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Structural Option
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December 3th, 2007

Technical Report 3

Executive Summary

This report provides a detailed description and analysis of the existing lateral force resisting system at the Heart Hospital. Swedish American Hospital recently completed construction of the new Heart and Vascular Center, also known as the Heart Hospital. This structure is designed as a 7 story patient facility located in Rockford, Illinois. Although the building was designed as a 100' tall building, it currently only stands 4 stories tall with mechanical units on the roof enclosed by a mechanical screen wall. The final phase of construction would be to enclose the current roof into a 5th floor mechanical space and complete the remaining two stories.

The existing gravity structural system makes use of composite action between rolled wide flange beams and concrete with metal deck. The two are connected by shear studs welded to the beams and embedded in the concrete. Typical interior spans are 32'-0" with shorter spans found towards the perimeter of the building, typically either 18'-0" or 22'-7". Typical beam sizes range from W12x14's to W27x146. The smaller W12's and W16's are found at the shorter 18'-0" and 22'-7" spans. The larger W18's and W21's are designed for the 32'-0" spans, or the shorter spans with heavier concentrated loads. The largest beams, W27x146, are part of the lateral framing system (moment frames). These members are connected by Bolted Flange Plate moment connections to W14x176 columns.

This report provides an in depth analysis of the lateral framing system used in the design of the Heart Hospital. A computer model was created in ETABS to thoroughly analyze the structure's response to lateral loads from wind and seismic forces. Only the lateral framing elements were modeled in the program and were connected together at each story level by a rigid diaphragm. In the model, columns were assumed to be pinned at the base. This is a conservative assumption to approximate the largest building drift and was also assumed by the design engineer. Results from the ETABS analysis are compared to hand calculations and other assumptions made at various points around the structure.

From the computer model analysis in ETABS, building drifts of 3.4" and 3.6" were calculated due to wind forces, whereas, a max design story drift of 11.5" was calculated for seismic forces (per ASCE 12.8.6). The design story drift due to seismic forces is lower than the allowable drift set by code, but the building drifts resulting from wind pressures exceed the assumed value of H/400. However, the H/400 limit is only based on engineering judgment and not set by code. The wind drifts produce ratios of roughly H/353 and H/333 respectively and are still considered acceptable for serviceability.

When looking at the lateral framing, there are eight total moment frames; four frames acting in each main direction. Also, six of the eight moment frames are positioned around the perimeter of the central framing core to help reduce torsional effects. Before running the ETABS analysis, it was assumed that each of the 4 frames in one direction took approximately 25% of the total base shear. Results from the ETABS analysis calculated the base shear of each of the frames, based on relative stiffness, and found that the results confirmed that assumption. Actual base shears of the moment frames were reasonably close to the estimated 25%.

Overall, this report assumes reasonable loads and distributions that are proven by calculations, and are comparable to those used in the original design by the design engineers.

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Structural System Overview

Introduction:

The Swedish American Hospital, located in Rockford, IL, is phase 2 in a 3 phase construction project on the Swedish American Health Center. Phase 2 ended with the completion of the 4 story Heart Hospital (see figure above). The Heart Hospital is designed for a total of 7 floors of patient wings based on a Certificate of Need for the city of Rockford and the surrounding areas. Phase 3 of the construction process is to frame in the existing roof of the Heart Hospital creating a 5th floor (functioning as a mechanical floor) and continue on to complete the 6th and 7th floors above. This report will analyze the lateral framing of the initial 7 story design.

Floor System:

The typical building floor framing system is made up of beams and girders acting compositely with a concrete floor slab. Floor sections show 3"-20 gauge LOK Floor galvanized metal deck with 3¼" of lightweight concrete (110 pcf) resting on the steel framing below. Composite action is achieved through 5" long ¾" diameter shear studs welded to the steel framing. Concrete is reinforced with 6x6-W5xW5 welded wire fabric. The span of the metal deck varies depending on the bay location. However, the direction is limited to east-west or northeast-southwest. This assembly has a 2 hour fire rating without the use of spray on fireproofing.

There is no "typical" bay in the structural framing system. However, columns located on the wings are spaced approximately 22'-7 ½" on center. Columns in the interior core area are spaced approximately 32'-0" on center with additional columns located around the core perimeter framing into the wings. The most common and longest span is 32'-0". Typical beam sizes range from W12x14's (typically spanning 10' to 12') to W27x146 (spans ranging from 22' to 32') with the larger beams acting as part of the moment framing system.

Roof System:

The roof framing system is very similar to the building floor framing system. Composite design is still used with 3 ¼" of lightweight concrete and 3"-20gauge LOK Floor metal deck on top of steel framing. Deeper steel beams and girders are used to help carry the heavier loads of the mechanical equipment on the roof.

The lobby roof is slightly different from the typical roof framing. It uses composite action but has a 1 ½" deep 20 gauge metal deck spanning north-south instead of the 3" metal deck used elsewhere on the building. Lower portions of the roof that see a heavier snow loads due to drift use a 3" deep 20 gauge metal deck.

Lateral System:

The lateral load resisting system consists of steel moment frames. The majority of the moment frames extend around the perimeter of the building with a few added moment frames on the interior to help stiffen the structure. Larger girders are framed into columns with bolted flange plate moment connections. The prefabricated steel pieces were bolted in place rather than welded to eliminate the need of preheating for welds. Shear walls were not part of the original design analysis; therefore, masonry cores such as the elevator and stairwell cores were not assumed to provide lateral support during the structural analysis.

Foundation:

The basement footprint is approximately one half of the square footage of the first floor plan. Hence, there are two slabs on grade: one for the basement and one for part of the first floor. Each slab on grade is 5" thick normal weight concrete (145pcf) with 4x4-W5xW5 welded wire fabric reinforcement.

Interior steel columns rest on spread footings with an allowable soil bearing capacity of 4ksf. Exterior columns and basement walls rest on continuous strip footings. Reinforced concrete pilasters are located where exterior columns rest on the basement wall. Footings below columns in the interior core area extend approximately 18' deep whereas the perimeter strip footings and footings located beneath the wings extend approximately 8' deep. All footings are required to extend a minimum of 4' deep for frost protection.

Columns:

Columns are laid out on two different intersecting grids: one running east-west and the other running northwest-southeast. All columns are ASTM A992 Grade 50 wide flange steel shapes. Columns are spliced between the 3rd and 4th floor. Columns acting as part of a moment frame are spliced 5'-6" above the 3rd floor elevation. Columns acting only as gravity columns are spliced 4'-6" above the 3rd floor elevation. All interior columns that extend to the basement level are also spliced 5'-6" above the 1st floor elevation. Future columns for the 6th and 7th floors are designed to be spliced with existing columns at the 5th floor elevation (current mechanical floor and roof).

Codes

Original Design Codes:

- International Building Code (IBC) 2003
  - with City of Rockford, IL amendment
- American Society of Civil Engineers (ASCE)
  - ASCE 7-02 - Minimum Design Loads for Buildings and Other Structures
- American Concrete Institute (ACI)
  - ACI 318-02 - Building Code Requirements for Structural Concrete
  - ACI 530-02 – Building Code Requirements for Masonry Structures
- American Institute of Steel Construction (ASIC)
  - LRFD 1999 - Load and Resistance Factor Design Specification for Structural Steel Buildings
  - AISC 341-02 – Seismic Provisions for Structural Steel Buildings

Thesis Design Codes:

- International Building Code (IBC) 2006
- American Society of Civil Engineers (ASCE)
  - ASCE 7-05 - Minimum Design Loads for Buildings and Other Structures
- American Concrete Institute (ACI)
  - ACI 318-05 – Building Code Requirements for Structure Concrete

Material Strengths

Concrete:

Normal Weight Concrete (columns, walls, foundations, slabs on grade)……...…4000psi
Light Weight Concrete (floor slabs on metal deck)………………………………4000psi
Reinforcement ……………………………………………………………………60ksi

Structural Steel:

Wide Flanges and Channels ……………………………………………………...50ksi
Angles, Bars and Plates…………………………………………………………...36ksi
Hollow Structural Sections (HSS)…………………………………………….. 46ksi
Bolts (A325X or A490X)…………………………………………………….3/4”dia
Shear Studs (5”long)………………………………………………………… 3/4”dia

Masonry:

Design Strength (F'_m)………………………………………………………….2000psi
Block……………………………………………………………………………...4000psi

Framing Plans

Lateral Framing System:

Swedish American Hospital's new Heart and Vascular Center is laterally supported with steel moment frames. The frames are designed to resist wind and seismic loads. Pieces of these frames are prefabricated then bolted together onsite. Bolted Flange Plate connections are used in place of welded connections (see Appendix A for details). Bolted connections eliminate the need for preheating steel for welded connections. Since steel erection began in mid February, eliminating the need for preheating helped speed up the erection process and keep the project on schedule.

The majority of the moment frames lie around the perimeter of the building, with some interior moment frames added to help stiffen the structure and reduce drift. Exterior moment frames help resist any torsion the structure might experience. Also, less interior moment frames help reduce the required depth of steel in interior spaces to minimize conflicts with HVAC systems. Moment frames allow for a more open architectural floor plan. Swedish American uses their open floor plan to help increase the amount of natural light that reaches their interior spaces. Braced frames and shear walls could create potential problems with door and window openings. All frames are assumed to be pin-supported on spread footings and concrete piers at the basement level. Assuming pinned connections at the base is a conservative assumption commonly used when analyzing a structure.

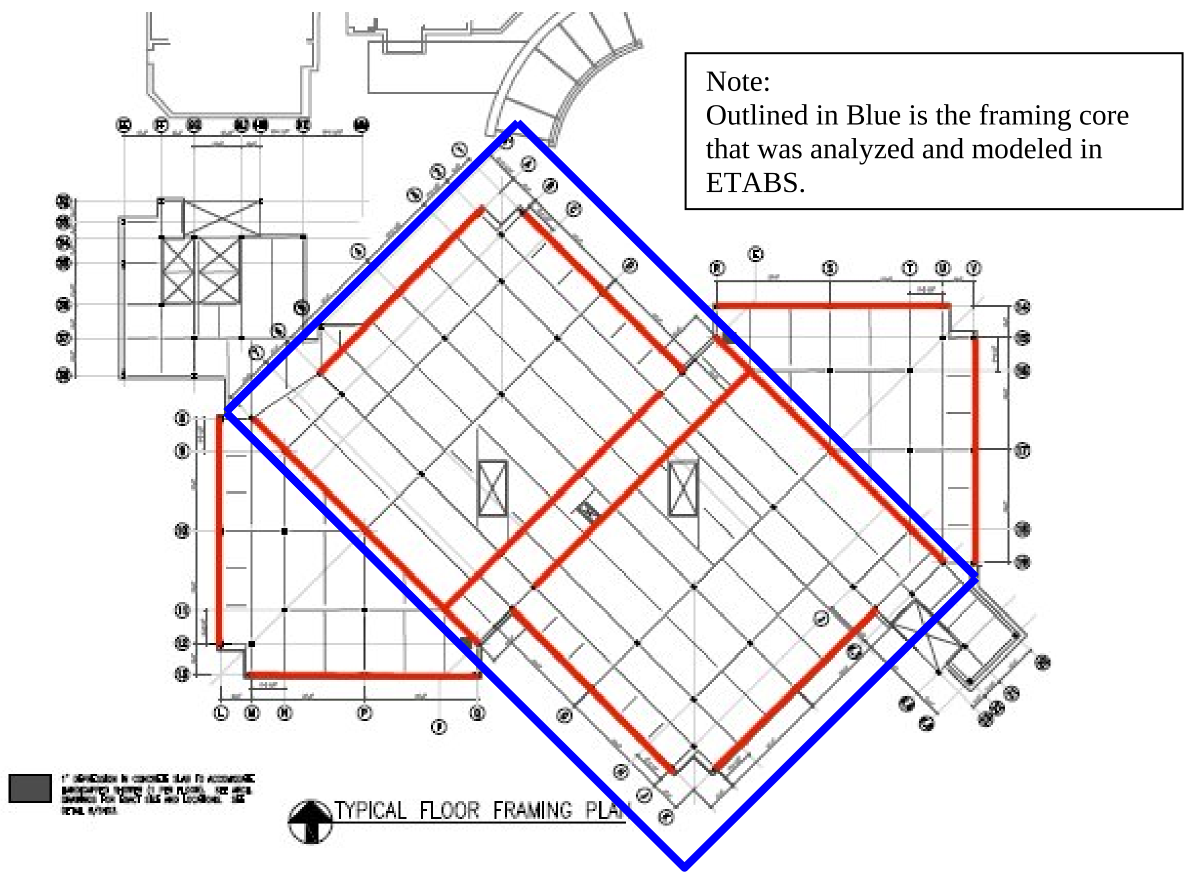

Figure 1: Framing Plan with Highlighted Moment Frames
*Typical moment frames are outlined in RED.

Lateral Loads

Wind Forces:

For wind pressures, the windward pressure acting along the height of the structure is in the form of a parabolic curve. A conservative assumption is to break the curve into a rectangular grid and find the effective pressure acting on an individual story. Windward pressures are calculated using equation 6.19 in ASCE 7-05 Section 6 (see the Wind Design Load Tables below and Appendix B for wind story shears, diagrams, and gust factors). Leeward pressure is assumed to be a constant along the back of the building and calculated using the total building height. Wind pressures are calculated in two main directions (usually acting perpendicular to the building face). Base shears resulting from wind for the Heart Hospital were 1045k (N-S direction) and 703k (E-W direction). Included in these values is a load factor of 1.6 for the applicable load combinations.

Level	Total Height	K_z	q	Wind Pressures (psf)					
				N-S Windward	N-S Leeward	N-S Side Wall	E-W Windwar d	E-W Leeward	E-W Side Wall
Roof	99.17	1.26	25.54	21.86	-10.79	-15.10	22.02	-8.71	-15.25
7	85.83	1.225	24.83	21.38	-10.79	-15.10	21.54	-8.71	-15.25
6	72.50	1.18	23.92	20.76	-10.79	-15.10	20.92	-8.71	-15.25
5	52.5	1.1	22.30	19.67	-10.79	-15.10	19.81	-8.71	-15.25
4	39.17	1.04	21.08	18.84	-10.79	-15.10	18.98	-8.71	-15.25
3	25.83	0.94	19.05	17.47	-10.79	-15.10	17.60	-8.71	-15.25
2	12.5	0.85	17.23	16.24	-10.79	-15.10	16.35	-8.71	-15.25

Level	Eff. Height	**Wind Design (NS - EW)**							
		Load (kips)		Shear (kips)		Moment (ft-k)		Factored Load (1.6W)	
		N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
		230'	165'	230'	165'	230'	165'	230'	165'
Roof	6.67	50	34	0	0	4963	3352	80	54
7	13.33	99	67	50	34	8528	5757	159	107
6	16.67	122	82	149	101	8853	5969	195	132
5	16.67	119	80	272	183	6239	4198	190	128
4	13.33	92	62	390	263	3608	2422	147	99
3	13.33	89	59	482	325	2292	1534	142	95
2	12.92	82	55	571	384	1027	684	131	88
1	6.25	39	26	0	0	0	0	62	41
Total	99.15	653	439	653	439	35509	23916	1045	703

Table 1: Effective Wind Pressures and Story Shears

Seismic Forces:

For seismic loading, the total base shear is calculated using ASCE 7-05 Sections 11 and 12 (see Seismic Load Table below and seismic load calculations in Appendix C). The Heart and Vascular Center has a base shear of approximately 518k. This base shear is divided over the entire story height and based on the height and weight of each story over the entire height and weight of the structure. This effective story shear is assumed to be taken at the floor level of each story. The story shear at the lowest level is small but increases with height for the building.

Seismic load table

Item	Design Value	Code Reference
Occupancy Category	IV	ASCE 7-05 Table 1-1
Site Class	D	* From Geotechnical Report
Spectral Acceleration for Short Periods (S_s)	0.17g	* From Geotechnical Report
Spectral Acceleration for One Sec. Periods (S_1)	0.06g	* From Geotechnical Report
Damped Design for Short Periods (S_{ds})	0.1813g	ASCE 7-05 Section 11.4.4
Damped Design for One Sec. Periods (S_{d1})	0.096g	ASCE 7-05 Section 11.4.4
Seismic Design Category	C	ASCE 7-05 Section 11.6.1.1
Seismic Force Resisting System	Ordinary Steel Moment Frames	ASCE 7-05 Table 12.2-1
Response Modification Factor (R)	3.5	ASCE 7-05 Table 12.2-1
System Overstrength Factor (Ω)	3.0	ASCE 7-05 Table 12.2-1
Deflection Amplification Factor (C_d)	3.0	ASCE 7-05 Table 12.2-1
Importance Factor	1.5	ASCE 7-05 Table 11.5-1
Approximate Period (Ta)	1.106	ASCE 7-05 Section 12.8.2.1
Upper Limit Period (CuTa)	1.7(Ta) = 1.88	ASCE 7-05 Section 12.8.2
Seismic Response Coefficient (Cs)	0.0219	ASCE 7-05 Section 12.8.1.1
Building Mass	23,650k	* From Massing Calcs
Design Base Shear	518k	

Table 2: Seismic Load Table

Vertical Distribution of Seismic Loads
ASCE 7-05 Section 12.8.3

Level	h (in ft)	W in kip	$w_x h_x^k$	C_{vx}	Fx
8th floor (future roof)	99.17	2568	6073655	0.27	142.21
7th floor (future)	85.83	2977	5516381	0.25	129.16
6th floor (future)	72.50	3376	4702945	0.21	110.11
5th floor mechanical	52.50	4047	3267362	0.15	76.50
4th floor	39.17	3091	1520994	0.07	35.61
3rd floor	25.83	3342	813938	0.04	19.06
2nd floor	12.50	3200	228522	0.01	5.35
1st floor	0.00	1049	0	0.00	0.00
		23650	22123797		518

Table 3: Effective Story Shears due to Seismic Loading

Design Base Shear (V) = 518 k
k (by interpolation) = 1.69

These results show that wind controls in both the North-South direction (also referred to as the “x” direction in the ETABS model) and in the East-West direction (referred to as the “y” direction in the ETABS model).

Load Cases

1) 1.4(D + F)
2) 1.2(D + F + T) +1.6(L + H) + 0.5(Lr or S or R)
3) 1.2D + 1.6(Lr or S or R) + (L or 0.8W)
4) 1.2D + 1.6W + L + 0.5(Lr or S or R)
5) 1.2D + 1.0E + L + 0.2S
6) 0.9D + 1.6W + 1.6H
7) 0.9D + 1.0E + 1.6H
8.) 1.0D + 1.0L + 1.0E

***** Load Cases are taken from ASCE 7-05 Chapter 2.

ETABS Model

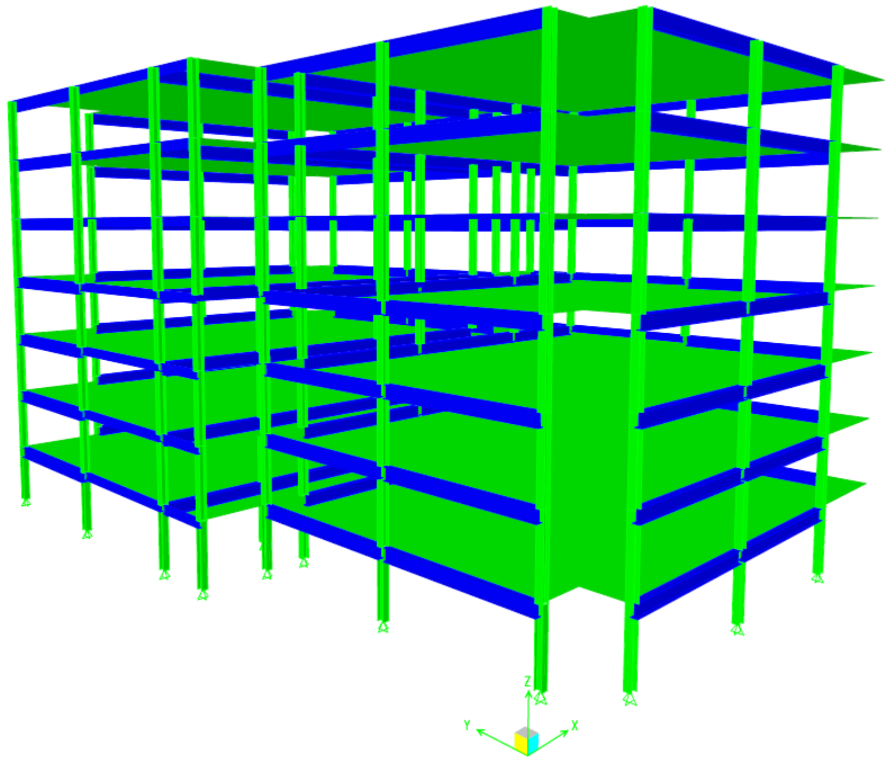

Figure 2: 3D Model of building with Floor Diaphragms

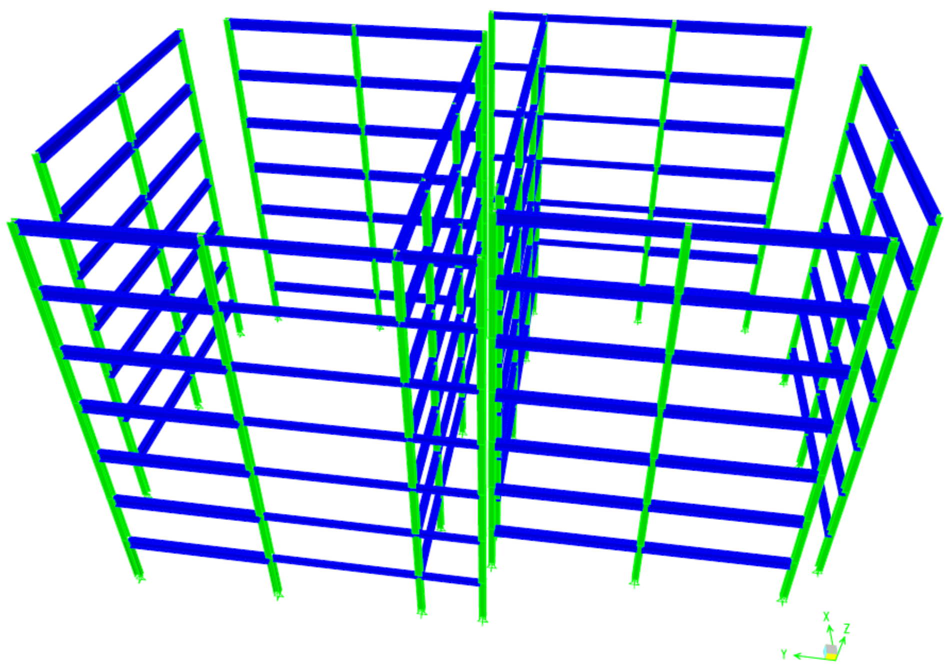

Figure 3: 3D Model of building without Floor Diaphragms

ETABS Model

The analysis of the lateral framing elements was completed using ETABS. A model of only the lateral framing elements was input into the program. This included all of the exterior moment frames and the two interior moment frames located along column lines E and F on the structural plans (column lines 6 and 5 respectively in the ETABS Model). A rigid diaphragm was assigned to each floor to connect all of the elements at that location. This enabled all the members at each floor to act together. Assigning a rigid diaphragm to each floor is acceptable based on ASCE 7-05 12.3.1.2. (see Appendix K for a floor plan of the ETABS Model)

The lateral loads from the wind and seismic forces are applied to each corresponding floor using the story forces calculated in their respective analyses (see Table 1 for the calculated wind story forces and Table 3 for the calculated seismic story forces). These story forces are imposed on the model as a static representation of the dynamic loads to achieve the same result. A load factor of 1.6 is assigned to the story forces in the wind analysis. This load factor is based on the load cases listed above on page 9. Similarly, building mass is assigned to the floor diaphragm at each level to account for the weight of the building. For Load Cases 6 and 7 listed above, a load factor of 0.9 is applied to the assigned mass of each diaphragm.

When calculating the story drift and building drift, only the service loads are applied to the structure, no load factors are assigned to the lateral forces because it is a serviceability calculation.

Load Distribution

Following ASCE 7-05 Chapter 6, hand calculations were used to determine the story shear forces for the wind pressures acting on both normal directions of the building. These story shear forces were then inserted into ETABS at the center of mass of each floor diaphragm. After running the analysis, the shear in each frame just above the base level was obtained and compared to the total shear force due to the wind pressures.

In the North-South direction (x-direction), each of the exterior moment frames (on column lines A and K on the structural plans, elevations 1 and 10 in the model) took 195.2k at the base. Each of the interior frames (on column lines E and F, elevations 5 and 6 in the model) took 271.4k. The 195.2k is approximately 18.7% of the total base shear, while the 271.4k is approximately 26% of the total base shear, 1045k.

In the East-West direction (y-direction), the all of the moment frames are situated along the exterior of the building (column lines 8, 7, 1, 20; elevations A, B, H, I in the model respectively). Each of the shorter frames, on column lines 7 and 1 (B and H), took 156.8k at the base. Each of the remaining two frames took 171.7k at the base. The 156.8k is approximately 22.3% of the total base shear, while the 171.7k is approximately 24.4% of the total base shear, 703k. (See Appendix D for detailed results of the load distribution to various frames and Appendix J for a floor plan of the building submitted by the structural engineer of record)

In both cases, the sum of the base shears adds up to roughly 90% of the total base shear. The remaining shear is accounted for in the columns that are not part of the lateral framing in that direction.

For a quick check of the load distribution, it is reasonable to assume each frame takes approximately 25% of the total base shear. This is because all of the beams and columns are the same size members (W27x146 beams and W14x176 columns) and the floor diaphragms are assumed to be rigid. Also, the typical moment frame includes 2-32' spans. The only exceptions are the interior North-South frames on column line E and F (elevations 5 and 6) and the East-West frames 8 and 20 (elevations A and I). Each of these four frames has an extra span. This 3rd span on the North-South frames extends an additional 22'-9". On the East-West frames, the 3rd span extends an additional 13'-4".

Since the 3rd spans on the North-South frames are longer than the extra spans on the East-West frames, it can be assumed that the two N-S frames will take a higher percentage of the base shear in that direction compared to the percentage of the base shear the E-W frames take in their direction. This assumption is confirmed when compared to the results given by ETABS. Also, as expected all four of the longer frames a larger percentage than the remaining typical moment frames acting in their respective directions.

Finally, since all except two of the moment frames are located around the perimeter of the building, it can be assumed that these perimeter frames will play a significant role in minimizing the torsion of the building.

Results

Drift:

After calculating the base shears for both the seismic forces and wind pressure forces, it is apparent that the wind pressures control in both normal directions of the building. The base shears are calculated are V=1045k in the North-South direction (x-direction in the model) and V=703k in the East-West direction (y-direction in the model). In the North-South direction, the wind story shears produce a maximum drift of 3.4" at the roof level (h=99.17'). In the East-West direction the story shears produce a maximum drift of 3.6" at the roof level. (see Appendix E for a list of the drifts calculated by the ETABS model)

Using an acceptable drift approximation of H/400, the acceptable drift is 3.0". The actual drifts of 3.4" and 3.6" have drift ratios of approximately H/353 and H/333 respectively.

To quickly check the seismic drift, an ETABS analysis shows that the roof of the building drifts a total of 5.7" in the x-direction and 3.5" in the y-direction. Following ASCE 7-05 12.8.6, the design story drift defined by the code is 11.4" in the x-direction and 7" in the y-direction. This is compared to the allowable story drift given by the equations in Table 12.12-1. The allowable story drift given for an Occupancy Category IV is 12". Both of the design drifts are below the allowable drift, therefore, the drift due to seismic forces meets code. (see Appendix F for drift calculations due to seismic forces)

Story Drift:

Assuming the lateral columns are pinned at the base (as assumed by the design engineer) the maximum story drifts due to wind shear forces are found between the base and 1st floor. The story drifts determined in the ETABS analysis are 1.25" in the x-direction and 1.02" in the y-direction. Again, using H/400 as an acceptable approximation of story drift, a value of 0.4 is calculated as an acceptable drift. (see Appendix D for a list of the story drifts)

This acceptable value is more than 3 times smaller than the larger calculated story drift. However, the connection detail for the base of the columns in the lateral system shows a 2” thick base plate with (12) 1¼” dia. anchor rods embedded 2’ into the concrete footing below. This stiff connection will provide some rigidity and help limit the story drifts at the base of the building, even though they are not considered moment connections in the analysis. (see Appendix G for a detail of the column base plate connection)

Overturning Moments:

When analyzing the wind story forces, a maximum overturning moment of 56815 ft-k is calculated acting in the North-South (x) direction. This moment is a result of wind pressures with a 1.6 load factor acting as story forces up the face of the building. A quick calculation to check the impact of the overturning moment is to multiply the weight of the building (23650k) by the distance from the center of mass to the edge of the lateral frame. Assuming that the center of mass is centered roughly in the middle of the building, the shortest moment arm of 32’ yields a resisting moment of 756800 ft-k. This is more than 13 times the value of the overturning moment due to wind. (see Appendix H for detailed hand calculations)

Similarly, the overturning moment due to seismic story forces is 39142 ft-k. By inspection, it is less than the overturning moment due to wind and therefore, will not control. Since the overturning moments due not control, there is no significant uplift on the lateral members. Therefore, there is a minimal impact on the existing foundations due to overturning.

Torsion:

Eight moment frames are utilized in the central framing core of the Heart Hospital to resist the lateral loads impacting the building. Six of the eight frames are used to frame in the perimeter of the framing core. These six frames are all part of lateral systems working to oppose the lateral loads in each of the two normal directions. However, all six frames also work together to resist any torsion acting on the structure.

All of the moment frames have relatively the same stiffness, as proven earlier on page 11 with the load distributions. Therefore, assuming a rigid diaphragm is connecting all of the frames together, there are 6 frames working together to resist any torsion on the building compared to only 4 frames working in either normal direction (x or y directions). Thus, it should be reasonable to assume that the lateral loads applied in the normal directions (acting on only 4 moment frames) will be the governing design criteria when compared to the torsional loads acting against 6 moment frames. Further analysis of torsional effects on the structure can be explored in detail during future investigations.

When analyzing torsion due to seismic forces, a torsional moment (M_t) is a result of accidental torsion defined by ASCE 12.8.4.2. This moment is taken into account in the ETABS model when the static seismic load cases are defined. For the Heart Hospital, the torsional moment (M_t) must be modified by a torsional amplification factor (A_x) per ASCE 12.8.4.3 because the structure falls under seismic design category C.

Lateral wind pressures are the controlling factor for lateral systems in the Swedish American Heart Hospital. ASCE 7-05 Equation 6-21 and Figure 6-9 are used to analyze the torsional effects of the resulting wind loads. Equation 6-21 provides a method to calculate the eccentricity (e) for a flexible diaphragm and Figure 6-9 lists 4 load cases that must be checked when analyzing torsion. (see Appendix I for the 4 load cases listed in Figure 6-9). Wind pressures and torsional moments from these load cases can then be applied to the building. Using a modeling program, like ETABS, torsional loads can be input into the model and the corresponding design moments and shears can be calculated in the members of the structure.

Conclusion

The building's response to seismic and wind loads was able to be determined using a thorough analysis of the lateral framing system at Swedish American Heart Hospital. The lateral framing was modeled in ETABS using user specified loads taken from the wind and seismic calculations performed by hand. Only the lateral framing elements were used in the model and connected at each floor using a rigid diaphragm. Lateral loads, calculated following ASCE 7-05, were input as story shears acting at the center of mass on each floor and used in the analysis of the lateral framing elements.

The columns and beams are sized heavy based on the shears and moments from ETABS and the hand calculations. However, these larger sizes could be attributed to the engineer of record wanting to maintain a uniform column and beam size if a larger size was required in a few localized areas. Also, additional dead load from the exterior façade bears on the exterior beams and columns and is not accounted for in the ETABS analysis. This could cause higher shears and moments in the members requiring larger sizes.

Spot check calculations done by hand confirmed the analysis and output from ETABS and prove the assumptions made by the design engineer were correct (see Appendix J for spot check hand calculations on beam and column framing members). One assumption that was not confirmed was an acceptable wind drift ratio of H/400. Drift calculations yielded results of 3.4" and 3.6" at the top of the building which have ratios of H/353 and H/333 respectively. These ratios do not meet the H/400 assumption; however a wind drift ratio of H/400 is not mandatory by code and the deflections of 3.4" and 3.6" are reasonably close for serviceability.

Allowable drift calculations for seismic loads were completed following ASCE 7-05 section 12.8.6. The story drifts calculated in the ETABS model and modified by ASCE Equation 12.8-15 were less than the allowable drift required by code and confirmed the design assumptions made by the engineer. Therefore, based on this lateral analysis, it is reasonable to assume that the loads and distributions calculated in this study are comparable to those used in the original design by the design engineers.

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December 3rd, 2007

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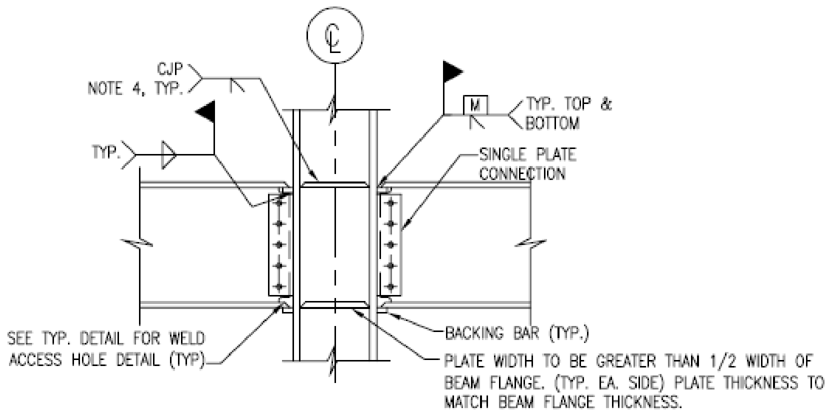
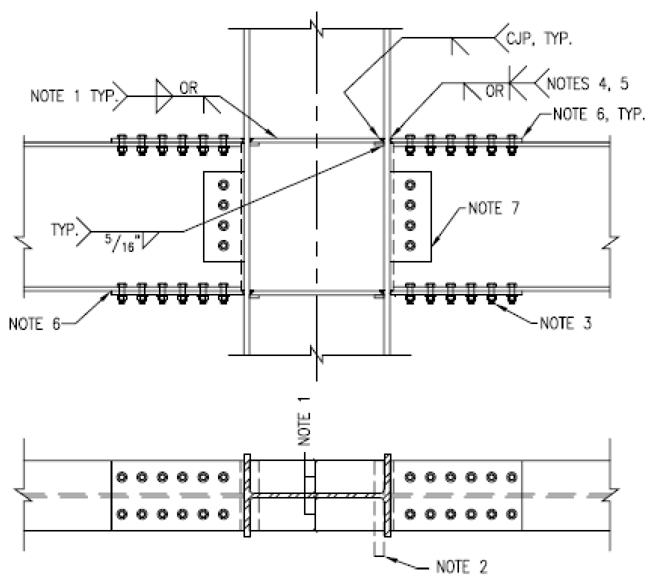
Appendix A

Typical and Alternate Beam to Column Moment Connections



NOTES:
1. MOMENT CONNECTION TO DEVELOP THE FULL CAPACITY OF THE BEAM.
2. SHEAR CONNECTION TO DEVELOP THE LOWER OF THE FACTORED REACTION LISTED IN THE SCHEDULE + 2.4(M_P/SPAN), OR 80% OF THE UNREDUCED SHEAR STRENGTH OF THE COPED WEB. BOLTS ARE SLIP CRITICAL.
3. INSTALL ALL BOLTS SNUG TIGHT PRIOR TO FIRST TORQUING. TENSION BOLTS FULLY PRIOR TO WELDING.
4. EDGE PREPARATION OF CONTINUITY PLATES AT FABRICATORS DISCRETION. BACK GOUGE ROOT PASS OF DOUBLE BEVEL GROOVE WELDS.
5. REMOVE BOTTOM FLANGE BACKING BAR, BACK GOUGE, AND INSTALL REINFORCING 5/16" FILLET ON TOP & BOTTOM OF BOTTOM FLANGE WELD.
6. WELD TOP FLANGE BACKING BAR CONTINUOUSLY TO COLUMN FLANGE OR CONTINUITY PLATE.
7. ADD A 5/16" REINFORCING FILLET TO TOP OF TOP FLANGE WELD.

TYPICAL BEAM-TO-COLUMN MOMENT CONNECTION DETAIL (STRONG AXIS)



BOLTED MOMENT CONNECTION NOTES:
1. MINIMUM WIDTH TO MATCH BEAM FLANGE, THICKNESS TO MATCH OR EXCEED FLANGE OF THICKER BEAM FLANGE AT CONNECTION.
2. REMOVE WELD TABS TO 1/4" MAXIMUM FROM EDGE OF CONTINUITY PLATE. GRIND END OF WELD SMOOTH, NOT FLUSH; DO NOT GOUGE COLUMN FLANGE.
3. ALL BOLTS PRETENSIONED; DESIGNED FOR BEARING. BOLT HOLES IN FLANGE PLATE ARE OVERSIZED, BOLT HOLES IN BEAM FLANGE ARE STANDARD.
4. SHOP WELD: WHEN USING SINGLE BEVEL PREPARATION, REMOVE BACKING AFTER WELDING, BACKGAUCE, AND REINFORCE WITH 5/16" MIN. FILLET WELD.
5. WHEN USING DOUBLE BEVEL PREPARATION, BACKGOUGE FIRST WELD BEFORE WELDING OTHER SIDE.
6. SHIMS BETWEEN BEAM FLANGE AND FLANGE PLATES ARE ALLOWED. USE FULL COVERAGE SHIM PLATES OR FULL DEPTH FINGER SHIMS.
7. HOLES IN SHEAR TABS ARE SHORT SLOTTED HORIZONTAL HOLES
HOLES IN BEAM WEB ARE STANDARD.
8. MOMENT CONNECTION SHALL DEVELOP THE FULL CAPACITY OF THE BEAM.

TS-1D ALTERNATE BOLTED STRONG AXIS BEAM-TO-COLUMN MOMENT CONNECTION DETAIL 3/4"=1'-0"

Appendix B

MAIN WIND-FORCE RESISTING SYSTEM (ASCE 7-05)

Swedish American Hospital - Rockford, IL

Basic Wind Speed (V) mph	90
Exposure Category	C
Importance Factor (I)	1.2
Wind Directionality Factor (K_d)	0.85
Topographic Factor (K_{zt})	1.0
Building Period (T)	1.106

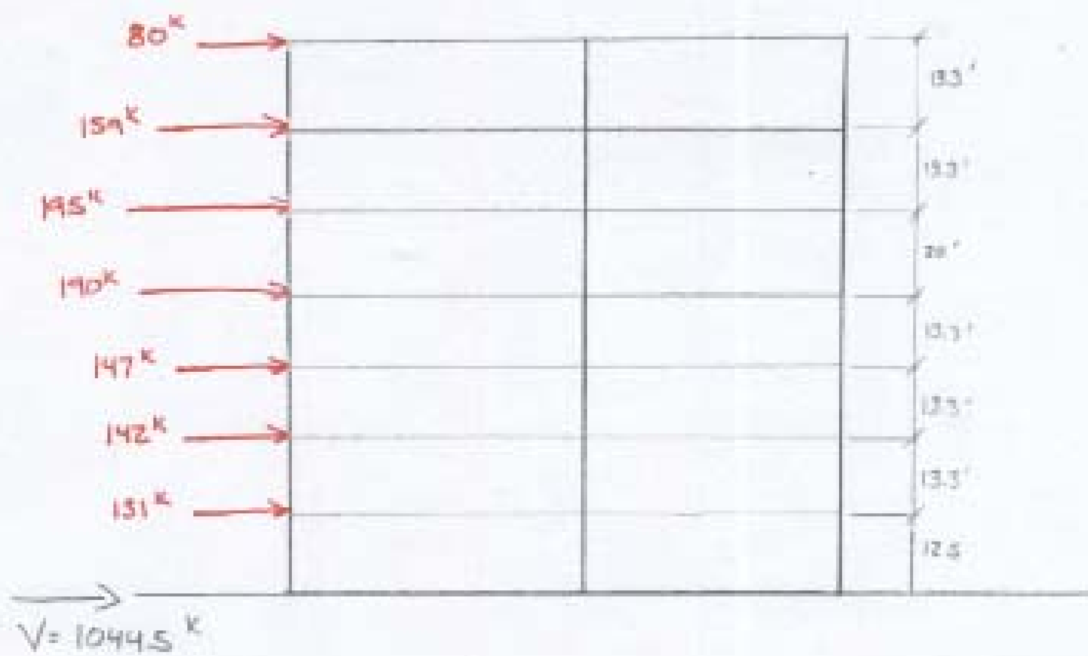
Number of Floors	7
Building Height (feet)	99.17
N-S Building Length (feet)	165
E-W Building Length (feet)	230
L/B in N-S Direction	0.7
L/B in E-W Direction	1.4

Gust Factor		
	Wind Direction	
Variable	N-S	E-W
Structure	Rigid	Rigid
B	230	165
L	165	230
h	99.17	99.17
z	59.502	59.502
ℓ	500	500
ε	0.2	0.2
α	0.15	0.15
b	0.65	0.65
β	0.2	0.2
L_z	562.6	562.6
c	0.2	0.2
I_z	0.18	0.18
g_q	3.4	3.4
g_v	3.4	3.4
Q	0.83	0.85
n_1	0.90	0.90
g_R	4.17	4.17
V_z	93.95	93.95
N_1	5.41	5.41
R_n	0.05	0.05
η_h	4.39	4.39
η_B	295293.52	211841.01
η_L	24.46	34.09
R_h	0.20	0.20
R_B	0.00	0.00
R_L	0.04	0.03
R	0.00	0.00
G_f	0.84	0.85

Wind Direction	$C_{p,\ windward}$	$C_{p,\ leeward}$	$C_{p,\ side\ wall}$	Gust Factor	Gcpi (+)	Gcpi (-)
N-S Direction	0.8	-0.5	-0.7	0.84	0.18	-0.18
E-W Direction	0.8	-0.4	-0.7	0.85	0.18	-0.18

Wind Pressure Story Shear Forces

WIND STORY SHEAR FORCES : X - DIRECTION
(FACTORED)

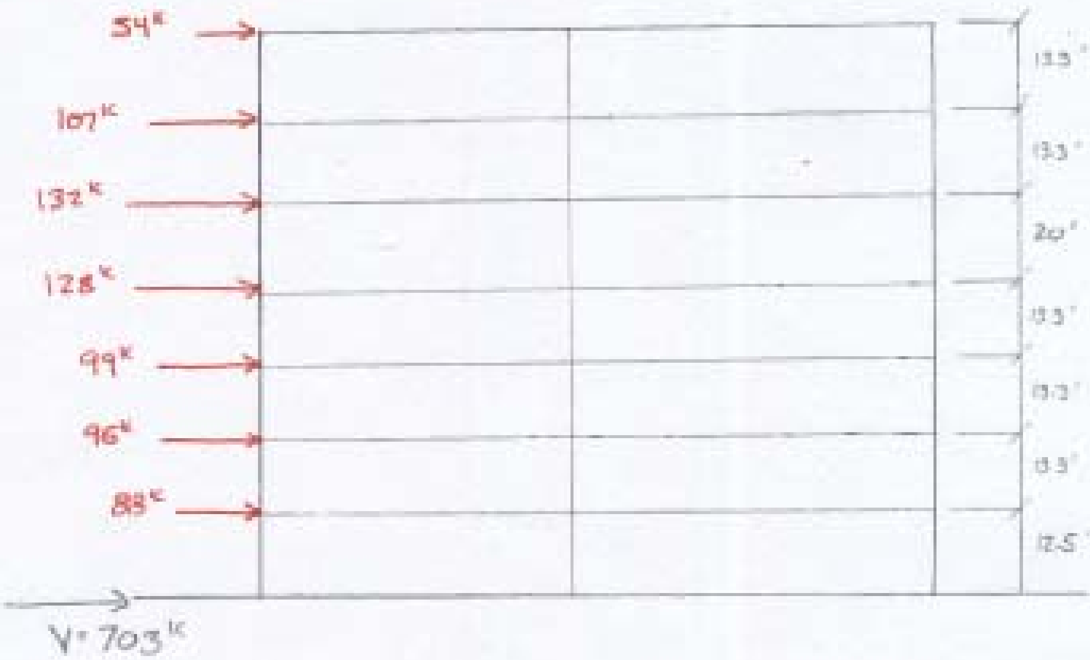


UNFACTORED STORY SHEARS

LEVEL	SHEAR (K)
ROOF (8)	50^K
7	99^K
6	172^K
5	119^K
4	42^K
3	89^K
2	82^K
	653^K TOTAL UNFACTORED

TOTAL FACTORED BASE SHEAR = 1044.5^K

WIND STORY SHEAR FORCES: Y - DIRECTION
(FACTORED)



UNFACTORED STORY SHEARS

LEVEL	SHEAR (K)
Roof (8)	34k
7	67k
6	82k
5	80k
4	62k
3	59k
2	55k

439k TOTAL UNFACTORED

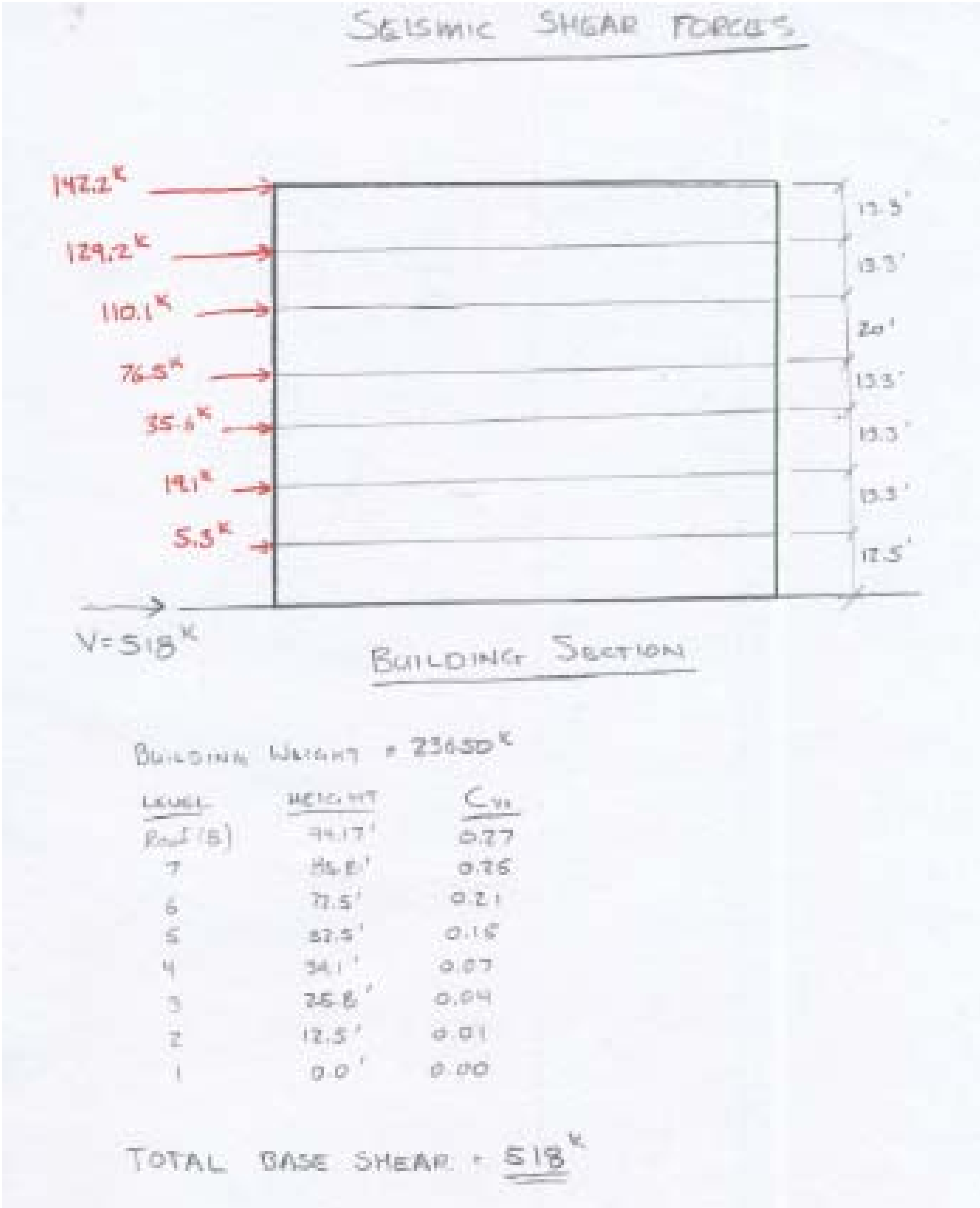
TOTAL FACTORED BASE SHEAR = 703k

Appendix C

Seismic Massing (Dead Loads)					
Level	Item	Quantity	Units	Unit Weight	Weight
				(ksf or klf)	(kip)
8th Floor (Future Roof)					
h = (ft)	6 ¼" LW conc. 3"-20g LOK-Floor deck	24900	sf	0.048	1195
99.17	Ponding	24900	sf	0.005	125
	Steel self weight	24900	sf	0.015	374
story height	Mechanical, electrical, floor/clg. Misc.	24900	sf	0.012	299
13.33	Snow	24900	sf	0.005	125
	Exterior wall - Precast	575	lf	0.566	325
	Exterior wall - Brick below	100	lf	0.375	38
	Exterior wall - brick above	100	lf	0.508	51
	Exterior wall - Aluminum curtain wall	175	lf	0.100	18
	Interior wall - Elevator core	100	lf	0.203	20
				Total (kips)	2568
7th Floor (Future)	6 ¼" LW conc. 3"-20g LOK-Floor deck	24993	sf	0.048	1200
h = (ft)	Ponding	24993	sf	0.005	125
85.83	Partition	24993	sf	0.015	375
story height	Steel self weight	24993	sf	0.012	300
13.33	Mechanical, electrical, floor/clg. Misc.	24993	sf	0.007	180
	Exterior wall - Precast	575	lf	1.133	652
	Exterior wall - Brick	100	lf	0.667	67
	Exterior wall - Aluminum curtain wall	175	lf	0.200	35
	Interior wall - Shaft	92	lf	0.267	25
	Interior wall - Elevator core	100	lf	0.200	20
				Total (kips)	2977
6th Floor (Future)	6 ¼" LW conc. 3"-20g LOK-Floor deck	24993	sf	0.048	1200
h = (ft)	Ponding	24993	sf	0.005	125
72.50	Partition	24993	sf	0.015	375
story height	Steel self weight	24993	sf	0.012	300
20.00	Mechanical, electrical, floor/clg. Misc.	24993	sf	0.007	180
	Exterior wall - Precast	575	lf	1.700	978
	Exterior wall - Brick	100	lf	1.000	100
	Exterior wall - Aluminum curtain wall	175	lf	0.300	53
	Interior wall - Shaft	92	lf	0.400	37
	Interior wall - Elevator core	100	lf	0.300	30
				Total (kips)	3376
5th Floor Mech	6 ¼" LW conc. 3"-20g LOK-Floor deck	24993	sf	0.048	1200
h = (ft)	Ponding	24993	sf	0.005	125
52.50	Steel self weight	24993	sf	0.015	375
story height	Mechanical, electrical, floor/clg. Misc.	24993	sf	0.012	300
13.33	Permanent Equipment	24993	sf	0.050	1250
	Exterior wall - Precast	575	lf	1.133	651
	Exterior wall - Brick	100	lf	0.667	67
	Exterior wall - Aluminum curtain wall	175	lf	0.200	35
	Interior wall - Shaft	92	lf	0.267	25
	Interior wall - Elevator core	100	lf	0.200	20
				Total (kips)	4047
4th Floor	Main Floor				
h = (ft)	6 ¼" LW conc. 3"-20g LOK-Floor deck	24993	sf	0.048	1200
39.17	Ponding	24993	sf	0.005	125
	Partition	24993	sf	0.010	250
story height	Steel self weight	24993	sf	0.015	375

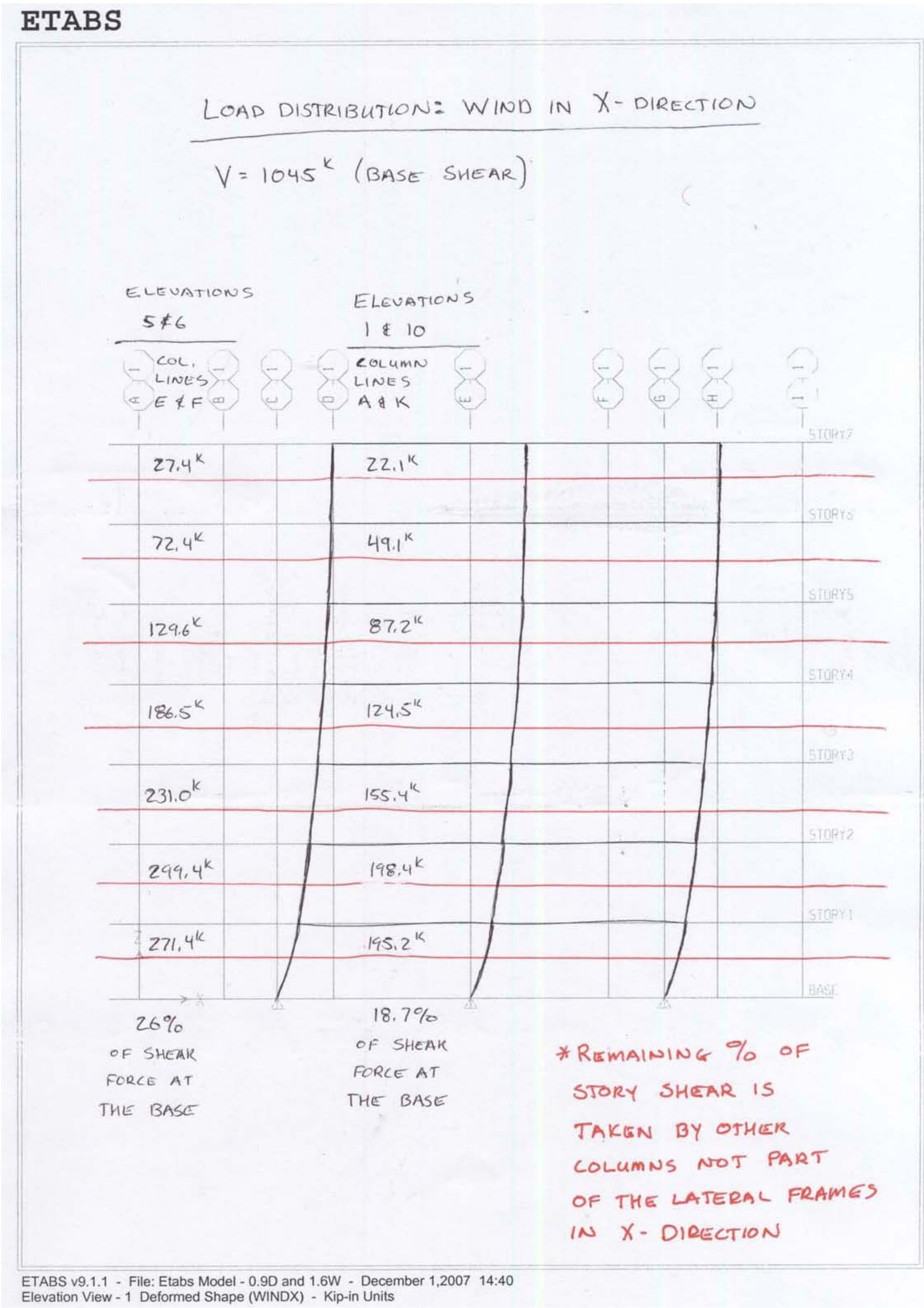
13.33	Mechanical, electrical, floor/clg. Misc	24993	sf	0.012	300
	Architectural Roof				
	6 ¼" LW conc. 3"-20g LOK-Floor deck	600	sf	0.048	29
	Ponding	600	sf	0.005	3
	Steel self weight	600	sf	0.015	9
	Snow	600	sf	0.005	3
	Exterior wall - Precast	575	lf	1.133	652
	Exterior wall - Brick	100	lf	0.667	67
	Exterior wall - Aluminum curtain wall	175	lf	0.200	35
	Interior wall - Shaft	92	lf	0.267	25
	Interior wall - Elevator core	100	lf	0.200	20
				Total (kips)	3091
3rd Floor	**Main Floor**				
h = (ft)	6 ¼" LW conc. 3"-20g LOK-Floor deck	24993	sf	0.048	1200
25.83	Ponding	24993	sf	0.005	125
	Partition	24993	sf	0.010	250
story height	Steel self weight	24993	sf	0.015	375
13.33	Mechanical, electrical, floor/clg. Misc.	24993	sf	0.012	300
	Lobby Roof				
	non composite roof	5300	sf	0.025	133
	Mechanical, electrical, floor/clg. etc	5300	sf	0.005	27
	steel self weight	5300	sf	0.015	80
	Snow	5300	sf	0.005	27
	Exterior wall - Precast	575	lf	1.133	652
	Exterior wall - Brick	100	lf	0.667	67
	Exterior wall - Aluminum curtain wall	325	lf	0.200	65
	Interior wall - Shaft	92	lf	0.267	25
	Interior wall - Elevator core	100	lf	0.200	20
				Total (kips)	3342
2nd Floor	**Main Floor**				
	6 ¼" LW conc. 3"-20g LOK-Floor deck	26283	sf	0.048	1262
h = (ft)	Ponding due to deck/purlin deflection	26283	sf	0.005	131
12.5	partition	26283	sf	0.010	263
	steel self weight	26283	sf	0.015	394
story height	Mechanical, electrical, floor/clg. etc	26283	sf	0.012	315
12.5	**Lobby Roof**				
	non composite roof	1170	sf	0.025	29
	steel self weight	1170	sf	0.015	18
	Mechanical, electrical, floor/clg. etc	1170	sf	0.005	6
	Snow (incl. drift)	1170	sf	0.005	6
	Exterior wall - Precast	575	lf	1.063	611
	Exterior wall - Brick	100	lf	0.625	63
	Exterior wall - Aluminum curtain wall	325	lf	0.188	61
	Interior wall - Shaft	92	lf	0.250	23
	Interior wall - Elevator core	100	lf	0.188	19
				Total (kips)	3200
1st Floor	6 ¼" LW conc. 3"-20g LOK-Floor deck	11650	sf	0.048	559
ground	Ponding due to deck/purlin deflection	11650	sf	0.005	58
h = (ft)	partition	11650	sf	0.010	117
0	steel self weight	11650	sf	0.015	175
	Mechanical, electrical, floor/clg. etc	11650	sf	0.012	140
				Total (kips)	1049
		Total Seismic Massing (kips) =			23650

Seismic Story Shear Forces

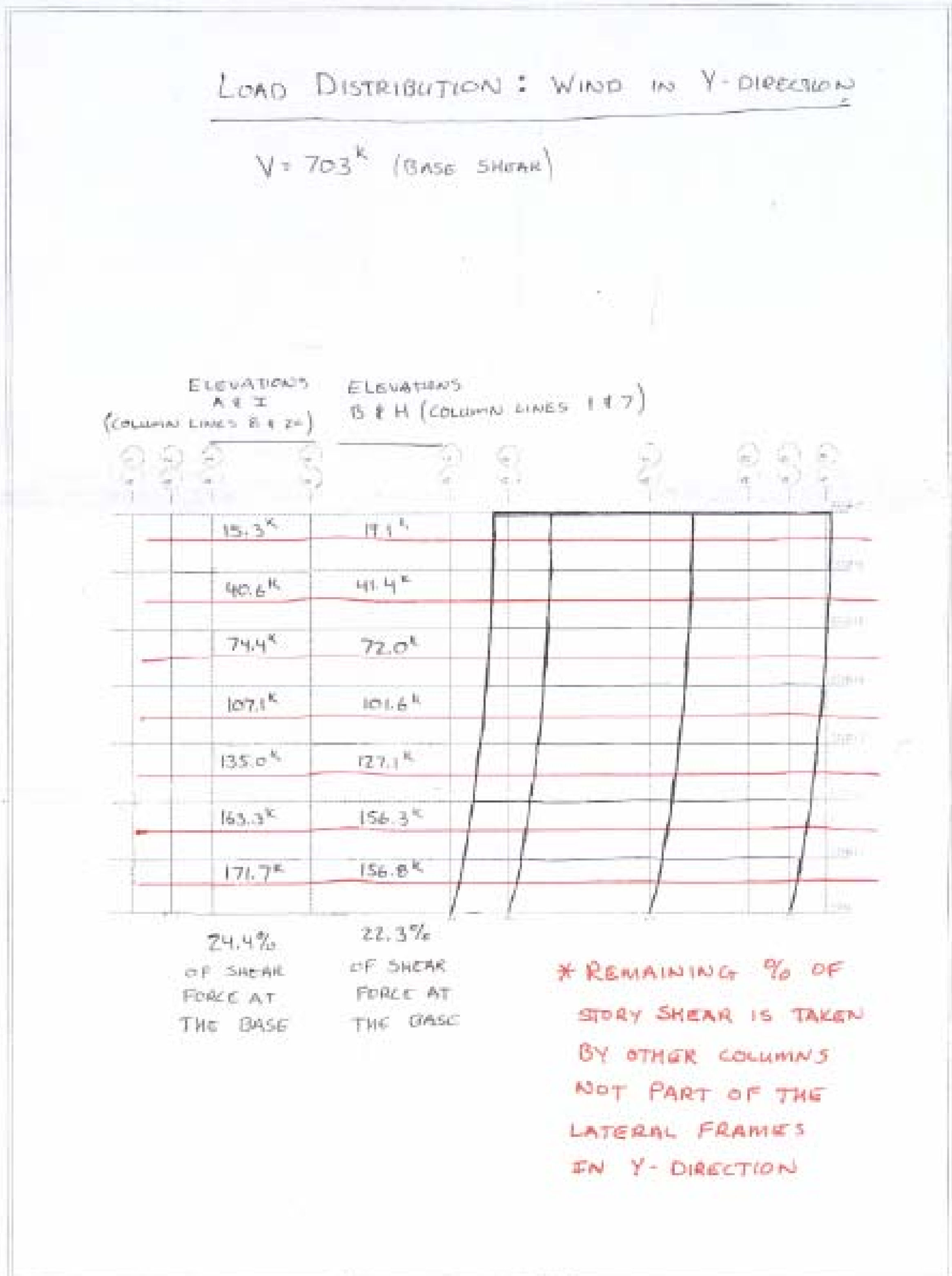


APPENDIX D

Load Distribution of Shear Forces between various lateral frames.



ETABS



ETABS v9.1.1 - File: Etabs Model - 0.9D and 1.6W - December 1,2007 14:40
Elevation View - A Deformed Shape (WINDY) - Kip-in Units

Appendix E

Story Drifts in the X and Y directions due to lateral forces from wind pressures.

X-Direction (N-S)

Story	Point	Load	UX	UY	Total Drift X (in)	Story Drift X (in)
STORY7	1	WINDDRIFTX	3.413	-0.029	3.41	0.09
STORY6	1	WINDDRIFTX	3.324	-0.024	3.32	0.17
STORY5	1	WINDDRIFTX	3.150	-0.019	3.15	0.29
STORY4	1	WINDDRIFTX	2.861	-0.014	2.86	0.40
STORY3	1	WINDDRIFTX	2.457	-0.009	2.46	0.51
STORY2	1	WINDDRIFTX	1.942	-0.005	1.94	0.69
STORY1	1	WINDDRIFTX	1.250	-0.002	1.25	1.25
BASE	1	WINDDRIFTX	0	0	0.00	0.00

Y-Direction (E-W)

Story	Point	Load	UX	UY	Total Drift Y (in)	Story Drift Y (in)
STORY7	20	WINDDRIFTY	-0.0192	3.6622	3.66	0.11
STORY6	20	WINDDRIFTY	-0.0158	3.5556	3.56	0.22
STORY5	20	WINDDRIFTY	-0.0125	3.3389	3.34	0.36
STORY4	20	WINDDRIFTY	-0.0092	2.9772	2.98	0.50
STORY3	20	WINDDRIFTY	-0.0061	2.4726	2.47	0.63
STORY2	20	WINDDRIFTY	-0.0034	1.8388	1.84	0.82
STORY1	20	WINDDRIFTY	-0.0014	1.0197	1.02	1.02
BASE	20	WINDDRIFTY	0	0	0.00	0.00

Appendix F

Story Drifts in the X and Y directions due to lateral forces from seismic forces.

X-Direction (N-S)

Story	Point	Load	UX	UY	Total Drift (in)	Design Drift (in)*	Allowable Drift (in)	Story Drift (in)
STORY7	20	QUAKE X	3.532	-0.035	3.532	7.06	12.00	0.184
STORY6	20	QUAKE X	3.348	-0.028	3.348	6.70	10.20	0.292
STORY5	20	QUAKE X	3.056	-0.022	3.056	6.11	8.64	0.398
STORY4	20	QUAKE X	2.658	-0.016	2.658	5.32	6.24	0.475
STORY3	20	QUAKE X	2.183	-0.010	2.183	4.37	4.68	0.527
STORY2	20	QUAKE X	1.656	-0.006	1.656	3.31	3.10	0.624
STORY1	20	QUAKE X	1.032	-0.002	1.032	2.06	1.50	1.032
BASE	20	QUAKE X	0.000	0.000	0.000	0.00	0.00	0.000

Y-Direction (E-W)

Story	Point	Load	UX	UY	Total Drift (in)	Design Drift (in)*	Allowable Drift (in)	Story Drift (in)
STORY7	20	QUAKE Y	-0.069	5.764	5.765	11.53	12.00	0.332
STORY6	20	QUAKE Y	-0.061	5.432	5.433	10.87	10.20	0.537
STORY5	20	QUAKE Y	-0.051	4.895	4.895	9.79	8.64	0.735
STORY4	20	QUAKE Y	-0.041	4.160	4.161	8.32	6.24	0.873
STORY3	20	QUAKE Y	-0.031	3.288	3.288	6.58	4.68	0.956
STORY2	20	QUAKE Y	-0.021	2.331	2.331	4.66	3.10	1.085
STORY1	20	QUAKE Y	-0.011	1.246	1.246	2.49	1.50	1.246
BASE	20	QUAKE Y	0.000	0.000	0.000	0.00	0.00	0.000

Occupancy Category = IV

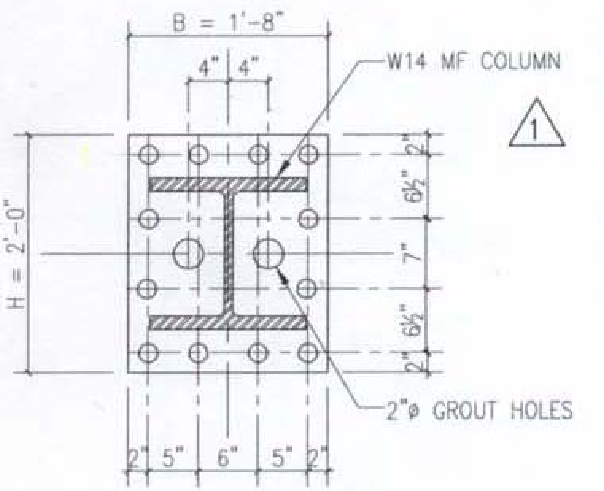
C_d = 3
I = 1.5

Design Drift $= \frac{C_d * d_{xe}}{I}$ (ASCE Eq. 12.8-15)

Allowable Drift = $0.01h_{sx}$ (ASCE Table 12.12-1)

APPENDIX G

Structural detail of column base plate connection.



BASE PLATE DETAIL
TYPE A

NOTES:
1.) ALL BOLT HOLES ½" OVERSIZE (1⅞"ø)

2.) ALL COLUMNS AND BASE PLATES ARE CENTERED ON GRID UNLESS OTHERWISE NOTED ON PLAN

TYPE	ANCHOR ROD		ROD MATERIAL	NUT MATERIAL	WELD	GROUT	NOTES
	DIAMETER	L					
A	1¼"ø	2'-0"	*	A563 GRADE A	¾	1½" MIN.	–
B	1¼"ø	2'-0"	*	A563 GRADE A	½	1½" MIN.	–
C	1¼"ø	1'-0"	*	A563 GRADE A	5/16	1½" MIN.	–
D	1¼"ø	1'-0"	*	A563 GRADE A	5/16	1½" MIN.	–

APPENDIX H

Hand calculations of overturning moments for wind and seismic forces.

OVERTURNING MOMENTS

WIND: (X direction controls over Y direction)

- FACTORED OVERTURNING MOMENT = $\boxed{56815^{k\cdot ft}}$ (From Spreadsheet calculations)

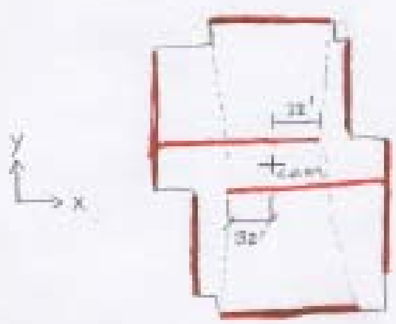
* quick check = TOTAL BASE SHEAR × ½ BLDG HEIGHT

= $1045^k \times 50' = 52250^k \leq \underline{56815^k}$ ✓OK

- RESISTING MOMENT DUE TO BLDG WEIGHT ACTING AT CENTER OF MASS

TOTAL BLDG WEIGHT = $\underline{23650^k}$ (Seismic massing spreadsheet)

→ moment ARM FROM CENTER OF MASS TO NEAREST END OF MOMENT FRAME = $\underline{32'}$



$23650^k \times 32' = \underline{756800}^{'k}$

$M_O = 56815^{'k}$

$M_R = 756800^{'k} \geq 56815^{'k}$ ✓OK

$F.S. = \frac{756800}{56815} = \underline{13.3}$ Rough FACTOR OF SAFETY

OVERTURNING MOMENTS

SEISMIC:

- OVERTURNING MOMENT (M_o) = $\boxed{39142^{'k}}$ (FROM SEISMIC SPREADSHEET)

  * quick check: seismic story shears are roughly in a triangular load shape

  Base shear = 518^k

  $M_o = 518 \times (\frac{2}{3})(100') = 34533^{'k} \approx \underline{\underline{39142}}^{'k}$ ✓ok

- Resisting Moment (M_R) = $756800^{'k}$

  (see M_R calcs on Wind OVERTURNING PAGE)

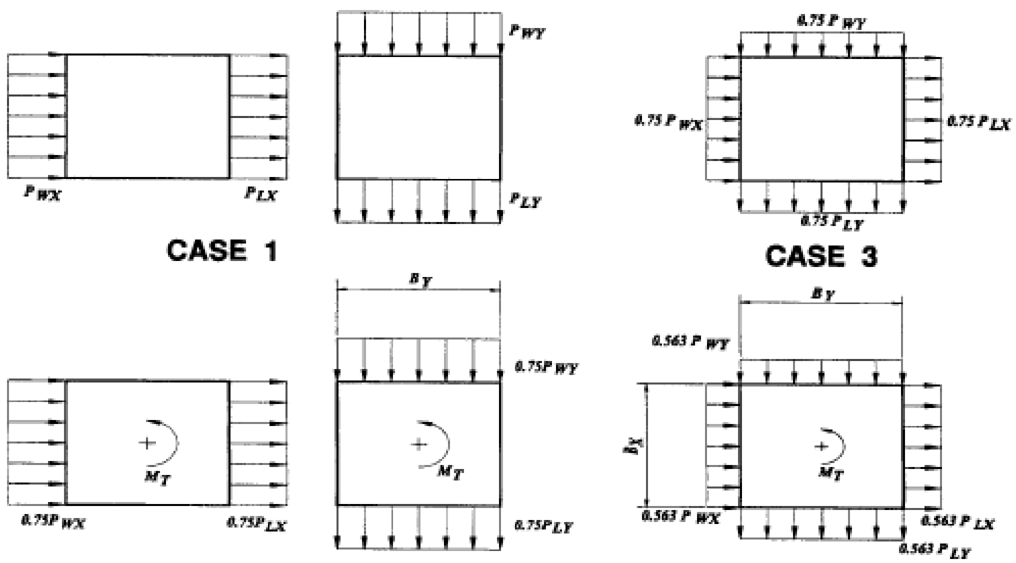
$M_R = 756800^{'k} \geq 39142^{'k}$ ✓ok

$F.S. = \frac{756800}{39142} = \underline{\underline{19.33}}$ Rough FACTOR OF SAFETY

APPENDIX I

ASCE 7-05 Chapter 6: Load cases for analyzing wind loads and effects of torsion.

Main Wind Force Resisting System – Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



$M_T = 0.75\,(P_{WX}+P_{LX})B_X e_X$ $\quad e_X = \pm\,0.15\,B_X$

$M_T = 0.75\,(P_{WY}+P_{LY})B_Y e_Y$ $\quad e_Y = \pm\,0.15\,B_Y$

CASE 2

$M_T = 0.563\,(P_{WX}+P_{LX})B_X e_X + 0.563\,(P_{WY}+P_{LY})B_Y e_Y$ $\quad e_X = \pm\,0.15\,B_X \quad e_Y = \pm\,0.15\,B_Y$

CASE 4

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

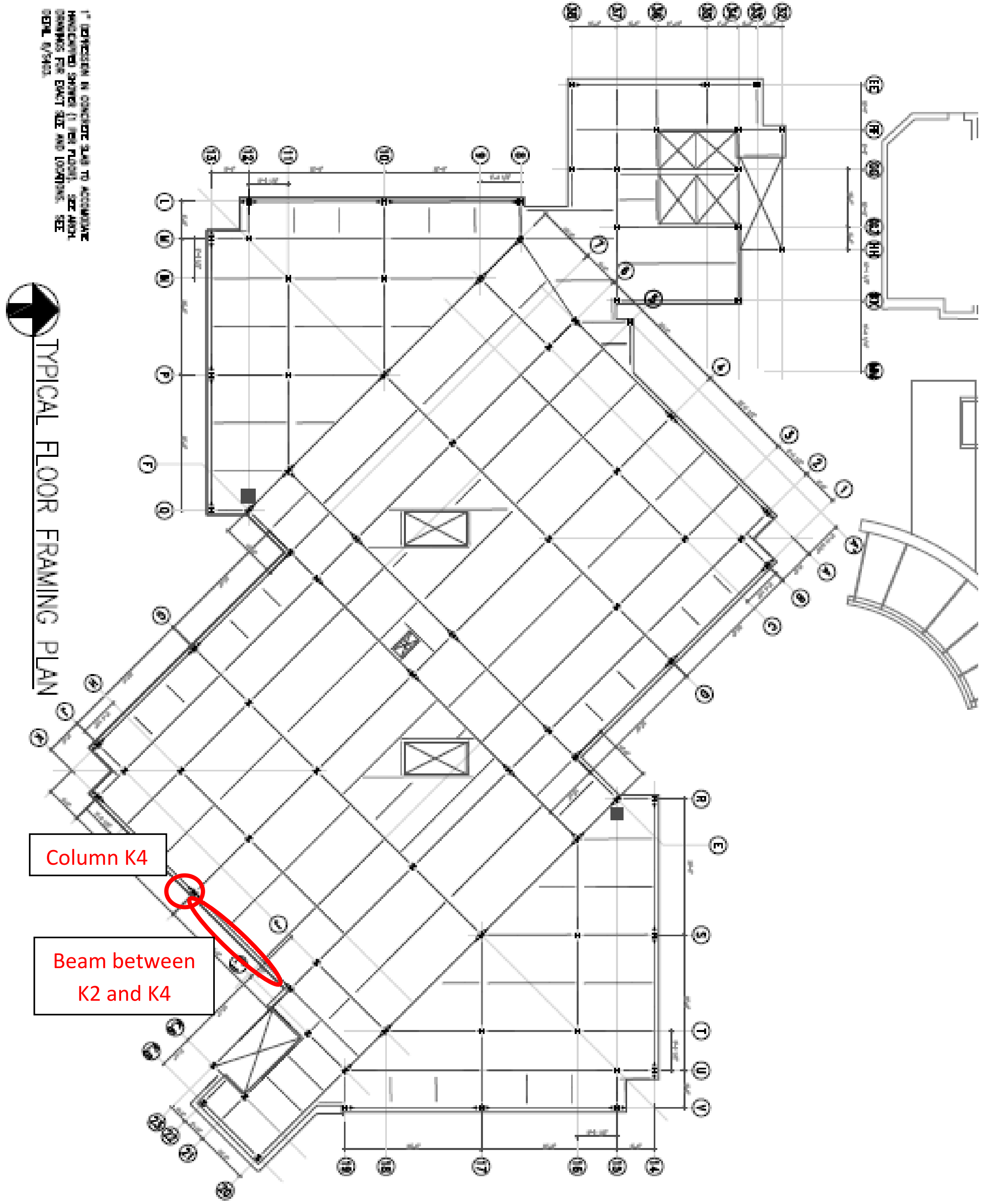
Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   - $e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T: Torsional moment per unit height acting about a vertical axis of the building.

APPENDIX J

Structural 1st Floor Plan submitted by the Engineer of Record.



Hand calculations for capacities of beam and column lateral framing members confirming the design of the original engineer.

FRAME BEAM: SPOT CHECK (BEAM ON 1st STORY BETWEEN COLUMNS K2 & K4 ON STR. PLANS)

LOAD COMBO = 0.9D + 1.6W

MOMENT FROM WIND (1.6W)

MAX POS MOMENT = 11939 + 103.7 = 12042.7 K-in

moment from DL (0.9D)

MAX NEG MOMENT = 9679 − 166 = 9513 K-in

$M_u = 12043^{K\text{-}in}$ $(1003.6^{K\text{-}ft})$

=> FROM TABLE 3-2 AISC 13th Ed

W30 x 90 $\phi M_p = 1060^{K\text{-}ft}$

W30 x 99 $\phi M_p = 1170^{K\text{-}ft}$

W30 x 116 $\phi M_p = 1420^{K\text{-}ft}$

$\boxed{W27 \times 146}$ | $\phi M_p = 1740^{K\text{-}ft}$

$\phi V_n = 497^K \geq 56^K$ → FROM ETABS MODEL

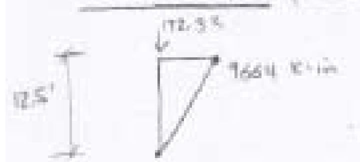
* ENGINEER COULD HAVE DESIGNED HEAVIER SIZE TO REDUCE THE DEPTH OF THE MEMBER. THIS ALLOWS FOR MORE FLEXIBILITY IN FLOOR AND FRAMING DESIGNS.

* ALSO, OTHER AREAS COULD HAVE REQUIRED LARGER MEMBER SIZES BASED ON LARGER LOADS, ENGINEER OF RECORD COULD HAVE WANTED UNIFORMITY IN MEMBER SIZES.

FRAME MEMBER : SPOT CHECK (COLUMN K4 ON STR. PLANS)

LOAD COMBINATION : 0.9 DL + 1.6 W

COLUMN K4 (COLUMN E1 IN ETABS MODEL)



→ MOMENT DUE TO WIND LOADS IN X - DIRECTION

$P_{eff} = P_u + m M_{ux}$ $\quad m = \frac{24}{d} = \frac{24}{14} = 1.71$

$= 172.3 + (1.71)\left(\frac{9664^{k\text{-}in}}{12''}\right)$

$= \underline{1549.4^k}$

$\underline{K_L = 12.5'}$ } TABLE 4-1 LRFD 2013th $\Rightarrow$ $\boxed{W14 \times 132}$ $\phi P_n = 1555^k$

W14 × 145 $\phi P_n = 1750^k$

W14 × 176 IS LARGER THAN W14 × 145

$\phi P_n = 2105^k \geq \phi P_n = 1555$ (W14 × 132)

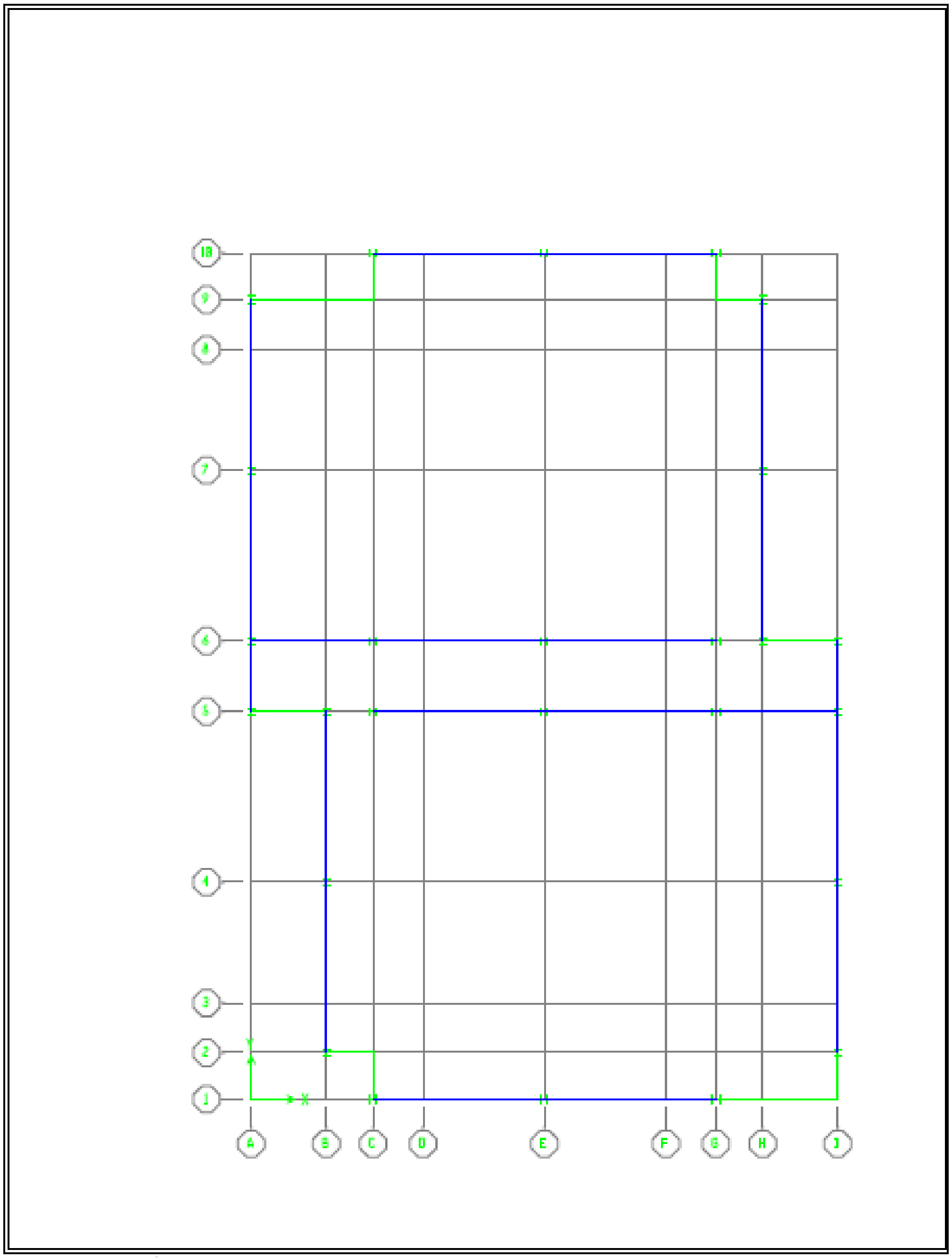
* INCREASED SIZE COULD BE DUE TO LONGER EFFECTIVE LENGTH ON THE FLOOR ABOVE ($K_L = 13.3'$).

* COULD ALSO BE DUE TO UNIFORMITY. OTHER AREAS MIGHT REQUIRE W14 × 176 COLUMNS AND ENGINEER OF RECORD WANTED TO KEEP ALL COLUMNS THE SAME SIZE.

Appendix K

Floor Plan of the framing model created in ETABS.

ETABS



ETABS v9.1.1 - File: Etabs Model - 0.9D and 1.6W - December 3,2007 12:16
Plan View - STORY1 - Elevation 150 - Kip-in Units