The Harry and Jeanette Weinberg Center

Mercy Hospital Medical Office Building
Baltimore, MD

Technical Assignment 3

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Structural Option

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Executive Summary

It is the intent of this report to analyze the lateral force resisting system in The Harry and Jeanette Weinberg Center.

Assumptions

In this report I assumed that loads were evenly distributed and able to find their way to the braced frames. In reality floor 2 is more of a balcony that surrounds an atrium. This could cause differences between design engineers analysis and my own.

Analysis Method

For this report I used ETabs to find the relative stiffness of the braced frames. Doing this also gave me a way to analyze the building taking into consideration torsional effects from the center of mass/load and the center of stiffness not corresponding with the same point. This caused eccentricities to develop about the center of stiffness and from this I was able to divide the loads up in a more logical manner to distribute them to the braced frames.

Strength/Drift Analysis

The Weinberg Centers' lateral force resisting system is controlled by service load deflections and not strength design. From previous spot checks I know that strength of the lateral bracing is more than sufficient. However a building cannot be designed using only strength criteria. Building occupants would not be comfortable working in a building that sways too much in the wind. In the case of The Weinberg Center building drift controls the design of the lateral bracing.

Table of Contents

Existing Conditions (See Appendix A-1 for general floor plan of The Weinberg Center)

The Weinberg Center is a 6 story medical office building located in downtown Baltimore, MD. It was constructed in 2002 using the 1997 Baltimore City Building Code and 1996 BOCA. This code assigns a 100psf live load to the floors. The design engineer used a 10 psf superimposed dead load for mechanical, electrical, plumbing, and finishes loads. Concrete is designed using The American Concrete Institute (ACI 318). Steel is designed using the "Load and Resistance Factor Design Specification for Structural Steel Buildings, Third Edition"

Lateral Force Resisting System (See Appendix A-2 for Braced Frame Elevations)

The lateral force resisting system is composed of 3 braced frames that run the entire height of the building, 103'-0" tall. These braced frames are located around the building core. Four smaller braced frames are located at the top of the glass/aluminum corner, and a few moment frames are located at the penthouse level. For the purpose of this report I did not analyze the bracing at the top of the façade corner. The 3 main frames are chevron braced with the exception of 1 diagonal brace in Braced Frame 3. Two of the braced frames carry lateral load in the E-W direction while the remaining braced frame carries the load in the N-S direction. The load is distributed to the braced frames through the framing on each floor.

Building Code

1997 Baltimore City Building Code
Maryland Building Performance Standards as amended effective April 7, 1997
1996 BOCA
Reference ASCE 7-02 for Wind and Seismic Load Calculations

Steel Code

"Load and Resistance Factor Design Specification for Structural Steel Buildings, Third Edition"
The American Institute of Steel Construction

Dead Loads

These loads are used in the Seismic Design of Building

Ceiling

- Metal Deck: 2psf
- Mechanical: 4psf
- Ceiling: 3psf
- Roofing: 5psf
- Framing: 7psf

Floor

- Concrete Slab: 45psf
- Mechanical: 4psf
- Flooring/ceiling: 2psf
- Framing: 7psf

Exterior brick curtain walls were assumed to weigh 15psf
Floor partitions were assumed to weigh 10psf

Lateral Loads

Wind: Loads are Designed using V=90mph in Exposure B

Height above ground	Windward		Leeward		Total	
	N-S	E-W	N-S	E-W	N-S	E-W
0-15	9.81	9.81	-7.63	-10.02	17.44	19.84
20	10.41	10.41	-7.63	-10.02	18.04	20.44
30	10.89	10.89	-7.63	-10.02	18.52	20.92
40	11.37	11.37	-7.63	-10.02	19.00	21.40
50	12.09	12.09	-7.63	-10.02	19.72	22.11
60	12.69	12.69	-7.63	-10.02	20.32	22.71
70	13.17	13.17	-7.63	-10.02	20.80	23.19
80	13.65	13.65	-7.63	-10.02	21.28	23.67
90	14.13	14.13	-7.63	-10.02	21.76	24.15
100	14.49	14.49	-7.63	-10.02	22.12	24.51

(See Appendix A-3 for Building Elevation and Summary of Windward Pressures)

Base Shear:

North-South Direction 157 kips

East-West Direction 335 kips

Overturning Moments

North-South Direction 6607 ft-kips

East-West Direction 14467 ft-kips

Seismic: Loads are based on a Seismic Use Group I, Site Class D (assumed since unknown), and Seismic Design Category B. Equivalent Lateral Force Analysis was used to determine the loads.

Height (ft)	Level	h^kWx	Cvx	Fx(k)
0	B	0	0	0
20	1	33342	0.028	8.5
32	2	76267	0.064	19.3
46	3	151802	0.126	38.1
60	4	210481	0.176	53.2
74	5	264854	0.221	66.8
88	6	325549	0.272	82.2
103	R	135783	0.113	34.2

(See Appendix A-4 for Building Elevation and Summary of Seismic Loads)

Base Shear:

V= 336.5 kips

Overturning Moment:

M= 21431.6 ft-kips

Strength Check (See Appendix A-5 for Strength check)

In Technical Assignment 1 I checked the design of the top bracing of the N-S lateral load resisting system. I ended up using a HSS 5x5x3/16". The design engineer used a HSS 8x8x1/4". Seismic loads controlled the strength design of my lateral element.

For Technical Assignment 3 I ended up reducing the loads on my building based on my faculty consultants' advice. This would have no adverse affect on the strength design of my lateral element; I will still end up with members being the same size or smaller if I were designing for strength alone. Seismic loads governed the strength design of Braced Frame 1 while Wind loads governed the strength design of Braced Frame 2 and 3.

Drift Analysis (See Appendix A-6 for Summary of Drift Analysis on each Braced Frame)

To analyze drift of the building I modeled each braced frame in ETabs and applied a arbitrary load to each floor in turn and noted the drift. (See Appendix A-7 for ETabs output) I was then able to assign each braced frame a relative stiffness with respect to each other. This in turn allowed me to analyze the building for torsional effects and distribute the loads to each braced frame more accurately. (See Appendix A-8 for torsional effect calculations)

Building drift in the North-South direction (Braced Frame 1) is calculated at 1.44" for wind service load. This is well with in the tolerable H/400 used as an industry standard building drift. In fact a more conservative drift limit of H/800 may have been used for several reasons. First, The Weinberg Center has immediate adjacencies to a parking garage and there is an elevated pedestrian walkway that connects into the building. These two elements may have been of concern to the design engineer and a small building drift could have been adopted. Average Story drift is .206" which is again with in tolerable allowances.

Building drift in the East-West direction (Braced frames 2 and 3) came out to be less than the tolerable 3" based on a H/400 drift limit. Braced frame 2 drifted 2.93" while braced frame 3 drifted 2.99". Average story drift came out to be .419" for BF-2 and .427" for BF-3, both of which are with in tolerable allowances.

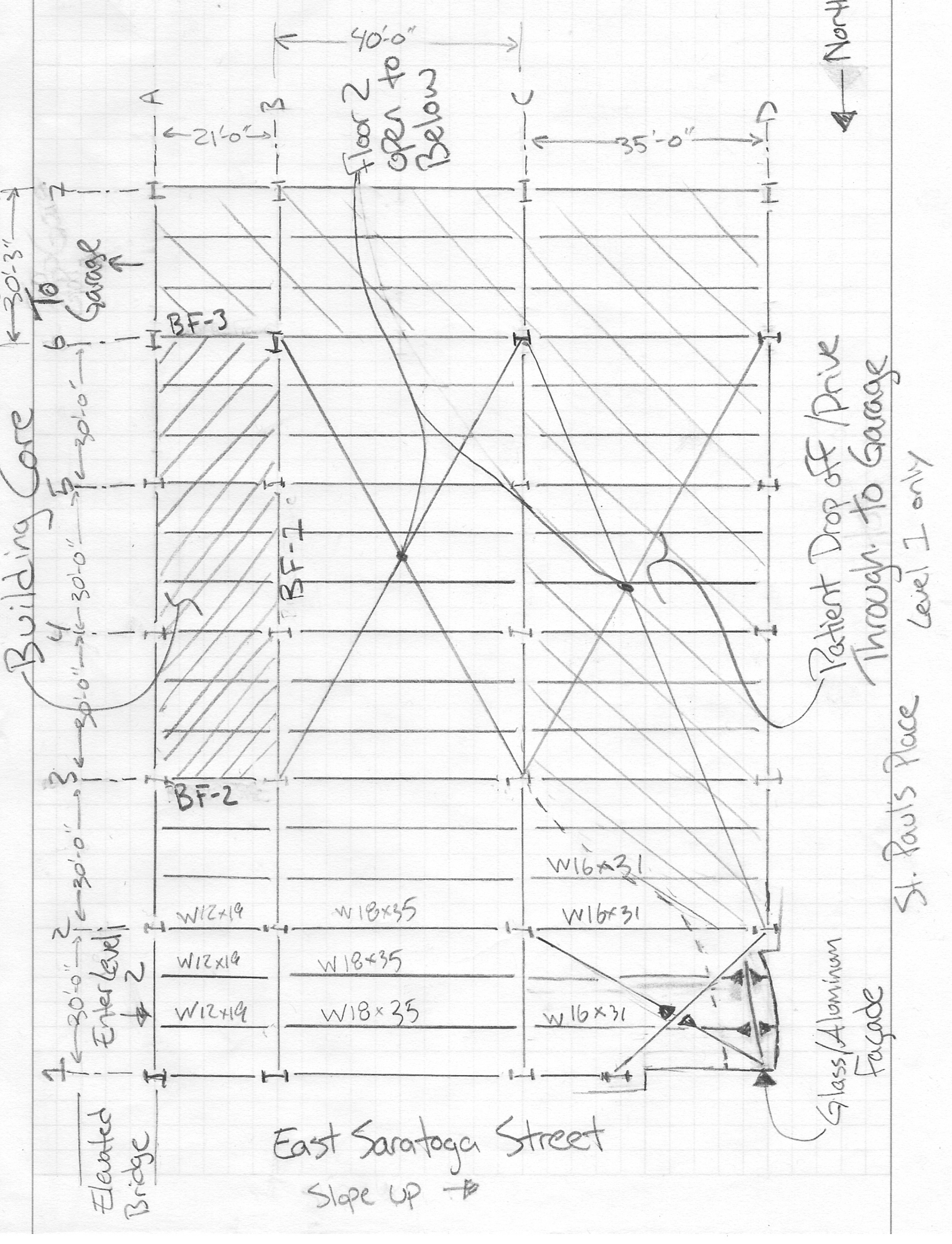
Areas of Concern

In order to perform my analysis I had to make a few general assumptions about the building. For distributing the load to the braced frames I assumed that load able to find its way through the flooring system to the braced frames. This assumption works well for all of the stories except for floor 2. Flooring is basically infill and connected to the braced frames in these floors. On floor 2 there is a 2 story atrium that takes up a good portion of the building footprint. This could cause significant problems in transferring the load from the floor and wall to the braces frame. It could also cause eccentricities to be induced because of the geometry of the floor. Second I assume that this building was designed to be adjacent to the parking garage, and that it was designed to stand alone under the loadings. This could cause errors in my calculations since I combined the loadings into one and analyzed the building for some worst case scenarios that may not happen.

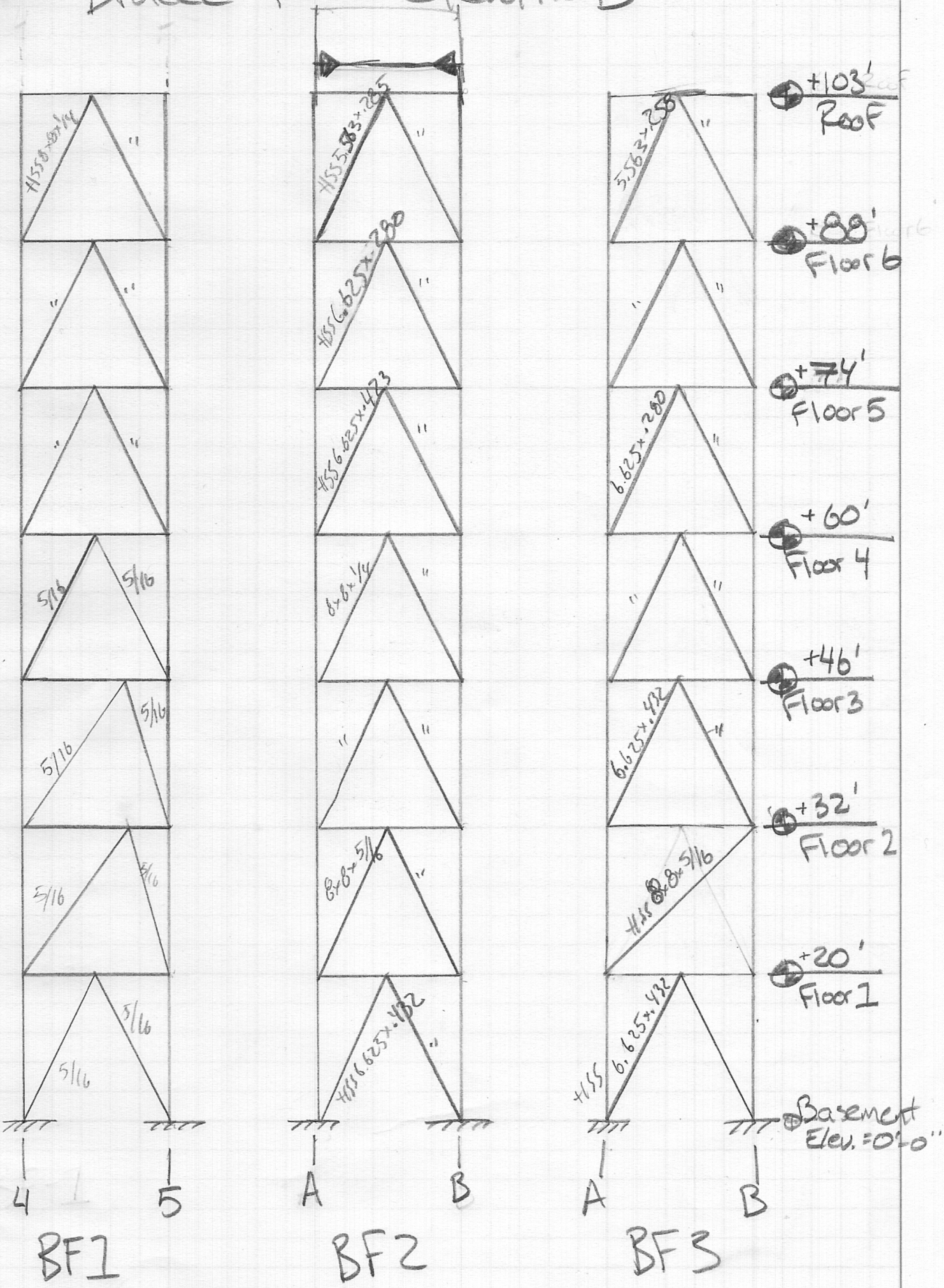
Conclusion

It is my opinion that limiting drift is the controlling factor in the design of the lateral force resisting system for The Weinberg Center. From my previous strength check I found that much smaller members could have been used to hold the building upright strength-wise, but service load deflections of wind on the building would not have been tolerable for comfort of the occupants. Members had to be sized larger than determined from strength calculations in order for the building drift to be reduced to a tolerable standard.

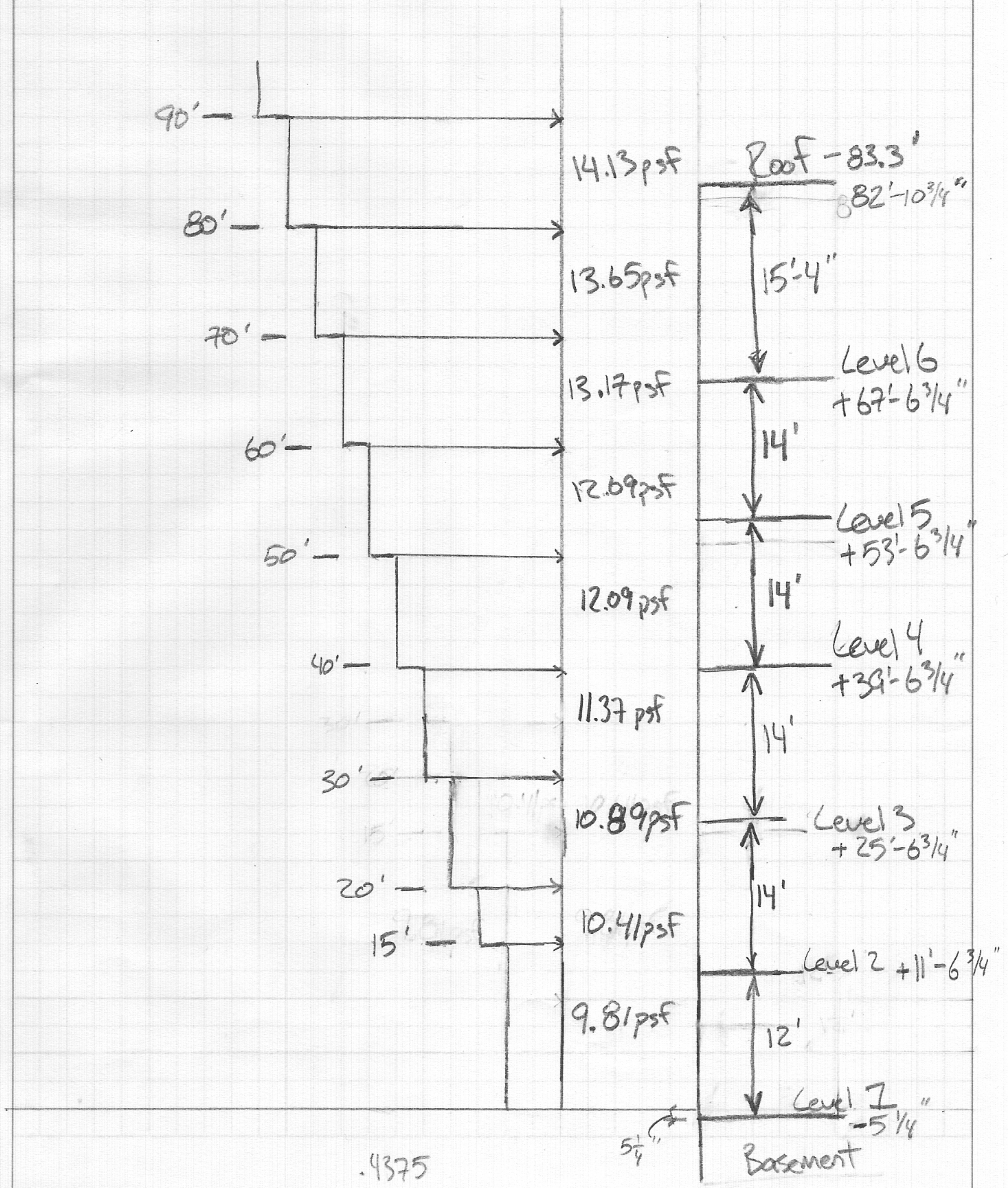
A-1 Floor Plan



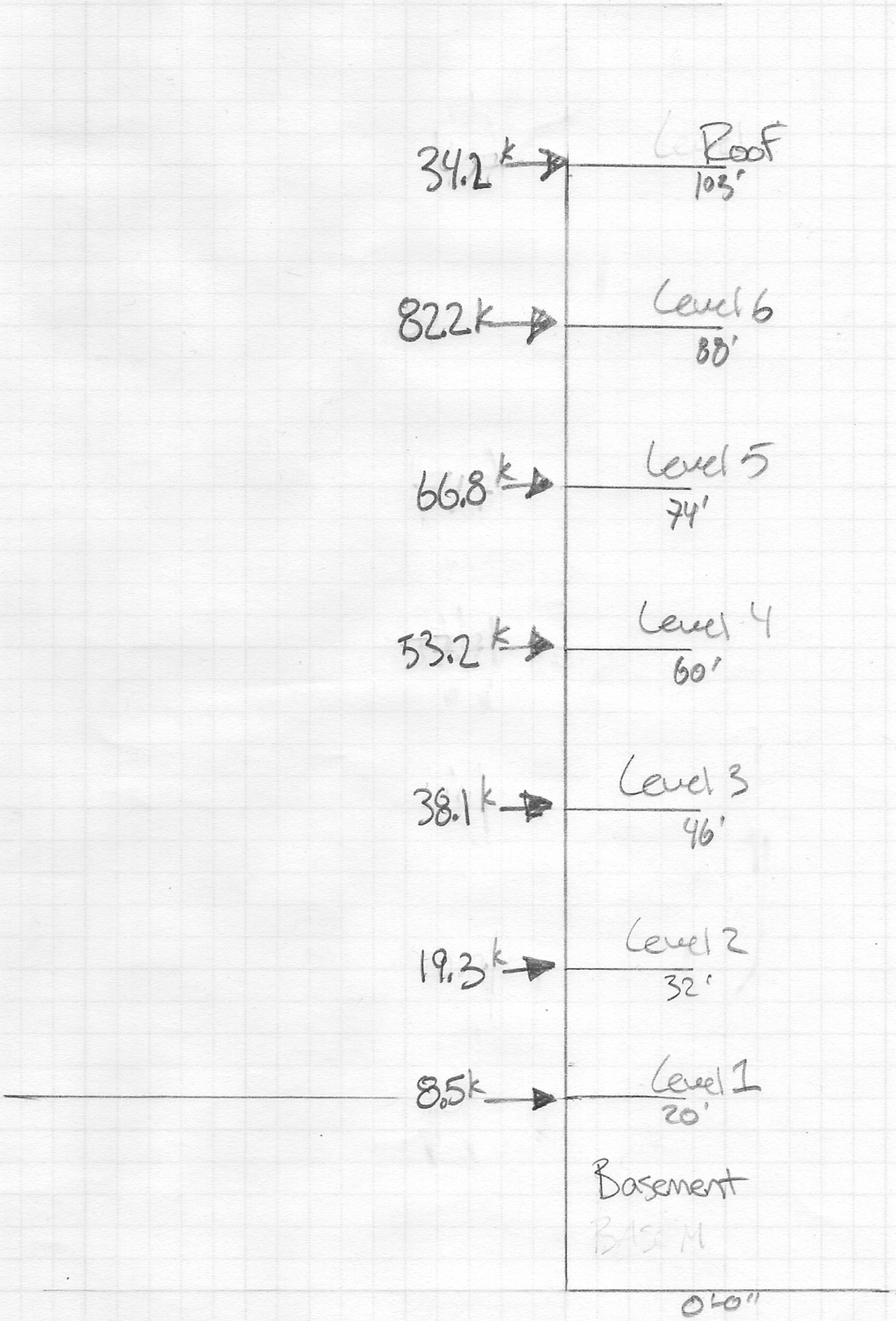
A-2 Braced Frame Elevations



A-3 WIND LOAD DIAGRAM

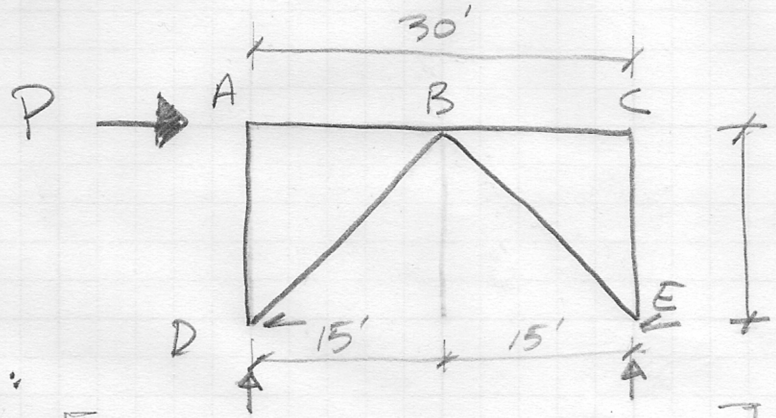


A-4 Seismic Load Diagram



Strength Check : Lateral Bracing

Look at Top Bay only in N-S Direction



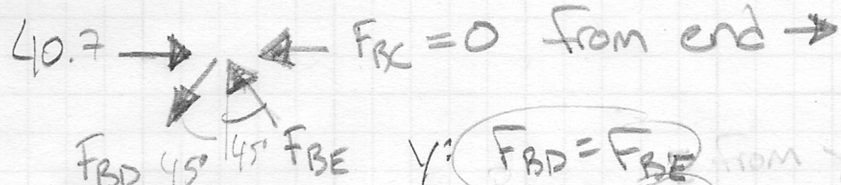
WIND:

$$P = [14.13(100)(3.3) + 13.65(100)(4.2)]\ 1.6$$
$$= 16.6^k$$

SEISMIC: $40.7^k\ (1.0)$

seismic controlls design of Top Bracing

$40.7 \rightarrow \quad \leftarrow F_{AB} = 40.7$



Y: $F_{BD} = F_{BE}$

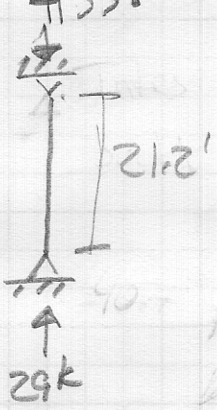
X: $40.7 = (F_{BD} + F_{BE}) \sin 45$

$40.7 = 2F_{BD} \sin 45$

$F_{BD} = F_{BE} = 29^k \rightarrow \pm 29^k$ T&C

Find Member that is 21.2' Long
& can take +/- 29k

HSS: $kL = 1(21.2) = 21.2$



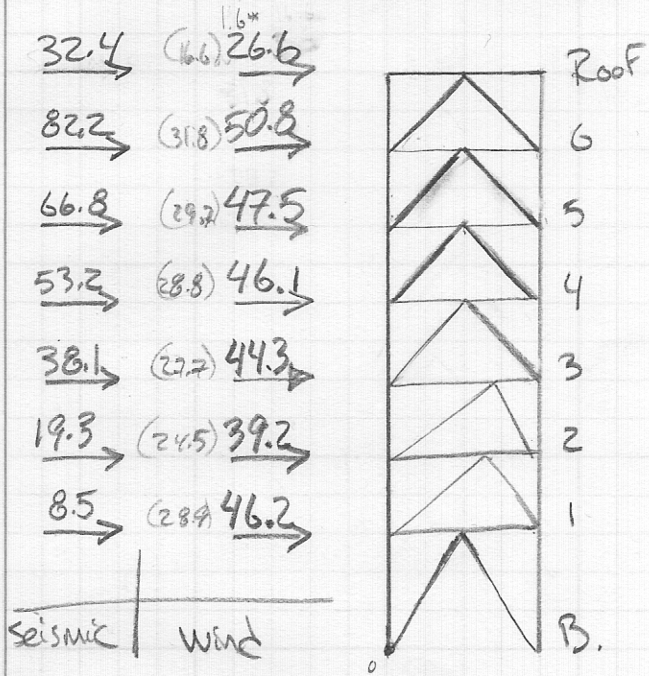
HSS 6x4x1/8
4x4x3/8
5x5x3/16 ← use

HSS 5x5x3/16" will work in compression.

Design Engineer Used 8x8x1/4"

maybe my loads are too small or could have been done for economy of scale due to Larger loads at lower levels plus simplified connection design & construction. These are knife-Edge connected into Frame which is a difficult connection to put together.

Braced Frame 1



seismic load factor: 1.0

wind load factor: 1.6

	overturning moments	Base shear
seismic	= 21101.5'k	300.5 k
wind	= 17707.4'k	300.7 k

Drift → using ETABS output:
using service loads:

seismic → 2.88" At roof.
.411" Avg. story drift.

wind → 1.44" At roof
.206" Avg. Story drift.

Typically only check wind drift, seismic just has to survive.

This building has immediate adjacencies, parking garage is right next to building plus skywalk is parallel to this direction deflection.

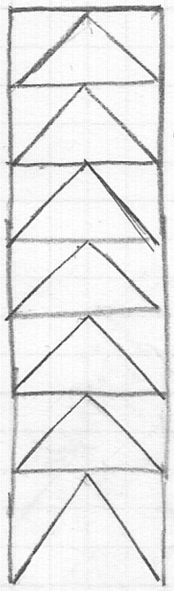
Designer could have used much higher deflection criteria than L/400 → which gives 3" for this building.
L/800 was probably used to be safe.

A-6.2

Braced Frame 2

seismic load factor = 1.0
wind load factor = 1.6

seismic	wind	
21.3 →	35.5 (22.2) →	Roof
53.9 →	68.0 (42.5) →	6
43.8 →	63.8 (39.9) →	5
34.9 →	62.0 (38.8) →	4
25.0 →	60.0 (37.5) →	3
12.7 →	53.3 (33.3) →	2
5.6 →	63.2 (39.5) →	1
		B



	over-turning moments	Base shear
seismic	13940.7'k	197.2 k
wind	23811.3'k	405.8 k

Drift ⇒ using ETABS output.

Seismic designed for strength only

wind ⇒ service loads only

= 2.93"

Avg story drift = .419"

This is in the ball park, L/400 gives 3" drift overall.

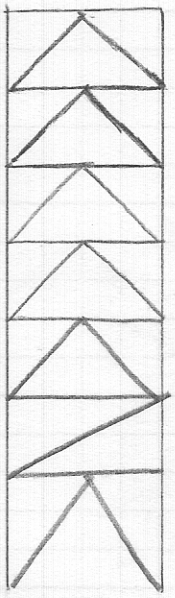
Designer probably use L/400 as drift limit for building.

A-6.3

Braced Frame 3

seismic load factor = 1.0
wind load factor = 1.6

seismic	wind	Level
17.3 →	29.0 (18.1) →	Roof
44.0 →	55.5 (34.7) →	6
35.7 →	52.0 (32.5) →	5
28.5 →	50.6 (31.6) →	4
20.4 →	49.0 (30.6) →	3
10.3 →	43.5 (27.2) →	2
4.5 →	51.5 (32.2) →	1
		B



	overturning Moment	Base shear
seismic	11363.7'k	160.7 k
wind	19425'k	330.9^k

Drift: wind only (service load)

wind ⇒ 2.99" at roof

Avg Story drift = .427"

This is almost dead on the criteria of L/400 = 3"

It is also not very far off the othe Braced Frame #2 at 2.93 or a .06" diferential deflection.

Designer Probably used L/400 for deflection criteria.

A-7

USING ETABS & modeling only the braced frames:

BF-1

10^k Load Applied at:

	point 13 Displacement at roof (in)
Roof	.164
6	.127
5	.098
4	.071
3	.050
2	.032
1	.018

BF-2

10^k Load Applied at:

	point 40 Displacement at roof (in)
Roof	.253
6	.202
5	.150
4	.106
3	.070
2	.040
1	.026

BF-3

10^k Load Applied at:

	Δ at roof (in) point 72
Roof	.329
6	.248
5	.187
4	.131
3	.086
2	.058
1	.029

A-8.1 Center of Stiffness

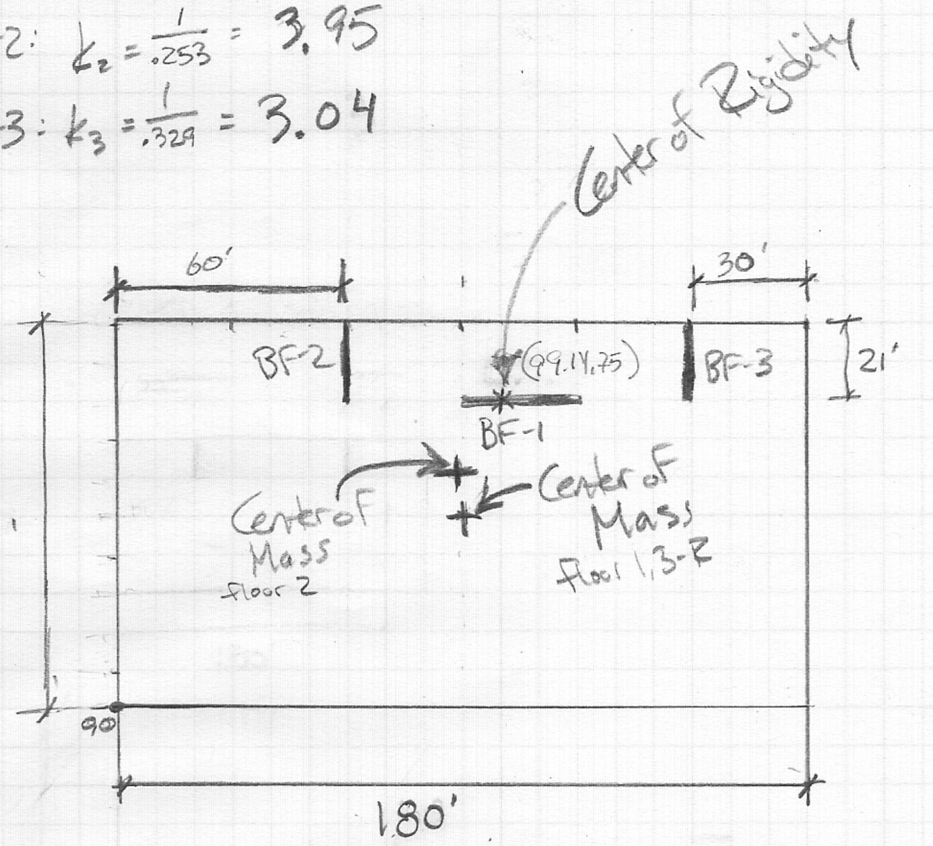
Relative Stiff. using the ETABS output

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

BF-1: $k_1 = \frac{1}{.164} = 6.10$

BF-2: $k_2 = \frac{1}{.253} = 3.95$

BF-3: $k_3 = \frac{1}{.329} = 3.04$



$$\bar{x}(3.95+3.04) = 3.95(60) + 3.04(150) \Rightarrow \bar{x} = 99.14'$$

$$\bar{y} = 75'$$

A-8.2

Center of Mass

floors 1, 3, 4, 5, 6, Roof have full Area

floor 2 has partial cutout of Floor Area

Walls form rectangle

Center of Mass for Floor 1, 3, 4, 5, 6, Roof are at center of geometry: $\bar{x} = 90'$, $\bar{y} = 48$

Center of Mass of floor 2 at:

$$\bar{x}: \frac{165(30)(96) + 105(30)(90) + 30(61)(60) + 14.67(44)(35).5}{30(96) + 30(90) + 61(60) + .5(44)(35)}$$

$\bar{x} = 87.89'$

$$\bar{y}: \frac{48(30)(96) + 81(30)(90) + 65.5(61)(60) + 23.33(44)(35)(.5)}{30(96) + 30(90) + 61(60) + .5(44)(35)}$$

$\bar{y} = 61.4'$

center of Mass for Building:

$$\bar{x} = \quad \bar{x} = \frac{6(90) + 1(87.89)}{7} = 89.7'$$

$$\bar{y}: \quad \bar{y} = \frac{6(48) + 1(61.4)}{7} = 49.9'$$

Eccentricities between center of load & Center of Stiffness

$$e_{x\,wind} = \frac{180}{2} - 99.14 = 9.14'$$

$$e_{y\,wind} = \frac{96}{2} - 75 = 27'$$

$$e_{x\,seismic} = 89.7 - 99.14 = 9.44'$$

$$e_{y\,seismic} = 25.1'$$

Torsional Moment $M = eP$

Wind: $M_x = 27\,P_x$ seismic $M_x = 25.1'\,P_x$

$M_y = 9.14\,P_y$ $M_y = 9.44'\,P_y$

Polar Moment of Inertia $J = \Sigma k_i d_i^2$

$$J = 3.95(39.14)^2 + 3.04(50.86)^2 + 6.10(0')$$

$$J = 13914.8$$

Det. Direct Shear:

$$F_{y_{1/2}} = \frac{3.95}{3.95+3.04} F_y = .565 F_y$$

$$F_{y_3} = (1-.565) F_y = .435 F_y$$

$F_{x_1} = F_x$ only one Frame In X-direction

Torsional Components to each Frame

$$F_T = \frac{k_d}{J} M$$

F_{T_1} ⇒ Is Always Zero : Center of Stiffness on BF-1 thus no ecc for Moment to develope

Wind:

x-dir: $F_{T_2} \Rightarrow \frac{3.95(39.14)(27P_x)}{13914.8} = .30P_x$

$F_{T_3} \Rightarrow \frac{3.04(50.86)(27P_x)}{13914.8} = .30P_x$

y-dir: $F_{T_2} \Rightarrow \frac{3.95(39.14)(9.14P_y)}{13914.8} = .10P_y$

$F_{T_3} \Rightarrow \frac{3.04(50.86)(9.14P_y)}{13914.8} = .10P_y$

seismic

x-dir: $F_{T_2} \Rightarrow \frac{3.95(39.14)(25.1P_x)}{13914.8} = .29P_x$

$F_{T_3} \Rightarrow \frac{3.04(50.86)(25.1P_x)}{13914.8} = .28P_x$

y-dir: $F_{T_2} \Rightarrow \frac{3.95(39.14)(9.44P_y)}{13914.8} = .10P_y$

$F_{T_3} \Rightarrow \frac{3.04(50.86)(9.44P_y)}{13914.8} = .10P_y$

A-8.5

Forces in Braced Frames w/ Torsional Effects.

BF-1 : $F_{x_1} = F_x \Rightarrow$ CS on BF-1

BF-2 : wind : $F_{y_2} = .565 F_y + .10 F_y$
seismic $F_{y_2} = .565 F_y + .1 F_y$ $\Big\} = .656 F_y$

BF-3 : wind : $F_{y_3} = .435 F_y + .1 F_y$
seismic : $F_{y_3} = .435 F_y + .1 F_y$ $\Big\} = .535 F_y$

F_x & F_y per floor used in drift Analysis

Braced Frame LOADINGS:

Seismic - Both Directions:

32.4 →	Roof
82.2 →	6
66.8 →	5
53.2 →	4
38.1 →	3
19.3 →	2
8.5 →	1
	B

WIND: North-South (wind along long dir.) ⊥ to 100' dim

16.6 →	Roof	103'
31.75k →	6	88'
29.7 →	5	74'
28.8k →	4	60'
27.7k →	3	46'
24.5k →	2	32'
28.9k →	1	20'
	B	0'-0"

load to Roof:

$$\left(\frac{15'}{2}\right)(22.12\,psf)(100') = 16.6^k$$

load to floor 6:

$$5.5'(22.12\,psf)(100') + 9'(21.76)(100')$$
$$= 31.75^k$$

load to Floor 5:

$$1'(21.76\,psf)(100') + 10'(21.28\,psf)(100') + 3'(20.8)(100) = 29.7^k$$

load to floor 4

$$7'(20.8)(100') + 7'(20.32)(100') = 28.8^k$$

load to floor 3

$$3'(20.32)(100') + 10'(19.72)(100') + 1'(19\,psf)(100') = 27.7^k$$

load to Floor 2

$$9'(19)(100) + 4(18.52)(100) = 24.5^k$$

load to Floor 1

$$\left[6(18.52) + 5(18.04) + 5(17.44)\right]100 = 28.9^k$$

A-8.7

WIND: EAST-WEST (wind perpendicular to Long face)

load to Roof: $7.5(24.51)(184) = 33.8^k$

load to Floor 6: $5.5(24.51)(184) + 9(24.15)(184) = 64.8^k$

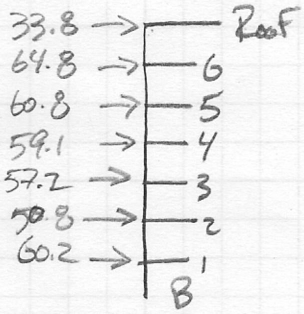
load to Floor 5: $1'(24.15)(184') + 10'(23.67)(184') + 3'(23.19)(184') = 60.8^k$

load to floor 4: $\left[7'(23.19) + 7'(22.71)\right] \times \frac{184}{1000} = 59.1^k$

load to Floor 3: $\left[3'(22.71) + 10(22.11) + 1'(21.40)\right] \frac{184}{1000} = 57.2^k$

load to floor 2: $\left[9'(21.40) + 4(20.92)\right] \frac{184}{1000} = 50.8^k$

load to Floor 1: $\left[6(20.92) + 5(20.44) + 5(19.84)\right] \frac{184}{1000} = 60.2^k$



WIND LOADS ARE BROKEN UP BACED ON RELATIVE STIFFNESS & TORSIONAL EFFECTS & THEN DISTRIBUTED TO EACH BRACED FRAME