Barshinger Life Science & Philosophy Building

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Structural Option
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Structural Technical Report #1

Structural Concepts / Structural Existing Conditions Report

Executive Summary

This report is a preliminary discussion of the building's structural systems and the loads cases that the systems are designed to support. Essentially, the superstructure is comprised of 6 ½" composite concrete slab-on-deck, supported by composite wide-flange members, carried upon wide-flange columns, and grounded with concrete piers and shallow concrete footings. Lateral forces are resisted by ten (10) concentrically braced steel frames located throughout the structure.

Simplified design calculations were performed using ASCE7-02 and the International Building Code (IBC) 2000 to determine the live, dead, snow, wind, and seismic loads acting on the building. The resulting loads are summarized in the table below.

Live	Offices	50 psf (+20 psf partitions)	
	Laboratories	60 psf	
	Public Spaces	100 psf	
Dead	Floor Loads	120 psf	
	Exterior Walls	45 psf	
Snow	Flat Roof	23.1 psf	[25 psf]*
	Sloped Roof	27.7 psf	[28 psf]*
Wind	N-S Base Shear	65.5 k	
	E-W Base Shear	143.2 k	
Seismic	Base Shear	846 k**	[865 k]*

*Design values in brackets if known **Controlling Lateral Load Case

Spot checks were performed on a typical floor bay and a lateral force resisting brace to validate the calculated load cases. In both cases, the spots checks produced results very similar to those of the design engineers.

1.0 The Building Program

The Barshinger Life Science and Philosophy Building will be the largest construction project in the long history of Lancaster, Pennsylvania's Franklin and Marshall College. The three-story Georgian Revival structure will house the departments of biology, psychology, and philosophy, as well as two interdisciplinary programs in biological foundations of behavior and scientific and philosophical students of mind. At a total cost of $45 million, the 102,000 square-foot building will include state-of-the-art classrooms and laboratories, a greenhouse, a multi-story atrium, a 125-seat lecture hall, a commons for meetings and gatherings, and a vivarium for the study of primates and rodents.

2.0 Structural System Overview

2.1 Superstructure

The building superstructure is comprised of composite slab-on-deck in combination with composite wide-flange steel beams supported by wide-flange columns bearing on concrete piers and shallow footings. The framing system is separated into approximately 20'x30' bays. Floor-to-floor heights are typically found to be 14-feet. A typical floor frame consists of 2-inch composite metal deck with 4 ½-inches of normal weight concrete above the flutes. The composite slab is then carried by W16x26 filler beams spaced 7-feet apart. Interior girders, of size W18x40, are typically carried by W12x65 columns, sized for ease of fabrication and erection considering the OSHA-required four anchor bolt pier connection. The basic framing plan can be found in Appendix A

2.2 Lateral Force Resisting Systems

The structure's main lateral force resisting system is composed of ten concentrically braced steel frames of varying sizes. These frames utilize wide-flange shapes for the vertical and horizontal members with ½-inch thick HSS shapes for the braces. The ten frames are located throughout the structure according to the Figure 2.2.1 below. The basic structure of each frame can be seen in Figure 2.2.2 on the next page.

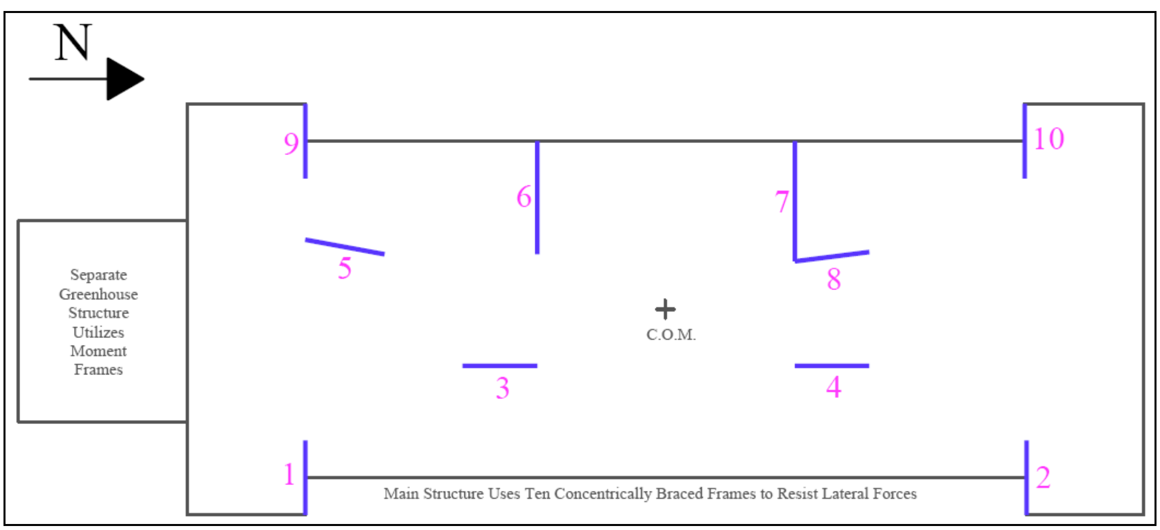


Figure 2.2.1 Layout of the 10 Concentrically Braced Frames

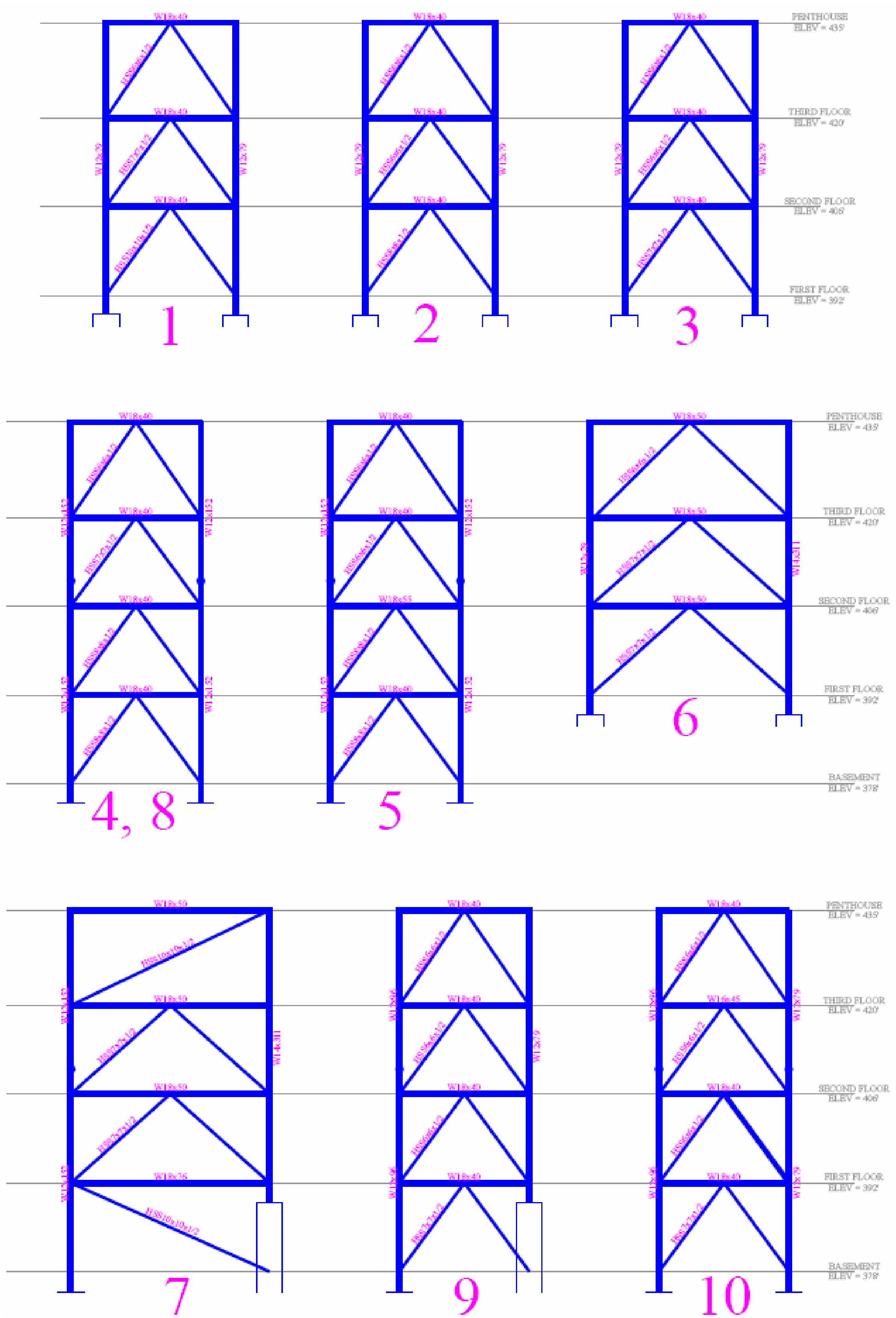


Figure 2.2.2 The 10 Concentrically Braced Frames in the Main Lateral Force Resisting System

The greenhouse wing on the southern exposure of the structure uses moment frames to resist the lateral forces. Large areas of glass were necessary to create the light, airy, and habitable space necessary for its greenhouse function. Moment frames were chosen over of the clumsier-looking braced frames due to the glass requirements as well as the lightweight nature of the structure that includes a glass and aluminum-framed barrel roof. The greenhouse wing is separated from the main building by an expansion joint in order to keep the lateral resisting systems separate.

The lateral system analysis for this report and Technical Report #3 will focus on the concentrically braced frames of the main building, and not the moment frames of the greenhouse.

2.3 Foundations

The superstructure of the Barshinger Building rests upon shallow foundations, specifically spread footings. In the geotechnical report for the site, Advanced GeoServices Corp. of West Chester, PA recommended that the foundations not exceed an allowable bearing pressure of 3,000 pounds per square foot (psf). Large footings will be located to transfer the loads from the braced frames into the ground and resist overturning moments. Test borings encountered intact rock at depths ranging from 3 to 23.5 feet. The recommendation put forth is to excavate the rock where necessary, then to supply a soil cushion in the excavated areas for the footings to bear on.

An alternative system would have been to bring the building loads directly onto the intact rock through the use of caissons in the deeper areas. This system certainly could have allowed for higher building loads and perhaps a larger structure overall and may warrant future investigation.

2.4 Cladding

The building employs a relatively heavy cladding system. The red brick façade is backed by concrete masonry units and certainly increased the seismic design loads on the structure. However, the cladding system is consistent with all of the other buildings on the Franklin and Marshall College campus.

2.5 Vierendeel Truss

The location and orientation of the large 125-seat lecture hall seemed, at least to me, to present an interesting challenge for the structural designers. The lecture hall is positioned in the center of the structure with half of the hall directly underneath two upper floors. Columns in the hall would obstruct views and create a cluttered audience. Therefore, the designers were forced to devise a method of spanning the entire 69-feet, while at the same time carrying the weight of the two upper floors. A Vierendeel Truss system, pictured in Figure 2.5.1 taken from the structural drawings, was selected to solve the problem. The truss requires exceptionally large wide-flange members that could present difficult erection issues for the contractor, including a special crane that is larger than what is required for the rest of the job. The Vierendeel Truss designed for the Barshinger Building could be a good candidate for future investigation in my thesis from both the structural and construction management perspectives.

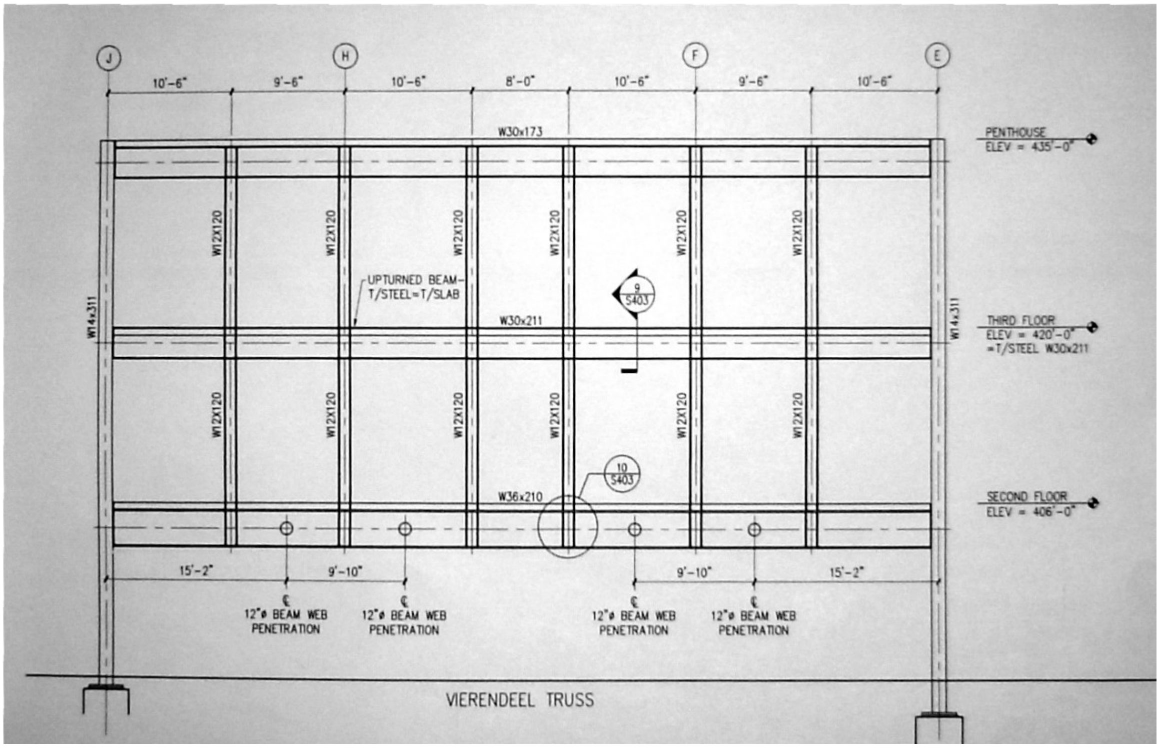


Figure 3.5.1 Vierendeel Truss

2.6 Material Strengths

The desired material strengths listed below in Figure 2.6.1 have been taken from the General Notes page of the Structural Drawings provided by Einhorn Yaffee Prescott, PC.

Concrete	f'_c	Unit Weight
Footings	3000 psi	150 pcf
Foundation Walls, Piers	4000 psi	150 pcf
Concrete on Metal Deck (Floor)	3500 psi	150 pcf
Concrete on Metal Deck (Roof)	3500 psi	150 pcf
Slabs on Grade	3500 psi	150 pcf
All Other Concrete	4000 psi	150 pcf
Reinforcing		
Typical Bars	ASTM A615	Grade 60
Welded Bars	ASTM A706	Grade 60
Welded Wire Fabric	ASTM A185	
Metal Deck Properties		
Roof Deck	3" Type "N"	20-gage
Composite Floor Deck	2" Type "B"	18-gage
Steel Members		
Wide-Flange Shapes	ASTM A992	
Channels & Angles	ASTM A36	
Pipe	ASTM A53	Grade B
Tubular Shapes	ASTM A500	Grade B
Base Plates	ASTM A36	
All Other Steel Members	ASTM A36	

Steel Connections	
High Strength Bolts	ASTM A325 or A490
Nuts & Washers	(Min. ¾" Diameter)
Anchor Rods	ASTM F-1554 Grade 55
Welding Electrode	E70XX
Metal Deck Welding Electrode	E60XX Min.
Masonry Properties	
Mortar	Type S
CMU Strength	F'_m = 1500 psi

Figure 2.6.1 Material Strengths & Properties for Design

2.7 Major Design Codes & Standards

The Barshinger Life Science and Philosophy Building was designed using the following major design codes and standards.

- International Building Code (IBC), 2000
- ASCE 7-98*
- ACI 315 "Manual of Standard Practice for Detailing Reinforced Concrete Structures"
- ACI 318 "Building Code Requirements for Reinforced Concrete"
- ACI 530 "Building Code Requirement for Masonry Structures"
- ACI 531 "Specifications for Masonry Structures"
- AISC "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings"

*I will use ASCE7-02 for the calculation of wind and seismic loads.

3.0 Design Loads

3.1 Live Loads

The minimum live load conditions for the building are taken from the IBC, 2000.

- Offices: (50 + 20 partition) psf
- Laboratories: 60 psf*
- Public Spaces, Exit Corridors, Stairs, Lobbies: 100 psf

*In order to simplify the design, the laboratory load can be increased to 70 psf to be equal to the minimum office load with the 20 psf partition allowance.

3.2 Dead Loads

Dead load estimates for the structure have been estimated using ASCE7-02, Table C3-1 where applicable. However a couple of the dead loads were selected based on engineering judgment and commonly used values.

- 6 ½" Normal Weight Slab: 12 psf/in x 6 ½" = 78 psf
- Metal Deck: 3 psf
- Framing Members: 10 psf
- MEP Equipment: 10 psf
- Carpet: + 1 psf

  120 psf

- Exterior Walls: 45 psf

3.3 Snow Loads

Flat roof and sloped roof snow loads were calculated using ASCE7-02. Then, they were compared to the design values used by the engineers. A detailed calculation can be found in Appendix C. A brief summation of the results is listed below. The engineers design loads are in brackets to the right of my calculated values for comparison. My own calculations produced results very similar to those of the professional structural engineers. Their design results are slightly more conservative.

- Flat Roof: p_f = 23.1 psf [25 psf]
- Sloped Roof: p_s = 27.7 psf [28 psf]

It may seem counter-intuitive that the sloped roof load is higher than the flat roof load. Common sense would dictate that the snow on the sloped roof should slide off before it approaches the flat roof load. However, in this structure, the sloped roof sections are merely screen roofs meant to keep the rooftop mechanical equipment out of sight from the ground. The difference can be explained by the fact that the flat roof is heated and the sloped roof is not.

The intersection of the sloped roof and the flat roof is a vertical wall where the snow drift effect needs to be designed for. Section 7.8 of ASCE7-02 has a procedure to determine the maximum drift load for the potential drift condition. The results of this analysis can be best explained in Figure 3.3.1 below.

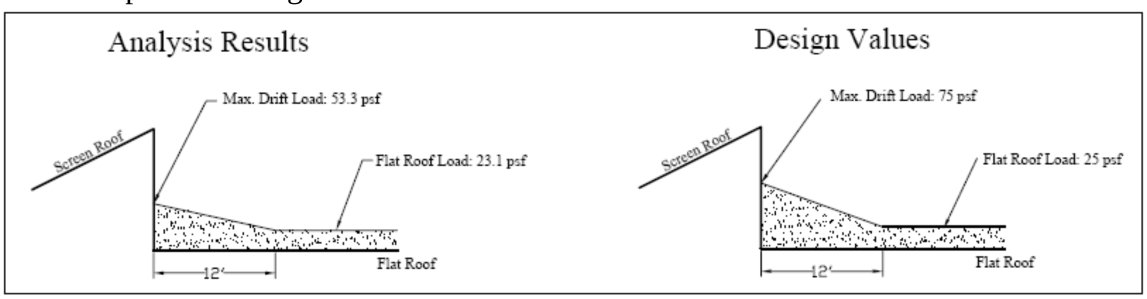


Figure 3.3.1 Snow Drift Effect

3.4 Wind Loads

The wind loads on the building were calculated using ASCE7-02. The detailed calculations can be found in Appendix D. Using the basic information for the building, I was able to estimate the lateral load placed on the building by the wind. For simplicity, the shape of the building was made rectangular with dimensions of 260'x110'. The dimensions match the largest width and length of the actual building shape, which resembles an elongated, flattened "H." The results of my analysis are represented by the story forces in Figure 3.4.1 below.

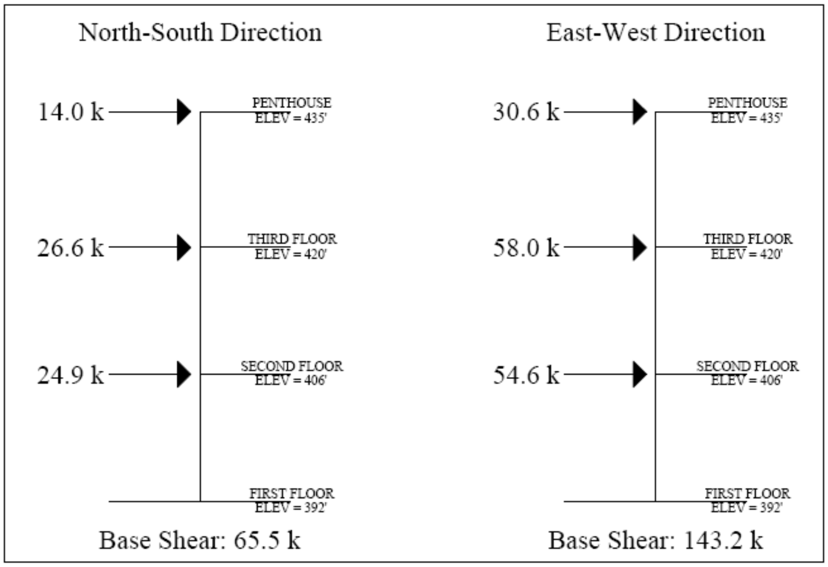


Figure 3.4.1 Story Wind Forces

3.5 Seismic Forces

The seismic loads on the building were calculated using ASCE7-02. The detailed calculations can be found in Appendix E. Using the basic information for the building, I was able to estimate the lateral load placed on the building by seismic forces. For simplicity, the shape of the building was made rectangular with dimensions of 260'x110'. The dimensions match the largest width and length of the actual building shape, which resembles an elongated, flattened "H." The results of my analysis are represented by the story forces in Figure 3.5.1 below. The resulting seismic forces were substantially larger than the wind forces in Section 3.4, and will therefore control the design of the lateral resisting system.

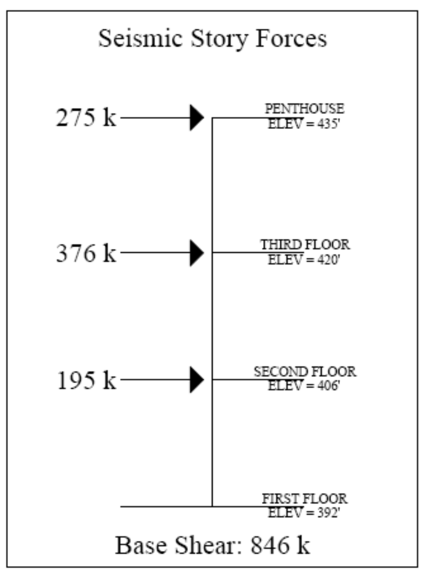


Figure 3.5.1 Story Seismic Forces

The designers of the structure had determined the seismic base shear to be 865 kips. Considering all the assumptions I had to use in calculating the weight of the building, the result was very close to designed value. The weight of the building is calculated in Appendix B.

4.0 Member Spot Checks

4.1 Floor Beams

The design for the floor system was checked using RAM Structural System. I constructed a simplified floor layout for the northern half of the structure and applied the design live and dead loads. Then, RAM was utilized to design the wide-flange members. Using Load Resistance Factor Design (LRFD) parameters, RAM designed the system using members slightly smaller than the actual designed system. However, when I adjusted the parameters to follow Allowable Stress Design (ASD), RAM produced a design with the same beam sizes as the original design. The girder sizes are slightly smaller than the actual design, but the difference is minimal. ASD was in fact the method used to design the structure. The ASD design results are pictured below in Figure 4.1.1.

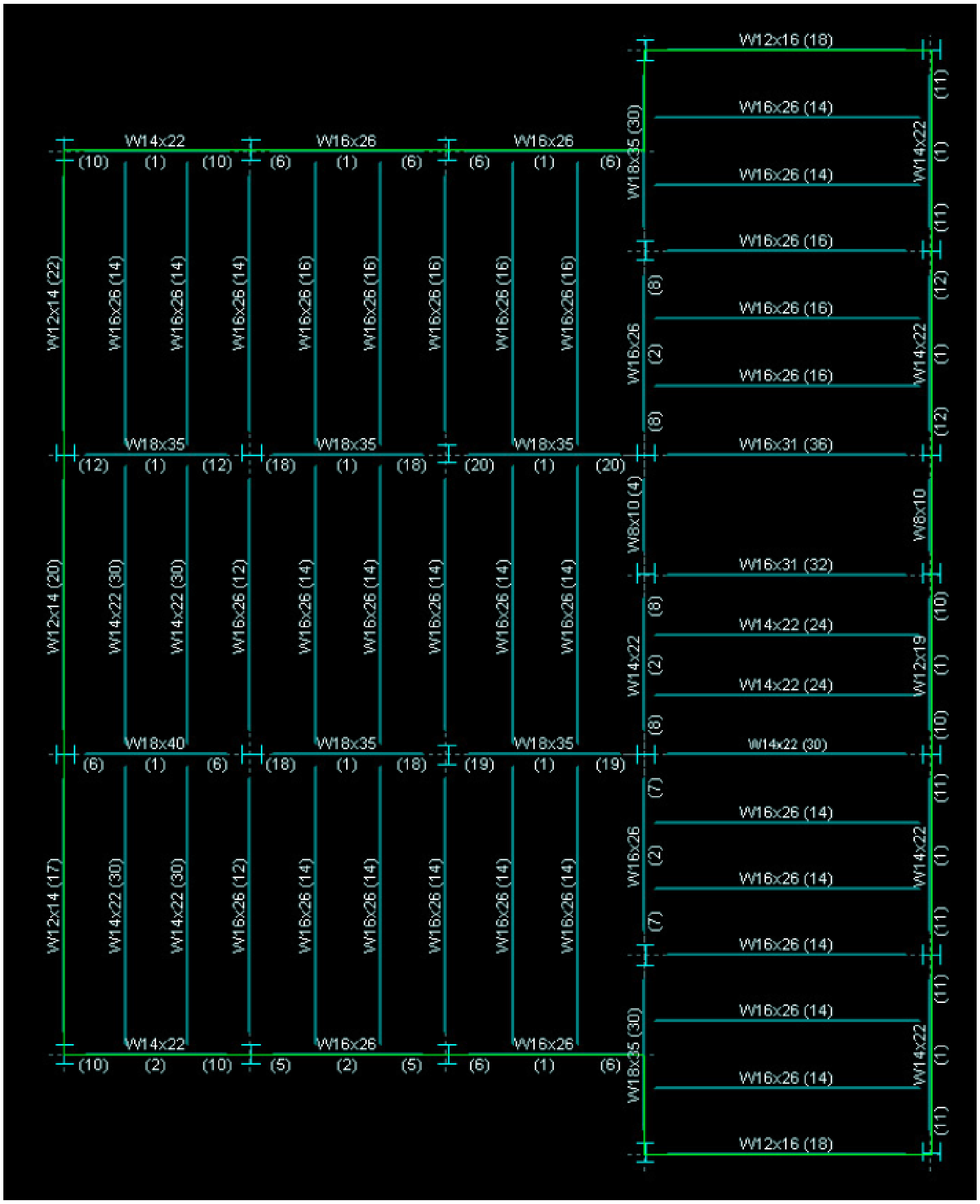


Figure 4.1.1 RAM Structural System Output for Beam Spot Check

4.2 Lateral Resisting Member

A lateral resisting member was checked using some very basic assumptions. This analysis is only preliminary and will be completed in much more depth in Technical Report #3. The concentrically-braced frames are placed throughout the plan as in Figure 4.2.1 below. For the purpose of this spot check, I analyzed Frame #6 and assumed that it resisted one-quarter of the load in the east-west direction. Frames #1, 2, 9, and 10 each take one-eight of the lateral load.

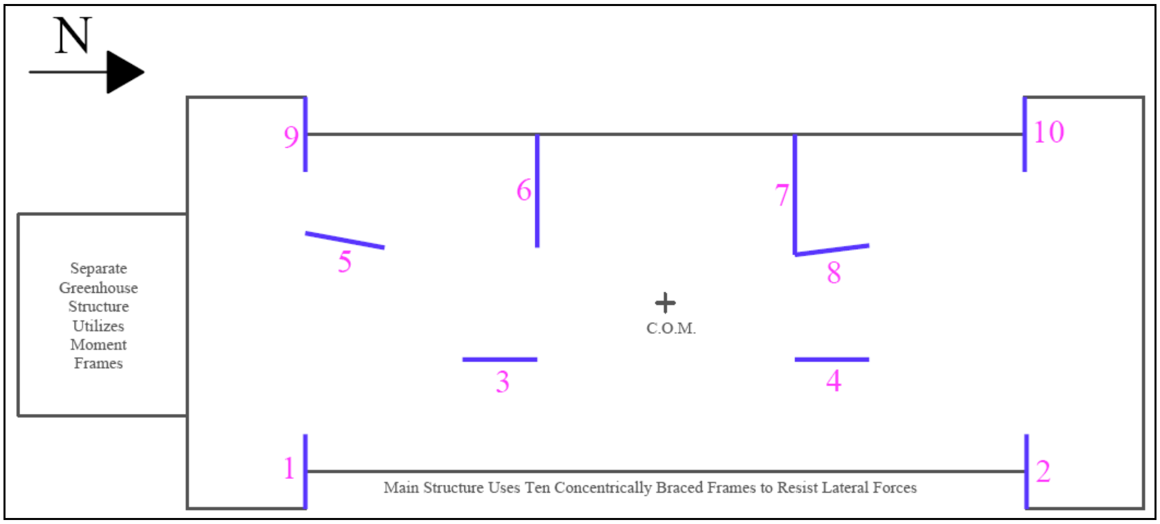


Figure 4.2.1 Layout of the 10 Concentrically Braced Frames

Then, I used simple truss analysis to find the force in the bottom bracing members. The calculations can be found in Appendix F. My results produced equal forces of 150 kips in each of the bracing members. This result compares favorably with the actual design values of 150 kips & 160 kips in the members. Even with all the approximation, the results are very similar. Some of the discrepancy is derived from the use of the seismic load that I calculated myself, instead of the design value. My analysis and the design values are compared in Figure 4.2.2 below with the design values in brackets.

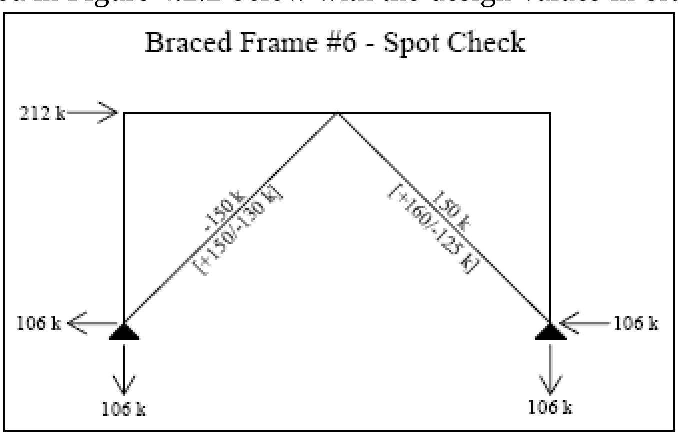
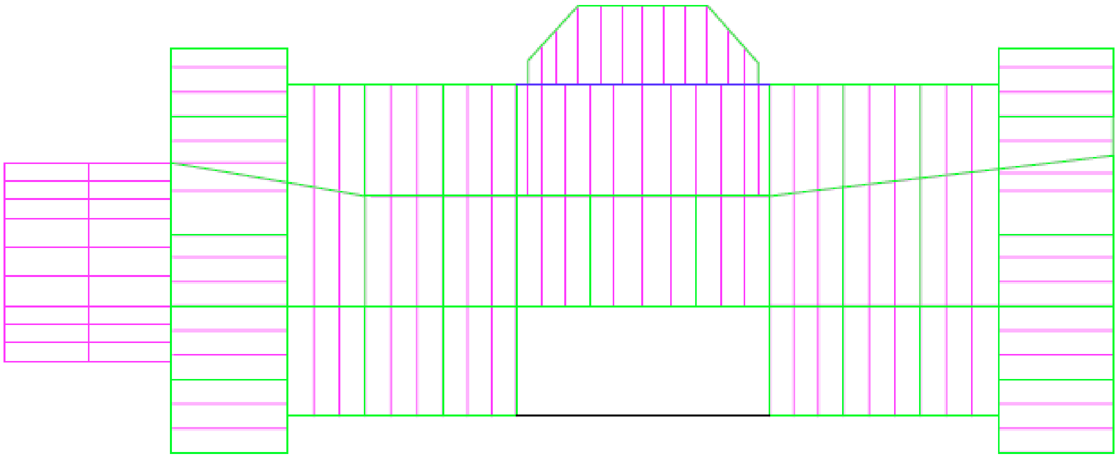


Figure 4.2.2 Braced Frame #6 – Member Force Spot Check

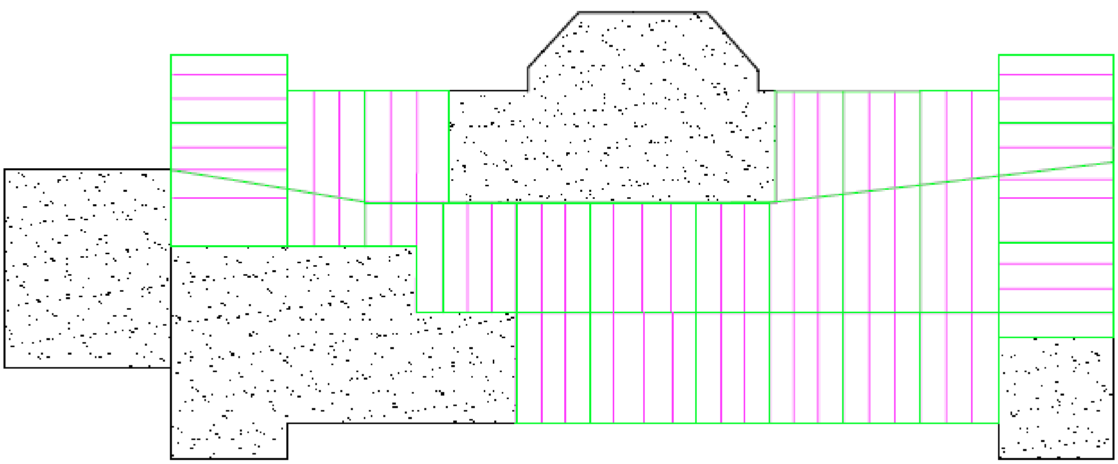
Appendix

Appendix	Description
A	**Simplified Structural Framing Plans**
B	**Dead/Live Load Requirements**
	Weight of Building Calculations
C	**Snow Load Calculations**
D	**Wind Load Calculations**
E	**Seismic Load Calculations**
F	**Lateral Brace Spot Check**

Appendix A
Simplified Structural Framing Plans

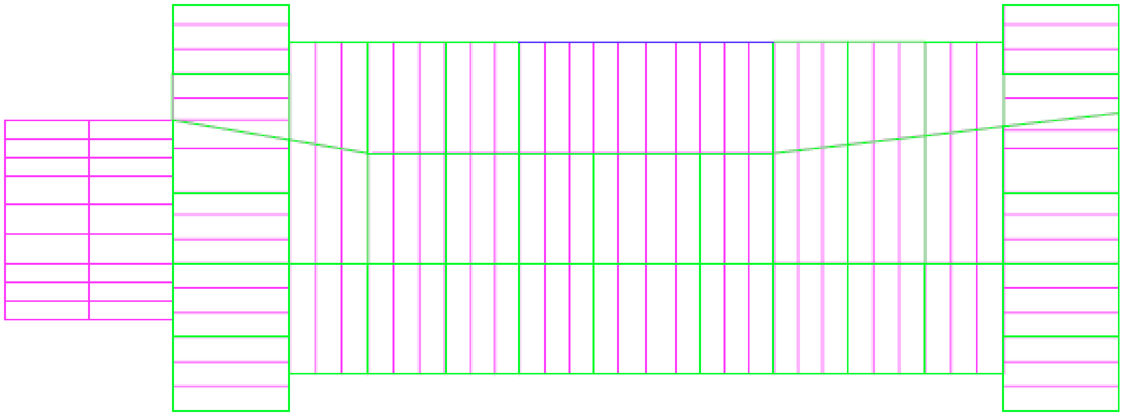


1st Floor Framing Plan

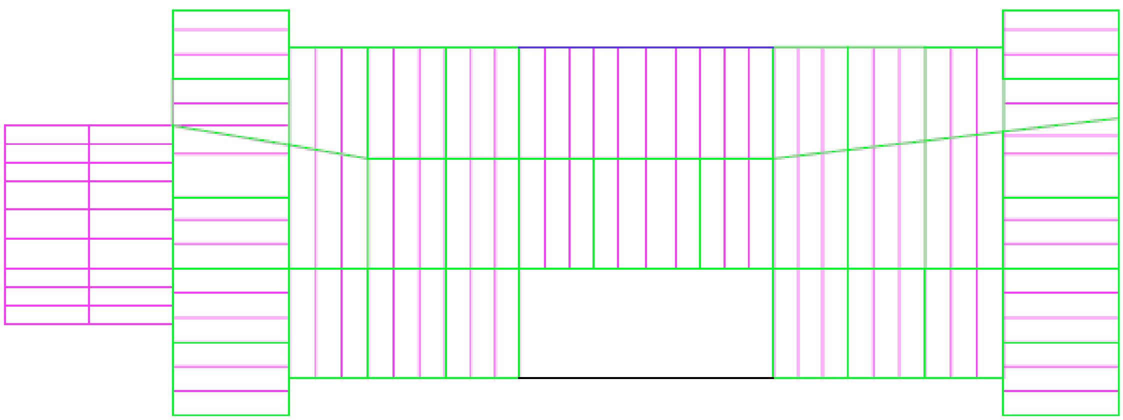


Basement Framing Plan

Framing Bay
Infill Beam
Vierendeel Truss



3rd Floor Framing Plan



2nd Floor Framing Plan

Framing Bay
Infill Beam
Vierendeel Truss

Appendix B

Dead & Live Load Requirements / Weight of Building Calculations

GRAVITY LOADS (ASCE 7-02, IBC 2000, + SOME EDUCATED GUESSES)

- DEAD LOADS
  - $6\frac{1}{2}$" NML. WT. CONCRETE SLAB : 12 PSF/IN × $6\frac{1}{2}$ = 78 PSF
  - METAL DECK : 3 PSF
  - FRAMING MEMBERS : 10 PSF
  - MEP EQUIPMENT : 10 PSF
  - EXT WALLS : 45 PSF
  - CARPET : 1 PSF
- PARTITIONS : 20 PSF
- LIVE LOADS
  - OFFICES : 50 + 20 PSF
  - LABORATORIES : 60 PSF
  - STAIRS / CORRIDORS : 100 PSF
- SNOW : 30 PSF (GROUND)
- ROOF DEAD : 60 PSF

FLOOR AREAS

114' × 260' → 30,000 SF PER FLOOR

→ 748'

STRUCTURE WEIGHT (FOR SEISMIC)

$W_{ROOF} = (30000)(60) + (14/2)(45)(748) = 2010$ K

$W_1 = (30000)(122) + (15/2)(45)(748) = 3885$ K

$W_2 = (30000)(122) + (15/2 + 14/2)(45)(748) = 4094$ K

$W_3 = (30000)(122) + (14/2 + 14/2)(45)(748) = 4079$ K

$W = W_{ROOF} + W_1 + W_2 + W_3 = 14{,}100$ K

Appendix C

Snow Load Analysis

SNOW LOAD (ASCE 7-02)

p_g = 30 PSF GROUND SNOW LOAD

C_e = 1.0 PARTIALLY EXPOSED

C_t = 1.0 FOR FLAT ROOF
= 1.2 FOR SCREEN ROOFS

I = 1.1 CATEGORY III BUILDING

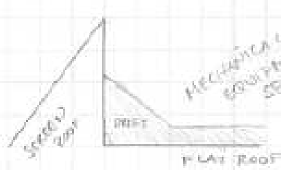
$p_f = 0.7 C_e C_t I p_g = 0.7(1.0)(1.0)(1.1)(30) = 23.1$ PSF

* A VALUE OF 25 PSF WAS USED FOR DESIGN

$p_s = C_s p_f = (1.0)[(23.1)(1.2)] = 27.7$ PSF

* A VALUE OF 28 PSF WAS USED FOR DESIGN

DRIFT – SCREEN ROOF – FLAT ROOF PROJECTION (SEC. 7.8)



$\gamma = 0.13(30) + 14 = 17.9$ PCF ≤ 30 PCF

$h_d = \begin{cases} 1.5' & \text{FIGURE 7-9} \\ W/4 = 12/4 = 3' & \leftarrow \text{CONTROLS} \end{cases}$

$h_b = (23.1)/(17.9) = 1.3'$

$h_d / h_b = 2.3$

* THE DESIGNED LOADS ARE
25 PSF FLAT ROOF LOAD
75 PSF MAX. DRIFT LOAD

Appendix D

Wind Load Analysis

WIND LOAD CALCULATIONS : NORTH-SOUTH DIRECTION [EAST-WEST]

- DESIGNED VALUES FROM GENERAL NOTES OF STRUCTURAL DRAWINGS
  - BASIC WIND SPEED, V_{3s} : 90 MPH
  - WIND IMPORTANCE FACTOR, I_W : 1.15
  - WIND EXPOSURE : B
  - HEIGHT & EXPOSURE ADJUSTMENT FACTOR : 1.19
  - P_{ROOF} : +15.9/-17.3 PSF, FIELD
  - P_{ROOF} : +15.9/-20.3 PSF, EDGE
  - P_{ROOF} : +15.9/-20.3 PSF, CORNER
  - P_{WALL} : +17.4/-18.8 PSF, FIELD
  - P_{WALL} : +17.4/-23.3 PSF, CORNER

$K_d = 0.85$ (TABLE 6-4)

C_p : WINDWARD → $C_p = 0.8$
LEEWARD → $C_p = -0.5$ [-0.3]

$K_{zt} = 1.0$

$G = 0.830$ [0.799]

K_z: (TABLE 6-3)

0'-15'	0.57
20'	0.62
25'	0.66
30'	0.70
40'	0.76
50'	0.81
60'	0.85

K_h : 0.83 FOR $h = 55'$

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

0'-15'	11.6 PSF
20'	12.6
25'	13.4
30'	14.2
40'	15.4
50'	16.4
60'	17.2

$q_h = 0.00256 K_h K_{ht} K_d V^2 I = 16.8$ PSF

WIND LOAD CALCS (CONT'D)

[EAST-WEST VALUES]

$P_w = qGC_p = q(0.83)(0.8)$

0-15'	7.7 PSF	[7.4]
20'	8.3	[8.0]
25'	8.9	[8.6]
30'	9.4	[9.0]
40'	10.2	[9.8]
50'	10.9	[10.5]
60'	11.4	[11.0]

$P_e = (16.8)(0.83)(-0.5) = -7.0 \quad [-6.7]$

NET PRESSURE (P_{NET})

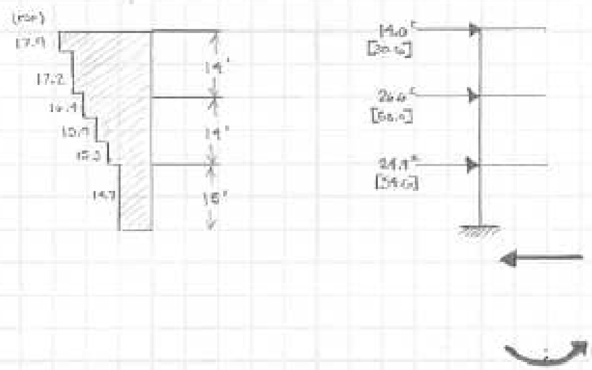
0-15'	(7.7) + (7.0) = 14.7 PSF	[14.1]
-20'	= 15.3	[14.7]
-25'	= 15.9	[15.3]
-30'	= 16.4	[15.7]
-40'	= 17.2	[16.5]
-50'	= 17.9	[17.2]
-60'	= 18.4	[17.7]

• FLOOR SHEAR LOADS

$F_1 = [(14.7)(7.5) + (15.3)(5) + (15.9)(2)](114) = 24.9\text{ k} \quad [54.6^k]$

$F_2 = [(15.9)(3) + (16.4)(5) + (17.2)(6)](114) = 26.6\text{ k} \quad [58.0^k]$

$F_3 = [(17.2)(4) + (17.9)(3)](114) = 14.0\text{ k} \quad [30.6^k]$



BASE SHEAR
65.5 k [143.2 k]

BASE MOMENT
1746.9'k [3816.8'k]

Appendix E

Seismic Load Analysis

SEISMIC LOAD CALCULATIONS (ASCE 7-02)

- DESIGN VALUES FROM GENERAL NOTES OF STRUCTURAL DRAWINGS
  - SEISMIC USE GROUP: II
  - SEISMIC DESIGN CATEGORY: B
  - $S_{DS} = 0.19$
  - $S_{D1} = 0.05$
  - SITE CLASS: B
  - DESIGN BASE SHEAR: 895 KIPS
  - SEISMIC RESISTING SYSTEM: CONCENTRICALLY BRACED FRAMES (STRUCTURAL STEEL SYSTEM NOT SPECIFICALLY DESIGNED FOR SEISMIC RESISTANCE.)
  - ANALYSIS PROCEDURE: EQUIVALENT LATERAL FORCE PROCEDURE

$I = 1.25$ (TABLE 9.1.4)

$S_{MS} = 25\ \%g$ (FIGURE 9.4.1.1 (a))

$S_{M1} = 6\ \%g$ (FIGURE 9.4.1.1 (b))

$F_a = F_v = 1.0$ (TABLE 9.4.1.2.4)

$S_{DS} = \frac{2}{3} S_{MS} = 0.167\ g$

$S_{D1} = \frac{2}{3} S_{M1} = 0.04\ g$

$T_o = 0.2\ S_{D1}/S_{DS} = 0.048\ s$

$T_s = S_{D1}/S_{DS} = 0.24\ s$

$R = 5$ RESPONSE MOD. FACTOR

$W_o = 2$ SYSTEM OVERSTRENGTH FACTOR

$C_d = 4.5$ DEFLECTION AMP. FACTOR

(TABLE 9.5.2.2 ORDINARY STEEL CONCENTRICALLY BRACED FRAMES

$C_s = \frac{S_{DS}}{R/I} = 0.06$

$T = T_a = C_t h_n^x = (0.02)(43)^{(0.75)} = 0.336 < C_u(0.1N) = 0.51$

$V = C_s W$

SEISMIC LOAD CALCS (CONT'D)

$V = C_s W = (0.06)(14100) = 846\ k$ BASE SHEAR

* VERY COMPARABLE TO DESIGN VALUE OF 895^k

$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$ $k = 1.0$ FOR $T \leq 0.5s$

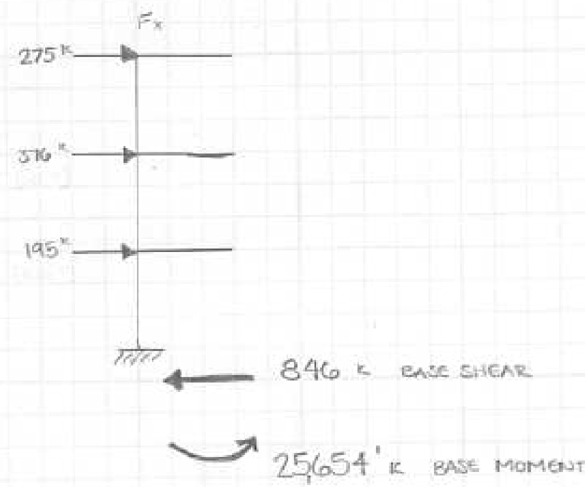
$C_{ROOF} = \frac{(2010)(43)}{(266131)} = 0.325$

$C_3 = \frac{(4079)(29)}{(266131)} = 0.444$

$C_2 = \frac{(4094)(15)}{(266121)} = 0.231$

$F_x = C_{vx} V$

$F_{ROOF} = 275\ k$
$F_3 = 376\ k$
$F_2 = 195\ k$



Appendix F

Lateral Bracing Member Spot Check