School of Engineering and Applied Science Building

Miami University, Oxford, OH

Technical Assignment 3

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AE 481W – Senior Thesis

The Pennsylvania State University

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Executive Summary

Miami (OH) University's School of Engineering and Applied Science Building consists of four stories above grade, three of which are designated for classrooms and labs for students, as well as faculty offices. The fourth floor is a mechanical penthouse floor under a mansard roof which houses the building's main HVAC equipment. The building also has three levels of below-grade parking. The new building will connect to the existing Benton Hall by way of a skywalk at the 2^{nd} through 4^{th} floor. The architectural voice of the new building is largely based upon the aesthetic concepts of Benton Hall.

The structure's gravity load system uses a steel frame with composite concrete floor slabs on steel columns. Lateral loads are resisted with steel moment frames in the longitudinal (east-west) direction and concentrically braced steel frames in the transverse (north-south) direction. The concrete floor slabs act as rigid diaphragms which transfer to load to each frame based on relative stiffness. This report will investigate the building's lateral framing system and check for its adequacy against both strength and serviceability requirements.

After calculating the lateral loads acting on the building using ASCE 7-05, seismic loads were found to control the structure's design in both directions of the building. A computer model was constructed using ETABS to help determine the distribution of these loads to each framing element. Building drift was calculated by the computer model and found to be well with the acceptable limitations. Spot checks of a few framing elements were checked against strength requirements and found to be sufficient. Finally, the building was checked against overturning the overturning moment caused by lateral forces, and the building's resisting moment caused by its self weight was found to be significantly higher than the overturning moment, so overturning will not be an issue.

Lateral Resistance System

- ### North-South Direction

The lateral system in the transverse (short) direction of the building consists of four (4) single bay concentrically braced steel frames from the ground floor to the mechanical floor, of roughly the same size. There is only one cross brace at each of the three levels of the brace, sloping up from south-to-north, and are made of steel tubing, ranging in size from HSS8x8x¼ to HSS10x10x½. Elevations of each braced frame and their locations on plan can be found in Appendix A of this report. Additionally, there are two (2) single-span moment frames that support the skywalk that connects the west end of the School of Engineering and Applied Science Building to Benton Hall. At the eastern end of the building, there is also a moment frame with wide flange columns and HSS20x12x5/8 steel tube beams beside the stairwell. For lateral resistance from the mechanical floor to the roof, the mansard roof around the perimeter braces the roof, but is helped by four (4) single-span moment frames, which frame into the columns' weak bending axes.

- ### East-West Direction

The longitudinal (long) direction of the building utilizes an ordinary moment frame system, comprised of a total of eight (8) frames. There are four (4) full height moment frames that run from the ground floor all the way to the roof in the southern half of the building. The remaining four (4) frames in the northern half of the building are only two (2) stories tall, and stop at the low roof where the building steps back at the second floor level. Refer to the framing plans in Appendix A for the locations of each frame. The moment frames use a partially restrained moment connection that has plates bolted to the flanges, which then are welded with full-penetration welds into the columns supporting the beams.

- ### Garage

There are three levels of below grade parking, mostly of which is directly beneath the main building. However, the northern end of the garage is below the exterior terrace in the rear of the building, where the grading drops down to approximately one level below the ground floor. This causes the weight of the ground floor to induce seismic forces, which are then transferred to the foundation through the exterior walls of the garage, which all act as shear walls. The walls range in thickness from 8" to 14" depending on their location. This report is focused primarily on the lateral resisting system above ground level, so the shear walls will have to be analyzed more carefully in upcoming reports.

Design Codes

The School of Engineering and Applied Science Building was designed using the 2002 Ohio Building Code (OBC) with reference to ASCE 7-98 for building load determination procedures. ACI 318-99 was used to design the concrete portions of the structure, and concrete masonry construction was designed using ACI 530.1, Specifications for Masonry Structures, and construction specification section 04810. The 1992 edition of AISC's Code of Standard Practice for Steel Buildings and Bridges, as modified by the construction documents, was used for design of steel members, and ANSI/AWS Structural Welding Code – Steel D1.1 was used for design of welds.

This report will use the more recent IBC 2006 with reference to ASCE 7-05 for building loads. ACI 318-05, Building Code Requirements for Structural Concrete, and the Load Resistance Factored Design procedure from the 13th edition of AISC's Manual of Steel Construction will be used for design of the concrete and steel structural members, respectively.

Load Combinations

The following load combinations from Chapter 2 of ASCE 7-05 were used in evaluating ultimate factored loads used to check member capacities and for building overturning:

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

Design Loads

• Dead Loads

Item	Weight
Concrete (Normal Weight)	150 pcf
Typical Floor	62.5 psf
Upper and Middle Garage 9" Slab	112.5 psf
Ground Floor 10" slab	125 psf
Ground Floor 12" slab	150 psf
Metal Deck	2 psf
Steel Framing	8 psf
Ceiling and Mechanical Allowance	
Typical Floor	15 psf
Mechanical Floor	25 psf
Roof	10 psf
Garage	10 psf
Partition Allowance	10 psf
Roof Materials	
4" Rigid Insulation	6 psf
Roof Membrane	1 psf
1/2" Gypsum Board	2 psf

• Live Loads

It is worthy to note that ASCE 7-05 does not specify live loads for labs such as the ones within the School of Engineering and Applied Sciences Building, which is what the majority of the space within the building is designated for. The designer chose to use a uniform load of 100 psf for upper level labs and 125 psf for labs at ground floor, which is what this report will use in the analysis.

Area	Design Live Load
Typical Floor	100 psf
Labs at Ground Level	125 psf
Mechanical Equipment Rooms	150 psf
Plaza	100 psf
Roof	25 psf
Parking Decks	50 psf
PSE Basement at Upper Garage Level	125 psf
Utility Tunnel	250 psf + 360 psf overburden

- ## Wind Loads

Wind loads determined for the School of Engineering and Applied Science Building were carried out under Section 6 of ASCE 7-05. Factors were based on building characteristics, location, and height of the building. Assumptions include the normalization of the building's shape into a rectangle, ignoring any indentations or extrusions in the façade, and that the walls around the mechanical floor are actually plumb rather than sloped as a mansard roof was made to simplify the analysis, which results in a conservative wind force at that level. The building was found to be rigid and was analyzed as such. It is worthy to note that a large expansion joint exists where the new building attaches to the existing Benton Hall which is fairly open. As such, wind loading in the East-West direction has two effective modes, one where the windward pressure is acting in combination with the internal pressure, and one where the leeward pressure acts with the internal pressure, but not a combination of the windward and leeward pressure on the whole building. The building is in occupancy category III since it is a college facility with a capacity of over 500 people, which results in a wind importance factor of 1.15. A summary of the analytical procedure is presented with this section. Refer to Appendix B for loading diagrams and a more detailed analysis.

Design Summary			
Design Parameter	Symbol	Value	ASCE 7-05 Reference
Occupancy category		III	Table 1.1
Wind design Wind method		Method 2	
Wind importance factor	I	1.15	Table 6-1
Exposure category		B	Section 6.5.6.3
Enclosure classification		Enclosed	
Wind directionality factor	k_d	0.85	Section 6.5.4.4 & Table 6-4
Topographical factor	k_z	1.00	Table 6.5.7.2
Basic wind speed	V	90 mph	Figure 6-1
Approximate building period	T_a	0.438 s	Equation 12.8-7
Gust effect factor	G	0.85	Section 6.5.8
North-South length		356.25 ft	
East-West length lower 2 levels		134.0 ft	
East-West length top 2 levels		86.0 ft	
Height above grade	h_n	61.33 ft	
Base shear N-S Wind	**V**	**413 k**	
Overturning moment N-S Wind	**M**	**13,776 ft-k**	
Base shear E-W Wind	**V**	**87 k**	
Overturning moment E-W Wind	**M**	**2572 ft-k**	

- ## Seismic Loads

Seismic loads determined for the School of Engineering and Applied Science Building were carried out under Section 11 of ASCE 7-05 using the equivalent lateral force design method. The ETABS computer model was helpful in determining the building's actual period in both the longitudinal and transverse directions, which allows for a more accurate calculation of the controlling C_s. The building is in occupancy category III since it is a college facility with a capacity of over 500 people, which results in a seismic importance factor of 1.25. Design assumptions and a summary of the analytical procedure are presented within this section. Refer to Appendix C for loading diagrams and a more detailed analysis.

Seismic Design Summary			
Design Parameter	Symbol	Value	ASCE 7-05 Reference
Occupancy category		III	Table 1.1
Site classification		C	Table 20.3-1
Seismic Design Category	SDC	B	Tables 11.6-1 & 2
Seismic importance factor	I	1.25	Table 11.5.1
Short period spectral response	S_s	0.171g	Section 11.4.1
Acceleration-based Site coefficient	F_a	1.2	Table 11.4-1
Maximum short period spectral response	S_{DS}	0.137	Equation 11.4-3
Spectral Response at 1 sec	S_1	0.073g	Section 11.4.1
Velocity-based site coefficient	F_v	1.7	Table 11.4-2
Maximum spectral response at 1 sec	S_{D1}	0.083g	Equation 11.4-4
Response modification factor	R	3.0	Table 12.2-1
Deflection amplification factor	C_d	3.0	Table 12.2-1
N-S building period	T	0.594 s	Calculated on ETABS
E-W building period	T	1.150 s	Calculated on ETABS
Long-period transition period	T_L	12 s	Figure 22-15
N-S Seismic design coefficient	C_S	0.0570	Section 12.8.1.1
E-W Seismic design coefficient	C_S	0.0300	Section 12.8.1.1
Height above grade	h_n	61.33 ft	
Base shear N-S loading	**V**	**836.1 k**	
Overturning moment N-S loading	**M**	**32,475 ft-k**	
Base shear E-W loading	**V**	**439.9 k**	
Overturning moment E-W loading	**M**	**18,189 ft-k**	

Serviceability Considerations

Drift limits for both seismic and wind loadings were compared with drift values computed by the ETABS computer model under service loads. Seismic drift at each story was evaluated against $\Delta_s = 0.015h_{sx}$ in accordance with IBC Table 1617.3. Wind drift for the entire building was evaluated against the commonly accepted engineering value of $\Delta_w = H/400$. The following table shows the calculated drift values of a point at the northeast corner of the building under both seismic and wind loads.

Seismic Story Drift				
Story	Height (ft)	ETABS Drift in x-direction (in)	ETABS Drift in y-direction (in)	Allowable Drift = $0.015h_{sx}$ (in)
Roof	61.33	2.147	1.441	11.04
Mech.	48.00	2.011	1.335	8.64
2nd	33.33	1.330	0.836	6.00
1st	16.67	0.683	0.365	3.00
Ground	4.00	0.008	0.001	0.72

Wind Story Drift				
Story	Height (ft)	ETABS Drift in x-direction (in)	ETABS Drift in y-direction (in)	Allowable Drift = H/400 (in)
Roof	61.33	0.288	0.318	1.84
Mech.	48.00	0.273	0.273	1.44
2nd	33.33	0.199	0.181	1.00
1st	16.67	0.110	0.080	0.50
Ground	4.00	0.001	0.001	0.12

Analysis and Conclusions

As expected from previous investigations, seismic forces control the design of the lateral system in both the north-south and east-west directions. The increased stiffness of the braced frames in the transverse direction of the building cause the fundamental period to be approximately half of that in the longitudinal direction where moment frames make the structure relatively flexible. This resulted in a seismic base shear of nearly twice the magnitude for the braced frames to resist than the moment frames. It is also worthy to note that since the ground floor is slightly above grade, that it induces seismic forces in the system of a much larger magnitude than any other floor since the two-way slab floor comprises 45% of the total building weight considered for seismic base shear. For future investigation, if there is a way to raise the grading around the building to be at the ground floor diaphragm's level around the building's perimeter everywhere except the garage level, seismic forces will be drastically reduced, thus resulting in a much simpler lateral resisting system.

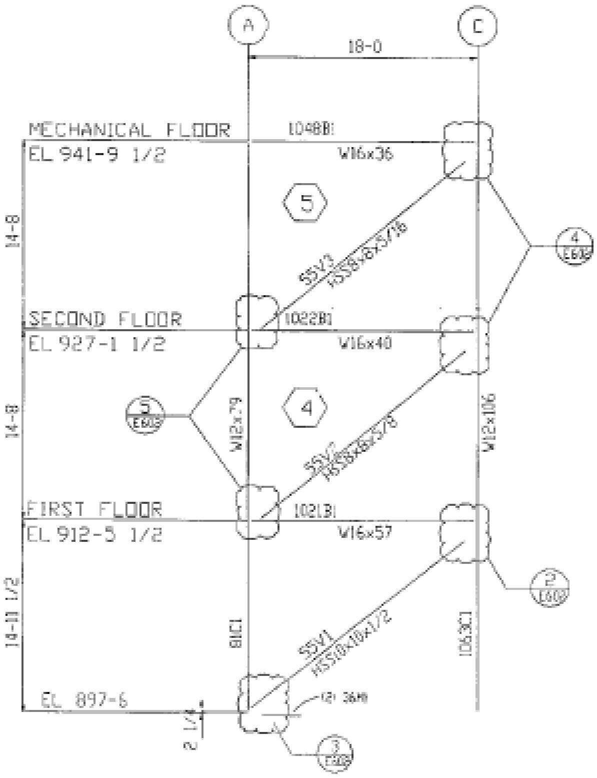
The ETABS computer model formed was a very helpful tool in determining distribution of lateral forces to individual resisting elements. By calculating relative stiffness of each frame, the program can accurately determine how much load is transferred to each brace and moment frame. The building's relatively symmetrical shape causes very little eccentric rigidities, so the 5% accidental building eccentricity caused small torsional shears to be induced in framing elements near the building's perimeter.

Using results from the computer model, strength checks were performed on a select number of lateral framing elements, all of which were found to be well within code limitations. ETABS was able to perform accurate drift calculations which were used to compare to industry standard limitations of H/400 for wind and $0.015h_{sx}$ for seismic drifts. Displacement for both load cases in each direction was found to be well within the accepted limits.

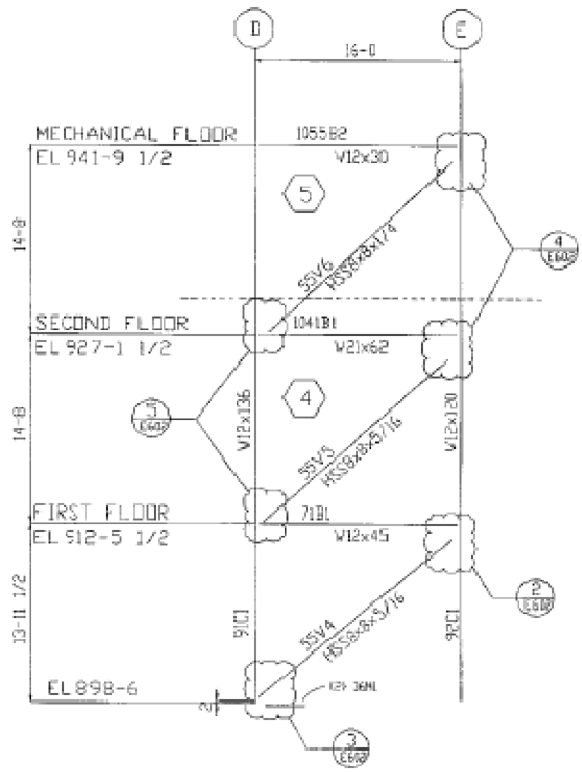
Finally, the structure was checked for possible overturning caused by lateral forces. The resisting moment caused by the building's self weight was found to be much higher than the largest overturning moment induced by lateral forces, so no net tension will need to be considered for foundation design.

Appendix A – Plans and Diagrams

Braced Frame Diagrams



Elevation at Lines 3 & 8
(Looking West)



Elevation at Lines 5 & 6
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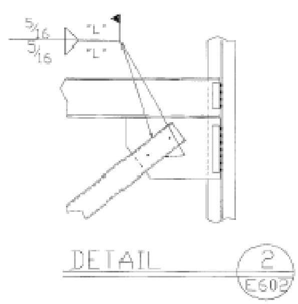
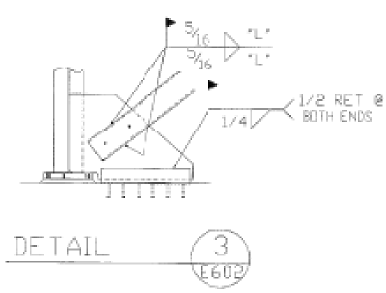
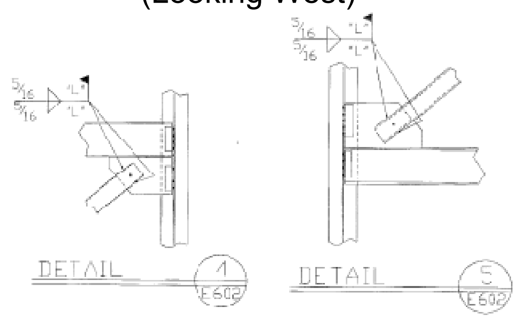


GRID LOCATION	"L" LENGTH OF FIELD WELD
C-3	20"
E-5	11"
E-6	11"
C-8	20"



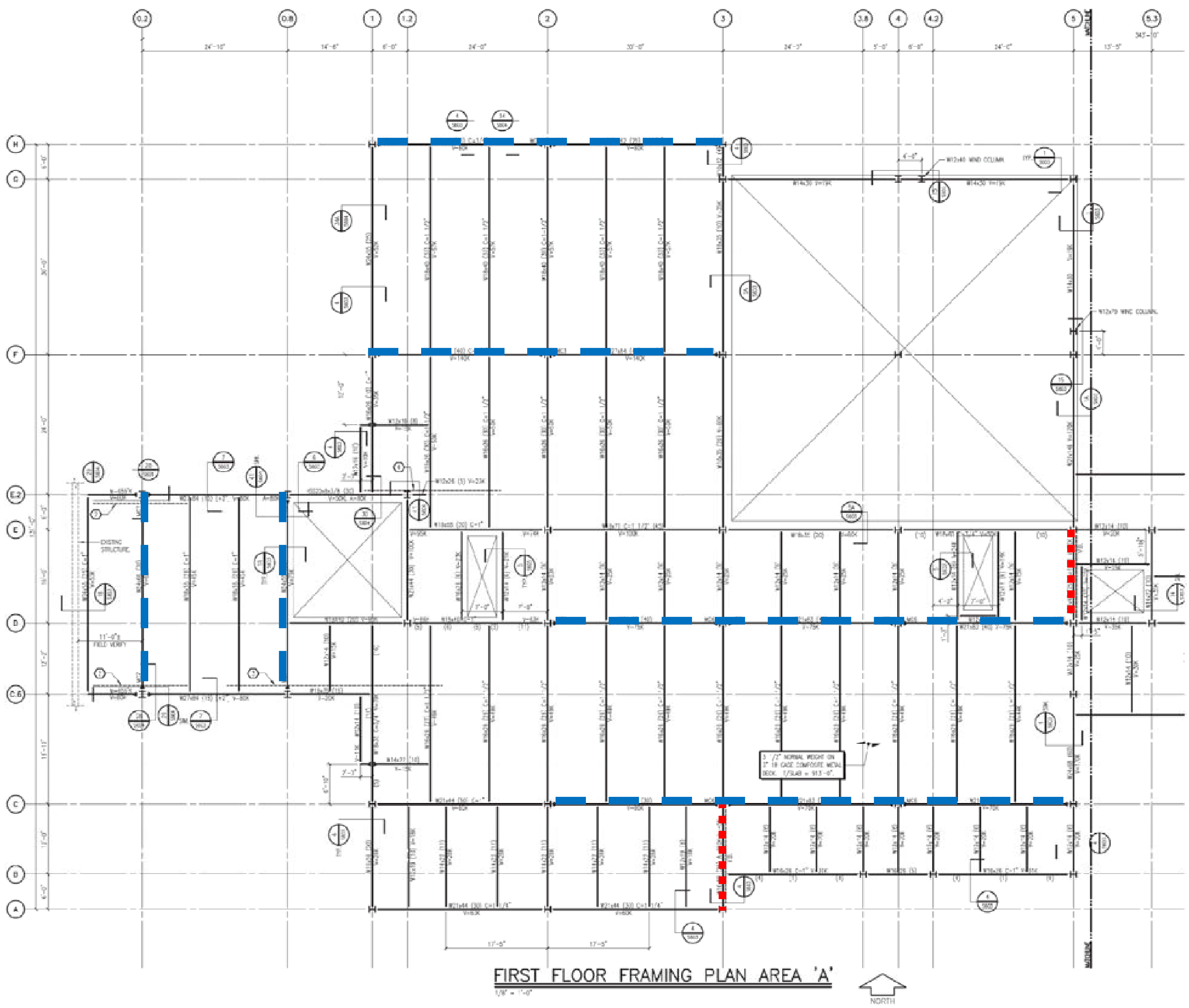
GRID LOCATION	"L" LENGTH OF FIELD WELD
A-3	20"
D-5	10"
D-6	10"
A-8	20"



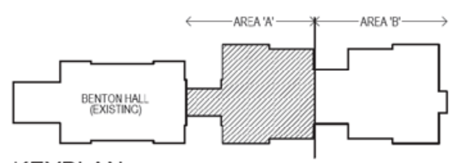
GRID LOCATION	ELEVATION	"L" LENGTH OF FIELD WELD
C-3	927'-1½	14"
C-3	941'-9½	8"
E-5	927'-1½	10"
E-5	941'-9½	8"
E-6	927'-1½	10"
E-6	941'-9½	8"
C-8	927'-1½	14"
C-8	941'-9½	8"

GRID LOCATION	ELEVATION	"L" LENGTH OF FIELD WELD
A-3	912'-5½	14"
A-3	927'-1½	8"
D-5	912'-5½	10"
D-5	927'-1½	8"
D-6	912'-5½	10"
D-6	927'-1½	8"
A-8	912'-5½	14"
A-8	927'-1½	8"

First Floor Framing Plan – Area 'A' (West half of building)



Legend

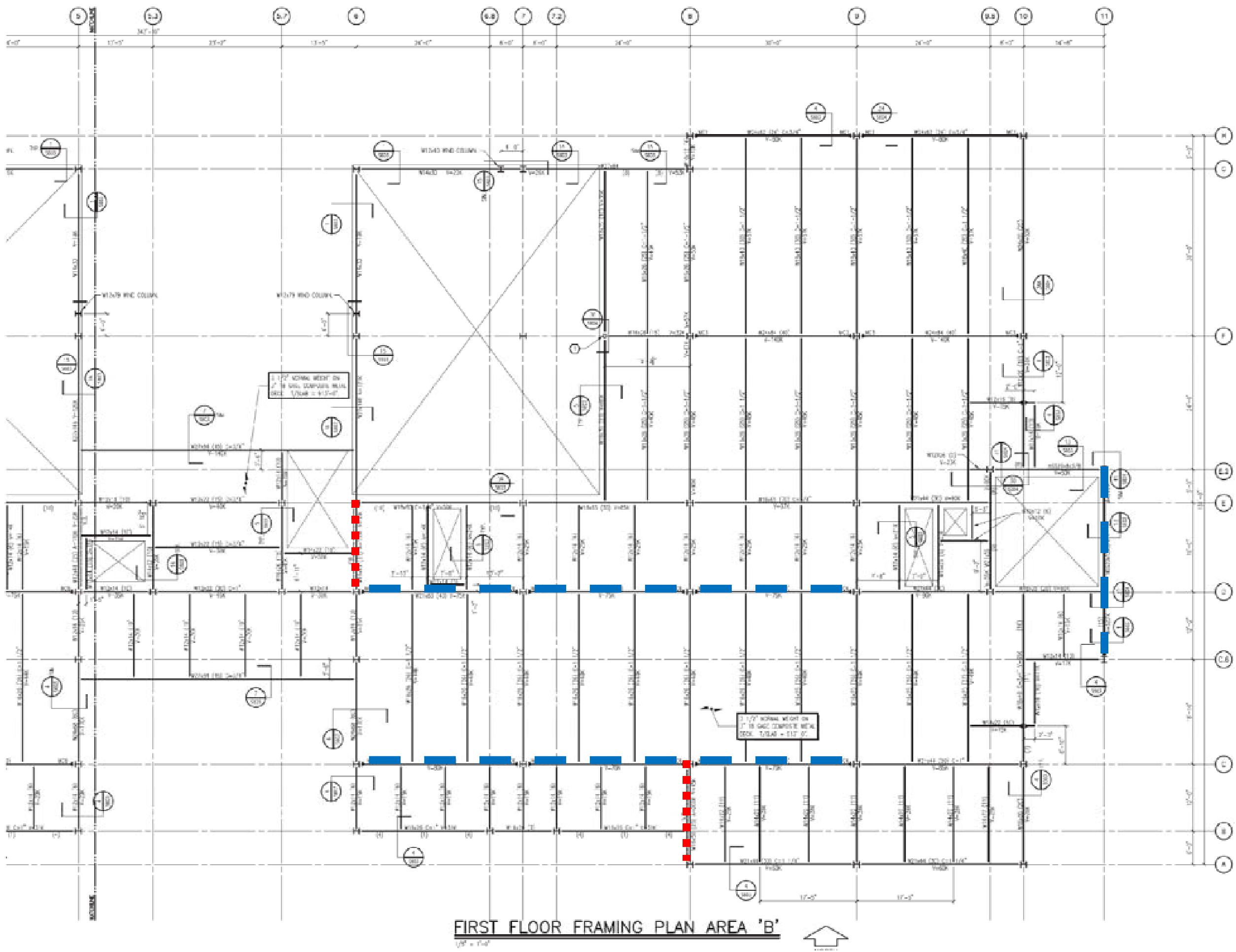


KEYPLAN

Braced Frame (red dotted line)

Moment Frame (blue dashed line)

First Floor Framing Plan – Area ‘B’ (East half of building)



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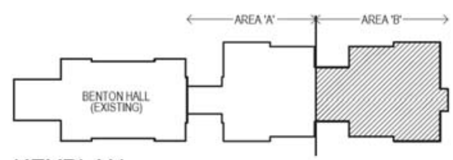
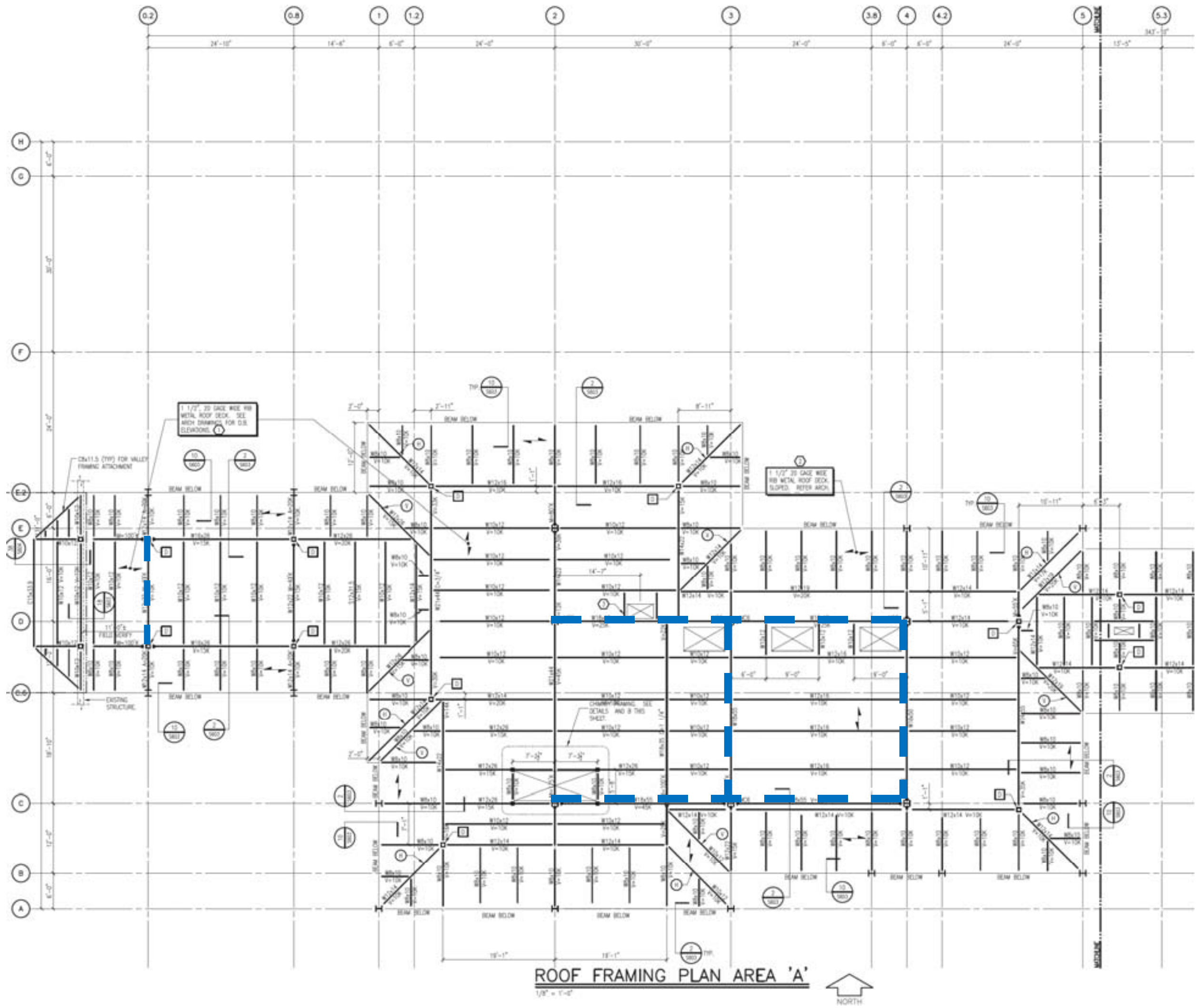


KEYPLAN

Braced Frame (red dotted line)

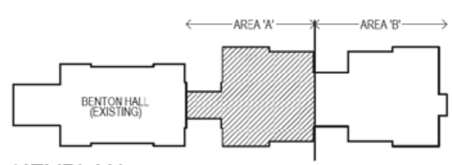
Moment Frame (blue dashed line)

Roof Framing Plan – Area 'A' (West half of building)



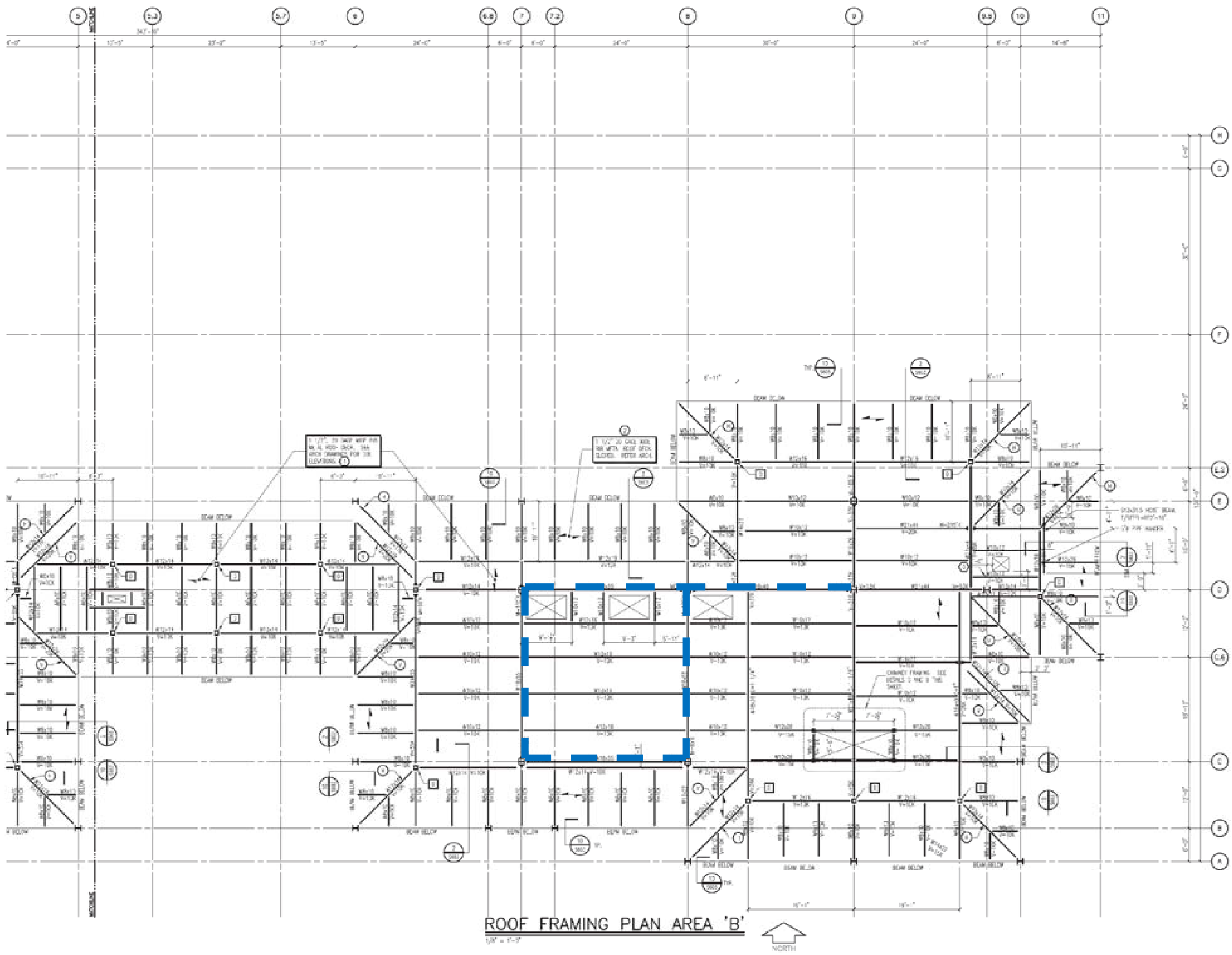
Legend

Moment Frame (blue dashed line)



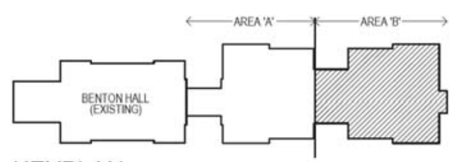
KEYPLAN

Roof Framing Plan – Area 'B' (East half of building)



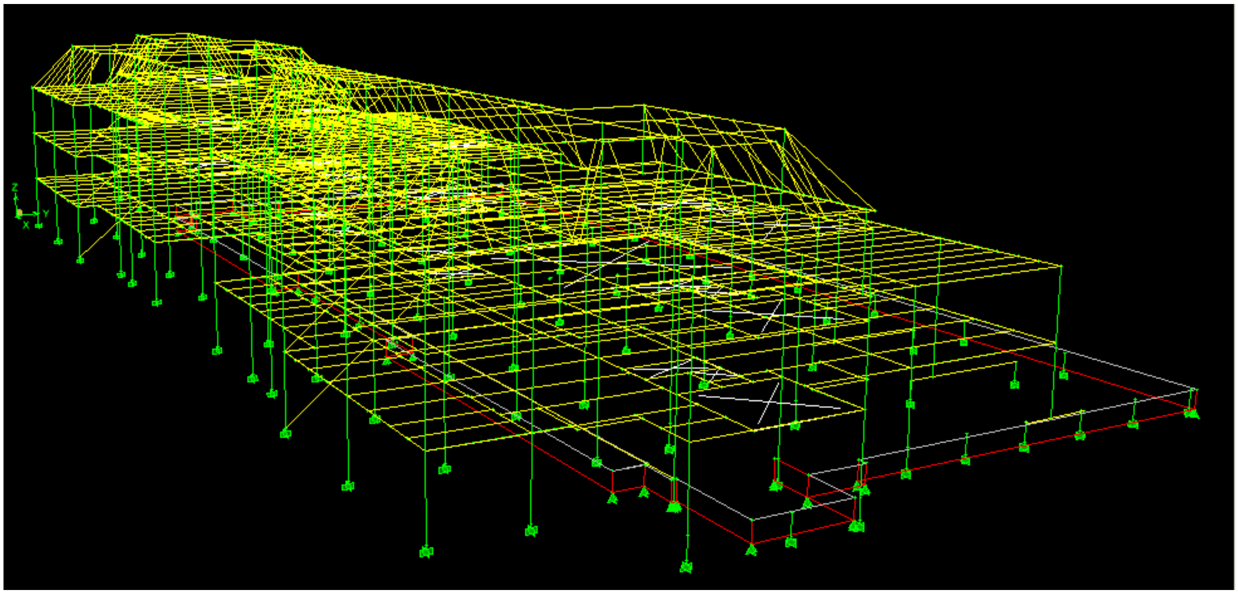
Legend

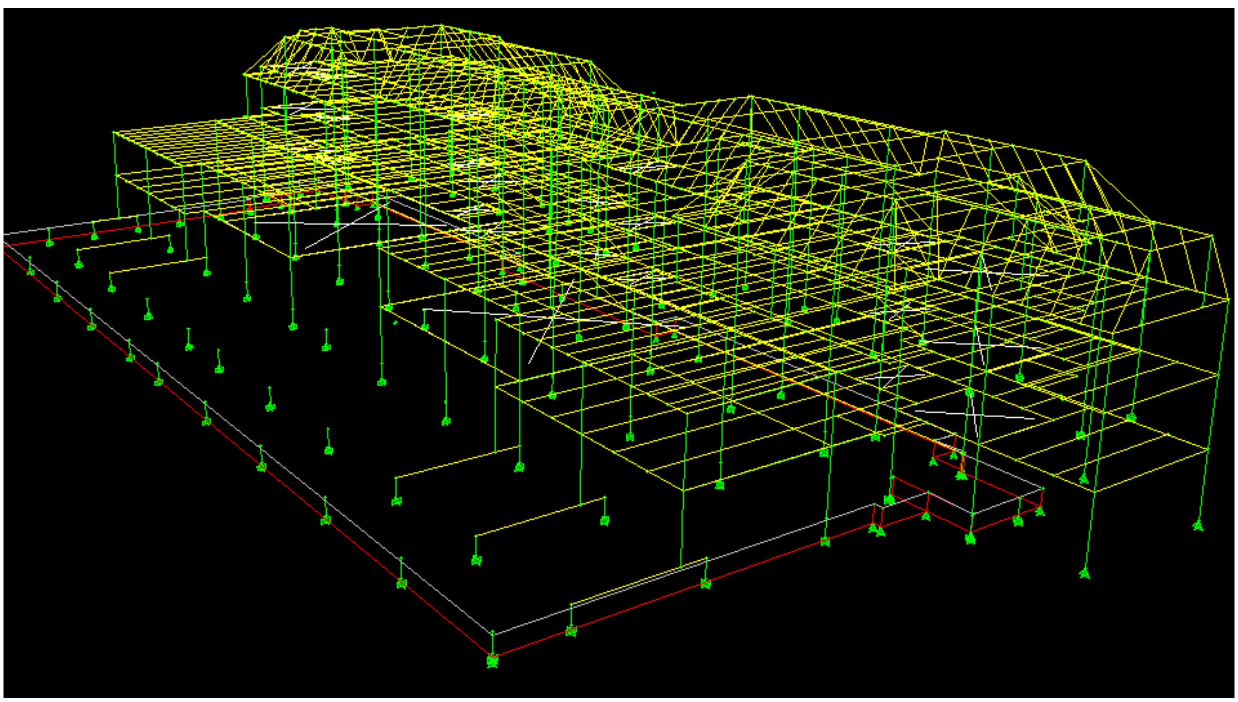
Moment Frame (blue dashed line)



KEYPLAN

ETABS Model



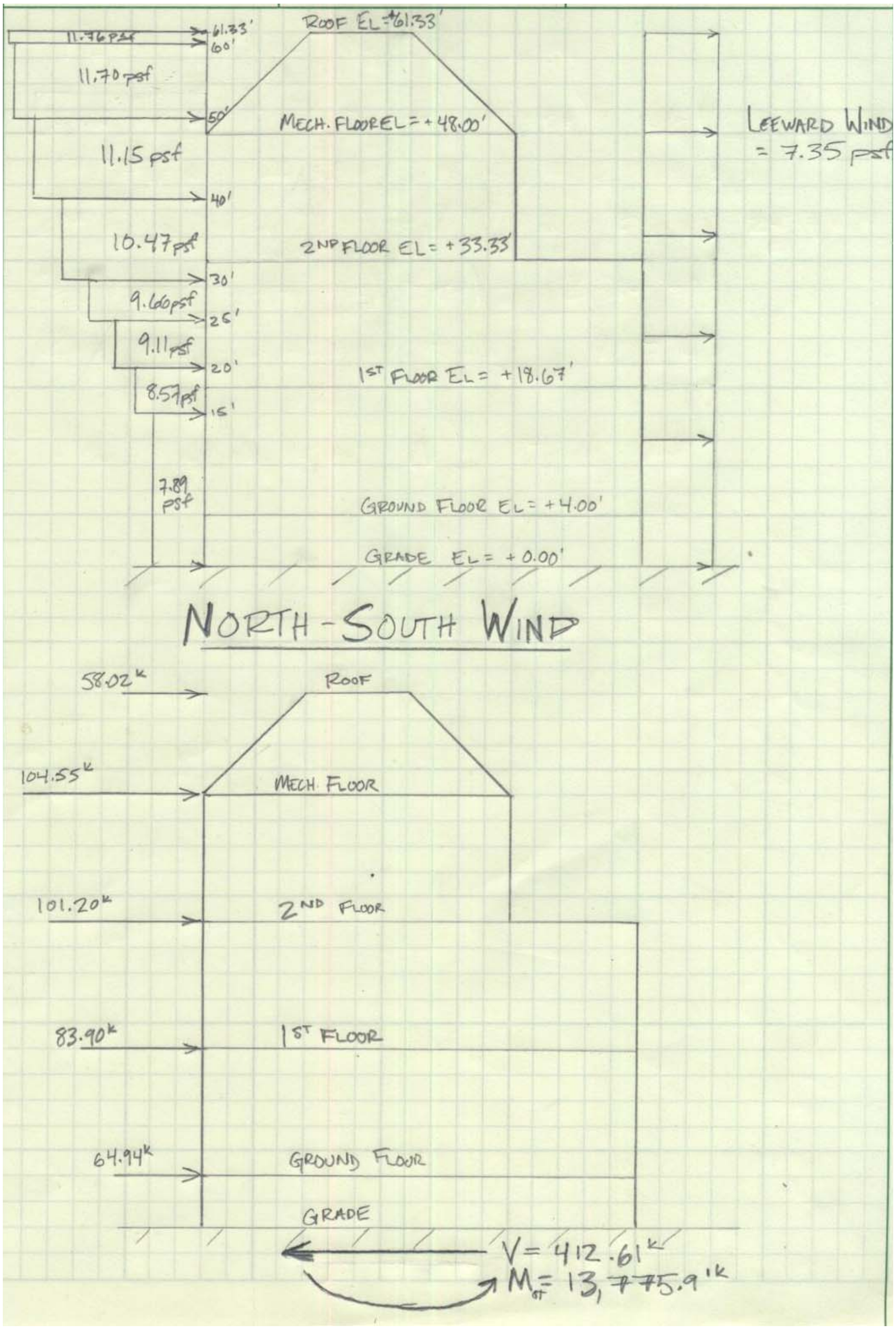


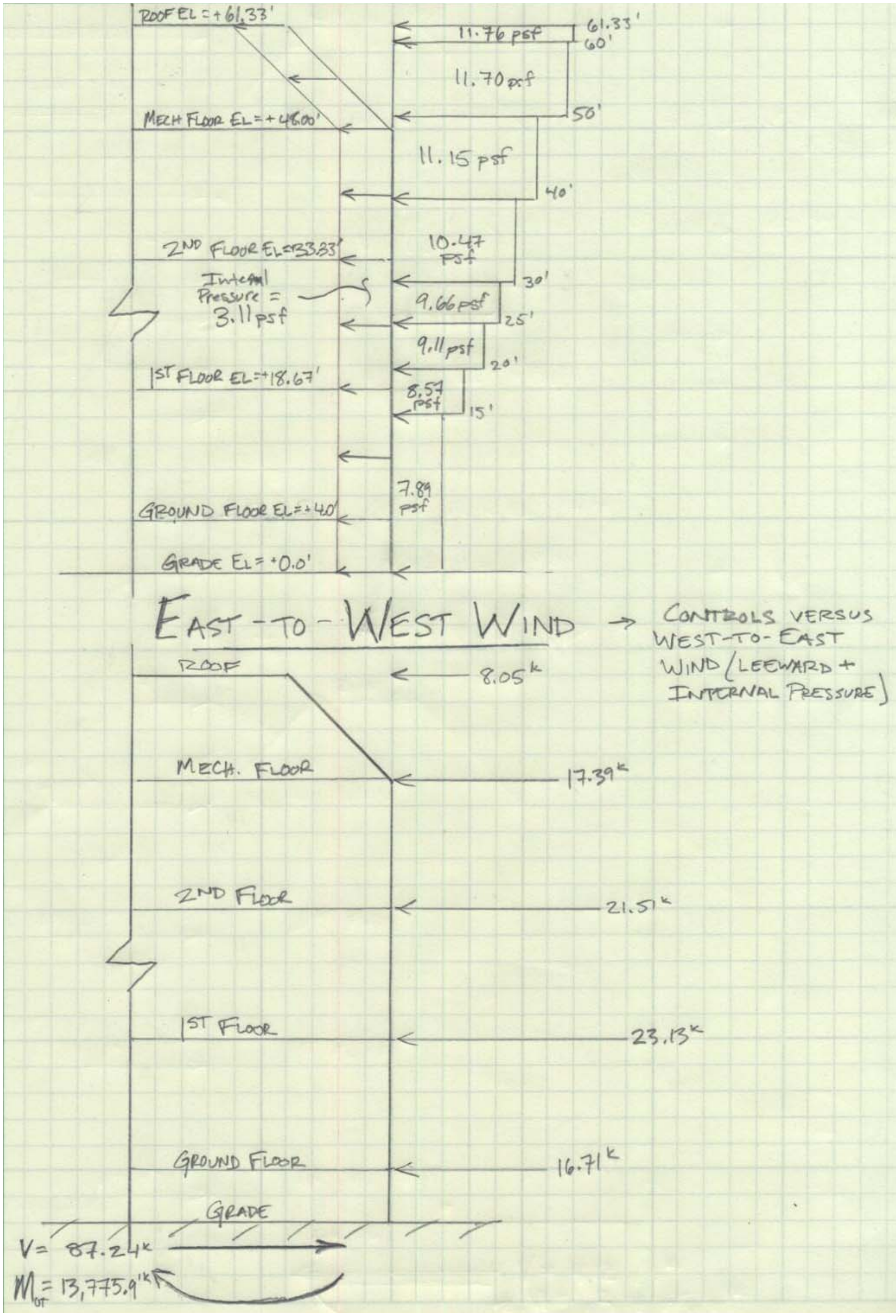
Appendix B – Wind Analysis

North-South Wind Loading						
Height above ground (ft)	Kz	qz (psf)	Pressure (psf)			
			Windward	Leeward	Sidewall	Internal
0-15	0.57	11.6	7.89	-7.35	-10.29	±3.11
20	0.62	12.6	8.57	-7.35	-10.29	±3.11
25	0.66	13.4	9.11	-7.35	-10.29	±3.11
30	0.70	14.2	9.66	-7.35	-10.29	±3.11
40	0.76	15.4	10.47	-7.35	-10.29	±3.11
50	0.81	16.4	11.15	-7.35	-10.29	±3.11
60	0.85	17.2	11.70	-7.35	-10.29	±3.11
61.33	0.86	17.3	11.76	-7.35	-10.29	±3.11

East-West Wind Loading						
Height above ground (ft)	Kz	qz (psf)	Pressure (psf)			
			Windward	Leeward	Sidewall	Internal
0-15	0.57	11.6	7.89	-3.88	-10.29	±3.11
20	0.62	12.6	8.57	-3.88	-10.29	±3.11
25	0.66	13.4	9.11	-3.88	-10.29	±3.11
30	0.70	14.2	9.66	-3.88	-10.29	±3.11
40	0.76	15.4	10.47	-2.94	-10.29	±3.11
50	0.81	16.4	11.15	-2.94	-10.29	±3.11
60	0.85	17.2	11.70	-2.94	-10.29	±3.11
61.33	0.86	17.3	11.76	-2.94	-10.29	±3.11

Wind Direction		North-South Wind		East to West Wind		West to East Wind	
Floor	Height above ground (ft)	Force (k)	Overturning Moment (ft-k)	Force (k)	Overturning Moment (ft-k)	Force (k)	Overturning Moment (ft-k)
Roof	61.33	58.02	3558.4	8.5	521.3	3.47	212.8
Mech.	48.00	104.55	5018.4	17.39	834.7	7.28	349.4
2nd	33.33	101.20	3373.0	21.51	716.9	10.68	356.0
1st	18.67	83.90	1566.4	23.13	431.8	13.74	256.5
Ground	4.00	64.94	259.8	16.71	66.8	10.62	42.5
Sum		412.61	13775.9	87.24	2571.6	45.79	1217.2


11.76 psf
61.33'
60'
11.70 psf
50'
11.15 psf
40'
10.47 psf
30'
9.66 psf
25'
9.11 psf
20'
8.57 psf
15'
7.89 psf
ROOF EL = +61.33'
MECH. FLOOR EL = +48.00'
2ND FLOOR EL = +33.33'
1ST FLOOR EL = +18.67'
GROUND FLOOR EL = +4.00'
GRADE EL = +0.00'
LEEWARD WIND = 7.35 psf
NORTH-SOUTH WIND
58.02k
ROOF
104.55k
MECH. FLOOR
101.20k
2ND FLOOR
83.90k
1ST FLOOR
64.94k
GROUND FLOOR
GRADE
V = 412.61k
M = 13,775.9'k



ROOF EL = +61.33'
11.76 psf
61.33'
60'
11.70 psf
50'
MECH FLOOR EL = +48.00'
11.15 psf
40'
10.47 psf
2ND FLOOR EL = 33.33'
30'
Internal Pressure = 3.11 psf
9.66 psf
25'
9.11 psf
20'
1ST FLOOR EL = +18.67'
8.57 psf
15'
7.89 psf
GROUND FLOOR EL = +4.0'
GRADE EL = +0.0'
EAST-TO-WEST WIND → CONTROLS VERSUS WEST-TO-EAST WIND (LEEWARD + INTERNAL PRESSURE)
ROOF
8.05k
MECH. FLOOR
17.39k
2ND FLOOR
21.51k
1ST FLOOR
23.13k
GROUND FLOOR
16.71k
GRADE
V = 87.24k
M_OT = 13,775.9'k

Wind Load Story Force Calculations

– North-South Wind

$P_{GRND} = (365.25')(11.33')(7.89 + 7.35) = \underline{64.94^k}$

$P_{1st} = (365.25')\left[(15'-11.33')(7.89 + 7.35) + (5')(7.35 + 8.57) + (5')(7.35 + 9.11) + (1')(7.35 + 9.66)\right] = \underline{83.90^k}$

$P_{2nd} = (365.25')\left[(30-26)(9.66 + 7.35) + (10')(10.47 + 7.35) + (0.67')(11.15 + 7.35)\right] = \underline{94.44^k}$

$P_{mech} = (365.25')\left[(50-40.67)(11.15 + 7.35) + (4.67')(11.70 + 7.35)\right] = \underline{104.55^k}$

$P_{roof} = (365.25')\left[(1.33')(11.76 + 7.35) + (60-54.67)(11.70 + 7.35)\right] = \underline{58.02^k}$

– East-West Wind (windward)

$P_{GND} = (134')(11.33')(7.89 + 3.11) = \underline{16.71^k}$

$P_{1st} = (134')\left[(15-11.33)(7.89 + 3.11) + (5)(8.57 + 3.11) + (5)(9.11 + 3.11) + (1')(9.66 + 3.11)\right] = \underline{23.13^k}$

$P_{2ND} = (134')\left[(4')(9.66 + 3.11) + (3.33')(10.47 + 3.11)\right] + \ldots + (86')\left[(6.67')(10.47 + 3.11) + (0.67')(11.15 + 3.11)\right] = \underline{21.51^k}$

$P_{mech} = (86')\left[(9.33')(11.15 + 3.11) + (54.67)(11.70 + 3.11)\right] = \underline{17.39^k}$

$P_{roof} = (86')\left[(5.33')(11.70 + 3.11) + (1.33)(11.77 + 3.11)\right] = \underline{8.50^k}$

Appendix C – Seismic Analysis

Project Location	Oxford, OH	
Project Latitude	39.505833°	
Project Longitude	-84.739167°	
Occupancy Category	III	
Seismic Importance Factor	1.25	
Site Classification	C	
S_s	0.171g	
F_a	1.2	
$S_{MS} = F_a S_s =$	0.205g	
$S_{DS} = (2/3)S_{MS} =$	0.137g	
S_1	0.073g	
F_v	1.7	
$S_{M1} = F_v S_s =$	0.124g	
$S_{D1} = (2/3)S_{M1} =$	0.083g	
Seismic Design Category	B	
Seismic Resisting System	Structural Steel System Not Specifically Detailed for Seismic Resistance	
Direction	N-S	E-W
R	3.0	3.0
C_d	3.0	3.0
h_n	61.33	61.33
C_u	1.6234	1.6234
C_t	0.02	0.028
x	0.75	0.8
$T_a = C_t h_n^x =$	0.4383 s	0.7539 s
$T_{max} = C_u T_a =$	0.7116 s	1.2238 s
T_{actual}*	0.5942 s	1.1496 s
T_L	12 s	12 s

* Note: T_{actual} calculated by ETABS

North-South Braced Frames

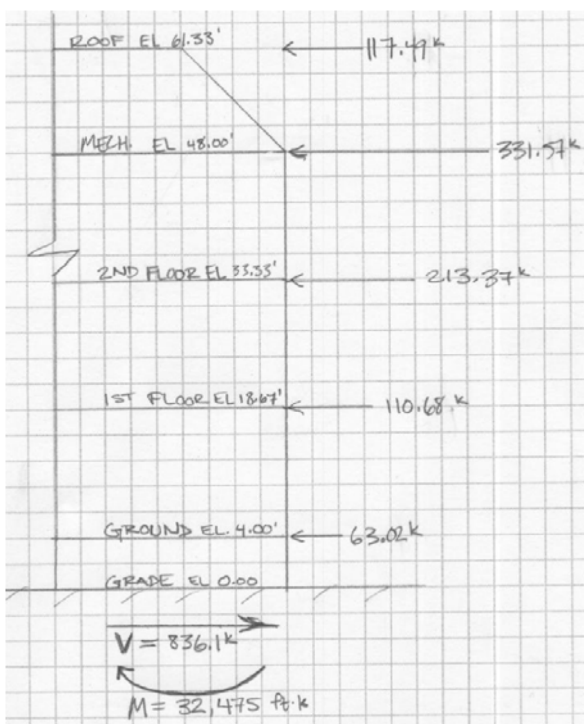
$$C_s = \min \left\{ \begin{array}{ll} S_{DS}/(R/I) = & 0.0570 \\ S_{D1}/(T(R/I)) = & 0.0580 \\ S_{D1}T_L/(T^2(R/I)) = & 1.1716 \end{array} \right\} \geq 0.01$$

Controlling $C_s = 0.0570$

$W = 14{,}669$ k

$V = C_sW = 836.1$ k

Lateral Seismic Force Distribution Through the Levels (North-South Braced Frames)								
Level	Story Height h_x (ft)	Story Weight W (k)	Exponent k	$\Sigma\, w_i h_i^k$	C_{vx}	Story Force f_x (k)	Shear V_x (k)	Moment M_x (ft-k)
Roof	61.33	707	1.0471	52637	0.1405	117.49	117.5	7206
Mech.	48.00	2579	1.0471	148552	0.3966	331.57	449.1	15916
2nd	33.00	2457	1.0471	95596	0.2552	213.37	662.4	7041
1st	18.67	2314	1.0471	49588	0.1324	110.68	773.1	2066
Grnd.	4.00	6612	1.0471	28233	0.0754	63.02	836.1	252
Sum		W = 14669		373690		**V = 836.1 k**		**M = 32475 ft-k**



East-West Moment Frames

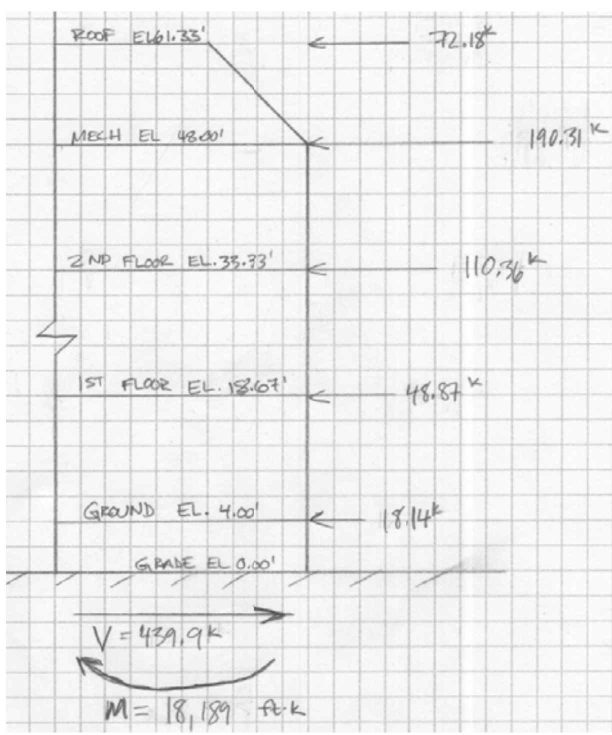
$$C_s = \min \left\{ \begin{array}{ll} S_{DS}/(R/I) = & 0.0570 \\ S_{D1}/(T(R/I)) = & 0.0300 \\ S_{D1}T_L/(T^2(R/I)) = & 0.3130 \end{array} \right\} \geq 0.01$$

Controlling $C_s = 0.0300$

W = 14,669 k

$V = C_sW = 439.9$ k

Lateral Seismic Force Distribution Through the Levels (East-West Moment Frames)								
Level	Story Height h_x (ft)	Story Weight W (k)	Exponent k	$\Sigma w_i h_i^k$	C_{vx}	Story Force f_x (k)	Shear V_x (k)	Moment M_x (ft-k)
Roof	61.33	707 k	1.3248	165093	0.1641	72.18	72.2	4427
Mech.	48.00	2579 k	1.3248	435271	0.4326	190.31	262.5	9135
2nd	33.00	2457 k	1.3248	252425	0.2509	110.36	372.9	3642
1st	18.67	2314 k	1.3248	111783	0.1111	48.87	421.7	912
Grnd.	4.00	6612 k	1.3248	41490	0.0412	18.14	439.9	73
Sum		W = 14669		1006062		**V = 439.9 k**		**M = 18189 ft-k**



Appendix D – Overturning

FLOOR & BUILDING DEAD LOADS

GROUND FLOOR = (38,217 SF)(173 PSF) = 6612 kips

1ST FLOOR = (26,320 SF)(98 PSF) = 2579 kips

2ND FLOOR = (21,197 SF)(98 PSF) + (11,520 SF)(33 PSF) = 2457 kips
(FLOOR) (ROOF)

MECH. FLOOR = (21,427 SF)(108 PSF) = 2314 kips

ROOF = (21,427 SF)(33 PSF) = 707 kips

TOTAL BUILDING WEIGHT (W) = 14,669 kips

OVERTURNING

RESISTING MOMENT OF FIRST FLOOR → ROOF FROM SOUTHERN EDGE OF BUILDING

$$M_R = \frac{\underbrace{(707^k)(432'')}_{ROOF} + \underbrace{(2314^k)(498'')}_{MECH} + \underbrace{(2457^k)(786'')}_{2ND\ FLR} + \underbrace{(2579^k)(786'')}_{1ST\ FLR}}{12\ in/ft}$$

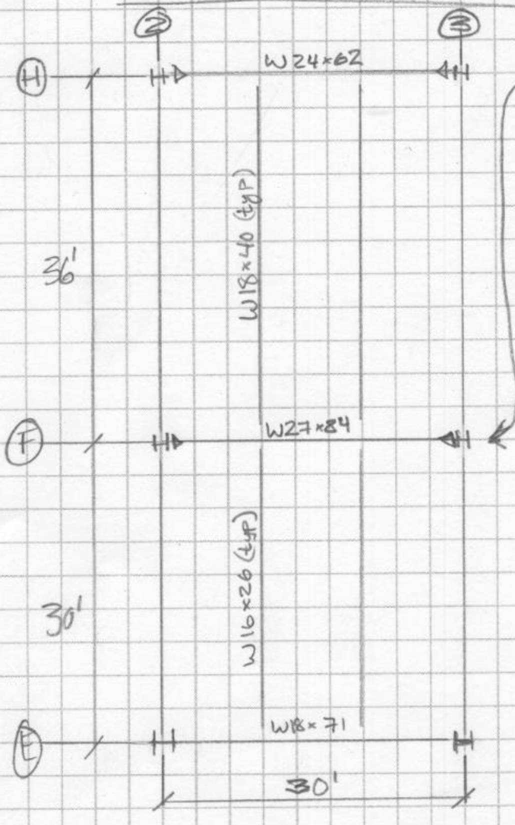
M_R = 451,341 ft·k

M_{OT} = 32,475 ft·k IN N-S DIR

$M_R \gg M_{OT}$ ∴ OVERTURNING IS NOT AN ISSUE

Appendix E – Spot Checks

SPOT CHECK OF COMPOSITE GIRDER



GIRDER F2-F3 @ the 1ST FLOOR

W27×84

$A_s = 24.8 \text{ in}^2$

3½ NWC slab on 3" comp deck

$f'_c = 4000$ psi

$w_D = 87.5$ psf

$w_L = 100$ psf

1.2D + 1.0E + L will control by inspection

P_u from W18×40:

$$P_u = \frac{36}{2}\left[1.2\left(40 + (10)(87.5)\right) + (10)(100)\right]$$

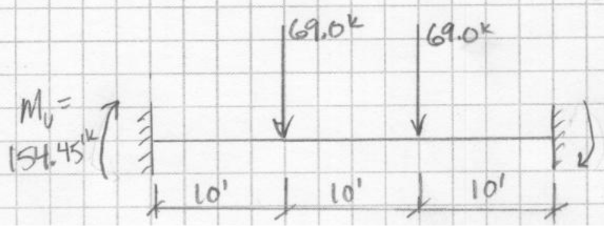
$$= 37{,}764 \text{ lb}$$

P_u from W16×26:

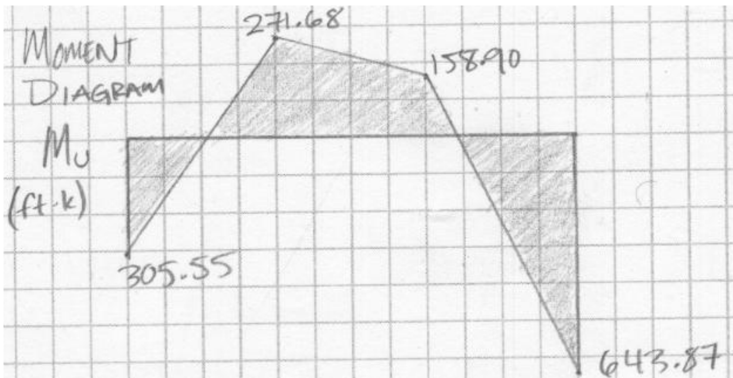
$$P_u = \frac{30}{2}\left[1.2\left(26 + (10)(87.5)\right) + (10)(100)\right]$$

$$= 31{,}218 \text{ lb}$$

Combined P_u = 69.0^k



Note: From ETABS model based on seismic load in the E-W dir.



$$\boxed{\phi M_n^- = 915.0^{\prime k} > 643.87^{\prime k} = M_U^- \quad \therefore \text{ ok}✓}$$

$\Sigma Q_n = A_s f_y = (24.8\text{ in}^2)(50\text{ ksi}) = 1240^k$

$b_{eff} = 1/4 \text{ span} = .25(30') = 7.5'$

$a_{req} = \dfrac{\Sigma Q_n}{.85\, b_{eff} f'_c} = \dfrac{1240^k}{.85(7.5')(12)(4\text{ ksi})} = 4.052''$

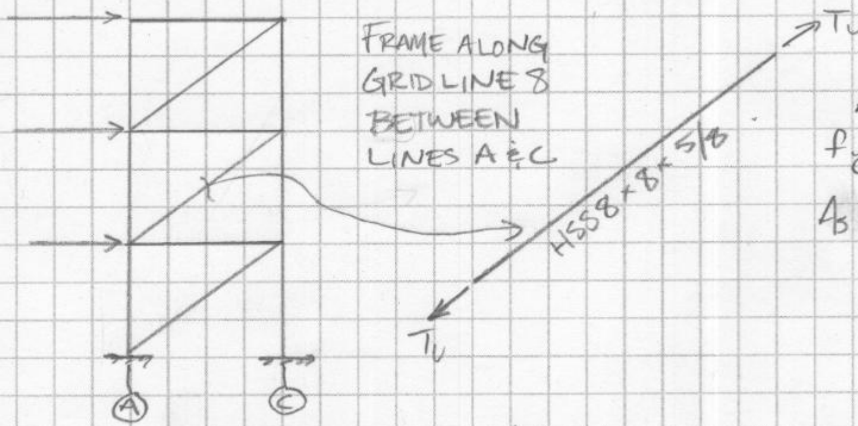
$Y2 = 6.5'' - \dfrac{4.052''}{2} = 4.474''$

TFL $\rightarrow \Sigma Q_n = 1240^k \geq 1240^k$ ok

$\phi M_n^+ (Y2 = 4.474'') = 1098.4^{\prime k}$ (from steel manual Table 3-19)

$$\boxed{\phi M_n^+ = 1098.4^{\prime k} \gg 271.68^{\prime k} = M_U \quad \therefore \text{ ok}✓}$$

Spot Check of a Lateral Brace in Tension



ASTM A-500 GRADE B

$f_y = 46$ ksi

$A_s = 16.4$ in²

$T_u = 304.02^k$ (from ETABS analysis)

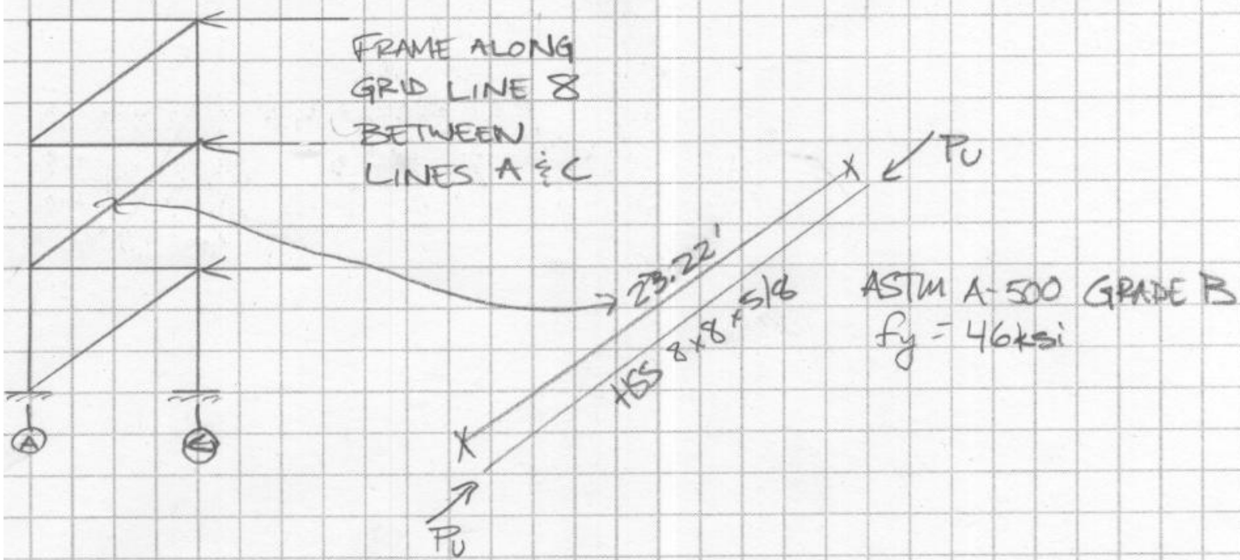
from 1.2D + 1.0E + L load combination

$\phi T_n = \phi A_s f_y$

$= 0.90(16.4\text{ in}^2)(46\text{ ksi})$

$\phi T_n = 678.96^k \geq T_u = 304.02^k \therefore$ ok ✓

SPOT CHECK OF LATERAL BRACE IN COMPRESSION



$P_u = 170.79^k$ (from ETABS analysis)
from 1.2D + 1.0E + L load combination

$L = \sqrt{(14.67ft)^2 + (18ft)^2} = 23.22'$

K = 1.0 for pinned-pinned compression member

$\frac{KL}{r} = \frac{1.0(23.22')}{2.99''}(12''/') = 93.2 < 200$ ok✓

FROM TABLE 4-4 OF AISC STEEL MANUAL,

@ KL = 24', for HSS 8×8×5/8,

$\phi P_n = 363^k \geq P_u = 170.79^k$ ∴ ok✓