Technical Report 3

Student Health Center

Penn State University

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Executive Summary

The Student Health Center (SHC) is a five story building on the Penn State campus that serves as a health care services and hospital facility. After completion in the fall of 2008, this building now houses University Health Services and Counseling and Psychological Services, two departments of Penn State's Division of Student Affairs.

The facility is 77 feet in height from the first level and is approximately 64,000 SF in area. It has a brick façade rising from the ground with large curtain wall on the south side the building. The structure is held up primarily by a steel frame. The overall structure sits on a mini-pile foundation through use of pile caps, piers, and grade beams. Composite steel with concrete slab on deck is use for the floor system throughout the SHC.

In this technical report, the lateral system was examined in greater detail. Calculations were done to determine building drift, loads on individual frames, strength, overturning due to lateral load, and other characteristics affecting design.

Allowable building drift was calculated for a critical frame in the building. Actual drift was then determined from lateral load analysis. The calculated actual displacement due to the wind load was within allowable limits but the seismic displacement exceeded the code-defined allowable drift. This may be due to a discrepancy in the assumptions of the engineer of record and me on load determination in a previous report and/or my hand calculated determination of stiffness. Overturning was not a factor on the frame selected for analysis. All of the critical members checked also turned out to be designed adequately for strength.

Introduction

The Student Health Center gives off a light and inviting atmosphere through use of a large curtain wall. This curtain wall works to let natural light into the building, as well as, expose the inner structure from the outside. This report is meant to examine the ability of this structure to resist lateral loads. The moment frame was tested for serviceability and strength. A STAAD model of every frame was create to check each frame's rigidity and to check deflections for serviceability. From this computer input and some hand calculations, conclusions were drawn as to how well the frame in place works to combat the imposed loads.

Structural Systems

Foundation:

The foundation of the SHC is composed of grade beams and piers that are supported by mini-piles with pile caps. The mini-piles are arranged in configurations of 1-5 piles per pile cap. They are to be at a depth of 45 feet and have an 80 ton allowable capacity. The partially-restrained moment frame employed in this building is either connected directly to a pile cap or to a concrete pier. The depth of these mini-piles will counteract the moment of the partially-restrained moment frame caused by lateral loads.

Floor System / Beams:

The floor system used in the SHC is composed of 3 1/4" lightweight concrete fill on 2"-20 gage galvanized composite floor deck LOK floor for a total slab thickness of 5 1/4". Also included are 3/4ϕ x 4" long shear studs equally spaced along the entire lengths of all interior beams and girders that are not part of the partially-restrained moment frame. The shear studs are not on the moment frame because the beams on the frame cannot be too rigid so that they can deform. This composite floor deck is supported by steel W-shape beams spanning between steel columns.

Columns:

The P.R. moment frame consists of W14 steel columns running from the foundation up to the roof level. Columns that are not part of the P.R. moment frame range in size and shape. Round HSS shapes are used both with and without concrete fill, as well as square HSS shapes and W shapes to resist gravity loads.

Roof / Penthouse Level:

The roof system is composed of 5 1/4" normal weight concrete fill on 3"-20 gage galvanized composite floor deck LOK floor for a total slab thickness of 8 1/4". The main roof is at the 6th level with a screen wall around the rooftop mechanical equipment. There is also a green roof around the perimeter of the main roof level *(Fig. 2)*. On the north end of the building, at the 5th level, there is another green roof *(Fig. 3)* that is nearly 20 feet wide and runs the length of the building.

Fig. 2 – Green Roof on Main Roof

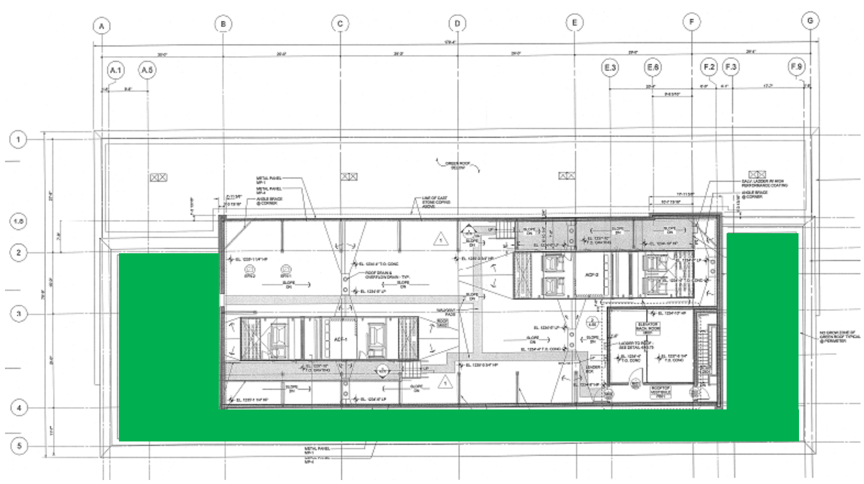


Fig 3 – Green Roof on 5th Floor

Lateral System:

A partially-restrained moment frame is used to resist lateral loads on the SHC. These frames are to have Flexible Moment Connections (FMC) designed by the steel fabricator per Part 11 of the AISC- Load & Resistance Factor Design Manual. A typical beam to column flange connection for these frames is detailed below *(Fig. 4)*. There are eight partially-restrained frames employed in this building, with seven running in the north/south direction, and one in the east/west direction *(Fig. 5)*. These frames run vertically up to the 5th Level or Main Roof Level of the building depending on the location. Frames are shown below in elevation (Fig. 6-8).

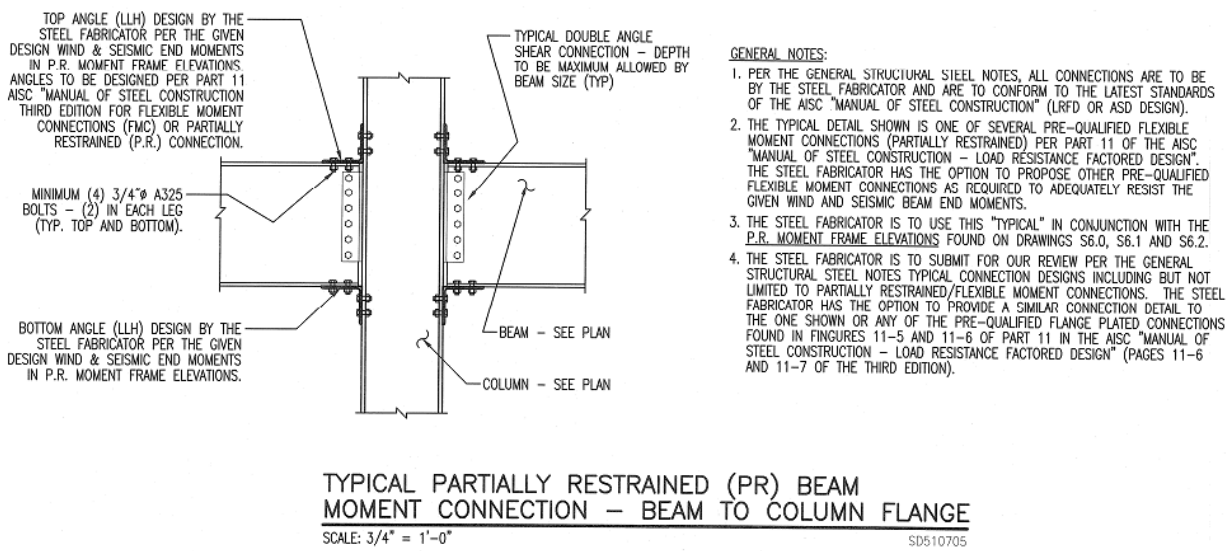


Fig. 4

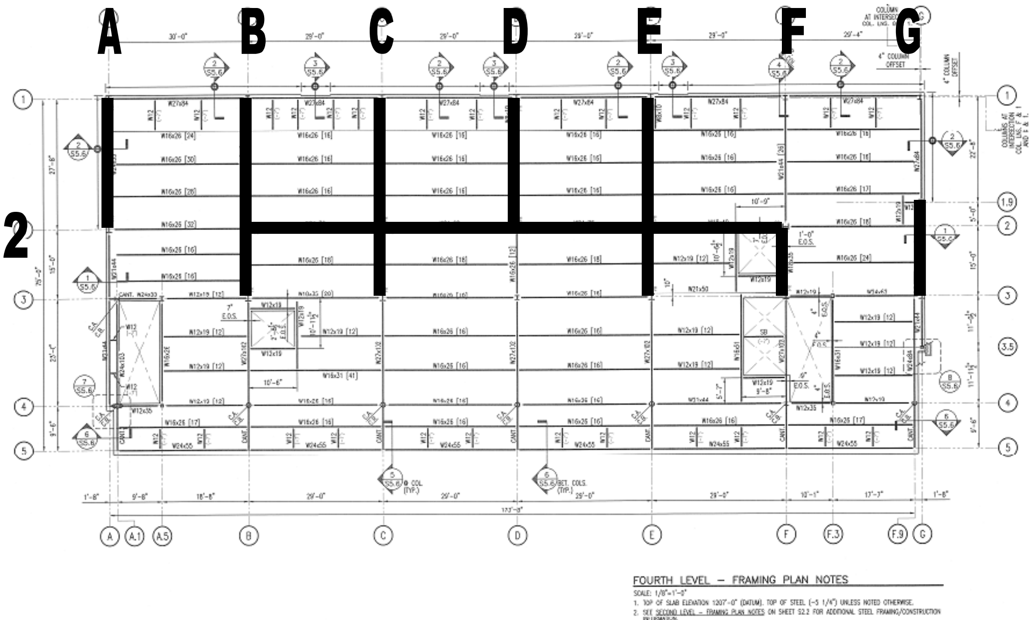


Fig. 5 – Partially-restrained Frame Locations

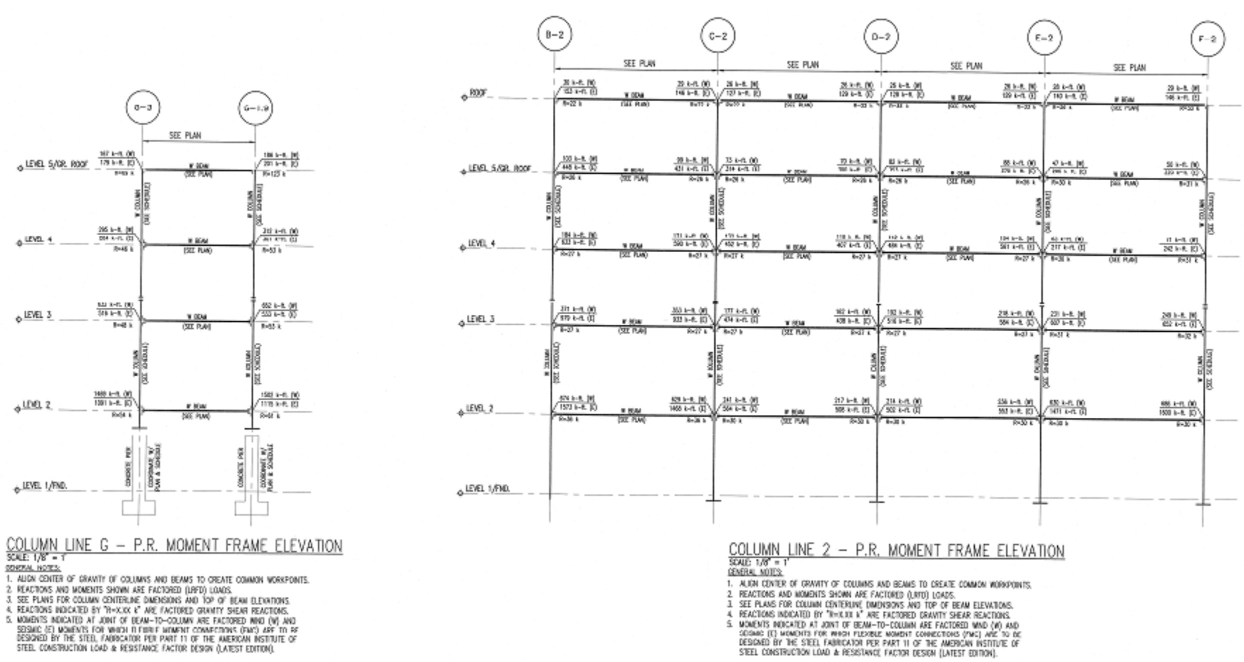


Fig. 6 – P.R. Moment Frame Elevations (G and 2)

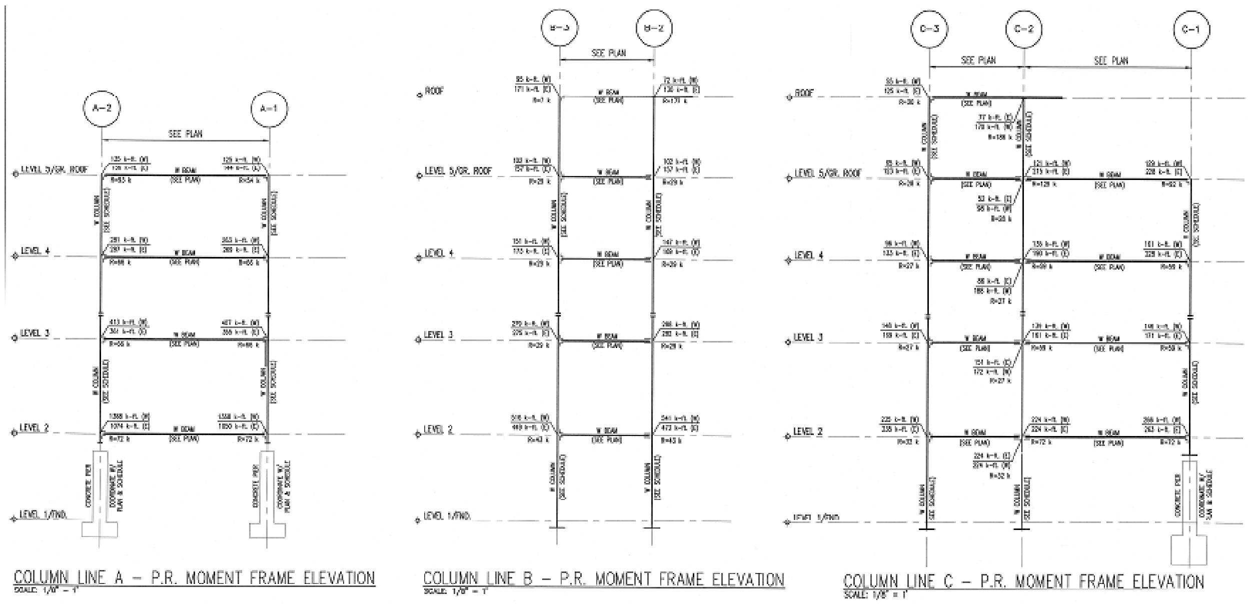


Fig. 7 – P.R. Moment Frame Elevations (A, B, and C)

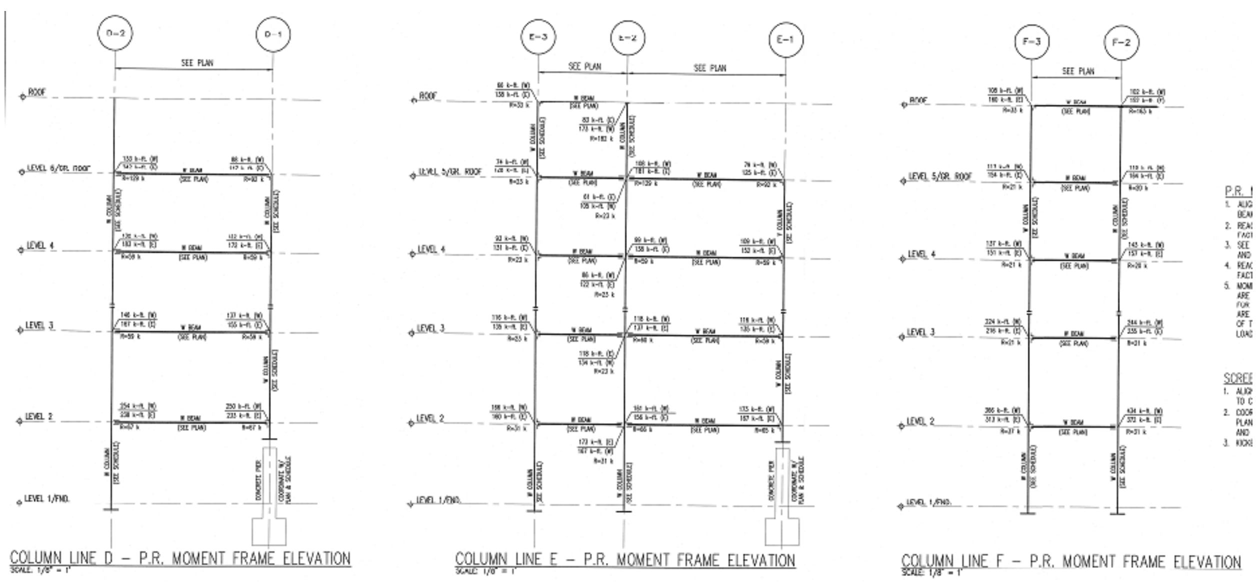


Fig. 8 – P.R. Moment Frame Elevations (D, E, and F)

Code and Design Requirements

Design Codes and References:

Codes used by Project Team:

International Building Code (IBC)/2003 with Borough Amendments
International Mechanical Code (IMC)/2003 with Borough Amendments
International Plumbing Code (IPC)/2003 with Borough Amendments
International Energy Conservation Code (IECC)/2003 with Borough Amendments
International Code Council Electrical Code (ICCEC)/2003
International Fire Code (IFC)/2003
ACI 318-05
AISC "Steel Construction Manual" (13th Edition)
ACI 530.1/ASCE 6/TMS 602 (2005)

Codes used for Thesis:

International Building Code (IBC)/2006
ACI 318-08
AISC "Steel Construction Manual" (13^{th} Edition)
ASCE 7-05

Deflection Criteria:

Maximum Floor Deflections:
- L/360 Live load
- L/240 Total load
- L/240 Roof

Maximum Lateral Deflections:
- L/400 - Drift due to wind
- $0.020h_{sx}$ - Drift due to seismic

Load Combinations:

1.4 (Dead)
1.2 (Dead) + 1.6 (Live) + 0.5 (Roof Live)
1.2 (Dead) + 1.6 (Roof Live) + 1.0 (Live or 0.8 Wind)
1.2 (Dead) + 1.6 (Wind) + 1.0 (Live) + 0.5 (Roof Live)
1.2 (Dead) + 1.0 (Seismic) + 1.0 (Live)
0.9 (Dead) + 1.6 (Wind)
0.9 (Dead) + 1.0 (Seismic)

Material Properties

Material	A.S.T.M.	Minimum Strength
Concrete		
Foundation Walls, Pile Caps, Slab on Grade, Retaining Walls, Footings	-	3000 PSI
Exterior Slabs, Curbs	-	4000 PSI
Reinforcement	A615 (Grade 60)	60 KSI
WWF	A185, A497	70 KSI
Structural Tubing, Round	A500 (Grade B)	42 KSI
Structural Tubing, Shaped	A500 (Grade B)	46 KSI
Steel Pipe	A53 (Type E, Grade B)	35 KSI
Rolled Shapes	A992	50 KSI
Other Rolled Plates	A36	36 KSI
Connection Bolts	A325	92 KSI
Anchor Bolts	A307	-
Threaded Rods	A36	36 KSI
Non-shrink Grout	C1107	8000 PSI
CMU	C90 (lightweight)	2800 PSI

Loads

Gravity Loads:

Dead Load:

Dead Loads were obtained using typical design values, material specifications, or educated assumptions. My values were very similar to values stated by the Engineer of Record.

Component	Obtained Values
2" Steel Deck (on floors 1-5)	2 PSF
3-1/4" Concrete on Deck (on floors 1-5)	43 PSF
3" Steel Deck (on main roof level)	2 PSF
5-1/4" Concrete on Deck (on main roof level)	82 PSF
Green Roof	25 PSF
Ceiling with Mechanical/Electrical	15 PSF
Floor Finish	3 PSF

Live Load:

Live Loads were taken from ASCE 7-05 along with an assumption for the mechanical rooms. My obtained values were once again very similar to the values on the drawings.

Building Location	Drawing Values	Obtained Values
Corridors (first floor)	100 PSF	100 PSF
Corridors (above first floor)	80 PSF	80 PSF
Procedure/Exam Rooms	50 PSF + 20 PSF partition	40 PSF + 15 PSF partition
Lobbies	100 PSF	100 PSF
Stairs	125 PSF	100 PSF
Mechanical Rooms	75 PSF	150 PSF
Offices	50 PSF + 20 PSF partition	50 PSF + 15 PSF partition
Light Storage	125 PSF	125 PSF
Heavy Storage	250 PSF	250 PSF

Snow Load:

Snow loads were determined using IBC 2006 and Centre Region Code.

$p_f = 0.7 \times C_e \times C_t \times I \times p_g = 30.8$ psf
$p_g = 40$ psf
$C_e = 1.0$
$C = 1.0$
$I = 1.1$

Lateral Loads:

Wind Load:

Wind loads were calculated using ASCE 7-05, Section 6.5. "Method 2 - Analytical Procedure" was used to determine wind loads in the N-S and E-W directions. The façade in each direction was assumed to be rectangular to simplify calculations.

The controlling base shear and overturning moment for wind loading were due to the wind in the N-S direction. These values were 337.93 K and 13,648 ft-K respectively. Wind Pressure Diagrams are shown in *(Fig. 9)*. Detailed calculations are shown in Appendix A.

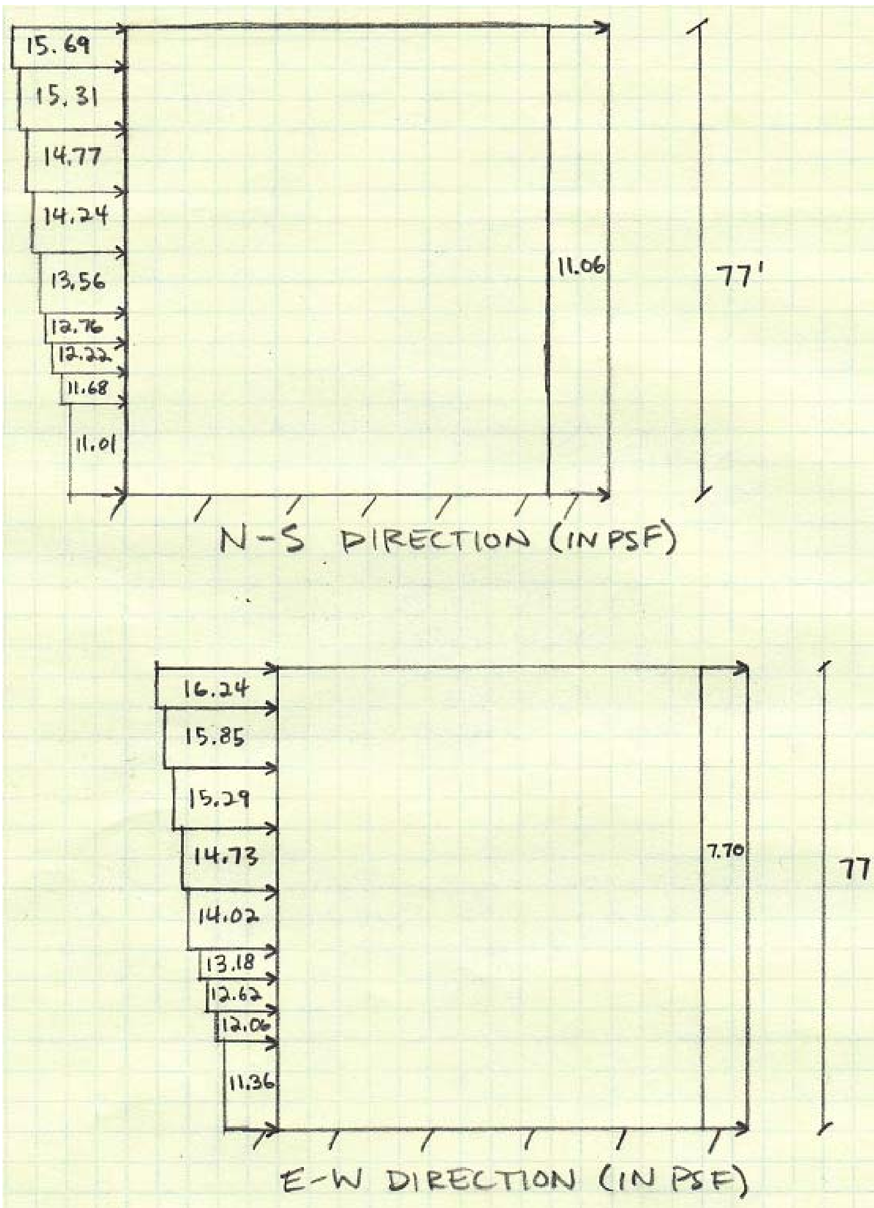


Fig 9 – Wind Diagrams

Seismic Load:

Using ASCE 7-05, Chapters 12, seismic loads were calculated. Information particular to the SHC was taken from the geotechnical report, the Centre Region Code, and the drawings. For details of these calculations, refer to Appendix B.

Level	h_x (ft)	Story Weight (k)	$h_x^k W_x$	C_{vx}	$F_x = C_{vx}V$	V_x (k)	M_x (ft-K)
Main Roof	84	469	95567	0.120	35	35	2936
5	70	1860	304530	0.383	111	146	10242
4	56	1356	169858	0.214	62	208	11672
3	43	1501	136944	0.172	50	258	11115
2	19	1535	52555	0.066	19	278	5277
1	14	1501	35623	0.045	13	291	4070
Total	84	8222	795077	1.0	**291**		**45311**

The base shear calculated was 291 K, which is fairly close to the base shear determined by the Engineer of Record, which was 252 K. Our difference in numbers could be explained by a difference in calculated building weight, in which I made a rough estimate for simplicity, or our interpretation of the code.

Relative Stiffness

To determine the amount of force that is directed to each frame, relative stiffness needed to be calculated. A model of each frame was created using STAAD.Pro 2006. A one kip load was applied to the top of each frame and the lateral displacement caused by that load was tabulated in the program. Displacements of the frames at each level are shown below.

Lateral Displacement										
				N-S Frames						E-W Frames
Level	**Floor Ht**	**Frame A**	**Frame B**	**Frame C**	**Frame D**	**Frame E**	**Frame F**	**Frame G**		**Frame 2**
Roof	14	0	0.14417	0.07941	0	0.08638	0.16572	0		0.03859
5th	13	0.06371	0.11313	0.05354	0.07918	0.06097	0.12976	0.06292		0.02779
4th	14	0.03969	0.07584	0.0357	0.05547	0.04204	0.08805	0.0316		0.01962
3rd	15	0.01305	0.03856	0.01435	0.02281	0.01838	0.04589	0.00761		0.01143
2nd	14	0	0.01267	0.00065	0.00057	0.00093	0.01602	0.00074		0.00406

From these displacements, stiffness can easily be calculated by taking the inverse. The stiffness of each frame in relation to the other frames in the same direction was tabulated from these numbers. The relative stiffness is important because this is how we know the amount of total lateral load to be distributed to each frame. A table with these values is shown here.

Lateral Rigidity										
									N-S Total	E-W Total
Level	**Floor Ht**	**Frame A**	**Frame B**	**Frame C**	**Frame D**	**Frame E**	**Frame F**	**Frame G**	**R (k/in)**	**Frame 2**
Roof	14	0.00	6.94	12.59	0.00	11.58	6.03	0.00	37.14	25.9
5th	13	15.70	8.84	18.68	12.63	16.40	7.71	15.89	95.84	36.0
4th	14	25.20	13.19	28.01	18.03	23.79	11.36	31.65	151.21	51.0
3rd	15	76.63	25.93	69.69	43.84	54.41	21.79	131.41	423.69	87.5
2nd	14	0.00	78.93	1538	1754	1075	62.42	1351	5861	246.3
Total	70	117.5	133.8	1667.4	1828.9	1181.4	109.3	1530.3	6569	447
Rel. Stiffness (%)		1.8	2.0	25.4	27.8	18.0	1.7	23.3	100	100

Direct Shear

The story shears due to wind and seismic loads were calculated in Technical Report 1 and are included in this report. The direct shear for each story on each frame was determined by multiplying the story shear forces by the relative stiffness. Results of these calculations are shown below.

Wind Direct Shear (V*Ri / ∑R), (in kips)

Level	Story Shear	Frame A	Frame B	Frame C	Frame D	Frame E	Frame F	Frame G		Story Shear	Frame 2
Roof	66.07	0.00	12.34	22.40	0.00	20.59	10.73	0.00		22.92	22.92
5th	62.05	10.16	5.72	12.09	8.18	10.62	4.99	10.29		21.43	21.43
4th	59.99	10.00	5.23	11.11	7.15	9.44	4.51	12.55		20.6	20.6
3rd	58.51	10.58	3.58	9.62	6.05	7.51	3.01	18.15		19.85	19.85
2nd	57.6	0.00	0.78	15.12	17.24	10.57	0.61	13.28		19.34	19.34
Base	304.2	30.7	27.7	70.4	38.6	58.7	23.9	54.3		104.1	104.1

Seismic Direct Shear (V*Ri / ∑R), (in kips)

Level	Story Shear	Frame A	Frame B	Frame C	Frame D	Frame E	Frame F	Frame G		Story Shear	Frame 2
Roof	111	0.00	20.73	37.64	0.00	34.60	18.03	0.00		22.92	22.92
5th	62	10.15	5.72	12.08	8.17	10.61	4.99	10.28		21.43	21.43
4th	50	8.33	4.36	9.26	5.96	7.87	3.76	10.46		20.6	20.6
3rd	19	3.44	1.16	3.13	1.97	2.44	0.98	5.89		19.85	19.85
2nd	13	0.00	0.18	3.41	3.89	2.39	0.14	3.00		19.34	19.34
Base	255.0	21.9	32.1	65.5	20.0	57.9	27.9	29.6		104.1	104.1

Torsional Shear

The shear force due to torsion is caused by the twisting of the structure due to eccentric lateral loads. This eccentricity is between the center of pressure and the center of rigidity for wind loading, and between the center of mass and the center of rigidity for seismic loading. For simplicity sake, the center of pressure and center of mass were the taken as the same value for calculations. A more extensive computer model will be completed in a later report to find these exact values. A table showing the centers of mass and rigidity, as well as, eccentricities is shown here.

Centers of Mass & Rigidity and Eccentricities

Level	COR (x)	COR (y)	COM (x)	COM (y)	e (x)	e (y)
Roof	85.8	48	87.5	27.5	1.7	20.5
5th	86.6	48	87.5	37.5	0.9	10.5
4th	90.0	48	87.5	37.5	2.5	10.5
3rd	97.5	48	87.5	37.5	10.0	10.5
2nd	105.6	48	87.5	37.5	18.1	10.5

The torsional shear was calculated using these values. Frame G ended up being the frame experiencing the most shear due to torsion and Frame 2, being the only frame running in the East/West direction at the center of rigidity, did not experience any torsional force. The torsion shear tables, showing the effects on each level, on each frame is shown below. Additional calculation tables are shown in Appendix C.

Wind Torsional Shear (V*e*Ri*d / (∑R*d^2))

Level	Story Shear	Frame A	Frame B	Frame C	Frame D	Frame E	Frame F	Frame G		Story Shear	Frame 2
Roof	66.07	0.00	0.68	0.59	0.00	0.64	0.64	0.00		22.92	0.00
5th	62.05	0.22	0.08	0.08	0.00	0.08	0.07	0.23		21.43	0.00
4th	59.99	0.62	0.21	0.24	0.01	0.17	0.17	0.73		20.6	0.00
3rd	58.51	2.40	0.56	0.86	0.13	0.34	0.34	3.27		19.85	0.00
2nd	57.6	0.00	0.56	6.74	2.91	1.15	0.24	8.82		19.34	0.00
	Base	3.2	2.1	8.5	3.1	2.4	1.5	13.0		104.1	0.00

Seismic Torsional Shear (V*e*Ri*d / (∑R*d^2))

Level	Story Shear	Frame A	Frame B	Frame C	Frame D	Frame E	Frame F	Frame G		Story Shear	Frame 2
Roof	111	0.00	1.15	1.00	0.00	1.07	1.08	0.00		22.92	0.00
5th	62	0.22	0.08	0.08	0.00	0.08	0.07	0.23		21.43	0.00
4th	50	0.51	0.18	0.20	0.01	0.15	0.14	0.61		20.6	0.00
3rd	19	0.78	0.18	0.28	0.04	0.11	0.11	1.06		19.85	0.00
2nd	13	0.00	0.13	1.52	0.66	0.26	0.05	1.99		19.34	0.00
	Base	1.5	1.7	3.1	0.7	1.7	1.5	3.9		104.1	0.00

Total Shear

The direct shear on the structural system is combined with the torsional shear to determine the maximum amount of shear force possible at each level on each frame. As shown in the following tables, Frame 2 needs to resist the largest amount of shear force in the E/W direction and Frame C in the N/S direction. Frame 2 is the only frame running N/S and Frame C is the largest E/W frame, so these values seem correct. As you can see, the wind values and seismic values are

Wind Total Shear

Level	Story Shear	Frame A	Frame B	Frame C	Frame D	Frame E	Frame F	Frame G		Story Shear	Frame 2
Roof	66.07	0.00	13.02	23.00	0.00	19.96	10.09	0.00		22.92	22.92
5th	62.05	10.38	5.80	12.18	8.18	10.54	4.91	10.06		21.43	21.43
4th	59.99	10.61	5.45	11.35	7.14	9.26	4.33	11.83		20.6	20.60
3rd	58.51	12.98	4.14	10.48	5.92	7.17	2.67	14.88		19.85	19.85
2nd	57.6	0.00	1.34	21.86	14.34	9.42	0.38	4.46		19.34	19.34
	Base	34.0	29.8	78.9	35.6	56.3	22.4	41.2		104.1	104.1

Seismic Total Shear

Level	Story Shear	Frame A	Frame B	Frame C	Frame D	Frame E	Frame F	Frame G		Story Shear	Frame 2
Roof	111	0.00	21.88	38.64	0.00	33.53	16.96	0.00		22.92	22.92
5th	62	10.38	5.80	12.17	8.17	10.53	4.91	10.05		21.43	21.43
4th	50	8.84	4.54	9.46	5.95	7.72	3.61	9.86		20.6	20.60
3rd	19	4.21	1.35	3.40	1.92	2.33	0.87	4.83		19.85	19.85
2nd	13	0.00	0.30	4.93	3.24	2.12	0.08	1.01		19.34	19.34
	Base	23.4	33.9	68.6	19.3	56.2	26.4	25.7		104.1	104.1

Drift

Drift characteristics were examined to determine the serviceability of the SHC. For wind loading, the drift cannot exceed L/400 and for seismic, it cannot exceed $0.020h_{sx}$. Frame A was chosen for analysis because it is the farthest frame from the center of rigidity. This is a preliminary analysis of drift effects and further investigation of the drift, through computer input of the entire structural steel system, will be done at a later date. Seismic loading is clearly the controlling factor in the analysis of Frame A for drift. Total drift was acceptable for wind effects but not for seismic. This discontinuity between my values and the actual design may be due to assumptions made in calculating rigidity and/or lateral loads in Technical Report 1. A check of previous calculations will be done before proceeding into further investigations of the lateral system in later reports.

Wind Effects on Frame A

Story	Story Drift	Allowable Drift	
5th	0.32	0.39	ok
4th	0.55	0.42	not ok
3rd	0.38	0.45	ok
Total	1.25	1.26	ok

Seismic Effects on Frame A

Story	Story Drift	Allowable Drift	
5th	0.29	0.26	not ok
4th	0.46	0.28	not ok
3rd	0.27	0.3	ok
Total	1.02	0.84	not ok

Overturning

A check was done to see if the lateral load caused an overturning moment large enough to cause uplift on a frame column. The analysis was done on the skinniest frame because this should create the greatest amount of uplift on the near column. After the uplift force was found, it was compared to the amount of dead load that would be counteracting the uplift. It was found that the lateral loads will not cause overturning. The calculations that led to this conclusion are in Appendix D.

Strength Check

Another analysis done in this report is the strength check of some critical members. This check uses the same frame as the drift check, Frame A, for consistency. One set of calculations is for a third floor beam and the other is for a second story column.

For the beam, maximum moment was found by combining the moment from lateral loads and gravity loads. The moment caused by lateral loads was found by portal method. The maximum moment was then verified to be less than the allowable moment. A serviceability check was also done, and the W21x68 beam was deemed adequate.

The maximum moment of the column was also determined by portal method. The combined loading of this moment and the compressive force was checked against the allowable for the W14x159 column. The designed column was determined to be a viable member by analysis. All calculations for these strength checks are shown in Appendix E.

Conclusion

Upon completion of Technical Report 3, a better understanding of how load is distributed through the structure is gained. Stiffness parameters were used to determine how much of the lateral load is applied to each individual frame. Stiffness was determined using the unit deflections calculated from a STAAD computer model of each frame. A determination of torsion effects on the structure was also made using eccentricities between the center of rigidity and the center of mass for analysis. All in all, the total shear on the frames due to wind as compared to seismic was very close. The moment caused by the seismic forces is significantly larger though, due to larger shear forces at higher stories, therefore seismic load will control design.

The only concern encountered in all of the checks of the current design of the SHC, was building drift. Several story drifts were determined to exceed the maximum allowable drift per code provisions. These discrepancies did not differ significantly; therefore the failure is possibly due to an accumulation of assumptions from all of the technical reports. Before further inspection concerning the lateral system, a reanalysis of previous calculations may need to be completed. Overturning was determined not to be an issue and all of the strength checks checked out.

For future analysis of the lateral force resisting system, an extensive computer model of the entire structure will have to be constructed and additional analysis will need to be done.

Appendix

A: Wind Calculations:

V_{3S} (mph)	90
K_d	0.85
Occupancy Category	III
I	1.15
Exposure Category	B
K_{zt}	1
Building Height (ft)	77
$n_1 = 22.2/H^{0.8}$	0.687
$g_Q = g_v$	3.4
g_R	4.644
$\dot{z}$	46.2
c	0.3
$I_{\dot{z}} = c(33/\dot{z})$^(1/6)	0.284
ℓ	320
ϵ	0.333
$L_{\dot{z}}$ = ℓ($\dot{z}$/33)^ϵ	358
B (East-West) (ft)	70
B (North-South) (ft)	180
Q (E-W) (Eq. 6-6)	0.858
Q (N-S)	0.813

ƀ	0.45
α	0.25
$V_{\dot{z}}$ (Eq. 6-14)	64.613
$N_1 = n_1 L_{\dot{z}}/V_{\dot{z}}$	3.808
R_n (Eq. 6-11)	0.060
$\eta = 4.6n_1h/V_{\dot{z}}$	3.768
R_h (Eq. 6-13)	0.230
$\eta = 4.6n_1B/V\dot{z}$ (E-W)	3.425
$\eta = 4.6n_1B/V\dot{z}$ (N-S)	8.808
R_B (Eq. 6-13) (E-W)	0.249
R_B (Eq. 6-13) (N-S)	0.107
$\eta = 15.4n_1L/V\dot{z}$ (E-W)	29.487
$\eta = 15.4n_1L/V\dot{z}$ (N-S)	11.467
R_L (Eq. 6-13) (E-W)	0.033
R_L (Eq. 6-13) (N-S)	0.083
R (Eq. 6-10) (E-W)	0.194
R (Eq. 6-10) (N-S)	0.130
G_f (Eq. 6-8) (E-W)	0.866
G_f (Eq. 6-8) (N-S)	0.829
GC_{pi}	0.18

	C_p (E-W)	C_p (N-S)
Windward Wall	0.8	0.8
Leeward Wall	-0.27	-0.5
Side Wall	-0.7	-0.7
	L/B = 2.57	L/B = 0.39

Wind Pressures (N-S)

Height (ft)	K_z	q_z (Eq. 6-15)	p_z (Eq. 6-19) (psf)	p_h (Eq. 6-19) (psf)	Total Pressure (psf)
0-15	0.57	11.55	11.01	-11.06	22.07
20	0.62	12.57	11.68	-11.06	22.74
25	0.66	13.38	12.22	-11.06	23.28
30	0.7	14.19	12.76	-11.06	23.82
40	0.76	15.40	13.56	-11.06	24.62
50	0.81	16.42	14.24	-11.06	25.30
60	0.85	17.23	14.77	-11.06	25.83
70	0.89	18.04	15.31	-11.06	26.37
77	0.918	18.61	15.69	-11.06	26.75

Wind Pressures (E-W)

Height (ft)	K_z	q_z (Eq. 6-15)	p_z (Eq. 6-19) (psf)	p_h (Eq. 6-19) (psf)	Total Pressure (psf)
0-15	0.57	11.55	11.36	-7.70	19.06
20	0.62	12.57	12.06	-7.70	19.76
25	0.66	13.38	12.62	-7.70	20.32
30	0.7	14.19	13.18	-7.70	20.88
40	0.76	15.40	14.02	-7.70	21.73
50	0.81	16.42	14.73	-7.70	22.43
60	0.85	17.23	15.29	-7.70	22.99
70	0.89	18.04	15.85	-7.70	23.55
77	0.918	18.61	16.24	-7.70	23.95

(N-S)

Level	Height (ft)	Force (K)	Shear (K)	Moment (ftK)
Penthouse Roof	77	33.70	33.70	2595.0
Main Roof	63	66.07	99.77	4162.3
5	49	62.05	161.82	3040.4
4	36	59.99	221.81	2159.8
3	22	58.51	280.32	1287.2
2	7	57.60	337.93	403.2
1	-7	0	337.93	0
		Total	337.93	13648.0

(E-W)

Level	Height (ft)	Force (K)	Shear (K)	Moment (ftK)
Penthouse Roof	77	11.73	11.73	903.5
Main Roof	63	22.92	34.66	1444.2
5	49	21.43	56.09	1050.1
4	36	20.60	76.68	741.4
3	22	19.85	96.53	436.7
2	7	19.34	115.88	135.4
1	-7	0	115.88	0.0
		Total	115.88	4711.3

B: Seismic Calculations:

Main Roof					
Dead Load w/o stl framing (psf)	Steel Framing Load (lbs)	Snow Load (psf)	Perimeter Wall Load (psf)	Perimeter Length (ft)	Level Ht (ft)
0	0	0	100	335	14
Level Weight (k)	469				

5th					
Dead Load w/o stl framing (psf)	Steel Framing Load (lbs)	Snow Load (psf)	Perimeter Wall Load (psf)	Perimeter Length (ft)	Level Ht (ft)
124	25000	30.8	10	330	14
99			100	335	
Level Weight (k)	1860				

4th					
Dead Load w/o stl framing (psf)	Steel Framing Load (lbs)	Snow Load (psf)	Perimeter Wall Load (psf)	Perimeter Length (ft)	Level Ht (ft)
63	127000	30.8	10	210	13
88			100	310	
Level Weight (k)	1356				

3rd					
Dead Load w/o stl framing (psf)	Steel Framing Load (lbs)	Snow Load (psf)	Perimeter Wall Load (psf)	Perimeter Length (ft)	Level Ht (ft)
63	151000	0	10	210	14
			100	310	
Level Weight (k)	1501				

2nd					
Dead Load w/o stl framing (psf)	Steel Framing Load (lbs)	Snow Load (psf)	Perimeter Wall Load (psf)	Perimeter Length (ft)	Level Ht (ft)
63	151000	0	10	210	15
			100	310	
Level Weight (k)	1535				

1st					
Dead Load w/o stl framing (psf)	Steel Framing Load (lbs)	Snow Load (psf)	Perimeter Wall Load (psf)	Perimeter Length (ft)	Level Ht (ft)
63	151000	0	10	210	14
			100	310	
Level Weight (k)	1501				

W, Total Building Weight (k)	8222

S_s (from Centre Region Code)	0.17
S_1 (from Centre Region Code)	0.06
Site Class (from Geotech Report)	D
F_a	1.6
F_v	2.4
$S_{MS} = F_aS_s$	0.272
$S_{M1} = F_vS_1$	0.144
$SD_S = 2S_{MS}/3$	0.181
$SD_1 = 2S_{M1}/3$	0.096
Seismic Design Category	B
R (ordinary steel moment frame)	3.5
C_d	3
I	1.25
Ct (Table 12.8-2)	0.028
x (Table 12.8-2)	0.8
$T_a = C_th_n^x$	0.970
T_L (Fig. 22-15)	6
C_s (Eq. 12.8-3)	0.035
W (k)	8222
V = C_sW (k)	**291**
k	1.2

C: Torsional Shear Tables:

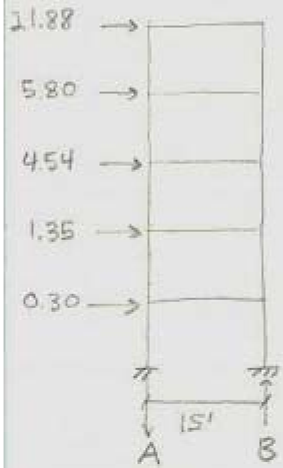
d (ft)						
85.80	55.80	26.80	2.20	31.20	60.20	89.20
86.64	56.64	27.64	1.36	30.36	59.36	88.36
90.03	60.03	31.03	2.03	26.97	55.97	84.97
97.45	67.45	38.45	9.45	19.55	48.55	77.55
105.60	75.60	46.60	17.60	11.40	40.40	69.40

(∑R*d^2)
63779
326831
560137
1815061
11085613

D: Overturning Calculations:

OVERTURNING CHECK

Check Frame B with the larger lateral load (shear) because of its small width. (15ft)

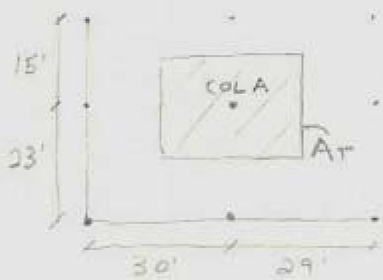


$M = (21.88 \times 70) + (5.8 \times 56) + (4.54 \times 43) + (1.35 \times 29) + (0.3 \times 14) =$

$M = 2095 \text{ ft}\cdot\text{k}$

UPLIFT ON COLUMN A

$A = 2095/15 = 140 \text{ K}$



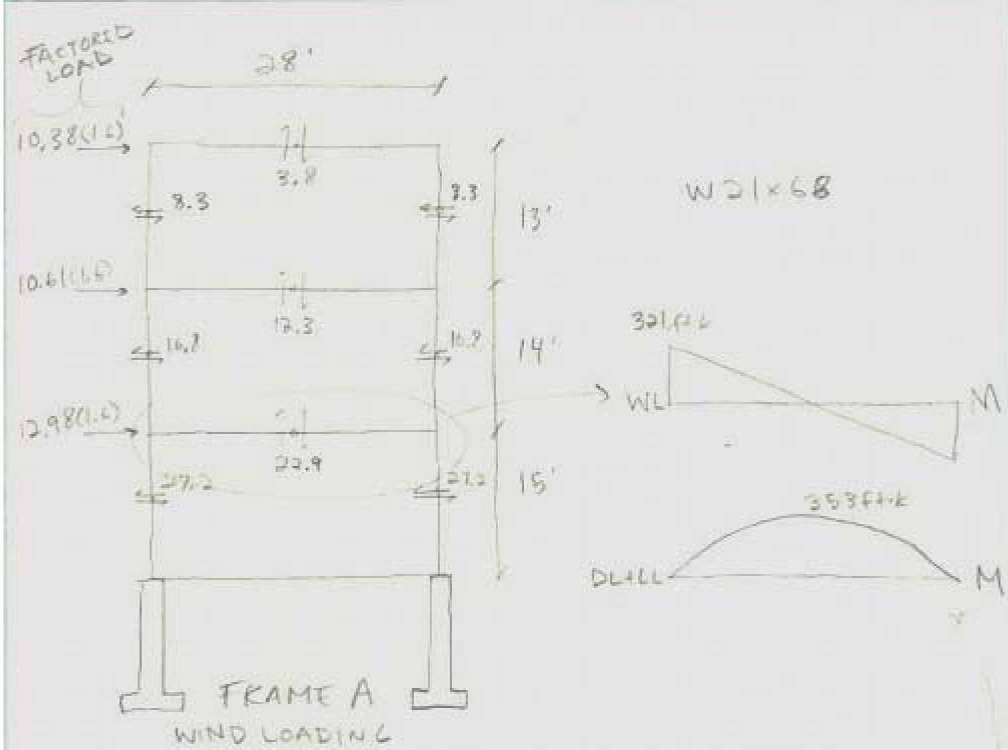
FLOOR WEIGHTS (DL)

MAIN ROOF	84 PSF
5th	197 PSF
4th	121 PSF
3rd	107 PSF
2nd	109 PSF
1st	+ 107 PSF
	725 PSF TOTAL

$725 \times (A_{T,\text{COLUMN A}}) = 725 \times (19 \times 29.5) = 406 \text{ k}$

$\boxed{406^{k} > 140^{k} \therefore \text{ NO UPLIFT ON FRAME B}}$

E: Strength Calculations:



DL = 107 PSF
LL ≈ 70 PSF

$$w_u = (1.2 \cdot 107 \times 15') + (1.6 \times 70 \times 15') = 3.6\ k/ft$$

$$M_u = \frac{w_u \ell^2}{8} = \frac{3.6(28)^2}{8} = 353\ ft\text{-}k$$

GRAVITY LOADS CONTROL

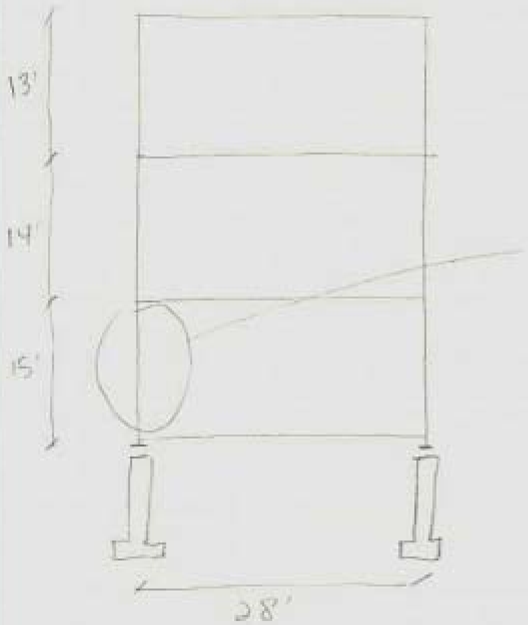
$\phi M_{n(W21\times68)} = 600\ ft\text{-}k > 353\ ft\text{-}k \quad \therefore OK$

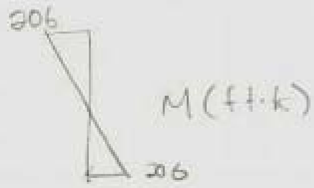
CHECK DEFLECTION:

$$\Delta = \frac{5 w \ell^4}{384 EI} = \frac{5(3.6)(28)^4(1728)}{384(29{,}000)(1480)} = 1.16''$$

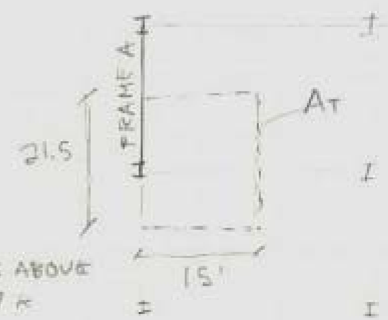
$$\ell/240 = \frac{28(12)}{240} = 1.4'' > 1.16'' \quad \therefore OK$$

$\boxed{\text{BEAM IS ADEQUATE}}$





FROM PORTAL ANALYSIS ON PREVIOUS PAGE



$A_T = 15(21.5) = 322.5\ ft^2$

$P_{LL} = 70\ PSF\ (322.5)(3) = 67.7^k$ ← FLOORS ABOVE

$P_{DL} = 107\ PSF\ (322.5)(4) = 138^k$

$P_u = 1.2(138) + 1.0(67.7) = 233^k$

FROM TABLE 6-1 IN STEEL MANUAL:

FOR W14×159: $p = 0.551 \times 10^{-3}\ \frac{1}{k}$ $b_x = 0.831 \times 10^{-3}\ \frac{1}{ft \cdot k}$

$KL = 15'$

$pP_u + b_x M_u = (0.000551 \times 233) + (0.000831 \times 206) = 0.30$

$0.30 < 1.0 \quad \therefore ok$

$\boxed{\text{COLUMN IS ADEQUATE}}$