School of Engineering and Applied Science Building

Miami University, Oxford, OH

Technical Assignment 1

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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 purpose of this report is to research the methods used to design the existing structural system for Miami University's new building for the School of Engineering and Applied Science. A description of the building's foundation, floor system, columns, and lateral resisting system are included within the report. Though the building was originally designed under the 2002 Ohio Building Code, this report utilizes the more recent 2006 International Building Code (IBC) with reference to ASCE 7-05 for calculations of loads.

Wind load analysis was performed using Method 2 on the main wind force resisting system (MWFRS). The pressures on the walls were distributed as loads on the floor diaphragms on the basis of tributary area. The seismic loads on the building were computed using the equivalent lateral force procedure. In addition, spot checks were performed on a typical composite beam, an interior column, and a vertical braced frame to check for member adequacy under my simplified analysis of the building. Any discrepancies obtained make no claim in any way that any of the designer's approaches, assumption, calculations, or resulting designs are incorrect or unsuitable.

Structural System

- ## Foundation

The lower level of the parking garage is a 5" slab on grade with a minimum 28-day compressive strength of 4500 psi, over 6" of granular subbase. It is reinforced with WWF 6x6 – W4.0xW4.0 wire mesh. The concrete columns, which carry the load from the main building above are supported by spread footings which range in size from 4'-0"x4'-0"x24" reinforced with (7)#5 bars each way to 9'-0"x9'-0"x42" reinforced with (15)#8 bars each way. The garage walls around the exterior are supported by 2'-0"x2'0" footings reinforced with (3)#9 top and bottom steel, while the wall footing running through the center of the garage is only 1'6" deep and reinforced with (2)#7 bottom bars. The School of Engineering and Applied Science Building's entrance plaza is a slab on grade with a minimum 28 day compressive strength of 4000 psi which varies by location from 5" thick reinforced with WWF 6x6 W4.0xW4.0 to 9" thick reinforced with #5 bottom bars at 12" O.C. and top WWF 6x6 W4.0xW4.0. The plaza is supported by drilled piers that range in size from 36" diameter, 12'-8" deep, to 60" diameter, 17'-4" deep. Grade beams run between the drilled piers and are typically 2'-0"x2'0". All footings, piers, and grade beams have a minimum concrete strength of 5000 psi.

- ## Floor System
  - ### Upper Floors

The first, second and mechanical floor of the School of Engineering and Applied Science Building utilizes a composite floor system with a typical concrete slab of 3½" on 3" 18 gage composite metal deck with normal weight concrete of minimum 28-day strength of 4000 psi, and is reinforced with WWF 6x6 W2.9xW2.9. The most typical bay is 30'-0"x30'-0" where the deck spans over (3) 10' spans on W16x26 beams with (26) ¾" diameter, 5" headed shear studs, and are cambered 1½". The beams frame into W21x83 girders at third-points, which have (40) shear studs of equal dimensions, and are not typically cambered. Girders in areas with larger tributary areas, in the north side of the building are W24x84's. These girders are also part of the lateral resisting system in the East-West direction and are supported with partially restrained moment connections at the columns. The roof is a mansard roof around the perimeter, sloping at a 12-12 pitch until it flattens off through the central part of the building. The roof does not have a composite slab, and is built of 4" rigid insulation on 1½" 20 gage wide rib roof deck, which spans on wide flange beams which are typically W8x10 on the pitched part of the roof, and are W10x12 or W12x16 in the central, flat area. The beams frame into girders which are generally W18x55.

- ### Garage

The middle and the upper levels of the garage, as well as the ground floor of the main building are comprised of a 2-way reinforced concrete slab with a minimum 28-day compressive strength of 5000 psi. The bay layout generally follows that of the columns above, typically 30'-0"x30'-0", from the main building to avoid the need for transfer slabs and girders. The middle and upper levels of the garage use a 9" flat slab with 10'-0"x10'-0"x8" drop panels at the columns. At the east end of the upper level, the slab turns into a 10" flat slab, and continues to turn into a 12" flat slab at ground floor, particularly on the northern half of the building. This is due to the fact that the live load on the ground floor is higher than anywhere else throughout the main building or garage. There are (3) transfer beams in this northern section of the main floor spanning north to south where the garage column layout doesn't exactly match that of the upper floors, which are 50" deep and are 36" or 48" wide. At the easternmost end of the building, there is a small section of slab where it is thickened to 14" to carry the some masonry walls.

Columns

- ### Upper Floors

Columns supporting the first floor through the roof are rolled W12 shapes with a yield strength of 50 ksi. Most of the columns contribute to the moment frame in the East-West direction, which range in size from W12x40 to W12x136. Where the columns continue all the way to the main roof through the mechanical floor, they are spliced just above the mechanical floor level. The base plates of gravity columns typically 1¼" – 1½" thick on 2" of non-shrink grout, with (4) anchor bolts embedded 16" into the ground floor concrete, and are assumed to act as pin connections. Columns acting as part of the moment frames or the vertical braces have heavier 2" – 2¼" thick, much larger in area so that the anchor bolts can be placed outside of the columns' projected area, unlike the gravity columns, and are assumed to act as fixed connections.

- ### Garage

The concrete columns in the garage are typically 24"x24", and have specified concrete strengths of either 4500 psi or 5000 psi depending on the location, and hence load, on the column. Reinforcement in the columns varies from (4)#11 bars to (12)#11 bars and splice at the middle level of the garage. The number of dowels at the base of the columns follows the number of reinforcement bars in the column, and are embedded to the bottom of the spread footing and hooked, creating a fixed connection.

- ## Lateral Resistance System

  - ### North-South Direction

The lateral system in the transverse (short) direction of the building consists of four 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. Diagrams can be found in Appendix A of this report. For lateral resistance from the mechanical floor to the roof, the mansard roof around the perimeter helps to brace the roof, but is helped by four single-span moment frames, which frame into the column's weak bending axis.

  - ### East-West Direction

The longitudinal (long) direction of the building utilizes an ordinary moment frame system. Two of the frames in the southern half of the building run the full length of the main building, and are the only two lateral resisting elements at the upper floors where the building steps back at the 2^{nd} floor level. The ground and 1^{st} floor also have four additional, shorter moment frames, two on each side of the rear entrance plaza at the center of the building. 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.

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 318-98 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 13^{th} edition of AISC's Manual of Steel Construction will be used for design of the concrete and steel structural members, respectively.

Material Strengths

The following table shall be referenced for required material strengths of all structural members:

Material Strength Specifications				
Concrete	Concrete Mix	Use	f'_c (psi)	Other Requirements
	Mix A	Footings, Drilled Piers, and Grade Beams	3000	
	Mix B	Elevated Parking Decks and Columns	5000	Air-Entrained, 7.5% Silica Fume
	Mix C	Garage Walls and Rails, Garage Slab on Grade, Concrete Pavements, and Exterior Concrete Exposed to Weather U.N.O.	4500	Air-Entrained
	Mix D	Ground Level Structural Deck	5000	
	Mix E	Stair Pan Fills	3000	#8 Aggregate
	Mix F	Interior Slabs on Metal Deck, Interior Building Slabs on Grade	4000	
	Mix G	Backfill (Lean) Concrete	1500	
	Mix H	Utility Tunnel	4000	
Reinforcing Steel	All Reinforcing		60 ksi Yield Strength	
	Reinforce all slabs as follows U.N.O., Furnish Mesh in Flat Sheets :			
	Slabs on Metal Deck		6x6-W2.9xW2.9 (42# WWF)	
	Interior and Exterior Slabs on Ground		6x6-W2.9xW2.9 (42# WWF)	
	Wearing Slabs Over Waterproof Membranes and Fill Slabs		6x6-W2.9xW2.9 (42# WWF)	
Structural Steel	Structural Shape or Part	ASTM Specification	f_y	
	Wide Flange Shapes	ASTM A-992	50 ksi	
	Rolled Shapes, Plates, and Bars	ASTM A-36	36 ksi	
	Hollow Structural Shapes (HSS Shapes)	ASTM A-500, Grade B	46 ksi	
	Steel Pipe	ASTM A-53, Type E or S, Grade B	35 ksi	
	Field Bolts	ASTM A-325, 3/4" Diameter, U.N.O	–	
	Anchor Bolts	ASTM F-1554, U.N.O.	36 ksi	
Masonry	CMUs	Material Strength (Net Area)	1900 psi	
		f'_m	1500 psi	
	Grout	Material Strength	3000 psi with 3/8" aggregate, 6" to 8" slump	

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. A summary of the analytical procedure is presented with this section.

Design Summary			
Design Parameter	Symbol	Value	ASCE 7-05 Reference
Building 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. Design assumptions and a summary of the analytical procedure are presented within this section.

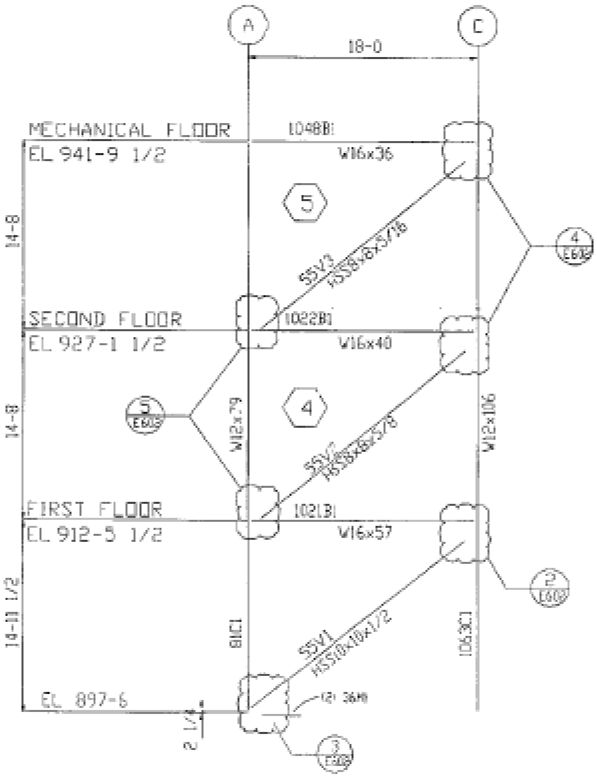
Seismic Design Summary			
Design Parameter	Symbol	Value	ASCE 7-05 Reference
Occupancy category		III	Table 1.1
Site classification		C	Table 20.3-1
Seismic Design Category	SDC	B	Tables 11.6-1 & 2
Seismic importance factor	I	1.25	Table 11.5.1
Short period spectral response	S_s	0.171g	Section 11.4.1
Acceleration-based 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
Effective approximate building period	T	0.712 s	Equation 12.8-7
Long-period transition period	T_L	12 s	Figure 22-15
Seismic design coefficient	C_S	0.0484	Section 12.8.1.1
Height above grade	h_n	61.33 ft	
Base shear	**V**	**710.7 k**	
Overturning moment	**M**	**28,026 ft-k**	

• Lateral Load Analysis and Conclusions

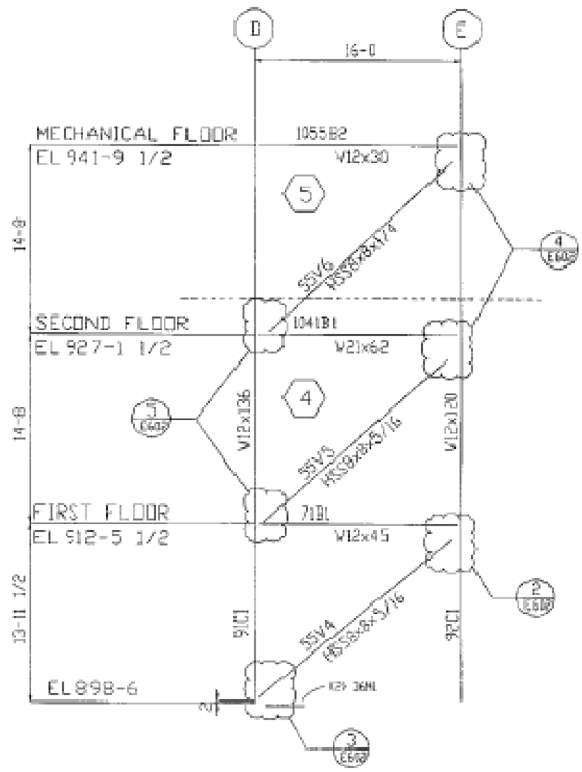
Based on my analysis, seismic forces clearly control the design of the lateral system in both directions. The base shear that the design professional calculated was 480 k in the N-S direction, and 395 k in the E-W direction. The reason they have a different base shear in each direction may be from a building specific analysis of the fundamental period, which will have to be investigated further at a later time. This analysis may have resulted in a longer period, which could explain why their base shear was so much lower than mine. Also, I used some conservative estimates of the building mass. A more detailed analysis of actual building weight may result in lighter building weight, and hence create a significantly lower base shear. Use of the newer codes may also account for some differences in seismic load calculations.

Appendix A – Plans and Diagrams

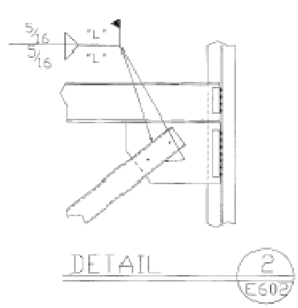
Braced Frame Diagrams



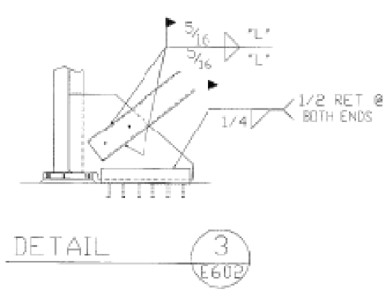
Elevation at Lines 3 & 8
(Looking West)



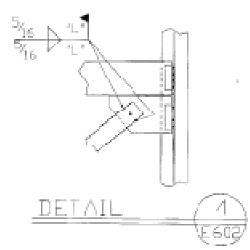
Elevation at Lines 5 & 6
(Looking West)



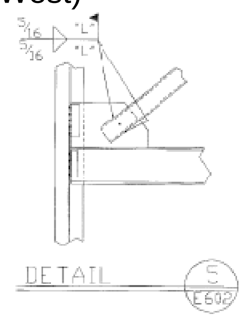
GRID LOCATION	"L" LENGTH OF FIELD WELD
C-3	20"
E-5	11"
E-6	11"
C-8	20"



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

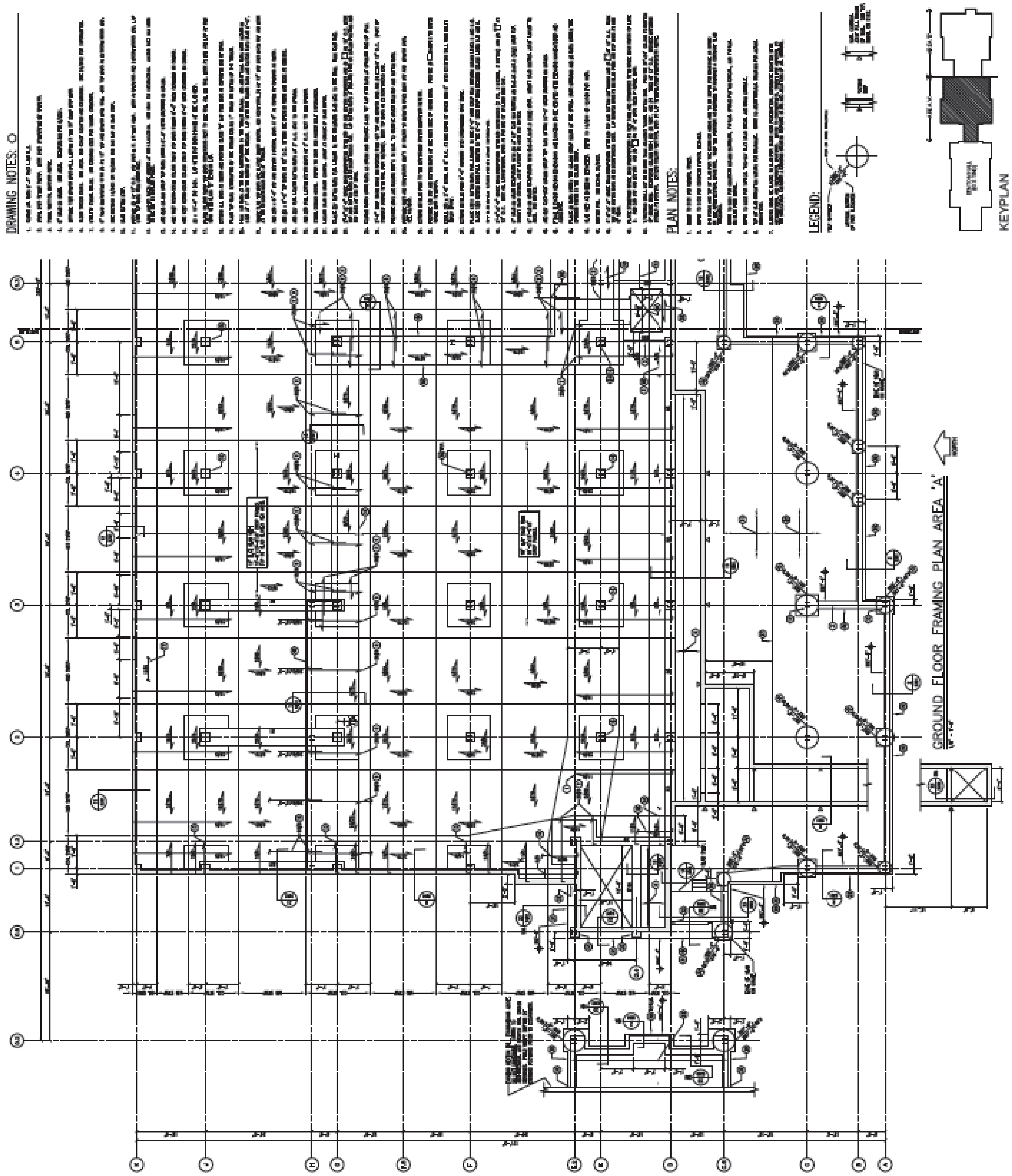


GRID LOCATION	ELEVATION	"L" LENGTH OF FIELD WELD
C-3	927'-1½	14"
C-3	941'-9½	8"
E-5	927'-1½	10"
E-5	941'-9½	8"
E-6	927'-1½	10"
E-6	941'-9½	8"
C-8	927'-1½	14"
C-8	941'-9½	8"

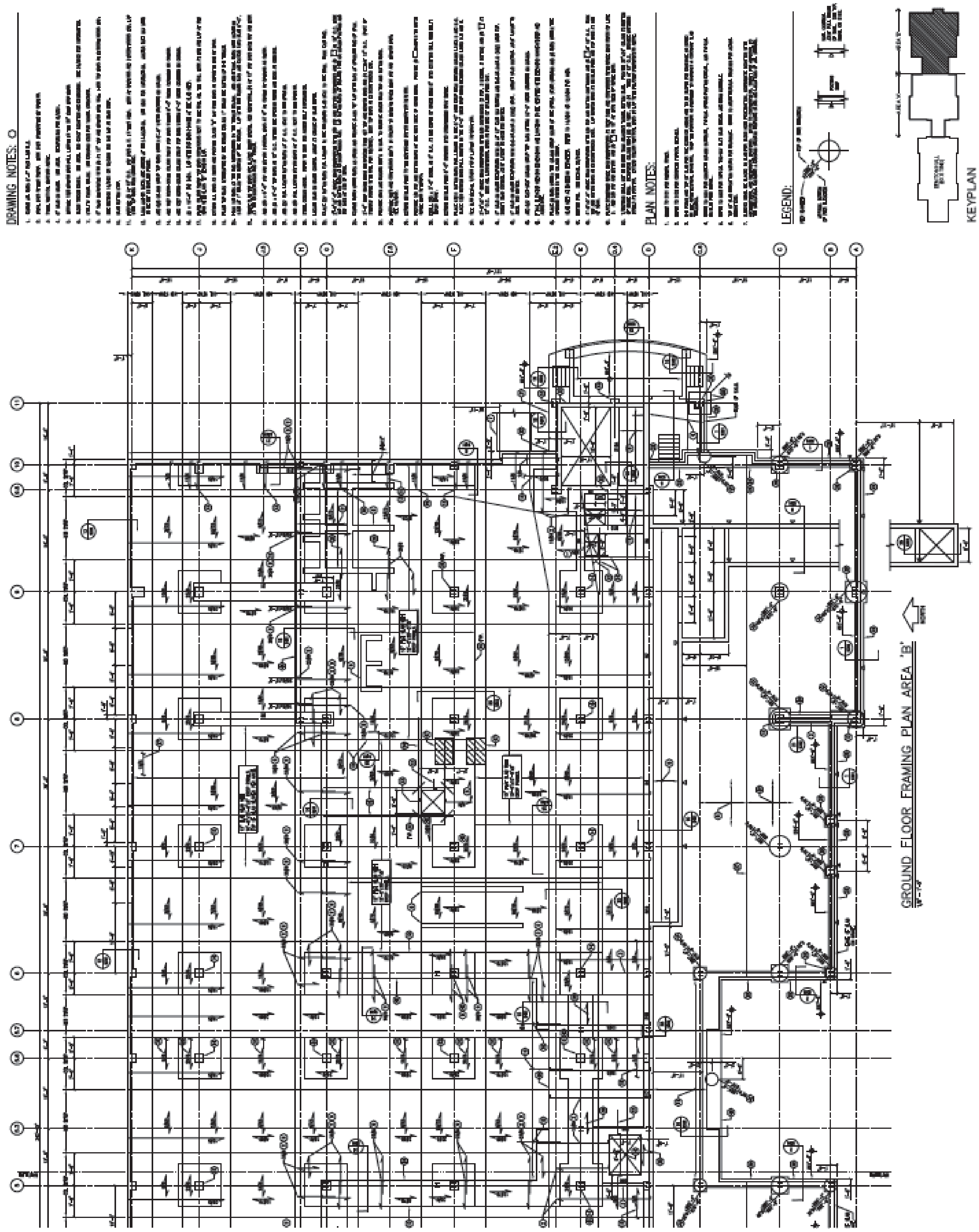


GRID LOCATION	ELEVATION	"L" LENGTH OF FIELD WELD
A-3	912'-5½	14"
A-3	927'-1½	8"
D-5	912'-5½	10"
D-5	927'-1½	8"
D-6	912'-5½	10"
D-6	927'-1½	8"
A-8	912'-5½	14"
A-8	927'-1½	8"

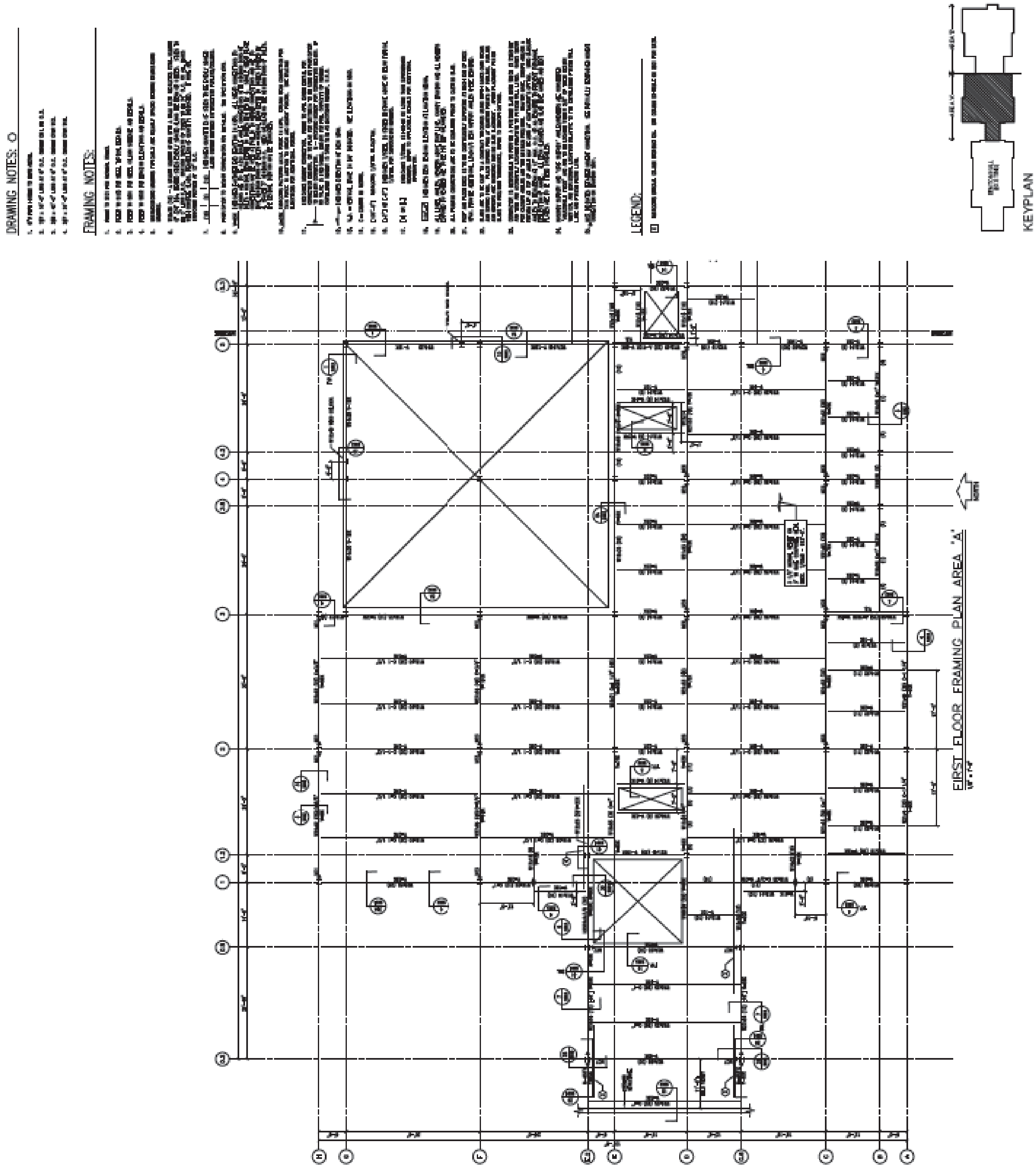
Ground Floor Framing Plan – Area 'A' (West half of building)



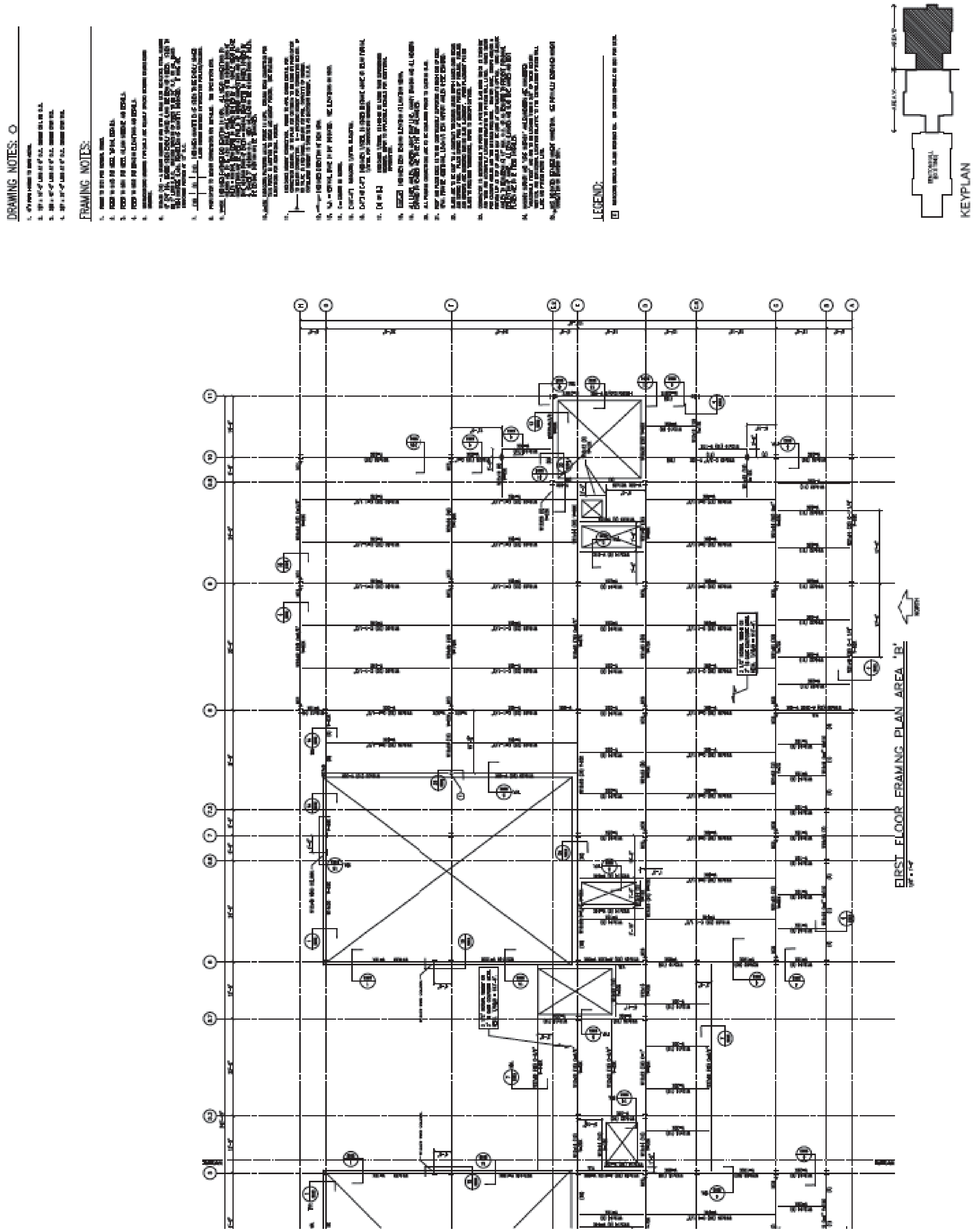
Ground Floor Framing Plan – Area 'B' (East half of building)



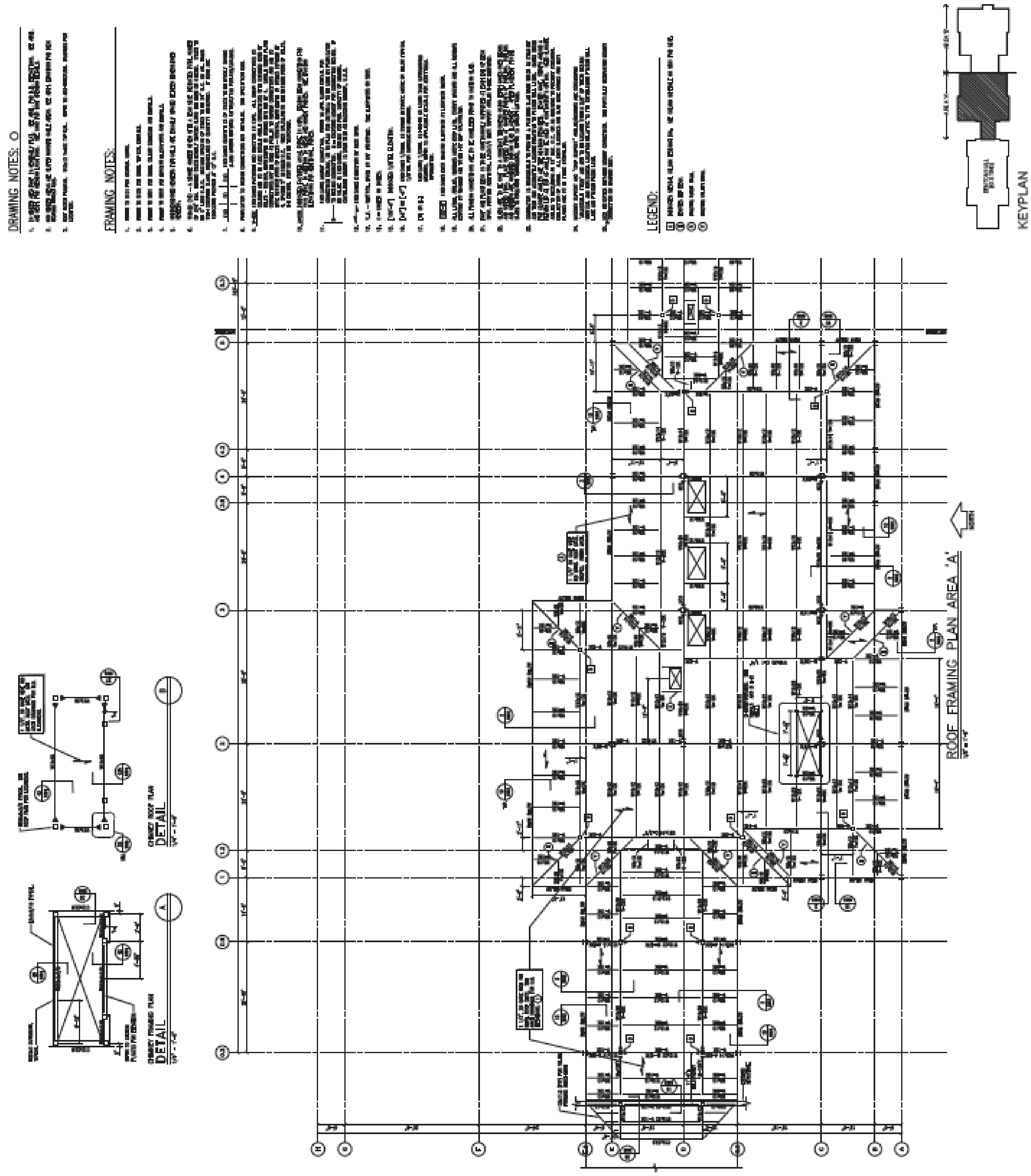
First Floor Framing Plan – Area 'A' (West half of building)



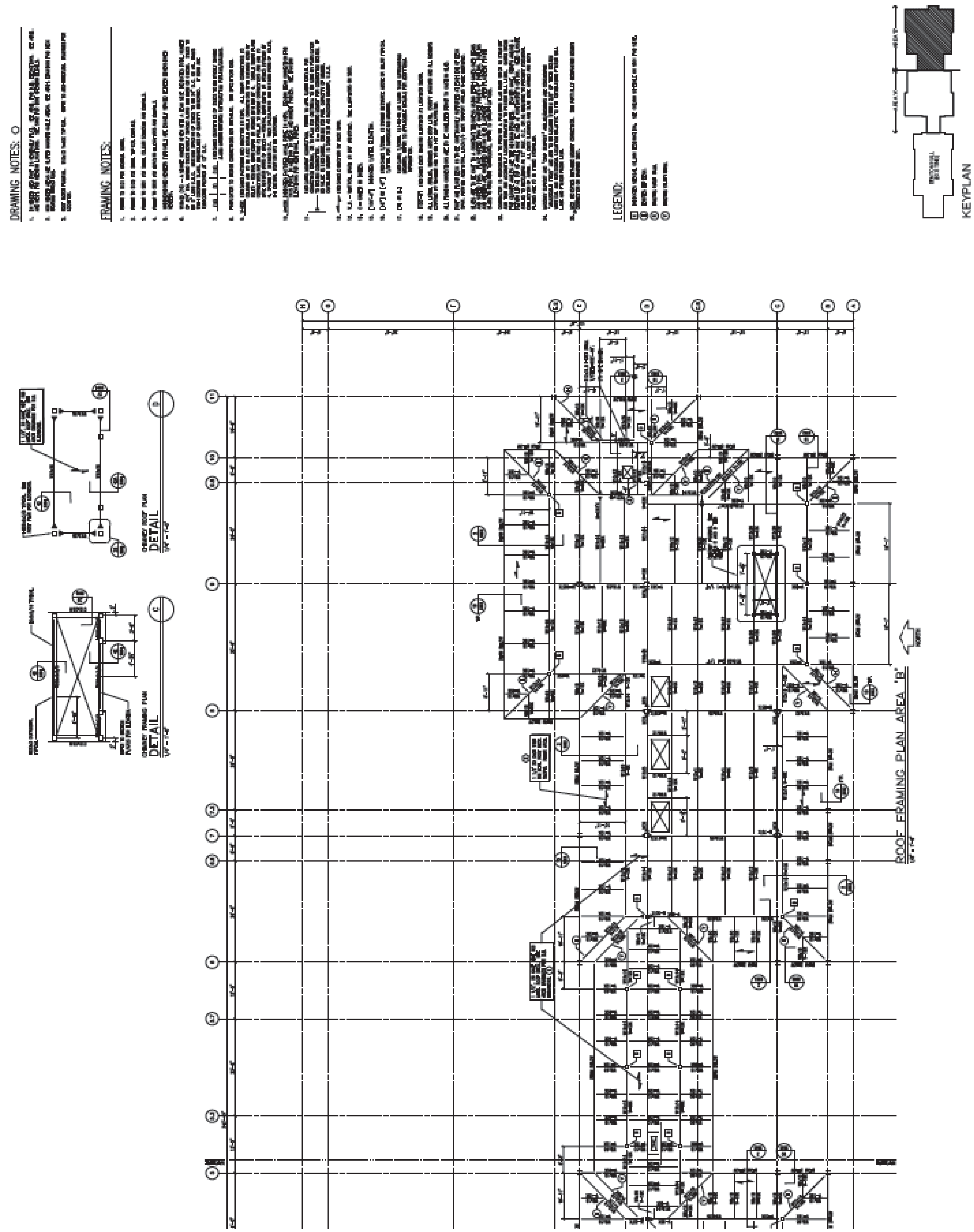
First Floor Framing Plan – Area 'B' (East half of building)



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



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

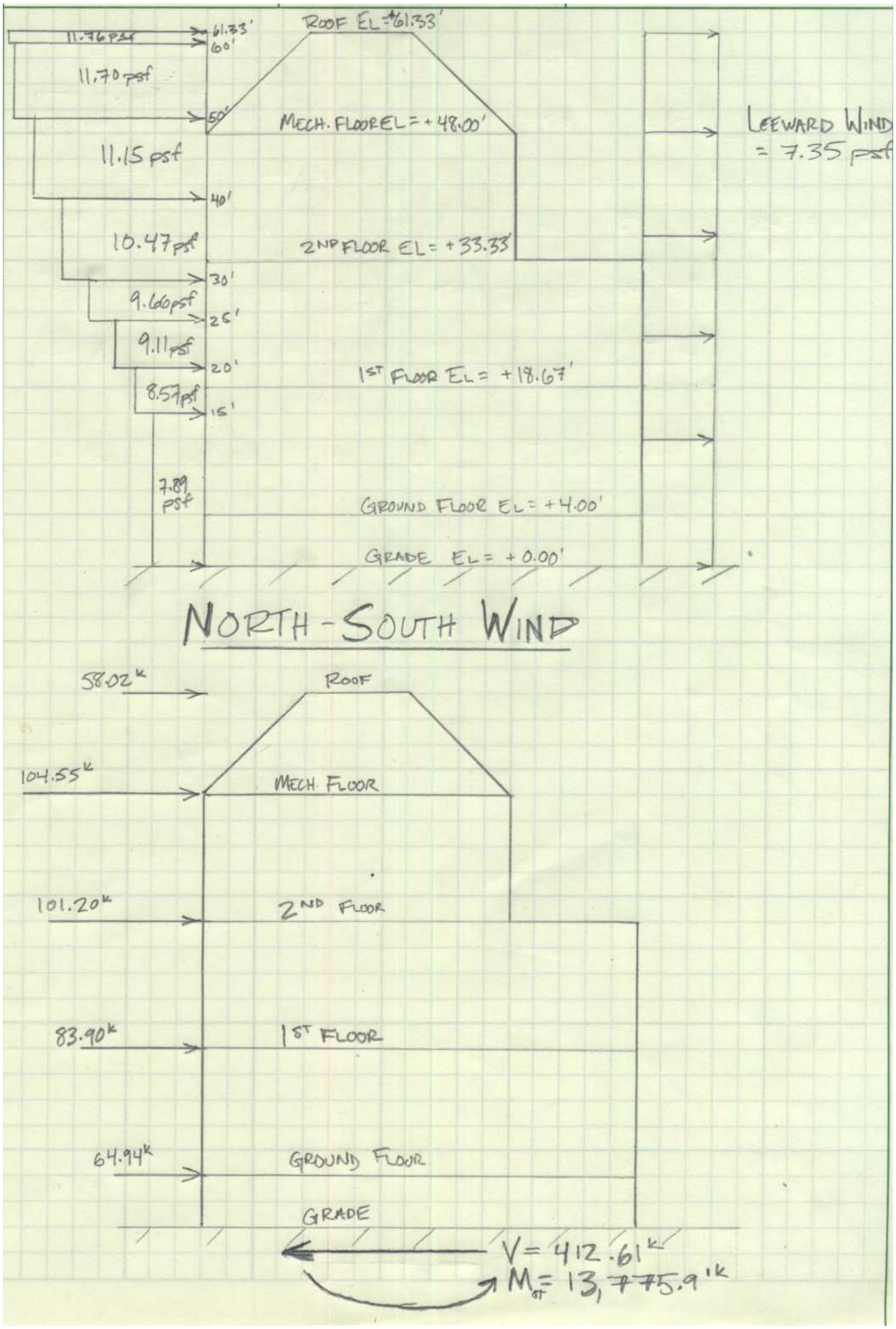


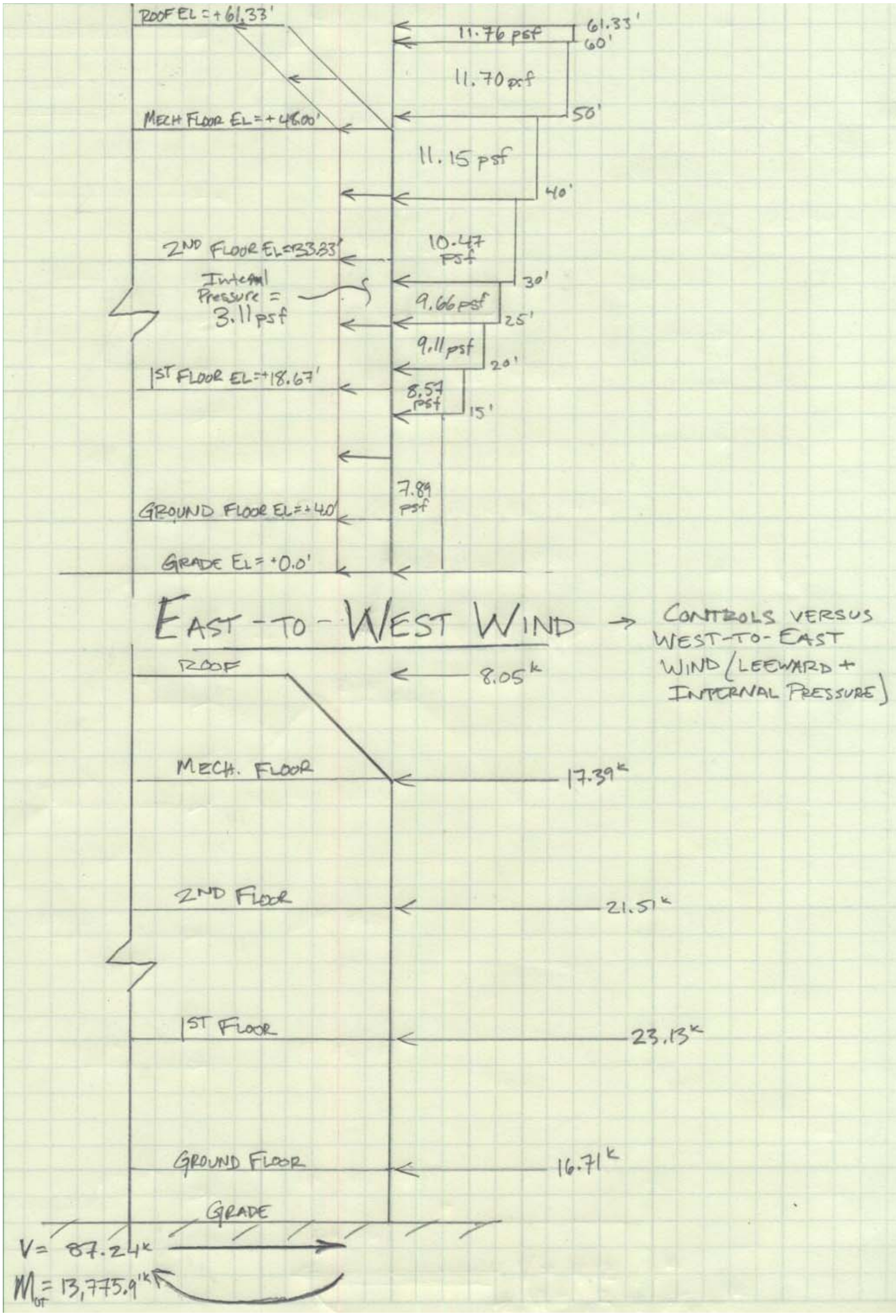
Appendix B – Wind Analysis

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'
ROOF EL = +61.33'
11.70 psf
50'
MECH. FLOOR EL = +48.00'
LEEWARD WIND
= 7.35 psf
11.15 psf
40'
10.47 psf
2ND FLOOR EL = +33.33'
30'
9.66 psf
25'
9.11 psf
20'
1ST FLOOR EL = +18.67'
8.57 psf
15'
7.89
psf
GROUND FLOOR EL = +4.00'
GRADE EL = +0.00'
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_OT = 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

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
R	3.0
C_d	3.0
h_n	61.33
C_u	1.6234
C_t	0.02
x	0.75
$T_a = C_t h_n^x =$	0.438 s
$T_{max} = C_u T_a =$	0.712 s
T_L	12 s

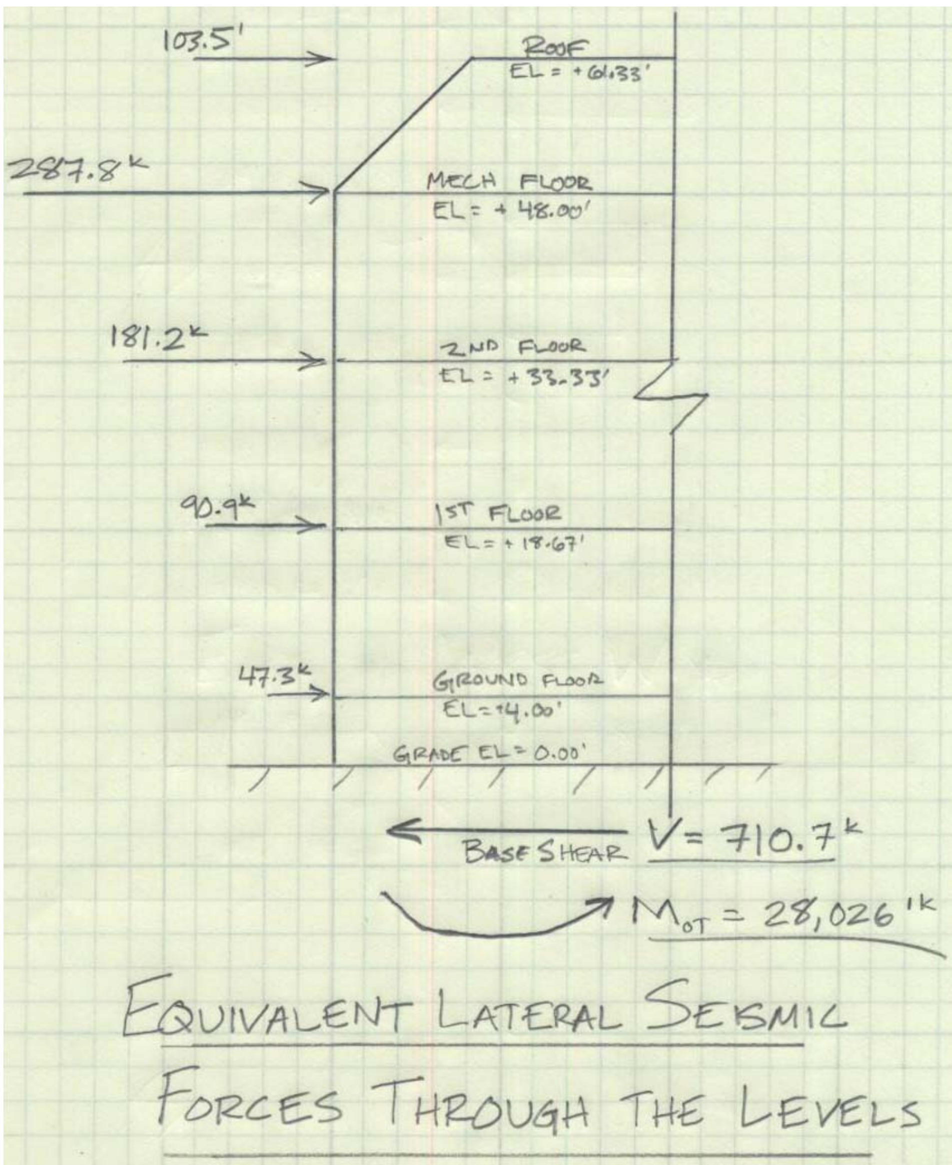
$$C_s = \min \left[\begin{array}{ll} S_{DS}/(R/I) = & 0.0570 \\ S_{D1}/(T(R/I)) = & 0.0484 \\ S_{D1}T_L/(T^2(R/I)) = & 0.8170 \end{array} \right] \geq 0.01$$

Controlling $C_s =$ **0.0484**

$W =$ 14669 k

$V = C_s W =$ **710.7 k**

Lateral Seismic Force Distribution Through the Levels								
Level	Story Height h_x	Story Weight w	Exponent k	$\Sigma\ w_i\ h_i^k$	C_{vx}	Story Force f_x	Shear V_x	Moment M_x
Roof	61.33 ft	707 k	1.1058	67018	0.1456	103.5 k	103.5 k	6345 ft-k
Mech.	48.00 ft	2579 k	1.1058	186437	0.4050	287.8 k	391.3 k	13816 ft-k
2nd	33.00 ft	2457 k	1.1058	117367	0.2550	181.2 k	572.5 k	5979 ft-k
1st	18.67 ft	2314 k	1.1058	58880	0.1279	90.9 k	663.4 k	1697 ft-k
Ground	4.00 ft	6612 k	1.1058	30625	0.0665	47.3 k	710.7 k	189 ft-k
Sum		W = 14669 k		460327		**V = 710.7 k**		**M = 28026 ft-k**



Appendix D – Load Calculations

Load Calculations

- Dead Loads
  - Upper Floors
  - Typ. composite slab

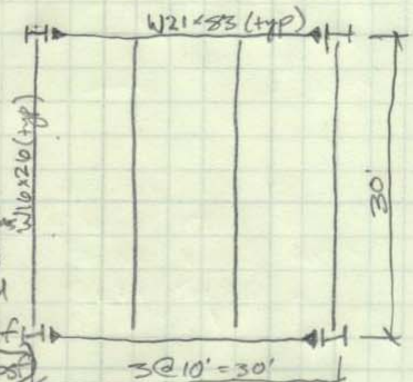
Equiv slab thickness = $3\tfrac{1}{2}" + 0.5(3") = 5"$

$w = 5"(1/12"/') (150\ pcf) = 62.5\ psf$
$+\ 2.5\ psf$ deck weight
$\boxed{65\ psf}$

- Typ Bay Steel Framing Weight

$W = \dfrac{(3\ bms)(30')(26\ plf) + (1\ grdr)(30')(83\ plf)}{(30')^2}$

$W = 5.4\ psf \Rightarrow$ use $\boxed{6\ psf}$ to account for shear studs & connections



Columns: Col. sizes vary from W12×40 to W12×136
Most typ. is W12×79, so that will be used to find appx. distr. load $W = \dfrac{(79\ plf)(14.67')}{(30')^2} = 1.3\ psf$
use W = 2 psf

- Ceiling & Mechanical Allowance (From Str. Gen. Notes)

$w = \boxed{15\ psf}$ on typ. floor

$W = \boxed{25\ psf}$ on mechanical equipment floor

$W = \boxed{10\ psf}$ on roof

- Wall & Partition Allowance
  - I am electing to use $\boxed{W = 10\ psf}$ for walls, which is lower than some engineers use because I feel ceiling & mechanical allowance is on the conservative side

- Roof Loads

1½" 20G. Deck = 2 psf
4" Rigid Insul. = 6 psf
½" Gypsum Board = 2 psf
Rubber Membrane = 1 psf

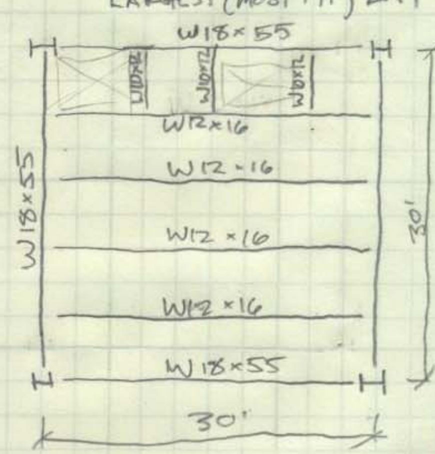
$\boxed{W = 11\ psf}$

- Steel Framing

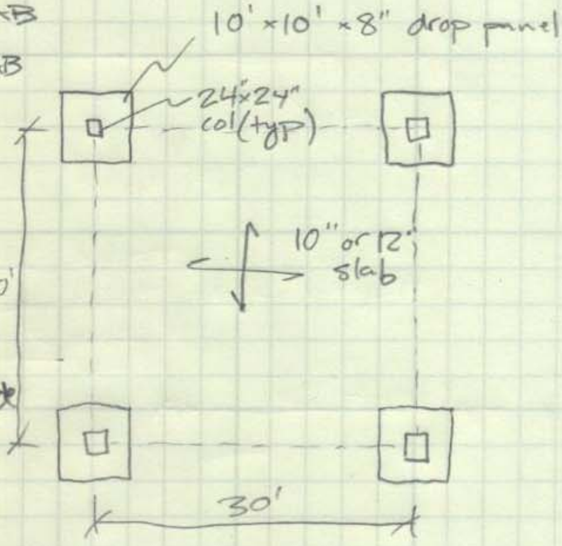
$W = \dfrac{2(55\ plf)(30') + 4(16\ plf)(30') + 3(12\ plf)(5')}{(30')(30')}$

$\boxed{W = 6.0\ psf}$

Largest (Most Typ) Bay



- GROUND FLOOR
  - TWO-WAY REINF. CONC. SLAB
  - APPX. 1/2 OF FLOOR IS 12" SLAB AND HALF IS 10" SLAB
    - FOR TOTAL FLOOR WEIGHT, EQUATE TO 11" SLAB



$W_{slab} = \left(\frac{11''}{12''/'}\right)(150\,pcf) = 137.5\,psf$

$W_{droppanel} = \frac{(10')(10')\left(\frac{8''}{12''/'}\right)(150\,pcf)}{(30')^2}$

$= 11.11\,psf$

$W_{col} = \frac{(2')(2')(150\,pcf)(4')}{(30')^2}$ eff height above grade

$= 2.67\,psf$

$W = 137.5 + 11.11 + 2.67 = 151.27\,psf$

use $\boxed{W = 152\,psf}$

DEAD LOAD SUMMARY

- GROUND FLOOR $\boxed{W_{DL} = 173\,psf}$

  $W = \underset{\text{conc floor + cols}}{152} + \underset{\text{mech. + ceiling allow.}}{15} + \underset{\text{steel cols}}{\tfrac{1}{2}(2)} + \underset{\text{partitions}}{\tfrac{1}{2}(10)} = 173$

- 1st, 2nd FLOORS $\boxed{W_{DL} = 98\,psf}$

  $W = \underset{\text{comp. slab}}{65} + \underset{\text{steel framing}}{6} + \underset{\text{col's}}{2} + \underset{\text{mech \& ceiling allow.}}{15} + \underset{\text{partitions}}{10} = 98$

- MECH. FLOOR $\boxed{W_{DL} = 108\,psf}$

  $W = 65 + 6 + 2 + \underset{\text{mech \& ceiling allow.}}{25} + 10 = 108$

- ROOF $\boxed{W_{DL} = 33\,psf}$

  $W = \underset{\text{roofing}}{11} + \underset{\text{steel framing}}{6} + \underset{\text{cols}}{\tfrac{1}{2}(2)} + \underset{\text{mech \& ceiling allow.}}{10} + \underset{\text{partitions}}{\tfrac{1}{2}(10)} = 33$

FLOOR & BUILDING DEAD LOADS

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

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

2ND FLOOR = $(\underset{\text{FLOOR}}{21{,}197}\text{ SF})(98\text{ PSF}) + (\underset{\text{ROOF}}{11{,}520}\text{ SF})(33\text{ PSF})$ = 2457 kips

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

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

TOTAL BUILDING WEIGHT (W) = $\boxed{14{,}669\text{ kips}}$

Snow Load and Drift Determination for Northern Wind on Low Roof

P_g	=	20.0 psf	Ground Snow Load	$l_{u\,(Lower)}$	=	54.00 ft	Use 25 if l_u is < 25 ft
C_e	=	1	Snow Exposure Factor	h_r	=	18.00 ft	Roof Projection
I	=	1.1	Importance Factor	h_c	=	$h_r - h_b$	
C_t	=	1	Thermal Factor	h_c	=	16.67 ft	
P_f	=	15.4 psf	Flat Roof Snow Load				
$P_{f(min)}$	=	22.0 psf	(Sect 7.3.4)				
Use P_f	=	**22.0 psf**					
γ	=	$13P_g + 14$					
γ	=	16.60 pcf	< 30 pcf				
use γ	=	**16.60 pcf**					
h_b	=	P_f / γ					
h_b	=	1.33 ft					
h_c / h_b	=	12.58	> 0.2				

Drift loads are required

Only Windward Drift Is Considered

h_d = $0.75*((0.43)(L_{u(Lower)})^{1/3}(P_g+10)^{1/4} - 1.5)$ For Roof Projection Condition

Use h_d = 1.73 ft

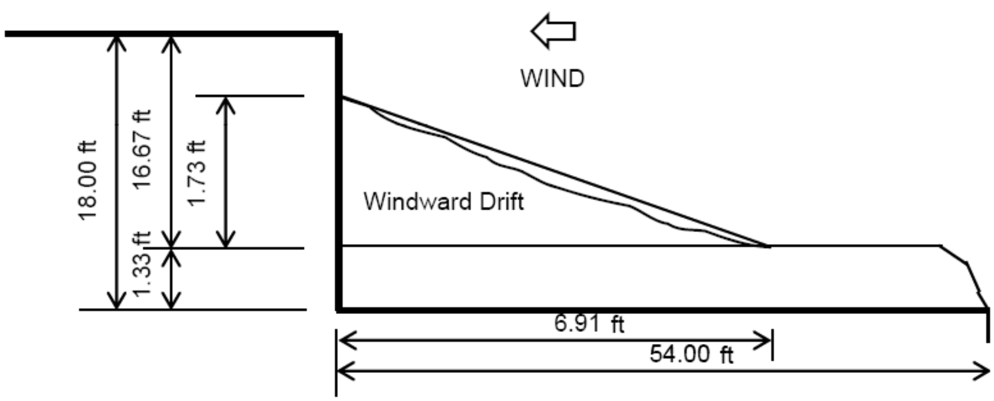
w = 4hd

w = 6.91 ft $\leq 8h_c$ = 133.4 ft

Use w = 6.91 ft

P_{max} = $\gamma\,(h_b + h_d)$

P_{max} = 50.7 psf This value is the **sum** of base snow intensity and drifted snow intensity



Snow Load and Drift Determination for Southern Wind on Low Roof

P_g	=	20.0 psf	Ground Snow Load	$l_{u\,(Upper)}$	=	71.00 ft	Use 25 if l_u is < 25 ft
C_e	=	1	Snow Exposure Factor	$l_{u\,(Lower)}$	=	48.00 ft	Use 25 if l_u is < 25 ft
I	=	1.1	Importance Factor	h_r	=	30.00 ft	Roof elevation differen
C_t	=	1	Thermal Factor	h_c	=	$h_r - h_b$	
P_f	=	15.4 psf	Flat Roof Snow Load	h_c	=	28.67 ft	
$P_{f(min)}$	=	22.0 psf	(Sect 7.3.4)				
Use P_f	=	**22.0 psf**					

γ = $13P_g + 14$

γ = 16.60 pcf < 30 pcf

use γ = 16.60 pcf

h_b = P_f / γ

h_b = 1.33 ft

h_c / h_b = 21.64 > 0.2

Drift loads are required

Leeward Drift

$h_d = (0.43)(L_{u\,(Upper)})^{1/3}(P_g+10)^{1/4} - 1.5$ For High Roof Condition

h_d = 2.67 ft

Windward Drift

$h_d = 0.75*((0.43)(L_{u(Lower)})^{1/3}(P_g+10)^{1/4} - 1.5)$ For High Roof Condition

h_d = 1.62 ft

Use h_d = 2.67 ft

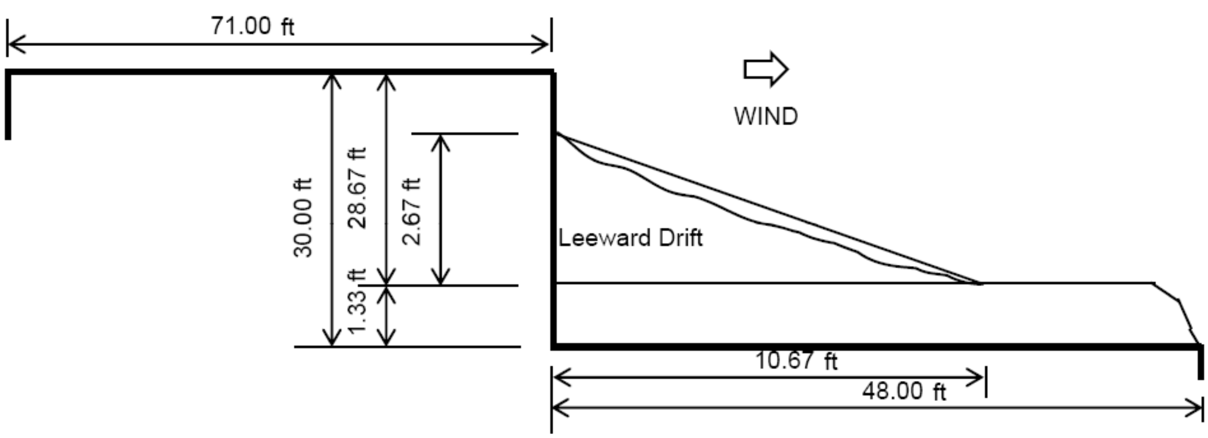
w = 4hd

w = 10.67 ft $\leq 8h_c$ = 229.4 ft

Use w = 10.67 ft

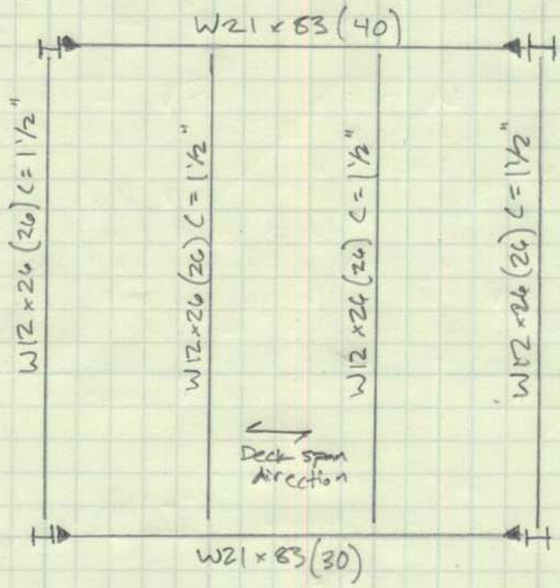
$P_{max} = \gamma\,(h_b + h_d)$

P_{max} = 66.3 psf This value is the **sum** of base snow intensity and drifted snow intensity



Appendix E – Spot Checks

SPOT CHECK OF COMPOSITE BEAM



NOTES:
(x) indicates num of 3/4" diam. shear studs
C = x" indicates beam camber

30' × 30' bay
3½" NWC CONC. SLAB ON 3" COMP. DECK
$f'_c = 4000\text{ psi}$

$A_s(W12\times26) = 7.56\text{ in}^2$

LOAD ON BEAM:

DEAD: (62.5 psf)(10') = 625 plf
Self-wt = 26 plf
Mech allowance = (15 psf)(10') = 150 plf
Partition allowance (10 psf)(10') = 100 plf
w_{DL} = 901 plf

LIVE: (100 psf)(10') = w_{LL} = 1000 plf

$$w_u = 1.2(901) + 1.6(1000) = 2681\text{ plf} = \boxed{2.681\text{ klf}}$$

$$M_u = \frac{w_u L^2}{8} = \frac{(2.681)(30')^2}{8} = \boxed{301.6\text{ ft}\cdot\text{k}}$$

$$V_u = \frac{w_u L}{2} = \frac{(2.681)(30')}{2} = \boxed{40.22^k}$$

$$\Sigma Q_n = A_s f_y = (7.56\text{ in}^2)(50\text{ ksi}) = 378^k$$

$$b_{eff} = \min\begin{cases} 1/4\text{ span} \\ \text{spacing}\end{cases} = \min\begin{cases} 1/4(30') = 7.5' \leftarrow \text{controls} \\ 10' \end{cases}$$

$$a_{req} = \frac{\Sigma Q_n}{0.85 b_{eff} f'_c} = \frac{378}{0.85(7.5\times12)(4\text{ ksi})} = 1.235"$$

$$Y2 = 6.5" - \frac{1.235}{2} = 5.88"$$

TFL → $\Sigma Q_n = 382^k > 378^k$ ok✓

$\phi M_n(Y2 = 5.88") = 342^{'k}$ (from AISC Steel Manual Table 3-19)

$342^{'k} > 301.6^{'k}$ ∴ ok✓

SPOT CHECK OF COMPOSITE BEAM (cont.)

Check deflection

$I_{LB} = 738.6 \text{ in}^4$ (from AISC Table 3-20)
$(Y2 = 5.88")$

$$\Delta_{total} = \frac{5wL^4}{384EI} = \frac{5(2.681)(30')^4}{384(29,000)(738.6)}(1728) = 2.28"$$

After 1½" camber $\Delta = 2.28" - 1.5" = .78"$

$L/240 = 1.5" > 0.78"$ ok ✓

$$\Delta_{LL} = \frac{5(1.6 \times 1 \text{ klf})(30')^4}{384(29000)(738.6)}(1728) = 1.36"$$

After 1½" camber $\Delta = 1.36" - 1.5" = -0.14"$ ok ✓

SPOT CHECK OF INTERIOR COLUMN

COLUMN D-8 FROM GROUND FLOOR TO ROOF

W12×58 FROM MECH. FLOOR TO ROOF, SPLICED WITH MOMENT SPLICE @ MECH FLOOR

W12×106 FROM GROUND FLOOR TO MECH. FLOOR.

LOAD ON W12×58

$P_U = 960\text{ SF}\left(1.2(33\text{ PSF}) + 1.6(25\text{ PSF})\right) = 54.92^k$

$KL = 1.2(13') = 15.6' \Longrightarrow$ use $KL = 16'$

$\phi P_n = 500^k >> 54.92^k$

LOAD ON W12×106 $\Longrightarrow$ See attached spreadsheet

$P_U = 458.6^k$

$KL = 1.2(14.67') = 17.6' \Rightarrow$ use $K = 18'$

$\phi P_n = 985^k >> P_u = 458.6^k$

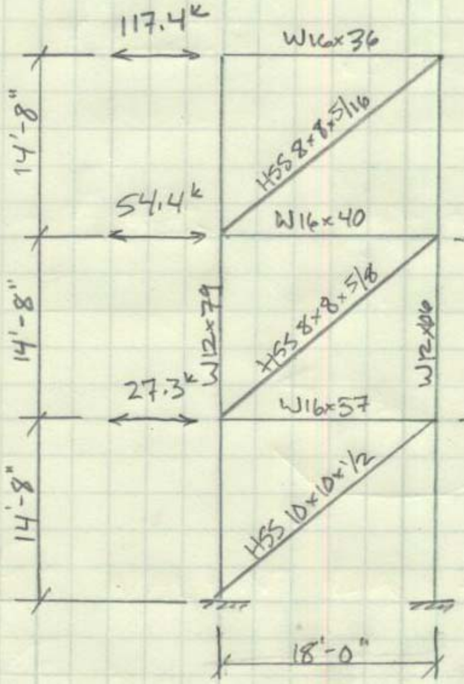
This column is used as part of the moment frame in the E-W direction, and a lateral analysis in that direction would be required to find applied moments. An interaction equation would then be used to check the column's capacity. This explains why the column's capacity is much larger than needed under pure gravity loads. Also, the structural general notes show that live load was not reduced on typical and mechanical floors, but I reduced the live load on the column per ASCE 7-05 Section 4.8.

Load on Column D-8 (Interior column)							
Floor	Tributary Area (sf)	Dead Load (psf)	Live Load, L_o (psf)	K_{LL}	Reduction Factor	Reduced Live Load (psf)	Factored Load 1.2D+1.6L (k)
Roof	690	33	25	4	0.536	13.39	42.10
Mech.	690	108	150	4	0.452	67.78	164.26
2nd	690	98	100	4	0.415	41.48	126.94
1st	690	98	100	4	0.400	40.00	125.30

Total Factored Load = $\mathbf{P_u}$ = **458.61 k**

CALCULATION OF LATERAL FORCES
FOR SPOT CHECK OF A VERTICAL BRACE

- Of the 4 V.B.'s in the North-South lateral resisting system, there are 2 identical pairs. Without doing a rigidity analysis and calculating the distribution of lateral load to each V.B., I am basing my distribution on the distribution of factored axial loads shown on bracing members in the structural drawings. Approximately 60% of the lateral goes to the 2 larger braced frames based on this very simplified analysis. To analyze one of these vertical braces, I will place 30% of my seismic lateral forces at each level. A diagram can be seen below:



shear from roof above

- Mech Floor $V = 0.30(287.8 + 103.5) = 117.4^k$

- 2nd Floor $V = 0.30(181.2) = 54.4^k$

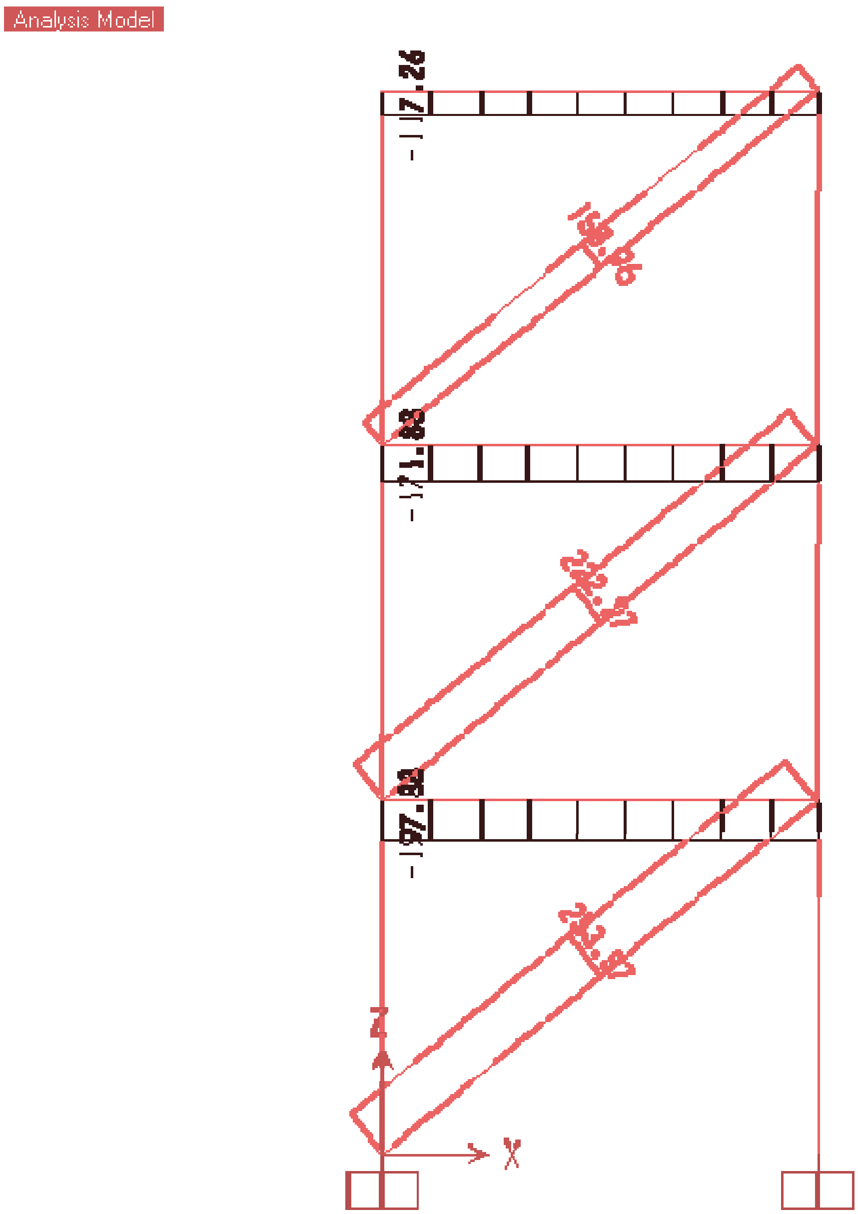
- 1st Floor $V = 0.30(90.9) = 27.3^k$

Note: columns bending in weak axis direction

Results from a SAP analysis are shown on the following pages. Beams were shown to fail in compression, because SAP doesn't know that the beams are fully braced from the composite slab, which would make the members passed.

SAP 2000 Analysis of Vertical Brace

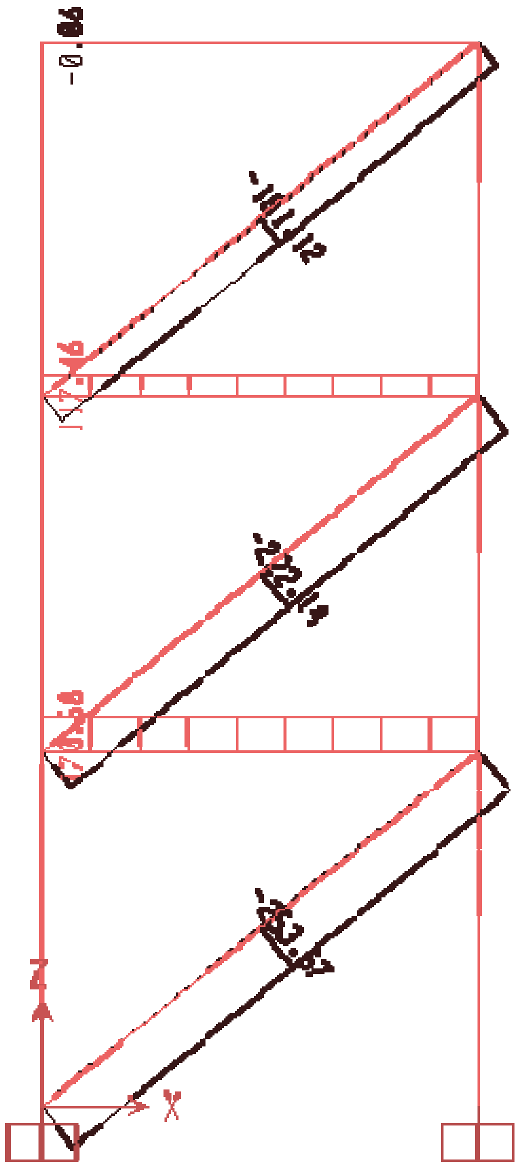
This SAP output used the assumptions of pin connections at all joints and moment connections at the base. This diagram ran a load case where seismic forces were applied from the South (left in diagram). Axial forces in members are displayed.



SAP 2000 Analysis of Vertical Brace

This SAP output used the assumptions of pin connections at all joints and moment connections at the base. This diagram ran a load case where seismic forces were applied from the North (right in diagram). Axial forces in members are displayed.

Analysis Model



SAP 2000 Analysis of Vertical Brace

This image from the SAP analysis is from the check of the structure. The red members (beams) are shown to fail in compression. The most logical explanation that would cause this is that SAP is assuming these members to buckle since it thinks that the member is unbraced, where in reality, the member would be fully braced along its' length by the composite concrete slab. By inspection, we can tell that these members are much larger than need for compression yielding since the axial forces are relatively small compared to the area of steel each member has to yield.