# Voorhees Replacement Hospital

# Voorhees, New Jersey

# Technical Report #3

Paul Stewart

Structural Option

Consultant: Dr. Ali M. Memari

December 1st, 2009

# Table of Contents

## Executive Summary

The Voorhees Replacement Hospital is a new hospital replacing the current Voorhees hospital due to its inability to expand and be renovated. The new building is 9 stories tall, approximately 140 feet tall. It consists of two parts, a main bed tower, and a services building.

In the third technical report an in depth analysis is preformed of the existing lateral system. The system is composed of moment and braced frames in both directions that are placed throughout the floor plans. The main bed tower and the services building are separated by a 1" expansion joint causing their two lateral systems to act independently. Since this is the case, only the services building is analyzed in this report.

A computer model of the hospital's services building is created for this report using ETABS. The model includes the entire lateral system of the building, while excluding the gravity system. Wind and seismic loads applied to this model are determined using hand calculations found in Appendix B and Appendix C. The model is used throughout the report to compare and assist with hand calculations.

The distribution of lateral forces to the braced frames and the moment frames is determined based on the stiffness of each frame. This method of distributing lateral forces is found through hand calculations by determining the amount of direct and torsional shear in each frame in the North – South direction on the $4^{th}$ floor. The center of rigidity is also found using ETABS and is compared to the center of mass in order to find the eccentricity of each floor in the building.

Lateral loads are also considered for the drift and strength requirements of the wind and seismic forces. These values are found in this report through ETABS and compared to values found using the Portal and Cantilever Methods of a specified moment frame. The values of the two methods are found to be similar and are also found to pass the strength and serviceability requirements.

The building's overall torsion and overturning moment are also found in this report through hand calculations. The building's torsion is found to be relatively small, while the overturning moment is found to rather large. Both the torsional effects, and the overturning moments will have to be considered in any future reports.

Specified members throughout the building are also checked for strength requirements. A brace in a braced frame is checked for strength, while a column located in a moment frame is also checked. Both spot checks are found to be acceptable by a sizable amount. Since this is the case, it is assumed that these members are sized for drift instead of strength.

## Introduction

The Virtua Voorhees Replacement Hospital is located in Voorhees, New Jersey (Latitude: 39.84° Longitude: -74.93°), immediately off Rt. 73. It will be replacing the old Voorhees hospital because of its inability to be renovated. The new hospital will have 9 floors, starting with a Garden Level continuing up through Floor 8. The building is broken up into two main areas, the main bed tower (referred to as Building A, or Northern Building in this report), and a services building (referred to as Building B, or Southern Building in this report). The building is also broken up into 7 smaller zones, for ease of reference in the drawings. Figure 1 shows how the building is split up.

The main bed tower, zones 1-3, is 8 levels and holds 350 individual patient rooms. It is a curved building with a curtain wall facing the majority of the site. This curtain wall allows residents to get an excellent view of the site as well as the wetlands that were protected during construction. The majority of the 8 floors in the main tower have the same floor plan with minor differences.

The services building, which holds zones 4 through 7, is attached to the main bed tower via a thin corridor. The services building houses most of the labs, offices, and surgical rooms needed in the hospital. These services are located between the ground floor and the 5$^{th}$ floor. Above the 5$^{th}$ floor, the building narrows, to match the width of the corridor connecting the bed tower and the services building. Mechanical spaces start on the 6$^{th}$ floor and continue up to the 9$^{th}$ floor. The services building also allows for future growth, with the potential to add more space on top of zone 6.

For the third technical report of the Voorhees Replacement Hospital the lateral system of the southern building, holding zones 4-6, is analyzed. Many different factors are looked at during this report. An ETABS model is created to compare to, and help with, the hand calculations preformed in this report. The lateral system is checked for story drift and overall drift and compared to serviceability standards and code requirements. Also, the building is checked for torsion effects, and overturning moments. Finally, spot checks are preformed on the building's lateral members to insure that they have enough strength to carry the applied loads.

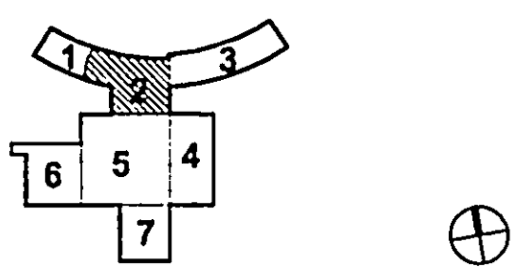


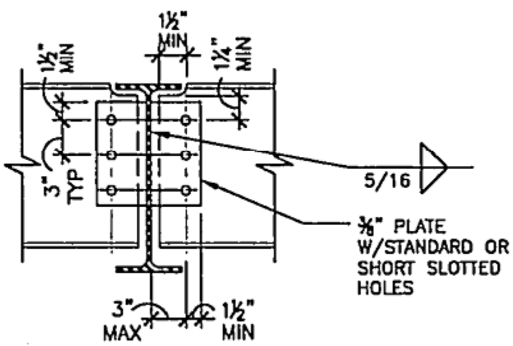

*Figure 1 – Key Plan*

## Structural System Overview

*Floor System*

The floor system of the Voorhees hospital is a composite steel/concrete system. In Building B the bay sizes are typically 31' – 4" x 31' – 4" or 31' – 4" x 29' – 4". 3 – ½" light weight concrete sits on top of 3" x 18 Gage composite steel deck. The total thickness of the concrete is 6 – ½" with 6x6-W2.1xW2.1 WWF.

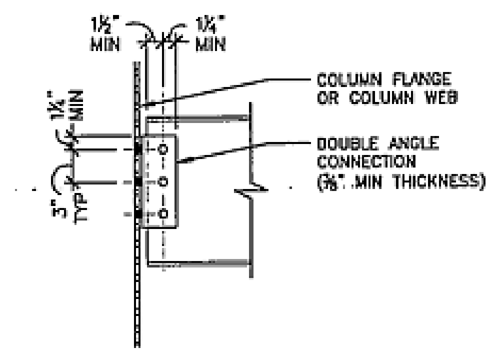
The steel deck is connected to the W-shape beams by ¾" diameter x 5" long shear studs allowing the two systems to work together in composite action. The beams then frame into larger W-shape girders via a single angle connection or a single plate connection. The beams are coped allowing them to connect to the girder's web so that the composite deck can sit on both the beams and the girders. A typical beam to girder connection is pictured below in Figure 2.




CONNECTION SCHEDULE

| BEAM SIZE | NUMBER OF ¾"ø A-325N BOLTS |
|---|---|
| W12, W14, C12 | 3 |
| W16 | 4 |
| W18 | 5 |
| W21 | 6 |
| W24, W27 | 7 |
| W30 & UP | 8 |

*Figure 2: Typical Single Plate Beam to Beam Connection Detail*




CONNECTION SCHEDULE

| BEAM SIZE | NUMBER OF ¾"ø A-325N BOLTS |
|---|---|
| W8 | 2 |
| W12, W14, W16 | 3 |
| W18, W21x44, 50, 57 | 4 |
| W21x62 & UP<br>W24x55, 62, 68 | 5 |
| W24x76 & UP | 6 |
| W30, W33, W36 | 7 |

*Figure 3: Typical Double Angle Beam to Column Connection Detail*

The W-shape girders frame into W-shape columns by either double angle connections, or by moment connections. The double angle connection is shown above in Figure 3.

*Columns*

Typical columns for the Voorhees Replacement Hospital are W14's. The columns are spliced every two floors, 4'-0" above the floor with either a bolted column splice or a welded splice making these connections rigid. The columns located in zone 6 are sized larger than typical to allow for future expansion to be built above.

*Lateral System*

The Voorhees Replacement Hospital uses a combination of braced framing and moment connections for its lateral system. In Building B the composite floor system and the roof deck act as a diaphragm to transfer loads to either the braced frames or the moment connections. In this building both braced frames and moment connections are used in both directions. The moment connections are typically spanning the exterior of the building while the braced frames are placed throughout the building, but typically are located in zone 5. The braced system consists of diagonal, square, HSS connected to W shapes. The braced frames are of two different styles, the bracing either frames from corner to corner of the bay, or from the lower corner to the midpoint of the top beam.

Building A and Building B are separated between zones 2 and 5 with a 1" expansion joint. This expansion joint allows the two building's lateral systems to act separately when forces are applied. Since the buildings act separately, only Building B will be analyzed in this report.

## Gravity Loads

Building live loads are determined by referencing ASCE 7-05. Table 1 below outlines the live loads:

| Live Loads | | |
|---|---|---|
| **Load Description** | **ASCE 07-05 Load (psf)** | **Assumed Partition Load (psf)** |
| **Labs** | 60 | 20 |
| **Operating Rooms** | 60 | 20 |
| **Private Rooms/Wards** | 40 | 20 |
| **Offices** | 50 | 20 |
| **Corridors above the 1st floor** | 80 | N/A |
| **Lobbies/1st floor corridors** | 100 | N/A |
| **Stairs and Exits** | 100 | N/A |
| **Storage** | 125 | N/A |
| **Mechanical Room** | 125 | N/A |
| **Roof Garden** | 100 | N/A |
| **Roof** | 20 | N/A |

*Table 1: Live Loads*

For the calculations in this report, a superimposed dead load of 15 psf is used to account for mechanical and other specialty equipment.

## Codes & Design Standards

*Codes Used for this Report*

American Concrete Institute (ACI 318), Building Code Requirements for Structural Concrete

American Institute of Steel Construction (AISC), Steel Construction Manual

International Building Code (IBC) 2006

American Society of Civil Engineers (ASCE 7-05), Minimum Design Loads for Buildings and Other Structures, 2005

*Material Strength Requirement Summary:*

*Cast-in-place Concrete:*

$f'_c$ = 3,500 psi @ 28 days for all lightweight concrete on metal decking

$f'_c$ = 4,000 psi @ 28 days for all other concrete types

*Concrete Masonry:*

Concrete Masonry Units: ASTM C90 Type “N-1”

Masonry Grout: $f'_c$ = 3,000 psi @ 28 days

Masonry Mortar: ASTM C270 (Type S uno)

*Steel Reinforcing:*

Reinforcing Bars: ASTM A615 (Grade 60)

Welded Bars & Anchors: ASTM A706 (Grade 60)

Deformed Bar Anchors: ASTM A496

Epoxy-Coated Reinforcing Bars: ASTM A775 or ASTM A934

Welded Wire Fabric: ASTM A185

*Structural Steel:*

W & WT Shapes: ASTM A992, $F_y = 50$ ksi

Plates & Shapes Other Than W: ASTM A36, $F_y = 36$ ksi

Rectangular HSS: ASTM A500, Grade B, $F_y = 46$ ksi

Round HSS: ASTM A500, Grade B, $F_y = 42$ ksi

Pipes: ASTM A53, Type E or S, Grade B, $F_y = 35$ ksi

Bolts: ASTM F1554, $F_y = 36$ ksi

Expansion Bolts: Hilti, Rawl, Thunderstuds, or National Fasteners

Adhesive Anchors/Grout: Sika, Hilti, Epcon

Headed Studs/Shear Connectors: ASTM A108

*Welds:*

All Types: E70XX

## Load Considerations

### Load Path

Lateral forces in both directions are transferred in essentially the same manner. The lateral forces that act on the building are transferred through the composite floor system or roof deck to either the moment frames or the braced frames, depending on the location in the building. The forces are then transferred to the columns which then are distributed down to the foundations. Since all member sizes are approximately the same size, there does not appear to be any weak links, or areas of concern. A more in depth analysis will be preformed however later in this report to find any possible weak links.

### Load Combinations

Five load cases will be considered in this report, seismic in the E-W direction, seismic in the N-S direction, wind case 1 in the N-S direction, wind case 1 in the E-W direction, and wind case 2. The wind cases are defined in ASCE 7-05 figure 6-9. Case 1 has 100% of the wind load in one direction, while case 2 has 75% of the wind load acting in each direction. Cases 3 and 4 are not checked and are assumed not to control due to the large spread out lateral frame, and the low eccentricity. They will be checked however in future studies. All lateral force values that are used can be found in Appendix B and C.

The following load combinations are considered in this report and are taken from section 2.3 in ASCE 7-05:

1.4(D + F)
1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)
1.2D + 1.6(Lr or S or R) + (L or 0.8W)
1.2D + 1.6W + L + 0.5(Lr or S or R)
1.2D + 1.0E + L + 0.2S
0.9D + 1.6W + 1.6H
0.9D + 1.0E + 1.6H

Because many of the factors listed above do not affect the lateral design, only combinations that involve dead, wind, and seismic loads are used for this report. The controlling equation is 1.2D + 1.0E + 1.0L + 0.2S. This is to be expected given that the seismic loads control in North – South direction over the wind loads, and the E-W wind and seismic loads are very similar.

## ETABS Model

The lateral system of Building B is modeled with ETABS. This model did not include any of the gravity columns or beams, only the braced frames and moment frames are modeled. The columns, braces, floor system, and the lateral beams are all modeled to match the sizes of the designed plans. Column splices and beam moment frame connections are assumed to be perfectly rigid. All columns are restrained at the base, while the braces are given partial fixity, releasing moment 3-3. The floor system is modeled to match the designed floor system, and is assumed to create a rigid diaphragm. Gravity loads are applied to the structure by giving uniform weight to the floor diaphragm. All five load cases stated above are used in the ETABS model. The seismic load cases are applied to the center of mass with no eccentricity. The ETABS model is shown below in Figure 4.

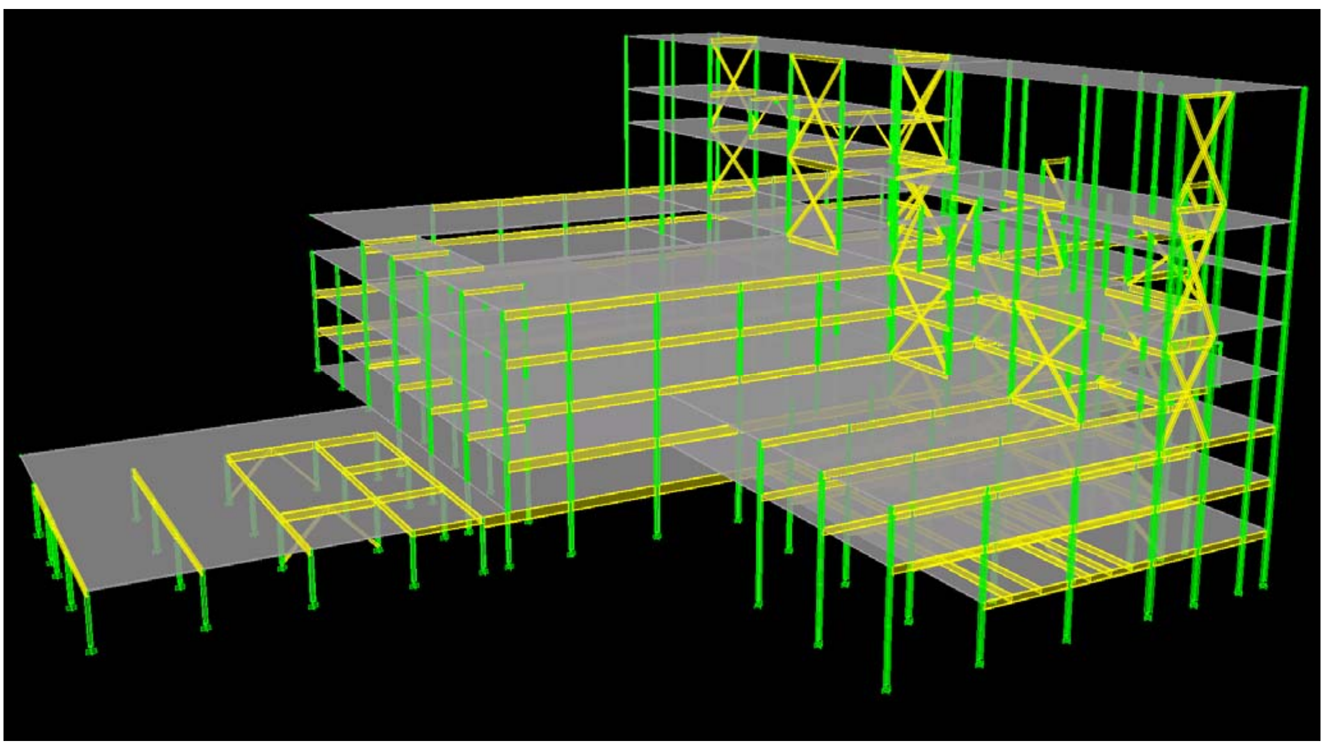


*Figure 4 – ETABS model*

## Lateral Force Analysis

### Wind Analysis

For the wind load calculations, ASCE-07 2005 is used. The values Building B are listed in Tables 2. The calculations can be found in Appendix B.

| Level | Height Above Ground (ft) | Wind Forces | | | | | |
|---|---|---|---|---|---|---|---|
| | | Load (kip) | | Shear (kip) | | Moment (ft-kip) | |
| | | N-S | E-W | N-S | E-W | N-S | E-W |
| 9 | 139.33 | 122.9 | 124.6 | 0 | 0 | 17123.7 | 17360.5 |
| 8 | 117.33 | 199.3 | 202.1 | 122.9 | 124.6 | 23383.9 | 23712.4 |
| 7 | 103.33 | 151.7 | 153.9 | 322.2 | 326.7 | 15902.5 | 15902.5 |
| 6 | 89.33 | 149.2 | 151.4 | 473.9 | 480.6 | 13328.0 | 13524.6 |
| 5 | 75.33 | 146.4 | 148.4 | 623.1 | 632.0 | 11028.3 | 11179.0 |
| 4 | 61.33 | 149.9 | 151.9 | 769.5 | 780.4 | 9193.4 | 9316.0 |
| 3 | 46.00 | 152.5 | 154.6 | 919.4 | 932.3 | 7015.0 | 7111.6 |
| 2 | 30.66 | 146.9 | 148.9 | 1071.9 | 1086.9 | 4504.0 | 4565.3 |
| 1 | 15.33 | 139.1 | 141.0 | 1218.8 | 1235.8 | 2132.4 | 2161.5 |
| **Total** | | **1357.9** | **1376.8** | **1357.9** | **1376.8** | **103611.2** | **104833.4** |

*Table 2 – Wind Forces on Building B*

**Seismic Analysis**

The seismic loads for this report are calculated using ASCE 7-05. It is assumed that due to the added geopiers used to densify the soil, that the site class is now D. The found seismic loads are listed in the Tables 3 and 4 below. The calculations can be found in Appendix C.

| Building B N-S Seismic Loads | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Level** | **Story Weight (kip)** | **Height (ft.)** | **$w_x h_x^k$** | **$C_{vx}$** | **Lateral Force (kip)** | **Story Shear (kip)** | **Moment (ft-kip)** |
| **R** | 1556.8 | 139.33 | 466234.47 | 0.164 | 252.125 | 252.125 | 35128.580 |
| **8** | 788.8 | 117.33 | 193702.12 | 0.068 | 104.748 | 356.873 | 12290.089 |
| **7** | 1294.5 | 103.33 | 274494.46 | 0.096 | 148.438 | 505.311 | 15338.101 |
| **6** | 515.75 | 89.33 | 92436.0 | 0.032 | 49.987 | 555.298 | 4465.294 |
| **5** | 3350.9 | 75.33 | 493241.3 | 0.173 | 266.729 | 822.027 | 20092.733 |
| **4** | 4709.4 | 61.33 | 546673.3 | 0.192 | 295.624 | 1117.651 | 18130.609 |
| **3** | 5212.5 | 46 | 434041.6 | 0.152 | 234.716 | 1352.367 | 10796.942 |
| **2** | 4133.2 | 30.66 | 215415.7 | 0.076 | 116.490 | 1468.857 | 3571.585 |
| **1** | 5592.2 | 15.33 | 130883.2 | 0.046 | 70.778 | 1539.635 | 1085.020 |
| **Sum** | **27154.05** | | **2847122.1** | **1.000** | **1539.635** | | **120898.952** |

*Table 3: The Southern Building N-S Seismic Loads*

| Building B E-W Seismic Loads | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Level** | **Story Weight (kip)** | **Height (ft.)** | **$w_x h_x^k$** | **$C_{vx}$** | **Lateral Force (kip)** | **Story Shear (kip** | **Moment (ft-kip)** |
| **R** | 1556.8 | 139.33 | 466234.470 | 0.164 | 216.107 | 216.107 | 30110.211 |
| **8** | 788.8 | 117.33 | 193702.122 | 0.068 | 89.784 | 305.891 | 10534.362 |
| **7** | 1294.5 | 103.33 | 274494.458 | 0.096 | 127.233 | 433.124 | 13146.943 |
| **6** | 515.75 | 89.33 | 92436.003 | 0.032 | 42.846 | 475.969 | 3827.395 |
| **5** | 3350.9 | 75.33 | 493241.341 | 0.173 | 228.625 | 704.595 | 17222.343 |
| **4** | 4709.4 | 61.33 | 546673.288 | 0.192 | 253.392 | 957.986 | 15540.522 |
| **3** | 5212.5 | 46 | 434041.608 | 0.152 | 201.185 | 1159.172 | 9254.522 |
| **2** | 4133.2 | 30.66 | 215415.651 | 0.076 | 99.849 | 1259.020 | 3061.358 |
| **1** | 5592.2 | 15.33 | 130883.194 | 0.046 | 60.666 | 1319.687 | 930.017 |
| **Sum** | **27154.05** | | **2847122.135** | **1.000** | **1319.687** | | **103627.673** |

*Table 4: The Southern Building E-W Seismic Loads*

The seismic values are found to control over the wind values in the North – South direction, while the wind values are found to control over the seismic in the East- West direction.

## Lateral Force Distribution

Distribution of lateral forces to the braced frames is determined based on the relative stiffness of each braced frame. Floor diaphragms are assumed to be infinitely rigid, and therefore distribute lateral loads to each frame based on their stiffness. The center of rigidity at level 4 is calculated using the south west corner of the building's grid line as the origin, and used to calculate the direct shear. Hand calculations are preformed for the North – South direction of level 4 and can be found in Appendix F. A more in depth analysis of the Shear from Torsion values can be found in the Torsion section of this report. The results of this hand calculation can be found in Table 6 below.

| Level 4 Shear Forces Due to North South Seismic | | | |
|---|---|---|---|
| Frame | Direct Shear | Shear from Torsion | Net Shear |
| | (kip) | (kip) | (kip) |
| La | 58.75 | -167.2 | -108.45 |
| Lb | 58.75 | -167.2 | -108.45 |
| M6a | 67.97 | 166.67 | 234.64 |
| M6b | 67.97 | 166.67 | 234.64 |

*Table 6 – Shear Forces Due to North – South Seismic Loads*

Table 7 below shows the x and y components, or the East – West and North – South components respectively, of the center of rigidity and the center of mass for each level as calculated by ETABS. The origin of the model is taken to be the south west corner of the grid line.

| ETABS Center of Mass and Center of Rigidity | | | | |
|---|---|---|---|---|
| Level | X Center of Mass | Y Center of Mass | X Center of Rigidity | Y Center of Rigidity |
| | (ft) | (ft) | (ft) | (ft) |
| 9 | 306.4 | 158.6 | 309.5 | 148.2 |
| 8 | 307.1 | 81.9 | 306.5 | 86.2 |
| 7 | 307.0 | 154.0 | 306.5 | 174.6 |
| 6 | 318.1 | 232.1 | 317.9 | 236.7 |
| 5 | 287.6 | 114.4 | 306.4 | 174.4 |
| 4 | 278.0 | 113.4 | 303.9 | 164.7 |
| 3 | 278.5 | 130.8 | 302.2 | 152.0 |
| 2 | 285.7 | 129.3 | 300.6 | 154.3 |
| 1 | 245.5 | 126.3 | 275.5 | 125.1 |

*Table 7 – Center of Mass and Rigidity Calculated by ETABS*

The difference in the center of mass and the center of rigidity appear to be minimal in the X direction, or the East – West direction. The difference in the Y direction, or the North – South direction however, appears to be more significant than the X direction, however the values are still small.

## Drift

Seismic and wind drifts are computed by the portal and cantilever methods, and through ETABS. The values are then compared to code drift limitations. Drift due to wind is compared to h/400 for the entire building in both directions. Seismic drift is compared to $0.015h_{sx}$, which is the allowable story drift, $\Delta_a$, per ASCE7-05, Table 12.12-1. The results are below in Tables 8, 9, 10, 11, and 12. The portal and cantilever calculations can be found in Appendix D

| Controlling Wind Drift: North – South Direction Through Portal/Cantilever Method | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Story | Story Height | Story Drift | Allowable Story Drift | | | Total Drift | Allowable Total Drift | | |
| | (ft) | (in) | (in) | | | (in) | (in) | | |
| 5 | 75.33 | 0.001340 | ≤ | 0.46 | Acceptable | 0.014860 | ≤ | 2.30 | Acceptable |
| 4 | 61.33 | 0.002514 | ≤ | 0.46 | Acceptable | 0.013520 | ≤ | 1.84 | Acceptable |
| 3 | 46 | 0.003298 | ≤ | 0.46 | Acceptable | 0.011006 | ≤ | 1.38 | Acceptable |
| 2 | 30.66 | 0.003864 | ≤ | 0.46 | Acceptable | 0.007708 | ≤ | 0.92 | Acceptable |
| 1 | 15.33 | 0.003844 | ≤ | 0.46 | Acceptable | 0.003844 | ≤ | 0.46 | Acceptable |

*Table 8 – Controlling Wind Drift through Portal and Cantilever Method*

| Controlling Wind Drift: East – West Direction | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Story | Story Height | Story Drift | Allowable Story Drift | | | Total Drift | Allowable Total Drift | | |
| | (ft) | (in) | (in) | | | (in) | (in) | | |
| 9 | 138.79 | 0.011076 | ≤ | 0.65 | Acceptable | 0.083184 | ≤ | 4.33 | Acceptable |
| 8 | 117.33 | 0.015192 | ≤ | 0.46 | Acceptable | 0.072108 | ≤ | 3.68 | Acceptable |
| 7 | 103.33 | 0.01998 | ≤ | 0.46 | Acceptable | 0.056916 | ≤ | 3.22 | Acceptable |
| 6 | 89.33 | 0.011472 | ≤ | 0.46 | Acceptable | 0.036936 | ≤ | 2.76 | Acceptable |
| 5 | 75.33 | 0.005184 | ≤ | 0.46 | Acceptable | 0.025464 | ≤ | 2.30 | Acceptable |
| 4 | 61.33 | 0.001032 | ≤ | 0.46 | Acceptable | 0.02028 | ≤ | 1.84 | Acceptable |
| 3 | 46 | 0.00738 | ≤ | 0.46 | Acceptable | 0.019248 | ≤ | 1.38 | Acceptable |
| 2 | 30.66 | 0.008616 | ≤ | 0.46 | Acceptable | 0.011868 | ≤ | 0.92 | Acceptable |
| 1 | 15.33 | 0.003252 | ≤ | 0.46 | Acceptable | 0.003252 | ≤ | 0.46 | Acceptable |

*Table 9 – Controlling Wind Drift in the East – West Direction*

| Controlling Wind Drift: North – South Direction | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Story | Story Height | Story Drift | Allowable Story Drift | | | Total Drift | Allowable Total Drift | | |
| | (ft) | (in) | (in) | | | (in) | (in) | | |
| 9 | 138.79 | 0.025056 | ≤ | 0.65 | Acceptable | 0.12828 | ≤ | 4.33 | Acceptable |
| 8 | 117.33 | 0.046176 | ≤ | 0.46 | Acceptable | 0.10322 | ≤ | 3.68 | Acceptable |
| 7 | 103.33 | 0.026928 | ≤ | 0.46 | Acceptable | 0.05705 | ≤ | 3.22 | Acceptable |
| 6 | 89.33 | 0.017844 | ≤ | 0.46 | Acceptable | 0.03012 | ≤ | 2.76 | Acceptable |
| 5 | 75.33 | 0.000552 | ≤ | 0.46 | Acceptable | 0.01228 | ≤ | 2.30 | Acceptable |
| 4 | 61.33 | 0.0021 | ≤ | 0.46 | Acceptable | 0.01172 | ≤ | 1.84 | Acceptable |
| 3 | 46 | 0.00414 | ≤ | 0.46 | Acceptable | 0.00962 | ≤ | 1.38 | Acceptable |
| 2 | 30.66 | 0.004368 | ≤ | 0.46 | Acceptable | 0.00548 | ≤ | 0.92 | Acceptable |
| 1 | 15.33 | 0.001116 | ≤ | 0.46 | Acceptable | 0.00112 | ≤ | 0.46 | Acceptable |

*Table 10 – Controlling Wind Drift in the North – South Direction*

| Seismic: East – West Direction | | | | | | |
|---|---|---|---|---|---|---|
| Story | Story Height | Story Drift | Total Drift | Allowable Drift | | |
| | (ft) | (in) | (in) | (in) | | |
| 9 | 138.79 | 0.015528 | 0.104808 | ≤ | 2.08185 | Acceptable |
| 8 | 117.33 | 0.022908 | 0.08928 | ≤ | 1.75995 | Acceptable |
| 7 | 103.33 | 0.012564 | 0.066372 | ≤ | 1.54995 | Acceptable |
| 6 | 89.33 | 0.01254 | 0.053808 | ≤ | 1.33995 | Acceptable |
| 5 | 75.33 | 0.008952 | 0.041268 | ≤ | 1.12995 | Acceptable |
| 4 | 61.33 | 0.00444 | 0.032316 | ≤ | 0.91995 | Acceptable |
| 3 | 46 | 0.01254 | 0.027876 | ≤ | 0.69 | Acceptable |
| 2 | 30.66 | 0.012612 | 0.015336 | ≤ | 0.4599 | Acceptable |
| 1 | 15.33 | 0.002724 | 0.002724 | ≤ | 0.22995 | Acceptable |

*Table 11 – Seismic Drift in the East – West Direction*

| Seismic: North – South Direction | | | | | | |
|---|---|---|---|---|---|---|
| Story | Story Height | Story Drift | Total Drift | Allowable Drift | | |
| | (ft) | (in) | (in) | (in) | | |
| 9 | 138.79 | 0.017244 | 0.076692 | ≤ | 2.08185 | Acceptable |
| 8 | 117.33 | 0.02244 | 0.059448 | ≤ | 1.75995 | Acceptable |
| 7 | 103.33 | 0.012564 | 0.037008 | ≤ | 1.54995 | Acceptable |
| 6 | 89.33 | 0.007164 | 0.024444 | ≤ | 1.33995 | Acceptable |
| 5 | 75.33 | 0.001116 | 0.01728 | ≤ | 1.12995 | Acceptable |
| 4 | 61.33 | 0.002952 | 0.016164 | ≤ | 0.91995 | Acceptable |
| 3 | 46 | 0.006072 | 0.013212 | ≤ | 0.69 | Acceptable |
| 2 | 30.66 | 0.005796 | 0.00714 | ≤ | 0.4599 | Acceptable |
| 1 | 15.33 | 0.001344 | 0.001344 | ≤ | 0.22995 | Acceptable |

*Table 12 – Seismic Drift in the North – South Direction*

As displayed in the tables above, all the values for drift are acceptable per serviceability requirements and code requirements. The hand calculated drifts preformed by the Portal and Cantilever methods are found to be similar to the ETABS calculated drifts. This shows that the ETABS model is an accurate representation of the building for drift.

## Torsion

Since the center of mass and the center of rigidity do not differ by a large amount, it is to be expected that the building's torsion is relatively small. The total building torsion consists of two parts, $M_t$, which is the inherent torsional moment. This is found by multiplying the distance between the center of rigidity and the center of mass by the story shear. The inherent torsional moment is then added to the accidental torsional moment, $M_{ta}$. $M_{ta}$ is found by multiplying the story shear by 5% of the building width at the specific level. The inherent torsional moment is added to the accidental torsional moment to find the total torsional moment, M. The values of $M_t$, $M_{ta}$, and M can be found in Table 13 below.

| Overall Building Torsion | | | | | | |
|---|---|---|---|---|---|---|
| Story | North – South Building Torsion | | | East – West Building Torsion | | |
| | $M_t$ | $M_{ta}$ | $M_{total}$ | $M_t$ | $M_{ta}$ | $M_{total}$ |
| | (kip-ft) | (kip-ft) | (kip-ft) | (kip-ft) | (kip-ft) | (kip-ft) |
| 9 | 781.59 | 567.2813 | 1348.869 | 1295.8 | 2552.244 | 3848.084 |
| 8 | 62.849 | 235.683 | 298.5318 | 869.03 | 4139.715 | 5008.745 |
| 7 | 74.219 | 333.9855 | 408.2045 | 3170.3 | 3152.411 | 6322.751 |
| 6 | 9.9974 | 112.4708 | 122.4682 | 696.44 | 3101.202 | 3797.642 |
| 5 | 5014.5 | 3636.316 | 8650.822 | 8904.0 | 3039.751 | 11943.75 |
| 4 | 7656.7 | 4030.242 | 11686.9 | 7792.5 | 3111.444 | 10903.91 |
| 3 | 5562.8 | 3199.883 | 8762.652 | 3277.5 | 3166.749 | 6444.269 |
| 2 | 1735.7 | 1702.676 | 3438.377 | 3722.5 | 3049.993 | 6772.493 |
| 1 | 2123.3 | 1080.532 | 3203.872 | 169.2 | 2888.174 | 3057.374 |
| Sum | | | **37920.7** | | | **58099.02** |

*Table 13 – Overall Building Torsion*

The torsional shear is also calculated for braced frames in the North – South direction of level 4. The results can be found below in Table 14. The values are found using the following formula:

$$F_{it} = (k_i * d_i * M_t) / (k_j * d_j^2)$$

Where $k_i$ is the stiffness of the frame, $d_i$, is the distance of the frame from the center of rigidity, $M_t$, is the total building torsion at level I, $k_{j,}$ is the sum of all the frame stiffness's, and $d_j$, is the sum of all the distances from the frames to the center of rigidity. The torsional shear values are then added to the direct shear values to find the total shear per frame.

| Level 4 Shear Forces Due to North South Seismic | | | |
|---|---|---|---|
| Frame | Direct Shear | Shear from Torsion | Net Shear |
| | (kip) | (kip) | (kip) |
| La | 58.75 | -91.17 | -32.42 |
| Lb | 58.75 | -91.17 | -32.42 |
| M6a | 67.97 | 90.88 | 158.85 |
| M6b | 67.97 | 90.88 | 158.85 |

*Table 14 – Level 4 Shear Forces due to Seismic Loading*

## Overturning

An analysis is performed to determine the overturning moments caused by the controlling forces. The controlling force in the North – South direction is the seismic load, while the controlling force in the East – West is the wind load. The overturning moment is found to be rather large and will need to be looked at in the future to determine if the foundations will be able to take the moment. A future analysis of the foundations will determine if they are of adequate size and depth to take the loads caused by the seismic and wind loads. The overturning moments can be found in Table 15 below.

| Level | Height Above Ground (ft) | Overturning Moments | | | | | |
|---|---|---|---|---|---|---|---|
| | | Load (kip) | | Shear (kip) | | Overturning Moment (ft-kip) | |
| | | N-S | E-W | N-S | E-W | N-S | E-W |
| 9 | 139.33 | 252.125 | 124.6 | 252.125 | 0 | 35128.580 | 17360.5 |
| 8 | 117.33 | 104.748 | 202.1 | 356.873 | 124.6 | 12290.089 | 23712.4 |
| 7 | 103.33 | 148.438 | 153.9 | 505.311 | 326.7 | 15338.101 | 15902.5 |
| 6 | 89.33 | 49.987 | 151.4 | 555.298 | 480.6 | 4465.294 | 13524.6 |
| 5 | 75.33 | 266.729 | 148.4 | 822.027 | 632.0 | 20092.733 | 11179.0 |
| 4 | 61.33 | 295.624 | 151.9 | 1117.651 | 780.4 | 18130.609 | 9316.0 |
| 3 | 46.00 | 234.716 | 154.6 | 1352.367 | 932.3 | 10796.942 | 7111.6 |
| 2 | 30.66 | 116.490 | 148.9 | 1468.857 | 1086.9 | 3571.585 | 4565.3 |
| 1 | 15.33 | 70.778 | 141.0 | 1539.635 | 1235.8 | 1085.020 | 2161.5 |
| **Total** | | **1539.635** | **1376.8** | **1539.635** | **1376.8** | **120898** | **104833** |

*Table 15 – Overturning Moments*

## Spot Checks

Lateral member spot checks are preformed for an HSS12x12x½ brace in a braced frame, and for a W14x211 column in a moment frame. Figure 5 shows the brace examined, while Figure 6 shows the column examined. The forces acting on the brace are determined using ETABS. The forces acting on the column are determined using the Portal and Cantilever methods which can be seen in Appendix D. The spot check analysis can be found in Appendix E.

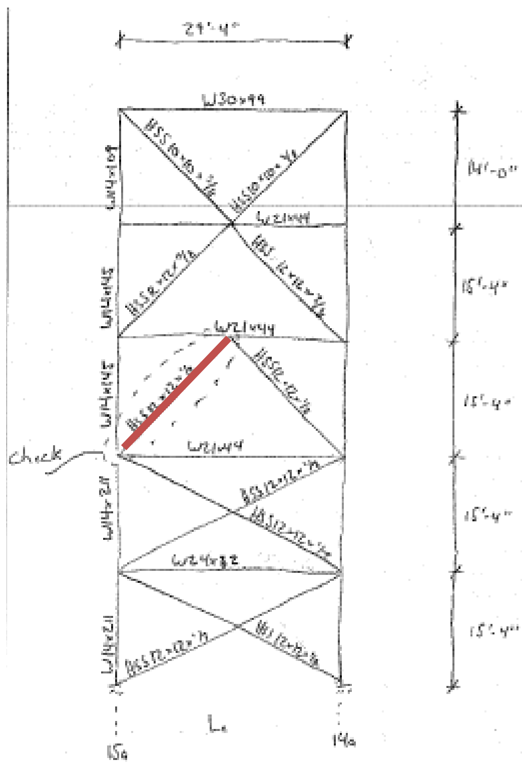



*Figure 5 – Brace Examined in Spot Check*

The spot check for the brace in the braced frame is found to be adequate. The load taken by the brace is found to be 70.15 kips, while the capacity of the brace multiplied by ϕ, which is found in Table 4.4 of AISC, is 698 kips. This value is much greater than what is applied; therefore the brace is of adequate size.

The spot check for the column located in the moment frame is also found to be adequate. Since $P_u/\phi P_n$ is less than 0.2 for this beam, the following equation is used:

$$P_u/2\phi P_n + (M_{ux}/\phi M_{nx} + M_{uy}/\phi M_{ny}) \leq 1.0$$

It is found that when inserting the values for the beam the equation above equals 0.44 which is less than 1.0.

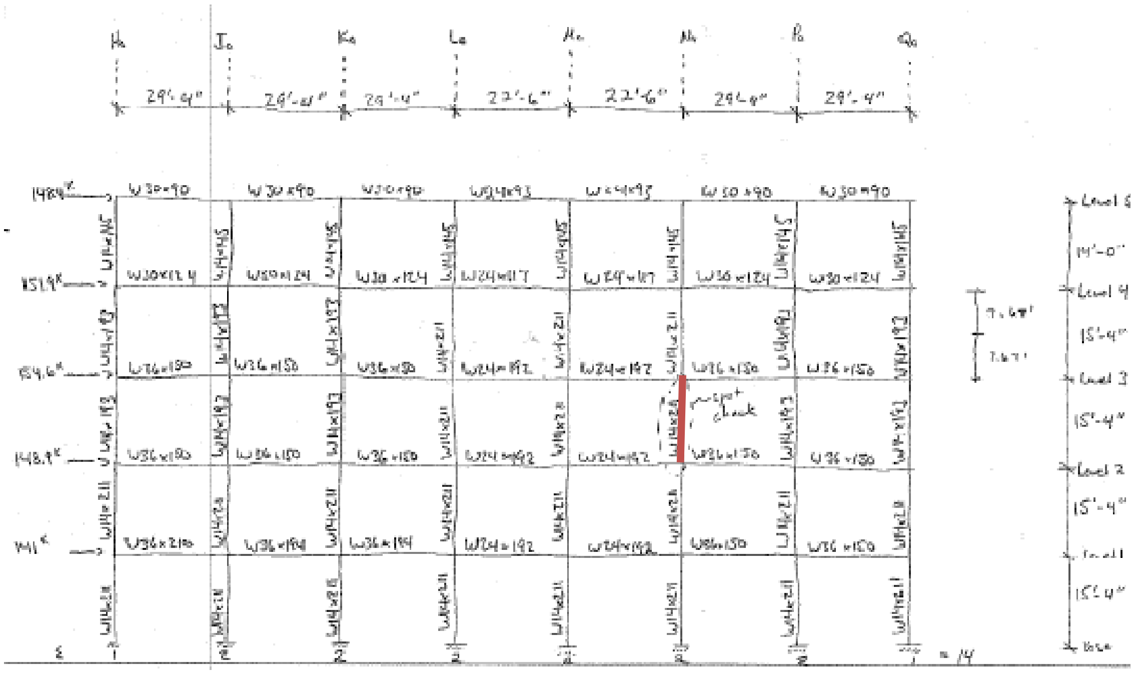



*Figure 6 – Column Examined in Spot Check*

Since both of these spot checks show that the members have much more capacity than what is needed, it can be assumed that they are sized according to drift requirements instead of strength requirements.

## Conclusions

In the third technical report an in depth analysis is preformed of the existing lateral system. The system is described and examined for many factors including lateral distribution, drift, torsion, and overturning moment.

An ETABS model is created in order to help and check hand calculations preformed in this report. The ETABS model is created imputing only the lateral system of the building. It is created so that it is as accurate to the original drawings as possible. The seismic and wind loads that are calculated in this report, found in Appendix B and C, are used so that any hand calculations should match the ETABS model.

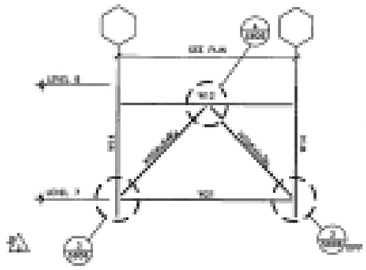
The distribution of lateral forces to the braced and moment frames is determined using hand calculations and is based on the stiffness of each frame. The amount of direct and torsional shear in the North – South frames of the 4$^{th}$ floor are determined through hand calculations. The center of rigidity is found using ETABS and is compared to the center of mass, also found in ETABS, to find the eccentricity of the building.

Lateral loads are also used to determine the amount of drift of the building. The values are found using the program ETABS and then compared to serviceability and strength requirements. The ETABS drift values are also compared to hand calculations of drift for a moment frame in the building. The hand calculation values are found using the Portal and Cantilever methods.

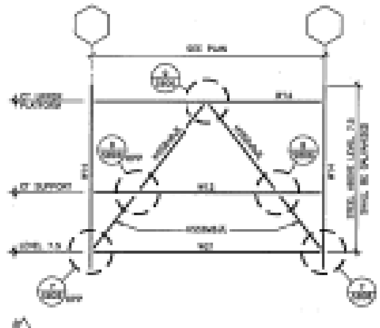
The building's torsional moment and overturning moment are also found in this report. The building's torsion is found using hand calculations and is relatively small. This value is also used to find the torsional shear in frames located on the 4$^{th}$ level.

Spot checks are also preformed on the lateral members to insure that they have enough strength to carry the lateral loads. A spot check of a brace in a braced frame and a column in a moment frame are preformed in order to make sure they have adequate strength. Both members are found to be sized larger than needed for strength, and are assumed to be sized according to drift requirements.

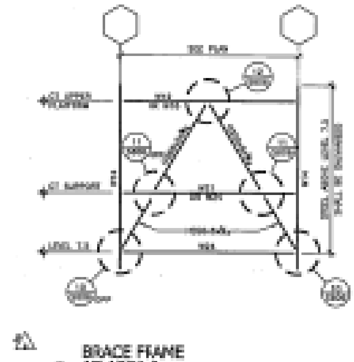
## Appendix A



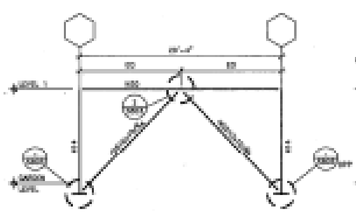
8 BRACE FRAME AT AREA 5



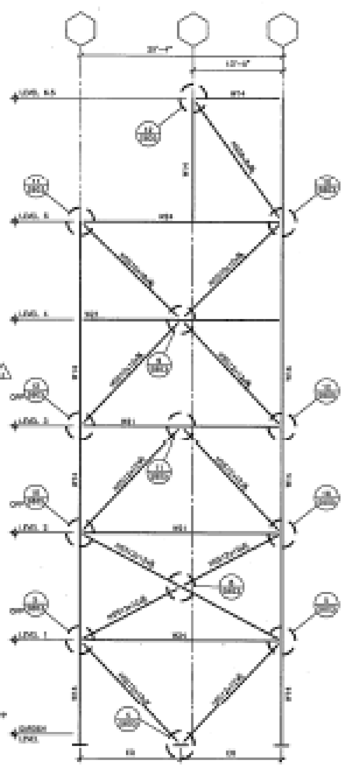
9 BRACE FRAME AT AREA 7



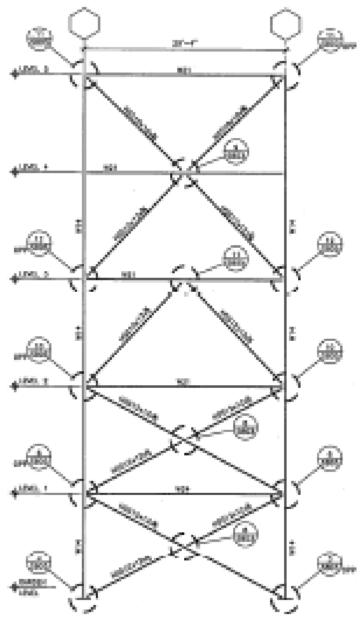
10 BRACE FRAME AT AREA 7



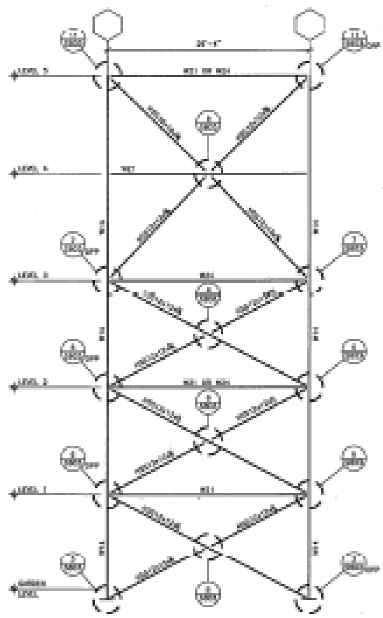
1 BRACE FRAME AT AREA 6



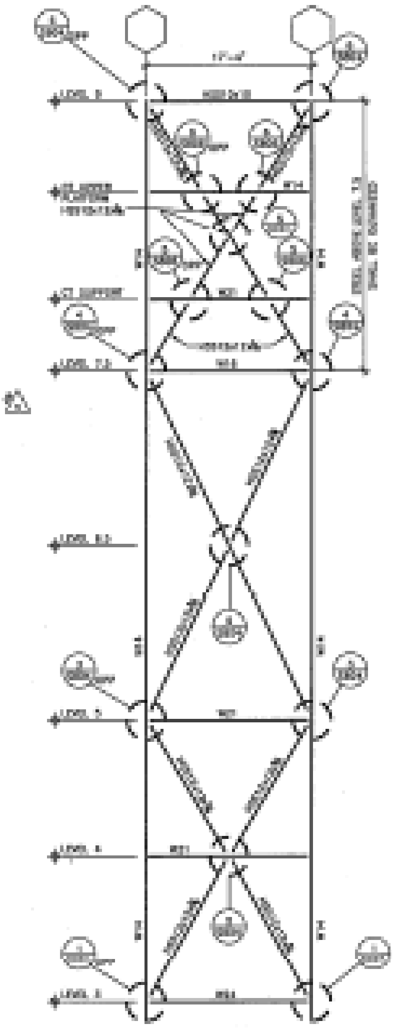
2 BRACE FRAME AT AREA 7



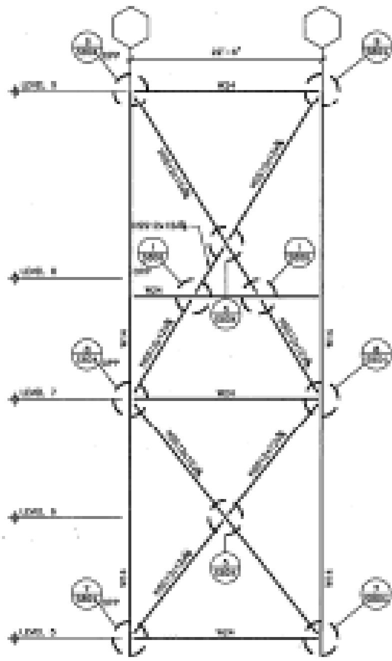
3 BRACE FRAME AT AREA 7



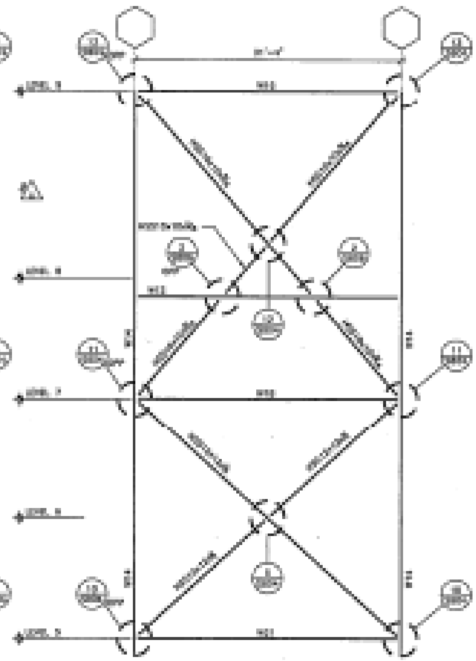
4 BRACE FRAME AT AREA 7



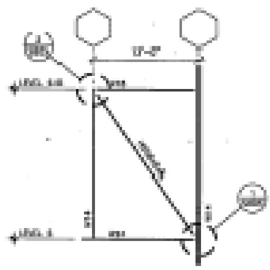
5 BRACE FRAME AT AREA 7



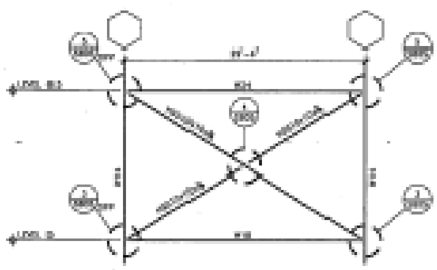
6 BRACE FRAME AT AREA 5



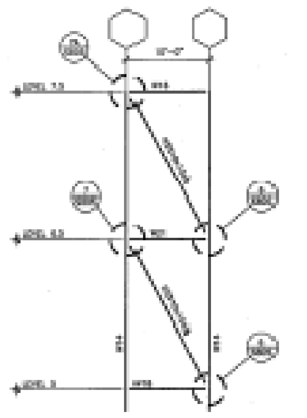
7 BRACE FRAME AT AREA 5



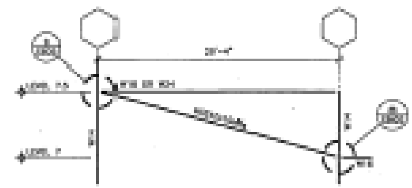
BRACE FRAME
AT AREA 7



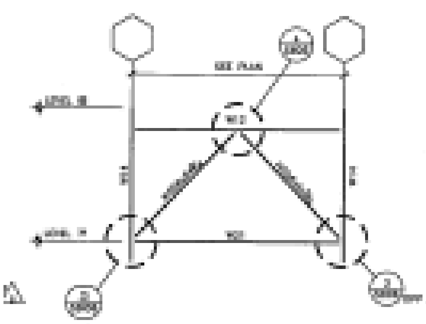
9 BRACE FRAME
AT AREA 2



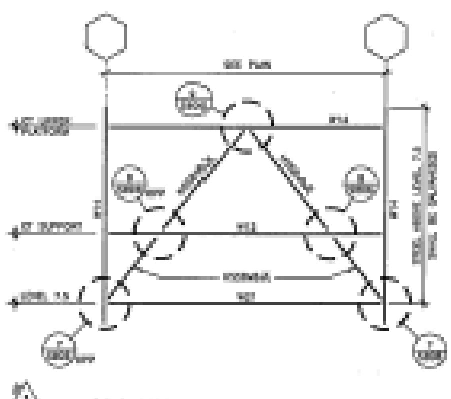
10 BRACE FRAME
AT AREA 7



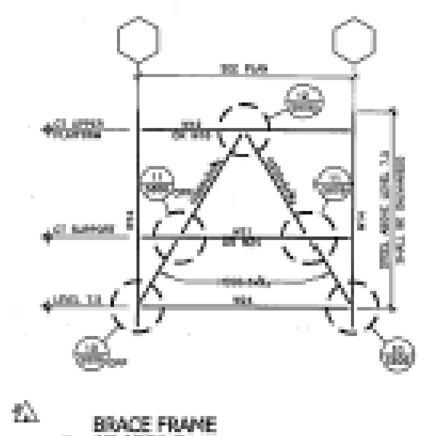
11 BRACE FRAME
AT AREA 7



8 BRACE FRAME
AT AREA 5



9 BRACE FRAME
AT AREA 7



10 BRACE FRAME
AT AREA 7

## Appendix B

Paul Stewart | Virtua Health Hospital | Tech #1 Wind Loads | 4

According to ASCE 07-05:

V = 100 mph per Fig 6-1

$K_d$ = .85 per Table 6-4

I = 1.15 per Table 6-1

Exposure Category = B per 6.5.6

6.5.7.1: all points not met

→ $K_{zt}$ = 1.0

$\beta$ = .01

N-S:
Area A = Areas 1-3: B = 74.166', h = 139'-4"
Area B = Areas 4-7: B = 295.875', h = 139'-4"

E-W:
Areas A: B = 242.4', h = 139'-4"
Areas B: B = 292', h = 139'-4"

story heights = 15'-4"

roof mean height = 139.33 ft

For exposure type B per Table 6-2:

$\alpha$ = 7.0
$z_g$ = 1200'
$\hat{a}$ = 1/7
$\hat{b}$ = 0.84
$\bar{\alpha}$ = 1/4.0
$\bar{b}$ = 0.45
c = 0.30
$\ell$ = 320'
$\bar{\epsilon}$ = 1/3.0

· Find $K_z$ per Table 6-3:

$K_z = 2.01(z/z_g)^{2/\alpha}$

| Level | Height (ft) | $K_z$ |
|---|---|---|
| 0 | 0 | 0 |
| 1 | 15.33 | 0.578 |
| 2 | 30.66 | 0.705 |
| 3 | 46 | 0.792 |
| 4 | 61.33 | 0.859 |
| 5 | 75.33 | 0.911 |
| 6 | 89.33 | 0.957 |
| 7 | 103.33 | 0.998 |
| 8 | 117.33 | 1.034 |
| 9 | 139.33 | 1.086 |

$K_h = 2.01(h/z_g)^{2/\alpha} = 1.086$

| Paul Stewart | Virtua Health Hospital | Tech #1 Wind Loads (2) | 5 |
|---|---|---|---|

- Find velocity pressures:

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

| Height | $q_z$ |
|---|---|
| 0 | 0 |
| 15.33 | 14.463 |
| 30.66 | 17.642 |
| 46 | 19.819 |
| 61.33 | 21.496 |
| 75.33 | 22.797 |
| 89.33 | 23.948 |
| 103.33 | 24.974 |
| 117.33 | 25.875 |
| 139.33 | 27.176 |

$$q_h = .00256\, K_h K_{zt} K_d V^2 I = .00256(1.086)(1)(.85)(100)^2(1.15) = 27.176$$

Gust Effects Factors:

$$n_1 = 22.2/H^{0.8} = 22.2/(139.33)^{.8} = .4276 \quad \text{per C6.5.8}$$

$n_1 < 1$ ∴ building is flexible per 6.2

$$g_Q = g_v = 3.4$$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{.577}{\sqrt{2\ln(3600 n_1)}} = 3.982 \qquad \text{Eq 6-9}$$

$$\bar{z} = .6h = .6(139.33) = 83.598$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .30\left(\frac{33}{83.6}\right)^{1/6} = .2569 \qquad \text{Eq 6-5}$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{83.6}{33}\right)^{1/3} = 416.23 \qquad \text{Eq 6-7}$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}} \qquad Q_{AME} = 0.845 \qquad \text{Eq 6-6}$$

$Q_{BME} = 0.783$

$Q_{ABW} = 0.796$

$Q_{BBW} = 0.784$

Paul Stewart | Virtua Health Hospital | Tech #1 Wind Loads (3) | 6

$\overline{V}_{\bar{z}} = \overline{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 82.265$ Eq 6-14

$N_1 = \frac{n_1 L_{\bar{z}}}{\overline{V}_{\bar{z}}} = 2.24$ Eq 6-12

$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = 0.083$ Eq 6-11

$\eta_1 = \frac{4.6 n_1 h}{\overline{V}_{\bar{z}}} = 3.29$

$R_h = \frac{1}{\eta_1} - \frac{1}{2\eta_1^2}\left(1-e^{-2\eta}\right) = .2578$

$\eta_2 = \frac{4.6 n_1 B}{\overline{V}_{\bar{z}}} =$ $\eta_{2A\,N\text{-}S} = 1.752$ $\eta_{2A\,E\text{-}W} = 5.724$

$\eta_{2B\,N\text{-}S} = 6.489$ $\eta_{2B\,E\text{-}W} = 6.897$

$R_B = \frac{1}{\eta_2} - \frac{1}{2\eta_2^2}\left(1-e^{-2\eta_2}\right) =$ $R_{B\,A\,N\text{-}S} = .4127$ $R_{B\,A\,E\text{-}W} = .1594$

$R_{B\,B\,N\text{-}S} = .1228$ $R_{B\,B\,E\text{-}W} = .1345$

$\eta_3 = \frac{15.4 n_1 L}{\overline{V}_{\bar{z}}} =$ $\eta_{3A\,NS} = 19.165$ $\eta_{3A\,EW} = 5.865$

$\eta_{3B\,NS} = 23.093$ $\eta_{3B\,EW} = 23.349$

$R_L = \frac{1}{\eta_3} - \frac{1}{2\eta_3^2}\left(1-e^{-2\eta_3}\right) =$ $R_{L\,A\,NS} = .0508$ $R_{L\,A\,EW} = .1559$

$R_{L\,B\,NS} = .0423$ $R_{L\,B\,EW} = .0418$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53 + .47 R_L)}$ Eq 6-10

$R_{A\,NS} = .6994$ $R_{A\,EW} = .4536$

$R_{B\,NS} = .3913$ $R_{B\,EW} = .3477$

$G_f = .925\left(\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right)$ $G_{f\,A\,NS} = 1.0227$ $G_{f\,A\,EW} = .8237$ Eq 6-8

$G_{f\,B\,NS} = .8059$ $G_{f\,B\,EW} = .9065$

The building is enclosed with no parapet, & is not a low-rise building.

Paul Stewart | Virtua Health Hospital | Tech #1 Wind Loads (4) | 7

From 6-6

Cp:

$GC_{pi}$ = +/-0.18

Windward wall: .8

Leeward walls: A-NS: -.2788
B-NS: -.5
A-EW: -.5
B-EW: -.5

Side walls: -.7

$p_z = q_z G_f C_p - q_h (GC_{pi})$ windward Eq 6-19

$p_h = q_h G_f C_p - q_h (GC_{pi})$ leeward Eq 6-19

Area 1,2,3 N-S

| height | Floor | $p_z$ | $p_h$ |
|---|---|---|---|
| 0 | 0 | 0 | 11.529 |
| 15.33 | 1 | 16.725 | |
| 30.66 | 2 | 19.326 | |
| 46 | 3 | 21.107 | |
| 61.33 | 4 | 22.479 | |
| 75.33 | 5 | 23.543 | |
| 89.33 | 6 | 24.485 | |
| 103.33 | 7 | 25.324 | |
| 117.33 | 8 | 26.062 | |
| 129.33 | 9 | 27.126 | |

Area 4,5,6,7 NS

| Floor | $p_z$ | $p_h$ |
|---|---|---|
| 0 | 0 | 15.842 |
| 1 | 14.216 | |
| 2 | 16.266 | |
| 3 | 17.669 | |
| 4 | 18.751 | |
| 5 | 19.589 | |
| 6 | 20.331 | |
| 7 | 20.993 | |
| 8 | 21.574 | |
| 9 | 22.413 | |

Paul Stewart | Virtua Health Hospital | Tech #1 Wind Loads (5) | 8

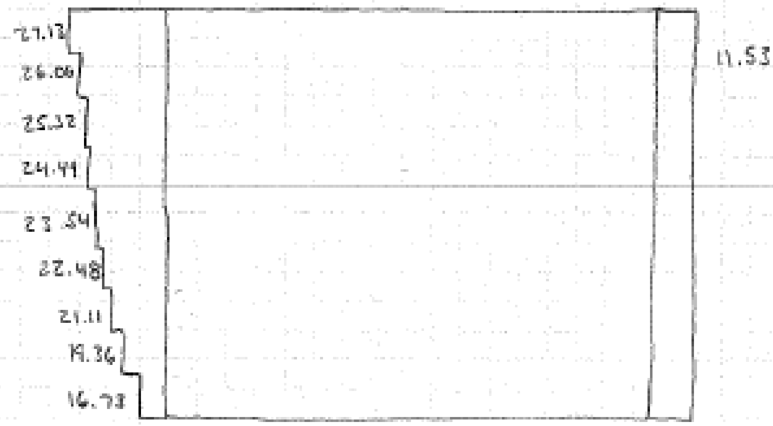
Area 1, 2, 3 E-W

| Floor | Pz | Ph |
|---|---|---|
| 0 | 0 | 16.084 |
| 1 | 14.422 | |
| 2 | 16.519 | |
| 3 | 17.952 | |
| 4 | 19.057 | |
| 5 | 19.914 | |
| 6 | 20.672 | |
| 7 | 21.349 | |
| 8 | 21.940 | |
| 9 | 22.800 | |

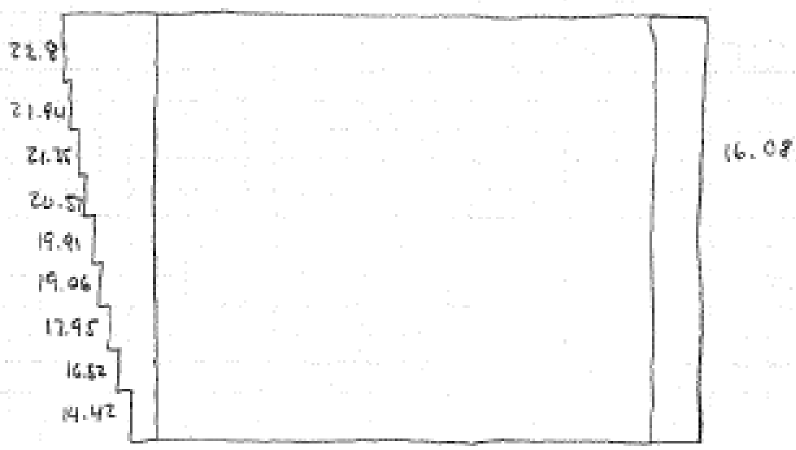
Area 4, 5, 6, 7 E-W

| Floor | Pz | Ph |
|---|---|---|
| 0 | 0 | 15.85 |
| 1 | 14.223 | |
| 2 | 16.274 | |
| 3 | 17.679 | |
| 4 | 18.861 | |
| 5 | 19.600 | |
| 6 | 20.343 | |
| 7 | 21.005 | |
| 8 | 21.586 | |
| 9 | 22.426 | |

Paul Stewart | Urban Health Hospital | Tech #1 Wind Loads (6) | 9

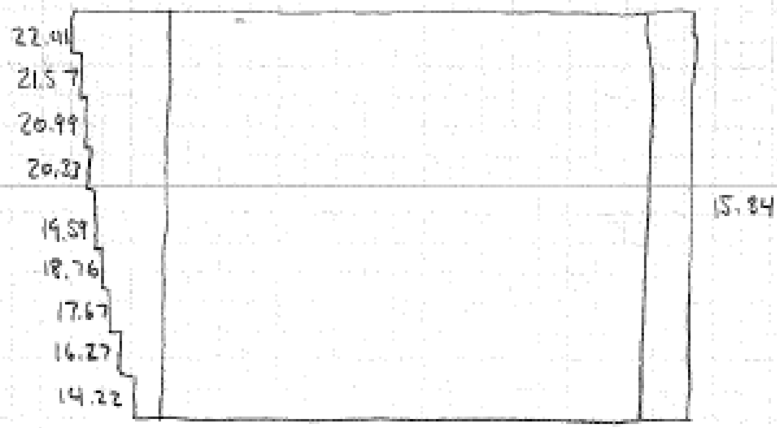



Area 1,2,3 N-S

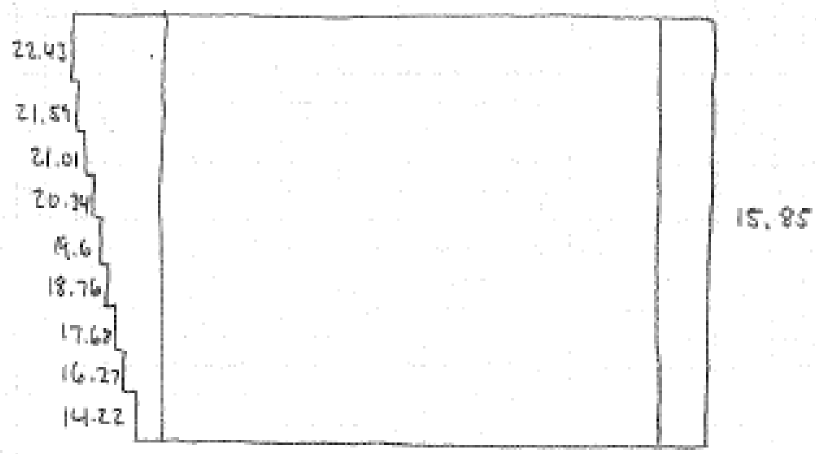



Area 1,2,3. E-W

Paul Stewart | Virtua Health Hospital | Tech #1 Wind Loads (7) | 10

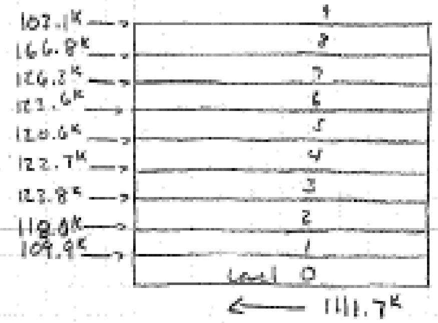


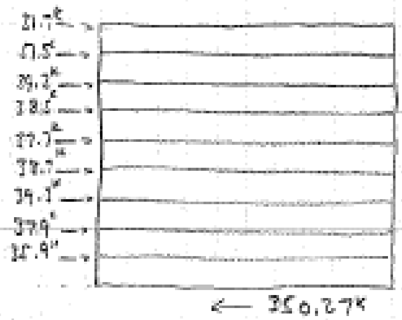

Areas 4,5,6,7 N-S




Areas 4,5,6,7 E-W

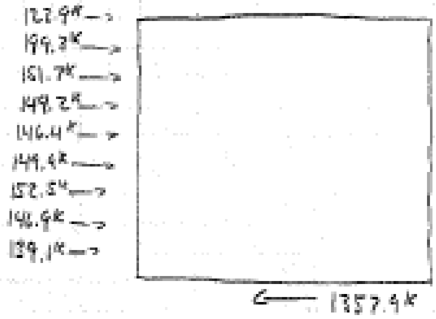
Paul Stewart | Virtua Health Hospital | Tech #1 Wind Loads (8) | 11

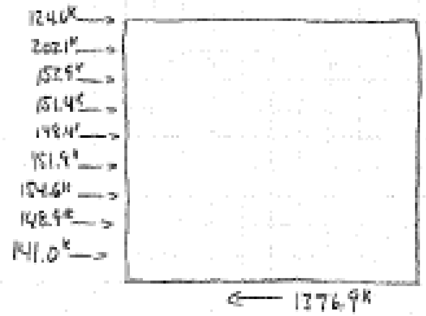



Areas 1,2,3 N-S

Areas 1,2,3 E-W




Areas 4,5,6,7 N-S

Areas 4,5,6,7 E-W

$M_{1-3\,NS}$ = 103.1(139.33) + 166.8(117.33) + 126.3(103.77) + 123.6(89.77) + 120.6K × 75.32 + 122.7(61.33) + 123.8(46) + 118.0(30.66) + 109.9(15.33)

= 85603.77 ft-K

## Appendix C

Paul Stewart | Virtua Health Hospital | Tech #1 | Seismic | 13

Design Data:

Location: Voorhees, New Jersey (Latitude: 39.84°, Longitude -74.93°)

Soil Classification: D

Occupancy: Medical

Material: Structural Steel

Structural System: Moment-resisting frames & Braced Frames

1) a) $S_S = 0.249$
$S_1 = 0.057$ — According to the USGS Ground Motion Parameter Calculator

b) Site Class D

c) $S_{MS} = 0.398$
$S_{M1} = 0.137$

d) $S_{DS} = 0.265$
$S_{D1} = 0.092$

2) a) $S_1 = 0.057 > 0.04$ & $S_S = 0.249 > 0.15$ ∴ can not be assigned to SDC A

b) For a health care facility or hospital having surgery or emergency treatment facilities, occupancy category is IV

c) $S_1 < .75$ ∴ not assigned to SDC E or SDC F

d) ∴ determine SDC from 11.6-1 & 11.6-2

$.167 \leq S_{DS} \leq .33$ ∴ use SDC = C

e) In N-S of Building A, uses concentrically braced frames

$T_a = C_t h_n^{x} = .02(139.33)^{.75} = 0.811$

In N-S of Building B, uses concentrically braced frames & moment frames:

$T_a = .02(139.33)^{.75} = .811$

Paul Stewart | Virtua Health Hospital | Tech #1 | Seismic (2) | 14

In E-W of Building A, Moment frames are used

$T_a = .028(179.23)^{.8} = 1.45$

In E-W of Bldg B, both systems are used

$T_a > .811$

$T_S = S_{D1}/S_{DS} = 2.88\,sec$

3) check if $T < 3.5T_S$

T for B.A. N-S $= 0.811 > 1.009$
T for B.B N-S $= 0.911 > 1.009$ $.35T_S = .35(2.88) = 1.009$
T for B A EW $= 1.45 < 1.008$
T for B B E-W $= 0.811 > 1.008$

a) determine R

$R_{BANS} = 3.25$ ← ordinary steel concentrically braced frames

$R_{BBNS} = 3$ ← not described

$R_{BAEW} = 3.5$ ← ordinary steel moment frames

$R_{BBEW} = 3.5$ ← ordinary moment frames

b) $I = 1.5$

c) $T_L = 6\,sec$ from Fig 22-15

d) $C_{SANS} = \frac{S_{D1}}{T(R/I)} = \frac{0.092}{.811(3.25/1.5)} = 0.0524$

$C_{SBNS} = \frac{0.092}{.811(3/1.5)} = 0.0567$

$C_{SAEW} = \frac{.092}{1.45(3.5/1.5)} = 0.0272$

$C_{SBEW} = \frac{.092}{.811(3.5/1.5)} = 0.0486$

Paul Stewart | Virtua Health Hospital | Tech #1 | Seismic (3) | 15

e) Dead Load for Building A:

Area #1: Area of 1 = 30.083'

4.03°
$R_1$ = 429.25'
$R_2$ = 459.33'

$C = \frac{2\pi r}{360} = 7.492\,ft/°$
$C = \frac{2\pi r}{360} = 8.0168\,ft/°$

$d = \frac{8.0168 + 7.492}{2} = 7.754\,ft/° \times 4.03° = 31.25\,ft$

Area of #1 = 31.25 × 30.083 = 940.1 ft²

concrete = 115 pcf = 115 (6.5/12) = 62.3 psf
metal deck = 12 pcf = 12 (3/12) = 3 psf

65.3 psf

beam weight = $\frac{31}{2} + 22 + 22 + 22 + \frac{22}{2} = \frac{92.5\,plf}{31.25\,ft} = 2.96\,psf$

girder weight = $76 + \frac{44}{2} = \frac{98}{30.083} = 3.26\,psf$

Total = 65.3 + 2.96 + 3.26 = 71.5 psf

column weights

Assume 7 psf

Total = 71.5 psf + 7 psf = 78.5 psf

Area of building A = 38547 ft²/floor

Total weight of Building A:

78.5 psf (38547 ft²)(9) = 27233.46 K

Paul Stewart | Virtua Health Hospital | Tech #1 Seismic (4) | 16

Building B

Deck weight = 65.3psf

beam weight = 26 + 26 + 26plf = $\frac{78plf}{29.33ft}$ = 2.66psf

girder weight = 35 + $\frac{50}{2}$ = $\frac{60}{31.33}$ = 1.92psf

assume 7psf column weight

Total weights = 65.3 + 2.66 + 1.92 + 7 = 76.9psf

Area of Building B = 63684ft² + 85293 + 68547 + 50972 + 47287 + 14625 + 14625 + 9893 + 2622 + 2622

= 360120ft²

Total weight of Building B:

360120 (76.9) = 27693.2$^K$

$V_{ANS}$ = .0514(27273.5) = 1427.03$^K$

$V_{BNS}$ = .0567(27693.2) = 1570.2$^K$

$V_{AEW}$ = .0272 (27233.5) = 740.75$^K$

$V_{BEW}$ = .0486(27693.2) = 1345.8$^K$

f) .5 < T < 2.5 sec

$k_{ANS}$ = .75 + .5T = 1.155

$k_{BNS}$ = 1.155

$k_{AEW}$ = 1.475

$k_{BEW}$ = 1.155

g) See excel sheet for Lateral Forces & Story Shear

$k_{ANS}$ = 1.155

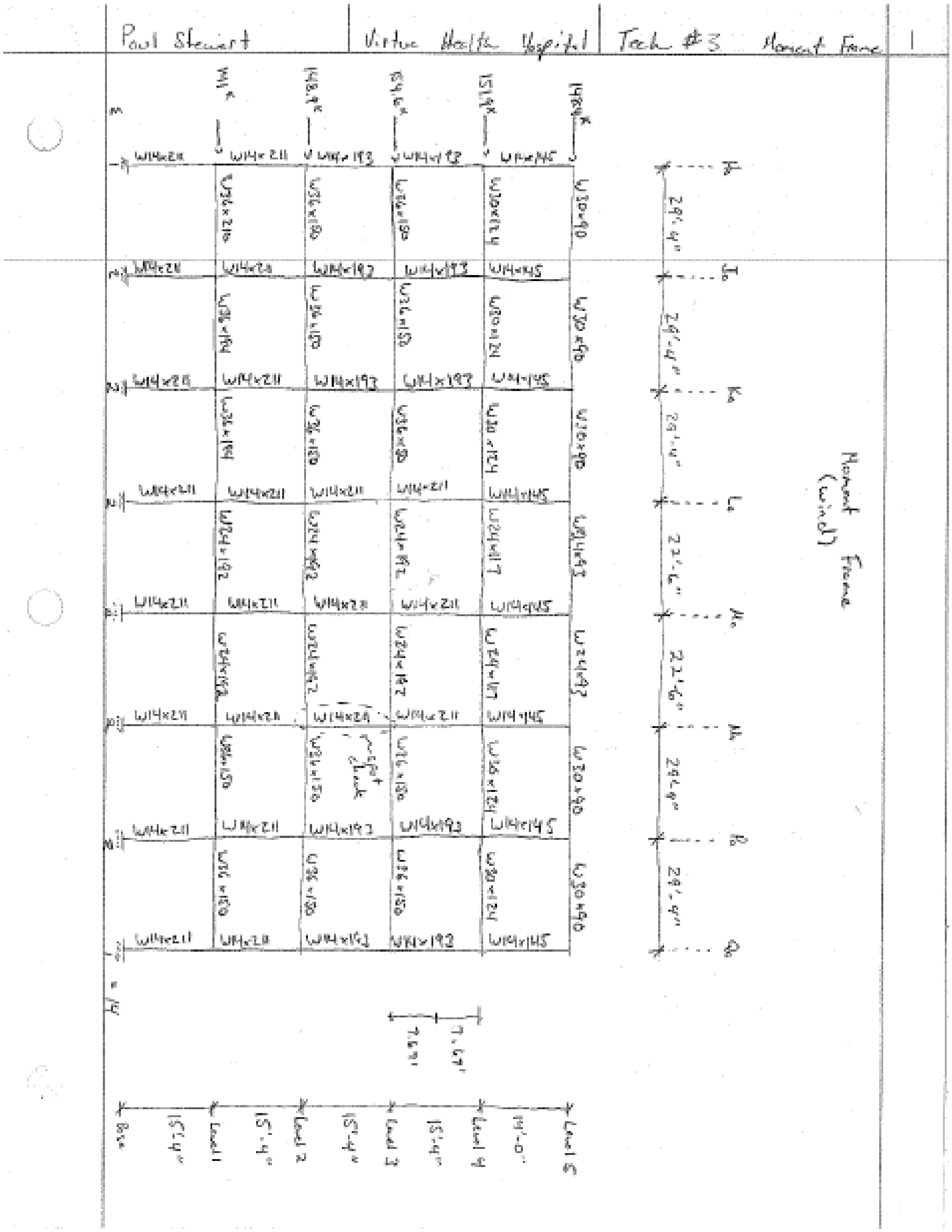
$K_{BNS}$ = 1.155

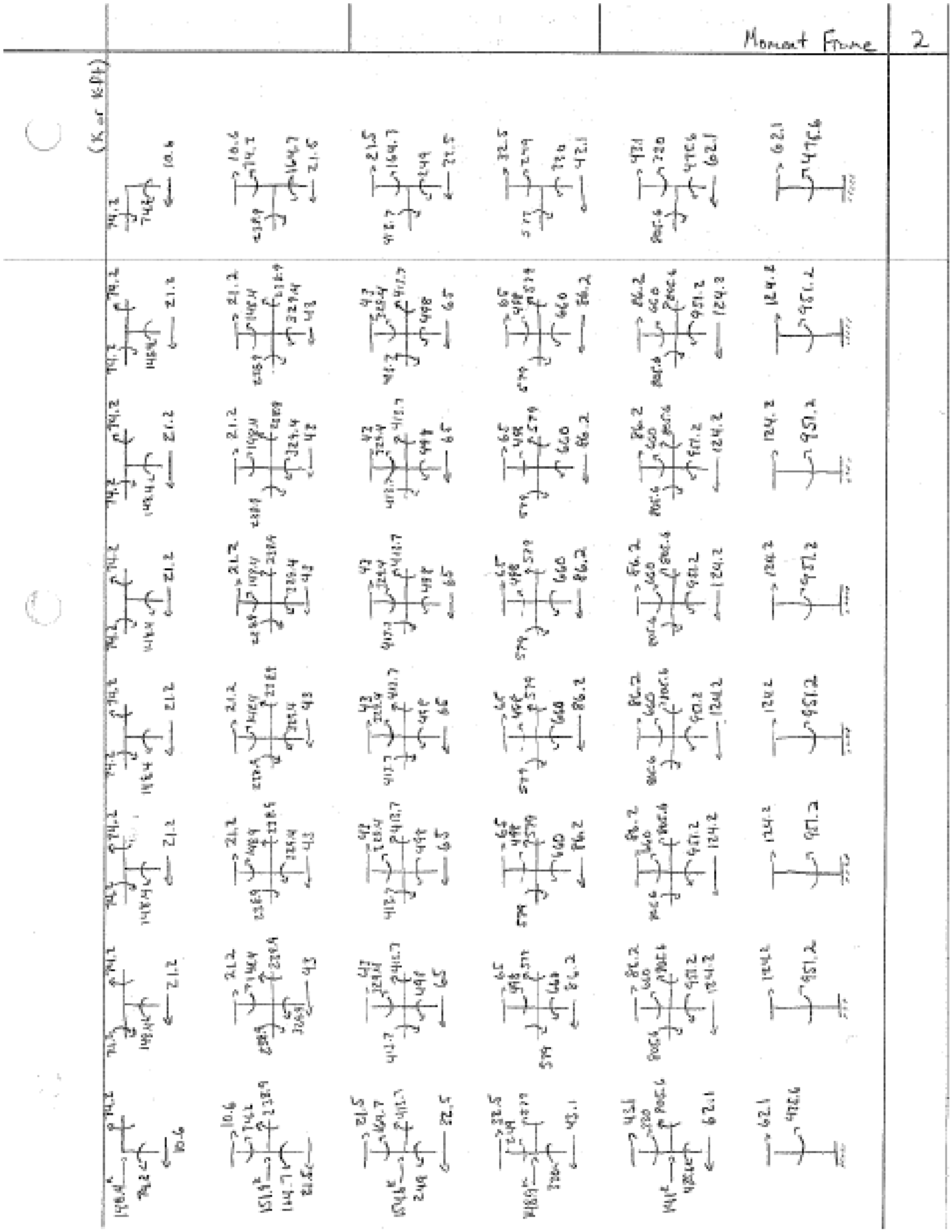
$K_{AEW}$ = 1.475

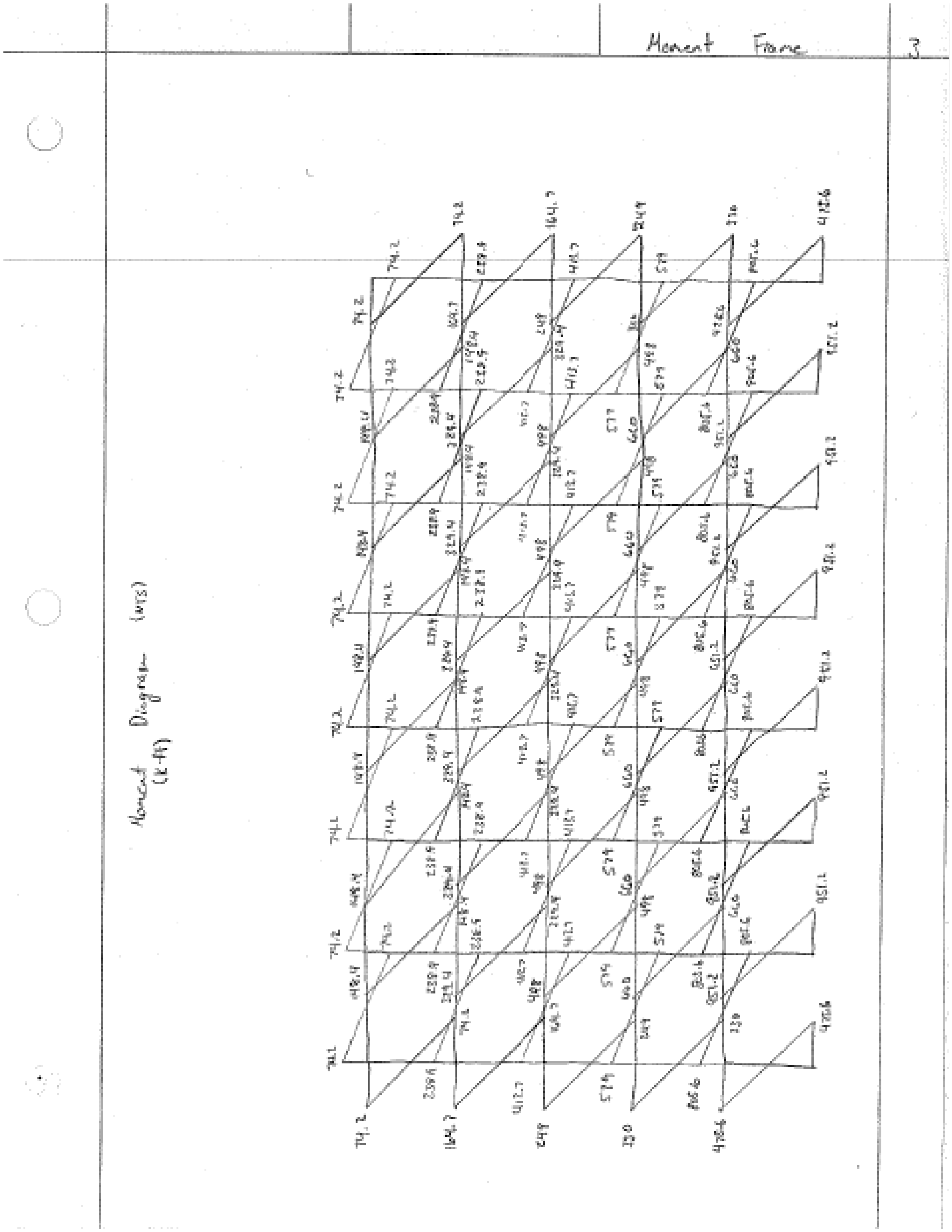
$K_{BEW}$ = 1.155

@ .5-1
@ 2.5-2

## Appendix D




Moment Frame
2
(K or K-ft)


Moment Frame
3
Moment Diagram (K-ft) (WTS)

Moment Frame 4

Cantilever Method:

| Size | Area (in²) | Moment of Inertia |
|---|---|---|
| W14×211 | 62.0 | 2660 |
| W14×193 | 56.8 | 2400 |
| W14×145 | 42.7 | 1710 |
| W24×192 | 56.3 | 6260 |
| W24×117 | 34.4 | 3540 |
| W24×94 | 27.7 | 2700 |
| W30×124 | 36.5 | 5360 |
| W30×90 | 26.4 | 3610 |
| W36×210 | 61.8 | 13200 |
| W36×194 | 57.0 | 12100 |
| W36×150 | 44.2 | 9040 |

$$\bar{x}_1 = \frac{29.33(62) + 58.66(62) + 88(62) + 110.5(62) + 133(62) + 162.33(62) + 191.66(62)}{(62 + 62 + 62 + 62 + 62 + 62 + 62 + 62)}$$

$= 96.69'$

$\bar{x}_2 = \bar{x}_1 = 96.69'$

$$\bar{x}_3 = \frac{29.33(56.8) + 58.66(56.8) + 88(62) + 110.5(62) + 133(62) + 162.33(56.8) + 191.66(56.8)}{(56.8 + 56.8 + 56.8 + 62 + 62 + 62 + 56.8 + 56.8)}$$

$= 97.14'$

$\bar{x}_4 = \bar{x}_3 = 97.14'$

$\bar{x}_5 = \bar{x}_1 = 96.69'$

Moment Frame 5

$P_i = \frac{M_{oy} A_i}{I}$ $I = \Sigma A_i y_i^2$

Story 1: $I = (62)(96.69)^2 + (62)(67.26)^2 + (62)(38.03)^2 + (62)(15.53)^2 + 62(6.49)^2 + (62)(36.3)^2 + (62)(65.63)^2 + (62)(94.96)^2$
$= 1876415$

Story 2: $I = I_1 = 1876915$

Story 3: $I = 56.8(97.14)^2 + 56.8(67.81)^2 + 56.8(38.49)^2 + 62(9.15)^2 + (62)(15.5)^2 + (62)(35.85)^2 + 56.8(65.19)^2 + 56.8(94.51)^2$

$= 1725838$

Story 4: $I = I_3 = 1725838$

Story 5: $I = 42.7(96.69)^2 + 42.7(67.36)^2 + 42.7(38.03)^2 + 42.7(15.53)^2 + 42.7(6.49)^2 + 42.7(36.3)^2 + 42.7(65.63)^2 + 42.7(94.96)^2$

$= 1291739$

Story 5:

$M_n = 148.4(7) = 1038.8^{k\text{-}ft}$

$P_H = \frac{1038.8(96.69)(42.7)}{1291739} = 3.32^k$ $P_I = \frac{1038.8(67.36)(42.7)}{1291739} = 2.31^k$

$P_K = \frac{1038.8(38.03)(42.7)}{1291739} = 1.31^k$ $P_L = \frac{1038.8(15.53)(42.7)}{1291739} = .533^k$

$P_M = .239^k$ $P_N = 1.25^k$ $P_P = 2.25^k$ $P_Q = 3.26^k$

Story 4:

$M_n = 151.9(7.67) + 148.4(14 + 7.67) = 4380.9^{k\text{-}ft}$

$P_H = 14^k$ $P_I = 9.78^k$ $P_K = 5.55^k$ $P_L = 1.44^k$

$P_M = 2.1^k$ $P_N = 5.69^k$ $P_P = 9.4^k$ $P_Q = 13.63^k$

Story 3:

$M_n = 154.6(7.67) + 151.9(7.67 + 15.33) + 148.4(14 + 15.33 + 7.67) = 10170.3^{k\,ft}$

$P_H = 32.51^k$ $P_I = 22.7^k$ $P_K = 12.88^k$ $P_L = 3.34^k$

$P_M = 4.88^k$ $P_N = 13.1^k$ $P_P = 21.82^k$ $P_Q = 31.62^k$

Moment Frame 6

Story 2:

$M_n = 148.9(7.67) + 159.6(7.67 + 15.33) + 151.9(7.67 + 15.33 + 15.33) + 148.4(7.67 + 15.33 + 15.33 + 14) = 18285.96^{k\text{-}ft}$

$P_H = 58.42^k$ $P_I = 40.7^k$ $P_K = 22.98^k$ $P_L = 9.38^k$

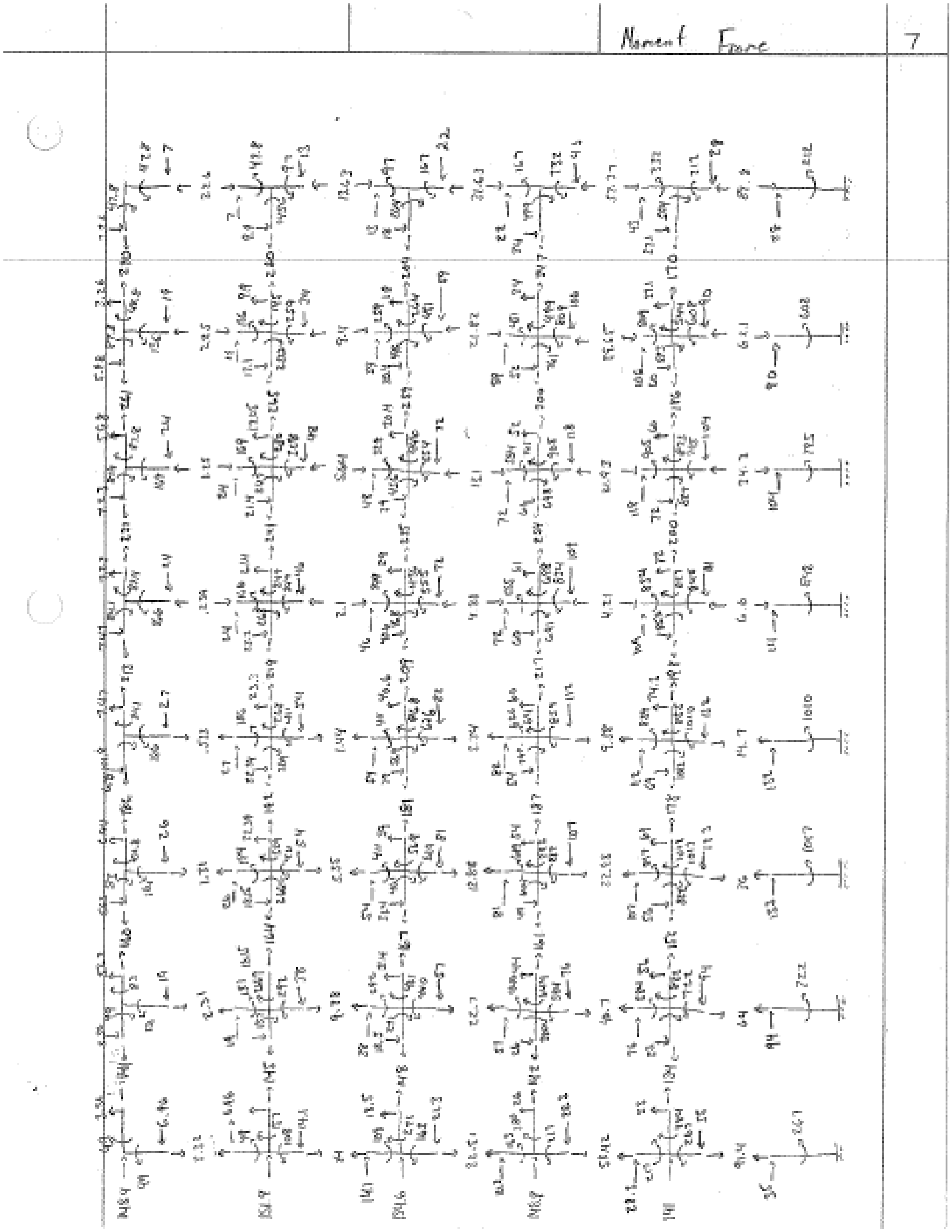
$P_M = 4.21^k$ $P_N = 21.95^k$ $P_P = 39.65^k$ $P_Q = 57.37^k$

Story 1:

$M_n = 28619.22^{k\text{-}ft}$

$P_H = 91.43^k$ $P_I = 63.7^k$ $P_K = 35.96^k$ $P_L = 14.69^k$

$P_M = 6.59^k$ $P_N = 34.33^k$ $P_P = 62.01^k$ $P_Q = 89.8^k$


Moment Frame
7

Moment Frame 8

$$U_1 = \frac{M_c L_c^2}{6EI_c} + \frac{L_c L_g}{12E}\left(\frac{M_{b-1}}{I_{b-1}} + \frac{M_b}{M_b}\right)$$

$$= \frac{475.6(15.33)^2}{6(29000)(2660)} + \frac{15.33(29.33)}{12(29000)}\left(\frac{805.6}{13200}\right) = 3.203\times10^{-4}\text{ ft} = .003844\text{ in}$$

$$u_2 = \frac{830(15.33)^2}{6(29000)(2660)} + \frac{15.33(29.33)}{12(29000)}\left(\frac{805.6}{13200} + \frac{529}{9040}\right) + .003844 = 0.007708\text{ in}$$

$$u_3 = \frac{249(15.33)^2}{6(29000)(2410)} + \frac{15.33(29.33)}{12(29000)}\left(\frac{529}{9040} + \frac{413.7}{9040}\right) + 0.007708 = 0.011006\text{ in}$$

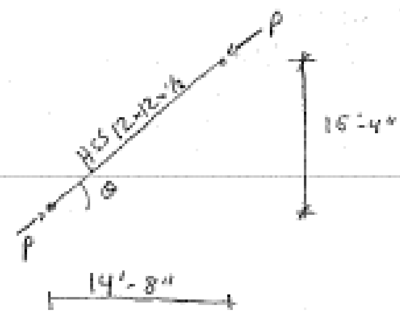
$$u_4 = \frac{164.7(15.33)^2}{6(29000)(2400)} + \frac{15.33(29.33)}{12(29000)}\left(\frac{413.7}{9040} + \frac{238.9}{5360}\right) + .011006 = 0.01352\text{ in}$$

$$u_5 = \frac{74.2(14)^2}{6(29000)(1710)} + \frac{(14)(29.33)}{12(29000)}\left(\frac{238.9}{5360} + \frac{74.2}{7610}\right) + .01352 = 0.01486\text{ in}$$

## Appendix E

Tech 3 | Spot checks | 10

Check brace (D42)




$P = 70.15^K$
$L_b = 21.21\,ft$

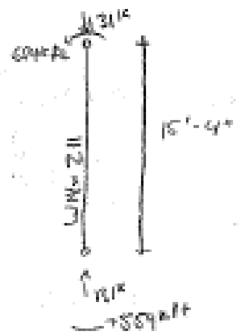
$\theta = \tan^{-1}\left(\frac{15.33}{14.66}\right) = 46.28°$

From Table 4-4 in AISC

$\phi P_n = 698^K > 70.15^K$ ok ✓

$\frac{P_u}{\phi P_n} = \frac{70.15}{698} = .101 > 1$ ∴ ok

check column in moment frame:




Forces from cantilever method

Assume $K = 1.67$

$KL = 15.33'(1.67) = 25.6$

$\phi M_p = 1460$  $\phi M_r = 887$  $L_p = 14.4$  $L_r = 86.4$
$BF = 7.99$

$\phi P_n = 1810^K$
$\phi M_{ny} = 731\,k\,ft$

$\phi M_{nx} = \phi M_p - BF(L_b - L_p) = 1460 - (7.99)(25.6 - 14.4) = 1370.3$

$\frac{P_u}{\phi P_n} = \frac{131}{1810} = .072 < .2$  $\frac{P_u}{2\phi P_n} + \left(M_{ux}/M_{nx}\phi + M_{uy}/M_{ny}\phi\right)$

$= \frac{131}{2(1810)} + \left(\frac{554}{1370.3}\right) = .44 \le 1.0$ ∴ good ✓

## Appendix F

Tech 3 | Lateral Force Distribution | 1

$F_{iy} = \frac{k_{iy}}{\Sigma k_{jy}} P_y$ $\quad F_{iy} = \frac{k_{iy}}{\Delta_y}$ $\quad k = \frac{P}{\Delta P}$

calculate lateral force distribution in level 4:

use forces from Y-direction (N-S) earthquake

N-S direction | drift
---|---
Brace M6a | 0.128078"
Brace M6b | 0.128078"
Brace La | 0.148169"
Brace Lb | 0.148169"

$P = 253.392^k$

Brace M6a = M6b
Brace La = Lb

$k_M = \frac{P}{\Delta P} = \frac{253.392}{.128078} = 1978.4$

$k_L = \frac{P}{\Delta P} = \frac{253.392}{.148169} = 1710.2$

$\bar{x} = \frac{\Sigma k_{iy} x_i}{\Sigma k_y} = \frac{2(1710.2)(137.66)}{(2(1710)+2(1978))} + \frac{2(1978.4)(172.66)}{(2(1710)+2(1978))} = 156.46 ft$

$F_{id} = \frac{k_{iy}}{\Sigma k_{jy}} P_y = \frac{1978.4}{7376}(253.392) = 67.97^k$ Brace M $\quad e_x = 25.15 ft$

$= \frac{1710.2}{7376}(253.392) = 58.75^k$ Brace L

$M_t$ found in torsion section

$F_{it} = \frac{k_i d_i P_y e_x}{\Sigma k_j d_j^2} = \frac{1978.4(16.2)(11686.9)}{(2)(1978.4)(16.2)^2 + 2(1710.2)(18.8)^2} = 166.67^k$ Brace M

$F_{it} = \frac{1710.2(18.8)(11686.9)}{(2)(1978.4)(16.2)^2 + 2(1710.2)(18.8)^2} = -167.2^k$ Brace L

$F_{total} = 166.67 + 67.97 = 234.64^k$ Brace M

$= -167.2 + 58.75 = -108.45^k$ Brace L