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Structural Option

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URS Office Building
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STRUCTURAL TECHNICAL REPORT 1

Structural Concepts / Structural Existing Conditions Report

Introduction

This report provides an overview of the structural system which support the URS Office Building located in Columbus, Ohio. The 5 story, 100,000 square foot building is the forerunner in design for the Arena District being developed by Nationwide Realty Investors. The curvature and the setback on the North facade of the building (facing Nationwide Boulevard) along with careful consideration for proportion gives distinction to the otherwise rectangular building. Designed as mercantile/office building, the URS Office Building provides retail area on the first floor and office area from second to fifth floor. Completed construction in January 2001, this design, bid, build project's total cost was $7 million.

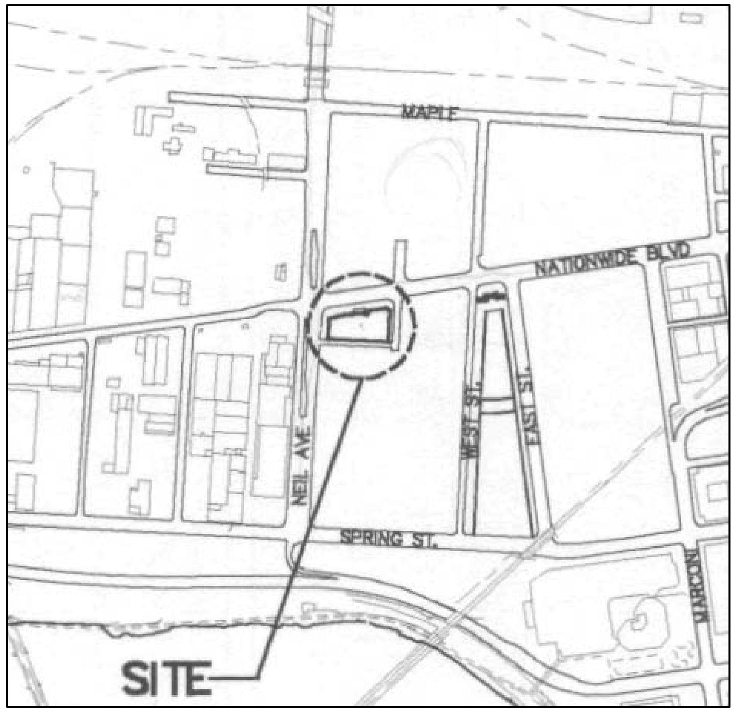


Figure 1

In the following sections are structural system description, codes applied, frame plan and elevation, design loads, and spot checks.

Structural System

URS Office Building is a steel frame structure surrounded by brick masonry veneer along with large punched windows which incorporate industrial glass. Structural steel used for beams, columns, and girders is ASTM A572 grade 50 wide-flange with yield strength of 50,000psi. Longest beam spans 33'4" and longest girder spans 32'. Typical bays are 30'x 30' with most bays being approximately a square. The chevron bracings resisting lateral loads are ASTM A500, Grade B tube steel with yield strength of 46,000psi.

Spread footings with minimum compressive strength at 28 days of 3000psi together with grade beams are employed as the foundation system. The size of footing varies from 4'x 4' to 14'x 14'. The grade beams also vary in width as well as depth. Both the spread footings and grade beams utilize bars #6, #7, #8, or #9 with #4 stirrups. The slab on grade is required minimum compressive strength at 28 days of 4000psi and the composite slabs are to be lightweight concrete with minimum strength of 3000psi. First floor slab on grade is 5” concrete slab. The composite slabs on floors 2 through 5 are composed of 2” steel deck and 3-1/4” light weight concrete.

Steel roof decks are galvanized 20 gage ASTM A653 grade 33 G90 zinc coated steel. The composite steel floor decks are galvanized 20 gage ASTM A653 grade 33 G60 steel. Headed studs ¾”ϕ x 4” spaced evenly across the steel members are used to achieve composite action.

CODES

For the URS Office Building structural design was performed under Ohio Basic Building Code 1998 (OBBC). OBBC was created by adopting BOCA National Building Code 1993. Structural standards for structural steel, cast in place concrete, pre-cast concrete, metal deck, and masonry are shown below in *Table 1*.

OHIO BASIC BUILDING CODE 1998	
STRUCTURAL STEEL	• AISC Manual of Steel Construction (ASD 91)
CAST IN PLACE CONCRETE	• ACI 318-95 – “Building Code Requirement for Reinforced Concrete” • ACI 301-89 – “Specification for Structural Concrete for Buildings”
PRECAST CONCRETE	• ACI 318 – “Building Code Requirement for Reinforced Concrete” • PCI MNL 120 – “PCI Design Handbook Pre-cast and Pre-stressed Concrete”
METAL DECK	• AISI – “Specification for the Design of Cold-formed Steel Structural Members” • SDI – “Design Manual for Composite Decks, Form Deck, and Roof Decks”
MASONRY	• ACI 530.1/ASCE 6/TMS 602

Table 1

In January 1 of 2002, State of Ohio adopted the 2000 International Building Code (IBC). Current building code in Ohio is the 2005 Ohio Building Code (OBC) based on 2003 IBC. Therefore throughout the report, 2003 IBC along with ASCE 7-05 will be used as the structural standard. Although calculations were performed using ASD 9th Edition of the steel manual, this report will be determined by LRFD 3rd Edition of the steel manual.

FRAME PLAN AND ELEVATION

Below in *Figure 2* is the largest bay which is 32'x 33'4". Typical girders are W24 and typical beams are W16. Second to fifth floor has identical framing plan. Roof frame plan varies from floors below, but bay size and location are the unchanged.

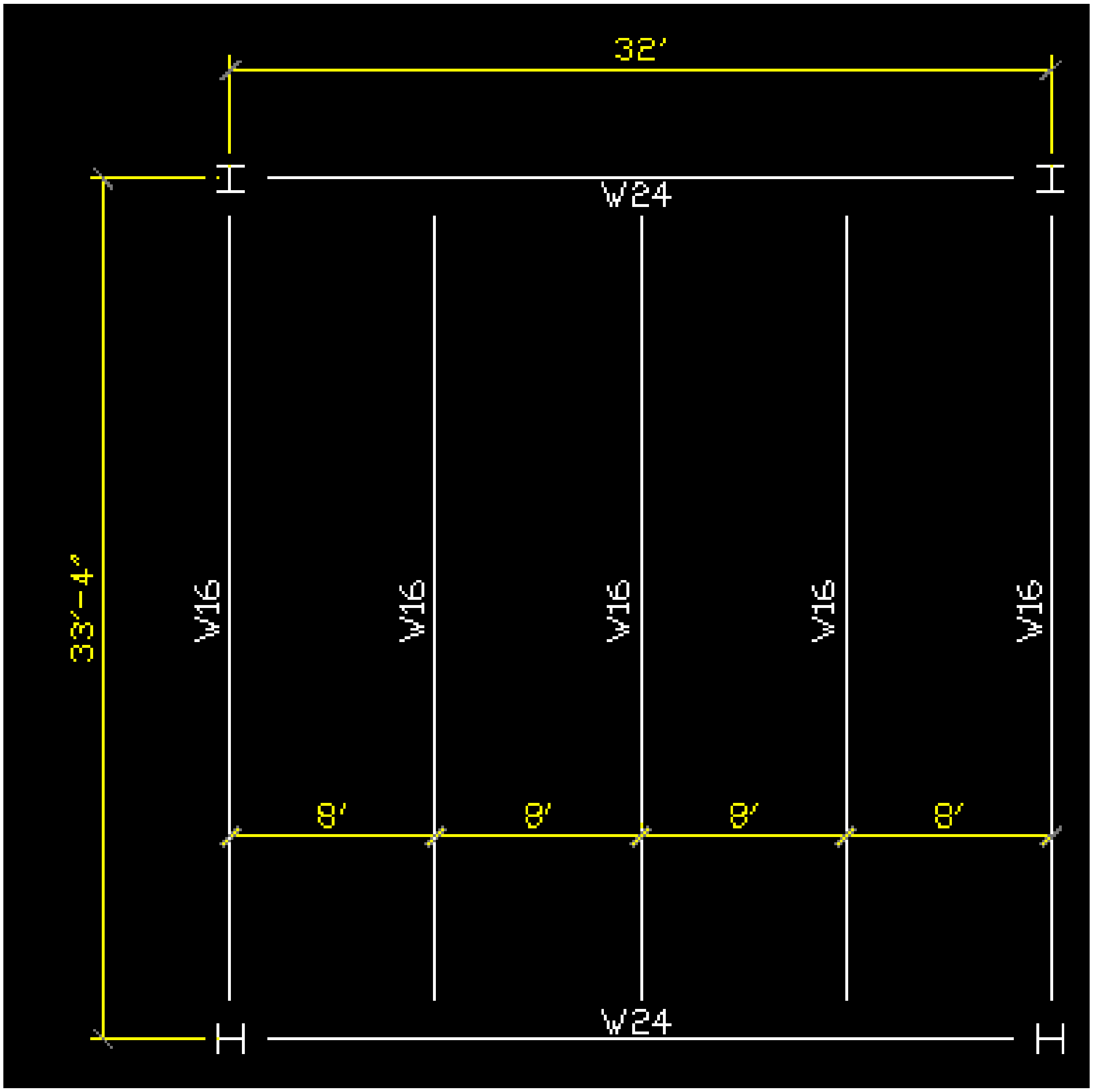


Figure 2

Three braced frames and moment frames along the perimeter exist to resist the lateral loads for URS Office Building (see *Figure 3a and 3c*). As shown in Figure 3b the chevron bracing is used for all three frames. The tube steel members which compose the chevron bracing have moment connections. Brace frame 1 (WB-1) resists east/west lateral loads and brace frames 2 and 3 (WB-2 and WB-3) resists north/south lateral loads.

Figure 3a

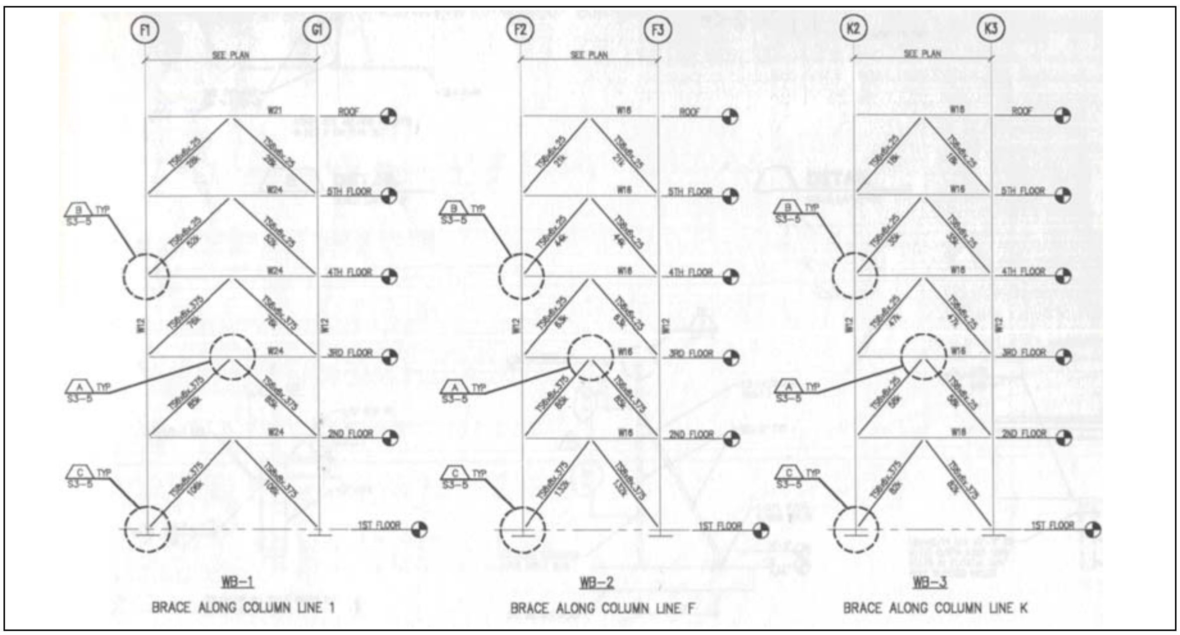


Figure 3b

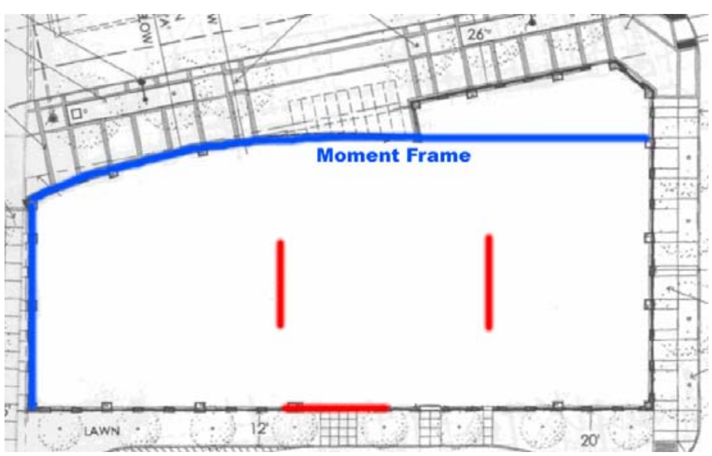


Figure 3c

LOADS

Loads are calculated by design parameters given in ASCE 7-05 in conjunction with 2003 IBC. Dead load will be calculated according to the actual weight of the permanent building components. Live load will be directly taken out of 2003 IBC. Snow, wind, seismic calculation will follow ASCE 7-05 procedures.

Gravity Load

Dead Loads (PSF) – actual weight of the permanent building components

- Structural Steel ------ 6.5 PSF
- Metal Deck ----------- 3 PSF
- Concrete -------------- 43 PSF
- MEP ------------------- 15 PSF
- Partition --------------- 20 PSF

- Total Dead Load ----- 87.5 PSF

Live Loads (PSF) – from 2003 IBC: Table 1607.1

- Roof Snow ----------- 25 PSF
- Office Floor ---------- 50 PSF
- Corridor --------------- 100 PSF
- Lobby ----------------- 100 PSF
- Retail ------------------ 100 PSF
- Penthouse Floor ------ 250 PSF
- Mechanical Unit ----- 150 PSF + weight of equipment

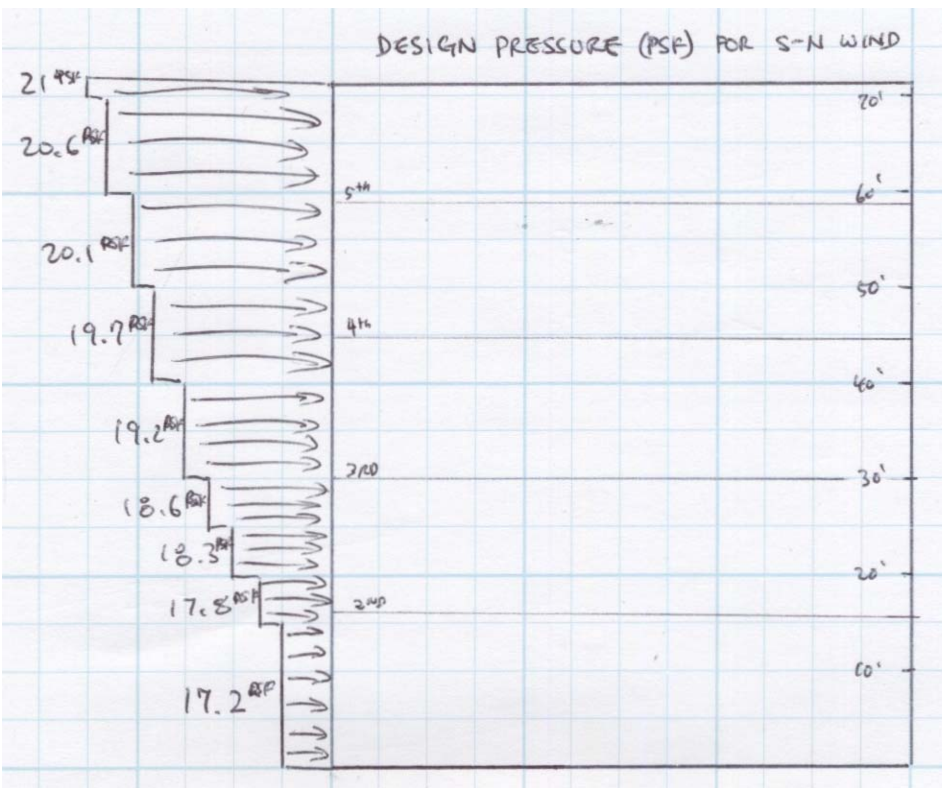
Snow Load (PSF) – from ASCE 7-05: Chapter 5

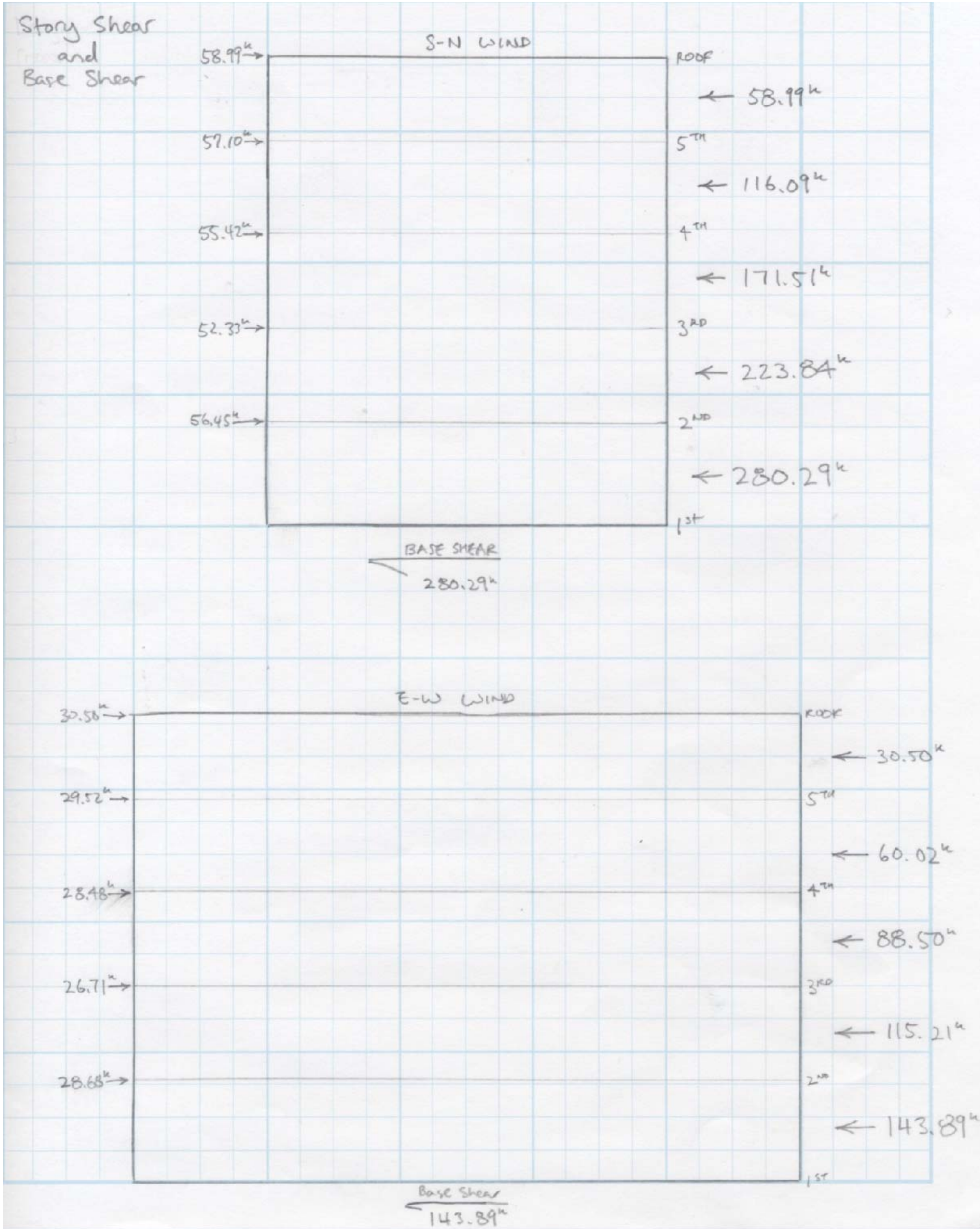
- Primary concern for snow load is next to the penthouse where drifting may occur. Following the guidelines in ASCE 7-05 snow load was calculated and drift was considered.
- The maximum drift calculated was 59.3 PSF

Lateral Load

Wind – from ASCE 7-05: Chapter 6

Height Above Ground (ft)	Windward plus Leeward Pressure
0-15	17.2 PSF
20	17.8 PSF
30	18.3 PSF
40	19.2 PSF
50	19.7 PSF
60	20.1 PSF
70	20.6 PSF
80	21.0 PSF
90	21.3 PSF




Story Shear and Base Shear
S-N WIND
58.99k
57.10k
55.42k
52.33k
56.45k
ROOF
5TH
4TH
3RD
2ND
1st
58.99k
116.09k
171.51k
223.84k
280.29k
BASE SHEAR
280.29k
E-W WIND
30.50k
29.52k
28.48k
26.71k
28.68k
ROOF
5TH
4TH
3RD
2ND
1ST
30.50k
60.02k
88.50k
115.21k
143.89k
Base Shear
143.89k

Seismic – from ASCE 7-05: Chapter 11

Seismic

	D+SID	AREA	W
2ND	87.5 PSF	20290 SF	1775^k
3RD	↓	↓	↓
4TH			
5TH			
ROOF	45 PSF		913^k
			$\Sigma = 8013$

$S_S = 12$

$S_1 = 5$

$S_{MS} = F_A S_S = 1.6(12) = 19.2$

$S_{M1} = F_V S_1 = 2.4(5) = 12$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(19.2) = 12.8$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(12) = 8$

Occupancy Category II → $I = 1.0$

SEISMIC DESIGN CATEGORY B

R → brace frames = 3.25

$T_a = 0.02(72)^{0.75} = 0.494$

$T = 1.7(0.494) = 0.84$

$T_L = 12$

$T \leq T_L$

$C_s = S_{DS}/\left(\frac{R}{I}\right) = 12.8/\left(\frac{3.25}{1}\right) = 3.94$

$C_{s\,max} = \frac{S_{D1}}{TR/I} = \frac{8}{0.84(3.25)/1} = 2.93$

base shear ≡ $V = C_s W = 0.0293 \times 8013 = 234.7^k$

$F_x = C_{vx} \cdot V$

$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$

$k = 1.12$ interpolated

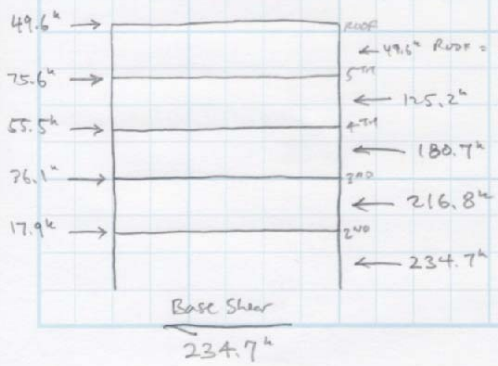
2ND $= \frac{1775(16)^{1.12}V}{} = \frac{39611V}{520095} = 0.076V = 17.9^k$

3RD $= \frac{1775(30)^{1.12}V}{} = \frac{80089V}{520095} = 0.154V = 36.1^k$

4TH $= \frac{1775(44)^{1.12}V}{} = \frac{122989V}{520095} = 0.236V = 55.5^k$

5TH $= \frac{1775(58)^{1.12}V}{} = \frac{167586V}{520095} = 0.322V = 75.6^k$

ROOF $= \frac{913(72)^{1.12}V}{} = \frac{109820V}{520095} = 0.211V = 49.6^k$



Spot Checks

Beams
To spot check the beams, a typical bay on the second floor was checked. RAM model showed the beams were adequate for the load provided in this report. Also hand calculation was performed which shows adequacy of the beam used.

Girders
A girder was taken from the same bay as the beam above. Again Ram model as well as the hand calculations agreed with the construction documents.

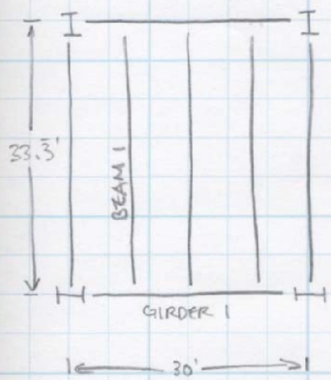
Columns
An interior column was taken from the first floor for spot check. The column listed on the column schedule was adequate in holding the loads provided in this report. RAM analysis was performed along with quick hand checks.

Bracing
Total factored load were taken into account for the bracing member calculations. Since dead, live, and wind load combination controlled in North-South lateral load, the loads were factored and applied to bracing two and three. Bracing one was checked against dead, live, and seismic combination loads which controlled East-West lateral load. Bracing members were assumed to take 80% of the lateral loads and moment frames were assumed to resist the rest. Bracing members along with moment frames were adequate to provide lateral resistance.

CALCULATIONS

Beam

LEVEL 2 → TYP. BAY



BEAMS SPACED EQUALLY

LL → 50 PSF
DL → 88 PSF

LL REDUCTION

$$L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right) = 50\left(0.25 + \frac{15}{\sqrt{500}}\right) = 46 \text{ PSF}$$

$A_T = 7.5' \times 33.\bar{3}' = 250$ SF

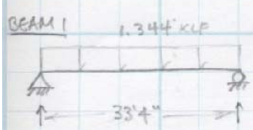
$A_I = 2A_T = 500$ SF

factored loads

$1.2D + 1.6L = 1.2(88) + 1.6(46) = 179.2$ PSF

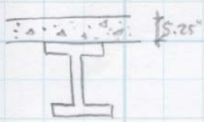
$P_u = 179.2$ PSF

$W_u = 179.2 \times 7.5 = 1344$ PSF $= 1.344$ KLF



$$M_u = \frac{wl^2}{8} = \frac{1.344(33.\bar{3})^2}{8} = 187'^k$$

try 16x26



assume $a = 1.5$

$Y_2 = 5.25" - 0.75" = 4.5"$

assume PNA @ 7

$\phi M_n = 228'^k$

$\Sigma Q_n = 96^k$

$$a = \frac{\Sigma Q_n}{0.85(f'_c)(b)} = \frac{96}{0.85(3)(90)} = 0.42 < 1.5 ✓$$

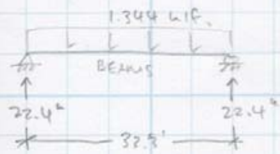
$b_{eff} = 7.5' \times 12 = 90"$

$\frac{33.3' \times 12}{4} = 100"$

W16x26 in composite action works

Girder

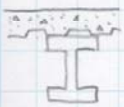
GIRDER 1



$P_1 = P_2 = P_3 = 44.8^k$

assume equal load from beams in next bay

$$M_u = \frac{P_2 \ell}{4} + \frac{P_1 bx}{\ell} + \frac{P_3 bx}{\ell} = \frac{44.8(30)}{4} + \frac{44.8(7.5)(15)}{30} + \frac{44.8(7.5)(15)}{30} = 672^{1k}$$



try 24x55
assume $a = 1.5$
$Y_2 = 4.5''$
assume PNA @ 7
$\phi M_n = 678^{1k}$

$\Sigma Q_n = 204^k$

$$a = \frac{204}{0.85(3)(90)} = 0.89 < 1 \checkmark$$

$b_{eff} = 33.3' \times 12 = 400''$
$= \frac{30' \times 12}{4} = 90''$

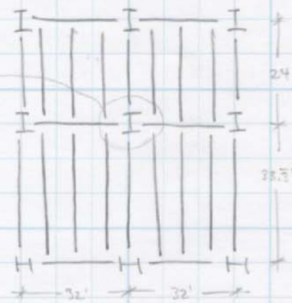
w 24x55 in composite action works

∴ From loads calculated composite beam and girders are adequate. The sizes chosen are also resonable.

Column

COLUMN DESIGN D-2

W12x72
KL = 16'
$\phi P_n = 670^k$



LL → 50 PSF
DL → 88 PSF

Roof LL → 25 PSF
Roof DL → 25 PSF (no conc, no partition

TRIB AREA = $32\left(\frac{24}{2} + \frac{33.\bar{3}}{2}\right) = 917$ SF

$A_T = 4 \times 917 = 3668$ SF

$A_I = 4A_T = 14672$ SF

LL RED

$L = L_o\left(0.25 + 15/\sqrt{A_I}\right) = 50\left(0.25 + 15/\sqrt{14672}\right) = 18.7$ PSF

$0.4L = 0.4(50) = 20$ PSF ⊕ CONTROLS

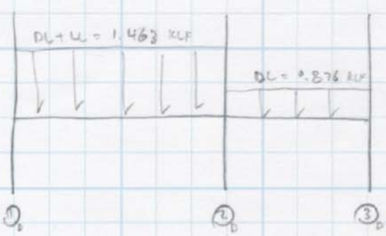
FACTORED LOADS

$1.4D = 1.4(88) = 123.2$ PSF

$1.2D + 1.6L = 1.2(88) + 1.6(20) = 137.6$ PSF

ROOF → $1.2(25) + 1.6(25) = 70$ PSF

$P_u = (137.6)(4 \times 917) + 70(917) = 568906$ lb $= 569^k$



DL+LL

factored loads from beams → 179 PSF

179 PSF $\times 8' = 1432$ PLF

$1432 + 1.2(26) = 1463$ PLF
self weight

DL

1.2×88 PSF $\times 8' = 845$ PLF

$845 + 1.2(26) = 876$ PLF
self weight

$FEM_{2-1} = \frac{1.463(33.\bar{3})^2}{12} = 135.5^{k}$

$FEM_{2-2} = \frac{0.876(24)^2}{12} = 42^{k}$

$\Delta FEM = 135.5 - 42 = 93.5^{k}$

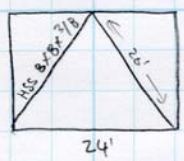
$M_u = \frac{\Delta FEM}{2} = \frac{93.5}{2} = 47^{k}$

$P_{eff} = 569 + \frac{24}{d}(M_u) = 569 + \frac{24}{12}(47) = 663^k < \phi P_n = 670$ ✓

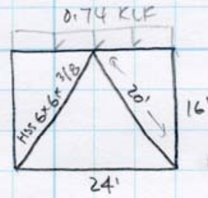
Bracing

BRACING for N-S lateral load $V = 1.6(280) = 448^k$

WB-2



WB-3



$0.74 \text{ KLF} \times 24 = 17.8^k$

$V = 448 + 17.8^k = \underline{466^k}$

braced frame $\equiv 0.8V = 373^k$

moment frame $\equiv 0.2V = 93^k$

conservative

DL = 88
LL = 50

$1.2D + 1.6L = 1.2(88) + 1.6(50) = 185.6$ PSF

$W_u = 185.6 \text{ PSF} \times 4' = 742.4$ PLF

LRFD 3rd Edition Table 4-6

$$KL = 20' \left\{ \begin{array}{l} \text{HSS } 8\times8\times3/8 \Rightarrow \phi P_n = 272^k \\ \text{HSS } 6\times6\times3/8 \Rightarrow \phi P_n = 141^k \end{array} \right\} 413^k > 373^k \checkmark$$

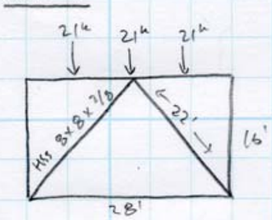
BRACING for E-W lateral load $V = 235^k$

$V = 235 + 21(3) = \underline{298^k}$

braced frame $\equiv 0.8V = 238^k$

moment frame $\equiv 0.2V = 60^k$

WB-1



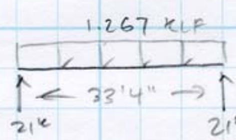
DL = 88
LL = 50

LL RED

$L = 50\left(0.25 + 15/\sqrt{466.\bar{6}}\right) = 47.2$ PSF

$A_T = 7' \times 33.\bar{3}' = 233.\bar{3}$

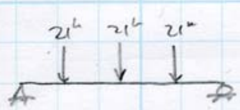
$A_I = 2A_T = 466.\bar{6}$



FACTORED LOAD

$1.2D + 1.6L = 1.2(88) + 1.6(47.2) = 181$ PSF

$W_u = 181 \times 7' = 1267$ PLF



LRFD 3rd Edition Table 4-6

$KL = 22'$ HSS $8\times8\times3/8 \Rightarrow \phi P_n = 250^k > 238^k \checkmark$

Snow Load Calculation

Ground Snow Load (p_g) =	20.0 psf	(Fig. 1608.2 p. 16-12)
Bldg. Classification Category =	III	(Table 1604.5 p. 16-5)
Snow Importance Factor (I_S) =	1.1	(Table 1604.5 p. 16-5)
Snow Exposure Factor (C_e) =	1.0	(Table 1608.3.1 p. 16-14)
Snow Thermal Factor (C_t) =	1.0	(Table 1608.3.2 p. 16-14)

Min Flat Roof Snow Load =	22.0 psf	(ASCE 7 Section 7.3)
Flat Roof Snow Load ASCE 7 Eq 7-1 (p_f) =	15.4 psf	(ASCE 7 Section 7.3)
Rain-on-snow Surcharge Load =	20.4 psf	(ASCE 7 Section 7.10)
Flat Roof Snow Load Used (p_f) =	**22.0 psf**	
Roof Slope Factor Unobstructed & Slippery (C_S) =	1.00	(ASCE 7 Fig 7-2 p. 84)
Roof Slope Factor All Others (C_S) =	1.00	(ASCE 7 Fig 7-2 p. 84)
Sloped Roof Snow Load Unobstruct. & Slippery (P_S) =	22.0 psf	
Sloped Roof Snow Load All Others (P_S) =	22.0 psf	
Sloped Roof Snow Load Used (P_S) =	**22.0 psf**	
Snow Density (γ) =	16.6 pcf	(ASCE 7 Eq 7-4 p. 79)

Location	Length of Upper Roof, l_u (ft)	Length of Lower Roof, l_u (ft)	Δ Roof Height (ft)	**Drift Width W_d (ft)**	**P_{max} (psf)**	**P_{min} = P_f (psf)**	**P at Edge of Roof (psf)**
Intermed. R	45	89	12.1	**9.0**	**59.3**	**22.0**	**22.0**

Wind Load Calculation (North – South)

Input Parameters:

Basic Wind Speed (V, mph) =	80	Building Length (L, ft) = (Parallel to Wind)	114.67
Exposure Category =	B	Building Width (B, ft) = (Normal to Wind)	204.67
Bldg. Classification Category =	III	Roof Slope (θ, deg.) = (Degrees from Horz.)	0 in per foot 0.00 deg. from horiz.
Wind Importance Factor (I_W) =	1.15	Internal Pressure Coef (GC_{pi}) =	0.18 (Pressure) -0.18 (Suction)
Mean Building Height (h, ft) =	76.666666		
Multipilers to obtain Topographic factor: K1 =	1		
K2 =	1		
K3 =	1		
Topographic Factor (K_{zt})=	1		
Wind Directionality Factor (K_d)=	0.85		

Calculated Parameters:			
Velocity Press. Exposure Coef at h (K_h)=	0.92	h/L =	0.669
Velocity Pressure at h (q_h) =	14.73 psf	L/B =	0.560
Gust Effect Factor (G) =	0.813	h/B =	0.375
Internal Pressure (q_hGC_{pi}) =	2.7 psf	α =	7
	-2.7 psf	z_g =	1200

MWFRS Wall Pressures

Height Above Ground	K_z	q_z (psf)	Side Wall External Pressure (psf)	Side Wall External +Int. Press. (psf)	Side Wall External - Int. Press. (psf)	Windward Wall External Pressure (psf)	Windward Wall External +Int. Press. (psf)	Windward Wall External - Int. Press (psf)	Leeward Wall External Pressure (psf)	Leeward Wall External +Int. Press (psf)	Leeward Wall External - Int. Press. (psf)	Windwrd +Leewrd (psf)
0-15	0.57	9.1	-8.4	-5.7	**-11.0**	5.9	**8.6**	3.3	-6.0	-3.3	**-8.6**	**17.2**
20	0.62	9.9	-8.4	-5.7	**-11.0**	6.5	**9.1**	3.8	-6.0	-3.3	**-8.6**	**17.8**
25	0.67	10.7	-8.4	-5.7	**-11.0**	7.0	**9.6**	4.3	-6.0	-3.3	**-8.6**	**18.3**
30	0.70	11.2	-8.4	-5.7	**-11.0**	7.3	**9.9**	4.6	-6.0	-3.3	**-8.6**	**18.6**
40	0.76	12.2	-8.4	-5.7	**-11.0**	7.9	**10.6**	5.3	-6.0	-3.3	**-8.6**	**19.2**
50	0.81	13.0	-8.4	-5.7	**-11.0**	8.4	**11.1**	5.8	-6.0	-3.3	**-8.6**	**19.7**
60	0.85	13.6	-8.4	-5.7	**-11.0**	8.9	**11.5**	6.2	-6.0	-3.3	**-8.6**	**20.1**
70	0.89	14.3	-8.4	-5.7	**-11.0**	9.3	**11.9**	6.6	-6.0	-3.3	**-8.6**	**20.6**
80	0.93	14.9	-8.4	-5.7	**-11.0**	9.7	**12.3**	7.0	-6.0	-3.3	**-8.6**	**21.0**
90	0.96	15.4	-8.4	-5.7	**-11.0**	10.0	**12.7**	7.3	-6.0	-3.3	**-8.6**	**21.3**

Wind Load Calculation (East – West)

Input Parameters: Basic Wind Speed (V, mph) =	80	Building Length (L, ft) = (Parallel to Wind)	204.67
Exposure Category =	B	Building Width (B, ft) = (Normal to Wind)	114.67
Bldg. Classification Category =	III	Roof Slope (θ, deg.) = (Degrees from Horz.)	0 in per foot 0.00 deg. from horiz.
Wind Importance Factor (I_W) =	1.15	Internal Pressure Coef (GC_{pi}) =	0.18 (Pressure) -0.18 (Suction)
Mean Building Height (h, ft) =	76.666666		
Multipilers to obtain Topographic factor: K1 =	1		
K2 =	1		
K3 =	1		
Topographic Factor (K_{zt})=	1		
Wind Directionality Factor (K_d)=	0.85		

Calculated Parameters:			
Velocity Press. Exposure Coef at h (K_h)=	0.92	h/L =	0.375
Velocity Pressure at h (q_h) =	14.73 psf	L/B =	1.785
Gust Effect Factor (G) =	0.832	h/B =	0.669
Internal Pressure (q_hGC_{pi}) =	2.7 psf	α =	7
	-2.7 psf	z_g =	1200

Height Above Ground	K_z	q_z (psf)	Side Wall External Pressure (psf)	Side Wall External +Int. Press. (psf)	Side Wall External - Int. Press. (psf)	Windward Wall External Pressure (psf)	Windward Wall External +Int. Press. (psf)	Windward Wall External - Int. Press (psf)	Leeward Wall External Pressure (psf)	Leeward Wall External +Int. Press (psf)	Leeward Wall External - Int. Press. (psf)	Windwrd +Leewrd (psf)
0-15	0.57	9.1	-8.6	-5.9	**-11.2**	6.1	**8.7**	3.4	-4.2	-1.6	**-6.9**	**15.6**
20	0.62	9.9	-8.6	-5.9	**-11.2**	6.6	**9.3**	4.0	-4.2	-1.6	**-6.9**	**16.1**
25	0.67	10.7	-8.6	-5.9	**-11.2**	7.1	**9.8**	4.5	-4.2	-1.6	**-6.9**	**16.7**
30	0.70	11.2	-8.6	-5.9	**-11.2**	7.5	**10.1**	4.8	-4.2	-1.6	**-6.9**	**17.0**
40	0.76	12.2	-8.6	-5.9	**-11.2**	8.1	**10.8**	5.4	-4.2	-1.6	**-6.9**	**17.6**
50	0.81	13.0	-8.6	-5.9	**-11.2**	8.6	**11.3**	6.0	-4.2	-1.6	**-6.9**	**18.1**
60	0.85	13.6	-8.6	-5.9	**-11.2**	9.1	**11.7**	6.4	-4.2	-1.6	**-6.9**	**18.6**
70	0.89	14.3	-8.6	-5.9	**-11.2**	9.5	**12.1**	6.8	-4.2	-1.6	**-6.9**	**19.0**
80	0.93	14.9	-8.6	-5.9	**-11.2**	9.9	**12.6**	7.3	-4.2	-1.6	**-6.9**	**19.4**
90	0.96	15.4	-8.6	-5.9	**-11.2**	10.2	**12.9**	7.6	-4.2	-1.6	**-6.9**	**19.7**

Seismic Calculations

Input Parameters:	
Building Height (h_n, feet) =	76.666666
Max. Considered Earthquake (MCE) Short Period (S_S) =	12.0%
MCE 1 sec. Period (S_1) =	4.6%
Site Soil Class =	D
Bldg. Classification Category =	II
Seismic Use Group =	I
Seismic Occupancy Importance Factor (I_E):	1
Response Modification Factor (R) =	3.25
Deflection Amplification Factor (C_d) =	4.5
C_T =	0.02
x=	0.75
Period Calculated from Analysis (T, sec.) =	0.84

Level	Story Height (ft.)	Elevation, h (ft.)	Surface DL+SDL (psf)	Floor Surface Area (sq.ft)	Additional StoryDL (kips)	Story Weight (w, kips)	w*h^k (kip-ft^2)	C_{Vx}	Story Force (kips)	Story Shear (kips)	Overturning Moment (k-ft)
L1	0	0	0	0	0	0	0	0.00000	0.0	235	11852
L2	16	16	87.5	20290	0	1775.375	45510	0.07219	17.0	235	8095
L3	14	30	87.5	20290	0	1775.375	94956	0.15061	35.4	218	5044
L4	14	44	87.5	20290	0	1775.375	148638	0.23576	55.4	183	2489
L5	14	58	87.5	20290	0	1775.375	205353	0.32572	76.5	127	709
roof	14	72	45	20290	0	913.05	136011	0.21573	50.7	51	0