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Structural Option
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Barshinger Life Science & Philosophy Building

Structural Technical Report #3

Lateral Systems Analysis & Confirmation Design

Executive Summary

Tech Report #3 further investigates the lateral force resisting system of the Barshinger Life Science and Philosophy Building. The structure utilizes a system of ten (10) concentrically braced steel frames placed throughout the building. The braces are composed of wide-flange A992 horizontal and vertical members with A992 ½-inch thick HSS diagonal braces. The loads experienced by the frames are calculated in detail in the report.

The analysis of the lateral system was completed using preliminary calculations from Tech Report #1. Distribution of lateral loads was accomplished according to the relative stiffness of the frames. STAAD.Pro structural analysis software was utilized to determine frame stiffness and to spot-check the diagonal braces of two critical frames.

The basic findings of this report are listed below:

- The seismic load on the building is more than 6 times greater than the wind load. As a result, the governing load combination from ASCE7-02 is 1.2D + 1.0E + 0.5L + 0.2S.
- Although the frames appear to be symmetrical located about the structure's major axes, varying rigidities produce an eccentricity from the building's center of mass, causing torsion in the structure that must be accounted for by the braced frames.
- Story drift and overall drift is well within the H/400 limit.
- The HSS diagonal braces are suitable for the applied lateral loads.
- Overturning is not a concern for the building given its low vertical profile and wide base.
- Overall, the system is fairly inefficient and does not approach its capacity under the assumed loadings.

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 Lateral Force Resisting System

2.1 Overview

The structure's main lateral force resisting system is composed of ten concentrically braced steel frames of varying sizes. These frames typically utilize W12 shapes for the vertical and W14 shapes for the horizontal members with ½-inch thick HSS shapes for the braces. All of the steel members in the frames are specified to be A992 steel. The ten frames are located throughout the structure according to the Figure 2.1.1 below. The basic structure of each frame can be seen in Figure 2.1.2 on the next page.

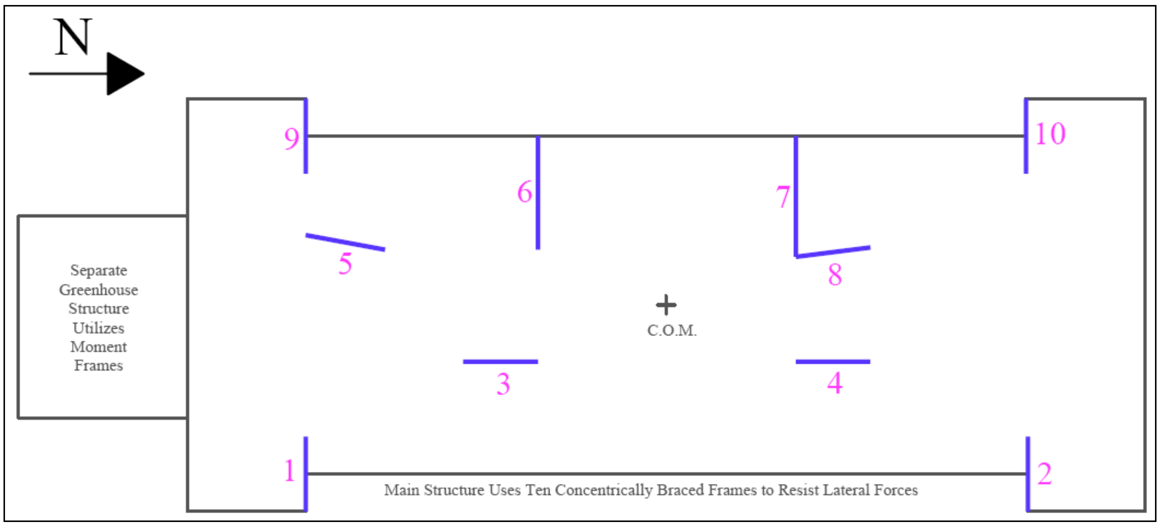


Figure 2.1.1 Layout of the 10 Concentrically Braced Frames

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 will focus on the concentrically braced frames of the main building, and not the moment frames of the greenhouse.

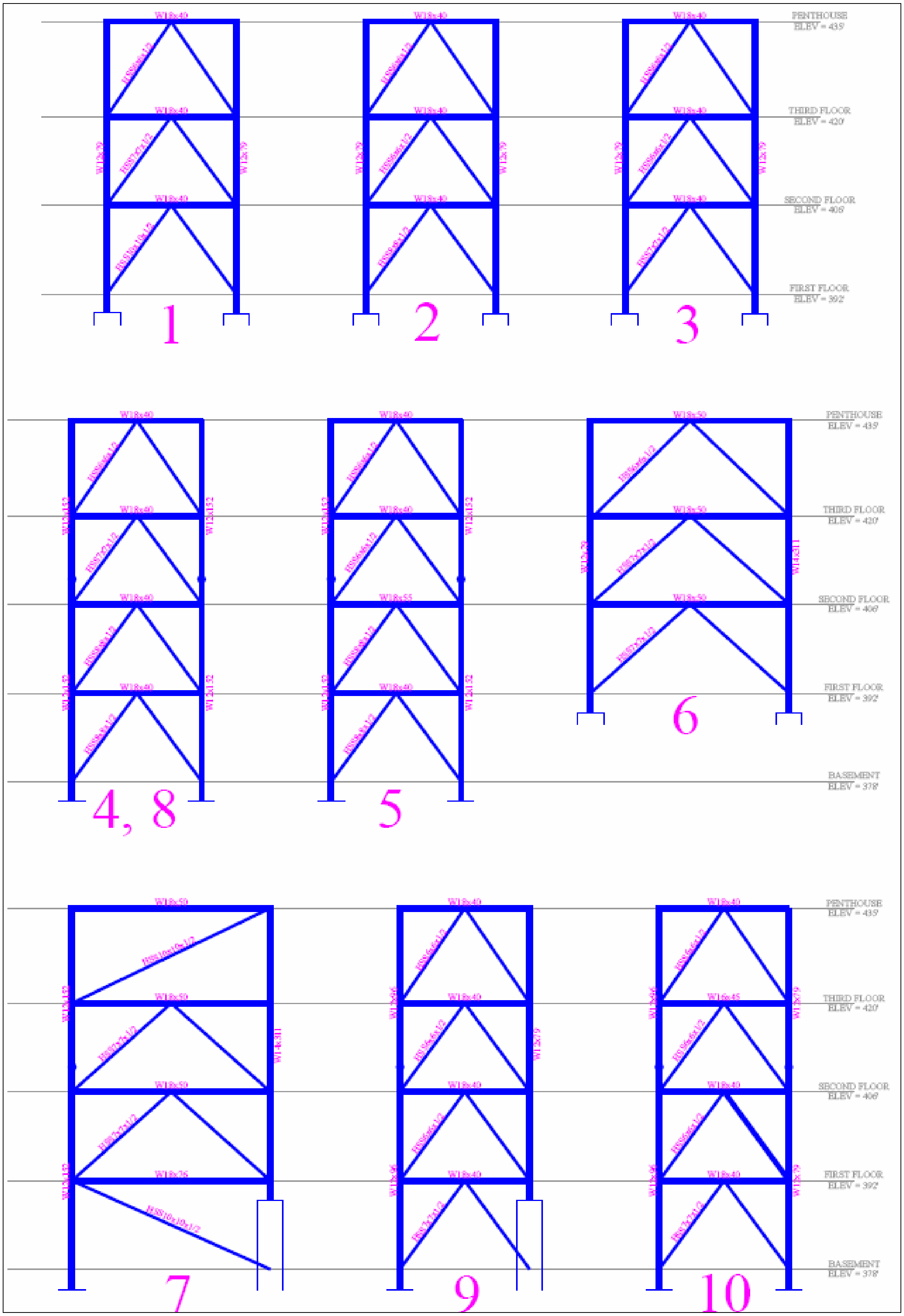


Figure 2.2.1 The Ten (10) Concentrically Braced Frames in the Main Lateral Force Resisting System

2.2 Load Distribution

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. All of the structural steel is specified as A992.

The use of composite slab construction, as described above, is very good for the distribution of lateral forces to the braced frames designed to resist them. I cannot foresee any problems areas for the transfer of lateral loads to the braced frames.

3.0 Lateral Loads

The lateral loads for the Barshinger Building were analyzed in Technical Report #1 using ASCE7-02. The calculations are also located in Appendices A and B for convenience. The results of that analysis are shown in the lateral loading diagrams pictured below in Figures 3.0.1 and 3.0.2. Seismic loads were found to be significantly larger than wind loads. This is due in large part to the low profile of the structure and the heavy nature of the materials used for the floor slabs and the exterior walls.

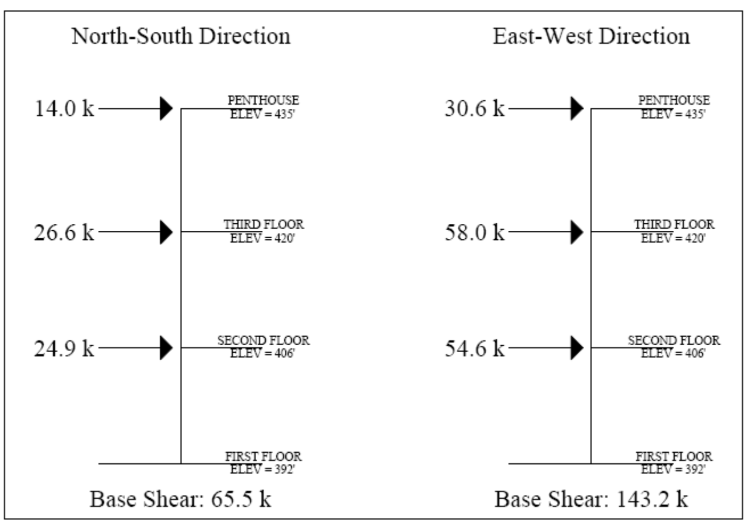


Figure 3.0.1 Story Wind Forces

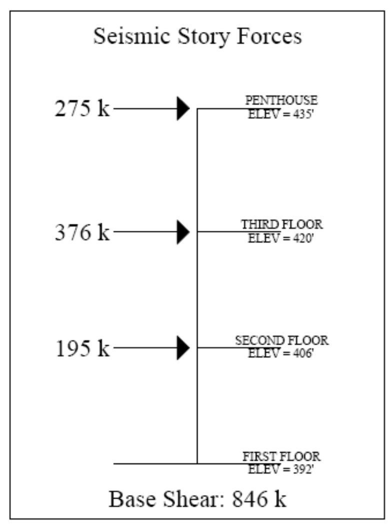


Figure 3.0.2 Story Seismic Forces

I will use the design base shear value of 865 kilo-pounds in the analyses for this report as it is more conservative and potentially more accurate than my calculated value.

4.0 Load Combinations

The load combinations are described in ASCE 7-02:

- 1.4 D
- 1.2D + 1.6L + 0.5S
- 1.2D + 1.6S + (0.5L or 0.8W)
- 1.2D + 1.6W + 0.5L + 0.5S
- 1.2D + 1.0E + 0.5L + 0.2S
- 0.9D + (1.6W or 1.0E)

The seismic forces are nearly six times larger than the wind forces. Therefore, it is no surprise that the controlling load combination is 1.2D + 1.0E + 0.5L + 0.2S. This is the load case that will be used to check critical members in the lateral force resisting system.

5.0 Lateral Load Distribution to Individual Frames

In my analysis, the lateral loads are distributed to the individual braced frames based on the stiffness of each frame. I used STAAD.Pro structural modeling software to determine the stiffness of each frame. Stiffness can be found by determining the maximum displacement of a frame caused by a 1 kilo-pound load. Then, I created an Excel spreadsheet to calculate the direct base shear experienced by each frame. The results are tabulated in Figure 5.0.1 below.

Frame	displ. per 1-kip load	k	Dir.	% Dir. Load	Direct Shear
1	0.63	1.59	E-W	2.9%	25.0
2	0.735	1.36	E-W	2.5%	21.5
3	0.91	1.10	N-S	5.5%	47.7
4	0.163	6.13	N-S	30.8%	266.0
5	0.152	6.58	N-S	33.0%	285.3
6	0.145	6.90	E-W	12.6%	108.8
7	0.032	31.25	E-W	57.0%	493.1
8	0.163	6.13	N-S	30.8%	266.0
9	0.202	4.95	E-W	9.0%	78.1
10	0.114	8.77	E-W	16.0%	138.4
Base Shear		865	kips		

Figure 5.0.1 Direct Shear Distribution

Although the frames appear to be placed evenly around the structure's center of mass, there is a significant eccentricity caused by the varying stiffness of the frames. The next stage of the analysis included locating the center of rigidity and the torsion shear loads. These results have been tabulated in Excel and can be seen in Figure 5.0.2 below.

Frame	k	x-coord	y-coord	k*d	k*d^2	(kd) / Σ(kd^2)	Torsional Shear
1	1.59	31.7	-	187.8	22226.5	0.0010	19.6
2	1.36	225.0	-	102.0	7653.1	0.0006	10.6
3	1.10	-	40.0	21.0	401.8	0.0001	0.4
4	6.13	-	40.0	117.3	2243.0	0.0007	2.2
5	6.58	-	70.0	71.6	778.7	0.0004	1.4
6	6.90	93.8	-	387.4	21756.3	0.0022	40.4
7	31.25	162.8	-	401.1	5147.0	0.0022	41.9
8	6.13	-	70.0	66.7	726.1	0.0004	1.3
9	4.95	31.7	-	585.8	69320.3	0.0033	61.1
10	8.77	225.0	-	657.9	49342.5	0.0037	68.7

C.O.R.	150.0	59.1
C.O.M.	128.3	55.2
e	21.7	4.0

Base Shear	865	kips	
Torsion (N-S)	3420	ft-kips	$M = V*e_y$
Torsion (E-W)	18741	ft-kips	$M = V*e_x$

Figure 5.0.2 Torsion Shear Distribution

In order to find the maximum lateral load that a frame could experience, the direct shear and the torsion shear loads were added. The total base shear of each braced frame is tabulated in Figure 5.0.3 below. The individual overturning moment are also calculated in the table.

Frame	Direct Shear	Eccen. Shear	Total Shear (kips)	Overturning Moment (ft-k)
1	25.0	19.6	44.6	1324
2	21.5	10.6	32.1	952
3	47.7	0.4	48.1	1425
4	266.0	2.2	268.3	11916
5	285.3	1.4	286.6	12732
6	108.8	40.4	149.2	4425
7	493.1	41.9	535.0	23763
8	266.0	1.3	267.3	11873
9	78.1	61.1	139.2	6185
10	138.4	68.7	207.1	9198

Figure 5.0.3 Individual Frame Base Shears & Overturning Moments

6.0 Critical Member Spot Checks

Two frames were fully analyzed using STAAD.Pro for individual member forces and story drifts. Frame #7 was selected for further evaluation because it experiences the highest lateral forces. Frame #3 was also selected for further evaluation because it represents the lowest stiffness and maybe be subject to high drift. The loading of each frame is detailed in Appendix C. A summary of the drift results is tabulated in Figure 6.0.1 below. The drift results were then compared with H/400, a common drift limit for designers. Excessive drift can jeopardize the building envelope, destroy the surface finishes, etc. The quick analysis resulted in acceptable story drifts. In fact, the braced frame system probably could be designed to be more efficient.

Frame	1st Story Drift	2nd Story Drift	3rd Story Drift	Total Drift	H/400	
3	0.369	0.152	0.196	0.717	1.26	OK
7	0.434	0.047	0.050	0.531	1.26	OK

Figure 6.0.1 Story Drift Values - Frame #3 & Frame #7

The individual bracing members were checked against the tensile and compressive limits found in AISC's "Manual of Steel Construction: LRFD," 3rd Edition. This comparison can be seen in Figure 6.0.2 below. All of the member forces were well under the allowable limits.

Frame	Bracing Member	Length (ft)	Maximum Axial Force	Allowable Force (k)		
				Tension	Compression	
3	6x6x$^1/_2$	18.0	14.1	318.0	203.0	OK
	7x7x$^1/_2$	17.2	82.8	378.0	303.0	OK
7	7x7x$^1/_2$	20.1	137.5	378.0	259.0	OK
	10x10x$^1/_2$	33.8	194.1	561.0	317.0	OK

Figure 6.0.2 Bracing Member Axial Forces - Frame #3 & Frame #7

7.0 Overturning

A quick analysis of building overturning was also completed. The building maintains a low profile with a ratio of length to height greater than 2 in the narrow direction. The structure also utilizes heavy building materials for the floor system and building envelope. When these two facts are looked at in combination, one can easily see that overturning is not an issue for the building. The numerical data to support this statement can be found in Appendix E.

8.0 Conclusions

The purpose of Tech Report #3 was to further investigate the lateral force resisting system. The Barshinger Life Science and Philosophy Building utilizes a system of ten concentrically braced steel frames placed throughout the building. The braces are composed of wide-flange A992 horizontal and vertical members with A992 HSS diagonal braces. A summary of the findings can be found in the Executive Summary on the first page.

This report has opened up the possibility for a lateral system redesign proposal. Architectural constraints probably forced the engineers to limit the number of frames in the building. The braced frames are not very efficient. Neither the drift limit nor the allowable axial forces are close to fully developed in the current system. The designed HSS braces are fairly large and force the building to have thicker walls around the braced frames. The column and beam shapes are consistent with the rest of the building skeleton, so the only changes would have to be made in the HSS shapes. By adding additional frames, the lateral force resisting system could be made more efficient overall and less bulky.

Appendix

Appendix	Description
A	**Wind Load Calculations**
B	**Seismic Load Calculations**
C	**Loading of Critical Frames**
D	**Overturning**

Appendix A

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})

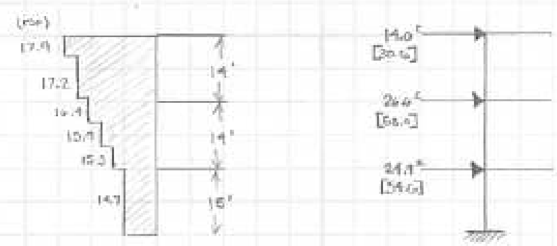
0-15'	(7.7)+(7.0) = 14.7 PSF	[14.1]
-20'	= 15.3	[14.7]
-25'	= 15.9	[15.3]
-30'	= 16.4	[15.7]
-40'	= 17.2	[16.5]
-50'	= 17.9	[17.2]
-60'	= 18.4	[17.7]

• FLOOR SHEAR LOADS

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

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

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



BASE SHEAR
65.5 k [143.2 k]

BASE MOMENT
1746.9'k [3816.8'k]

Appendix B

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)

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

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

$T_0 = 0.2\ S_{D1}/S_{DS} = 0.048\ s$

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

$R = 5$ RESPONSE MOD. FACTOR

$W_0 = 2$ 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 (CON'T'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 ≤ 0.5 s

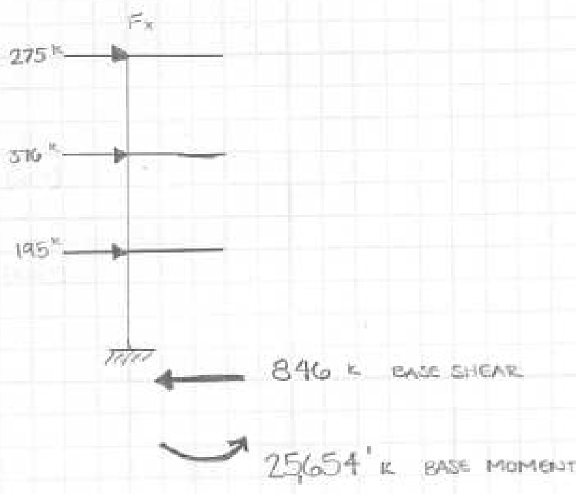
$C_{ROOF} = \frac{(2010)(43)}{(266131)} = 0.325$

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

$C_2 = \frac{(4094)(15)}{(266131)} = 0.231$

$F_x = C_{vx} V$

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

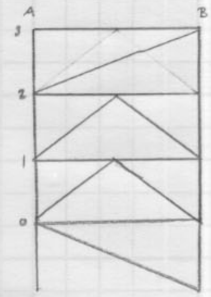


Appendix C

Loading of Critical Frames

LOADING CRITICAL FRAMES — APPLY GRAVITY LOADS TO COLUMNS

FRAME #7



TRIBUTARY AREAS OF COLUMN A + B DIFFER

TRIB A: (20)(30) = 600 FT²

TRIB B: (15)(10) + (15)(35) = 675 FT²

→ USE 675 FT² TO SIMPLIFY

D_{ROOF}: (60 PSF)(675)/1000 = 40.5 K

D_{FLOOR}: (147 PSF)(675)/1000 = 99.3 K

L_{ROOF} (S): (28 PSF)(675)/1000 = 18.9 K

L_{FLOOR}: (100 PSF)(675)/1000 = 67.5 K

SEISMIC

BASE SHEAR: 535 K

E_1 = 535 (195/846) = 123 K

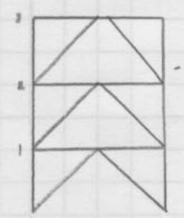
E_2 = 535 (376/846) = 238 K

E_3 = 535 (275/846) = 174 K

DISTRIBUTE TO FLOORS ACCORDING TO RATIO OF FLOOR LOAD TO TOTAL BASE SHEAR IN FULL BUILDING ANALYSIS (APPENDIX B)

FRAME #3

TRIBUTARY AREA: (20)(30) = 600 FT²



D_{ROOF}: (60)(600)/(1000) = 36 K

D_{FLOOR}: (147)(600)/(1000) = 88.2 K

L_{ROOF}: (28)(600)/(1000) = 16.8 K

L_{FLOOR}: (100)(600)/(1000) = 60 K

SEISMIC

BASE SHEAR: 48.1 K

E_1 = 48.1 (195/846) = 11 K

E_2 = 48.1 (376/846) = 21.4 K

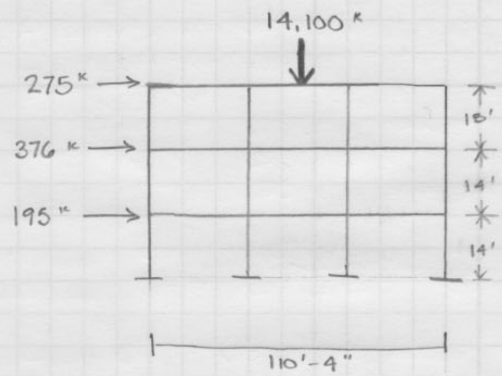
E_3 = 48.1 (275/846) = 15.6 K

Appendix D

Overturning

OVERTURNING

E-W DIRECTION - CRITICAL DIRECTION



$\circlearrowright M = 25{,}654\ 'k$ ∴ FROM SEISMIC ANALYSIS

RESISTING MOMENT OF DEAD LOAD

$\circlearrowleft M = (14100)(110.33/2) = 777{,}850\ 'k$

$25{,}654\ 'k \ll 777{,}850\ 'k$

EVEN WITH AN UPLIFT MOMENT EQUAL TO OVERTURNING THE DEAD LOAD CAN RESIST OVERTURNING.