4.86 |
| A3 | -0.11 | 9.31 | -6.08 | 6.24 | 0.98 | 13.30 | 7.53 | -4.15 | -13.40 | 24.71 | 15.46 |
| A4 | 0.11 | 9.52 | 6.08 | -6.24 | 13.14 | 0.82 | 6.59 | 14.75 | 24.00 | -14.11 | -4.86 |

**TABLE E.6 – Level 3 Resultant Forces and Eccentricities**

| | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $P_y$ (k) | 61.84 | 0.00 | 46.38 | 46.38 | 0.00 | 0.00 | 46.38 | 34.81 | 34.81 | 34.81 | 34.81 |
| $e_x$ (ft) | -0.98 | 0.00 | 33.22 | -35.18 | 0.00 | 0.00 | -0.98 | 33.22 | 33.22 | -35.18 | -35.18 |
| $P_x$ (k) | 0.00 | 35.33 | 0.00 | 0.00 | 26.50 | 26.50 | 26.50 | 19.89 | 19.89 | 19.89 | 19.89 |
| $e_y$ (ft) | 0.00 | -0.23 | 0.00 | 0.00 | 18.97 | -19.43 | -0.23 | 18.97 | -19.43 | 18.97 | -19.43 |
| $M_x$ (ft-k) | -60.60 | -60.60 | 1540.62 | -1631.52 | 1540.62 | -1631.52 | -39.35 | 779.12 | 1543.01 | -1602.10 | -838.20 |
| $M_y$ (ft-k) | 8.13 | 8.13 | 502.71 | -514.90 | 502.71 | -514.90 | - - | - - | - - | - - | - - |

**TABLE E.7 – Level 3 Lateral Loads on Each Frame (Kips)**

| Frame | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|---|---|---|
| B1 | 31.59 | -0.67 | 6.07 | 41.32 | -17.12 | 18.13 | 23.63 | 8.75 | 0.26 | 35.21 | 26.72 |
| B2 | 30.24 | 0.67 | 40.31 | 5.06 | 17.12 | -18.13 | 22.75 | 26.06 | 34.55 | -0.39 | 8.09 |
| A1 | -0.09 | 8.74 | -5.71 | 5.85 | 0.91 | 12.48 | 7.07 | -3.88 | -12.56 | 23.18 | 14.50 |
| A2 | 0.09 | 8.93 | 5.71 | -5.85 | 12.34 | 0.77 | 6.18 | 13.83 | 22.51 | -13.23 | -4.55 |
| A3 | -0.09 | 8.74 | -5.71 | 5.85 | 0.91 | 12.48 | 7.07 | -3.88 | -12.56 | 23.18 | 14.50 |
| A4 | 0.09 | 8.93 | 5.71 | -5.85 | 12.34 | 0.77 | 6.18 | 13.83 | 22.51 | -13.23 | -4.55 |

### TABLE E.8 – Level 4 Resultant Forces and Eccentricities

| | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $P_y$ (k) | 65.05 | 0.00 | 48.79 | 48.79 | 0.00 | 0.00 | 48.79 | 36.62 | 36.62 | 36.62 | 36.62 |
| $e_x$ (ft) | -0.98 | 0.00 | 33.22 | -35.18 | 0.00 | 0.00 | -0.98 | 33.22 | 33.22 | -35.18 | -35.18 |
| $P_x$ (k) | 0.00 | 37.16 | 0.00 | 0.00 | 27.87 | 27.87 | 27.87 | 20.92 | 20.92 | 20.92 | 20.92 |
| $e_y$ (ft) | 0.00 | -0.22 | 0.00 | 0.00 | 18.98 | -19.42 | -0.22 | 18.98 | -19.42 | 18.98 | -19.42 |
| $M_x$ (ft-k) | -63.75 | -63.75 | 1620.75 | -1716.38 | 1620.75 | -1716.38 | -41.68 | 819.59 | 1622.90 | -1685.48 | -882.17 |
| $M_y$ (ft-k) | 8.17 | 8.17 | 528.93 | -541.20 | 528.93 | -541.20 | - - | - - | - - | - - | - - |

### TABLE E.9 – Level 4 Lateral Loads on Each Frame (Kips)

| Frame | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|---|---|---|
| B1 | 33.23 | -0.71 | 6.39 | 43.47 | -18.01 | 19.07 | 24.86 | 9.21 | 0.28 | 37.04 | 28.11 |
| B2 | 31.82 | 0.71 | 42.40 | 5.32 | 18.01 | -19.07 | 23.93 | 27.42 | 36.34 | -0.42 | 8.51 |
| A1 | -0.09 | 9.20 | -6.01 | 6.15 | 0.96 | 13.12 | 7.44 | -4.08 | -13.21 | 24.38 | 15.25 |
| A2 | 0.09 | 9.38 | 6.01 | -6.15 | 12.98 | 0.82 | 6.49 | 14.54 | 23.67 | -13.92 | -4.79 |
| A3 | -0.09 | 9.20 | -6.01 | 6.15 | 0.96 | 13.12 | 7.44 | -4.08 | -13.21 | 24.38 | 15.25 |
| A4 | 0.09 | 9.38 | 6.01 | -6.15 | 12.98 | 0.82 | 6.49 | 14.54 | 23.67 | -13.92 | -4.79 |

### TABLE E.10 – Level 5 Resultant Forces and Eccentricities

| | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $P_y$ (k) | 67.54 | 0.00 | 50.66 | 50.66 | 0.00 | 0.00 | 50.66 | 38.03 | 38.03 | 38.03 | 38.03 |
| $e_x$ (ft) | -0.98 | 0.00 | 33.22 | -35.18 | 0.00 | 0.00 | -0.98 | 33.22 | 33.22 | -35.18 | -35.18 |
| $P_x$ (k) | 0.00 | 38.57 | 0.00 | 0.00 | 28.93 | 28.93 | 28.93 | 21.72 | 21.72 | 21.72 | 21.72 |
| $e_y$ (ft) | 0.00 | -0.21 | 0.00 | 0.00 | 18.99 | -19.41 | -0.21 | 18.99 | -19.41 | 18.99 | -19.41 |
| $M_x$ (ft-k) | -66.19 | -66.19 | 1682.82 | -1782.11 | 1682.82 | -1782.11 | -43.57 | 850.83 | 1684.76 | -1750.17 | -916.24 |
| $M_y$ (ft-k) | 8.10 | 8.10 | 549.38 | -561.53 | 549.38 | -561.53 | - - | - - | - - | - - | - - |

### TABLE E.11 – Level 5 Lateral Loads on Each Frame (Kips)

| Frame | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|---|---|---|
| B1 | 34.51 | -0.74 | 6.63 | 45.13 | -18.70 | 19.80 | 25.81 | 9.56 | 0.29 | 38.46 | 29.19 |
| B2 | 33.04 | 0.74 | 44.03 | 5.53 | 18.70 | -19.80 | 24.84 | 28.47 | 37.73 | -0.43 | 8.83 |
| A1 | -0.09 | 9.55 | -6.24 | 6.38 | 0.99 | 13.61 | 7.73 | -4.24 | -13.72 | 25.32 | 15.84 |
| A2 | 0.09 | 9.74 | 6.24 | -6.38 | 13.48 | 0.85 | 6.74 | 15.10 | 24.57 | -14.46 | -4.98 |
| A3 | -0.09 | 9.55 | -6.24 | 6.38 | 0.99 | 13.61 | 7.73 | -4.24 | -13.72 | 25.32 | 15.84 |
| A4 | 0.09 | 9.74 | 6.24 | -6.38 | 13.48 | 0.85 | 6.74 | 15.10 | 24.57 | -14.46 | -4.98 |

**TABLE E.12 – Level 6 Resultant Forces and Eccentricities**

| | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $P_y$ (k) | 69.39 | 0.00 | 52.04 | 52.04 | 0.00 | 0.00 | 52.04 | 39.07 | 39.07 | 39.07 | 39.07 |
| $e_x$ (ft) | -0.68 | 0.00 | 33.52 | -34.88 | 0.00 | 0.00 | -0.68 | 33.52 | 33.52 | -34.88 | -34.88 |
| $P_x$ (k) | 0.00 | 39.64 | 0.00 | 0.00 | 29.73 | 29.73 | 29.73 | 22.32 | 22.32 | 22.32 | 22.32 |
| $e_y$ (ft) | 0.00 | -5.34 | 0.00 | 0.00 | 13.86 | -24.54 | -5.34 | 13.86 | -24.54 | 13.86 | -24.54 |
| $M_x$ (ft-k) | -47.19 | -47.19 | 1744.48 | -1815.26 | 1744.48 | -1815.26 | 123.39 | 1000.17 | 1857.26 | -1672.01 | -814.93 |
| $M_y$ (ft-k) | 211.70 | 211.70 | 412.11 | -729.66 | 412.11 | -729.66 | - - | - - | - - | - - | - - |

**TABLE E.13 – Level 6 Lateral Loads on Each Frame (Kips)**

| Frame | Case 1 NS | Case 1 EW | Case 2 NS+ | Case 2 NS- | Case 2 EW+ | Case 2 EW- | Case 3 | Case 4 ++ | Case 4 +- | Case 4 -+ | Case 4 -- |
|---|---|---|---|---|---|---|---|---|---|---|---|
| B1 | 35.22 | -0.52 | 6.64 | 46.19 | -19.38 | 20.17 | 24.65 | 8.42 | -1.10 | 38.11 | 28.59 |
| B2 | 34.17 | 0.52 | 45.40 | 5.85 | 19.38 | -20.17 | 27.39 | 30.65 | 40.17 | 0.96 | 10.48 |
| A1 | -2.41 | 7.51 | -4.68 | 8.29 | 2.75 | 15.72 | 6.03 | -5.79 | -15.53 | 24.58 | 14.84 |
| A2 | 2.41 | 12.32 | 4.68 | -8.29 | 12.12 | -0.86 | 8.84 | 16.95 | 26.69 | -13.42 | -3.68 |
| A3 | -2.41 | 7.51 | -4.68 | 8.29 | 2.75 | 15.72 | 6.03 | -5.79 | -15.53 | 24.58 | 14.84 |
| A4 | 2.41 | 12.32 | 4.68 | -8.29 | 12.12 | -0.86 | 8.84 | 16.95 | 26.69 | -13.42 | -3.68 |

## Torsional Shear Values for each Frame

**TABLE E.14 - Level 1 Torsional Shears for Load Case 4+-**

| Frame | Torsional Coefficient (1/Ft) | Moment (Ft-K) | Torsional Shear (K) |
|---|---|---|---|
| B1 | -0.01111 | 930.69 | -10.34 |
| B2 | 0.01111 | 930.69 | 10.34 |
| A1 | -0.01136 | 930.69 | -10.58 |
| A2 | 0.01136 | 930.69 | 10.58 |
| A3 | -0.01136 | 930.69 | -10.58 |
| A4 | 0.01136 | 930.69 | 10.58 |

**TABLE E.15 - Level 2 Torsional Shears for Load Case 4+-**

| Frame | Torsional Coefficient (1/Ft) | Moment (Ft-K) | Torsional Shear (K) |
|---|---|---|---|
| B1 | -0.01111 | 1645.87 | -18.29 |
| B2 | 0.01111 | 1645.87 | 18.29 |
| A1 | -0.01136 | 1645.87 | -18.70 |
| A2 | 0.01136 | 1645.87 | 18.70 |
| A3 | -0.01136 | 1645.87 | -18.70 |
| A4 | 0.01136 | 1645.87 | 18.70 |

**TABLE E.16 - Level 3 Torsional Shears for Load Case 4+-**

| Frame | Torsional Coefficient (1/Ft) | Moment (Ft-K) | Torsional Shear (K) |
|---|---|---|---|
| B1 | -0.01111 | 1543.01 | -17.14 |
| B2 | 0.01111 | 1543.01 | 17.14 |
| A1 | -0.01136 | 1543.01 | -17.53 |
| A2 | 0.01136 | 1543.01 | 17.53 |
| A3 | -0.01136 | 1543.01 | -17.53 |
| A4 | 0.01136 | 1543.01 | 17.53 |

**TABLE E.17 - Level 4 Torsional Shears for Load Case 4+-**

| Frame | Torsional Coefficient (1/Ft) | Moment (Ft-K) | Torsional Shear (K) |
|---|---|---|---|
| B1 | -0.01111 | 1622.90 | -18.03 |
| B2 | 0.01111 | 1622.90 | 18.03 |
| A1 | -0.01136 | 1622.90 | -18.44 |
| A2 | 0.01136 | 1622.90 | 18.44 |
| A3 | -0.01136 | 1622.90 | -18.44 |
| A4 | 0.01136 | 1622.90 | 18.44 |

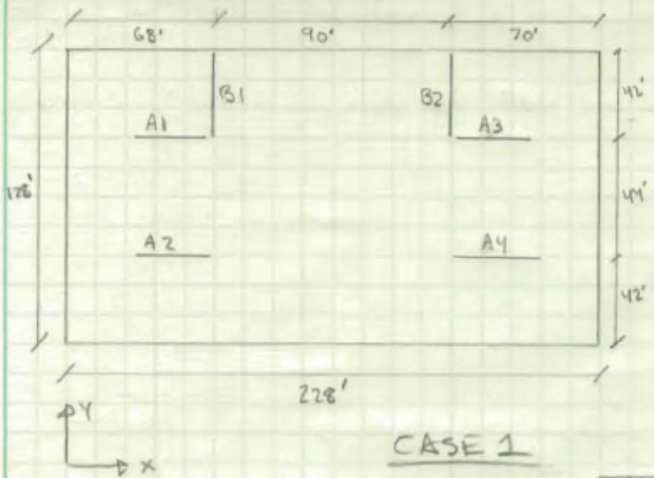
**TABLE E.18 - Level 5 Torsional Shears for Load Case 4+-**

| Frame | Torsional Coefficient (1/Ft) | Moment (Ft-K) | Torsional Shear (K) |
|---|---|---|---|
| B1 | -0.01111 | 1684.76 | -18.72 |
| B2 | 0.01111 | 1684.76 | 18.72 |
| A1 | -0.01136 | 1684.76 | -19.15 |
| A2 | 0.01136 | 1684.76 | 19.15 |
| A3 | -0.01136 | 1684.76 | -19.15 |
| A4 | 0.01136 | 1684.76 | 19.15 |

**TABLE E.19 - Level 6 Torsional Shears for Load Case 4+-**

| Frame | Torsional Coefficient (1/Ft) | Moment (Ft-K) | Torsional Shear (K) |
|---|---|---|---|
| B1 | -0.01111 | 1857.26 | -20.64 |
| B2 | 0.01111 | 1857.26 | 20.64 |
| A1 | -0.01136 | 1857.26 | -21.11 |
| A2 | 0.01136 | 1857.26 | 21.11 |
| A3 | -0.01136 | 1857.26 | -21.11 |
| A4 | 0.01136 | 1857.26 | 21.11 |

Lateral forces for each frame at Floor 6 | Tech Report #3 | Page 1 of 2



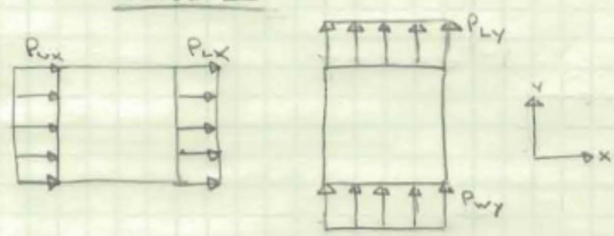

Floor 6

$P_{wx} = 15.53$ psf
$P_{wy} = 15.26$ psf
$P_{Lx} = 6.93$ psf
$P_{Ly} = 6.81$ psf

$K_{frame\,A} = 16.6$ k/in

$K_{frame\,B} = 161.8$ k/in

CASE 1




center of mass: $x = 113.68'$
$y = 69.34'$

center of rigidity: $x = 113'$
$y = 64'$

$e_y = 69.34 - 64 = 5.34$ ft
$e_x = 113.68 - 113 = 0.68$ ft

$P_x = [P_{wx} + P_{Lx}] B_x h = (15.53 + 6.93)(128)(13.79)$

$P_x = 39.64^k$

$P_y = [P_{wy} + P_{Ly}] B_y h = (15.26 + 6.81)(228)(13.79)$

$P_y = 69.39^k$

Direct shear

for frames B1 and B2: $F_{iy} = \frac{k_{iy} P_y}{\Sigma k_{iy}} = \frac{161.8(69.39)}{2(161.8)} = 34.7^k$

for frames A1-A4: $F_{ix} = \frac{k_{ix} P_x}{\Sigma k_{ix}} = \frac{16.6(39.64)}{4(16.6)} = 9.91^k$

torsional shear

$F_{it} = \frac{k_i d_i Pe}{\Sigma k_i d_i^2} \quad \theta_t = \frac{Pe}{\Sigma k_i d_i^2} \Rightarrow F_{it} = k_i d_i \theta_t$

for frames B1 and B2: $\theta_t = \frac{P_y e_x}{\Sigma k_i d_i^2} = \frac{(69.39)(.68)}{(161.8)(45^2 + 45^2)} = 7.201e\text{-}5$ rad

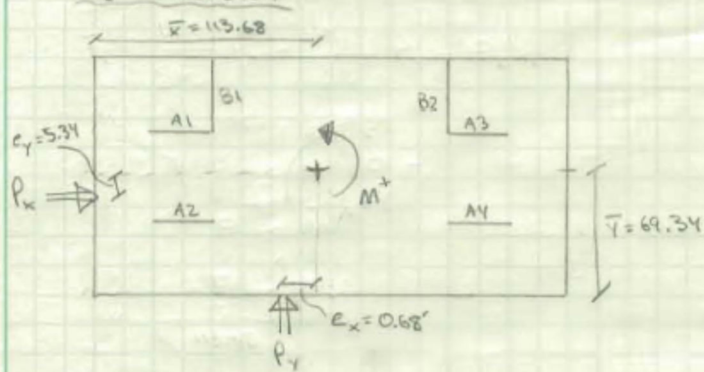
for frames A1 to A4: $\theta_t = \frac{P_x e_y}{\Sigma k_i d_i^2} = \frac{(39.64)(5.34)}{(16.6)(22)^2(4)} = .00659$ rad

| Lateral forces for each frame at Floor 6 | Tech Report # 3 | Page 2 of 2 |
|---|---|---|

Frames B1 and B2: $F_{it} = \theta_t k_i d_i = (7.201e\text{-}5)(161.8)(45) = .5243^k$

Frames A1, A2, A3, A4: $F_{it} = \theta_t k_i d_i = (.00659)(16.6)(22) = 2.407^k$

Sign convention




Total Shear

$$F_i = \frac{k_{xi}}{\Sigma k_{xi}} P_x + \frac{k_{yi}}{\Sigma k_{yi}} P_y + \theta_t k_i d_i$$

CASE 1 (NS)

$M = P_y e_x - P_x e_y = 69.39(-.68) - 0 = -47.19$

Frame B1: 0 + 34.7 + .5243 = 35.22
Frame B2: 0 + 34.7 − .5243 = 34.18
Frame A1: 0 + 0 − 2.407 = −2.407
Frame A2: 0 + 0 + 2.407 = 2.407
Frame A3: 0 + 0 − 2.407 = −2.407
Frame A4: 0 + 0 + 2.407 = 2.407

CASE 1 (EW)

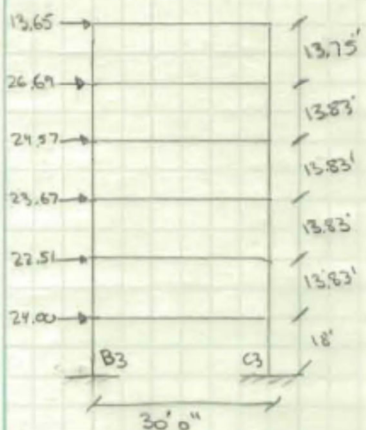
$M = P_y e_x - P_x e_y = 0 - 39.64(-5.34) = +211.68$

Frame B1: 0 + 0 + (−.5243) = −.5243
Frame B2: 0 + 0 + (.5243) = .5243
Frame A1: 9.91 + 0 + (−2.407) = 7.503
Frame A2: 9.91 + 0 + (+2.407) = 12.32
Frame A3: 9.91 + 0 + (−2.407) = 7.503
Frame A4: 9.91 + 0 + (+2.407) = 12.32

## APPENDIX F – OVERTURNING & STRENGTH CHECK CALCULATIONS

Overturning check | Tech Report #3 | Page 1 of 1

check overturning moment on Frame A2




$M_{overturning} = 24.00(18) + 22.51(31.83) + 23.67(45.66) + (24.57)(59.49) + 26.69(73.32) + 13.65(87.07)$

$M_{overturning} = 6836.49$ ft·k

$\text{Uplift force} = \frac{M}{L} = \frac{6836.49 \text{ ft·k}}{30 \text{ ft}} = 227.88^k$

Find the dead load for column C3

DL = Self weight + SDL

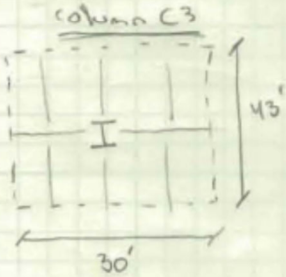
Floor 1: $D_L = \frac{2857.79(1000)}{228(128)} = 97.92$ psf

Floors 2-5: $D_L = \frac{2681.15(1000)}{228(128)} = 91.87$ psf

Floor 6: $D_L = \frac{2678.3(1000)}{228(128)} = 91.77$ psf

Roof: $D_L = 20$ psf

column C3




$P_{DL} = (97.92\text{psf})(30\times43) + (91.87\text{psf})(30\times43)(5) + 91.77\text{psf}(30\times43) + 20\text{psf}(30\times43)$

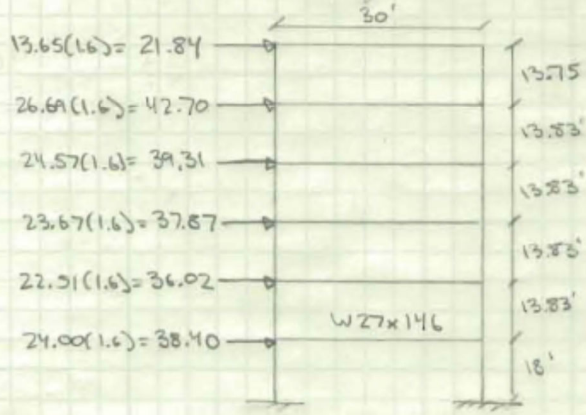
$P_{DL} = 863061.6^{lb} = 863.1^k$

$\boxed{863.1^k > 227.88^k \quad \checkmark \text{ok}}$

*overturning will not be a problem

Strength Checks | Tech Report # 3 | Page 1 of 2

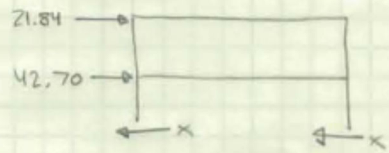
Evaluate frame A2 using the portal method





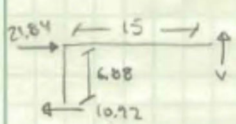
$\Sigma H = 0 = 21.84 - 2x$

$x = 10.92^k$



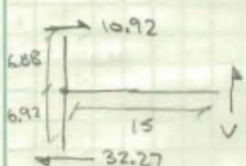

$\Sigma H = 0 = 21.84 + 42.70 - 2x$

$x = 32.27$



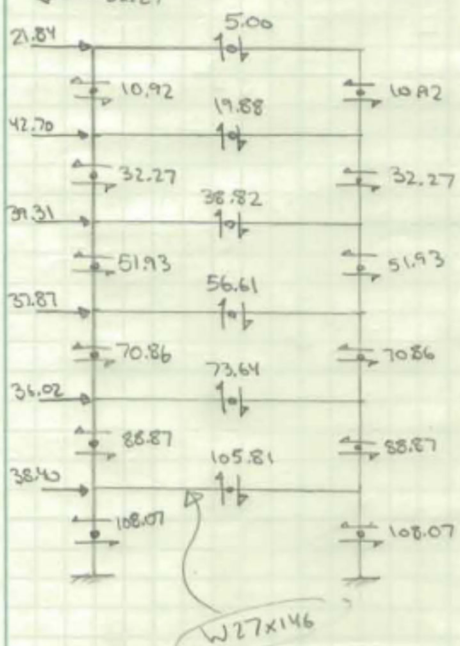

$\Sigma M = 10.92(6.88) - V(15) = 0$

$V = \frac{10.92(6.88)}{15} = 5$

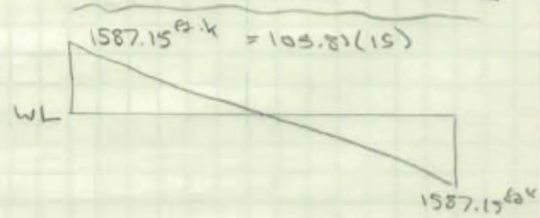



$\Sigma M = 32.27(6.92) + 10.92(6.88) - V(15)$

$V = 19.89^k$




Check the W27x146 on level 1




Level 1: DL = 97.92 psf

LL = 100 psf

$LL_{red} = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) = 50\left(.25 + \frac{15}{\sqrt{4(30)(43)(5)}}\right)$

$LL_{red} = 50(.343) \Rightarrow .343 < .4 \Rightarrow$ use .4

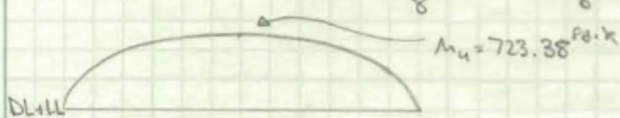
$LL = 50(.4) = 20\,psf$

$w_u = (1.2)(97.92)(43) + (1.6)(20)(43)$

$w_u = 5052.672 + 1376 = 6.43^k/ft$

Strength checks | Tech Report # 3 | Page 2 of 2

$w_u = 6.43^{k}/ft$ $\quad M_u = \frac{w_u l^2}{8} = \frac{6.43(30)^2}{8} = 723.38$ ft·k




$1587.15 > 723.38 \Rightarrow$ wind loads control

For a W27x146 $\Rightarrow \phi M_n = 1740$ ft·k $> 1587.15$ ✓ ok

check deflection

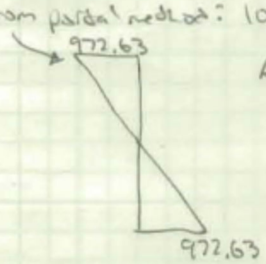
$$\Delta = \frac{5wl^4}{384EI} = \frac{5(6.43)(30)^4(1728)}{384(29000)(5660)} = 0.714 \text{ in}$$

$$\frac{L}{240} = \frac{30(12)}{240} = \frac{360}{240} = 1.5 \text{ in} \Rightarrow 1.5 \text{ in} > .714 \text{ in} \checkmark \text{ ok}$$

$\boxed{\text{W27x146 Beam is Adequate}}$

## COLUMN STRENGTH CHECK

- Check W14x311 at column line C3
- from portal method: $108.07(9) = 972.63$ ft·k




$A_T$ from overturning check $= 43(30) = 1290$ ft²

$P_{LL} = 20\text{psf}(1290 \text{ ft}^2)(5 \text{ stories}) = 129^k$

$P_{DL} = (97.92 + 91.87(4) + 91.77 + 70)(1290 \text{ ft}^2)$

$P_{DL} = 744.55$ ft·k

$P_u = 1.2(744.55) + 1.6(129) = 1099.86^k$

From table 6.1 in steel manual $\Rightarrow KL = 18'$
for a W14x311 column
$p = .295 \times 10^{-3}$ 1/k
$b_x = .398 \times 10^{-3}$ 1/ft·k

$$pP_u + b_x M_u = (.295e\text{-}3)(1099.86^k) + (.398e\text{-}3)(972.63) = .712$$

$.712 < 1.0$ ✓ ok

$\boxed{\text{Column is Adequate}}$