St. Elizabeth Hospital Boardman Campus Inpatient Facility

Boardman, Ohio

Josh Behun

Structural Option

Technical Report #3

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Executive Summary

The purpose of this report is to complete a thorough analysis of the lateral resisting system in use within the St. Elizabeth Boardman Hospital. The majority of the focus for this study is placed on the system used throughout the portion of the building that would receive that largest amount of lateral forces; the new seven story patient tower addition. The building contains expansion joints with Teflon slide bearings which separates the individual sections of the hospital and allows the tower to move independently from the rest of the building, making it possible to conduct the study in this fashion. A computer modeling program known as RAM Structural System was used to quickly and efficiently analyze the lateral forces and the respective framed bracing system, as well as evaluate the story drift imposed upon the building by such lateral forces. The wind forces applied to the building are presumably the more significant lateral forces controlling the building's design, even though the seismic loadings are slightly larger. The wind loads are a bigger factor in design due to serviceability and ensuring a minimal story drift in a more recurring situation, where as for the seismic considerations the building merely has to be able to resist the lateral forces without failure.

Conclusion

From the analysis conducted, it appears that the lateral framing designed for the hospital is a suitable system for resisting the lateral forces applied upon the building. The total lateral drift for the building fell well within the allowable limit of H/400 for the wind force analysis, and though the local story drifts at a few floors had slightly exceeded the allowable limit, the exceeding drifts were not excessive enough to pose a severe concern. The seismic forces also fell well within their respective limiting drift of $0.015h_{sx}$, which seems to produce an allowable drift that may be a bit large, though it is not imperative that the bracing system completely dampen the seismic reactions of the building to meet the specific serviceability comfort levels as much as it should to effectively control the wind forces. In all, the lateral resisting system is perfectly capable of controlling the lateral forces that the Ohio region is accustomed to, as well as providing a level of serviceability for personal comfort against common high speed winds.

Building Description

The St. Elizabeth Hospital Boardman Campus Inpatient Facility is 65 million dollar renovation to an already existing two story building located in Boardman, Ohio. The renovation consists primarily of a seven story, 25,000 square foot patient tower addition, as well as some modifications to the preexisting two story diagnostic wing. Appendix A shows the plans for the building as they are expected to be updated with this current renovation and a possible future renovation to come. The patient tower is constructed using a steel framing system, which includes a façade system that is constructed using a brick veneer and a curvilinear aluminum panel curtain wall system that exists on the north facing elevation of the hospital. The remainder of the building, including the preexisting areas, is primarily masonry construction. The total height of the new building tops off at around 103 feet, plus a penthouse that contains a stairway for access to the rooftop HVAC equipment. The hospital began the construction for the new patient tower addition during October of 2005, and has recently finished in August of 2007.

Introduction to Structural System

Foundation

The foundation for the St. Elizabeth Hospital Inpatient Facility consists of 16" diameter auger cast grout injected piles with a capacity of 50 tons and an f'c of 4000 psi, including (4) #6 vertical bars for the top 20' of the piles and #3 ties spaced at 16" on center. The vertical reinforcement from each pile is to extend 18" into its corresponding pile cap or grade beam with a 90° hook of 2'-0" in length. Several of the column piers will be constructed on existing footings, subsequent reinforcement bars are to be drilled and grouted into the existing footing with Hilti epoxy adhesives, providing a minimum embedment of 8".

Super Structure

The framing for the structural system consists typically of wide flange structural steel members. The typical column size for the building is within the range of W12x40 to W12x136, while there are a minimal number of W10 and W14 columns throughout the atypical areas of the new addition. The girders for the building are on average W30x90 where the façade is brick and W18x40 where the outer façade is the aluminum panel curtain wall system. The floor to floor height of each story two through seven is 14'-8" tall while the floor to floor height for the first floor is 15'-4" in height.

Floor System

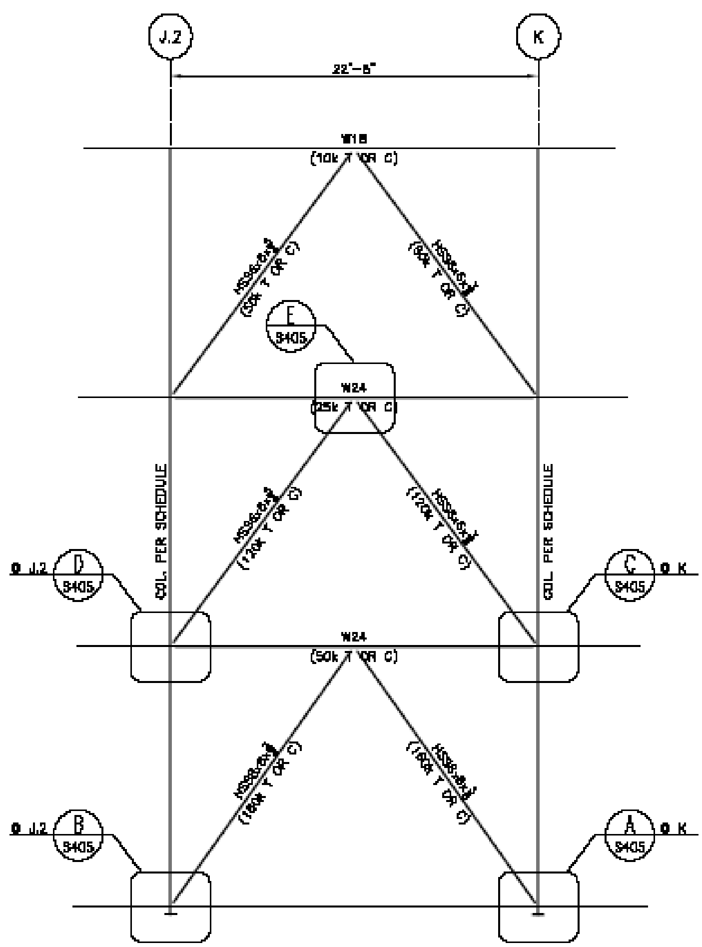
The floor system of the St. Elizabeth Hospital Inpatient Facility is a concrete slab system comprised of a 4" light weight concrete topping slab on 2" – 20 gage galvanized composite decking with 5" long ¾" diameter shear studs and a 6x6-W2.1xW2.1 welded wire fabric reinforcement system. The majority of the beams for the floor framing are 21" in depth with a typical span of 34'. On the first two floors, the new addition's floor systems are connected to the existing floor slabs as well as the masonry walls by ½" diameter Hilti adhesive anchors spaced at 24" on center, with a minimum embedment of 4½".

Roofing

The roofing system is a flat roof which consists of structural steel members similar to that of the floor system. The area where the HVAC units rest has a slab of 4½" light weight concrete on 2"- 20 gage galvanized composite decking with 6x6-W2.1xW2.1 welded wire fabric reinforcement. While the remainder of the roof area, including the penthouse roof, is constructed of 1½"-20 gage galvanized wide ribbed steel roof deck.

Existing Lateral Bracing System

The lateral resisting system in place at the St. Elizabeth Boardman Hospital consists of a number of braced frames strategically placed throughout the superstructure. The majority of the bracing frames used along the exterior of the building contain chevron type bracings, or K braces, and are located against the eastern most side of the building, where the aluminum panel curtain wall system meets the brick façade. There is also a large section of bracings amongst the elevator shafts that consist primarily of chevron style bracings as well, except for a two column section along the western most side of the elevators that is constructed using a set of singular cross bracings. Aside from the typical bracings throughout the building, there are also a small number of interior framed sections that contain knee bracings for added lateral support. All of the bracing members used throughout the framing system are square HSS members ranging in size from 5x5x3/8" to 9x9x1/2".
(Lateral bracing plans and typical elevations are shown in Appendix B)

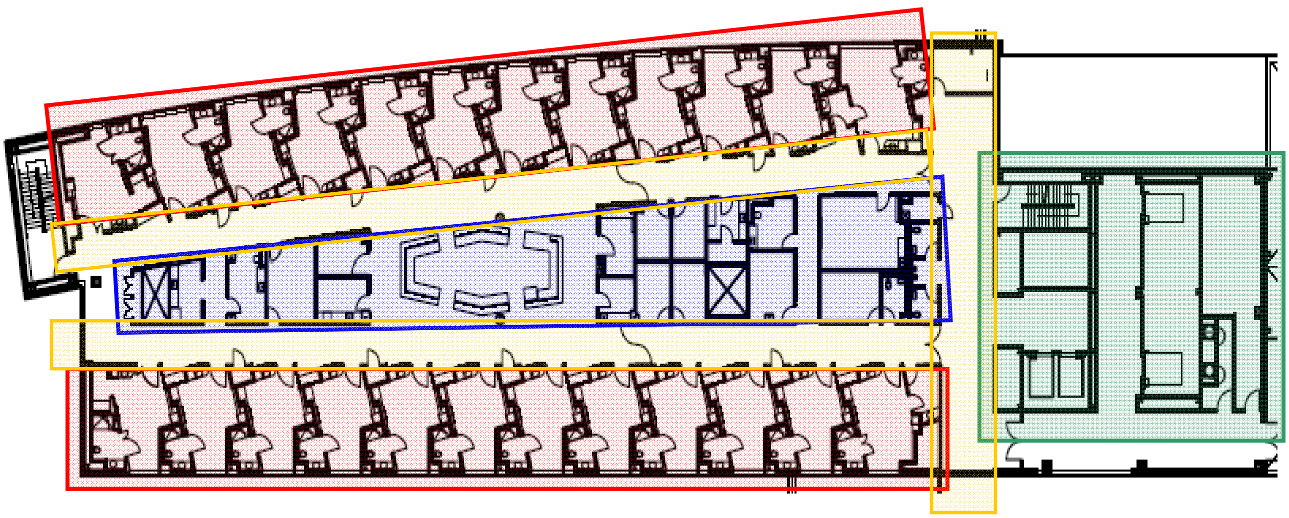


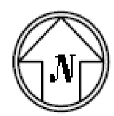
Typical chevron braced frame used within building
(Elevation 6 along western wall shown)

Typical Framing Plans for new Inpatient Facility

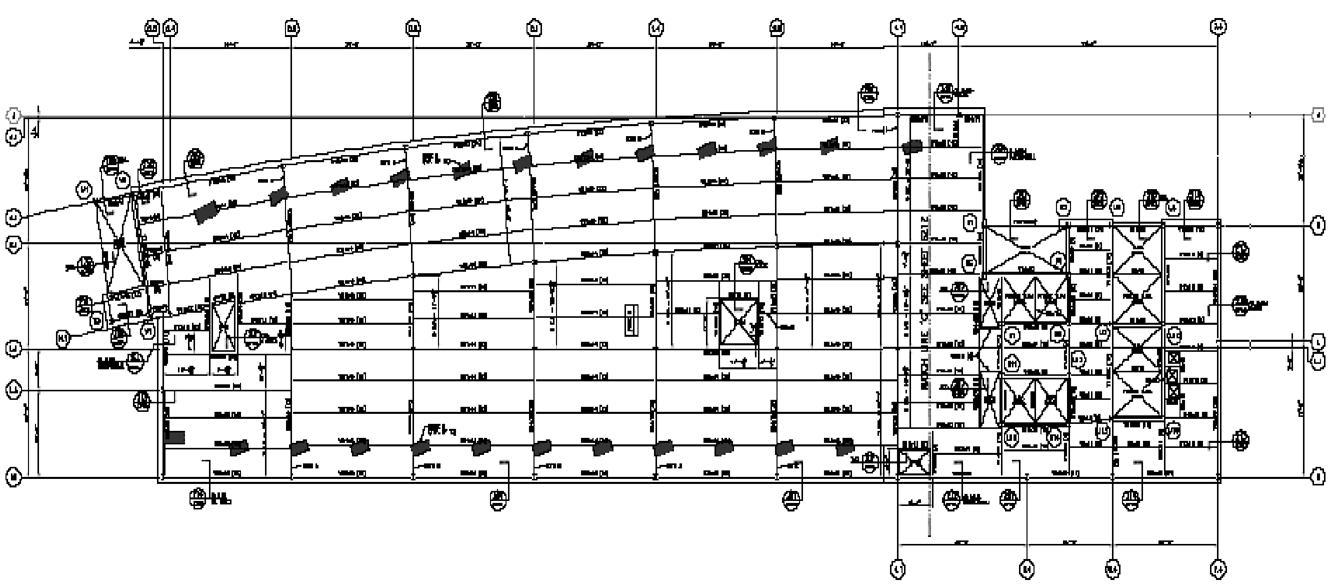
Typical Floor Plan for Seven Story Addition

- Showing **patient rooms**, **nurse's station**, **elevator core**, **and corridors**.





Typical Framing Plan for Seven Story Addition



Loading Conditions

Building Codes

The building codes used for design parameters of the hospital and its new additions are:

- The Ohio Building Code, 2005
- The International Building Code, 2003

As well as the ASCE-05 manual, which was utilized as a reference guide for design considerations including wind, snow, and seismic analysis.

Load Combinations

The load combinations considered for the lateral analysis were those of the ASCE-05 handbook, where the most significant forces would be the dead, live, and snow loads for the gravity influence, and the wind and or seismic forces for the lateral loading influence.

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

Design Criteria

Main areas of concern shown on floor plan above

Live Loads

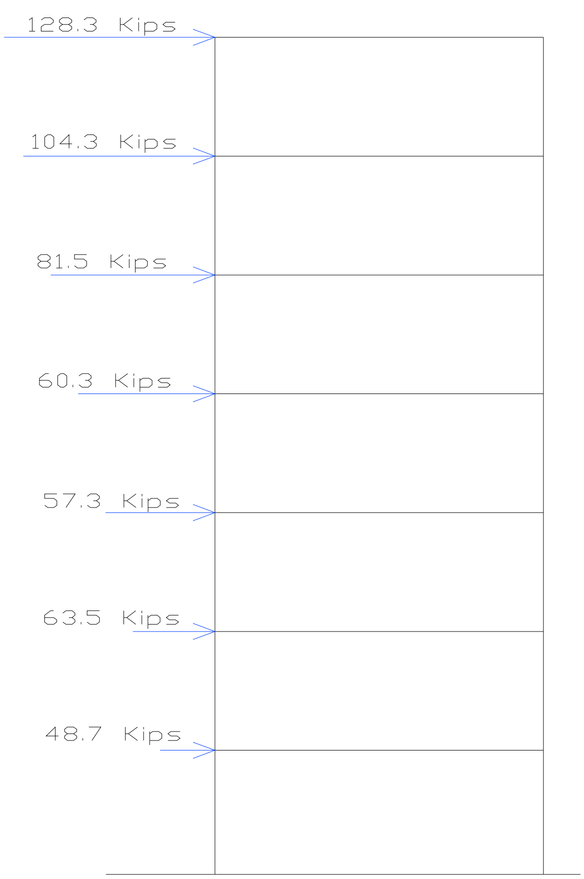
Roof……………………………………….. 30 psf
Public Areas………………………….…..100 psf
Lobbies…………………………………... 100 psf
First Floor Corridors……………………….. 100 psf
Corridors above First Floor…………………80 psf
Patient Rooms……………………….…...60 psf
Light Storage…………………………….……125 psf
Catwalks……………………….…………25 psf
Mechanical…………………..………………175 psf
Stairs……………………….……………… 100 psf

Lateral Loading

From previous lateral load calculations done with technical assignment #1, the seismic loading produces the largest forces that have the capability of controlling the design of the hospital building and its lateral resisting system. However, for serviceability reasons it is more appropriate to design the lateral system to be able to efficiently react to the wind forces, which would most likely be a much more significant and recurring loading circumstance in the Ohio region. The design engineer had conducted the initial design by separating the building into three distinct sections, with the seven story patient tower addition being the portion of the building most significantly affected by lateral loadings. Seeing as the building segments are separated by expansion joints containing Teflon slide bearings, the calculations performed in this report will be conducted in a similar fashion.

Seismic Loading

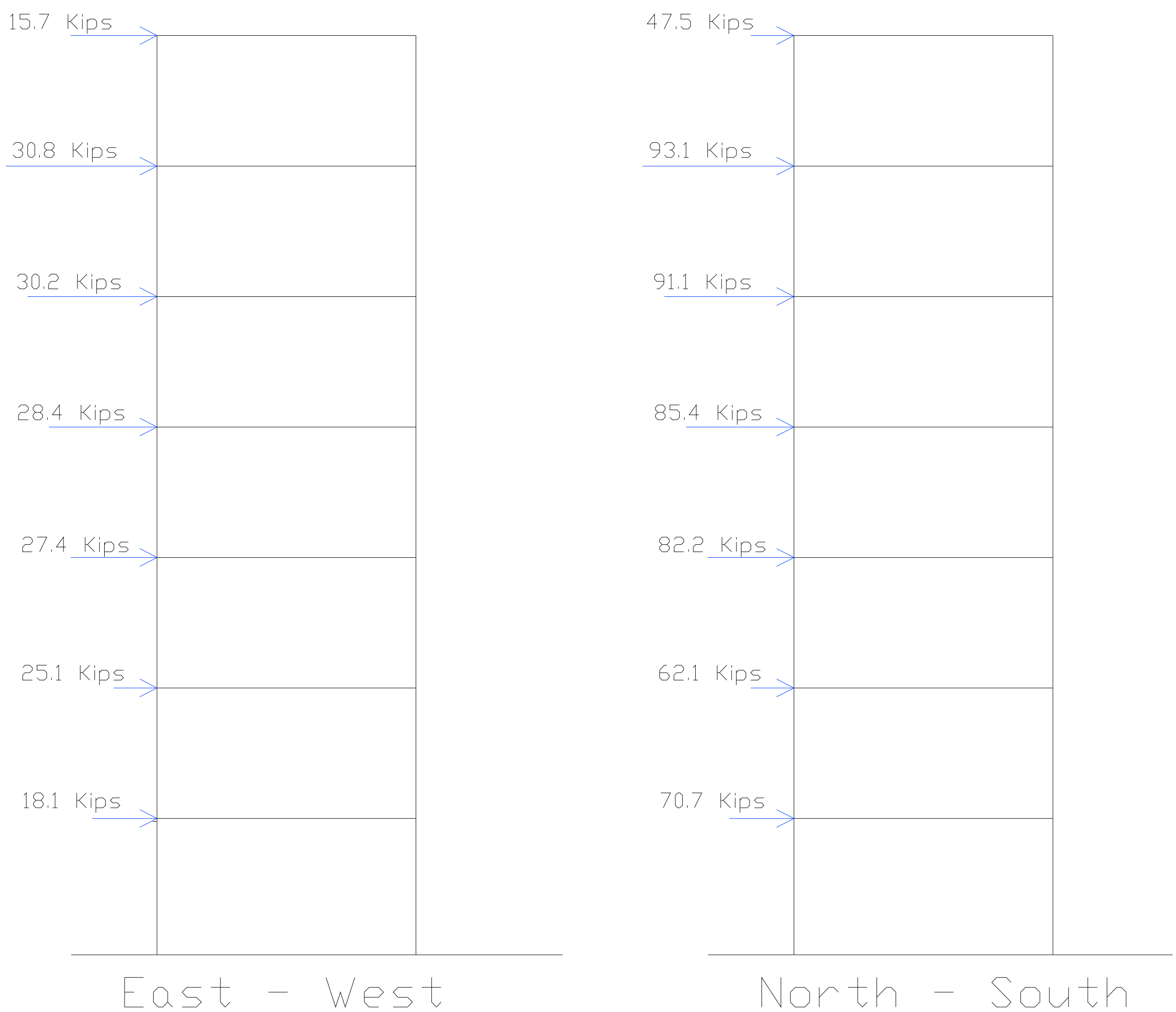
The seismic analysis was performed utilizing the procedure specified in ASCE-05. The loads were determined using the Equivalent Lateral Force analysis method, and are based on a seismic exposure group III, with a site class D and seismic performance category C. The original design had been conducted by separating the building into distinct sections, due to expansion joints between each section that would allow them to react differently to seismic force application. In this way it is possible to analyze the patient tower addition as a separate entity. (Supporting calculations provided in Appendix D.2)



Base Shear = 544 Kips
Overturning Moment = 37,330 Ft-Kips

Wind Loading

The wind analysis was also performed utilizing the methods specified in ASCE-05. The loads were determined using method II, the analytical procedure, and are based on a fully enclosed flexible building with an exposure category C, an importance factor of 1.15, and a maximum wind velocity of 90 mph. When comparing the two main directions of the wind forces, although the east-west directional wind pressures are higher, the wind forces in the north-south direction were determined to be significantly larger due to a greater amount of surface area for them to act upon. Being that the wind forces are a continuous force being applied to the building, they will govern the design of the building's lateral resisting system for controlling comfort and serviceability on a regular basis.
(Supporting calculations provided in Appendix D.1)

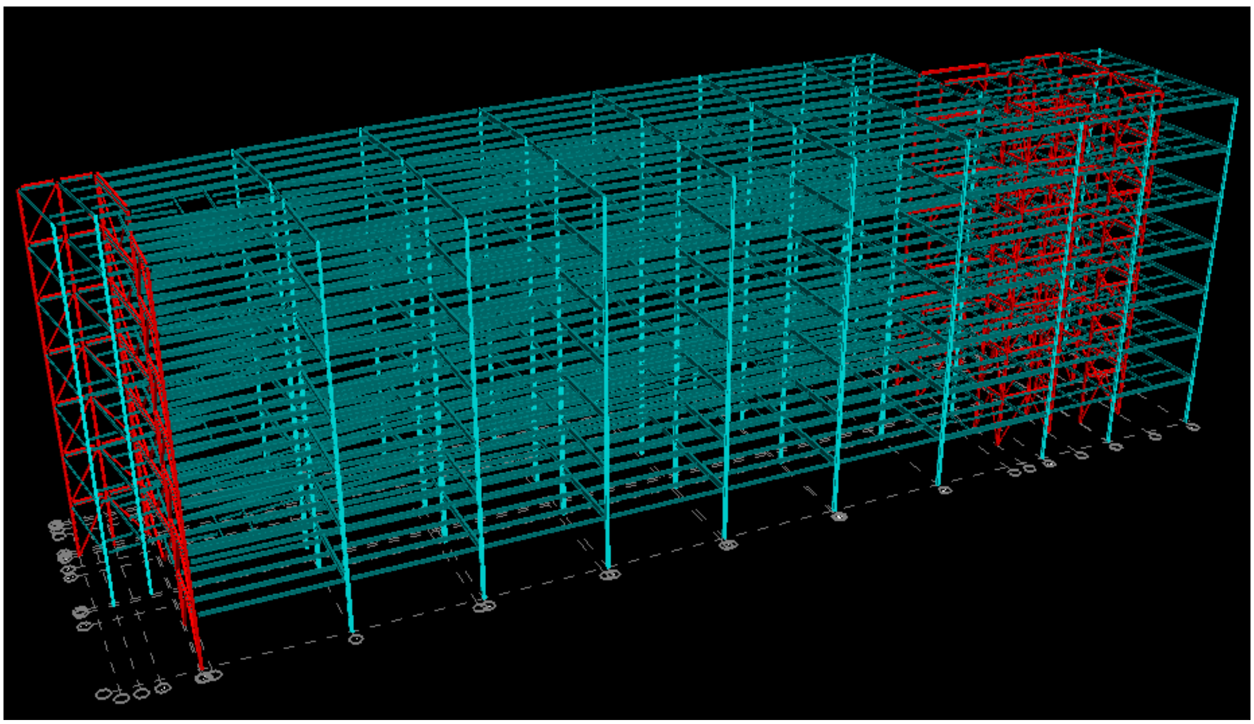


Base Shear = 175 Kips
Overturning Moment = 10,528 Ft-Kips

Base Shear = 532 Kips
Overturning Moment = 31,590 Ft-Kips

Analysis Method

The analysis performed for the lateral resisting system was conducted using the computer modeling software RAM Structural System. The RAM program allows users to construct a database of building structures, making it easier to design and analyze any type of building structure. Once the building's framing plan has been modeled and designed, and all significant loads have been entered, the RAM software can begin to determine the effect the loadings will have on the building's framing system. At that point the modeling software can also determine the story drift and overall lateral building movement and prepare it to be compared against standard drift allowances for personal comfort and serviceability within the building.



Three dimensional view of building framing and lateral framing (lateral framing shown in red)
– Courtesy of RAM Structural System

Story Drift

The story drift for the building was calculated using RAM Structural System. The seismic forces ended up being the largest lateral force affecting the building, though the design of the lateral system is controlled by the wind forces. Seeing as the wind forces are a more common occurrence affecting the building's serviceability, the lateral framing will be designed to support the displacement caused by the wind load. Where as opposed to the designing for seismic forces, the serviceability is not as much of a factor, thus the lateral system has only to ensure the building can resist the seismic forces without failure. For reacting against wind forces, the standard allowable story drift for a building is H/400, which makes the maximum controlling displacement for the building 3.1". Although the story drifts caused by the wind forces for a few individual floors are slightly exceeding the limit of 0.43", the maximum displacement for the entire building comes to 2.62", which falls well within the allowable limit. For reactions to the seismic forces placed upon the building, the allowable story drift is controlled by the limiting factor of $0.015h_{sx}$, which makes the allowable drift 18.72". Being that the actual total seismic drift comes out around 3.33", the lateral system more than compensates for the seismic forces that are possible in the northern Ohio region.

Story Drift for Wind Loads in North – South Direction

Floor	Height (Ft)	Floor to Floor Height	Max Displacement		Allowable Drift (H/400)	Story Drift		Allowable Story Drift (H/400)
			X (Inch)	Y (Inch)		X (Inch)	Y (Inch)	
7th	103.33	14.67	2.62	0.450	3.10	0.42	0.086	0.43
6th	88.67	14.67	2.20	0.364	2.66	0.45	0.087	0.43
5th	74	14.67	1.75	0.277	2.22	0.47	0.083	0.43
4th	59.33	14.67	1.28	0.194	1.78	0.45	0.074	0.43
3rd	44.67	14.67	0.83	0.120	1.34	0.4	0.08	0.43
2nd	30	14.67	0.430	0.040	0.90	0.256	0.024	0.43
1st	15.67	15.33	0.174	0.016	0.47	0.174	0.016	0.47

Story Drift for Seismic Loading

Floor	Height (Ft)	Floor to Floor Height	Max Displacement		Allowable Drift ($0.015h_{sx}$)	Story Drift		Allowable Story Drift ($0.015h_{sx}$)
			X (Inch)	Y (Inch)		X (Inch)	Y (Inch)	
7th	103.33	14.67	3.327	0.364	18.72	0.585	0.074	2.64
6th	88.67	14.67	2.742	0.290	15.96	0.616	0.072	2.64
5th	74	14.67	2.126	0.218	13.32	0.615	0.068	2.64
4th	59.33	14.67	1.511	0.150	10.68	0.566	0.061	2.64
3rd	44.67	14.67	0.945	0.089	8.04	0.472	0.049	2.64
2nd	30	14.67	0.473	0.040	5.40	0.293	0.032	2.64
1st	15.67	15.33	0.180	0.008	2.82	0.180	0.008	2.82

Load Distribution

To determine the distribution of the lateral forces acting upon the individual braced frame members, the calculated story forces were applied directly to the center of mass for each floor. The appropriate centers of mass for each respective story are labeled in Appendix C. The load cases used to create the most relative loading combinations are those mentioned in the loading conditions section, on page seven of this report. The distribution of the loads appears to be dispersed rather evenly across the framing of the building. Though, since the majority of the lateral resisting frames are located within the area of the elevator core, it is evident that those specific locations are more vital areas of lateral bracing.

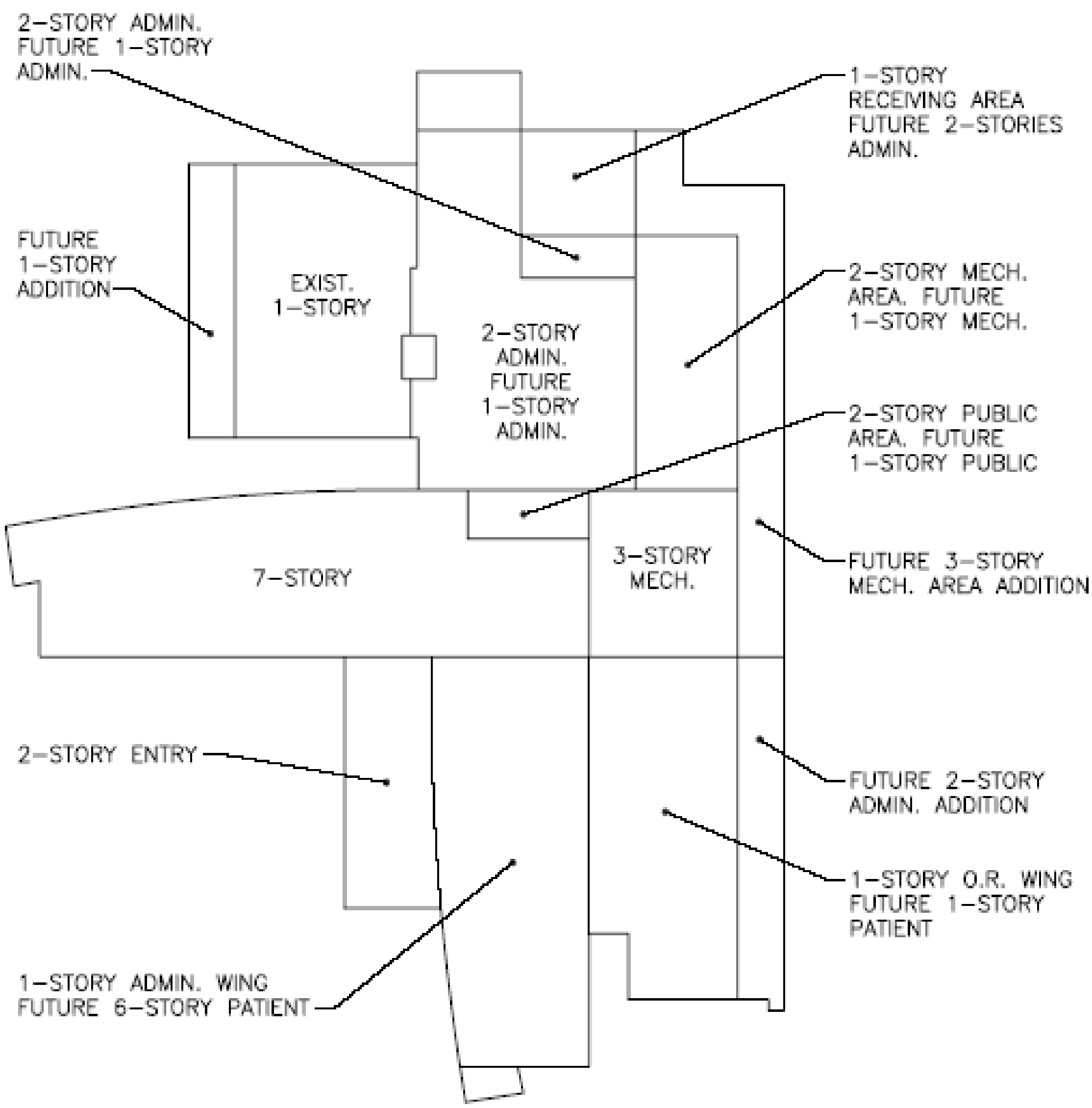
From calculations performed by hand, the specific bracing members investigated, frame numbers 11 and 24, appear to be designed large enough to resist the forces that are to be applied upon them. The spot check calculations performed for the lateral bracing individual members are provided in Appendix F. The loading cases evaluated were also calculated utilizing the RAM Structural System program. The moment diagrams of the two frames considered, as produced by the RAM program, are shown in Appendix E.

Conclusions

After completing an analysis of the lateral system currently in use at the St. Elizabeth Hospital Inpatient Facility, it has been determined that the existing system is adequate for controlling serviceability and resisting the lateral effects caused by wind and seismic forces. The total lateral drift for the entire building is well within the allowable limit of H/400 for the wind analysis, and though the local story drifts do slightly exceed that serviceability limit at a few specific stories, the exceeding drifts are very small and do not appear to be large enough to pose a noticeable drift worth extensive concern. The seismic analysis also provides a story drift that falls well within its allowable limit of $0.015h_{sx}$, which produces an allowable lateral displacement that seems large, though the seismic forces do not necessarily need to be dampened enough to meet serviceability concerns but only controlled enough to ensure the building frame will not reach a point of failure. From the basic analysis, the torsion effects of the lateral forces appeared to be nonexistent and thus was not a factor impacting the building's lateral system design. In all, the lateral resisting system that was initially designed for the hospital is more than adequate for controlling any lateral displacement caused by wind or seismic forces that are common to the northern Ohio region.

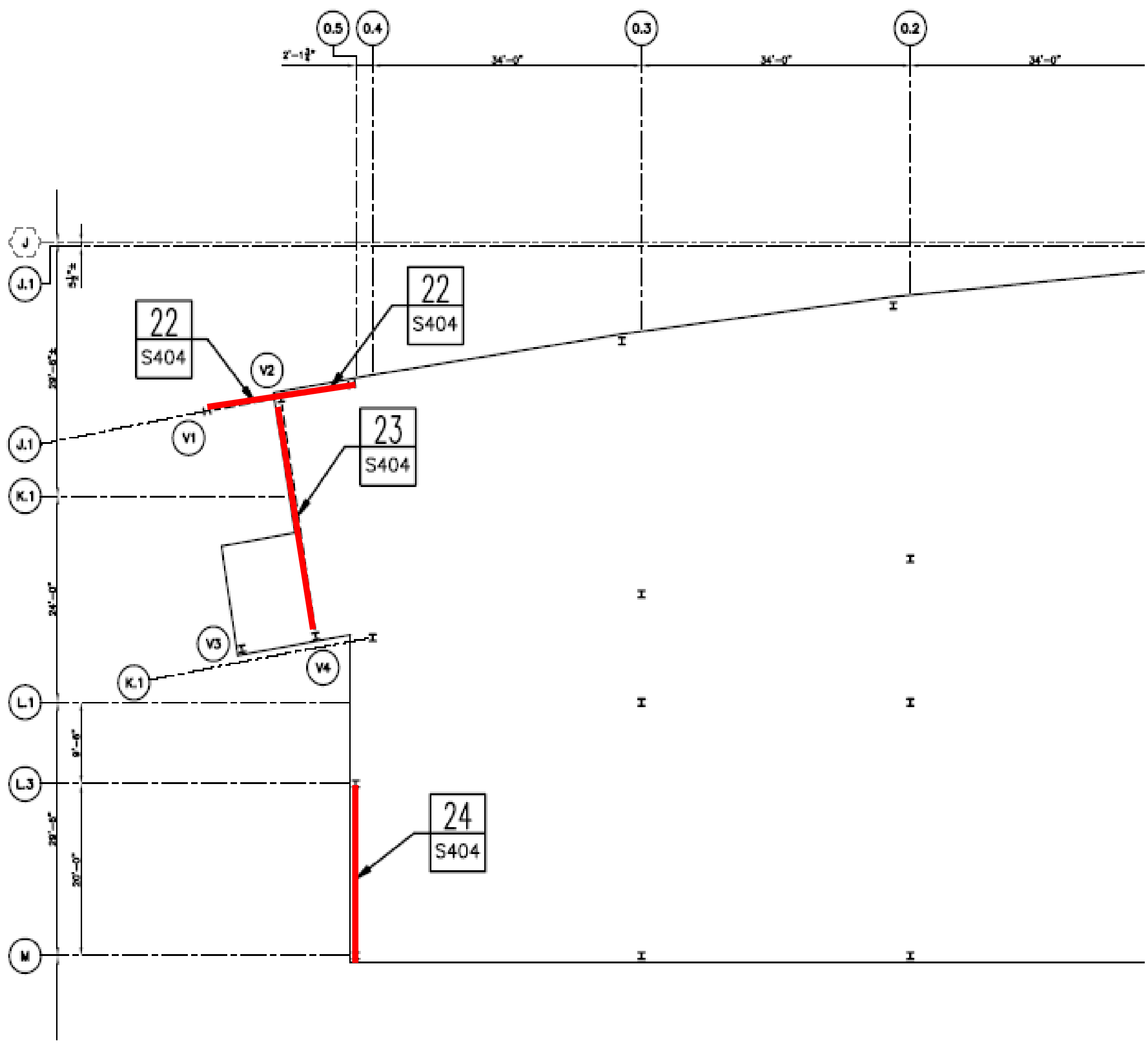
Appendix

Appendix A – Complete Building Plan

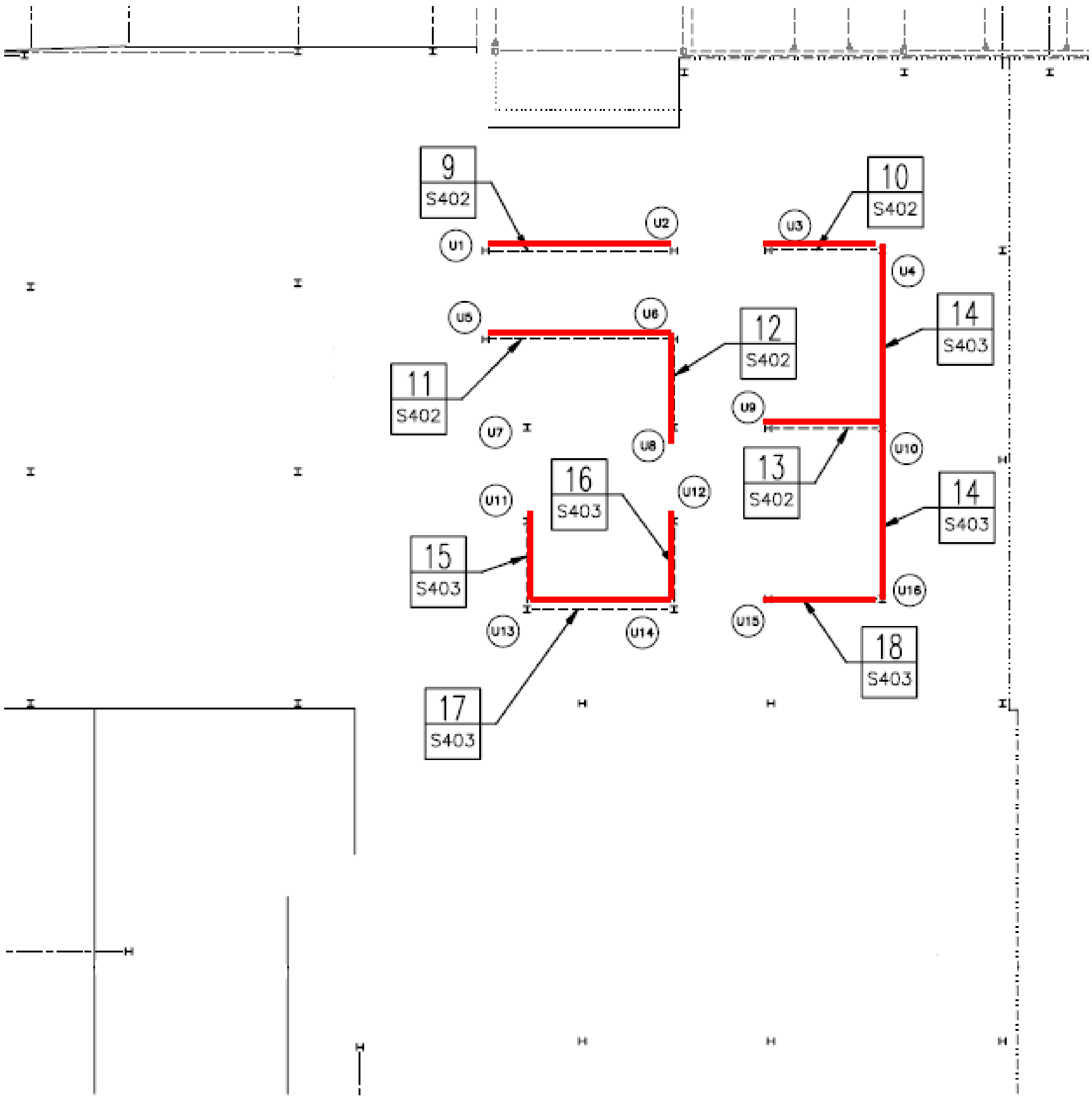


FUTURE EXPANSION KEY PLAN

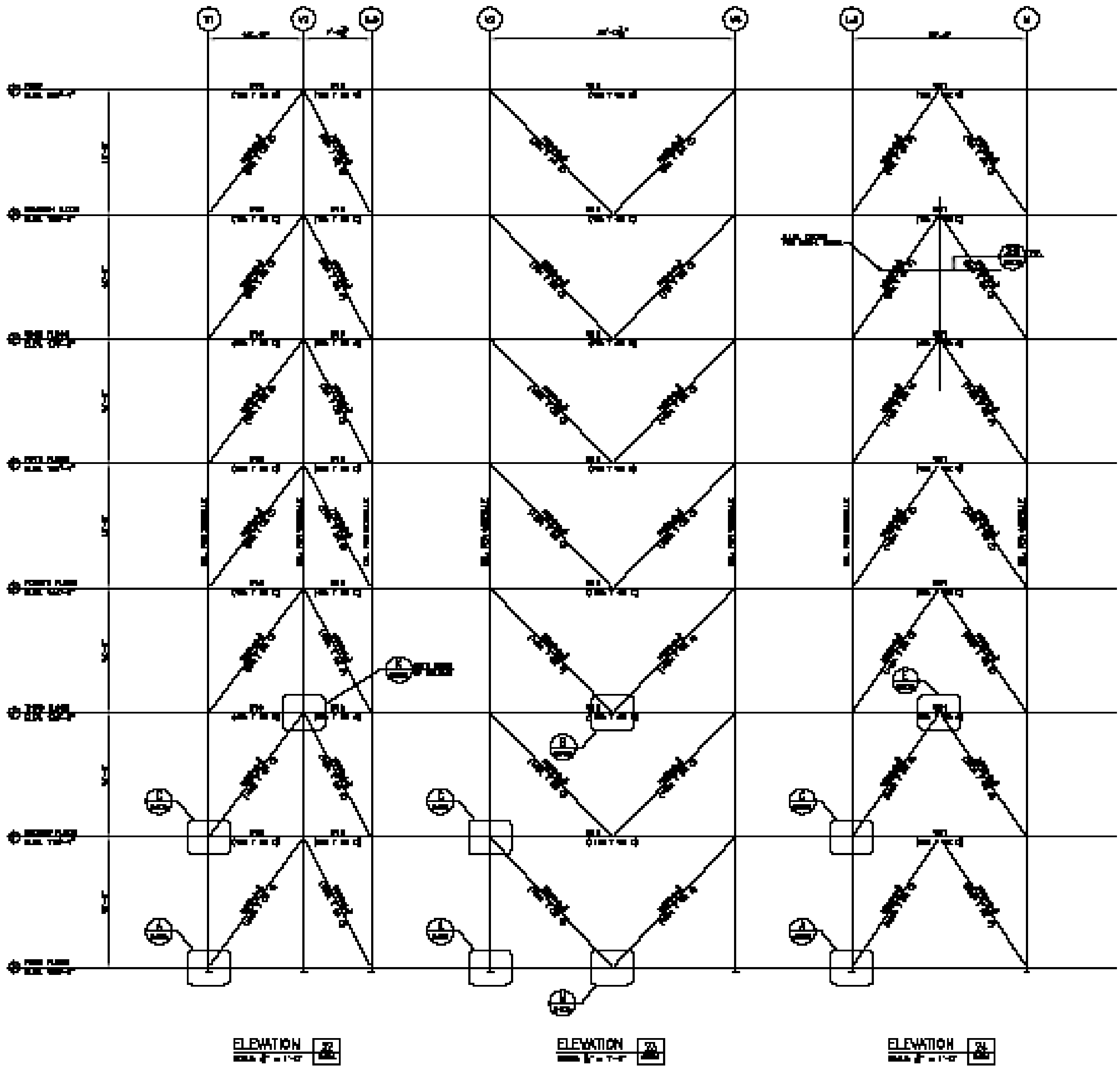
Appendix B – Bracing Plan at Eastern Side of Building



Bracing Plan around Elevators at Western Side of Building

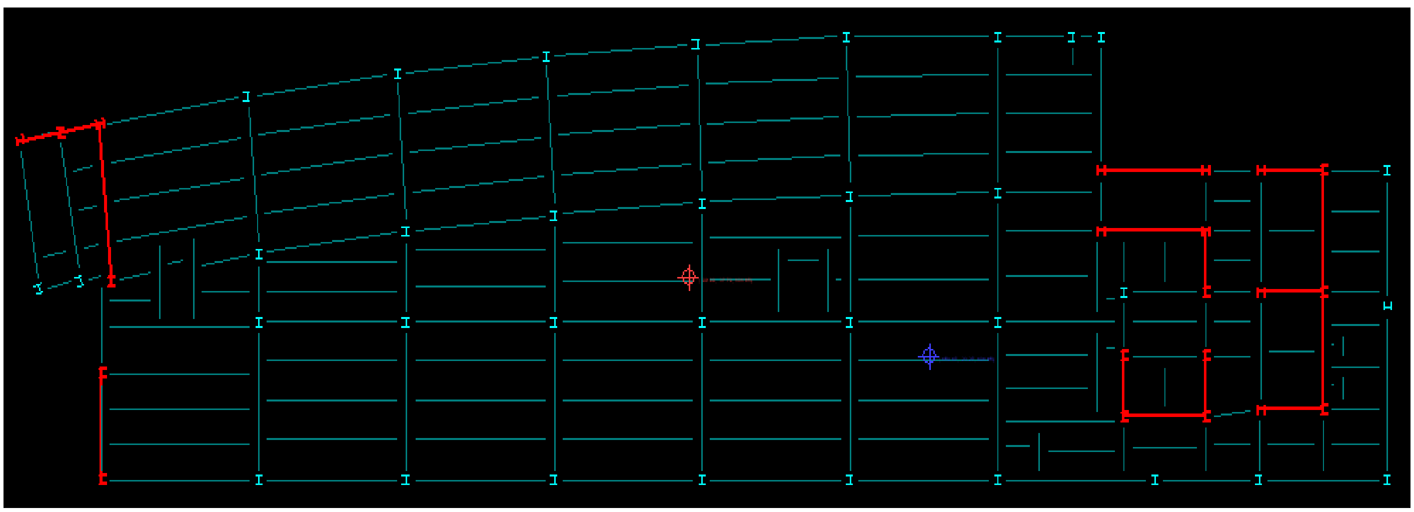


Typical Framing Elevation Showing Bracing Variations



Appendix C – Center of Mass

Floor	Center of Rigidity		Center of Mass	
	X	Y	X	Y
7th	154.17	23.27	98.99	37.79
6th	152.05	25.71	99.23	37.80
5th	148.21	29.32	99.18	37.80
4th	144.16	33.64	99.12	37.81
3rd	139.77	38.33	99.08	37.81
2nd	133.24	43.23	99.07	37.81
1st	125.65	44.90	99.08	37.80



Center of Mass for the building shown in **RED**
Center of Rigidity for the building shown in **BLUE**
(Lateral resistant framing also in red)

Appendix D.1 – Lateral Load Analysis - Wind

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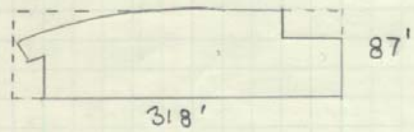
Wind Speed = 90 mph

$I = 1.15$

Category C

Flexible Structure



n_1 = frequency

$= \frac{1}{1.22} = 0.81967$

$$g_r = \sqrt{2\ln(3600(n_1))} + \frac{0.577}{\sqrt{2\ln(3600(n_1))}} = 4.142$$

g_q and $g_v = 3.4$

$\bar{z} = .6h = .6(104) = 62.4 > \bar{z}_{min} = 15$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{62.4}{33}\right)^{1/5} = 568$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}(V)\left(\frac{88}{60}\right) = 0.65\left(\frac{62.4}{33}\right)^{1/6.5}\left(\frac{88}{60}\right)90 = 94.6$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.81967(568)}{94.6} = 4.92$$

N-S

$\underline{R_h}$ $$n_h = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(.81967)(104)}{94.6} = 4.15$$

$\underline{R_B}$ $$n_B = \frac{4.6 n_1 EB}{\bar{V}_{\bar{z}}} = \frac{4.6(.81967)(318)}{94.6} = 12.67$$

$\underline{R_L}$ $$n_L = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4(.81967)(87)}{94.6} = 11.6$$

Note: equation n for R_B contains a symbol E which has been determined to be a mistake in the code. Both ASCE '02 and '05 contain typo's in this equation.

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$$R_L = \frac{1}{n} - \frac{1}{2n^2}\left(1 - e^{-2n}\right)$$

$$= \frac{1}{11.6} - \frac{1}{2(11.6)^2}\left(1 - e^{-2(11.61)}\right) = 0.0825$$

$$R_B = \frac{1}{12.67} - \frac{1}{2(12.67)^2}\left(1 - e^{-2(12.67)}\right) = 0.0758$$

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{318+104}{568}\right)^{.63}}} = 0.81$$

damping ratio β is assumed to be 5%

$$R = \sqrt{\frac{1}{\beta} R_N R_h R_B (0.53 + 0.47 R_L)}$$

$$= \sqrt{\frac{1}{0.05}(0.0513)4.15(0.0758)(0.53 + 0.47(0.0825))} = 0.43$$

$$R_N = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(4.92)}{(1 + 10.3(4.92))^{5/3}} = 0.0513$$

$$I_{\bar{z}} = c\left(\frac{10}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{10}{62.4}\right)^{1/6} = 0.147$$

$$G_f = 0.925\left[\frac{1 + 1.7(I_{\bar{z}})\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right]$$

$$= 0.925\left[\frac{1 + 1.7(0.147)\sqrt{3.4^2(0.81)^2 + 4.142^2(0.43)^2}}{1 + 1.7(3.4)(0.147)}\right]$$

$$= 0.91$$

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E-W

R_B $n_B = \frac{4.6(0.81967)(87)}{94.6} = 3.47$

$R_B = \frac{1}{3.47} - \frac{1}{2(3.47)^2}\left(1 - e^{-2(3.47)}\right) = 0.267$

R_L $n_L = \frac{15.4(0.81967)(318)}{94.6} = 42.43$

$R_L = \frac{1}{42.43} - \frac{1}{2(42.43)^2}\left(1 - e^{-2(42.43)}\right) = 0.0234$

$Q = \sqrt{\frac{1}{1 + .63\left(\frac{87+104}{548}\right)^{.63}}} = 0.871$

$R = \sqrt{\frac{1}{0.05}(0.0513)(4.15)(0.267)(0.53 + 0.47(0.0234))}$

$= 0.784$

$G_f = 0.925\left[\frac{1 + 1.7(0.147)\sqrt{3.4^2(0.871)^2 + 4.142^2(0.784)^2}}{1 + 1.7(3.4)(0.147)}\right]$

$= 1.05$

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pressure

$P = q G_f C_p - q_i (\pm GC_{pi})$

Enclosed Building
$GC_{pi} = \pm 0.18$
$C_p = 0.8$ windward
-0.5 leeward

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$
$= 0.00256 (K_z)(1)(0.85) 90^2 (1.15)$
$= 20.27 K_z$

N-S

leeward

$P = q_h G_f C_p - q_h(-GC_{pi})$
$= 20.23(0.91)(-0.5) - (20.23)(-0.18)$
$= -5.56$

windward

$P = q_z G_f C_p - q_i(GC_{pi})$
$= q_z (0.91)(0.8) - 20.23(0.18)$

E-W

leeward

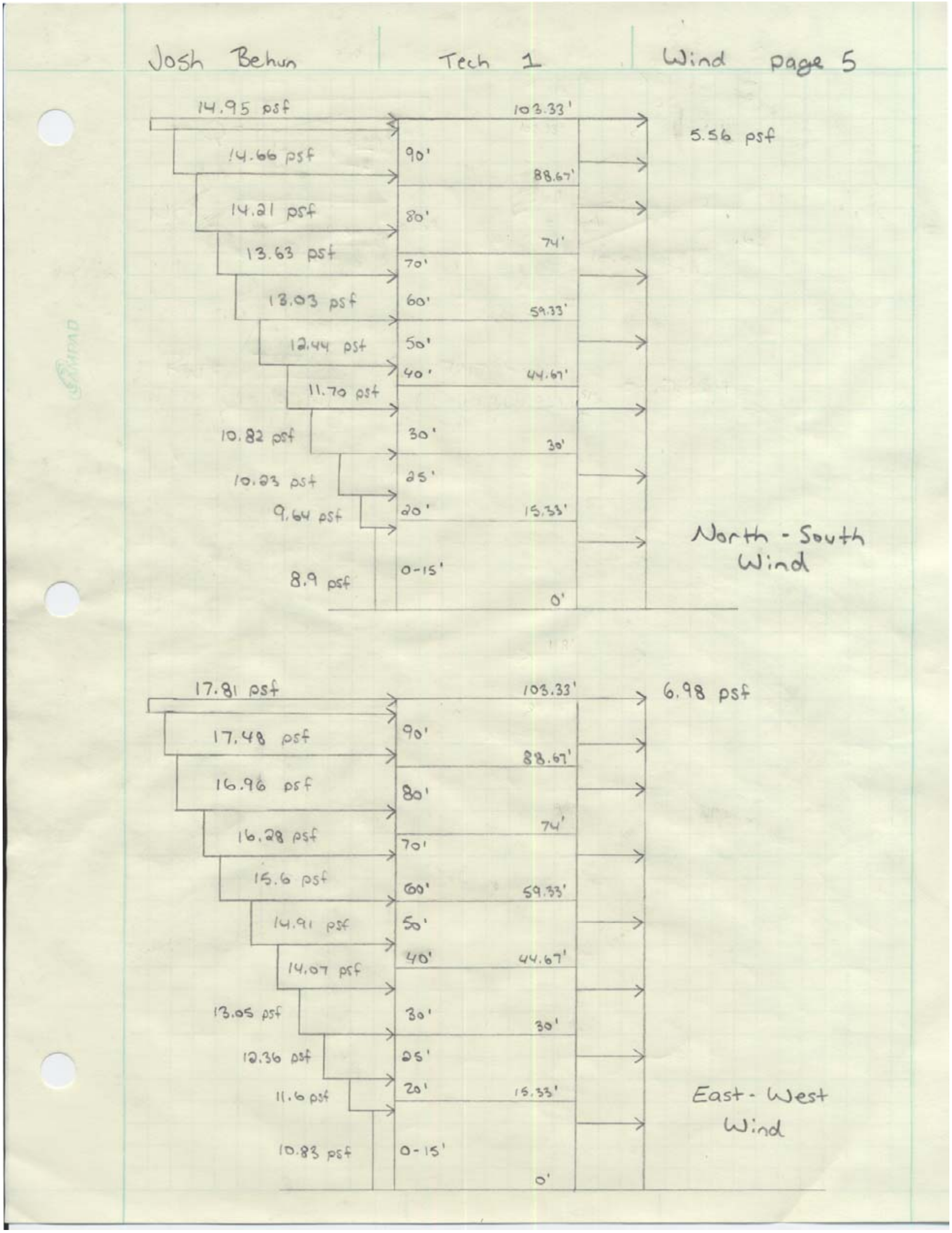
$P = 20.23(1.05)(-0.5) - 20.23(-0.18)$
$= -6.98$

windward

$P = q_z (1.05)(0.8) - 20.23(0.18)$

Flexible Building Properties		
	N-S	E-W
B	87’	318’
L	318’	87’
n1	0.8197	0.8197
h	104’	104’
$h_{min} = 0.6h$	62.4	62.4’
g_R	4.142	4.142
g_Q & g_v	3.40	3.40
R_n	0.0513	0.0513
I_z	0.147	0.147
V_z	94.60	94.60
R_h	4.15	4.15
ηh	4.15	4.15
β	5%	5%

	N-S Direction	E-W Direction
ηL	11.6	42.43
R_L	0.0825	0.0234
ηB	12.67	3.47
R_B	0.0758	0.267
Q	0.81	0.871
R	0.43	0.784
Gf	0.91	1.05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Tech 1
Wind page 5
14.95 psf
103.33'
5.56 psf
14.66 psf
90'
88.67'
14.21 psf
80'
74'
13.63 psf
70'
13.03 psf
60'
59.33'
12.44 psf
50'
40'
44.67'
11.70 psf
10.82 psf
30'
30'
10.23 psf
25'
9.64 psf
20'
15.33'
North - South Wind
8.9 psf
0-15'
0'
17.81 psf
103.33'
6.98 psf
17.48 psf
90'
88.67'
16.96 psf
80'
74'
16.28 psf
70'
15.6 psf
60'
59.33'
14.91 psf
50'
40'
44.67'
14.07 psf
13.05 psf
30'
30'
12.36 psf
25'
11.6 psf
20'
15.33'
East - West Wind
10.83 psf
0-15'
0'

Wind Pressure

North – South Wind Pressures					
Height (ft)	Kz	qz	Windward Pressure (psf)	Leeward Pressure (psf)	Total
0-15	0.85	17.23	8.90	-5.56	14.46
20	0.9	18.24	9.64	-5.56	15.20
25	0.94	19.05	10.23	-5.56	15.79
30	0.98	19.87	10.82	-5.56	16.38
40	1.04	21.08	11.70	-5.56	17.26
50	1.09	22.09	12.44	-5.56	18.00
60	1.13	22.91	13.03	-5.56	18.60
70	1.17	23.72	13.63	-5.56	19.19
80	1.21	24.53	14.21	-5.56	19.78
90	1.24	25.14	14.66	-5.56	20.22
110	1.26	25.54	14.95	-5.56	20.51
120	1.31	26.55	15.69	-5.56	21.25

North – South Wind Loading								
Floor	Height (ft)	Tributary Height (ft)	Windward Pressure (psf)	Leeward Pressure (psf)	Total (psf)	Story Force (k)	Total Shear (k)	Overturning Moment (ft-k)
Ground	0	0	0	0	0	0	**V = 532**	**M = 31590**
2	15.33	15	9.64	-5.56	15.20	70.7	532.1	1084
3	30	14.67	10.53	-5.56	15.79	62.1	461.4	1863
4	44.67	14.67	11.70	-5.56	17.26	82.2	399.3	3672
5	59.33	14.67	12.74	-5.56	18.30	85.4	317.1	5067
6	74	14.67	13.63	-5.56	19.19	91.1	231.7	6741
7	88.67	14.67	14.21	-5.56	19.77	93.1	140.6	8255
Roof	103.33	7.33	14.95	-5.56	20.51	47.5	47.5	4908

East – West Wind Pressures					
Height (ft)	Kz	qz	Windward Pressure (psf)	Leeward Pressure (psf)	Total
0-15	0.85	17.23	10.83	-6.98	16.39
20	0.9	18.24	11.6	-6.98	17.24
25	0.94	19.05	12.36	-6.98	17.92
30	0.98	19.87	13.05	-6.98	18.61
40	1.04	21.08	14.07	-6.98	19.63
50	1.09	22.09	14.91	-6.98	20.47
60	1.13	22.91	15.60	-6.98	21.16
70	1.17	23.72	16.28	-6.98	21.84
80	1.21	24.53	16.96	-6.98	22.52
90	1.24	25.14	17.48	-6.98	23.04
110	1.26	25.54	17.81	-6.98	23.37
120	1.31	26.55	18.66	-6.98	24.22

East – West								
Floor	Height (ft)	Tributary Height (ft)	Windward Pressure (psf)	Leeward Pressure (psf)	Total (psf)	Story Force (k)	Total Shear (k)	Overturning Moment (ft-k)
Ground	0	0	0	0	0	0	**V = 175**	**M = 10528**
2	15.33	15	11.60	-6.98	18.58	18.1	175.1	278
3	30	14.67	12.71	-6.98	19.69	25.1	157.6	753
4	44.67	14.67	14.07	-6.98	21.05	27.4	132.5	1224
5	59.33	14.67	15.26	-6.98	22.24	28.4	105.1	1685
6	74	14.67	16.28	-6.98	23.26	30.2	76.7	2235
7	88.67	14.67	16.96	-6.98	23.94	30.8	46.5	2731
Roof	103.33	7.33	17.81	-6.98	24.79	15.7	15.7	1622

Appendix D.2 – Lateral Load Analysis – Seismic

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$S_s = 0.1518$
$S_1 = 0.0558$

Site Class C

$F_a = 1.2 \qquad F_v = 1.65$

$S_{ms} = F_a(S_s) = 1.2(0.1518) = 0.18216$

$S_{m1} = F_v(S_1) = 1.65(0.0558) = 0.09207$

$S_{Ds} = \frac{2}{3} S_{ms} = \frac{2}{3}(0.18216) = 0.12144$

$S_{D1} = \frac{2}{3} S_{m1} = \frac{2}{3}(0.09207) = 0.06138$

from table 12.8-1 $\quad S_{D1} \leq 0.1 \quad C_u = 1.7$

Concentrically Braced Frame & Existing Masonry Shear Wall

from table 12.8-2

$C_t = 0.02 \qquad x = 0.75$

$T_a = C_t(\text{height})^x$

$= 0.02(118')^{0.75} = 0.716$

period $= T = C_u T_a = 1.7(0.716) = 1.22$

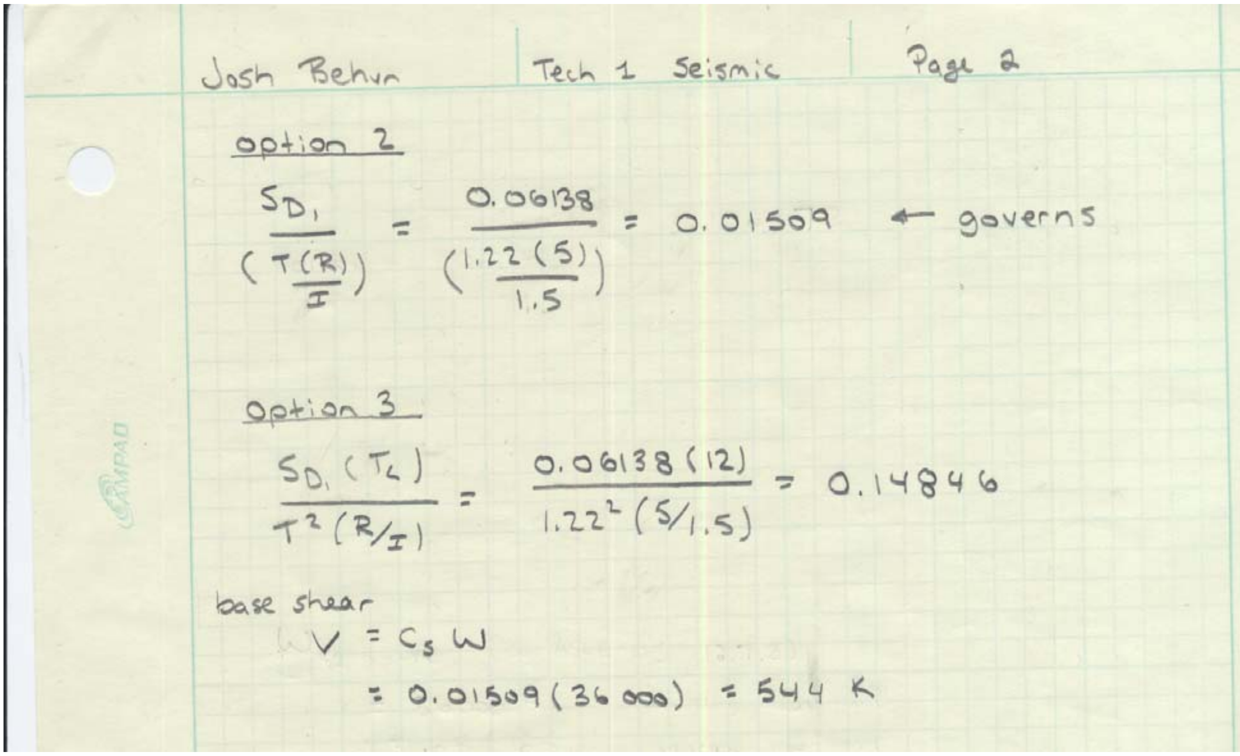
frequency $= \frac{1}{1.22} < 1 \quad \therefore$ Flexible

$R = 5$
$I = 1.5$

from table 22.15
$T_L = 12\ s$

<u>option I</u>

$\frac{S_{Ds}}{(R/I)} = \frac{0.12144}{(5/1.5)} = 0.03643$



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option 2

$$\frac{S_{D1}}{\left(\frac{T(R)}{I}\right)} = \frac{0.06138}{\left(\frac{1.22(5)}{1.5}\right)} = 0.01509 \quad \leftarrow \text{governs}$$

option 3

$$\frac{S_{D1}(T_L)}{T^2(R/I)} = \frac{0.06138(12)}{1.22^2(5/1.5)} = 0.14846$$

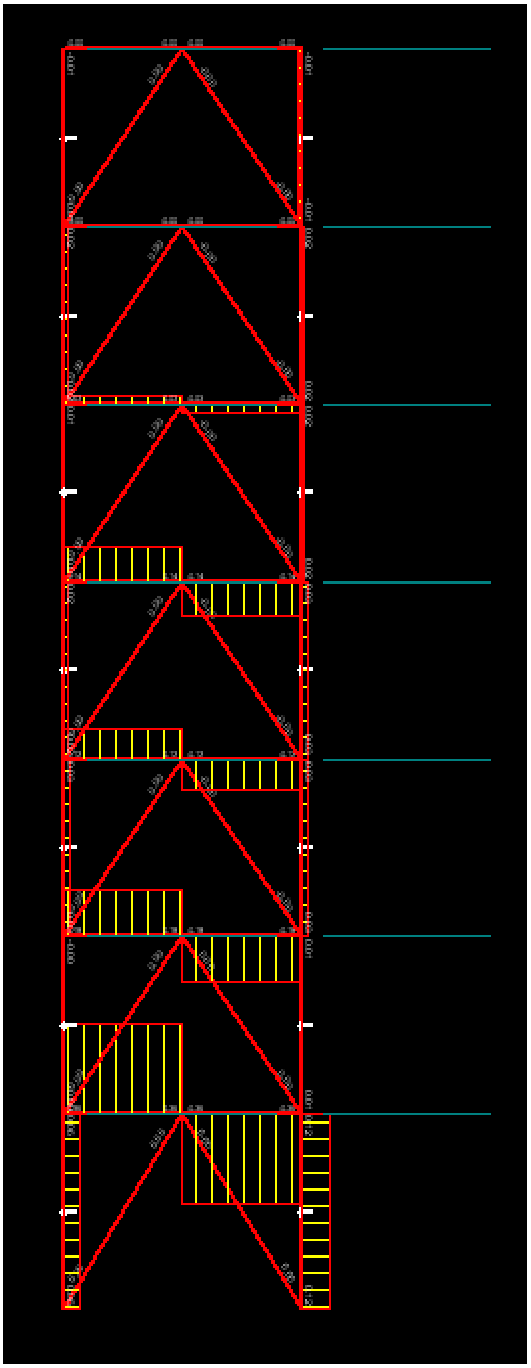
base shear

$$V = C_s W$$

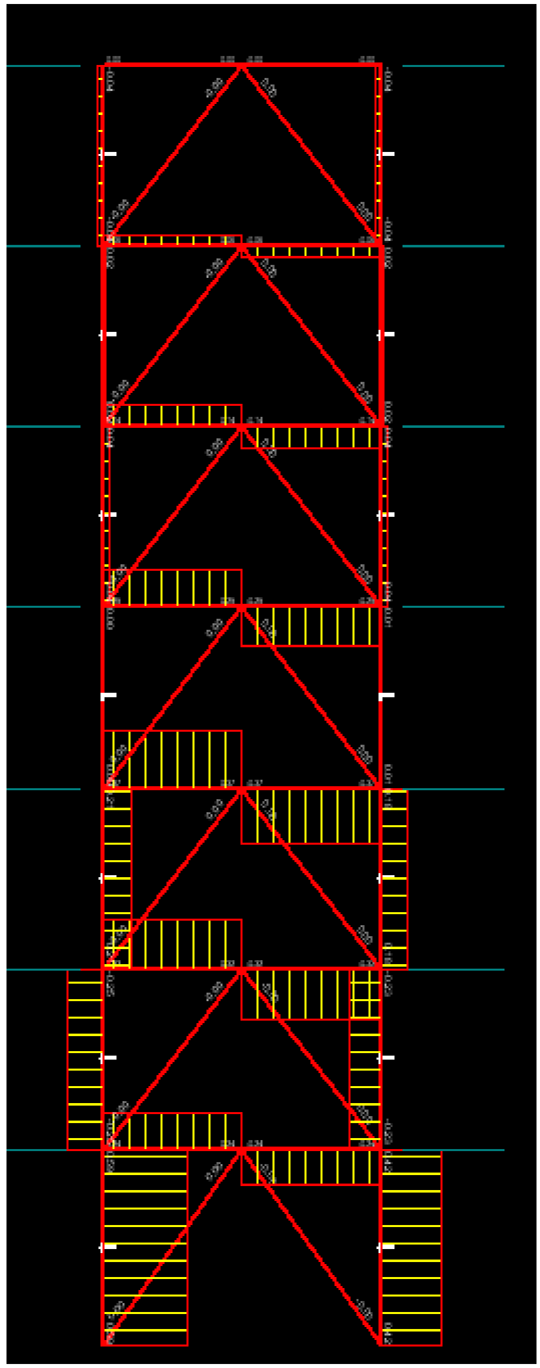
$$= 0.01509(36\,000) = 544 \text{ K}$$

Lateral Seismic Force Distribution								
Level	Weight (kips)	Story Height h (ft)	Exponent k	Wx*hx^k (kips)	Cvx	Story Force Fx (kips)	Vx (kips)	Mx (ft-kips)
1	12,510	15.33	1.36	512383	0.0896	48.7	48.7	747
2	6,545	30	1.36	668030	0.1168	63.5	112.2	1905
3	3,435	44.67	1.36	602488	0.1053	57.3	169.5	2560
4	2,460	59.33	1.36	634730	0.1109	60.3	229.8	3580
5	2,460	74	1.36	857215	0.1498	81.5	311.3	6030
6	2,460	88.67	1.36	1096255	0.1916	104.3	415.6	9250
7	2,460	103.33	1.36	1349842	0.2359	128.3	543.9	13257
Sum	32330	104		5720943	1.0	**V = 544 K**		**M = 37330 Ft-K**

Appendix E – Moment diagrams



Frame at Elevation 24



Frame at Elevation 11

Appendix F – Spot Checks

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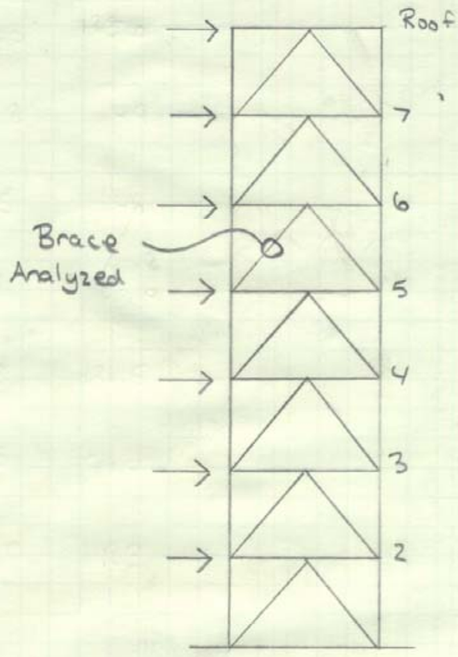
Frame page 1

Frame @ Elevation 24
(Along Eastern Wall)

seismic factor = 1.0
wind factor = 1.6



HSS 6x6 x 3/8

F_y = 36 ksi
A_s = 7.58 in²
L = 17.47'

Tension Force in Brace
From RAM output $T_u = 110^K$

$\phi T_N = \phi A_s F_y$
$= 0.9(7.58)(36) = 245.5^K$

$\phi T_N > T_u$ ok
$245.5 > 110$

Compression In Brace
@ $P_u = 110^K$ K for pin-pin connection = 1

$\frac{KL}{r} = \frac{17.47(1)}{2.28}(12) = 91.95 < 200$ ok

@ KL = 18'

$\phi P_N = 172^K > P_u = 110^K$ ok

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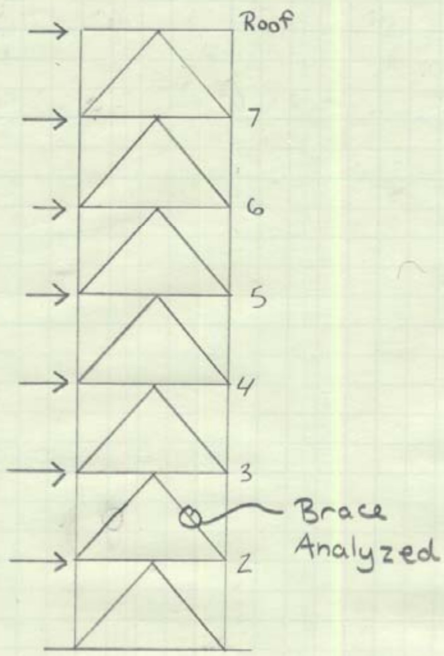
Frame page 2

Frame ⑨ Elevation 11



HSS $6 \times 6 \times 3/8$

$F_y = 36$

$A_s = 7.58 \text{ in}^2$

$L = 18.95'$

Tension Force in Brace

$= 80^k$

$\phi T_N = \phi A_s F_y$

$= 0.9(7.58)36 = 245.5^k > 80$ ok

Compression Force

@ $P_u = 80^k$

$$\frac{KL}{r} = \frac{18.95(1)}{2.28}(12) = 99.7 < 200 \text{ ok}$$

@ $KL = 19'$

$\phi P_N = 160^k > P_u = 80^k$ ok