William W. Wilkins Professional Building

Columbus, Ohio

Technical Assignment 3

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Michelle Benoit
Structural Option
AE 481W
Senior Thesis
Faculty Advisor:
Dr. Boothby

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Executive Summary

This report consists of a detailed analysis of the lateral system in the William W. Wilkins Professional Building. To begin wind and seismic loads are determined according to ASCE-7-05. Following are strength, drift and overturning moment checks on the braced frames.

The William W. Wilkins Professional Building is a 6 story, 112,000 sq. ft. medical office building located in Columbus, Ohio. Costing approximately $7.4 Million, it is essentially an addition to the Grant Riverside hospital across the street. These buildings are connected by a pedestrian bridge from the third floor. Enclosed by brick veneer, precast concrete and spandrel glass panels the exterior is non-load bearing. The floor system is designed for composite action supported on W12 columns. Lateral framing consists of five braced frames utilizing tube steel. Two frames run North-South with the remaining three running East-West.

The center of mass of the building is only slightly off center from the center of stiffness in the Wilkins building. It was found, due to the wonderful symmetry of the frame placement that torsion effects are negligible. Load distribution consisted of, on average, 31.5% being taken by WB1-1 and WB1-7 and 36.5% being taken by WB3 for the East-West distribution. The distribution in the North-South direction was equal.

According to base shear evaluations wind in the East-West direction controls while seismic controls in the North-South direction. However, all load cases were evaluated in RISA. From the RISA analysis, it was found that, in general, all members are ok. Six floor beams were found to be overstressed. This could be due to design differences between ASD and LRFD or the inability of RISA to take into account the composite action of the beams. It was also found that inter story and total building drift were ok by ASCE-7-05. Overturning moments were also evaluated for the building. It was found that the weight of the building is more than sufficient to resist the overturning moment from wind and seismic forces.

Introduction

The William W. Wilkins Professional Building is a 6 story, 112,000 sq. ft. medical office building located in Columbus, Ohio. Costing approximately $7.4 Million, it is essentially an addition to the Grant Riverside hospital located across the street. These buildings are connected by a pedestrian bridge from the third floor. Enclosed by brick veneer, precast concrete and spandrel glass panels the exterior is non-load bearing.

The floor system in the Wilkins building is designed for composite behavior. Floor slabs consist of 3 ½″ normal weight concrete on 2″ 18 gage composite steel deck reinforced with W2.1xW2.1 welded wire fabric (WWF). ¾″ diameter by 4 ½″ long headed studs are spaced evenly across members to transfer loads. Roof framing is designed as non-composite. Columns are ASTM 992 Grade 50 rolled W12 steel shapes with splices on the third and fifth floors. The building is supported on caissons drilled approximately 25′ down.

For this report, ASCE-7-05 was used to calculate wind and seismic loading. When analyzing the lateral resisting systems in RISA, IBC 2000 Strength design was used to determine load cases.

Included in this report is a detailed analysis of the lateral system in the Wilkins building. This includes calculated wind and seismic loads, distribution of forces, analysis of frames, drift check and overturning moment check.

Lateral System

Lateral loads are resisted in the Wilkins building using braced frames. Two frames spanning North-South are located near the elevator shafts. Frames spanning East-West are split with one located near the elevator shafts, one on the exterior South-West bay and one on the exterior North-West bay. See Figure 1 below for frame locations. Lateral bracing in these frames are ASTM A500 Grade B tubes ranging in size from TS5x5x.1875 to TS8x8x.25. A typical braced frame is shown in Figure 2 below. The tube steel is welded to gusset plates that connect to the main framing members.

Figure 1: Braced Frame Locations

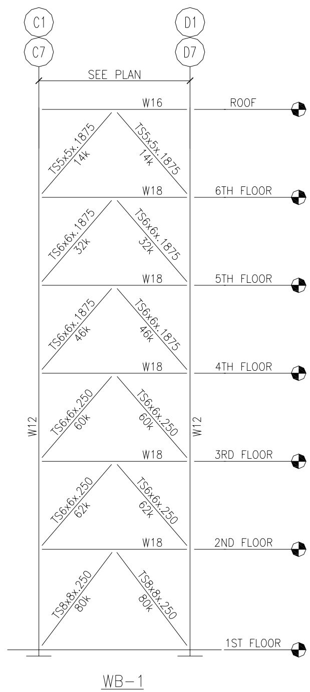


Figure 2: Typical Braced Frame

Lateral Loads

<u>My calculated wind loads:</u>

For complete calculations, see Appendix.

$P = qGCp - q_i(GCp_i)$						
	Windward		Leeward		Total	
height	N-S	E-W	N-S	E-W	N-S	E-W
0-15'	6.83	6.83	-4.68	-7.04	11.51	13.87
20'	7.43	7.43	-4.68	-7.04	12.11	14.47
25'	7.91	7.91	-4.68	-7.04	12.59	14.95
30'	8.39	8.39	-4.68	-7.04	13.07	15.43
40'	9.11	9.11	-4.68	-7.04	13.78	16.15
50'	9.71	9.71	-4.68	-7.04	14.38	16.75
60'	10.19	10.19	-4.68	-7.04	14.86	17.23
70'	10.67	10.67	-4.68	-7.04	15.34	17.71
80'	11.15	11.15	-4.68	-7.04	15.82	18.19
84.67	11.27	11.27	-4.68	-7.04	15.94	18.31

Table 1: Wind Loads

<u>My calculated seismic loads:</u>

For complete calculations, see Appendix.

Story	F_x(k)
2	11.88
3	23.06
4	34.01
5	44.92
6	55.91
R	23.07

Table 2: Seismic Loads

Load Combinations

Based on ASCE 7 the load cases evaluated during analysis were:

1.4D
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)

Distribution of Forces

It was found that in the East-West direction wind controls, where as, in the North-South direction seismic controls. As stated above, there are only two braced frames in the North-South direction; thus, the load distribution is equal. On the other hand, in the East-West direction, there are three braced frames. The stiffness of each floor was determined by applying a 100^k load at each floor individually and determining the deflection at that floor, remembering that stiffness is equal to $1/\Delta$. Based on these relative stiffnesses, the percentage of the load taken by each braced frame was calculated.

	% of Load Taken		
Floor	WB1-1	WB1-7	WB3
2	31.5	31.5	37
3	30.5	30.5	39
4	32.5	32.5	35
5	32.5	32.5	35
6	32	32	36
Roof	31.5	31.5	37

Table 3: Force Distribution

It was determined that the torsional component associated with the braced frames was negligible. The order of magnitude due to torsion was typically around 0.02^k.

	WB1-1	WB1-7	WB3	WB2-E	WB2-W
Distance	82.300	104.867	19.467	16.165	16.165
Stiffness	0.540	0.540	0.630	0.790	0.790
kd^2	3657.577	5938.427	238.747	206.433	206.433
F	0.018	0.023	0.005	0.005	0.005
J	10247.617				
M	25.600				

Table 4: Torsion Forces

Analysis

RISA Analysis:
Each braced frame was modeled in RISA. In modeling the braced frames, the existing sizes of members, as designed by the Engineer of Record, were imputed. A design check was then done based on the above noted load cases. It was found that the braced frames, in general, were acceptable. In WB1-1 and WB1-7 the second, third, and fourth floor members were slightly over stressed. This may be due to design method differences. The Engineer of Record used ASD whereas I utilized LRFD. Another possibility for the differences is that RISA does not consider the beams acting compositely. Further reasons for this inconsistency may be due to code changes as I used ASCE-7-05 to calculate loads whereas the Engineer of Record used ASCE-7-98, or I may have calculated different floor loads acting on each braced frame than originally used in the design.

Drift Check:
The largest total building drift observed was 1.23". The largest inter story drift occurred between floors four and five with a value of 0.241". From ASCE-7-05, the allowable drift can be calculated from the equation:

$$\Delta = 0.02H$$

This results in a typical inter story drift value of 0.266" and a total building drift of 1.69". Therefore, the Wilkins building is satisfactory in relation to drift.

Overturning Moment:
The largest overturning moment in the East-West direction is obtained from wind loading with a value of 11,509$'^{k}$. The largest overturning moment in the North-South direction is obtained from seismic loading with a value of 10,719$'^{k}$. The resisting moment is calculated below.

Building dead weight = 8035.20^k
Resisting Moment = $8035.20^k(50.835') = 408{,}469'^k$
$= 8035.20^k(93.58') = 751{,}962'^k$

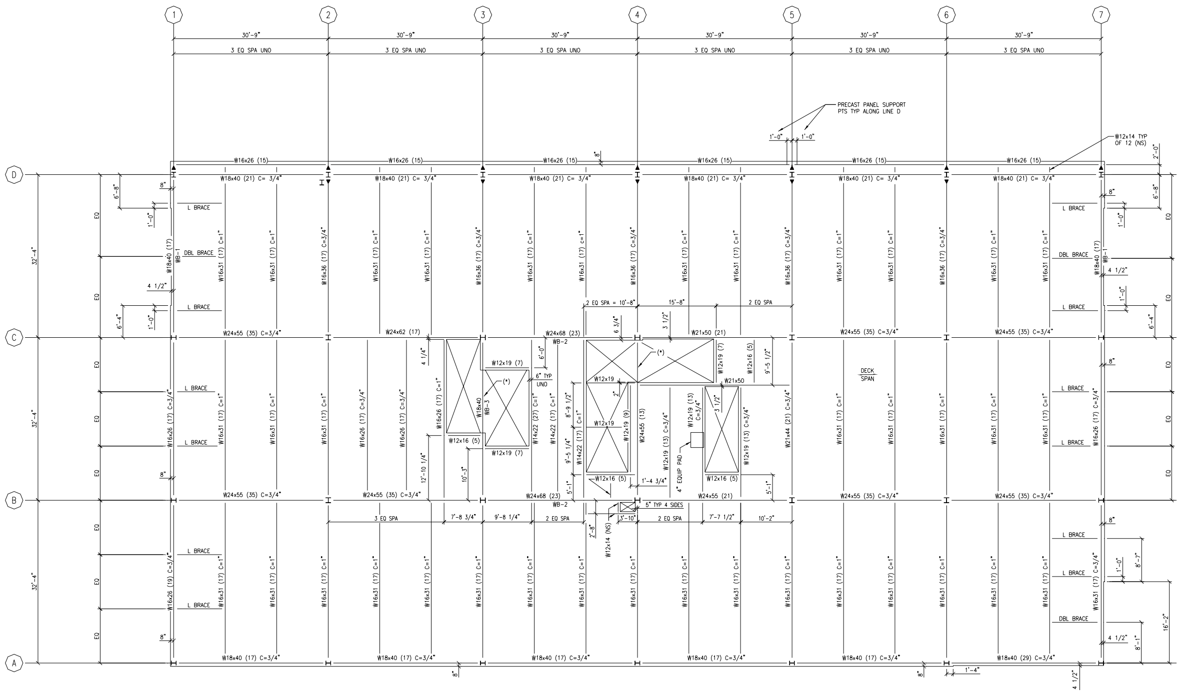
From the resisting moment values, it is easy to see that the dead weight of the building can resist the overturning moment of the building.

Conclusion

In conclusion, the lateral bracing is sufficient to resist both wind and seismic loads for Columbus, OH. This is the case in both strength and drift. The few floor beams that were calculated to be overstressed may have been found so due to code changes, differing loads, or differing design methods from the original design. Furthermore, the dead weight of the Wilkins building is sufficient to resist the overturning moment induced by wind loads.

Appendix

A1. Typical Floor Plan



A2. Wind Loads

height	K_z	q_z
0-15'	0.57	10.05
20'	0.62	10.93
25'	0.66	11.63
30'	0.7	12.34
40'	0.76	13.40
50'	0.81	14.28
60'	0.85	14.98
70'	0.89	15.69
80'	0.93	16.39
84.67	0.94	16.57

G	0.85
Gcpi	0.18

Cp		
	Leeward	Windward
N-S	0.332	0.8
E-W	0.5	0.8

$P=qGCp - q_i(GCp_i)$						
	Windward		Leeward		Total	
height	N-S	E-W	N-S	E-W	N-S	E-W
0-15'	6.83	6.83	-4.68	-7.04	11.51	13.87
20'	7.43	7.43	-4.68	-7.04	12.11	14.47
25'	7.91	7.91	-4.68	-7.04	12.59	14.95
30'	8.39	8.39	-4.68	-7.04	13.07	15.43
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84.67	11.27	11.27	-4.68	-7.04	15.94	18.31

			Story Force (kip)		Cumulative Shear (kip)		OM (ft-kip)	
Floor	Height	Trib. Ht	N-S	E-W	N-S	E-W	N-S	E-W
1	0.00	0.00	0.00	0.00	110.30	236.70	5396.45	11508.87
2	16.33	14.84	18.00	39.70	110.30	236.70	293.94	648.30
3	29.67	13.34	18.10	39.20	92.30	197.00	537.03	1163.06
4	43.00	13.33	19.30	41.40	74.20	157.80	829.90	1780.20
5	56.33	13.34	20.30	43.20	54.90	116.40	1143.50	2433.46
6	69.67	14.17	22.50	47.60	34.60	73.20	1567.58	3316.29
Roof	84.67	7.50	12.10	25.60	12.10	25.60	1024.51	2167.55

Wind Loads

Bldg: $h = 84'-8'' = 84.67'$
$101'-6''$ x $187'-2''$
↑ E-W ↑ N-S

Category II ⇒ $I = 1.0$

$V = 90\text{mph}$

$K_d = 0.85$
$K_{zt} = 1.0$

Exposure = B
K_h = Case 2 → 0.94

K_z - varies

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

$= 0.00256(1.0)(0.85)(90)^2(1.0) K_z$
$= 17.6256 K_z$

flexible if $f \le 1.0\text{Hz}$ $\quad f = \frac{1}{T}$

$T_a = C_t h_n^{x} \quad = 0.02(84.67)^{0.75} = 0.56$

Table 12.8-2 $\quad C_T = 0.02$
$x = 0.75$

$f = \frac{1}{0.56} = 1.8 > 1.0 \quad \therefore$ Rigid

$G = 0.85$

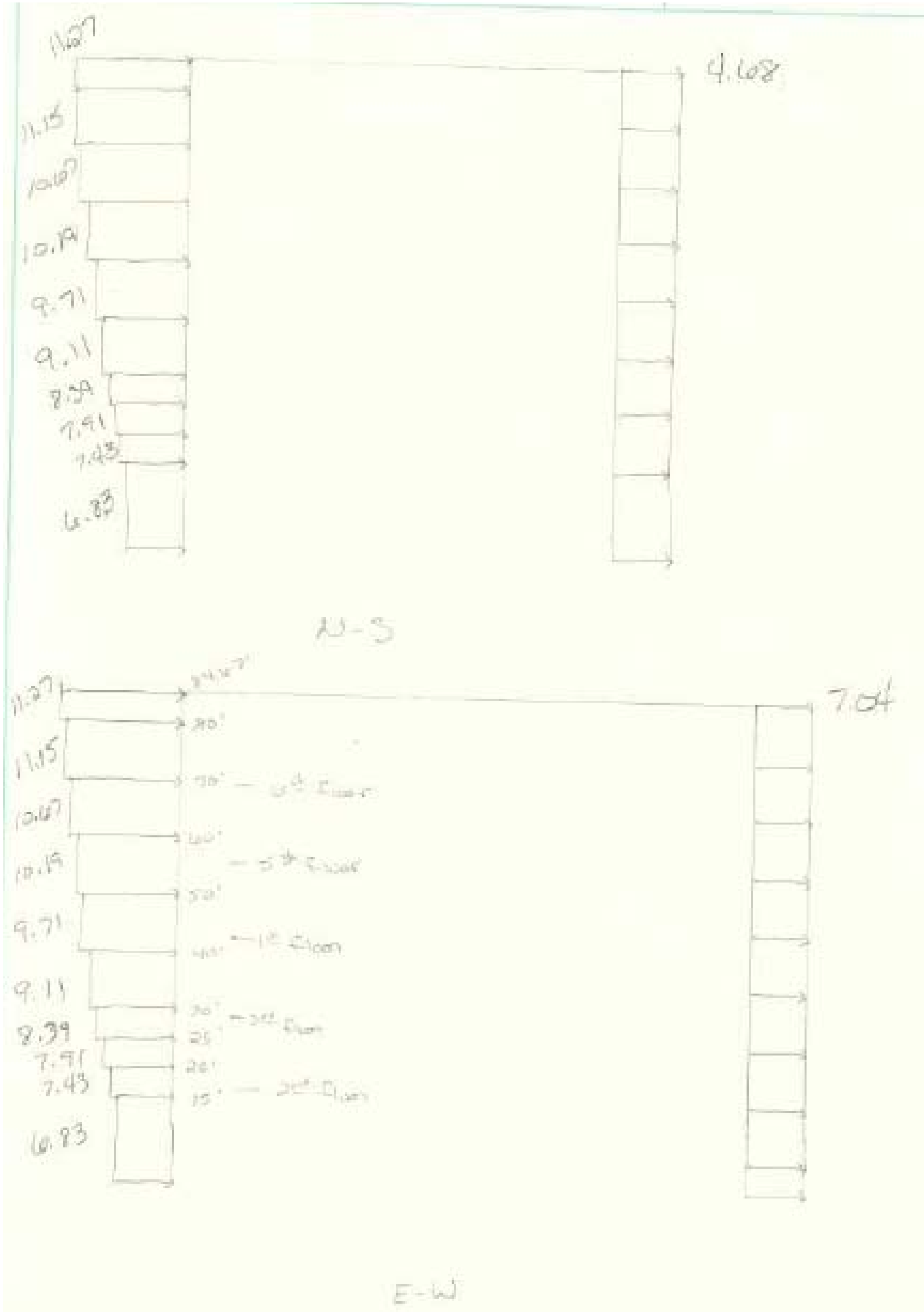
$GC_{pi} = \pm 0.18$ fig. 6-5

C_p: windward: 0.8
fig. 6-6 leeward: N-S = $187'-2'' / 101'-6'' = 1.84 \Rightarrow 0.332$
E-W = $101'-6'' / 187'-2'' \Rightarrow 0.5$

$P = q G C_p - q_i (GC_{pi})$

for windward $\quad q = q_z$
$q_i = q_h$

for leeward $\quad q_i = q_h$
$q = q_h$



2nd Floor

N-S = [6.83(6.835) + 7.43(5) + 7.91(3) + 4.68(14.835)] (101'-8")
= [107.56 + 69.43] (101'-8") = 18^k

E-W = [107.56 + 7.04 (14.835)] (187'-2")
= 39.7^k

3rd Floor

N-S = 101'-8" [6.335(9.11) + 8.39(5) + 7.91(2) + 4.68(13.335)]
= 18.1^k

E-W = 187'-2" [13.335(7.04) + 115.5]
= 39.2^k

4th Floor

N-S = 101'-8" [9.11(3.665) + 9.71(9.665) + 4.68(13.33)]
= 19.3^k

E-W = 187'-2" [7.04(13.33) + 127.24]
= 41.4^k

5th Floor

N-S = 101'-8" [9.71(0.333) + 10.19(10) + 10.67(3) + 4.68(13.335)]
= 20.3^k

E-W = 187'-2" [7.04(13.335) + 137.16]
= 43.2^k

6th Floor

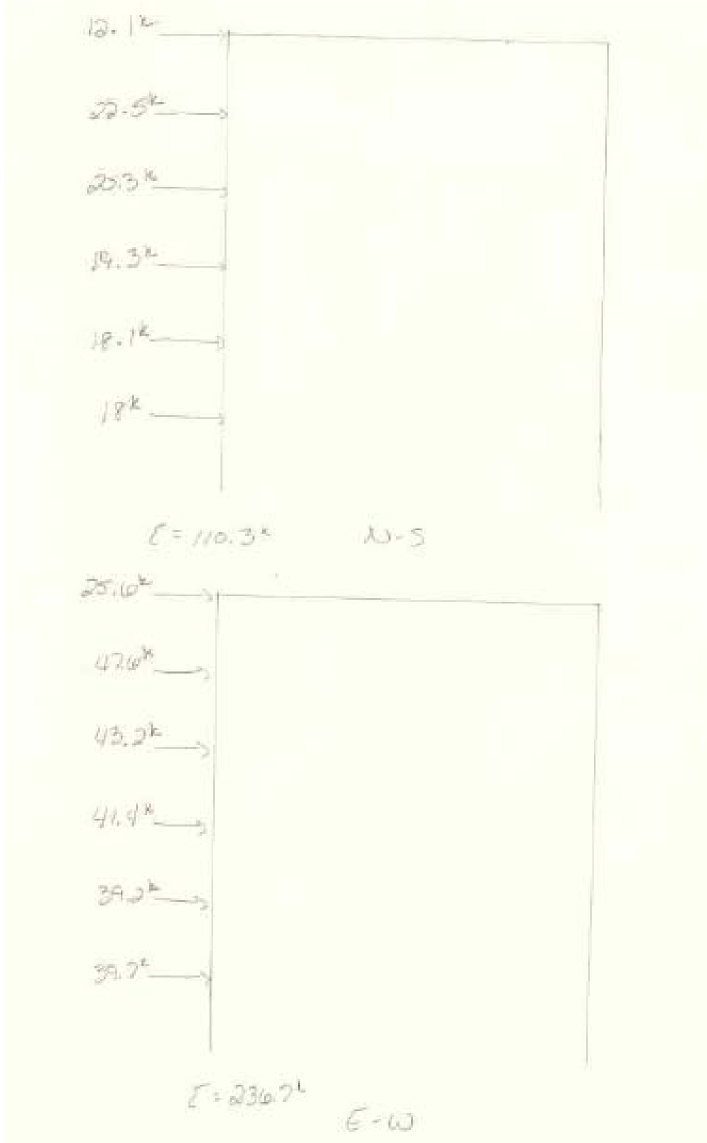
N-S = 101'-8" [10.67(7) + 11.15(7.17) + 4.68(14.17)]
= 22.5^k

E-W = 187'-2" [7.04(14.17) + 154.65]
= 47.6^k

Roof

N-S = 101'-8" [11.15(2.83) + 11.27(4.67) + 4.68(7.5)]
= 12.1^k

E-W = 187'-2" [7.04(7.5) + 84.2]
= 25.6^k



A3. Seismic

Story	V	k	h_x	Area (sq. ft.)	W_x (k)	$h_x^k W_x$	C_{vx}	F_x(k)	OM (ftk)
2	192.84	1.03	16.33	18023.40	1441.87	25603.69	0.06	11.88	10719.06
3	192.84	1.03	29.67	18906.30	1512.50	49680.23	0.12	23.06	684.1249
4	192.84	1.03	43.00	19029.30	1522.34	73279.97	0.18	34.01	1462.474
5	192.84	1.03	56.33	19029.30	1522.34	96777.58	0.23	44.92	2530.164
6	192.84	1.03	69.67	19029.30	1522.34	120461.98	0.29	55.91	3895.203
R	192.84	1.03	84.67	19029.30	513.79	49699.04	0.12	23.07	1953.043

$\sum W_x =$	8035.20
$\sum h_x^k W_x =$	415502.49

Earthquake Loads

Occupancy Cat. II $\Rightarrow I = 1.0$

Site Class C

$R = 5$

$S_s = 0.15$

$S_1 = 0.06$

$F_v = 1.7$

$F_a = 1.2$

$S_{MS} = F_a S_s$

$= 1.2(0.15) = 0.18$

$S_{M1} = F_v S_1$

$= 1.7(0.06) = 0.1$

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

$= \frac{2}{3}(0.18) = 0.12$

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

$= \frac{2}{3}(0.1) = 0.067$

Design Category

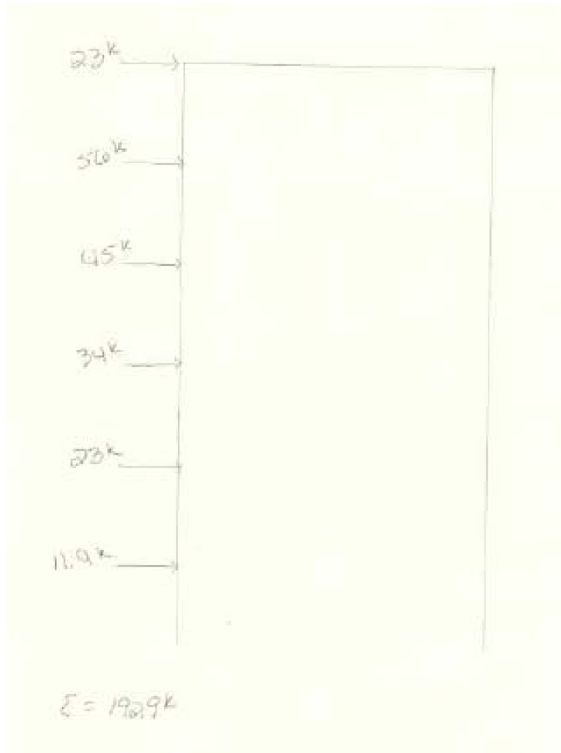
$S_{DS} = A$

$S_{D1} = A$

$C_s = \frac{S_{DS}}{\frac{R}{I}} = \frac{0.12}{5} = 0.024$

$T_a = 0.56$

$C_s \le \frac{S_{D1}}{T_a\left(\frac{R}{I}\right)} = \frac{0.067}{0.56(5)} = \boxed{0.024}$



Site class determined from geotechnical report.

Load distribution on entire building.

A4. Drift Calculations

WB-3

100k @ roof $\Delta H = 1.584$ @ roof $k = 0.63$

100k @ 2nd floor $\Delta H = .163$ @ 2nd floor $k = 6.13$

@ 3rd floor $\Delta H = .256$ @ 3rd floor $k = 3.9$

@ 4th floor $\Delta H = .479$ @ 4th floor $k = 2.09$

@ 5th floor $\Delta H = 0.75$ @ 5th floor $k = 1.33$

@ 6th floor $\Delta H = 1.087$ @ 6th floor $k = 0.92$

WB1-1 / WB1-7

2nd floor = 0.189" $k = 5.29$

3rd floor = 0.33" $k = 3.03$

4th floor = 0.509" $k = 1.96$

5th floor = 0.82" $k = 1.22$

6th floor = 1.215" $k = 0.82$

roof = 1.838" $k = 0.54$

$$k = \frac{1}{\Delta}$$

WB-3 2nd floor = 37%

3rd floor = 39%

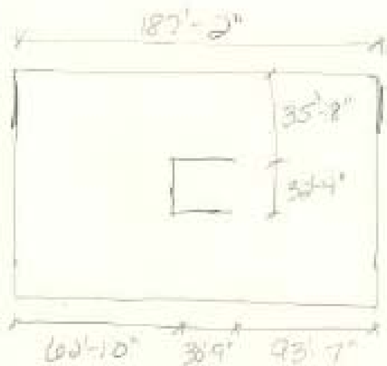
4th floor = 35% % Lateral forces

5th floor = 35%

6th floor = 36%

roof = 37%

WB 2	A	K
2nd Floor	0.166	6
3rd Floor	0.285	3.51
4th Floor	0.425	2.35
5th Floor	0.605	1.65
6th Floor	0.835	1.2
Roof	1.267	0.79



CM = (93'-7", 53'-10")

$\bar{y}(12) = 6(35.67) + 6(68) \Rightarrow \bar{y} = 51.835'$

$\bar{x}(5.29 + 5.29 + 6.13) = 5.29(187.167) + 6.13(62.833)$

$\bar{x} = 82.3'$; $e = 1'$

3rd = 81.54'

4th = 82.9'

5th = 82.7'

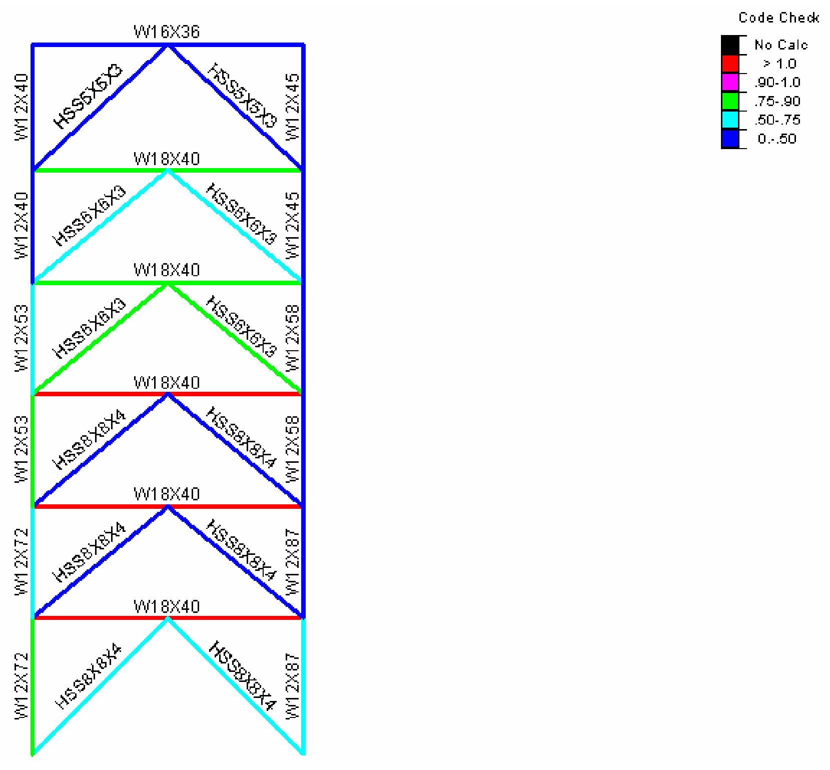
6th = 80.5'

Roof = 82.3'

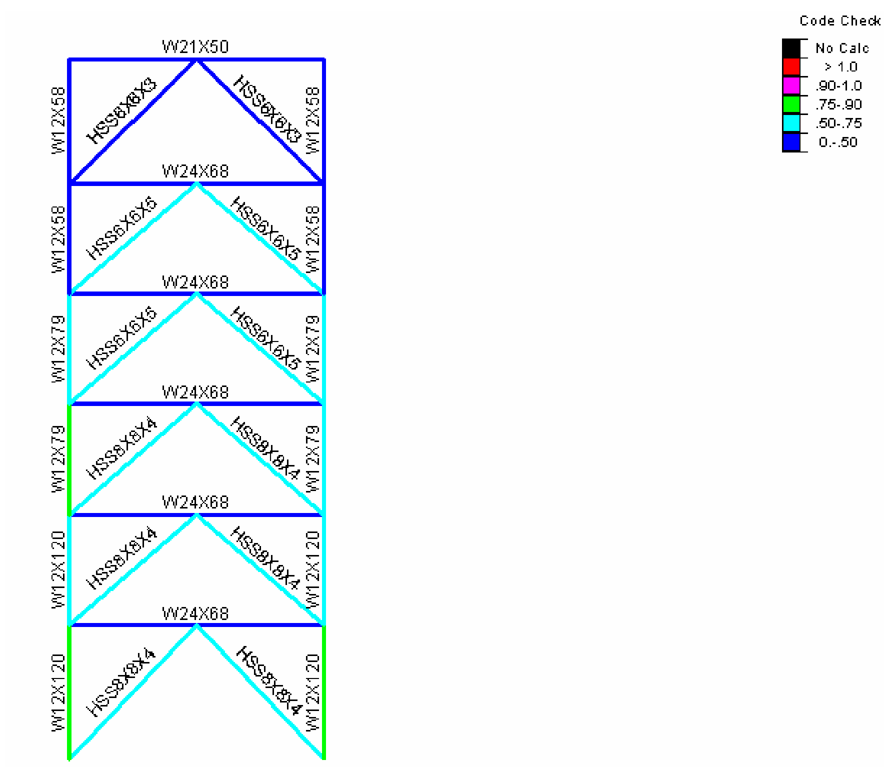
A5. RISA Analysis

The below figures are the various braced frames. The color code noted gives the results from equations H1-1a and H1-1b from the LRFD manual. Anything red is a failing member.

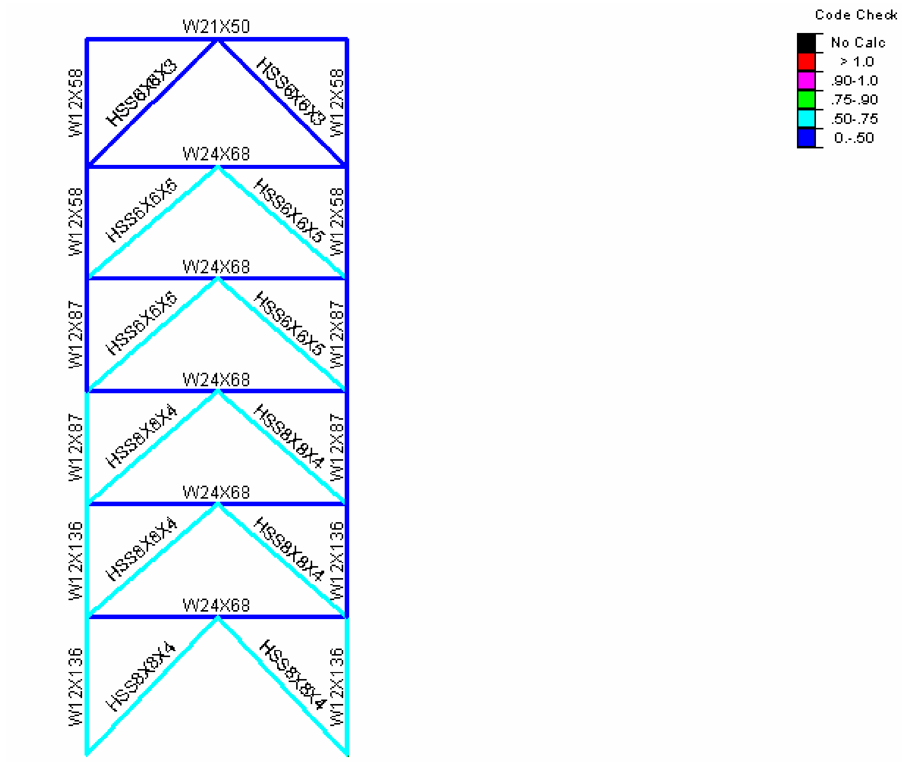
WB1-1 and WB1-7:



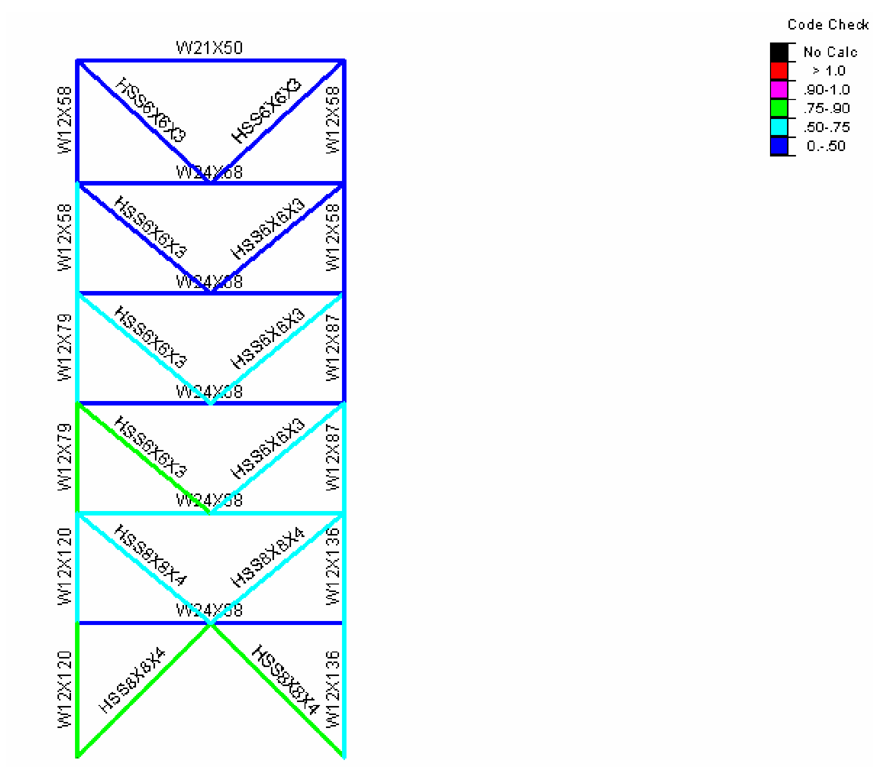
WB2-E:



WB2-W:


W21X50
HSS6X6X3
HSS6X6X3
W12X58
W12X58
W24X68
HSS6X6X5
HSS6X6X5
W12X58
W12X58
W24X68
HSS6X6X5
HSS6X6X5
W12X87
W12X87
W24X68
HSS8X8X4
HSS8X8X4
W12X87
W12X87
W24X68
HSS8X8X4
HSS8X8X4
W12X136
W12X136
W24X68
HSS8X8X4
HSS8X8X4
W12X136
W12X136
Code Check
No Calc
> 1.0
.90-1.0
.75-.90
.50-.75
0.-.50

WB3:


W21X50
HSS6X6X3
HSS6X6X3
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W24X68
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HSS8X8X4
W12X120
W12X136
W24X68
HSS8X8X4
HSS8X8X4
W12X120
W12X136
Code Check
No Calc
> 1.0
.90-1.0
.75-.90
.50-.75
0.-.50