The Harry and Jeanette Weinberg Center

Mercy Hospital Medical Office Building

Baltimore, MD

Technical Assignment 1

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Structural Option

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Executive Summary

The intent of this report is to analyze existing conditions and design procedures used in the structural design of The Harry and Jeanette Weinberg Center in Baltimore, MD.

The Harry and Jeanette Weinberg Center is a medical office building located in Baltimore, Maryland. It consists of 6 floors above grade, and 1 partially below. The building occupancy classification is Building Use Group B, Business. Typical uses are administrative, doctor and clinical office suits, and conference rooms with occupant load less than 50 persons. The type of construction used is type 2A using noncombustible and protected materials and having a sprinklered wet-pipe fire-suppression system. On the first floor (seen at street level in the picture above) is a drive through patient drop off that has a connection on the far right leading to the parking garage located to the rear of the building. An elevated pedestrian bridge spans across East Saratoga Street and connects The Weinberg Center to Mercy Medical Center.

The structural design code use in the design of The Weinberg Center is BOCA 1996. Several of the members that I spot checked ended up being smaller than those used in the building. This could be because I made very general assumptions during the design of these members. For instance, my slab design assumed that the slab would be unshared which forced me to use a thicker decking.

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Structural System

The structural system of The Weinberg Center spans 6 floors above grade and 1 below. Floor to floor heights are 20'-0" from the basement to level 1, 14'-0" for level 1 through level 5 and 15'-0" for level 6. The building is built using steel framing with composite action slab on deck. The columns are set up on a maximum bay size of 30'-0" (N-S) by 40'-0" (E-W). Lateral forces are resisted by braced frames located at the buildings core. All structural steel is A572 Grade 50.

Foundation

The foundation is composed of straight shaft drilled caissons, spread footing, slab-on-grade and a concrete retaining wall along the west elevation. Caissons bear on rock at depths exceeding 36'-0" in order to reach bearing capacities of 90ksf. Spread footings are all 12" thick. Assumed bearing pressures for spread footings and slab-on-grade is 2.0ksf. Slab-on-grade is divided into quadrants between column areas and is typically 6" thick with a maximum thickness of 10" in the North-West corner. The concrete retaining wall is 15" thick, 22'-0" high and carries minimal loads from the floor above.

Caisson diameters: 3'-6" 4'-0" 4'-6" 7'-0"
Spread footings dimentions: 4'-0"x4'-0" 4'-6"x4'-6" 5'-6"x5'-6"
Concrete Strength f 'c
Drilled Caissons: 3500psi
Spread Footings: 3000psi
Walls & Piers: 4000psi
Slab-on-grade: 3000psi
Deformed Bar Reinforcing Strength: fy=60ksi

Columns

The columns of The Weinberg Center are all W14 shapes. They range in size from a W14x24 at the penthouse level down to a W14x283 in the basement. Columns are typically spliced at the floor 1, floor 3 and floor 5. The longest columns are 29'-1" tall and are located on the top floors. All columns are ASTM A572 GR50.

Floor System

The floor system is a made up of simply supported girders (typical sizes are W21x50 or W21x44) that span 30'-0" column to column in the N-S direction and simply supported infill beams (typical sizes are W16x26 and W18x35) span 40'-0" at 10'-0" on center in the E-W direction. Infill beams that span more that 30'-0" are cambered upward in the middle by 1-7/8". Girders that span 30'-0" are cambered up in the middle by 1" to 1-1/8". A 1-way slab-on-deck utilizing composite action is used to carry floor loads to the beams. The slab is 3.25" lightweight concrete (strength f'c=3000 psi on a 2"-20 gage deck with 6x6-W1.4xW1.4 welded wire fabric. The maximum span for the slab on deck is 10'-0", the typical beam spacing. The main lobby on floor 1 is 2 stories high so floor 2 only runs around the North, West and South walls. The glass/aluminum corner is framed out by running a diagonal beam to truncate the corner, and then cantilevering beams off the diagonal to the façade. The cantilevered beams are moment connected into the diagonal girder, opposite the

Structural System Continued

cantilevered beams is another moment connected beam tying back into the structural system to balance any torsion effects (See appendix for typical bay and glass/aluminum façade corner framing). All structural steel is fy=50ksi while all plates and angles are fy=36ksi steel. The roof is framed out in the same way the floors are except that none of the roof shapes are cambered. The roof girders range from a W21x44 to a W24x62 while the beams range from W16X26 to W18x40. The high roof framing for the glass/aluminum corner is more simplified than the floor framing and composes of W14 and smaller shapes.

Lateral Force Resisting System

The lateral force resisting system composes of 3 braced frames that run the entire height of the building 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. The 3 main frames are chevron braced with the exception of 1 diagonal brace. 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 and Code Requirements

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

Concrete:

The American Concrete Institute (ACI)

Steel:

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

Loads

Live Loads

- Foundation and Basement Level 150psf
- Levels 1-6 100psf
- Roof/High Roof 30psf

Dead Loads

- Concrete Slab
  - Foundation and Basement 61psf
  - Levels 1-6 41psf
- Metal Deck 2psf

Loads Continued

- Mech, ducts, etc. 4psf
- Ceiling 3psf
- Floor Covering 1psf
- Roofing 5psf
- Insulation 4psf
- Framing 7psf ←Assumed

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 2 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	$h^k Wx$	Cvx	Fx(k)
0	B	0	0	0
20	1	46288	0.035	11.8
32	2	81309	0.062	20.9
46	3	162011	0.123	41.4
60	4	224636	0.172	57.9
74	5	291140	0.222	74.7
88	6	347975	0.265	89.2
103	R	157616	0.121	40.7

Loads Continued

Base Shear:

V= 336.5 kips

Overturning Moment:

M= 23852 ft-kips

Spot Checks

Spot checks can be found in appendix A-7.

<u>Floor Slab:</u> My floor slab design is thinner than what was used in the building. This could be explained because the designers may have been looking for a way to increase the moment arm in the composite design by making the slab thicker than what was required. They would have wanted to do this because the beam spans are rather large in some parts of the building (40'-0") and it would be more economical to decrease the beam size by utilizing a more efficient composite action design.

<u>Composite Beam:</u> The composite beam that I designed was not far off from what is used in The Weinberg Center. The design engineer used a W18x35 beam with 48 shear studs and had a camber of 1-7/8". The beam I designed was also a W18x35 but I used 54 shear studs and a camber of ½". My design may differ from the engineers if I were to farther look into using more of the concrete slab for composite action. A larger camber could have been used to mitigate service performance deflection.

<u>Composite Girder:</u> My composite girder design is the same weight but much shallower than what the design engineer used. I came up with a W18x50 with 68 studs while the design engineer used a W21x50 with 71 studs. A deeper beam would be used to lessen deflection of the system under service load.

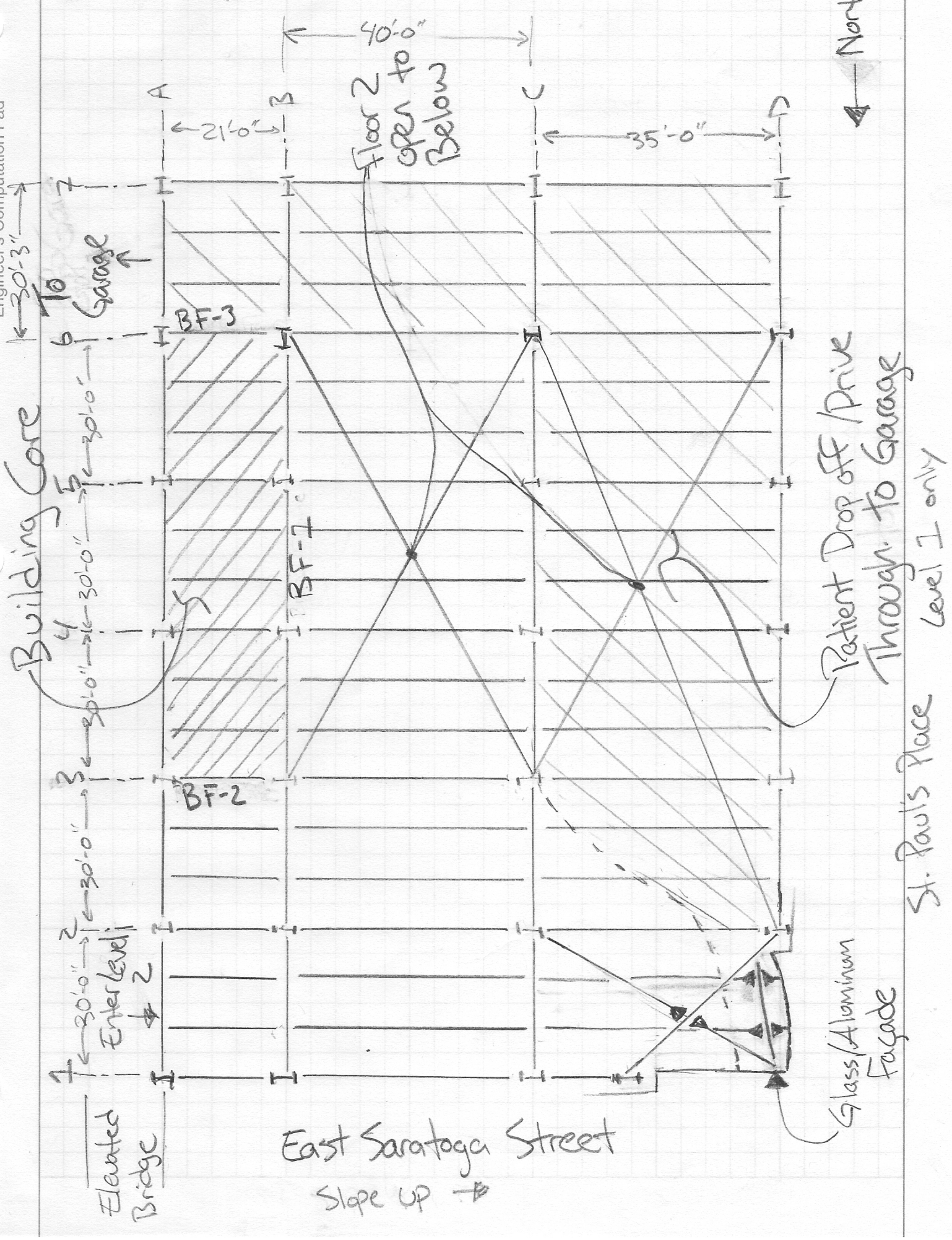
<u>Column:</u> In general my columns ended up being smaller than what was used by the design engineer. This is most probably because I only designed for gravity loads and did not look at what effect wind or seismic would have on column axial forces. Also my loads may have been too light for the design. I took into account live load reductions but the design engineer may not have reduced them as much as I did and would have added in the weight of rooftop equipment. I also neglected any residual moment that would be caused by wind or seismic and load patterning on the floors.

<u>Lateral Element:</u> 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 design of my lateral element and I made a lot of general assumptions that the design engineer may not have. Things I did not take into account are parapet loading on the building and any loading on the penthouse wall. This may account for some of the load that may be missing. A larger section on the upper floors could have been used in order to simplify design and construction of the building. Knife connections were used to tie the chevron bracing into the frame and using

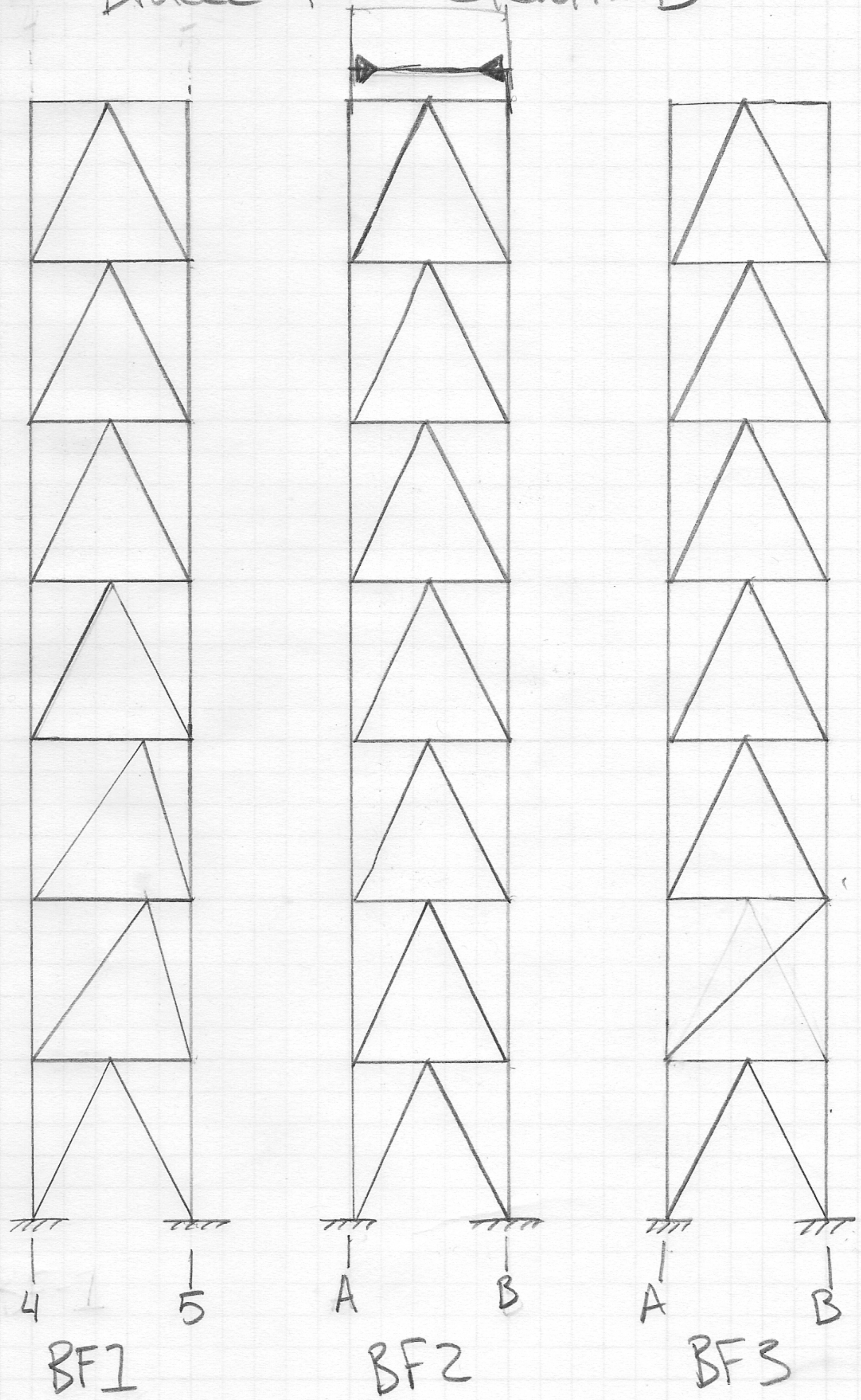
Spot Checks Continued

one brace size could be more economical than having to design and construct several different knife-edge connections.

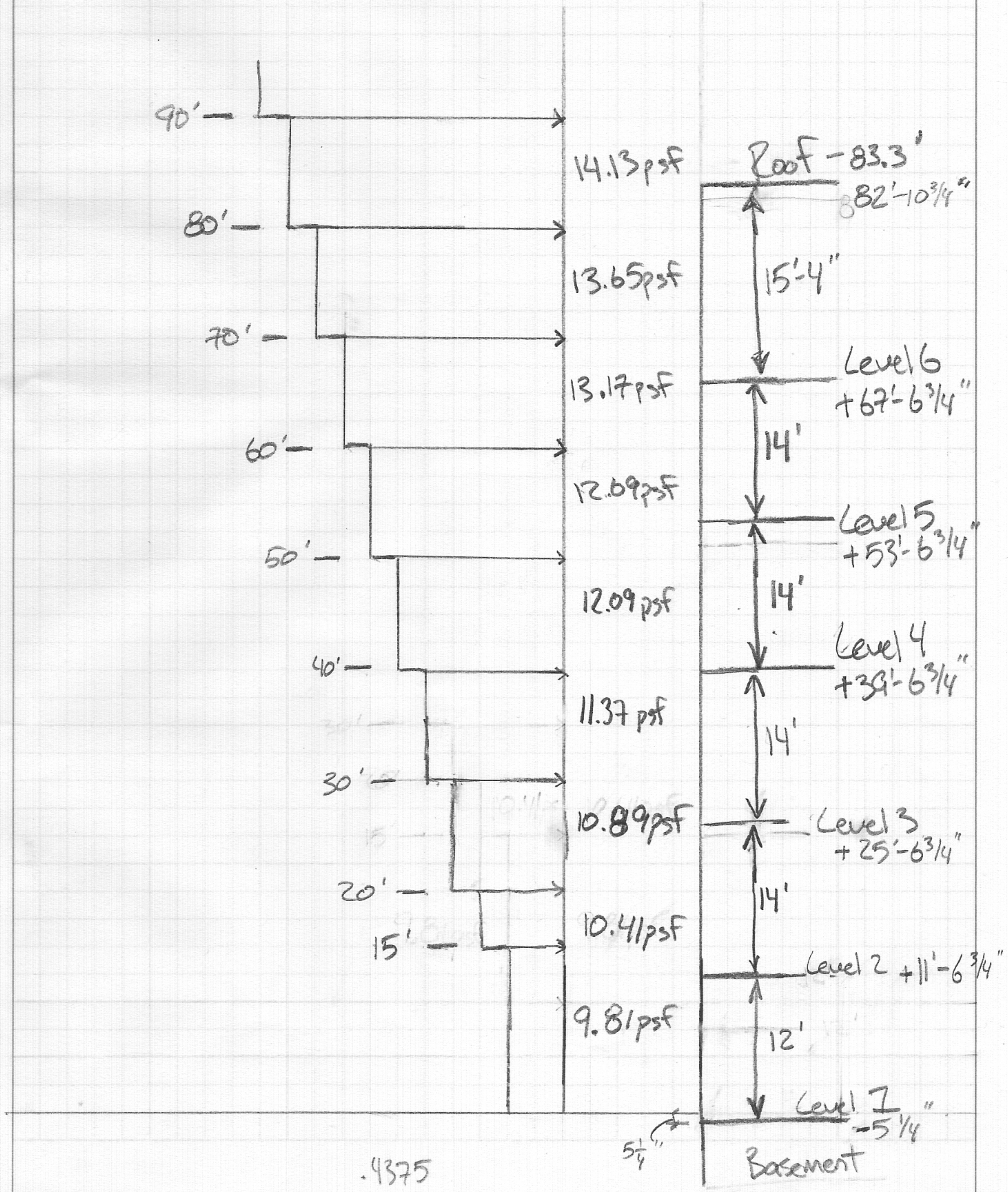
A-1 Floor Plan



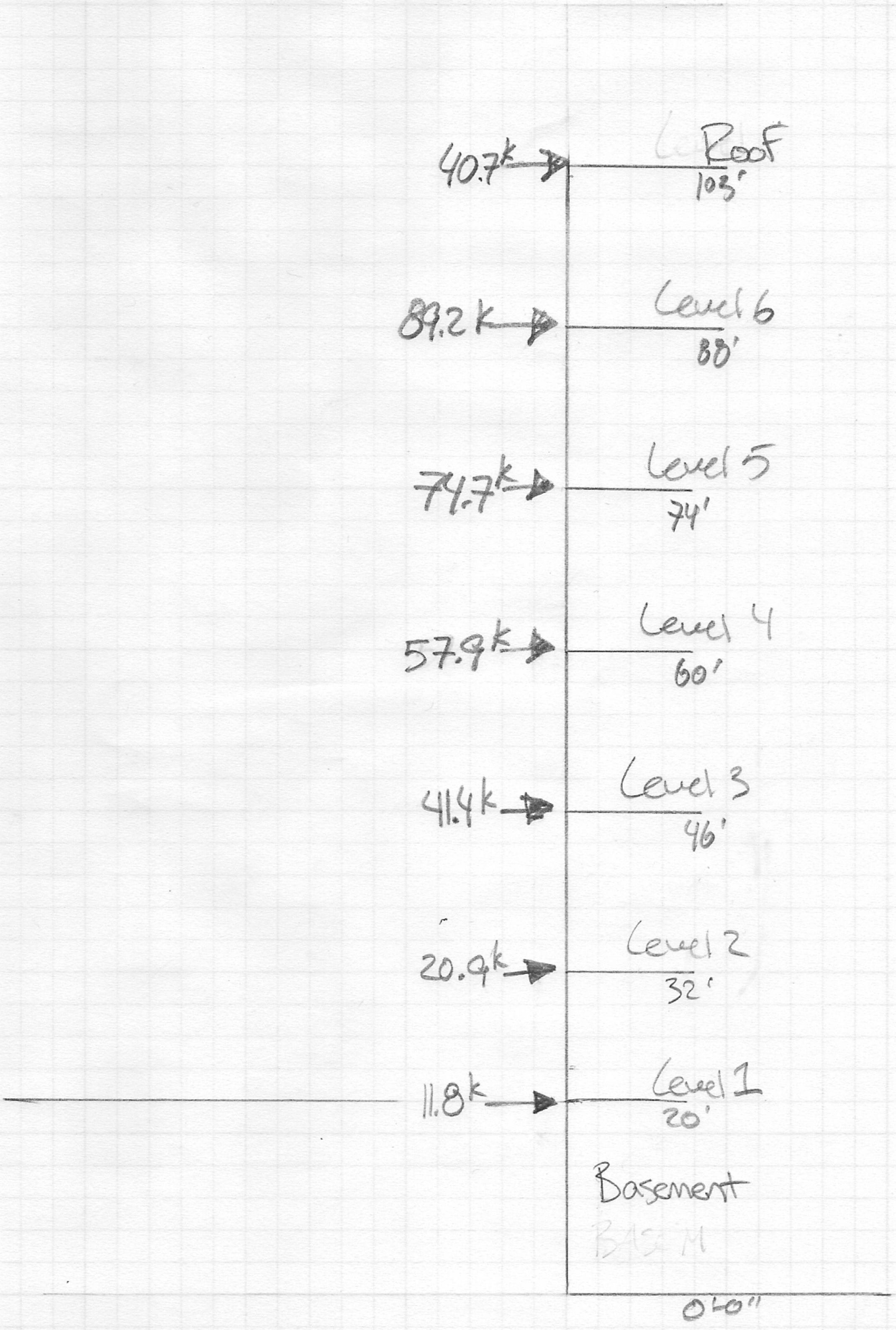
A-2 Braced Frame Elevations



A-3 WIND LOAD DIAGRAM



A-4 Seismic Load Diagram



Kevin Clouser 9-24-06

WIND LOAD CALCS. Method 2

1 Basic Wind Speed V = 90 MPH

Wind Directionality Factor $k_d = 0.85$

2 Importance Factor I = 1.0

nonhurricane, building catagorie II

3 Exposure Catagory B

Exposure B, case 2

Height (ft)	k_z
0-15	0.57
20	0.62
25	0.66
30	0.70
40	0.76
50	0.81
60	0.85
70	0.89
80	0.93
90	0.96
100	0.99

4 k_{zt} - unknown for now

parking garage immediatly adjacent to windward face

$k_{zt} = 1.0$ for now

5 G Assume Rigid Building

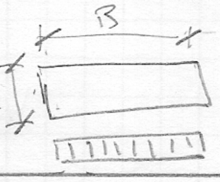
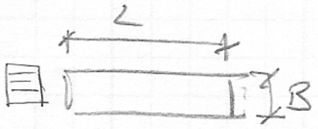
G = 0.85

6 Enclosure Classification

Enclosed Building

A-5.2

7. Internal Pressure Coefs $GC_{pi} = \pm 0.18$
8 External Pressure Coefs.

Wind Direction:		
$\frac{L}{B}$:	$\frac{100}{184.25} = 0.54$	$\frac{184.25}{100} = 1.84$
$\frac{h}{L}$:	$\frac{85.58}{100} = 0.83$	$\frac{8}{184.25} = 0.45$
$\theta = 0°$		

	C_p
Windward Wall	0.80 – use with q_z
Leeward Wall	–0.5 – use with q_h
Side Wall	–0.7 – use with q_h

	C_p
Windward wall	0.80 – use with q_z
Leeward Wall	1 –0.5 1.84 –0.33 ← use with q_h 2 –0.3
Side Wall	–0.7 ← use with q_h

Roof $\theta = 0$, $h/L = 0.83$

dist from windward edge	C_p – use with q_h
0 to 41.7'	–1.3**, –0.18
> 41.7'	–0.7, –0.18

Roof $\theta = 0$, $\frac{h}{L} = 0.45$

dist from Windward edge	C_p – use with q_h
0 to 83.3	–0.9, –0.18
83.3 to 166.6	–0.5, –0.18
> 166.6	–0.3, –0.18

** Area applied = 42.8(184.25)
$A = 7885.9 \text{ ft}^2$

$-1.3(.8) = -1.04$

A-5.3

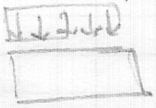
9. Velocity Pressure q_z & q_h

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

See spread sheet for q_z calcs.

10 Design Wind Load

$$p = qG_{?}C_p - q_i(GC_{pi})$$



Direction 1

Direction 2

See Spreadsheet for Calcs.

A-5.4

1	V=	90.00
	Kd=	0.85

2	I=	1.00

3	Exposure B
	case 2

kz:

height	kz		
0-15	0.57		
20	0.62		
25	0.66		
30	0.70		
40	0.76		
50	0.81		
60	0.85		
70	0.89		
80	0.93	kh=	0.94
90	0.96		
100	0.99		

4	Kzt=	1.00

5	G=	0.85

6	Enclosed Building

7	Gcpi=	0.18
		-0.18

8	External Pressure Coefs Cp

Wind Direction

top of parapet=85.6

Direction 1		
L=	100.00	
B=	184.25	
h=	83.30	
L/B=	0.54	
h/L=	0.83	
Wall	Cp	
Windward	0.80	use with qz
Leeward	-0.50	use with qh
Side	-0.70	use with qh
Roof	use with qh	
0-41.7	-1.04	-0.18
>41.7	-0.70	-0.18

Direction 2		
L=	184.25	
B=	100.00	
h=	83.30	
L/B=	1.84	
h/L=	0.45	
Wall	Cp	
Windward	0.80	use with qz
Leeward	-0.33	use with qh
Side	-0.70	use with qh
Roof	use with qh	
0-83.3	-0.90	-0.18
83.3-100.0	-0.50	-0.18

A-5.5

>166.6	-0.30	-0.18

9

Velocity Pressure qz

height	qz		
0-15	10.05		
20	10.93		
25	11.63		
30	12.34		
40	13.40		
50	14.28		
60	14.98		
70	15.69		
80	16.39		
90	16.92	qh=	16.57
100	17.45		

10

Design Wind Loads

	Height	Direction 1		Direction 2	
Windward	0-15	3.85	9.81	3.85	9.81
	20	4.45	10.41	4.45	10.41
	30	4.93	10.89	4.93	10.89
	40	5.41	11.37	5.41	11.37
	50	6.13	12.09	6.13	12.09
	60	6.73	12.69	6.73	12.69
	70	7.21	13.17	7.21	13.17
	80	7.68	13.65	7.68	13.65
	90	8.16	14.13	8.16	14.13
	100	8.52	14.49	8.52	14.49

Leeward	Direction 1	Direction 2
	-10.02	-7.63
	-4.06	-1.67

Sidewall	Direction 1	Direction 2
	-12.84	-12.84
	-6.88	-6.88

Roof	Direction 1			Direction 2	
0-41.7	-17.63	-5.52	0-83.3	-15.66	-5.52
	-11.66	0.45		-9.69	0.45

A-5.6

>41.7	-12.84	-5.52	83.3-166.6	-10.02	-5.52
	-6.88	0.45		-4.06	0.45
			>166.6	-7.21	-5.52
				-1.24	0.45

Seismic Design Loads

Seismic Use Group : I (building Cat. II)

Occupancy Importance Factor $I = 1.0$ (SUG-I)

Site Class → for now unknown (Assume D)

$S_s = .22$
$S_1 = .07$

$S_{MS} = F_a S_s = 1.6(.22) = 0.325$

$S_{M1} = F_v S_1 = 2.4(.07) = 0.168$

$S_{DS} = 2/3\, S_{MS} = \frac{2}{3}(0.325) = 0.217$

$S_{D1} = 2/3\, S_{M1} = \frac{2}{3}(0.168) = 0.112$

Seismic Design Catagorie :

$S_{DS} = 0.217$, SUG-I

T9.4.2.1a → SDCat. = B

$S_{D1} = 0.112$, SUG-I

T9.4.2.1b → SDcat = B

T9.5.2.5.1 → Equivalent Lateral Force Analysis
9.5.5

$V = C_s W \qquad C_s = \frac{S_{DS}}{R/I}$

Lateral System Type: R=3 Cd=3

$T_a = 0.028(83)^{0.8} = 0.96$

Gross Floor Areas

1st Floor - 10815 sf
2nd - 14052
3rd 18365
4th 18365
5th 18389
6th 18179
Roof 18179

Snow Load - 20psf

Loads: Roof:
MH. Deck - 2 psf
Mech - 4 psf
Ceiling - 3 psf
Roofing - 5 psf
Insulation - 4 psf
Framing - 7 psf
25 psf

Floor: Conc. Slab - 46
Deck - 2
Mech - 4
Ceiling - 3
Floor Covering - 1
Framing - 7
63 psf

partition - 10 psf
Ext. Walls - 15 psf

Floor 1 - Topping Slab
+ 26 psf
= 88 psf - Floor 1 only
part. 10
Ext. wall 15

R=3, Cd=3

A-6.3

$W_{Roof} = 18179(25+.2(20)) = 527^k$

$W_6 = 18179(63+10) + 10 \times 2(100+184)(15) = 1472^k$

$W_5 = 18389(63+10) + 15(2)(100+184)(14) = 1462^k$

$W_{3\&4} = 18365(63+10) + 15(2)(100+184)(14) = 1460^k \times 2$

$W_2 = 14052(63+10) + 15(2)(100+184)(14) = 1145^k$

$W_1 = 10815(88+10) + 15(2)(100+184)(12) = 1162^k$

$C_s = \frac{0.217}{3/1} = 0.072$

$C_s \leq \frac{0.112}{.96(3/1)} = 0.039$

$T = 0.96$

$> 0.014(.217) I = .009$ ✓

$C_s = 0.039$

$V = 0.039(8628) = 336.5^k$

.5 → 1
2.5 → 2
T = .96
$k = 1.23$

$F_x = C_{vx} V$ $\qquad C_{vx} = \frac{w_x h_x^k}{\Sigma w_x h_x^k}$

	Level	$h^k w_x$	C_{vx}	F_x
	B	0	0	0^k
20'	1	46288.5	0.035	11.8
32'	2	81309.3	0.062	20.9^k
46'	3	162011.2	0.123	41.4^k
60'	4	224636.8	0.172	57.9^k
74'	5	291140.	0.222	74.7^k
88'	6	347975	0.265	89.2^k
103'	R	157616	0.121	40.7^k
		1,310,975		

overturning Moment $= \Sigma F_x h_x = 23852.7'k$

A-7.1

Spot Check: Floor System

Slab span = 10'-0"

Loads: Live: 100 psf

Reference: USD design Manual & catalogue of products

2" LOK Floor Leightweight Conc.

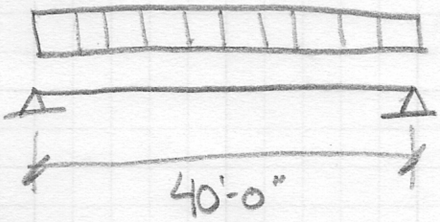
Max Unshored 3 spans continuous
need 19 gage w/ 5.5" slab depth → 3.5" concrete
& will support 270 psf of Live Load.

Design Engineer used:

2" Deck 20 gage min.
w/ 5.25" of concrete.

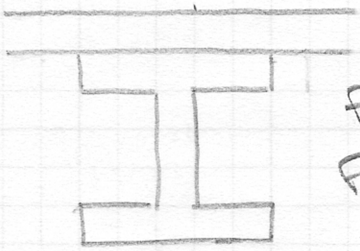
probably choose this in order to increase eccentricity for better composite action. Also would have had to be shored durring construction, I chose unshored slab.

Spot Check: Composit Beam Design.



Beam Spacing = 10'-0"

Live: 100 psf
Dead: 63 psf



$F_y = 50\,ksi$
$F'_c = 3000\,psi$

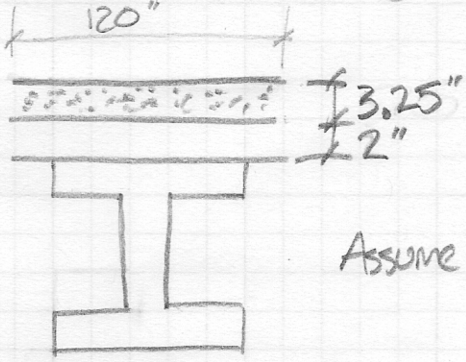
$b_{eff} = \frac{40'}{4} \leq 10'$

$\Rightarrow$ use 120"

$b_{eff} = 120"$

$w_u = 1.2(63) + 1.6(100) = 236\,plf\,(10') = 2.4\,klf$

$$M = \frac{wl^2}{8} = \frac{2.4\,klf\,(40)^2}{8} = 480'k$$



Assume: $a = 1"$ $\quad Y_2 = 5.25 - \frac{1}{2}$

$Y_2 = 4.75"$ $\qquad Y_2 = 4.5" \rightarrow a = 1.5"$

	M	ΣQ_N	
W18x46	482'k	169/17.1 = 98 + 46(40) = 1938	
W18x40	480	272/17.1 (10)	1759
W18x35	494	451/17.1 (70)	1663 ← Most economical

W18x35 $\qquad \Sigma Q_N = 451 \qquad b_{eff} = 120"$

$\phi M_n = 482'k$

$$a = \frac{169}{.85(3)(120)} = 1.47 < 1.5$$

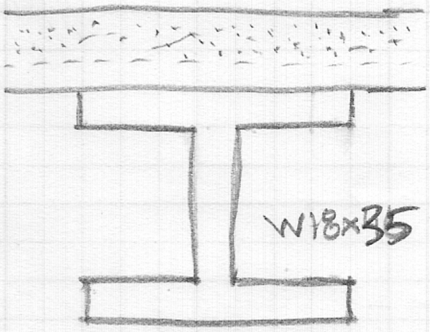
ok

$$Y_2 = 5.25 - \frac{1.47}{2} = 4.515$$

A-7.3

$\Sigma Q_n = \frac{451}{17.2\ k/stud} = 26.2 \rightarrow$ use 27 in $\frac{1}{2}$ span

$27(17.2) = 464^k$



3ksi (120)(a) = 464

a = 1.288" < 1.5 Assume
-ok

$$\boxed{\text{use: W18x35 w/ 54 studs } 3/4"}$$

LL = 20psF = 200plF
DC = .41psF(10)+35 = 445plF

check $\Delta = \frac{5(.445)(40)^4}{384(29000)(510)}$

$\Delta = 1.75"$

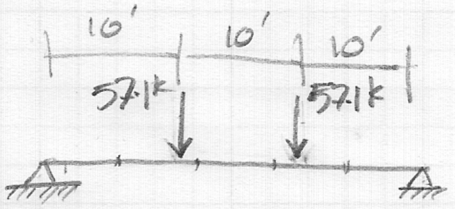
$\Delta \leq \frac{L}{360} : \frac{40(12)}{360} = 1.33"$ ⇒ need to camber beam
$\frac{1}{2}"$ to drop construction deflection to allowable

Designer Used: W18x35 w/ 48 shear studs.
Also cambered beam by 1-7/8"

probably to account for LL Δ

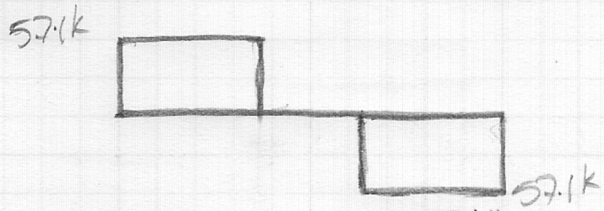
A-7.4

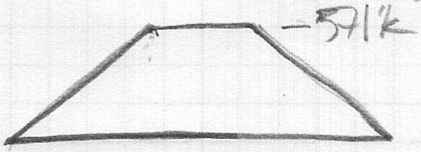
Spot Check: Composite Girder Design



$\frac{30(12)}{4} = 90'' = b_{eff}$

$\frac{30+40(12)}{2} = 180''$





W18×50 $\phi M_p = 379$ $\phi M_n = 589$ (PNA: BFL) $\Sigma Q_N = 306$ $\frac{306}{17.1}(10) = 178 + 1500 = 1679$

W18×55 $\phi M_p = 420$ $\phi M_n = 586$ (PNA: 7) $\Sigma Q_n = 202$ $\frac{202}{17.1}(10) + 1650 = 1767$

W18×50 more economical:

$a = \frac{589}{.85(90)(3)} \Rightarrow a = 2.56 \Rightarrow Y_2 = 5.25 - \frac{2.56}{2}$

$Y_2 = 3.97 \rightarrow$ not conservative

try W18×50 PNA = BFL $Y_2 = 4$ $\phi M_n = 577^k$

$\frac{577}{.85(90)(3)} = a = 2.51$

$Y_2 = 5.25 - \frac{2.51}{2} = 3.995$ – ok

$\frac{577}{17.1} = 33.7 \rightarrow 34$ studs in ½ length

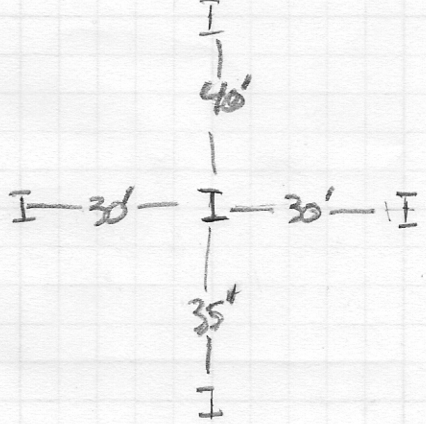
$\boxed{\text{Use W18×50 w/ 68 studs}}$

Designer used W21×50 w/ 71 studs

probably to limit deflection of member

A-7.5

Spot Check: Column



Loads: Roof: → 30 live 25 dead
Floor 3-6 → 63 dead 100 live
Floor 2: → Nothing-open
Floor 1: 88 dead
100 Live

Tributary Area: 30×37.5
$= 1125\ ft^2$

$1.2D + 1.6L$

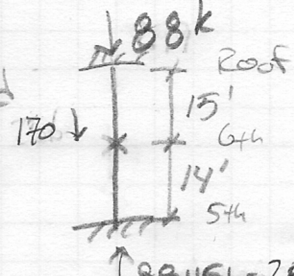
Roof: $1.2(25) + 1.6(30) = 78\,psf\,(1125) = 88^k$

Floors 3-6: $1.2(63) + 1.6(100) = 236\,psf\,(1125) = 266^k$

Floor 1: $1.2(88) + 1.6(100) = 266\,psf = 299^k$

See Live Load Reductions

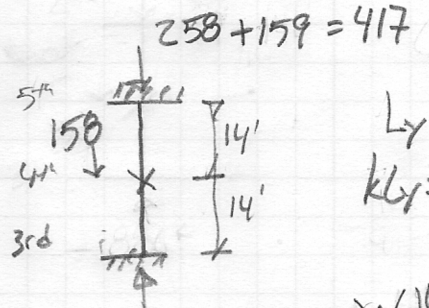
Assume fixed framing at top & bottom



$L_x = L_y \Rightarrow L_y$ controlls

$kL_y = .8\,(15) = 12'$

$\Rightarrow$ W14x43 $\phi_c P_n = 350 > 258^k$

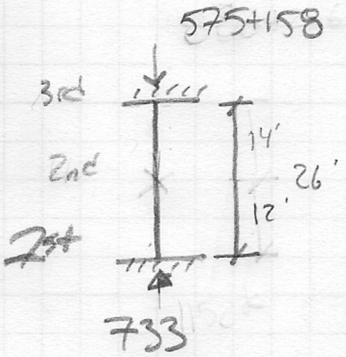


$L_y = 14'$

$kL_y = (0.8)(14) = 11.2'$

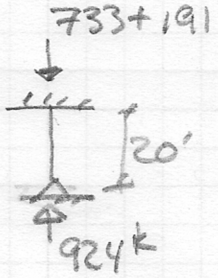
W14x61 $\phi_c P_n @ 12' = 591^k$

A-7.6



$kL_y = .68(26) = 17.7'$

W14×90 $\phi_c P_n = 878 @ 18'$



$kL_y = .8(20) = 16'$

W14x90 $\phi_c P_n = 925 @ 16'$

Design Engineer Used: W14x43
W14x68
W14x109
W14x132

Mine are generally lighter
most probably b/c I haven't added
wind Axial Effects yet.

A-7.8

Live Load Reductions $\quad A_I = (40+35)(60) = 4500$

$$L = L_0\left(.25 + \frac{15}{\sqrt{A_I}}\right)$$

Roof: 88^k

$$\text{Floor 6: } 1.2(63) + 1.6(100)\overbrace{\left(.25 + \frac{15}{\sqrt{4500}}\right)}^{.47\ \checkmark} = 151\text{ psf}$$

$$151(1125) = 170^k$$

$$\text{Floor 5: } 1.2(63) + 1.6(100)\underbrace{\left(.25 + \frac{15}{\sqrt{2 \cdot 4500}}\right)}_{.41 \to ok} = 141\text{ psf}$$

$$141(1125) = 159^k$$

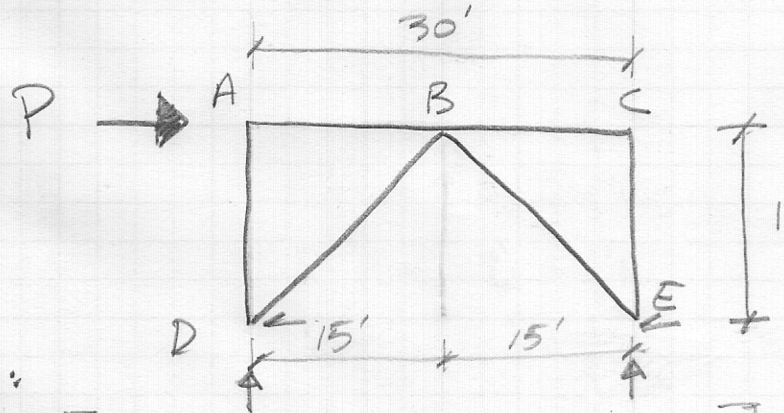
$$\text{Floor 4: } 1.2(63) + 1.6(100)\underbrace{\left(.25 + \frac{15}{\sqrt{4500 \cdot 3}}\right)}_{.38 \to \text{use } .4} = 140\text{ psf}(1125) = 158^k$$

$$\text{Floor 3: } 1.2(63) + 1.6(100)(.4) = 140\text{ psf} = 158^k$$

$$\text{Floor 1: } 1.2(88) + 1.6(100)(.4) = 170\text{ psf} = 191^k$$

Spot Check : Lateral Bracing

Look at Top Bay only in N-S Direction



WIND:

$$P = \left[14.13(100)(3.3) + 13.65(100)(4.2)\right] 1.6$$

$$= 16.6^k$$

SEISMIC: $40.7^k (1.0)$

seismic controlls design of Top Bracing

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

$40.7 \rightarrow \quad \leftarrow F_{BC} = 0$ from end →

F_{BD} 45° 45° F_{BE}

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

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

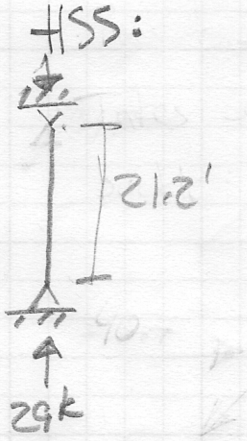
$40.7 = 2F_{BD} \sin 45$

$F_{BD} = F_{BE} = 29^k \rightarrow +/- 29^k$ T&C

A-7.10

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

HSS:



$kL = 1(21.2) = 21.2$

HSS 6×4×1/8
4×4×3/8
5×5×3/16 ← use

HSS 5×5×3/16" will work in compression.

Design Engineer Used 8×8×1/4"

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.