Technical Report 3

University Hospitals Case Medical Center Cancer Hospital

11100 Euclid Avenue Cleveland, Ohio

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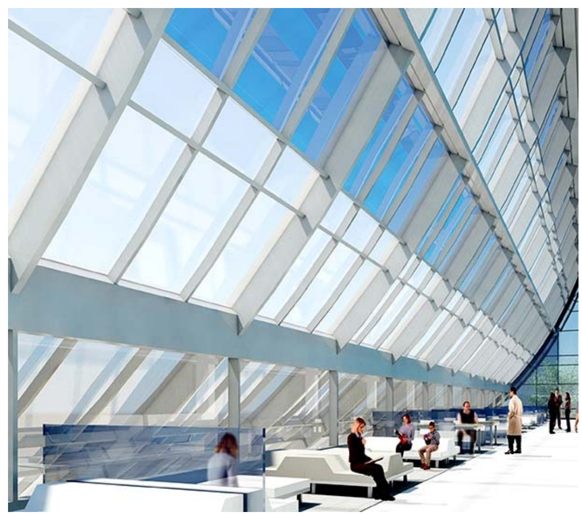
http://www.engr.psu.edu/ae/thesis/portfolios/2009/dcm234

Executive Summary

The University Hospitals Case Medical Center Cancer Hospital is a 9 story research and patient care facility located in Cleveland, Ohio. Its infrastructure consists of steel and steel composite members which have been carefully arranged in order to conform to the modular architectural design system known as the *Universal Grid*, allowing full optimization of available space for varying use. Sloped curtain walls envelope the Cancer Hospital, consisting of exterior glazing and curved steel. The new Cancer Hospital will serve as an addition to the adjacent Case Medical Center which will integrate medical services once spread through 7 different buildings.

The purpose of this report is to evaluate is to evaluate the existing lateral force resisting system of the Cancer Hospital and make note of any significant areas of concern. The system includes 4 typical steel "chevron" braced frames rising the full height of the building, a singly braced frame also rising the full height of the building, and a singly braced frame which rises to the 4th level roof of the lower "L" shaped base.



Lateral loads from wind and seismic effects were determined in accordance with ASCE7-05 and IBC 2006. These loads were then distributed to each frame through calculation utilizing relative stiffness. A 3-Dimensional ETABS model has been assembled and determined to be an accurate representation of the lateral system in the Cancer Hospital based on manual calculation checks. Using output from the model, an evaluation of torsion, drift, member strength and foundation effect has been conducted on the system.

The existing lateral force resisting system utilized in the Cancer center has been found to be adequate in carrying the required loads which are controlled by wind in both directions. As the forces were transferred in calculation to each respective frame, a sizeable amount of torsion was found to be present in the building due to an uneven distribution of stiffness and non-symmetrical frame locations. Irregular bracing configurations were found which contributed to this uneven distribution however all lateral members were determined efficient in carrying required loads. Although the existing Cancer Hospital design complies with all drift limits required by code, additional measures should be taken to decrease this amount of drift in order to adhere to the sensitive movement requirements of the surgery and imaging rooms. Axial forces from overturning moment were taken into consideration and determined to be adequately carried by columns and foundations. Uplift forces in the east/west braced frames were found not to exceed gravity loads however braced frames in the north/south direction require special consideration in foundation design for uplift.

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Introduction

The University Hospitals Case Medical Center Cancer Hospital will integrate patient care and cancer research in a new and innovative way. Architecturally, the Cancer Hospital will reflect this cutting edge link by joining adjacent buildings together while serving as a primary gateway to the UHCMC campus located in Cleveland, Ohio.

The Cancer Hospital design fulfills the wishes of former facility cancer patients in creating an appealing and comfortable environment as opposed to the sterile feel of the past. This is accomplished through use of strong architectural accents including the Cancer Hospital's most dominating feature, its curved facade. A universal grid system consisting of 31'-6" modular bays has been incorporated into design to optimize floor space for varying uses. Clinical pods have been designed for treatment of specific patient populations.

Medical services which were previously distributed among seven facilities will now be performed under one roof to optimize cancer research, education, and patient care while providing an architecturally appealing exterior as well as a warm and inviting natural interior.

This report will evaluate the existing lateral force resisting system of the Cancer Hospital and make note of any significant areas of concern. The current system includes 4 typical steel "chevron" braced frames rising the full height of the building, a singly braced frame also rising the full height of the building, and a singly braced frame which rises only to the 4th level roof of the lower "L" shaped base. The 5 braces which do rise the full height are all located in the tower portion of the structure.

Thorough the analysis of lateral loads caused by both wind and seismic effects determined from ASCE7-05 and IBC 2006, the required capacity of each respective frame will be able to be evaluated. In order to more accurately determine the strength, torsion, drift, and overturning characteristics of the structure, an *ETABS* model will be used upon acceptance as an accurate representation of the lateral system in the Cancer Hospital. This *ETABS* model will be checked for accuracy using manual calculation of the distribution of loads on each frame based on relative stiffness.

Once results have obtained from calculation and model output, conclusions will be drawn revealing any concerns or shortcomings of the existing lateral system of the University Hospitals Case Medical Center.

Existing Structural Systems

Foundation

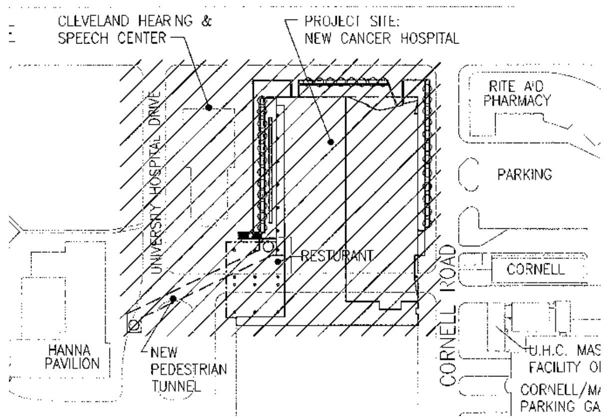


Figure 1

The Cancer Hospital consists of drilled piers transferring load to caissons for the gravity columns with the combined use of grade beams for the lateral force resisting frames. The drilled gravity piers/caissons range 30" to 60" in diameter depending on location. The drilled piers/caissons receiving lateral load are typically 66" in diameter. Along the south side, 36" thick spread footings, typically 48" by 72", have been used to carry gravity load along the existing adjacent Case Medical Center Hospital. The grade beams which carry the lateral load to the drilled piers/caissons are typically 24" by 24" and consist of Grade 60, #7 reinforcement bars. All foundations are made from concrete having a compressive strength of 4000psi with the exception of the caissons and spread footings, which have a strength of 3000psi.

The soil on site has been classified as hard shale (see Figure 1). Thus, giving the caissons used in the foundation an end bearing capacity of 50kpf with a skin friction capacity of 10psi below the first 5' of shale. The typical minimum penetration depth for the gravity piers/caissons is 3'-0" and for the lateral, 16'-6".

Floor System

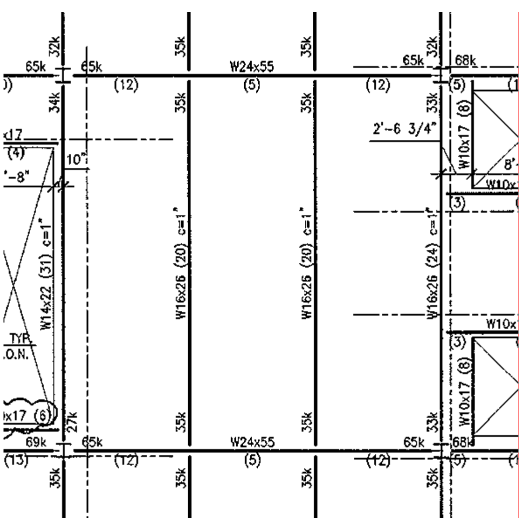


Figure 2

Being a primarily steel structure, the Cancer Hospital has a fairly typical composite steel beam and girder framing system. The typical composite floor slab is 5-1/4" thick using 3000psi lightweight composite concrete, an 18 gauge 2" galvanized steel deck, and 3-1/2" metal studs. This composite floor slab is used on all but the 2nd and 8th floors. The second floor requiring a thicker slab with normal weight concrete due the vibration requirements of the surgery and imaging rooms and the 8th due to the increased load from the mechanical system. The slab used on these floors consists of 6-1/2" thick 3000psi normal weight concrete, an 18 gauge 2" galvanized steel deck, and 3-1/2" metal studs. Both decks are reinforced with 6x6 Welded Wire Fabric; W4.5xW4.5 for the first floor, W3.5xW3.5 for the second and eighth floors, and W2.1x2.1 for the remaining floors.

Framing

Bay sizes conform to the universal grid, having a typical size of 31'-6" by 31'-6". Infill beams are typically W16x26 around the interior and W14x22 around the exterior framing into W24x68 girders (see Figure 2). For the larger breaks in the slab, such as the elevator shafts, HSS 8x4x1/4 tubes have been used. On the 4th and roof level, moment connections are utilized in conjunction with cantilevered beams in order to support the curved exterior façade. Smaller breaks used for mechanical, plumbing, etc., consist typically of W10x17. Columns consist of a typical W14 member decreasing in size with elevation and spliced every other floor starting with the second. All steel members conform to ASTM A-992, Grade 50 unless otherwise noted.

Ground Level

At the ground level, a 6" thick slab-on-grade is used with Grade 60 #5 reinforcement bars spaced @ 18"oc EW. The slab rests on a 10 mils min. vapor barrier on compacted granular material over a 2000psi mud slab. In the northeastern and southeastern section of the building special research equipment has been placed requiring a 12" thick slab-on-grade with Grade 60 #5 reinforcement bars placed @ 12"oc EW.

Machine Room

A 31'-0" by 63'-0" machine room is located on the 8th floor. Framing is similar to the rest of the structure however with shorter spans and larger members to account for the additional weight. Beams range from W21 beams to W40 beams depending on specific equipment.

Roof System

The roof of the Cancer Center is a sloped deck with a 63'-0' by 63'-0" elevator penthouse perched at the southern corner. The roof slopes downward along the east and west sides of the building and allows drainage to the center third. The roof system consists of a 3"x20ga type 'N' galvanized steel deck. The roof deck rests typically rests on W14x22 beams framing into W21x44 girders with W18x35 beams being used to support mechanical equipment spaced uniformly across the building's center. Roof decks lower than the top of the 8th level consist of 1.5"x20ga. type 'B' galvanized steel deck (see Figure 3).

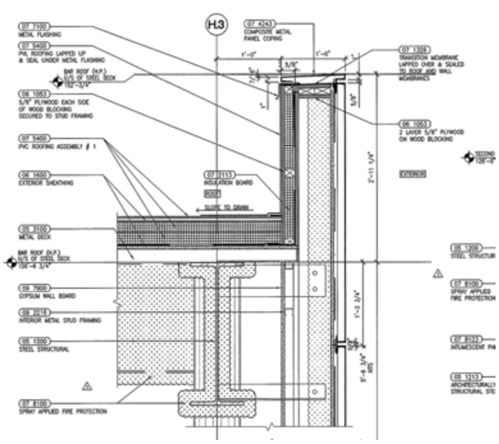


Figure 3

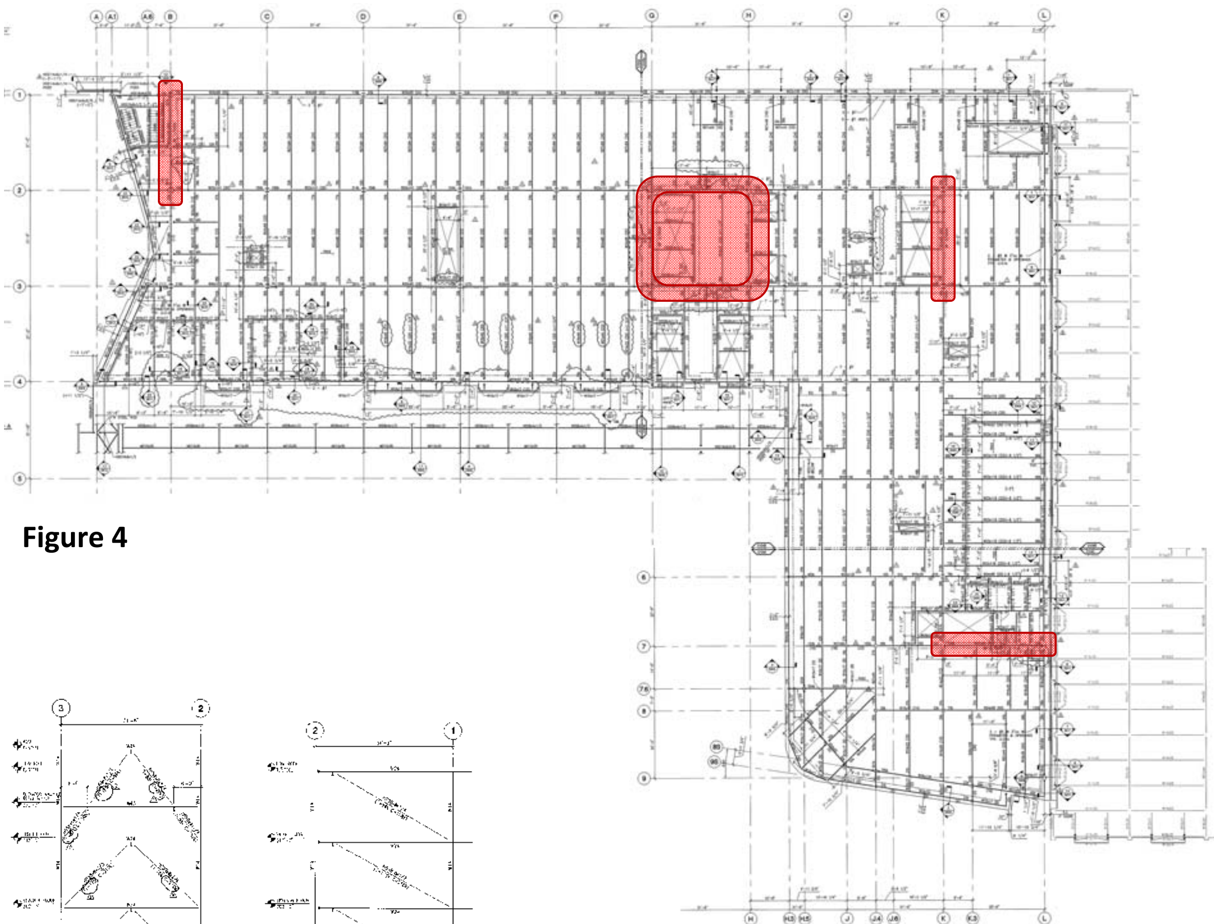


Figure 4

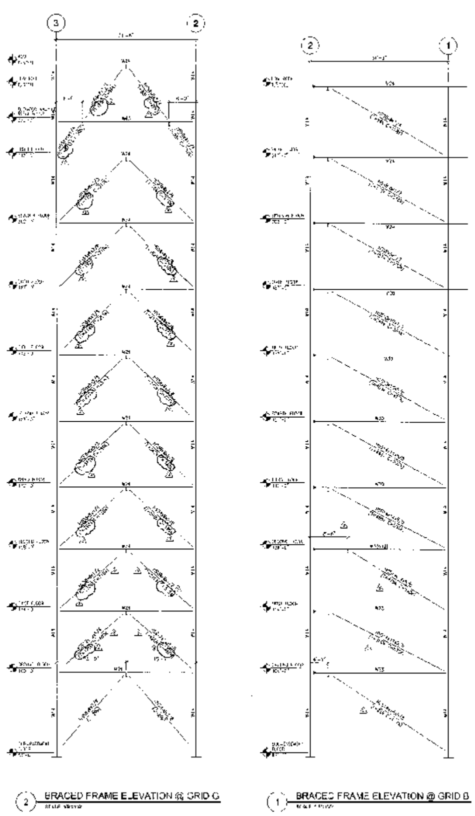


Figure 5

Lateral System

Lateral forces are resisted by a series of concentrically braced frames located at the center of the building near the main elevator core and along isolated points of the exterior bays (see Figure 4). This system consists of four chevron braces and two diagonal braces, which are used both in the north/south direction as well as the east/ west direction. Each brace typically consists of a 31'-6" wide W24 beam, a 15'-0' tall W14 column, and two HSS8 size diagonal members (see Figure 5). Structural brace members beyond the 8th floor increase in size due to increased lateral loads.

Code and Design Requirements

Codes

IBC 2006 *International Building Code*

ASCE-7-05 *Design Code for Minimum Design Loads*

LRFD Specifications for Structural Steel Design – Unified Version, 2005

References

ETABS V9.2.0

Lateral Loads

Seismic

All tables, figures, and equations used in calculation of seismic loads were done so in accordance with Chapter 12 of ASCE 7-05. Although the structure falls under design category A allowing simplified procedure, the Equivalent Lateral Force Procedure was utilized in order to gain greater accuracy in the development of results. Due to the complexity and diversity of loads on each floor of the Cancer Center, a Load Key Diagram was obtained from the structural consultant in order to accurately calculate effective story weight to be used in the Equivalent Lateral Force Procedure (see Appendix D).

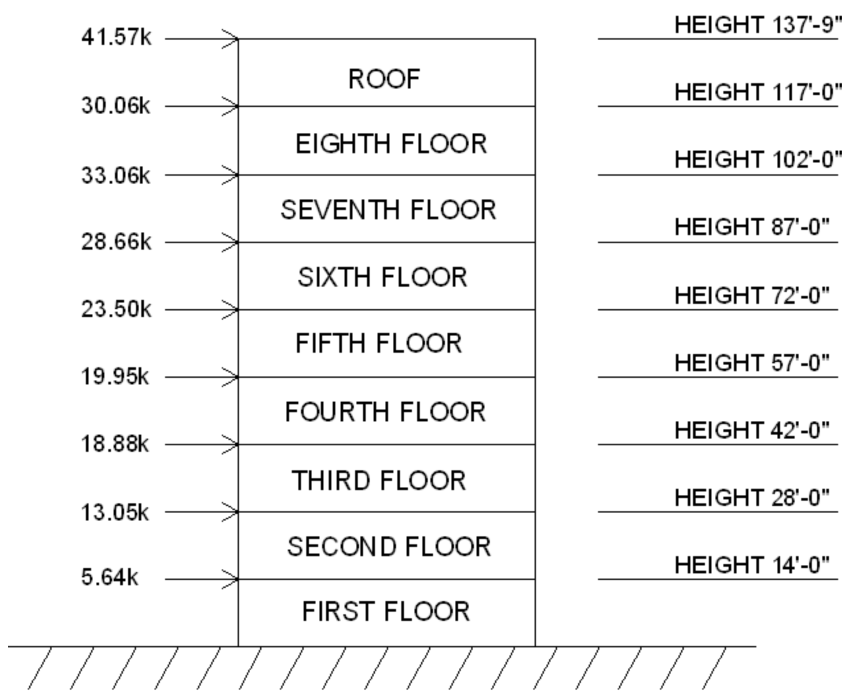


Figure 6

Superimposed line and area dead loads from the diagram can be applied to each respective zoned area on each of the 9 levels. The penthouse level weight has been neglected due to its small amount of contribution to the period. After calculation, these loads were determined to include self-weight. The dead load distribution is shown in the following Tables 1 through 3: (see Appendix A for calculations).

GRAVITY LOAD ROOF – LEVEL 5				
Level	**Description**	**Load**	**Area / Dist**	**Total(lb)**
Roof	Roof Load	25psf -41psf	28200 ft^2	800747
	Building Envelope	300plf - 500plf	1921 ft	713100
	Ceiling Partition	5psf	26791 ft^2	133955
	Suspended Mechanical Equipment	10psf	26791 ft^2	267910
	Interior Shafts	225plf	812ft	182700
8th	Floor Load	30psf -70psf	28315 ft^2	883095
	Building Envelope	300plf	814 ft	244200
	Ceiling Partition	5psf	26791 ft^2	133955
	Suspended Mechanical Equipment	10psf	26791 ft^2	267910
	Interior Shafts	225plf	812ft	182700
7th	Floor Load	47psf	28516 ft^2	1340252
	Building Envelope	300plf	814 ft	244200
	Ceiling Partition	5psf	28516 ft^2	142580
	Suspended Mechanical Equipment	10psf	28516 ft^2	285160
	Interior Shafts	225plf	812ft	182700
6th	Floor Load	47psf	28518 ft^2	1340346
	Building Envelope	300plf	814 ft	244200
	Ceiling Partition	5psf	28518 ft^2	142590
	Suspended Mechanical Equipment	10psf	28518 ft^2	285180
	Interior Shafts	225plf	812ft	182700
5th	Floor Load	47psf	28188 ft^2	1324836
	Building Envelope	300plf	814 ft	244200
	Ceiling Partition	5psf	28188 ft^2	140940
	Suspended Mechanical Equipment	10psf	28188 ft^2	281880
	Interior Shafts	225plf	812ft	182700

Table 1

GRAVITY LOAD LEVEL 4 – LEVEL 1				
Level	**Description**	**Load**	**Area / Dist**	**Total(lb)**
4th	Floor Load	47psf	28062 ft^2	1318914
	Building Envelope	300plf - 360 plf	1289 ft	409740
	Ceiling Partition	5psf	28062 ft^2	140310
	Suspended Mechanical Equipment	10psf	28062 ft^2	280620
	Interior Shafts	225plf	812ft	182700
3rd	Floor Load	47psf	40492 ft^2	1903124
	Building Envelope	300plf	1006 ft	301800
	Ceiling Partition	5psf	40492 ft^2	202460
	Suspended Mechanical Equipment	10psf	40492 ft^2	404920
	Interior Shafts	225plf	812ft	182700
2nd	Floor Load	47psf	41393 ft^2	1945471
	Building Envelope	300plf - 560plf	1006 ft	357180
	Ceiling Partition	5psf	41393 ft^2	206965
	Suspended Mechanical Equipment	10psf	41393 ft^2	413930
	Interior Shafts	225plf	812ft	182700
1st	Floor Load	47psf	41662 ft^2	1958114
	Building Envelope	300plf	336 ft	100800
	Ceiling Partition	5psf	41662 ft^2	208310
	Suspended Mechanical Equipment	10psf	41662 ft^2	416620

Table 2

Table 3

GRAVITY LOAD	
Level	**Load(lb)**
Roof	2098412
8	1711860
7	2194892
6	2195016
5	2174556
3	2995004
2	3106246
1	2683844
Total Wt.	19159830

Table 4

SEISMIC FACTORS			
S_s	19.20%	S_{ds}	0.1536
S_1	5.10%	S_{d1}	0.058
Site Class	C	Occupancy Cat.	IV
F_a	1.2	Seismic Des. Cat.	A
F_v	1.7	C_s	0.01
S_{ms}	0.2304	T_a	1.31
S_{m1}	0.0867	V	214.9k
R	7	I	1.5

The maximum lateral story force was determined to be 41.57k at the roof level (see Table 4). This value is reasonable due to the low amount of seismic activity in Cleveland, Ohio. Base shear was found to be 214.9 kips, placing it within 1% error when compared to value of 213.6k calculated by the consultant engineer (see Figure 5). For further calculation details see Appendix A.

SEISMIC FORCES					
Level	wx	hi	Σwihi	Cvx	Story Force(k)
Roof	2098412	132	1431970472	0.19	41.57
8	1711860	117	1431970472	0.14	30.06
7	2194892	102	1431970472	0.16	33.60
6	2195016	87	1431970472	0.13	28.66
5	2174556	72	1431970472	0.11	23.50
4	2332284	57	1431970472	0.09	19.95
3	2995004	42	1431970472	0.09	18.88
2	3106246	28	1431970472	0.06	13.05
1	2683844	14	1431970472	0.03	5.64

Table 5

Wind

As in the seismic analysis, all tables, figures, and equations for calculation of wind loads were done so in accordance with chapter 6 of ASCE 7-05. Method 2 of the Main Wind-Force Resisting Systems, also known as the Analytical Method, was used in determination of lateral wind pressures. For the approximate calculations of this report, Case I of ASCE7-05 Figure 6-9 has been assumed to be the most conservative and were analyzed in the both the north/south direction as well as east/west direction accordingly (see Figure 7).

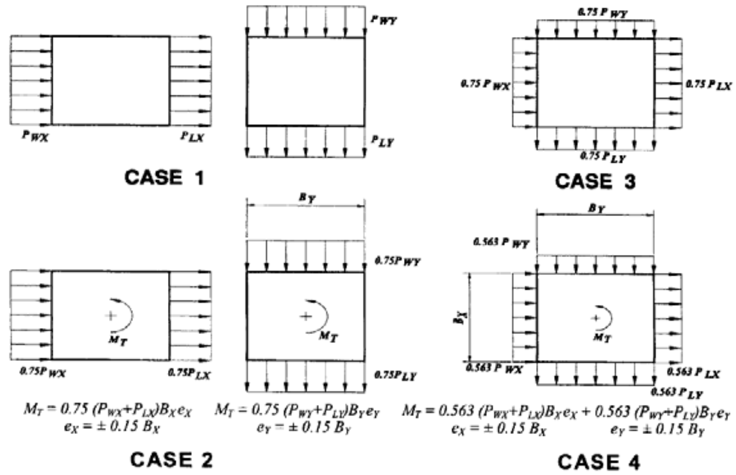


Figure 7

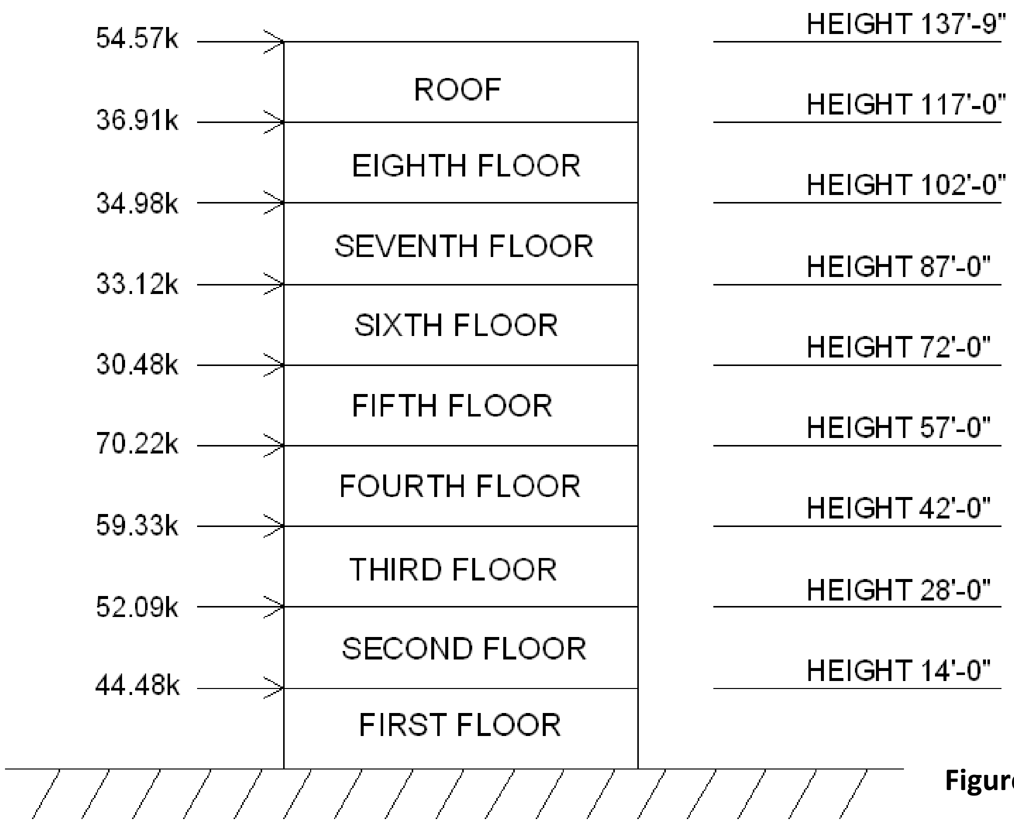


Figure 8

East – West Direction

Different gust factors resulted due to flexibility (see Table 6). A conservative approach was taken in east/west direction in order to account for the vertical "L" shape caused by the lower 4 story, southern wing of the Cancer Hospital. Since the code is unclear about applying wind pressures to non-uniform shapes, a rectangular shape was used in calculation. This will cause the lateral forces to be larger than in actuality (see Appendix D for North Elevation).

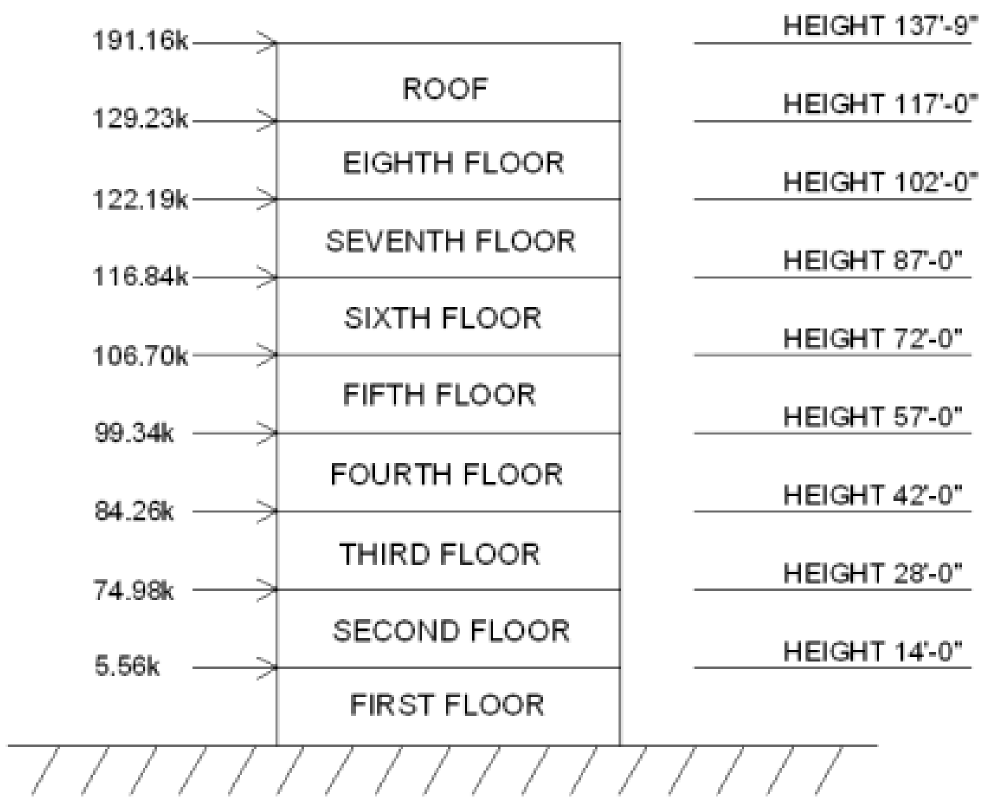


Figure 9

North – South Direction

For this analysis internal pressures and roof top uplift pressures have been ignored, however, overturning moment has been determined. The maximum point load was calculated to be 191k in the north/south direction and 54.57K in the east /west direction at the roof level (see Figures 8 – 9). The wind pressure and story shear for each are included in the following Tables 7 through 9: (see Appendix A for calculations).

Wind Factors			
V	90mph	n	0.4
Kd	0.85	G	.89/.84
I	1.15	qz	20.27
Exp. Cat.	B	qi	22.7
Kzt	1	qh	22.7
Kh	1.2	Cp	0.8

Table 6

Table 7

WIND ANALYSIS				
	Story	Tributary Height (ft)	Kz	qz (psf)
Windward	Penthouse	16.33	1.12	22.7
	High Roof	7.17	1.08	21.9
	Low Roof	13.58	1.06	21.5
	8	15	1.03	20.9
	7	15	0.99	20.1
	6	15	0.95	19.3
	5	15	0.89	18.0
	4	15	0.84	17.0
	3	14	0.77	15.6
	2	14	0.68	13.8
	1	14	0.57	11.6
Leeward		154.1	1.12	22.7
Side		154.1	1.12	22.7

Table 8

NORTH - SOUTH DIRECTION					
Story	Tributary Height (ft)	External Pressure qGC_p (psf)	Forces (k)	Story Shear (k)	Overturn Moment (ft-k)
Roof	20.75	22.63	191.16	95.50	95.50
8	15	21.04	129.23	224.50	6363.25
7	15	20.27	122.19	346.50	12078.25
6	15	19.14	116.84	462.50	19578.25
5	15	17.68	106.70	568.50	28743.25
4	15	16.28	99.34	667.50	39445.75
3	14	14.29	84.26	751.50	51097.25
2	14	12.73	74.98	825.50	63473.25
1	14	10.70	63.56	888.50	76808.25

Table 9

EAST - WEST DIRECTION					
Story	Tributary Height (ft)	External Pressure qGC_p (psf)	Forces (k)	Story Shear (k)	Overturn Moment (ft-k)
Roof	20.75	21.36	54.57	27.29	27.29
8	15	19.86	36.91	64.20	1818.43
7	15	19.13	34.98	99.18	3452.98
6	15	18.06	33.12	132.30	5598.28
5	15	16.69	30.48	162.78	8220.58
4	15	15.37	70.22	233.00	11598.13
3	14	13.49	59.33	292.33	15787.50
2	14	12.01	52.09	344.42	20626.67
1	14	10.10	44.48	388.90	26141.83

Controlling Load

Wind loads were compared to seismic loads in both the north/south and east/west directions. Wind was found to control over seismic in both orientations (see Table 10 and Charts 1 - 2). This is as expected due to the low seismic region in which the Cancer Hospital is located. The north/south wind load is significantly larger than the east/west direction due to the increase in pressure area of on the north and south face. A difference in wind force from levels 1-4 to levels 5-R can be noticed in the east/west direction. This is caused by the decrease in the pressure zone size as the transition is made from the lower "L" shape to the upper tower levels.

LATERAL LOADS				
Story	Wind N/S (k)	Wind E/W (k)	Seismic N/S (k)	Seismic E/W (k)
Roof	191.00	54.57	41.57	41.57
8	129.00	36.91	30.05	30.05
7	122.00	34.98	33.06	33.06
6	116.00	33.12	28.66	28.66
5	106.00	30.48	23.5	23.50
4	99.00	70.22	19.95	19.95
3	84.00	59.33	18.88	18.88
2	74.00	52.09	13.05	13.05
1	63.00	44.48	5.6	5.60
Ground	0.00	0.00	0.00	0.00

Table 10

Chart 1

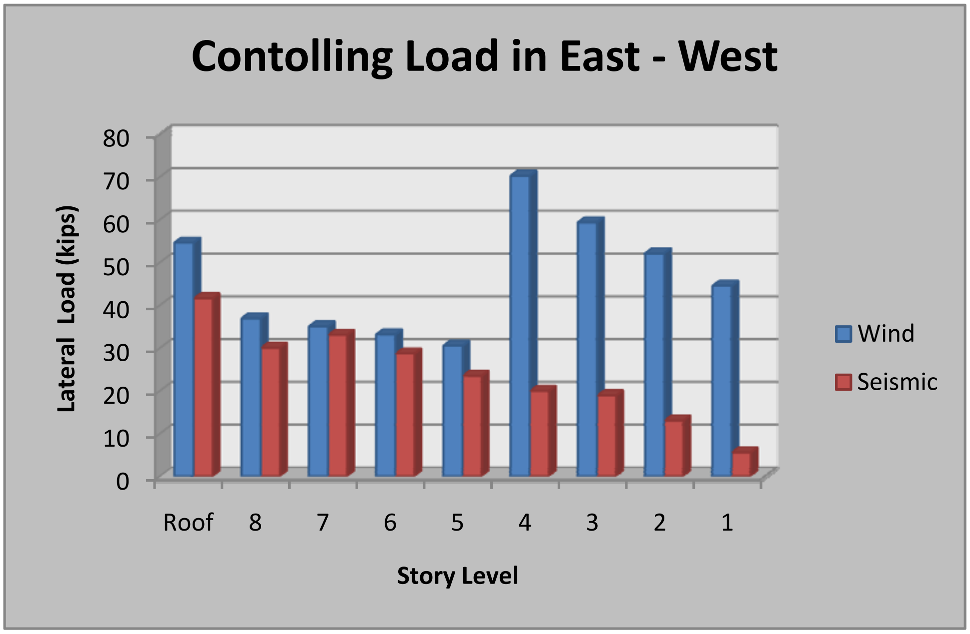
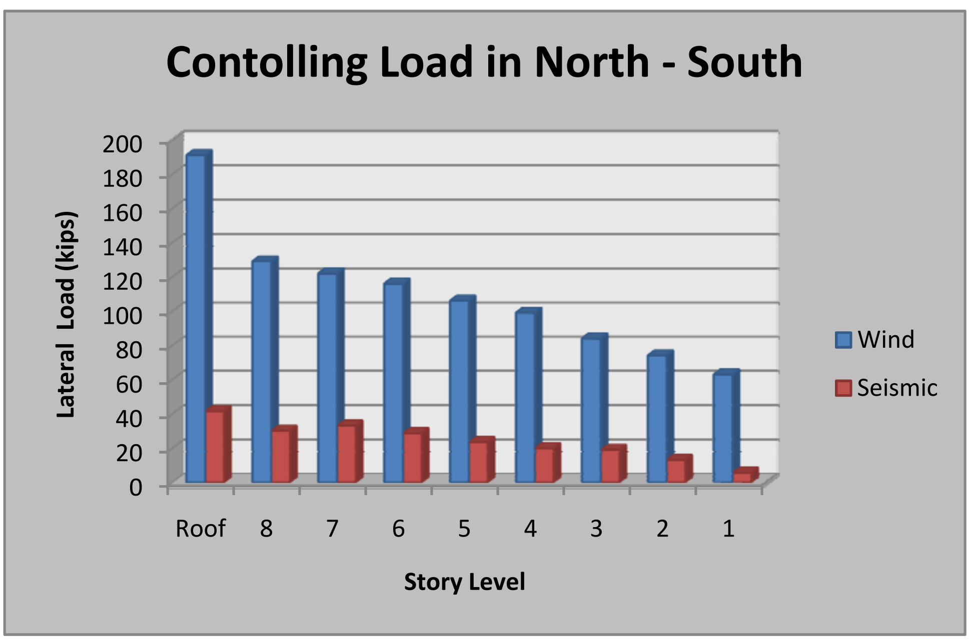


Chart 2



OVERALL STORY SHEAR					
Story	Tributary Height (ft)	Wind N/S (k)	Story Shear N/S (k)	Wind E/W (k)	Story Shear E/W (k)
Roof	20.75	191.00	95.50	54.57	27.29
8	15	129.00	224.50	36.91	64.20
7	15	122.00	346.50	34.98	99.18
6	15	116.00	462.50	33.12	132.30
5	15	106.00	568.50	30.48	162.78
4	15	99.00	667.50	70.22	233.00
3	14	84.00	751.50	59.33	292.33
2	14	74.00	825.50	52.09	344.42
1	14	63.00	814.50	44.48	336.81
Base Shear	0	0.00	984.00	0.00	344.42

Table 11

Load Path

From the external face, lateral load will travel immediately into the rigid floor and roof diaphragms which in turn distribute it to the six braced frames in the Cancer Hospital through relative stiffness. 3 braced frames located at gridline B,G, and K resist the loads in the north/south direction at all levels. The remaining 3 braced frames located at gridline 2,3, and 7 resist loads in the east/west direction. Frames 2 and 3 provide lateral resistance in all levels however frame 7 on only resists load up to the fourth level. This is due to its location in the lower "L" shape which discontinues at level 4 (see Figure 10).

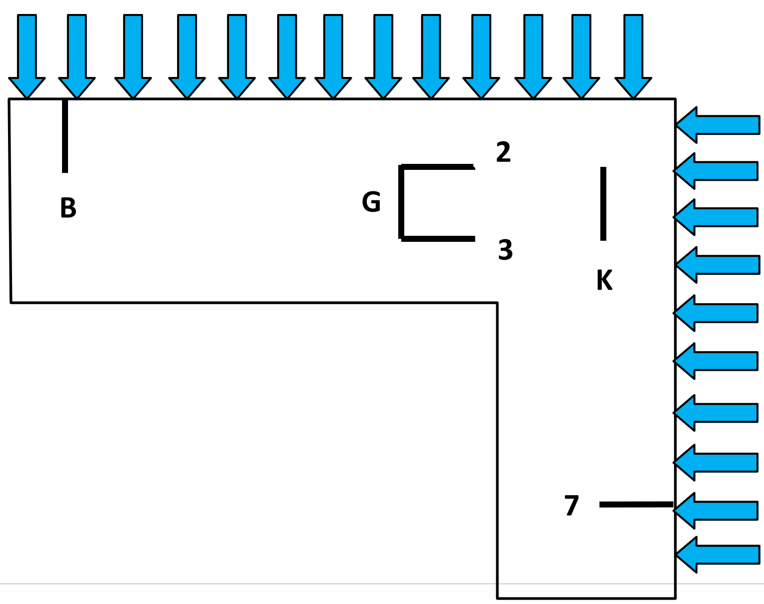


Figure 10

Once the lateral load has reached the frame at each story, it is then distributed to the members. The resulting horizontal force is transferred as shear and is primarily taken by either a “chevron” or single brace depending on the frame. In the case of the single brace, this wind load is the only load calculated in the strength design however, in the case of the “chevron” braces, the wind load is taken into account in conjunction with the gravity and live loads from the center point of the above beam (see Figure 11). On frame B and 7, the braces have been stepped back 4’-0” from the beam and column corner joint forcing a moment into the beam and a moment connection. This step back has presumably been constructed due to the location of these frames at the perimeter of the structure. The effect of this special connection has been assumed to be negligible due to the small amount of eccentricity that it endues on the load path. The remaining amount of shear which has not been absorbed by the brace is transferred into the columns. This remaining shear is minimal and will not be taken into account for the approximate methods of calculation in this report. In addition to the shear force in the braces, the overall over-turning of the frame causes a tensile force in the columns of the receiving side and a compressive force in the columns of the opposite side (see Figure 11). The beam has been assumed to take no axial load since it contributes minimally in comparison to the diaphragm.

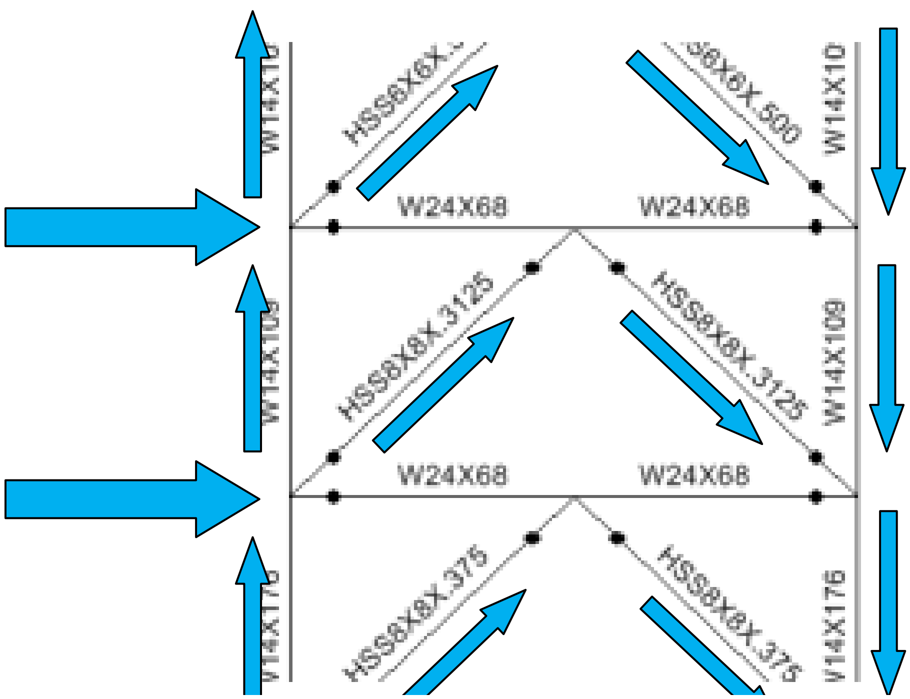


Figure 11

Frame Contribution

Stiffness

Stiffness values have been calculated through use of a lateral model created using the structural software *ETABS*. All lateral members in the existing design were accurately modeled and rigid diaphragms were assigned to each floor. The roof diaphragm has been neglected for the purposes of this report due to its limited contribution to the frame. Once a working model was completed, a unit load of 1000k was applied on the roof level in the north/south direction at the center of mass. After this load had been applied, section cuts were made directly beneath the roof level to determine the how much each of the 3 frames resisting in that direction took from the 1000 kip load. This load was then divided by the deflection the frames experienced at the roof level in accordance with the following equation;

$$P = K \cdot d$$

After the values were found at the roof level in the north/south direction and determined to be acceptable, this process was repeated for each level and then also in the east/west direction (see Table 12)

Table 12

RELATIVE STIFFNESS (k/in)						
Level	EAST-WEST DIRECTION			NORTH - SOUTH DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	46.125	49.26937984	0	39.0918138	57.3964687	63.90529695
8th	82.93278689	80.35255354	0	66.7638191	111.5603015	70.76130653
7th	122.8762136	119.3737864	0	89.77852349	133.590604	111.0838926
6th	160.9606557	158.9016393	0	122.8125	162.9208333	128.8125
5th	242.6521739	235.1884058	0	218.6346154	191.9122807	190.4912281
4th	251.4055944	268.8811189	164.8741259	322.1944444	369.0092593	229.1296296
3rd	391.3483146	420.4719101	278.6853933	433.5205479	495.9863014	437.8493151
2nd	723.6530612	875.3061224	407.9183673	1057.851064	984.787234	54.14893617
1ST	1760.52381	1744.904762	1013.909091	2029.75	2261.3125	1844.6875

Rigidity

Of the 6 braced frames in the Cancer Hospital, only the frame located at gridline 7 does not rise to the full height of the structure. This will cause a different rigidity in levels 1 through 4 located in the lower "L" shape as opposed to levels 5 through 9 located in the upper tower. In order perform calculations, a theoretical square footprint has been assumed and a point in the extreme bottom left of this footprint has been selected. This point will serve as the zero point in which the distance to the frames will be based on (see Figure 12). Once these distances have been calculated in both the north/south as well as the east/west direction, the center of rigidity was calculated for each level using the following equations (see Table 13 for calculated values);

$$x = \Sigma K_i x_i / \Sigma K_i$$

$$y = \Sigma K_i y_i / \Sigma K_i$$

.

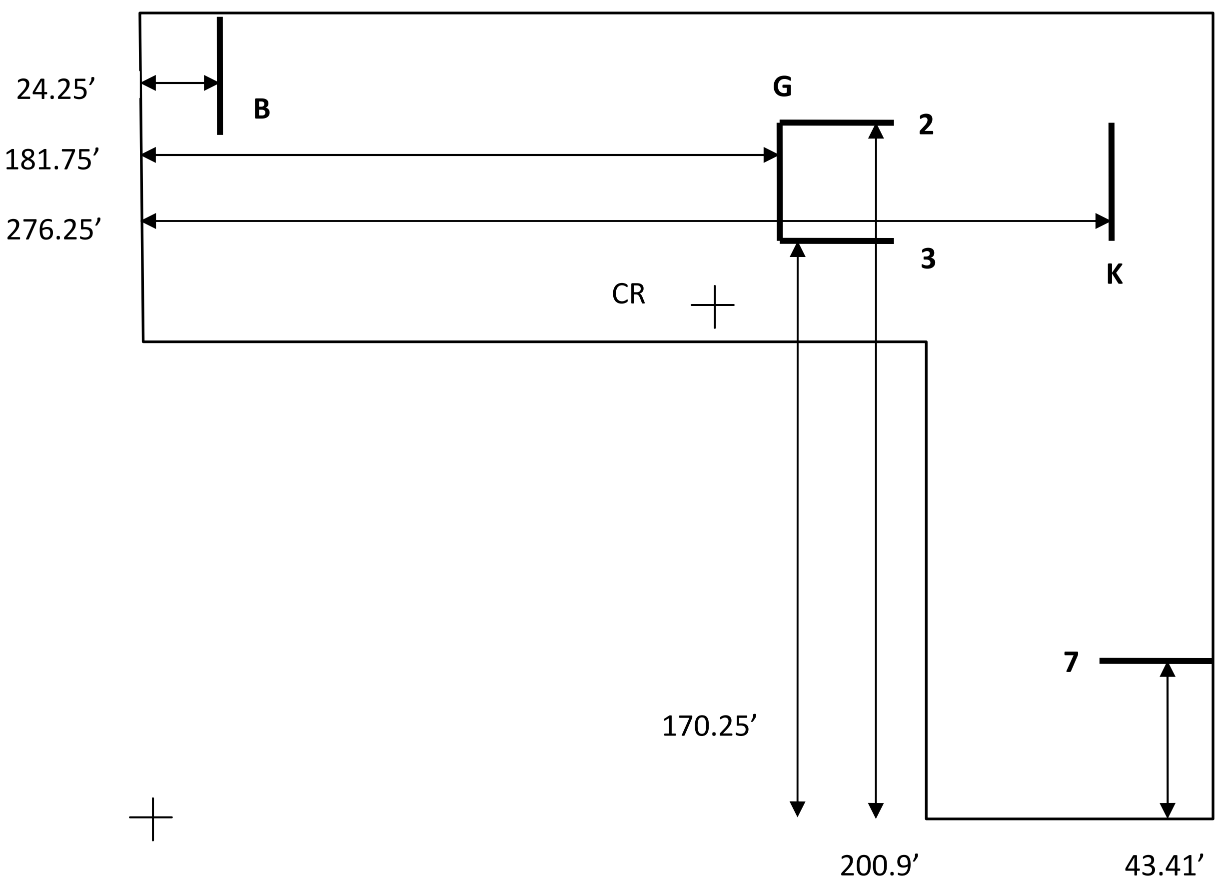


Figure 12

RIGIDITY		
Level	NORTH - SOUTH	EAST - WEST
Roof	105.5957495	151.547828
8th	101.7555933	144.959525
7th	101.8614401	146.3818148
6th	102.4930708	142.1965655
5th	101.7699418	133.8245927
4th	146.6987028	133.4760194
3rd	149.3782366	139.8499726
2nd	144.1657082	107.0494225
1ST	141.5963084	137.5052144

Table 13

In addition to the center of rigidity, the center of mass was also calculated for future use. In order to find this figure, the centroids of both legs in the "L" shape bottoms were combined. For this approximate calculation, mass has been assumed to be uniformly distributed throughout the structure (see Table 14).

CENTER OF MASS		
Level	NORTH - SOUTH (IN)	EAST - WEST (IN)
Roof	154.6	185.1
8th	154.6	185.1
7th	154.6	185.1
6th	154.6	185.1
5th	154.6	185.1
4th	186.2	152.2
3rd	186.2	152.2
2nd	186.2	152.2
1ST	186.2	152.2

Table 14

Frame Force

Direct Shear

Once relative stiffness values had been found, the external forces were then distributed as direct shear to each frame. Calculations were performed in a spreadsheet using the following equation where ΣK_i is the sum of the relative stiffness of each in-plane frame;

$$F_{direct\ shear} = F_i \cdot K_i / \Sigma K_i$$

Torsion

In addition to the direct shear forces, each frame will take a torsional force at every level caused by the non-symmetrical nature of both the building and the location of the braced frames. This torsion was found using a spreadsheet utilizing the following equation;

$$F_{torsion} = M_i \cdot K_i d_i / \Sigma K_i d_i$$

In this equation, the moment is caused by the force at the center of pressure multiplied by the eccentricity of the center of rigidity. In the case of seismic lateral loads, the eccentricity is from the center of mass to the center of rigidity. Each distance multiplied by the relative stiffness in the previous equation is measured from the location of the frame to the location of the center of mass.

Though analysis of obtained data, the Cancer Center appears to have significant torsional forces concentrated primarily at the braces closest to the outskirts of the building in both directions. The frames which are most affected include frame B, K, and 7. This torsional force is most likely due to the "L" shaped base, in addition to the off-symmetrical location of the braced frames in the tower (see Figure 13). Investigation will be conducted in a future report to evaluate the effect of using a different structural configuration or material to reduce this torsional value.

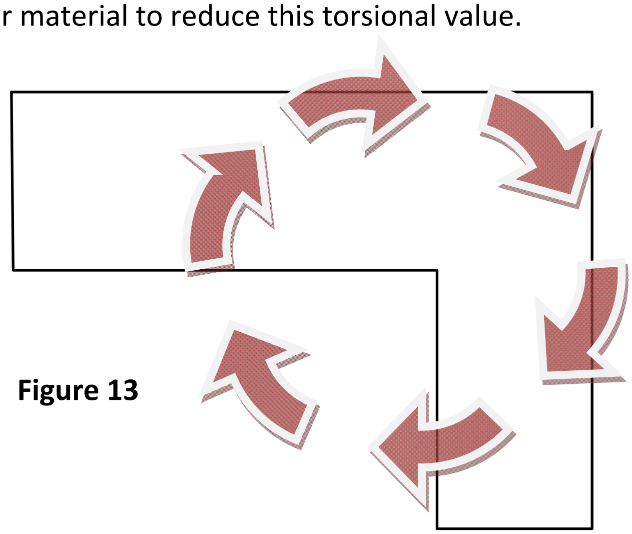


Figure 13

After both direct shear and torsion for each frame at every level were found, the forces were then added together to determine the total force at each level (see Tables 15 - 18). Detailed spreadsheets used in calculating direct shear, torsion, and total force are provided in Appendix B.

TOTAL WIND FORCE NORTH – SOUTH						
Level	EAST – WEST DIRECTION			NORTH – SOUTH DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	1.240278687	0.449942102	0	43.90534703	69.55184905	80.84165279
8th	1.115356341	0.367014903	0	32.39268463	59.07335952	39.35386719
7th	1.073500892	0.35419318	0	30.89918768	49.82558346	43.35283322
6th	1.073475697	0.35991297	0	32.29008863	46.41741085	38.40071999
5th	2.546582094	0.838274081	0	33.18909731	35.77230379	39.46593351
4th	1.539867382	0.559326408	-2.324370176	29.97269392	41.77878042	28.72068046
3rd	1.249788276	0.456043848	-2.048479354	23.23058438	31.79960367	30.84095022
2nd	1.654922722	0.679835622	-2.147162838	31.29110835	36.55193919	2.255153606
1ST	0.984812681	0.331497374	-1.305434699	18.05023047	24.27955935	21.84951896

Table 15

TOTAL WIND FORCE EAST – WEST						
Level	EAST – WEST DIRECTION			NORTH – SOUTH DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	25.2915617	27.78602982	0	2.226500091	-1.210760669	-4.34393277
8th	17.76544566	17.84090249	0	1.901379267	-1.176720982	-2.405097947
7th	16.79894642	16.92653353	0	1.661416972	-0.915626458	-2.453390145
6th	15.72352767	16.13863801	0	1.735045089	-0.852473823	-2.171878593
5th	14.63927911	14.72513341	0	1.815632604	-0.590266387	-1.887962545
4th	23.05226208	26.57163859	20.9948538	8.367937613	-3.549554801	-7.102168658
3rd	19.08961658	22.07367999	18.77487203	5.872138304	-2.488241371	-7.078163755
2nd	15.8654339	21.52010621	14.37170995	10.25719327	-3.536573248	-0.626618601
1ST	15.59301161	16.59091608	12.28118757	4.814865074	-1.9867279	-5.222443121

Table 16

TOTAL SEISMIC FORCE NORTH – SOUTH						
Level	EAST – WEST DIRECTION			NORTH – SOUTH DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	20.36932607	21.56785884	0	-0.549344044	0.298730804	1.071777902
8th	15.5212867	14.87275517	0	-0.501322498	0.31025725	0.634134247
7th	17.05829206	16.38646407	0	-0.508500929	0.28024085	0.750895885
6th	14.68712219	14.32656251	0	-0.486185135	0.238875694	0.608592303
5th	12.14270545	11.63532225	0	-0.453392766	0.147399044	0.471454719
4th	8.268199527	8.173406641	3.369772703	-2.922586248	1.239717661	2.480503729
3rd	7.637410594	7.594189693	3.412078417	-2.297061468	0.973349584	2.768834177
2nd	5.604178091	6.060909504	1.486765488	-3.159787712	1.088180124	0.193033484
1ST	2.450163227	2.252580484	0.90022316	-0.745150204	0.307466705	0.808227125

Table 17

TOTAL SEISMIC FORCE EAST – WEST						
Level	EAST – WEST DIRECTION			NORTH – SOUTH DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	0.269439127	0.09774578	0	9.582275347	15.17445845	17.63443087
8th	0.2588618	0.085180077	0	7.553154181	13.76904165	9.170873172
7th	0.289302907	0.09545322	0	8.365923304	13.48540003	11.73131247
6th	0.264877166	0.088807532	0	8.004567798	11.50255442	9.514160646
5th	0.209172956	0.068854748	0	8.095005838	7.650980622	7.919474537
4th	0.947959779	0.344327664	-1.430908573	4.061599736	9.238705045	7.447330362
3rd	0.861969864	0.314530118	-1.412821279	3.688846343	7.821763782	8.814512167
2nd	0.898523556	0.369109876	-1.165780349	3.424073616	7.217306244	0.530046036
1ST	0.26866362	0.090434746	-0.356131495	1.107369863	2.371330128	2.491843635

Table 18

ETABS Model

As previously stated, an *ETABS* model has been constructed using the existing design of the Cancer Hospital. Even though the sub-ground level is actually below the surface and not susceptible to direct lateral loads, the lateral members have been modeled in these areas to increase consistency with the existing design. The diaphragms between braces have been modeled as rigid in order to follow the method of stiffness calculation described previously (see Figure 14 - 15).

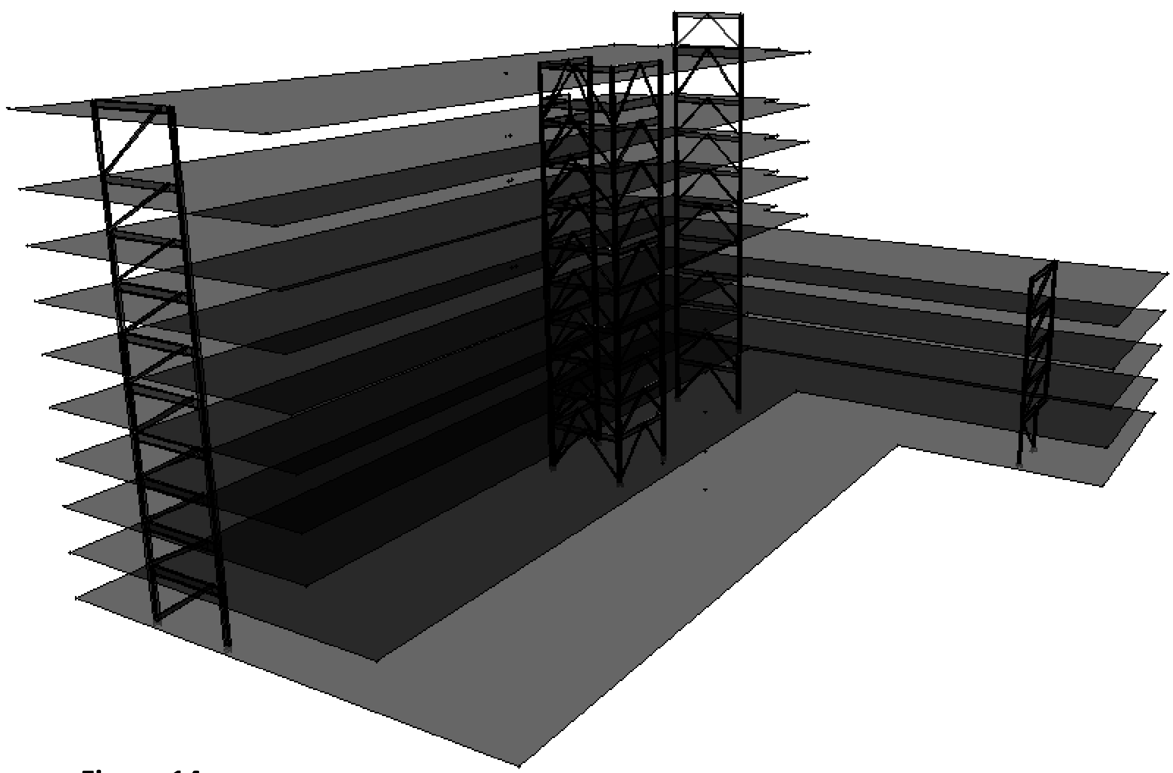


Figure 14

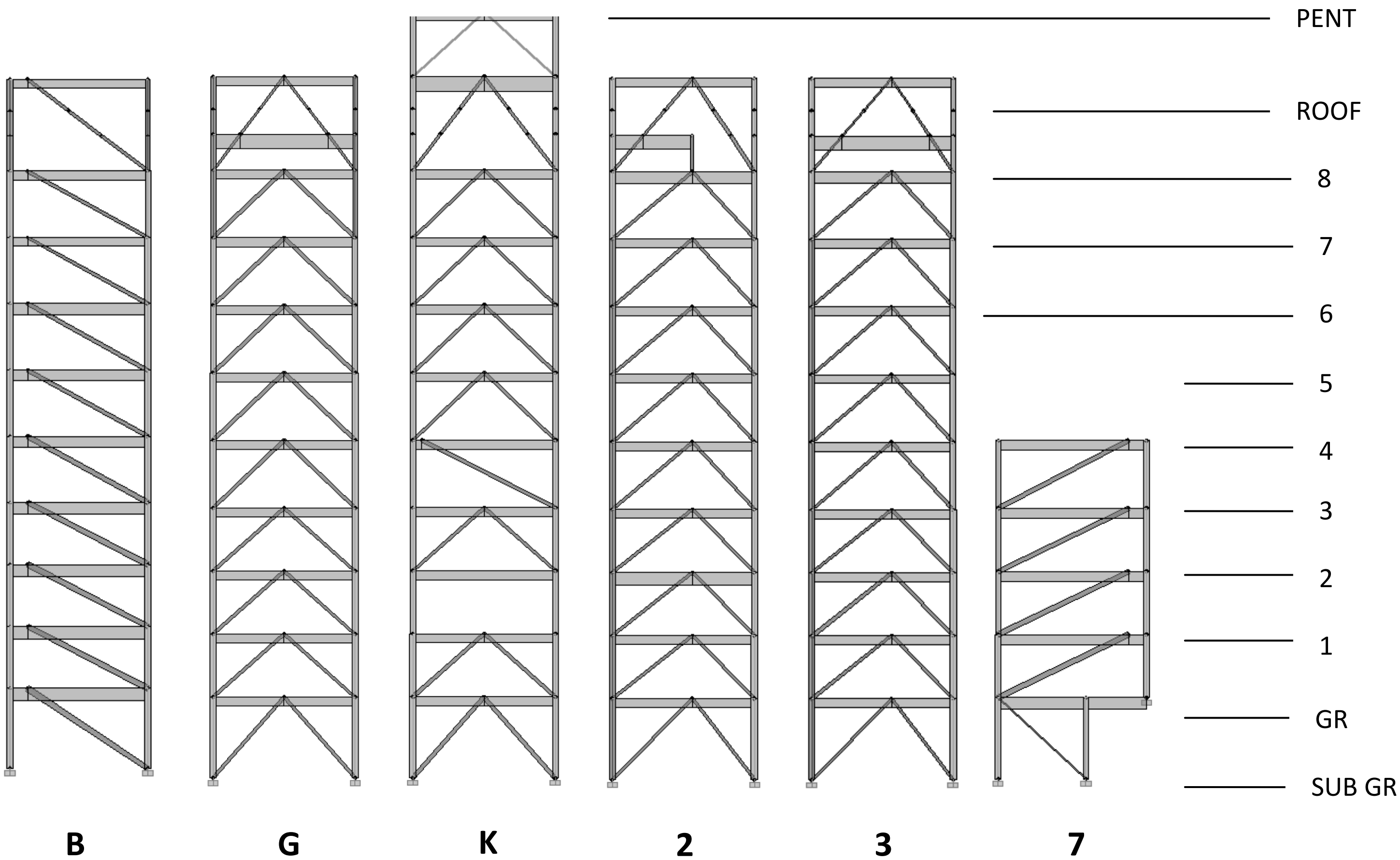


Figure 15

Upon completion of the modeling of all significant lateral members, un-factored wind and seismic loads previously calculated were applied in the proper orientation. Initially, the model produced reasonable values for period:

Mode 1 (x): 2.343sec

Mode 2 (y): 1.978sec

Mode 3 (z): 1.610sec

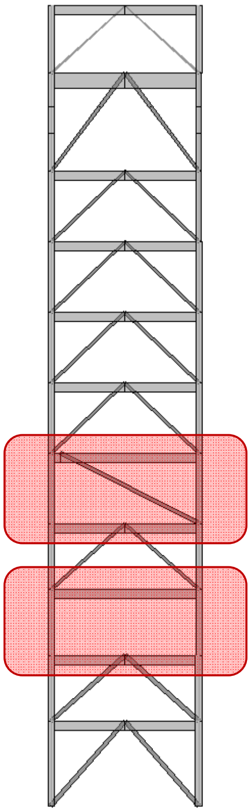
Further investigation was then performed by selecting a representative frame from both the north/south direction and the east/west direction. The frames selected include frame K for analysis in the north/south direction and frame 2 for analysis in the east/west direction. Since wind controls in both directions, seismic loads were not considered. Using the shear cut command in *ETABS*, values for shear were obtained and compared to results of the manual load calculation using spreadsheets (see tables 19 - 20). Any additional *ETABS* data is available upon request.

NORTH - SOUTH DIRECTION USING FRAME K					
Story	Tributary Height (ft)	Forces (k)	Frame Shear (k)	ETABS Frame Shear (k)	% Error
Roof	20.75	80.84	40.42	47.39	17.24%
8	15	39.35	79.77	71.91	9.86%
7	15	43.35	123.13	110.74	10.06%
6	15	38.40	161.53	139.34	13.74%
5	15	39.47	200.99	193.24	3.86%
4	15	28.72	229.71	181.00	21.21%
3	14	30.84	260.56	245.81	5.66%
2	14	2.26	262.81	38.94	85.18%
1	14	21.85	284.66	275.21	3.32%

Table 19

EAST - WEST DIRECTION USING FRAME 2					
Story	Tributary Height (ft)	Forces (k)	Frame Shear (k)	ETABS Frame Shear (k)	% Error
Roof	20.75	25.29	12.65	26.29	107.91%
8	15	17.76	30.41	48.88	60.76%
7	15	16.79	47.20	65.33	38.43%
6	15	15.72	62.92	80.36	27.73%
5	15	14.63	77.55	98.41	26.91%
4	15	23.05	100.60	93.34	7.21%
3	14	19.05	119.65	111.36	6.92%
2	14	15.86	135.51	131.49	2.96%
1	14	15.59	151.10	152.01	0.61%

Table 20



Upon observation of the resulting error between manual calculation and the *ETABS* model, the percentage is well underneath the acceptable limit with the exception of two isolated points. The first area of error is on the 2nd and 4th level of frame K (see figure 16). Immediately, it is seen that both of these areas in the frame have a sizeable decrease in stiffness due to the 4th level having a single brace and the 2nd level having no brace at all. This large decrease in stiffness creates a shear reversal in the modeled frame causing the results from *ETABS* to be varied from the manual calculation. These special locations seem ineffective and future investigation will be conducted in order to find a more efficient approach. The second area of error can be seen in frame 2 from the 5th level to the Roof. In this area the ETABS model has taken a more conservative approach analyzing the lateral loads by creating a pressure zone that is more balanced with the increased size of the zone in the lower “L” shape. In the manual calculations, a less conservative approximation was made based strictly on area.

The center of rigidity from previous manual calculation was also compared to the value used in the ETABS model (see Table 21). Results yielded a sizeable difference in the north/south direction above level 4. This variation has been found to be caused by the consideration of the out-of-plane contribution of members acting in the opposite direction in the model. This consideration provides a more accurate result when compared to the manual calculation.

Figure 16

Upon comparison of results and evaluation of areas of concern, the ETABS model has been accepted as an accurate representation of the existing structural system in the Cancer Hospital. The model will be used to obtain values for deflection and member forces used in strength checks.

RIGIDITY COMPARISON						
Level	NORTH – SOUTH DIRECTION (in)			EAST – WEST DIRECTION (in)		
	Hand Calc	Etabs	%Error	Hand Calc	Etabs	%Error
Roof	105.5957495	155.37425	32.04%	151.547828	174.92875	13.37%
8th	101.7555933	150.3958333	32.34%	144.959525	171.2143333	13.37%
7th	101.8614401	149.1854167	31.72%	146.3818148	168.1620833	13.37%
6th	102.4930708	146.8034167	30.18%	142.1965655	164.4718333	13.37%
5th	101.7699418	143.0035	28.83%	133.8245927	158.8278333	13.37%
4th	146.6987028	138.6399167	5.81%	133.4760194	152.2090833	13.37%
3rd	149.3782366	133.9120833	11.55%	139.8499726	152.5019167	13.37%
2nd	144.1657082	125.7456667	14.65%	107.0494225	154.4185833	13.37%
1ST	141.5963084	158.651	10.75%	137.5052144	154.0863333	13.37%

Table 21

Design Checks

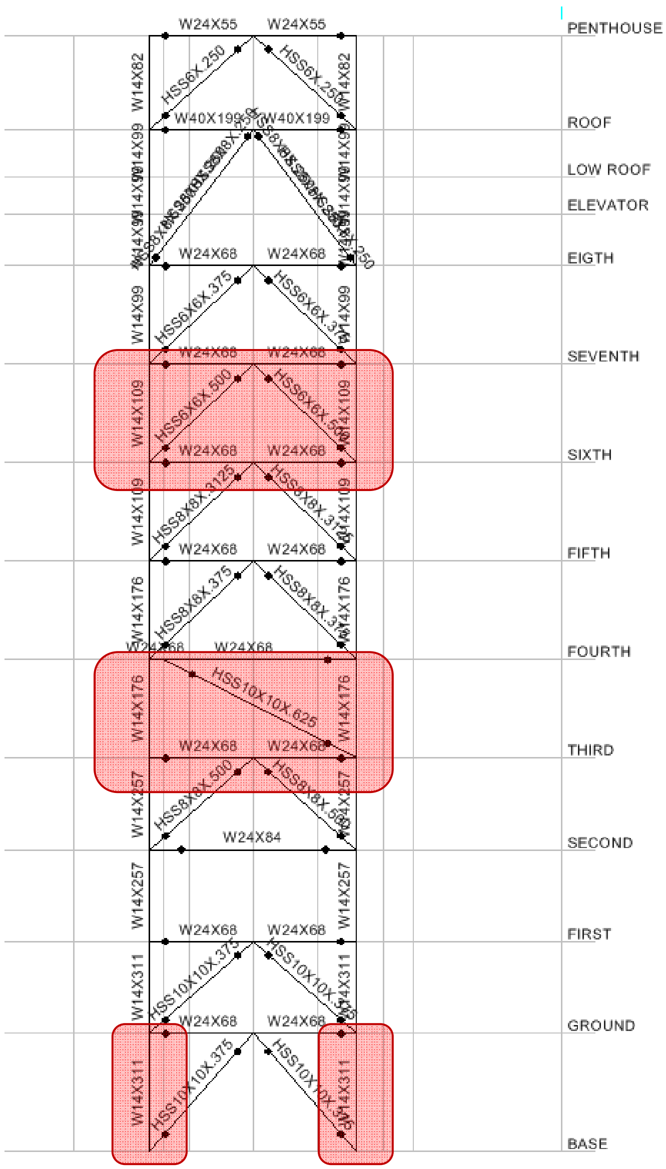


Figure 17

Member Strength

Critical members in the lateral system of the existing Cancer Hospital design have been checked to meet strength requirements. As mention previously, the *ETABS* model has been proven to be an accurate representation of the lateral system utilized in the Cancer Hospital and required service loads have been determined based on program output. Of the lateral system, Frame K is exposed to the highest amount of direct shear, torsion, and also has the most irregularities. For these reasons, Frame K has been chosen for member strength spot checks. Both the “chevron” brace configuration and the single brace configuration have been analyzed in addition to the columns at the base (see Figure 17). Loads on members were determined upon finding the controlling load combination in accordance with ASCE7-05. See appendix C for detailed calculations.

Applicable load combinations considered include:

1.4D

1.2D + 1.6 L

1.2D + 1.6W+1.0L

.9D + 1.6

Chevron Brace @ 6th Level

The chevron brace on the 6th level has been checked for strength (see Figure 18). Not only has lateral loading been considered, but also the dead and live load contribution from the beam on the 7th floor above. The 1.2D + 1.6W +1.0L load combination controls yielding an axial load of 152.3k. Using the AISC Steel Manual as a design aid, the HSS6x6x.5 has been found to have an allowable axial force of 159.92k, making this member adequate to carry the load.

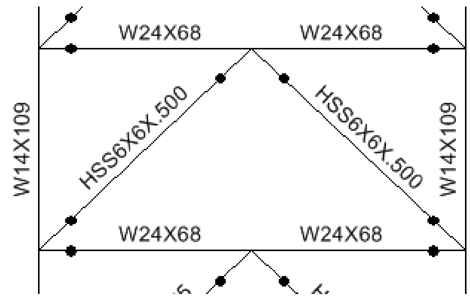


Figure 18

Single Brace @ 3th Level

The single brace on the 3rd level has been analyzed for strength (see Figure 19). This brace is susceptible to lateral loading only. The 1.2D + 1.6W +1.0L load combination also controls in this frame, producing a required axial load of 304.94k. With the use of the AISC Steel Manual for design, the HSS10x10x.625 member was found to have an allowable axial force of 388.58k thus making it acceptable to carry the required load.

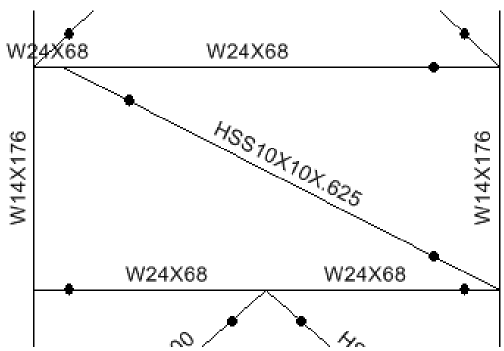


Figure 19

Columns @ Base

The columns at the base level have also been chosen for a strength check (see Figure 20). In addition to the dead and live load from the floors above, these two members are exposed to an axial load at the receiving side of the lateral load, and a compressive load at the opposite side, due to overturning. Both members have been checked to handle the combination of the lateral compressive load in addition to the gravity and live loads of the stories above. The controlling load combination was determined to be 1.2D + 1.6W + 1.0L, yielding a required load of 1856.9k. With the use of the AISC Steel Manual, the W14x311 columns were found to have a maximum axial load capacity of 1846.3k. Although this strength is less than what is required, a conservative approach was taken in determining the loads from the floors above. This capacity, being within 5%, has been determined to be acceptable to carry the axial load (see Table 22).

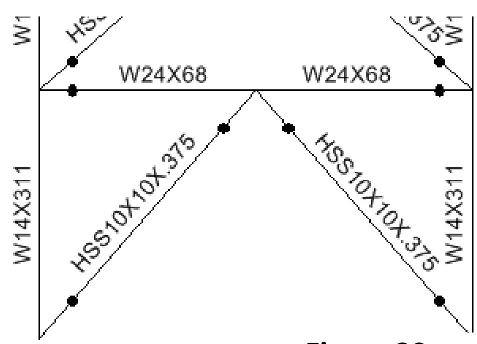


Figure 20

COLUMN LOAD TAKE DOWN				
Level	Dead	Live	Reduction	Area
Pent	40 psf	30 psf	15.6 psf	744 ft^2
Roof	56 psf	100 psf	52 psf	744 ft^2
8th	45 psf	150 psf	78 psf	744 ft^2
7th	62 psf	60 psf	32 psf	744 ft^2
6th	62 psf	60 plf	32 psf	744 ft^2
5th	62 psf	60 psf	32 psf	744 ft^2
4th	62 psf	60 plf	32 psf	744 ft^2
3rd	62 psf	60 psf	32 psf	744 ft^2
2nd	62 psf	125 psf	65 psf	744 ft^2
1st	62 psf	60 psf	32 psf	744 ft^2
Ground	56 psf	100 psf	52 psf	744 ft^2

Table 22

Drift Analysis

The Cancer Hospital has been checked for both story and overall drift from not only wind lateral loads, but also seismic in accordance with ASCE7-05 and IBC 2006. Drift results have been found using program output from the ETABS model. By referencing code, acceptable drift limits were found for both wind and seismic at each level (see Table 23 - 24). The seismic allowable drift limit is found from the equation 0.010 h_{sx} and the wind allowable drift limit is found using H/400. Drift at all stories due to all lateral loads in each direction were found to be within the limits of code. The maximum drift was determined to be 3.16" at the roof level and caused by wind in the north/south direction. Although this amount of drift is within the acceptable limit, a value this high presents a concern on the levels where medical imaging and surgeries are performed. Further investigation will be conducted in order to determine the effect of the existing drift on the strict movement criteria in these spaces.

DRIFT FROM WIND LOADS				
Level	**Story Height (ft)**	**N/S Drift (in)**	**E/W Drift (in)**	**Code Allowable H/400 (in)**
Roof	137.75	3.1563	1.4941	4.1325
8th	117	2.6198	1.1818	3.51
7th	102	2.2735	1.0358	3.06
6th	87	1.9091	0.8854	2.61
5th	72	1.5342	0.7092	2.16
4th	57	1.206	0.5501	1.71
3rd	42	0.8938	0.3902	1.26
2nd	28	0.6231	0.2383	0.84
1st	14	0.2224	0.1104	0.42
Ground	0	0	0	0

Table 23

DRIFT FROM SEISMIC				
Level	**Story Height (ft)**	**N/S Drift (in)**	**E/W Drift (in)**	**Code Allowable 0.010 h_{sx} (in)**
Roof	137.75	0.7265	1.0833	1.3775
8th	117	0.6055	0.8466	1.17
7th	102	0.5263	0.7356	1.02
6th	87	0.441	0.6193	0.87
5th	72	0.3527	0.4806	0.72
4th	57	0.2791	0.3443	0.57
3rd	42	0.2066	0.2337	0.42
2nd	28	0.1436	0.1363	0.28
1st	14	0.0498	0.0603	0.14
Ground	0	0	0	0

Table 24

Overturning Moment

Axial Effect

Overturning moment has been calculated and taken into account in the member design check (see Tables 25 -26). In the existing system of the Cancer Hospital, axial loads from overturning cause compressive and tensile forces in the columns making up each braced frame. The largest overturning moment has been found to be in the bottom columns of Frame K. These columns have been determined through a representative strength check to be adequate to carry the required load.

Foundation

Foundations in the existing system will be exposed to uplift force caused by overturning moments. The magnitude of the uplift forces depends on the direction of the lateral load. Gravity loads have been found to exceed the uplift force in all east/west frames and require no special foundation. In frame K it was found that uplift forces in the lower three levels exceeded the 469.5k gravity load. For this reason special foundation design for uplift force is required for frames in the north/south direction.

NORTH - SOUTH DIRECTION USING FRAME K						
Story	Tributary Height (ft)	Forces (k)	Frame Shear (k)	Overturn Moment (ft-k)	Uplift Force (k)	Foundation Effect
Roof	20.75	80.84	40.42	40.42	1.28	NONE
8	15	39.35	79.77	2578.93	81.87	NONE
7	15	43.35	123.13	4707.01	149.43	NONE
6	15	38.40	161.53	7448.24	236.45	NONE
5	15	39.47	200.99	10773.47	342.01	NONE
4	15	28.72	229.71	14610.10	463.81	NONE
3	14	30.84	260.56	18742.96	595.01	YES
2	14	2.26	262.81	22972.41	729.28	YES
1	14	21.85	284.66	27370.61	868.91	YES

Table 25

EAST - WEST DIRECTION USING FRAME 2						
Story	Tributary Height (ft)	Forces (k)	Frame Shear (k)	Overturn Moment (ft-k)	Uplift Force (k)	Foundation Effect
Roof	20.75	25.29	12.65	12.65	0.40	NONE
8	15	17.76	30.41	847.64	26.91	NONE
7	15	16.79	47.20	1619.32	51.41	NONE
6	15	15.72	62.92	2634.82	83.65	NONE
5	15	14.63	77.55	3877.94	123.11	NONE
4	15	23.05	100.60	5403.67	171.55	NONE
3	14	19.05	119.65	7179.00	227.90	NONE
2	14	15.86	135.51	9142.08	290.22	NONE
1	14	15.59	151.10	11325.31	359.53	NONE

Table 26

Summary

In this report, the wind and seismic loads have been found and applied to the existing lateral system. The lateral system was then checked for compliance with code and design capacity in the areas of strength, drift, and impact on foundation. An *ETABS* model was created and used to aid in these checks once it was determined to be an accurate representation of the Cancer Hospital structure. From the results obtained, the building has been determined to adhere to all limits of code and design however several key topics were developed as areas of interest.

Torsion

All lateral frames were determined to be adequate in carrying the combined force from direct shear and torsion however the loads on each frame did not appear to be very evenly distributed over the entire structure. Frames such as B,7, and K located around the outward perimeter of the building were shown to take more load than the frames located on the interior. This has caused a sizeable amount of torsion in the structure. This is to be expected in a non-symmetrical building however methods of decreasing this uneven distribution will be investigated and results will be presented in a future report.

Bracing Irregularity

In certain frames throughout the lateral structure, irregular bracing has been identified. This has caused stress reversals in the frames which hold these atypical configurations and aids in the uneven distribution of forces to the structure. Additional research will be conducted to determine the reason for these irregular bracing patterns and alternatives will be evaluated.

Member Strength

Critical members in the lateral structure of the Cancer Hospital have been checked for strength to resist loads both from lateral contributions as well as predetermined live and dead loads if applicable. Member strength capacity was found to be typically within 5-10% of the required loads. Not only have the members been found to be adequate to carry the required loads, but also efficient in design.

Drift

The existing structure complies with all required drift limits for both wind and seismic loads however the maximum drift from wind in the north/south direction causes a significant drift of 3.14”. In a normal structure this would not be a concern however, due to the low amount of movement required in the imaging and surgery rooms, an effort should be made to decrease this value.

Overturning Moment

In addition to the direct lateral shear, axial forces from overturning moment were also considered. Though a spot check, the columns in the lateral frames were determined to capable of carrying this addition load. Uplift forces were evaluated against the gravity loads transferred axially to the foundation. Gravity loads were found to exceed the uplift force in all east/west frames and require no special foundation design however this is not the case in the north/south direction. In frame K it was found that uplift forces in the lower three levels exceeded the 469.5k gravity load. For this reason special foundation design for uplift force is required for frames in the north/south direction.

AREAS OF CONCERN						
	EAST-WEST DIRECTION			NORTH - SOUTH DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Torsion	NO	NO	YES	YES	NO	YES
Irregularity	NO	NO	YES	YES	YES	YES
Strength	NO	NO	NO	NO	NO	NO
Drift	NO	NO	NO	YES	YES	YES
Overturning	NO	NO	NO	YES	YES	YES

Table 27

Conclusions

This report has evaluated the effect of wind and seismic loads on the existing lateral system design of the University Hospitals Case Medical Center Cancer Hospital. The lateral force resisting system has been found to be adequate to carry the required loads controlled by wind in both directions. As forces transferred to each frame were determined, a sizeable amount of torsional force was found and determined to be caused by uneven distribution of stiffness and location of frames. Irregular bracing configurations were found contributing to this uneven distribution however all lateral members were efficiently sized and found capable of carrying the required load. Although the building complies with all drift limits required by code, additional measures should be taken to decrease this drift in order to adhere to the sensitive movement requirements of the surgery and imaging rooms. Axial forces from overturning moment were taken into consideration and determined to be effectively carried by columns and foundations. Uplift forces in the east/west braced frames were found not to exceed gravity loads however braced frames in the north/south direction require special consideration in foundation design for uplift.

Appendix A

Load Calculation

Seismic Calc

Using Chap 12 ASCE 7-05

- Design Spectral Response Accel.

  $S_s = 19.2\%$ (www.usgs.gov)
  $S_1 = 5.1\%$

  Site Class C

  $F_a = 1.2$
  $F_v = 1.7$

  $S_{MS} = 1.2(.192) = .2304$ (table 12.2-1)
  $S_{M1} = 1.7(.051) = .0867$

  $S_{DS} = \frac{2}{3}(.2304) = .1536$

  $S_{D1} = \frac{2}{3}(.087) = .058$

- Occupancy Cat: IV (table 11.5-1)
  Seismic Design Cat.: A
- Use Equivalent Lateral Force Method

  $V = C_s(W)$

- DL (units k, ft)

  see table 1 through 3)

Load Calculation

AMPAD

• Calculate seismic base shear:

$I = 1.5$
$R = 7$ (eccentric braced frame, non-moment)

$C_s = \frac{S_{DS}}{(R/I)} = \frac{.1534}{(7/1.5)} = .033$

$T_a = C_t h_n^x$ $\quad C_t = .03 \quad x = .75 \quad h_n = 172.08'$

$T_a = .03(154.08)^{.75}$ (eq. 12.8-7)
$= 1.87$ sec (table 12.8-2)

$C_s \le \frac{S_{D1}}{T(R/I)}$

$\le \frac{.058}{1.87(7/1.5)}$

$= .009$

$C_s\ .009 \le .01$

$C_s = .01$ (12.8-5)

$V = C_s W$
$= .01(21408.2014\ 16)$
$= 206.7$

$\boxed{V = 204.9k}$ (eq. 12.8)

$V_{ACT} = 213.6^k$

Error $\le 5\%$ ✓ okay

Load Calculation

Wind:

- $V = 90\,mph$ Fig 6-1 (6.5.4)
- $K_d = .85$ Table 6-4 (6.5.4.4)
- $I = 1.15$ Table 6-1 (6.5.5)
- Wind Exp. Cat. : B
- $K_{zt} = 1.0$ not situated on hill, ridge, or escarpment
- $K_h = 1.12$ (Table 6-3)
- $q_p = .00256 K_h K_{zt} K_d V^2 I$

  $= .00256(1.12)(1.0)(.85)(90)^2(1.15)$

  $= 22.7\,psf$
- $p_p = q_p\, GC_{pn}$ → parapit

  $= 22.7(1.5) = 34.1\,psf$ for windward parapit

  $= 22.7(-1.0) = 22.7\,psf$ for leeward parapit

parapit height = 4'-3"

$F = 34.1(4.25) = 144.9\,plf$ windward

$F = 22.7(4.25) = 96.5\,plf$ leeward

- Determine wether flexible or rigid

  $n = \frac{22.2}{H^{.8}} = \frac{22.2}{(184)^{.8}}$ (ASCE 7-05; C6.5.8)

  $n = .40 < 1$ **flexible**

Load Calculation

• Gust Factors $\boxed{\text{North-south}}$

$g_Q = g_v = 3.4$; $n_1 = .40$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{.577}{\sqrt{2\ln(3600 n_1)}}$$

$= 3.82 + .15$

$g_R = 3.97$ Eq 6-9

$\bar{z} = .6(154)$
$= 92' > z_{min} = 30'$ ✓okay (table 6-2)

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$ $c = .3$ (table 6-2)

$= .3\left(\frac{33}{92}\right)^{1/6}$

$= .25$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$ $\ell = 320'$; $\bar{\epsilon} = 1/3.0$ (table 6-2)

$= 320\left(\frac{92}{33}\right)^{1/3.0}$

$= 450.4$

* $Q = \sqrt{\dfrac{1}{1 + .63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}}$ $B = 304.25$

$= \sqrt{\dfrac{1}{1.63}}$

$= .781$

$V_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$ $\bar{\alpha} = 1/4.0$; $\bar{b} = .45$ (table 6-2)

$= .45\left(\frac{92}{33}\right)^{1/4.0}(90)\left(\frac{88}{60}\right)$

$= 76.8$ mph

$N_1 = \dfrac{n_1 L_{\bar{z}}}{V_{\bar{z}}} = \dfrac{.46(450.4)}{76.8}$

$= 2.7$

Load Calculation

$$R_n = \frac{7.47(N_1)}{(1+10.3(N_1))^{5/3}}$$
$$= \frac{7.47(2.7)}{(1+10.3(2.7))^{5/3}}$$
$$= .074$$

$$\eta = 4.6 n_1 h/\bar{V}_z$$
$$h = 4.6(.46)\left(\frac{154}{76.8}\right)$$
$$= 4.24$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$$
$$= \frac{1}{4.24} - \frac{1}{2(4.24)^2}(1-e^{-2(4.24)})$$
$$= .21$$

$$\eta_B = 4.6 n_1 B/\bar{V}_z$$
$$= 4.6(.46)\frac{(304.20)}{76.8}$$
$$= 8.38$$

$$R_B = \frac{1}{8.38} - \frac{1}{2(8.38)^2}(1-e^{-2(8.38)})$$
$$= .12 - .007(.999)$$
$$= .113$$

$$\eta_L = 15.4(n_1)(L)/\bar{V}_z \qquad L = 228.33'$$
$$= 15.4(.46)\frac{(228.33)}{76.8}$$
$$= 20.6$$

$$R_L = \frac{1}{20.6} - \frac{1}{2(20.6)^2}(1-e^{-2(20.6)})$$
$$= .048 - .0011(1)$$
$$= .047$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53+.47R_L)} \qquad \beta = 1.0 \text{ (steel)}$$
$$= \sqrt{\frac{1}{.01}(.074)(.21)(.113)(.53+.47(.047))}$$
$$= .37$$

Load Calculation

$$G_f = .925\left(\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

$$= .925\left(\frac{1 + 1.7(.25)\sqrt{(3.4)^2(.781)^2 + (3.97)^2(.371)^2}}{1 + 1.7(3.4)(.25)}\right)$$

$$= .925\left(\frac{2.4}{2.5}\right)$$

$$= .89$$

$$\underline{\underline{G_f = .89}}$$

Load Calculation

• Gust Factor $\boxed{\text{East - West}}$ (using previous)

$g_Q = g_V = 3.4$; $n_1 = .46$

$g_R = 3.97$

$\bar{z} = 92'$

$I_{\bar{z}} = .25$

$L_{\bar{z}} = 450.4$

$Q = .771$

$\bar{V}_{\bar{z}} = 76.8 \text{ mph}$

$N = 2.7$

$R_n = .074$

$\eta_h = 4.24$

$R_h = .21$

$\eta_B = 4.6 n_1 B / \bar{V}_{\bar{z}}$

$= 4.6(.46)\left(\frac{223.33}{76.8}\right)$

$= 6.15$

$R_B = \frac{1}{6.15} - \frac{1}{2(6.15)^2}(1 - e^{-2(6.15)})$

$= .16 - .013(.99)$

$= .147$

$\eta_L = 15.4 n_1 L / \bar{V}_{\bar{z}}$

$= 15.4(.46)\frac{(304.25)}{76.8}$

$= 28.06$

$R_L = \frac{1}{28.06} - \frac{1}{2(28.06)^2}(1 - e^{-2(28.06)})$

$= .035 - .00063(1)$

$= .034$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53 + .47 R_L)}$

$= \sqrt{\frac{1}{.01}(.074)(.21)(.147)(.53 + .47(.034))}$

$= .35$

Load Calculation

$$G_f = .925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

$$= .925\left(\frac{1 + 1.7(.25)\sqrt{(3.4)^2(.781)^2 + (3.97)^2(.85)^2}}{1 + 1.7(3.4)(.25)}\right)$$

$$= .925\left(\frac{2.27}{2.5}\right)$$

$$= .84$$

$$\underline{\underline{G_f = .84}}$$

Load Calculation

• Determine velocity pressures q_z ; q_h

$q_z = .00256\, K_z K_{zt} K_d V^2 I$
$= .00256\, K_z\, 1.0(.85)(90)^2(1.15)$
$= 20.27$

$q_z = 20.27 K_z$
(see excel chart)

• Determine C_p

E-W
Windward wall: $C_p = .8$
Leeward wall $(L/B = 304/223.3 = 1.36)$ $C_p = -.428$
Side wall: $C_p = .7$

N-S
Windward wall: $C_p = .8$
Leeward wall: $(L/B = 223.3/304 = .73)$ $C_p = -.5$
Side wall: $C_p = -.7$

• Determine q_i
$q_i = q_h = 22.7$ psf

• Determine GC_{pi}
$GC_{pi} = +.18, -.18$ for enclosed structure

Appendix B

Spreadsheet Output

TOTAL SEISMIC FORCE NORTH - SOUTH						
Level	EAST - WEST DIRECTION			NORTH - SOUTH DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	20.36932607	21.56785884	0	-0.549344044	0.298730804	1.071777902
8th	15.5212867	14.87275517	0	-0.501322498	0.31025725	0.634134247
7th	17.05829206	16.38646407	0	-0.508500929	0.28024085	0.750895885
6th	14.68712219	14.32656251	0	-0.486185135	0.238875694	0.608592303
5th	12.14270545	11.63532225	0	-0.453392766	0.147399044	0.471454719
4th	8.268199527	8.173406641	3.369772703	-2.922586248	1.239717661	2.480503729
3rd	7.637410594	7.594189693	3.412078417	-2.297061468	0.973349584	2.768834177
2nd	5.604178091	6.060909504	1.486765488	-3.159787712	1.088180124	0.193033484
1ST	2.450163227	2.252580484	0.90022316	-0.745150204	0.307466705	0.808227125

SEISMIC DIRECT SHEAR NORTH - SOUTH						
Level	X - DIRECTION			Y - DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	20.09988694	21.47011306	0	0	0	0
8th	15.2624249	14.7875751	0	0	0	0
7th	16.76898915	16.29101085	0	0	0	0
6th	14.42224502	14.23775498	0	0	0	0
5th	11.9335325	11.5664675	0	0	0	0
4th	7.320239748	7.829078977	4.800681275	0	0	0
3rd	6.775440729	7.279659574	4.824899696	0	0	0
2nd	4.705654535	5.691799628	2.652545837	0	0	0
1ST	2.181499607	2.162145738	1.256354655	0	0	0

Spreadsheet Output

SEISMIC TORSION FORCE NORTH - SOUTH						
Level	X - DIRECTION			Y - DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	0.269439127	0.09774578	0	-0.549344044	0.298730804	1.071777902
8th	0.2588618	0.085180077	0	-0.501322498	0.31025725	0.634134247
7th	0.289302907	0.09545322	0	-0.508500929	0.28024085	0.750895885
6th	0.264877166	0.088807532	0	-0.486185135	0.238875694	0.608592303
5th	0.209172956	0.068854748	0	-0.453392766	0.147399044	0.471454719
4th	0.947959779	0.344327664	-1.430908573	-2.922586248	1.239717661	2.480503729
3rd	0.861969864	0.314530118	-1.412821279	-2.297061468	0.973349584	2.768834177
2nd	0.898523556	0.369109876	-1.165780349	-3.159787712	1.088180124	0.193033484
1ST	0.26866362	0.090434746	-0.356131495	-0.745150204	0.307466705	0.808227125

Spreadsheet Output

TOTAL SEISMIC FORCE EAST - WEST						
Level	EAST - WEST DIRECTION			NORTH - SOUTH DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	0.269439127	0.09774578	0	9.582275347	15.17445845	17.63443087
8th	0.2588618	0.085180077	0	7.553154181	13.76904165	9.170873172
7th	0.289302907	0.09545322	0	8.365923304	13.48540003	11.73131247
6th	0.264877166	0.088807532	0	8.004567798	11.50255442	9.514160646
5th	0.209172956	0.068854748	0	8.095005838	7.650980622	7.919474537
4th	0.947959779	0.344327664	-1.430908573	4.061599736	9.238705045	7.447330362
3rd	0.861969864	0.314530118	-1.412821279	3.688846343	7.821763782	8.814512167
2nd	0.898523556	0.369109876	-1.165780349	3.424073616	7.217306244	0.530046036
1ST	0.26866362	0.090434746	-0.356131495	1.107369863	2.371330128	2.491843635

SEISMIC DIRECT SHEAR EAST - WEST						
Level	X - DIRECTION			Y - DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	0	0	0	10.13161939	14.87572764	16.56265296
8th	0	0	0	8.054476679	13.4587844	8.536738924
7th	0	0	0	8.874424233	13.20515918	10.98041659
6th	0	0	0	8.490752932	11.26367872	8.905568343
5th	0	0	0	8.548398604	7.503581579	7.448019817
4th	0	0	0	6.984185983	7.998987384	4.966826633
3rd	0	0	0	5.985907811	6.848414198	6.045677991
2nd	0	0	0	6.583861328	6.12912612	0.337012552
1ST	0	0	0	1.852520067	2.063863423	1.68361651

Spreadsheet Output

SEISMIC TORSION FORCE EAST - WEST						
Level	X - DIRECTION			Y - DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	0.269439127	0.09774578	0	-0.549344044	0.298730804	1.071777902
8th	0.2588618	0.085180077	0	-0.501322498	0.31025725	0.634134247
7th	0.289302907	0.09545322	0	-0.508500929	0.28024085	0.750895885
6th	0.264877166	0.088807532	0	-0.486185135	0.238875694	0.608592303
5th	0.209172956	0.068854748	0	-0.453392766	0.147399044	0.471454719
4th	0.947959779	0.344327664	-1.430908573	-2.922586248	1.239717661	2.480503729
3rd	0.861969864	0.314530118	-1.412821279	-2.297061468	0.973349584	2.768834177
2nd	0.898523556	0.369109876	-1.165780349	-3.159787712	1.088180124	0.193033484
1ST	0.26866362	0.090434746	-0.356131495	-0.745150204	0.307466705	0.808227125

Spreadsheet Output

TOTAL WIND FORCE NORTH - SOUTH						
Level	EAST - WEST DIRECTION			NORTH SOUTH DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	1.240278687	0.449942102	0	43.90534703	69.55184905	80.84165279
8th	1.115356341	0.367014903	0	32.39268463	59.07335952	39.35386719
7th	1.073500892	0.35419318	0	30.89918768	49.82558346	43.35283322
6th	1.073475697	0.35991297	0	32.29008863	46.41741085	38.40071999
5th	2.546582094	0.838274081	0	33.18909731	35.77230379	39.46593351
4th	1.539867382	0.559326408	-2.324370176	29.97269392	41.77878042	28.72068046
3rd	1.249788276	0.456043848	-2.048479354	23.23058438	31.79960367	30.84095022
2nd	1.654922722	0.679835622	-2.147162838	31.29110835	36.55193919	2.255153606
1ST	0.984812681	0.331497374	-1.305434699	18.05023047	24.27955935	21.84951896

WIND DIRECT SHEAR NORTH - SOUTH						
Level	X - DIRECTION			Y - DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	0	0	0	46.4340804	68.1767353	75.90805868
8th	0	0	0	34.55272997	57.7365559	36.62157663
7th	0	0	0	32.786055	48.78570866	40.56652384
6th	0	0	0	34.26046582	45.4493121	35.93425957
5th	0	0	0	38.70894041	33.97778995	33.72619465
4th	0	0	0	34.7201477	39.76498107	24.69134623
3rd	0	0	0	26.56114223	30.38832359	26.82635918
2nd	0	0	0	37.11088285	34.54770235	1.899619801
1ST	0	0	0	20.78165116	23.15251018	18.88688367

Spreadsheet Output

WIND TORSION FORCE NORTH - SOUTH						
Level	X - DIRECTION			Y - DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	1.240278687	0.449942102	0	-2.528733362	1.375113754	4.933594104
8th	1.115356341	0.367014903	0	-2.160045343	1.336803615	2.73229056
7th	1.073500892	0.35419318	0	-1.886867323	1.039874801	2.78630938
6th	1.073475697	0.35991297	0	-1.97037719	0.968098747	2.46646042
5th	2.546582094	0.838274081	0	-5.519843091	1.79451384	5.739738859
4th	1.539867382	0.559326408	-2.324370176	-4.747453778	2.013799353	4.029334227
3rd	1.249788276	0.456043848	-2.048479354	-3.330557844	1.41128008	4.014591039
2nd	1.654922722	0.679835622	-2.147162838	-5.819774503	2.004236841	0.355533806
1ST	0.984812681	0.331497374	-1.305434699	-2.731420688	1.12704917	2.96263529

Spreadsheet Output

TOTAL WIND FORCE EAST - WEST						
Level	EAST - WEST DIRECTION			NORTH - SOUTH DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	25.2915617	27.78602982	0	2.226500091	-1.210760669	-4.34393277
8th	17.76544566	17.84090249	0	1.901379267	-1.176720982	-2.405097947
7th	16.79894642	16.92653353	0	1.661416972	-0.915626458	-2.453390145
6th	15.72352767	16.13863801	0	1.735045089	-0.852473823	-2.171878593
5th	14.63927911	14.72513341	0	1.815632604	-0.590266387	-1.887962545
4th	23.05226208	26.57163859	20.9948538	8.367937613	-3.549554801	-7.102168658
3rd	19.08961658	22.07367999	18.77487203	5.872138304	-2.488241371	-7.078163755
2nd	15.8654339	21.52010621	14.37170995	10.25719327	-3.536573248	-0.626618601
1ST	15.59301161	16.59091608	12.28118757	4.814865074	-1.9867279	-5.222443121

WIND DIRECT SHEAR EAST - WEST						
Level	X - DIRECTION			Y - DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	26.38360274	28.18219501	0	0	0	0
8th	18.74723775	18.16396725	0	0	0	0
7th	17.74418119	17.23840631	0	0	0	0
6th	16.66879275	16.45556475	0	0	0	0
5th	15.47692203	15.00086547	0	0	0	0
4th	25.76645685	27.55751623	16.89788193	0	0	0
3rd	21.29313021	22.87773525	15.16317854	0	0	0
2nd	18.7821899	22.71829788	10.58739422	0	0	0
1ST	17.32900936	17.17526954	9.980007105	0	0	0

Spreadsheet Output

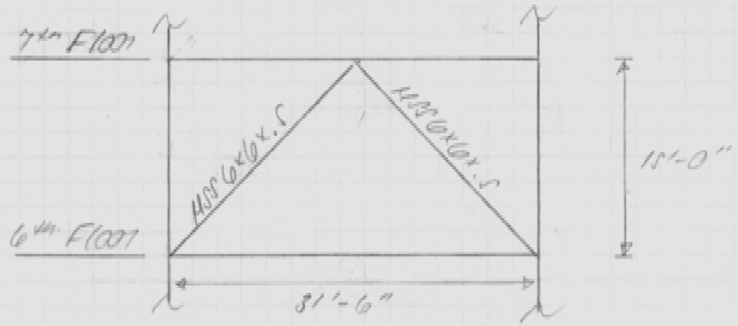
WIND TORSION FORCE EAST - WEST						
Level	X - DIRECTION			Y - DIRECTION		
	FRAME 2	FRAME 3	FRAME 7	FRAME B	FRAME G	FRAME K
Roof	-1.092041039	-0.396165189	0	2.226500091	-1.210760669	-4.34393277
8th	-0.981792085	-0.323064758	0	1.901379267	-1.176720982	-2.405097947
7th	-0.945234771	-0.311872782	0	1.661416972	-0.915626458	-2.453390145
6th	-0.945265072	-0.316926746	0	1.735045089	-0.852473823	-2.171878593
5th	-0.837642919	-0.275732068	0	1.815632604	-0.590266387	-1.887962545
4th	-2.714194764	-0.985877631	4.096971878	8.367937613	-3.549554801	-7.102168658
3rd	-2.203513632	-0.804055259	3.61169349	5.872138304	-2.488241371	-7.078163755
2nd	-2.916756	-1.198191676	3.784315731	10.25719327	-3.536573248	-0.626618601
1ST	-1.735997755	-0.58435346	2.301180469	4.814865074	-1.9867279	-5.222443121

Appendix C

Strength Checks

Strength Check:

Frame K - Sixth Floor



Brace 1

• Loads

Dead

Floor Load + S.W.	47 psf
Ceiling Partition	5 psf
Suspended Mech Equip	10 psf
	62 psf

Live

Floor Load — 60 (reducible)

$L = L_0\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$ — $K_{LL} = 2.0$ (ASCE 7-05 Table 4-2)

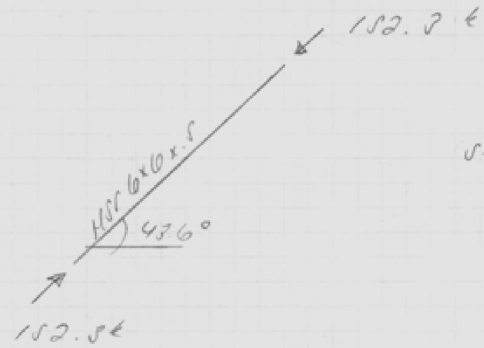
$= 60\left(.25 + \frac{15}{\sqrt{2.0(330.75)}}\right)$ — $A_T = (10.5)(31.5)$ $= 330.75\ ft^2$

$= 50\ psf$

$L = 50\ psf$

Strength Checks

Controling load case = $1.2D + 1.6W + L$



$\sin 43.6 = \frac{15'}{L}$

$L =$

• Axial strength

$r = 2.23$ (AISC Steel Design Manual 13th Ed.)

$\frac{KL}{r} = \frac{1.0(21.75)(12)}{2.23} = 117.04$

$4.71\left(\sqrt{\frac{29000}{50}}\right) = 113 < 117.04$

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{(3.14)^2(29000)}{(117.04)^2} = 20.87$ ksi

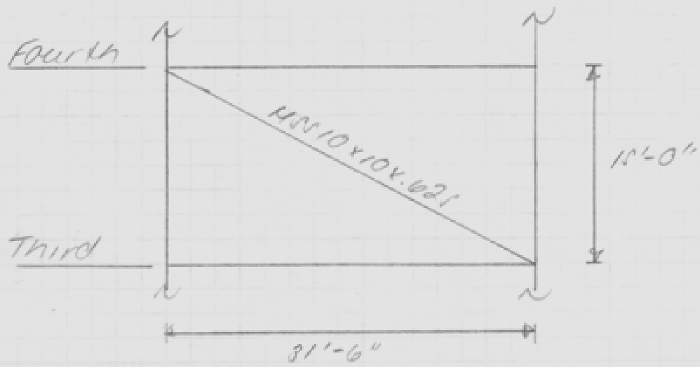
$F_{CR} = .877(20.87) = 18.3$ ksi

$P_n = F_{CR}(A_g) = 18.3(9.71) = 177.69^k$

$\phi P_n = 159.92 > P_u = 152.3^k$ ✓ okay

Strength Checks

Frame K - Third Floor

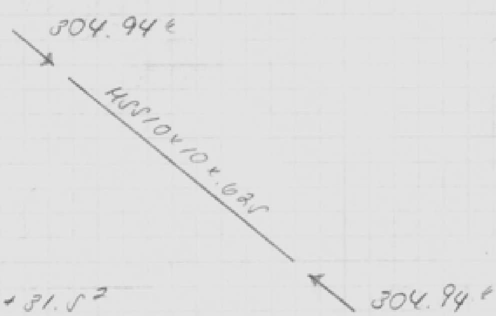


Brace 2

- Loads (only wind taken)

1.6 Wind controls   ASCE 7-05

Wind (from Etabs) $1.6(190.59) = 304.94^k$



$L = 15^2 + 31.5^2$
$L = 34.89'$

Strength Checks

• Axial Strength

$$\frac{KL}{r} = \frac{1.0(34.89)(12)}{3.8} = 110.17$$

$$4.71\left(\sqrt{\frac{29000}{50}}\right) = 113 > 110.17$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{(3.14)^2(29000)}{(110.17)^2} = 23.55$$

$$F_{cr} = .658^{\left(\frac{50}{23.55}\right)}(50) = 20.56$$

$$P_n = (F_{cr})(A_g) = 20.56(21.0)$$

$$\phi P_n = .9(431.76) = 388.58^k > P_u = 304.94^k \quad \checkmark \underline{\text{okay}}$$

Strength Checks

$w_D = (62\,psf)(10.5) = .651\ ^{k}/_{ft}$
$w_L = (50\,psf)(10.5) = .525\ ^{k}/_{ft}$

$D = \dfrac{(.651)(31.5)}{2} = 10.25^{k}$

$L = \dfrac{(.525)(31.5)}{2} = 8.27^{k}$

$\theta = \tan^{-1}\left(\dfrac{180}{189}\right)$

$\theta = 43.6^{\circ}$

$P_D = \dfrac{10.25}{\cos(90 - 43.6)}$
$= 14.86^{k}$

$P_L = \dfrac{8.27^{k}}{\cos(90 - 43.6)}$
$= 11.99^{k}$

$P_D = 14.86^{k}$
$P_L = 11.99^{k}$
$P_W = 76.51^{k}$ (from etabs)

Load Combination: (ASCE 7-05 2.3.2)

- $1.4D$

  $1.4(14.86) = 20.81^{k}$

- $1.2D + 1.6L$

  $1.2(14.86) + 1.6(11.99) = 197.5^{k}$

- $1.2D + 1.6W + L$

  $1.2(14.86) + 1.6(76.51) + 11.99 = 152.3^{k}$

- $.9D + 1.6W$

  $.9(14.86) + 1.6(76.51) = 135.8^{k}$

Strength Checks

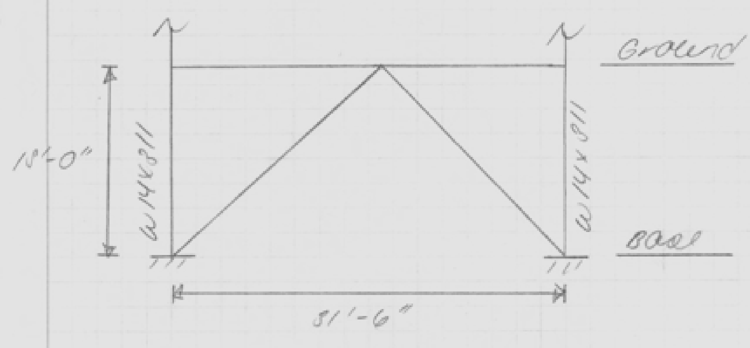
Frame K - Sub Basement Level

• Load Reduction

$$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$$

$$= L_o\left(.25 + \frac{15}{\sqrt{(4.0)(744)}}\right)$$

$$= L_o(.52)$$



Columns

• Loads

Dead (from column gravity load spreadsheet) 631 psf

Live Reduced (from spreadsheet) 323.3 psf

$P_D = (631)(744)$
$= 469.5^k$

$P_L = (434)(744)$
$= 323.3^k$

$P_W =$ (from Etabs) $= 606.42^k$

Strength Checks

Load Combination

- $1.4D$

  $1.4(469.5) = 657.3^k$

- $1.2D + 1.6L$

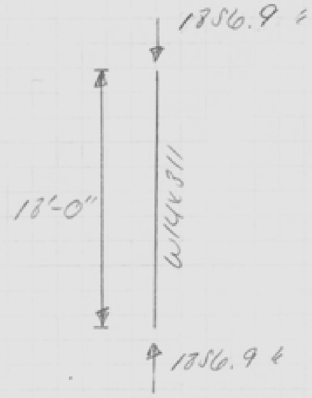
  $1.2(469.5) + 1.6(323.3) = 1080.7^k$

- $1.2D + 1.6W + L$

  $1.2(469.5) + 1.6(606.4) + 323.3 = 1856.94^k$

- $.9D + 1.6W$

  $.9(469.5) + 1.6(606.4) = 1392.8^k$



$\underline{1.2D + 1.6W + L \text{ controls}}$

- Axial strength

$$\frac{KL}{r} = \frac{1.0(18)(12)}{4.2} = 51.4$$

$r = 4.2$ (AISC Steel Manual 13th Ed.)

$$4.71\left(\sqrt{\frac{29000}{50}}\right) = 113 > 51.4$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{(3.14)^2(29000)}{(51.4)^2} = 108.2$$

Strength Checks

$F_{cr} = .658^{\left(\frac{50}{108.2}\right)}(50)$

$= 20.21$

$P_n = F_{cr}(A_g)$

$= 20.21(91.4)$

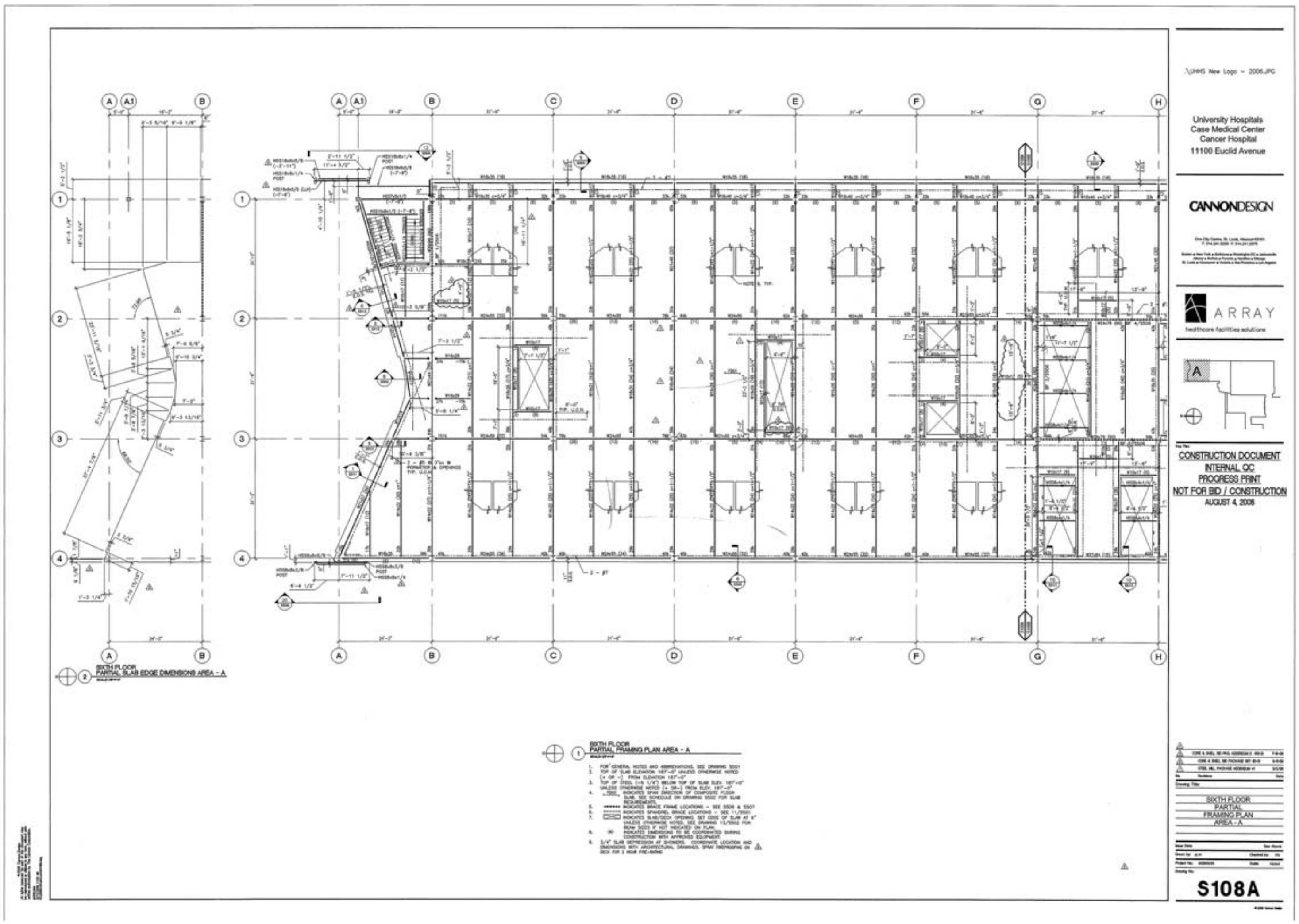
$= 1846.28^k$

$\phi P_n = 1846.3^k < P_u \doteq 1856.9^k \rightarrow$ within 5% error

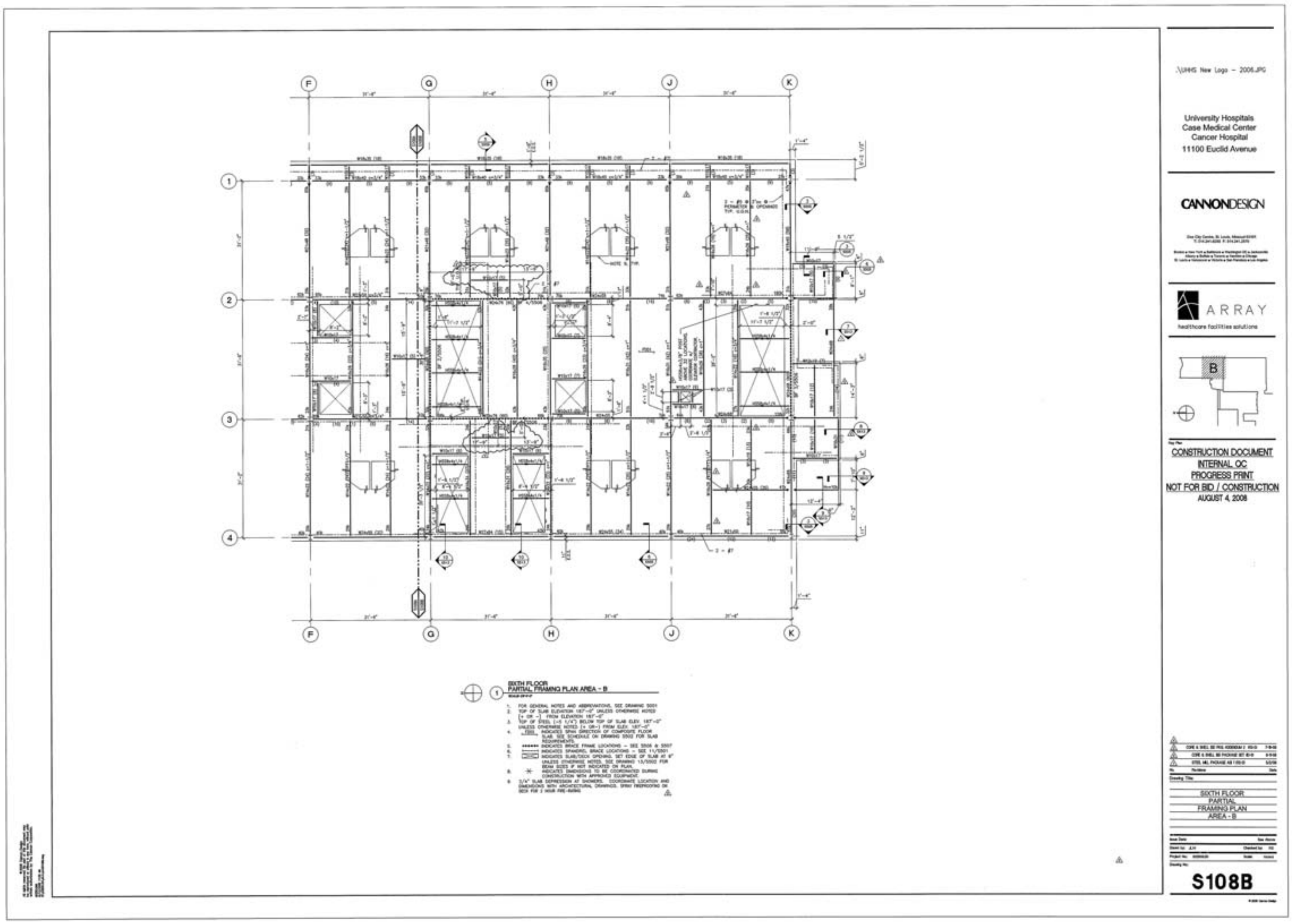
✓ <u>okay</u>

Appendix D

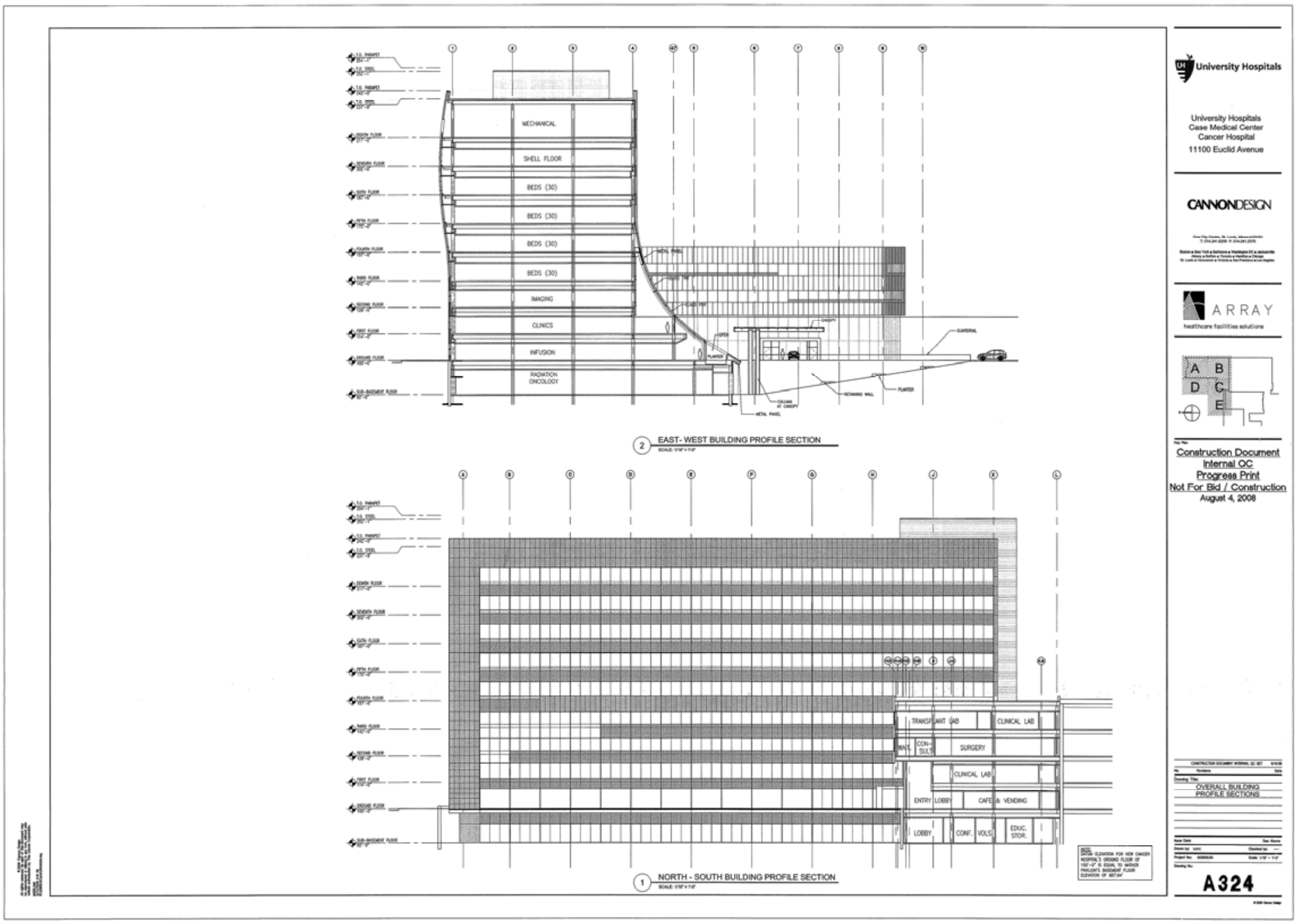
Level 6 Floor Plan



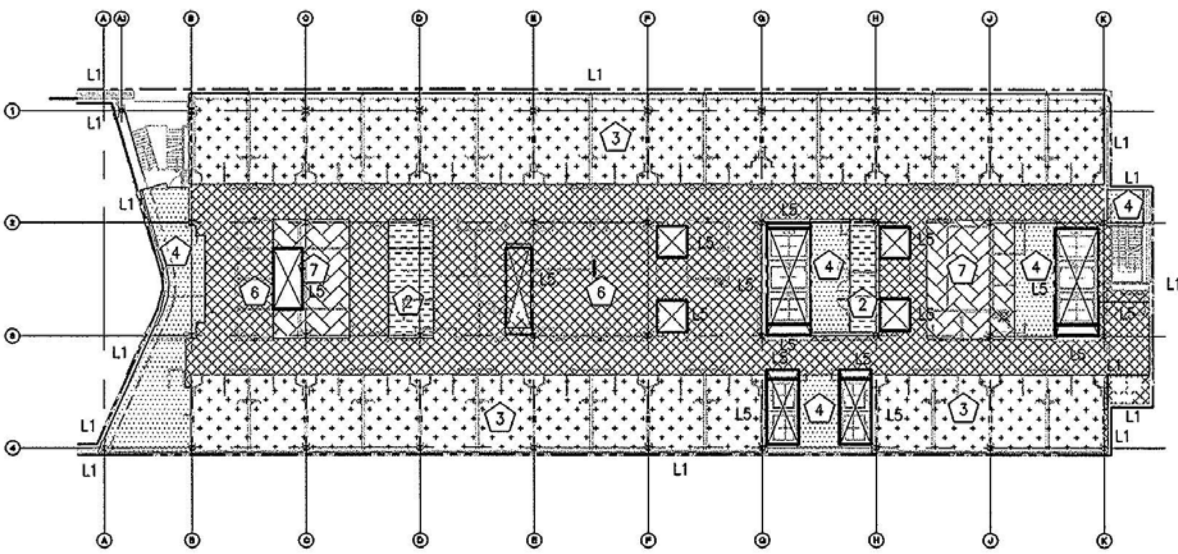
Level 6 Floor Plan



Elevation


MECHANICAL
SHELL FLOOR
BEDS (30)
BEDS (30)
BEDS (30)
BEDS (30)
IMAGING
CLINICS
INFUSION
RADIATION ONCOLOGY
EAST- WEST BUILDING PROFILE SECTION
TRANSPLANT LAB
CLINICAL LAB
SURGERY
CLINICAL LAB
ENTRY LOBBY
CAFE & VENDING
LOBBY
CONF.
VOLS.
EDUC. STOR.
NORTH - SOUTH BUILDING PROFILE SECTION
University Hospitals
University Hospitals
Case Medical Center
Cancer Hospital
11100 Euclid Avenue
CANNONDESIGN
ARRAY
healthcare facilities solutions
A
B
C
D
E
Construction Document
Internal QC
Progress Print
Not For Bid / Construction
August 4, 2008
OVERALL BUILDING PROFILE SECTIONS
A324

Loading Diagram



2 LEVEL 5 AND 6
LOAD KEY DIAGRAM
SCALE: 1/32=1'-0"

SUPERIMPOSED LOADS

SURFACE LOAD SCHEDULE					
LABEL	PATTERN	DL (psf)	LL (psf)	REDUCTION TYPE	MASS DL (psf)
1		47	370	Unreducible	102.5
2		47	150	Unreducible	103.2
3		47	40	Reducible	75.7
4		41	100	Reducible	41
5		47	60	Reducible	102.5
6		47	60	Reducible	75.7
7		47	125	Unreducible	124
8		25	30	Unreducible	31
9		30	150	Unreducible	175
10		340	100	Unreducible	385
11		70	175	Unreducible	181
12		30	270	Unreducible	390
13		31	53	Unreducible	53
14		150	100	Unreducible	195
15		80	150	Unreducible	215

LINE LOAD SCHEDULE			
LABEL	DL (k/ft)	MASS DL (k/ft)	LL
L1	.3	.3	
L2	.56	.42	
L3	.36	.36	
L4	.5	.5	
L5	.225	.225	
L6	.4	.4	

SNOW DRIFT LOAD SCHEDULE						
LABEL	PATTERN	DL (psf)	MASS DL (psf)	MIN SL (psf)	MAX SL (psf)	WIDTH (ft)
D1		25	31	30	58	8.2
D2		25	31	30	83	14.25
D3		25	31	30	75	12.32
D4		25	31	30	64	9.8
D5		25	31	30	50	6.2
D6		25	31	30	78	13
D7		25	31	30	108	20