

Letter of Transmittal

November 14, 2015

Dr. Linda M. Hanagan. PhD, PE
The Pennsylvania State University
212 Engineering Unit A
University Park, PA 16802
lhanagan@engr.psu.edu

Dear Dr. Hanagan,

The attached document, Technical Report IV – Lateral System Analysis Study contains a comprehensive analysis of the lateral load resisting system of 706 Madison Avenue. This report documents strength and serviceability analyses of the existing lateral framing system of the building through manipulation of a computer model and presentation of hand calculations as well as sketches, inclusive of material submitted in Notebook A and B.

The computer analysis results, including wind loads, seismic loads, base shear, drifts etc. are validated by manual calculations in the structure notebook. In addition, the controlling load case and overall building torsion would be found and used to check the important elements of lateral force-resisting system.

Thank you for taking time to review and evaluate my report. I look forward to your comments and discussing my work with you in the future.

Sincerely,

Yong Yue

The Pennsylvania State University
Architectural Engineering Class of 2017

706 Madison Avenue | New York, USA

Lateral System Analysis Study

Structural Notebook Submission C



Submitted to: Dr. Linda Hanagan, Advisor

Prepared by: Yong Yue [Structural Option]

Prepared on: November 14th, 2016

Executive Summary

706 Madison Avenue is a 48,500 square-foot, high-end retail building located on the southwest corner of Madison Avenue and 63rd Street of the upper east side of Manhattan, New York. The building consists of a 3-story existing landmarked building and a five-story horizontal extension on two sides.

The existing landmarked building was built in 1920 and was initially constructed with masonry walls, steel columns, cinder concrete slabs, and marble and brick façades. Back to 1920s, building codes didn't require seismic design for structures. So the old building wasn't designed to resist seismic load; however, the masonry walls and core stairwells in the building have been designed for wind.

The addition took place on March 2015. It is still under construction and scheduled to be done in January 2016. The structural system consists of steel columns, concrete slab with composite metal deck, a mat foundation and moment frames for a lateral load resisting system. However, the addition's lateral load resisting system is independent from the existing building.

The building was designed based on the 2008 New York City Building Code. The exterior of building also needs to meet the historical requirements, which are regulated by Landmark Preservation Commission (LPC).

Table of Contents

1	Introduction	3
1.1	Purpose.....	3
1.2	Scope.....	3
1.3	Site Plan and Location Plan	3
1.4	Building Codes & Reference Standards	4
2	Gravity Load	5
2.1	Roof Construction.....	5
2.2	Floor Construction	6
2.3	Typical Exterior Wall Detail.....	7
2.4	Snow Load	9
3	Wind Load	11
3.1	Calculations.....	11
4	Seismic Load	20
4.1	Calculations.....	12
5	Typical Member Spot Checks for Gravity Loads	24
5.1	Composite Steel	24
6	Alternative Framing Systems for Gravity Loads	25
6.1	Alternative 1 – Steel Joists.....	25
6.2	Alternative 2 – One-Way Slab.....	25
6.3	Alternative 2 – Hollow Core Planks System	25
7	Comparisons	25
7.1	Weights per bay	25
7.2	Cost per bay	25
8	Lateral System Analysis Study	25
8.1	Modeling Information for Lateral Load Analysis.....	57
8.2	Model Validation	57
8.3	Lateral System Checks.....	57
8.4	Member Spot Checks for Lateral Loads	57

[1] Introduction

1.1 Purpose

This report is written to develop a detailed analysis of the lateral force-resisting system on 706 Madison Avenue. The existing lateral framing system of the building will be analyzed based on the strength and serviceability criteria.

1.2 Scope

Enclosed in the report is analysis of the steel moment frame system, 3D modeling documentation, and hand calculations used to verify results. The content of the report consists of modeling information, model validation, lateral system checks and member spot checks for lateral loads. Sketches and excel spreadsheets will be presented to describe/compare the data.

1.3 Site Location and Plan

As shown in the figure above (Figure 1 & 2), 706 Madison Avenue is located at the southwest corner of Madison Avenue and East 63rd Street, which is in a historical district at the upper east side of Manhattan, New York. The shape of the site is basically a rectangular, with a demension 90'x100'.

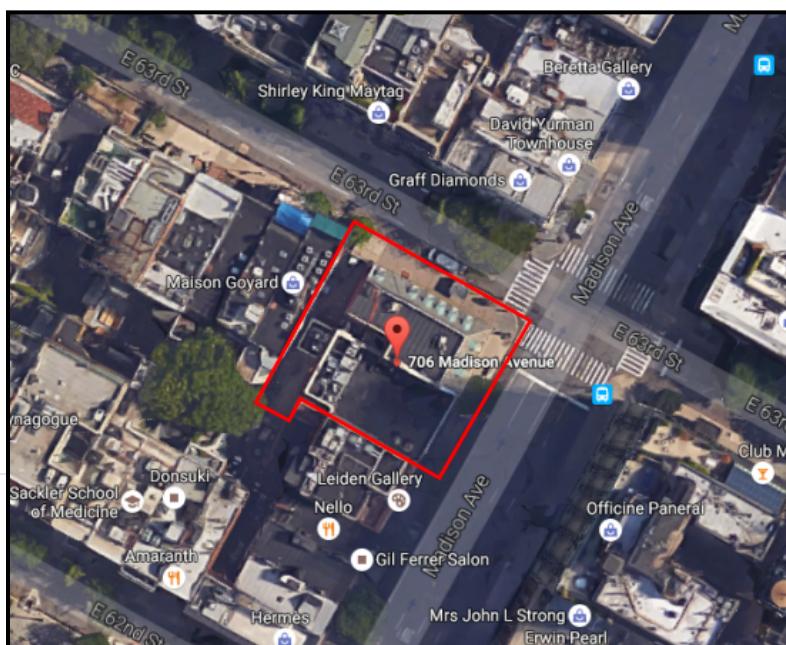


Figure 1
Site Context



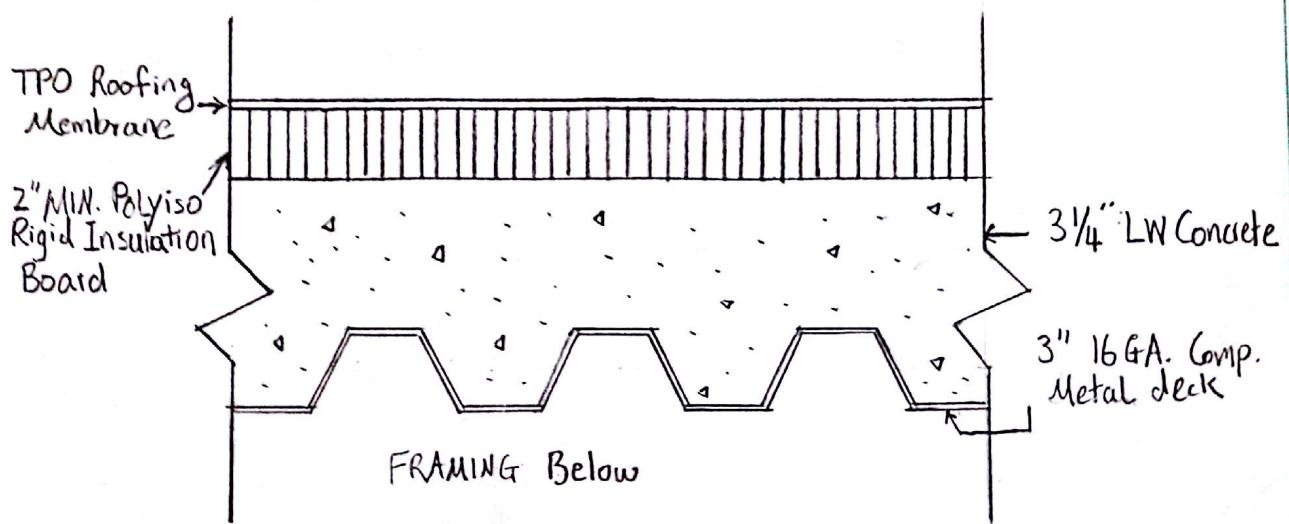
Figure 2 Site Context (Google Map)

1.4 Building Codes & Reference Standards

- A. New York City Building Codes (NYCBC), 2008 with Current Revisions
- B. ASCE 7 – 02 Minimum Design Loads for Buildings and Other Structures

[2] GRAVITY LOAD

2.1 Cross section of roof construction



- Roof Loading

- Dead Load: (according to ASCE 7-02 Table C3-1)

Roofing Membrane:	1 PSF
2" Rigid Insulation:	3 PSF
3 1/4" LW Concrete	46 PSF
3" 16 GA. Comp. deck	
Framing:	7 PSF
Miscellaneous:	<u>10 PSF</u>

$$DL_r = 67 \text{ PSF}$$

- Live Load:

$$LL_r = 20 \text{ psf} \quad (\text{according to ASCE 7-02 C4.9.1 Min. Roof Live Loads})$$

- Snow Load:

$$\text{Ground Snow Load; } P_g = 25 \text{ psf} \quad (\text{ASCE 7-02 Figure 7-1})$$

$$\text{Exposure Factor; } C_e = 0.9 \quad (\text{ASCE 7-02 Table 7-2 for Terrain Category B and a fully exposed roof})$$

$$\text{Thermal Factor; } C_t = 1.0 \quad (\text{ASCE 7-02 Table 7-3})$$

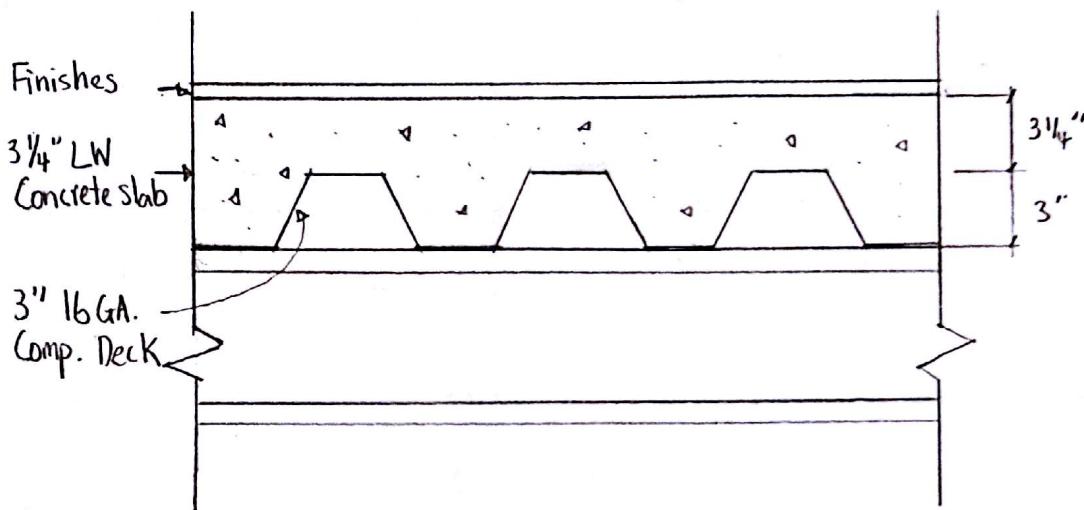
$$\text{Important Factor; } I_s = 1.0 \quad (\text{ASCE 7-02 Table 7-4})$$

$$\begin{aligned} \text{Flat Roof Snow Load; } P_f &= 0.7 (C_e C_t I_s P_g) \\ &= 0.7 (0.9)(1)(1)(25) = 16 \text{ PSF} < 20 \text{ PSF (Min)} \end{aligned}$$

Thus, use $P_f = 20 \text{ PSF}$.

GRAVITY LOAD (cont)

2.2 Cross section of floor construction



- Floor Loading
 - Dead Load

Finishes:	2 psf
Concrete Slab + deck:	46 psf
Framing:	7 psf
Columns:	1 psf
Miscellaneous:	<u>10 psf</u>

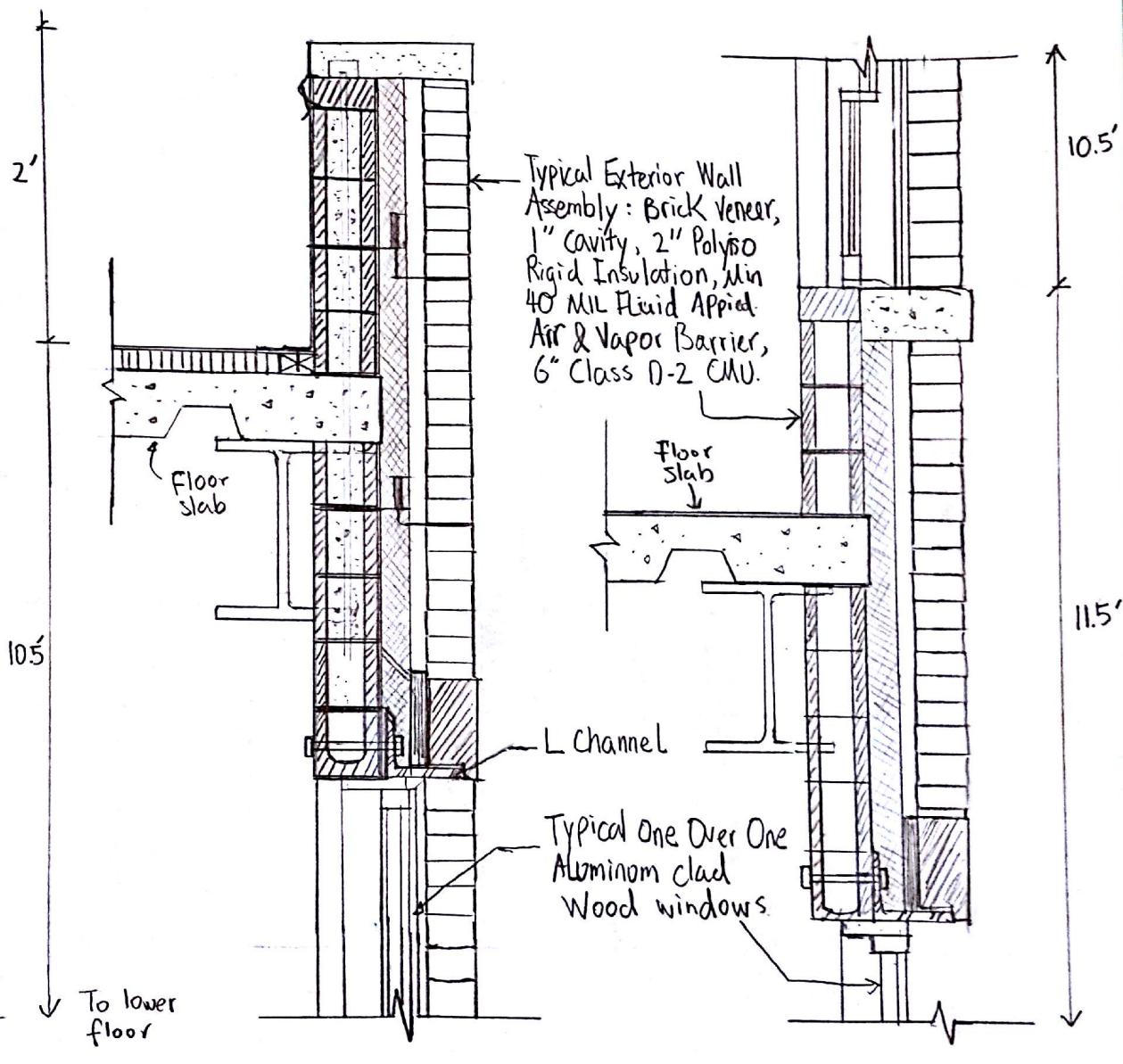
$$DL_f = 66 \text{ psf}$$

- Live load:

	Number Noted in Drawings	Code Minimum (ASCE 7-02)
Retail - 1st Floor	105 psf	100 psf
Retail - Upper Floors	75 psf	75 psf
Public Assembly Space	100 psf	100 psf
Stairs and Exits	100 psf	100 psf
Storage	125 psf	125 psf
Sidewalk	600 psf	250 psf
Elevator Machine Room	125 psf	150 psf

GRAVITY LOAD (cont.)

2.3 Gross section of typical wall details with load path description and dead load.



Exterior Wall Detail at Roof

Exterior Wall Detail at Floor

- Load Path:

Wall loads act on the L channels, L channels transfer loads into 6" Concrete masonry unit (CMU) through the bolts. CMUs transfer loads to the concrete slab with composite metal deck, concrete slabs transfer load to the transverse beams, the transverse beams transfer loads to the columns, and the columns transfer loads down to foundation.

GRAVITY LOAD (Cont.)

- Dead Load of Wall (From ASCE7-02 Table C3-1)

Hollow CMU wythes, 6", full grout:	55 psf
Clay brick wythes, 4":	39 psf
Rigid Insulation, 2":	1/2 psf
Vapor Barrier:	1/2 psf
	DL _w = 96 psf

For Roof:

$$W_{wall} = 96 \left(\frac{10.5}{2} + 2 \right) = 696 \text{ Plf}$$

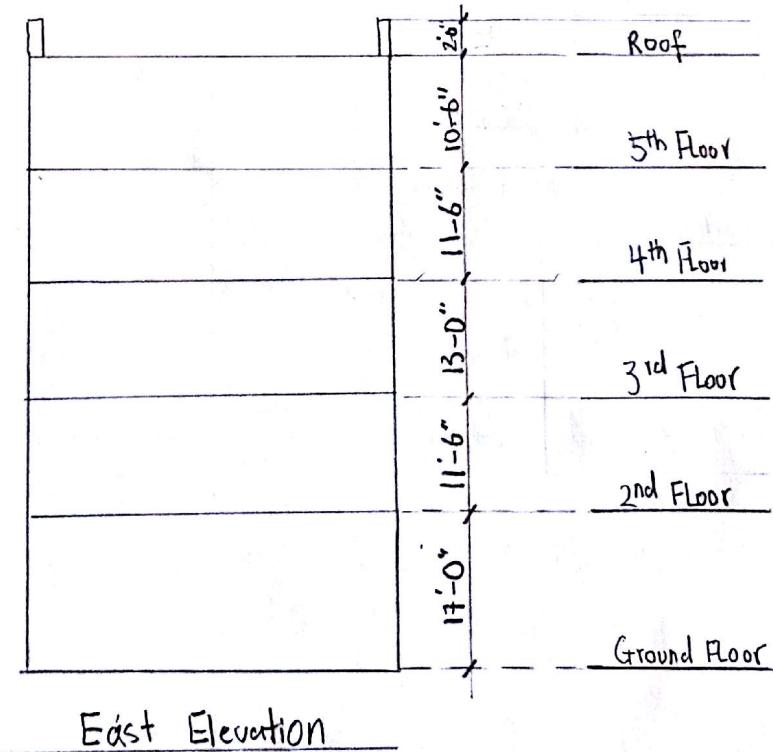
For floors:

$$W_{wall}(5) = 96 \left(\frac{10.5}{2} + \frac{11.5}{2} \right) = 1056 \text{ Plf}$$

$$W_{wall}(4) = 96 \left(\frac{11.5}{2} + \frac{13}{2} \right) = 1176 \text{ Plf}$$

$$W_{wall}(3) = 1176 \text{ Plf}$$

$$W_{wall}(2) = 96 \left(\frac{11.5}{2} + \frac{17}{2} \right) = 1368 \text{ Plf}$$



2.4 Snow Load

- Drift @ Parapet (Windward drift)

$$P_g = 25 \text{ PSF}$$

$$P_f = 20 \text{ PSF}$$

(From previous calcs.)

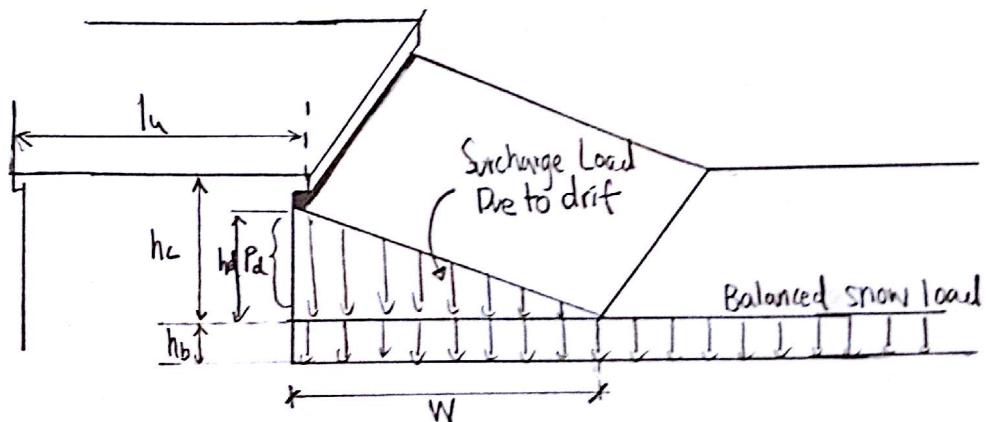
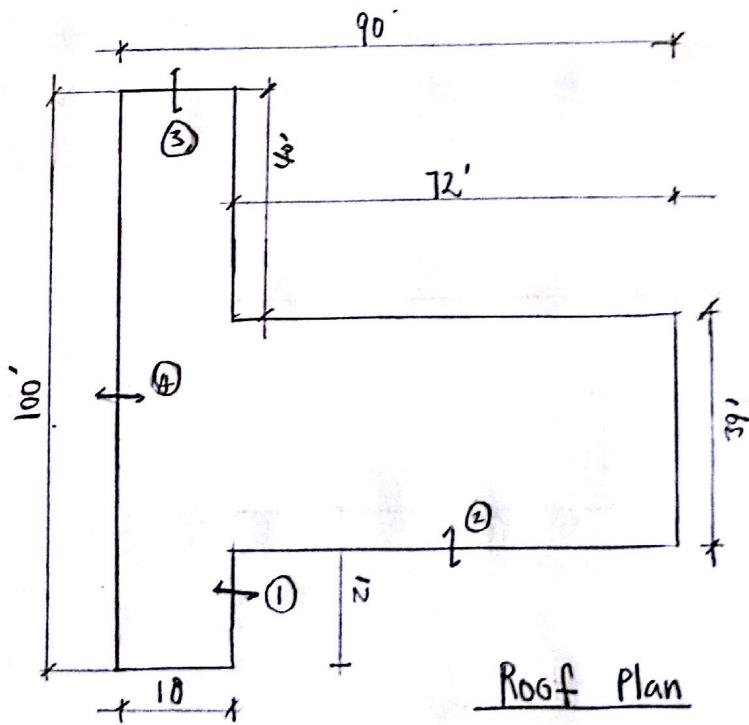


Figure 7.8 Snow Drifts on Lower Roof (ASCE 7-02)

$$\gamma = 0.13 P_g + 14 = 0.13(25) + 14 = 17.25 \text{ PCF} \text{ (but no more than } 30 \text{ PCF)}$$

$$h_b = 20 / 17.25 = 1.16 \text{ ft}$$

$$h_c = 2' - 1.16' = 0.84'; \quad \frac{h_c}{h_b} = \frac{0.84}{1.16} = .72 > 0.2 \Rightarrow \text{drift load must be considered.}$$



Roof Plan

Snow Load (cont.)

Parapet Section ①:

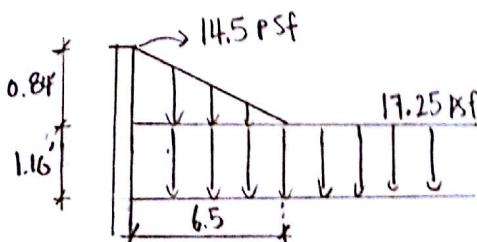
$$l_u = 18' < 25 \therefore \text{use } l_u = 25'$$

$$h_d = 0.75 [0.43 \sqrt[3]{25} \sqrt[4]{25+10} - 1.5] = 1.17 > h_c \therefore \text{Same drift for } ②, ③, ④$$

$$W = 4h_d^2/h_c \text{ & } h_d = h_c$$

$$W = \frac{4(1.17)^2}{0.84} = 6.5' < 8h_c = 8(0.84) = 6.72' \therefore W = 6.5'$$

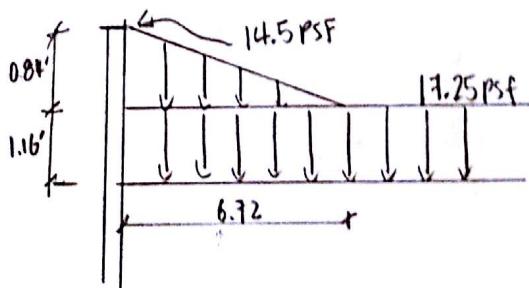
$$P_d = Y h_d = Y h_c = 17.25(0.84) = 14.5 \text{ psf}$$



Section 1

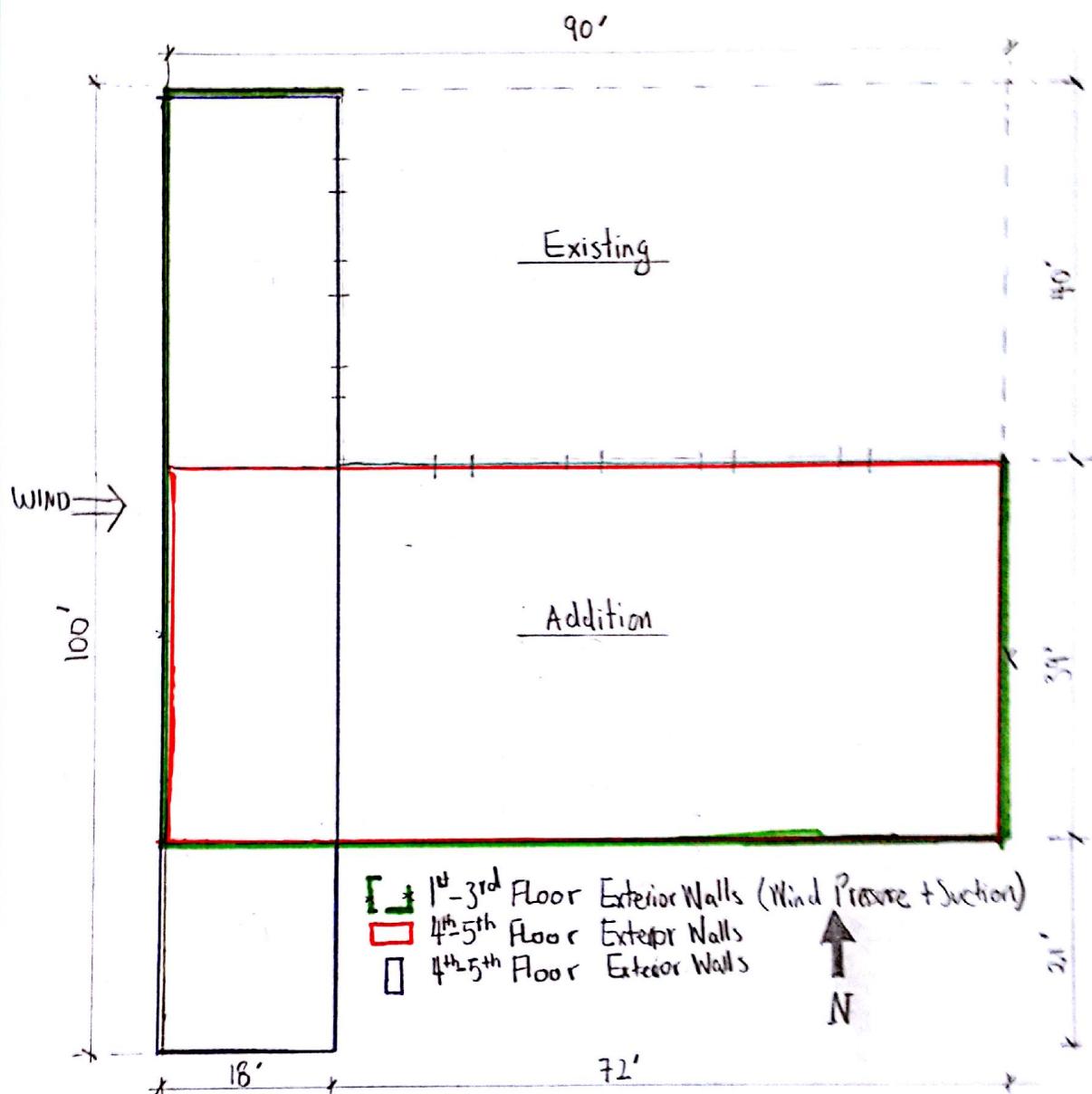
$$②: h_d = 0.75 [0.43 \sqrt[3]{39} \sqrt[4]{25+10} - 1.5] = 1.54 > h_c$$

$$W = \frac{4(1.54)^2}{0.84} = 11.2' > 8h_c = 6.72' \therefore W = 6.72' \text{ for } ②, ③, ④$$



$$P_d = Y h_c = 17.25(0.84) = 14.5 \text{ psf}$$

[3] WIND LOAD



- Risk Category: II (ASCE7-02 Table 1-1)
- Wind Speed: 98 mph (Figure 6-1, 3-sec gust)
- Wind Directionality Factor: $K_d = 0.85$ (Table 6-4)
- Wind Important Factor: $I_w = 1.0$ (Table 6-1)
- Exposure Category : B (Section 6.5.6.3)
- Topographic Factor : $K_{zt} = 1$ (1.0 except for isolated escarpments, ridges & hills Section 6.5.7)

WIND LOAD (cont.)

- Velocity Pressure Coefficients, k_z (Table 6-3)

Story	Ht. z (ft)	Stay Ht (ft)	k_z	I	k_{zt}	k_d	V^2	q_2 (psf)
1	0	17	0.7	1	1	.85	9604	14.6
2	17	11.5	0.7	1	1	.85	9604	14.6
3	28.5	13	0.7	1	1	.85	9604	14.6
4	41.5	11.5	0.77	1	1	.85	9604	16.1
5	53	10.5	0.82	1	1	.85	9604	17.1
Roof	63.5		.86	1	1	.85	9604	18.0

$$q_z = 0.00256 k_z k_{zt} k_d V^2 I \quad (\text{Eq. 6-15})$$

- Gust Effect Factor:

For structure steel moment resisting frame buildings:

$$\eta_g = 22.2/h^{0.8} = 22.2/(63.5)^{0.8} = .802 < 1 \quad (\text{ASCE 7-10, Eq. 26.9-2})$$

∴ Not rigid.

$\beta = 0.01$; Conservative for steel (ASCE 7-10, structural damping)

Exposure B $\Rightarrow \bar{z} = 1/4, \bar{b} = 0.45, l = 320', \bar{z} = 1/3, c = 0.3$ (ASCE 7-02 Table 6-2)

$$\bar{z} = 0.6(63.5) = 38.1' > 30'$$

$$L_{\bar{z}} = l \left(\frac{\bar{z}}{33} \right)^{\bar{z}} = 320 \left(\frac{38.1}{33} \right)^{1/3} = 336 \quad (\text{Eq. 6-7})$$

$$I_{\bar{z}} = c \left(\frac{33}{8} \right)^{1/6} = 0.3 \left(\frac{33}{38.1} \right)^{1/6} = .293 \quad (\text{Eq. 6-8})$$

Section	h (ft)	Bew (ft)	Lew (ft)
C ①	63.5	79	90
D ②	63.5	100	18
E ③	63.5	39	90

WING Load (cont.)

For ① Wind E-W →

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{8+h}{L_z} \right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63 \left(\frac{79+63.5}{336} \right)^{0.63}}} = .855$$

For ②

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{100+63.5}{336} \right)^{0.63}}} = .845$$

③

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{39+63.5}{336} \right)^{0.63}}} = .818$$

} Eq. 6-6

$$\bar{V}_z = \bar{b} \left(\frac{\bar{z}}{33} \right)^2 \sqrt{\left(\frac{8g}{60} \right)} = 0.45 \left(\frac{38.1}{33} \right)^{1/4} (98) \left(\frac{8g}{60} \right) = 67.0 \text{ ft/s (Eq. 6-14)}$$

R , the resonant response factor, $R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$ (Eq. 6-10)

$$N_1 = \frac{n_1 L_z}{\bar{V}_z} = \frac{802 \times 336}{67.0} = 4.02$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{1/3}} = \frac{7.47 \times 4.02}{(1 + 10.3 \times 4.02)^{1/3}} = .058 \quad (\text{Eq. 6-11})$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2} (1 - e^{-2\eta}) \quad (\text{Eq. 6-13a})$$

$$R_h = .245 \quad \eta = 4.6 n_1 h / \bar{V}_z = 3.50$$

$$R_B = .203 \quad \eta = 4.6 n_1 B / \bar{V}_z = 4.35 \quad \text{①}$$

$$R_B = .165 \quad \eta = 5.5 \quad \text{②}$$

$$R_B = .359 \quad \eta = 2.15 \quad \text{③}$$

$$R_L = .181 \quad \eta = 4.6 n_1 L / \bar{V}_z = 4.96 \quad \text{①}$$

$$R_L = .570 \quad \eta = .99 \quad \text{②}$$

$$R_L = .181 \quad \eta = 4.96 \quad \text{③}$$

$$R(①) = \sqrt{\frac{1}{0.01} (.058)(.245)(.203)(0.53 + 0.47 \times .181)} = .421$$

$$R(②) = \sqrt{\frac{1}{0.01} (.058)(.245)(.165)(0.53 + 0.47 \times .570)} = .433$$

$$R(③) = \sqrt{\frac{1}{0.01} (.058)(.245)(.359)(0.53 + 0.47 \times .181)} = .560$$

$$g_R = \left[2 \ln(3600 n_1) + \frac{5.77}{2 \ln(3600 n_1)} \right] = 4.14 \quad (\text{Eq. 6-9})$$

WING Load (cont.)

(Eq. 6-8)

$$G_f = 0.925 \left(\frac{1 + 1.7 I_2 \sqrt{g_a^2 Q^2 + g_a^2 R^2}}{1 + 1.7 g_a I_2} \right) = \left(\frac{1 + 1.7(7.93) \sqrt{(0.9)^2 (8.5)^2 + (1.14)^2 (1.21)^2}}{1 + 1.7(3.4)(2.93)} \right) = .9232 \approx .92 \quad (1)$$

$$G_f = 0.925 \left(\frac{1 + 1.7(2.93) \sqrt{(3.4)^2 (8.15)^2 + (1.14)^2 (4.33)^2}}{1 + 1.7(3.4)(2.93)} \right) = .9227 \approx .92 \text{ For } (2)$$

$$G_f = 0.925 \left(\frac{1 + 1.7(2.93) \sqrt{(3.1)^2 (8.78)^2 + (1.14)^2 (5.60)^2}}{1 + 1.7(3.4)(2.93)} \right) = .99 \text{ For } (3)$$

- Enclosed Building \Rightarrow Internal Pressure Coefficient, $G_C p_i = \pm 0.18$ (Fig. 6-5)
 - External Pressure Coefficient, C_p (Wind E-W) (Fig. 6-6)
- For (1) : $h/B = 1.14$; $h/L = .706$ $A = 79 \times 100 = 7900 \text{ ft}^2 \Rightarrow$ Red. factor = 0.8

- Walls : $C_{p, \text{windward, EW}} = 0.8$
 $C_{p, \text{leeward, EW}} = -0.47$
 $C_{p, \text{sidewall, EW}} = -0.7$

1	-0.5
1.14	-0.47
2	-0.3

For (2) : $h/B = 1.18$, $h/L = 3.53$. $A = 100 \times 18 = 1800 \text{ ft}^2 \Rightarrow R.f. = .8$

- Walls : $C_{p, \text{windward, EW}} = -0.8$
 $C_{p, \text{leeward, EW}} = -0.5$
 $C_{p, \text{sidewall, EW}} = -0.7$

- Roofs : $C_{p, \text{roof, EW}} (0-9) = -1.3 \times 0.8 = -1.04$
 $C_{p, \text{roof, EW}} (9-18) = -0.7$

For (3) : $h/B = 2.31$, $h/L = .706$, $A = 39 \times 100 = 3900 \text{ ft}^2 \Rightarrow R.f. = .8$

- Walls : $C_{p, w, EW} = .8$
 $C_{p, L, EW} = -0.28$
 $C_{p, S, EW} = -0.7$

2	-0.3
2.31	-0.2
4	

- Roofs : $C_{p, \text{roof, EW}} (0-31.75) = -.96$
 $C_{p, \text{roof, EW}} (31.75-63.5) = -.82$
 $C_{p, \text{roof, EW}} (63.5-90) = -.58$

h/L	$C_p D - \frac{h}{L}$	$h_2 - h$	$h - 2h$
0.5	-0.9	-0.9	-0.5
0.706	-0.96	-0.82	-0.58
1.0	-1.04	-0.7	-0.7

WIND LOAD (cont.)

- Pressure on the parapets $k_z @ 65.5 = 0.87$

$$G_{\text{Cp}} = +1.5 \text{ (windward)} \\ = -1.0 \text{ (leeward)}$$

$$q_p = 0.00256 (0.87) (0.85) (1) (9604) (1) = 18.2 \text{ psf}$$

$$P_p, \text{ windward} = 18.2 \times (1.5) = 27.3$$

$$P_p, \text{ leeward} = 18.2 \times (-1.0) = -18.2$$

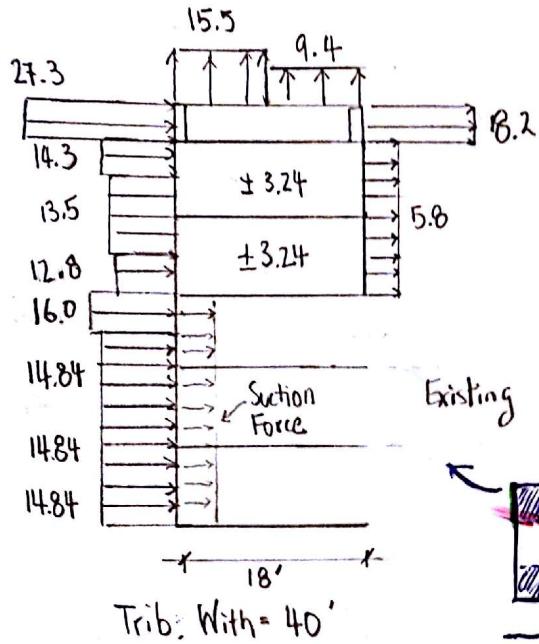
- Pressure on each surface : $P = q_z G_{\text{Cp}} - q_i (G_{\text{Cp}})$ (Eq. 6-23)

use $G_f = 0.99$ (Conservative)

Location	z (ft)	q_z psf	C_p	$q_z G_{\text{Cp}}$ (psf)	G_{Cp}	$q_h G_{\text{Cp}}$ (psf)	Net Pressure (psf)	
							$q_z G_{\text{Cp}} - q_n (+G_{\text{Cp}})$ (psf)	$q_z G_{\text{Cp}} - q_n (-G_{\text{Cp}})$ (psf)
Windward	0	14.6	0.8	11.6	0.18	3.24	8.36	14.84
	17	14.6		11.6			8.36	14.84
	28.5	14.6		11.6			8.36	14.84
	41.5	16.1		12.8			9.56	16.04
	53	17.1		13.5			10.3	16.74
	65.5	18.0	✓	14.3	↓	↓	11.1	17.54
Leeward (2)	All	18.0	-0.5	-9.0			-12.2	-5.8
	(3)	All	18.0	-0.28	-5.0	0.18	3.24	-8.2
Parapet (W)	65.5	18.2			1.5			27.3
	(L)	65.5	18.2		-1.0			-18.2
Roof (2)	0-9			-1.04	-18.7		-21.9	-15.5
	9-18	63.5	18.0	-0.7	-12.6		-15.8	-9.4
	0-31.75			-0.96	-17.3	0.18	3.24	-20.5
	31.75-63.5	63.5	18.0	-0.82	-14.8			-14.1
	63.5-90			-0.58	-10.4			-11.6
							-13.6	-7.2

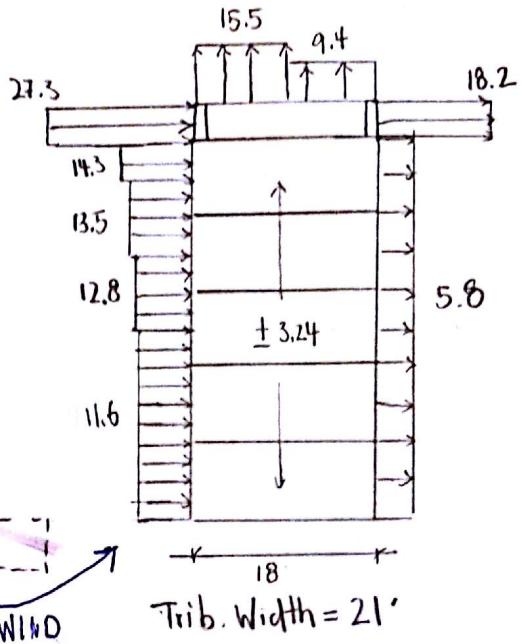
WIND LOAD (cont.)

North Part of ② (with Suction) Wind E-W



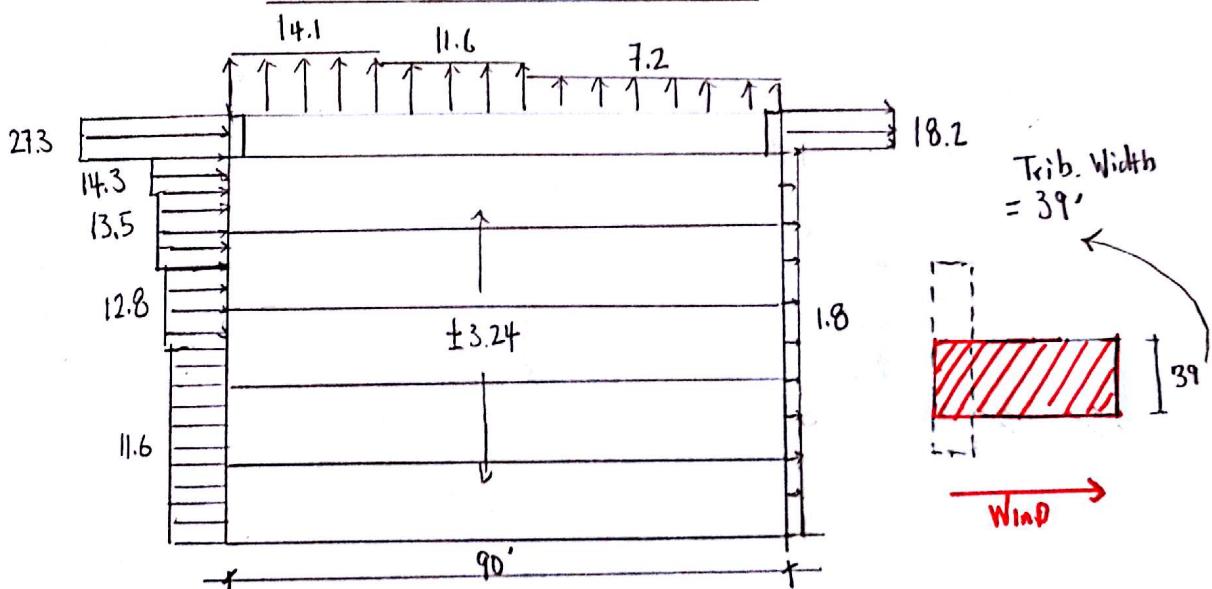
$$V = [14.8(17+11.5+6.5) + 16(6.5) + (12.6+5.8)(5.75) + (13.5+5.8)(5.75+5.25) + (14.3+5.8)(5.25) + (27.3+18.2)(2)] \times 40 \\ = 45.5 \text{ K}$$

South Part of ② (without suction) Wind E-W



$$V = [(1.6+5.8)(17+11.5+6.5) + (12.8+5.8)(6.5+5.75) + (13.5+5.8)(5.75+5.25) + (14.3+5.8)(5.25) + (27.3+18.2)(2)] \times 21 \\ = 26.0 \text{ K}$$

③ Wind E-W



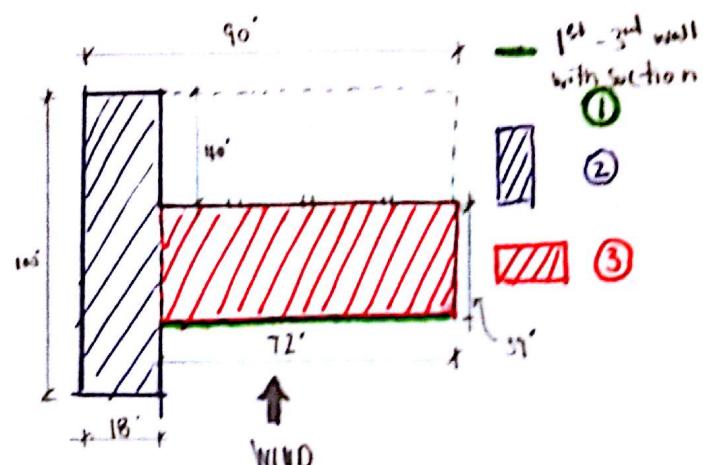
$$V = [(11.6+1.8)(17+11.5+6.5) + (12.8+1.8)(6.5+5.75) + (13.5+1.8)(5.75+5.25) + (14.3+1.8)(5.25) + (27.3+18.2)(2)] \times 39 = 38.4 \text{ K}$$

$$* \text{Base Shear (E-W)} = 45.5 + 26 + 38.4 = 110 \text{ K}$$

WIND LOAD (cont.)

* N-S Direction

Section	h (ft)	B_{NS} (ft)	L_{NS} (ft)
①	63.5	72	79
②	63.5	18	100
③	63.5	72	39



For ① & ③:

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{32 + 63.5}{336} \right)^{0.63}}} = .859 \quad \left\{ \text{(Eq. 6-6)} \right.$$

For ②:

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{18 + 63.5}{336} \right)^{0.63}}} = .892 \quad \left\{ \text{from previous calc.} \right.$$

R , the resonant response factor, $R = \sqrt{\frac{1}{\rho} R_n R_h R_L (0.58 + 0.47 R_L)}$ (Eq. 6-10)

$$N_1 = 4.02, R_n = .058, V_{\infty} = 67.0' \quad \left[\text{from previous calc.} \right]$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2} (1 - e^{-2\eta}) \quad \left\{ \text{(Eq. 6-13a)} \right.$$

$$R_h = .245$$

$$R_B = .220$$

$$R_D = .570$$

$$\eta = \frac{4.6 n_1 B}{V_{\infty}} = 3.96 \quad \text{for } ① \& ③$$

$$\eta = .991 \quad \text{for } ②$$

$$R_L = .203$$

$$R_L = .165$$

$$\beta_L = .359$$

$$\eta = \frac{4.6 n_1 L}{V_{\infty}} = 4.35 \quad \text{for } ①$$

$$= 5.50 \quad \text{for } ②$$

$$= 2.15 \quad \text{for } ③$$

$$R(①) = \sqrt{\frac{1}{0.01} (.058)(.245)(.220)(.58 + 0.47 \times .203)} = .460 \quad \left\{ \text{from previous calc.} \right.$$

$$R(②) = \sqrt{\frac{1}{0.01} (.058)(.245)(.57)(.58 + 0.47 \times .165)} = .730 \quad \left\{ \text{from previous calc.} \right.$$

$$R(③) = \sqrt{\frac{1}{0.01} (.058)(.245)(.220)(.58 + 0.47 \times .359)} = .484 \quad \left\{ \text{from previous calc.} \right.$$

WIND Load (cont.)

- $g_r = 4.14$ (From previous calcs.)

- $G_f = 0.925 \left(\frac{1 + 1.7(0.293) \sqrt{(3.4)^2(8.59)^2 + (4.14)^2(4.6)^2}}{1 + 1.7(0.293)(3.4)} \right) = 0.9398 \approx 0.94 \quad (1)$

$$G_f = 0.925 \left(\frac{1 + 1.7(0.293) \sqrt{(3.4)^2(8.92)^2 + (4.14)^2(7.3)^2}}{1 + 1.7(3.4)(0.293)} \right) = 1.076 \approx 1.08 \quad (2) \leq \underline{\text{Governing}}$$

$$G_f = 0.925 \left(\frac{1 + 1.7(0.293) \sqrt{(3.4)^2(8.59)^2 + (4.14)^2(1.484)^2}}{1 + 1.7(3.4)(0.293)} \right) = 0.9478 \approx 0.95 \quad (3)$$

- External Pressure Coefficient, C_p (Wind N-S) [Fig. 6-6]

For (1): $L_B = 1.1$, $h_L = 8.04$

- Walls: $C_{p,w,ns} = 0.8$
 $C_{p,L,ns} = -0.48$
 $C_{p,S,ns} = -0.7$

1	-0.5
1.1	-0.48
2	-0.3

For (2): $L_B = 5.56$, $h_L = 6.85$ $A = 1800 \Rightarrow R_f = 0.8$

- Walls: $C_{p,w,ns} = 0.8$
 $C_{p,L,ns} = -0.2$
 $C_{p,S,ns} = -0.7$

h_L	$0-h_L$	h_L-h	$h-2h$
0.5	-0.9	-0.9	-0.5
0.85	-0.44	-0.85	-0.55
1.0	-1.04	-0.7	-0.7

For (3): $L_B = 0.542$, $h_L = 1.63$ $A = 39 \times 72 = 2808 \Rightarrow R_f = 0.8$

- Walls: $C_{p,w,ns} = 0.8$
 $C_{p,L,ns} = -0.5$
 $C_{p,S,ns} = -0.7$

- Roofs: $C_{p,rf}(0-31.75) = -1.04$
 $C_{p,rf}(31.75-33.5) = -0.7$

WIND LOAD (cont.)

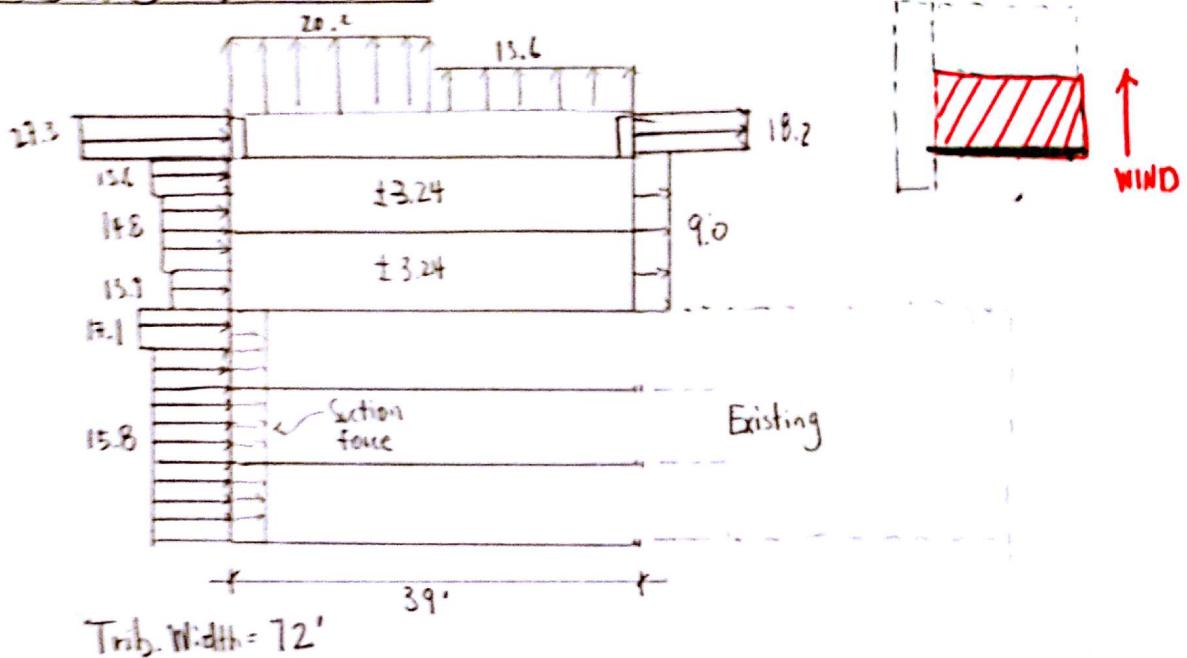
• Pressure on each surface : $P = q_a G_{sp} - q_n G_{pi}$ (Eq. 6-23)

Use $G_f = 1.08$

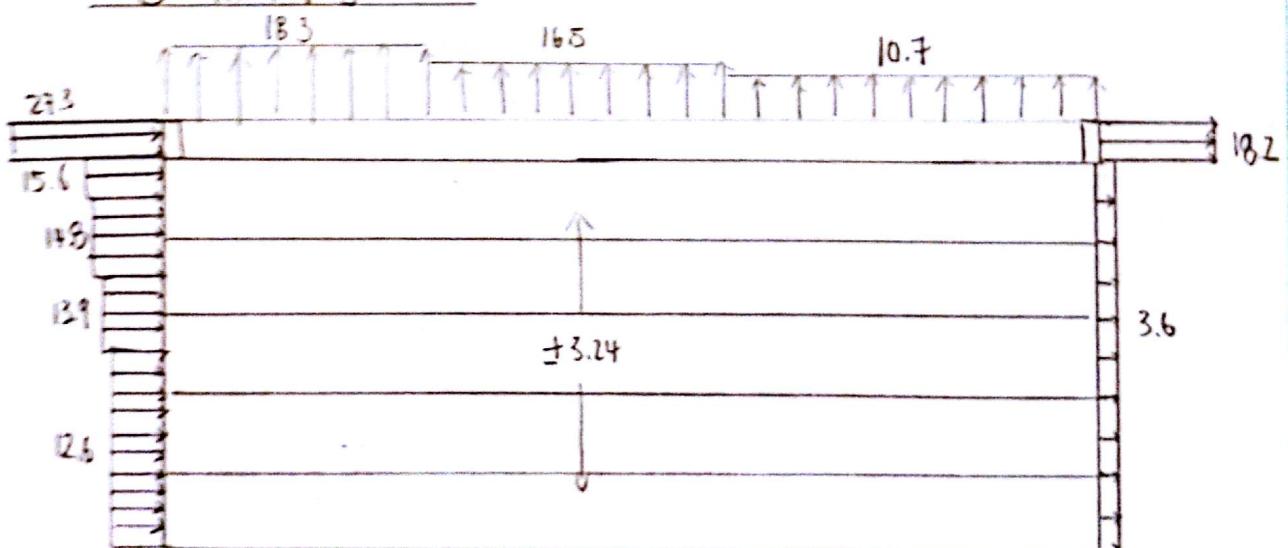
Location	z (ft)	q_a (psf)	c_p	$q_a G_{sp}$ (psf)	G_{sp}	$q_n G_{pi}$ (psf)	$q_a G_{sp} - q_n G_{sp}$ (psf)	$q_a G_{sp} - q_n (-G_{pi})$ (psf)
Windward	0	14.6	0.8	12.6	0.18	3.24	9.4	15.8
	17	14.6		12.6			9.4	15.8
	28.5	14.6		12.6			9.4	15.8
	41.5	16.1		13.9			10.7	17.1
	53	17.1		14.8			11.6	18
	63.5	18.0	↓	15.6			12.4	18.8
Leeward	(2) ALL	18.0	-0.2	-3.6			-6.8	-0.4
	(3) ALL	18.0	-0.5	-9.0	↓	↓	-12.2	-5.8
Parapet (W)	65.5	18.2			1.5			27.3
	(L)	65.5	18.2		-1.0			-18.2
Roof	0-31.5			-9.4	-18.3		-21.5	-15.1
	(2) 31.5-63.5	63.5	18.0	-8.5	-16.5	0.18	-19.7	-13.3
	63.5-100			-5.5	-10.7	3.24	-13.9	-7.5
	(3) 0-31.5			-1.04	-20.2		-23.4	-17.0
	31.5-39			-0.7	-13.6		-16.8	-10.6

WIND LOAD (cont.)

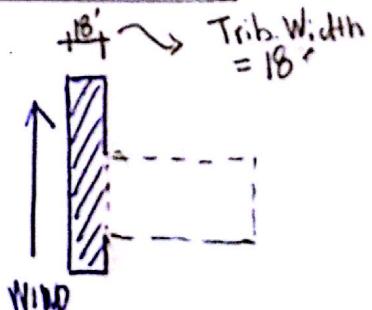
• ① & ③ Wind N-S



• ② Wind N-S



$$\text{* Base Shear (N-S)} = 92 + 21.2 = 113.2 \text{ k}$$



[4] SEISMIC LOAD

- Structure not exempt (ASCE 7-02 § 9.1.2)
- Site Class D ($\frac{S_{D1}}{S_{D2}} = \frac{0.114}{0.367} < 0.5$)
- $S_s = 0.365$ $S_{D2} = 0.367$
 $S_1 = 0.071$ $S_{D1} = 0.114$ (From USGS Design Maps Report)
- Seismic design category
 - Table 9.4.2.1a, $0.33 \leq S_{D2} < 0.5 \rightarrow C$
 - Table 9.4.2.1b, $0.067 \leq S_{D1} < 0.133 \rightarrow B \therefore SDLC$.
- Table 9.5.2.5.1, ELF is permitted, use LEF Procedure.
- Response Modification Factor, R (Table 9.5.2.2)
 - Ordinary Steel Moment Frames
 - No height limit
 - $R = 3.5, C_d = 3, W_o = 3$ (noted $R=3$ in design)
- Risk < Category II \Rightarrow Seismic Use Group I (Table 9.1.3)
 - \Rightarrow Seismic Important Factor, $I_e = 1.0$ (Table 9.1.4)
- Fundamental Period of the Building, T_a

$$T_a = C_t \cdot h_n^x \quad (\text{Eq. 9.5.5.3.2-1})$$

$$C_t = 0.028, x = 0.8 \quad (\text{Table 9.5.5.3.2})$$

$$T_a = 0.028 (63.5)^{0.8} = 0.775$$

• Seismic Response Coefficient, C_s

$$C_s = \frac{S_{D2}}{R/I_e} = \frac{0.367}{3/1} = 0.122 \quad (\text{Eq. 5.5.5.2.1-1})$$

$$\therefore C_s = \frac{S_{D1}}{T(R/I_e)} = \frac{0.114}{0.775 \times 3} = 0.049 \Leftarrow \text{Controls} \quad (\text{Eq. 5.5.5.2.1-2})$$

$$\therefore C_s = 0.049$$

$$\text{check: Max} \left| \frac{0.049 S_{D2}}{0.01} \right| = 0.16 \quad 0.049 > 0.01 \quad \checkmark$$

$$\therefore C_s = 0.049$$

SEISMIC LOAD

- Effective Total Seismic Weight

$$P_f = 20 \text{ psf} < 30 \text{ psf} \Rightarrow \text{Snow load is not considered} \quad (\S 9.5.3)$$

$$\begin{aligned} W_{\text{Roof}} &= (18 \times 100 + 72 \times 39)(67 + 20) + 2(100 + 90)(696) \\ &= 665.4 \text{ k} \end{aligned}$$

$$\begin{aligned} W_{\text{Floor}} &= 4(18 \times 100 + 72 \times 39)(100) + 2(100 + 90)(1056 + 1176 + 1176 + 1368) \\ &= 3658.1 \text{ k} \end{aligned}$$

$$\begin{aligned} W_{\text{Total}} &= W_{\text{Roof}} + W_{\text{Floors}} \\ &= 3658.1 + 665.4 \\ &= 4323.5 \text{ k} \end{aligned}$$

- Seismic Base Shear.

$$V = C_s W = 0.049 * 4323.5 = 212 \text{ k} \quad (\text{Eq. 9.5.5.2.1})$$

- Vertical Distribution of Seismic Forces (F_x)

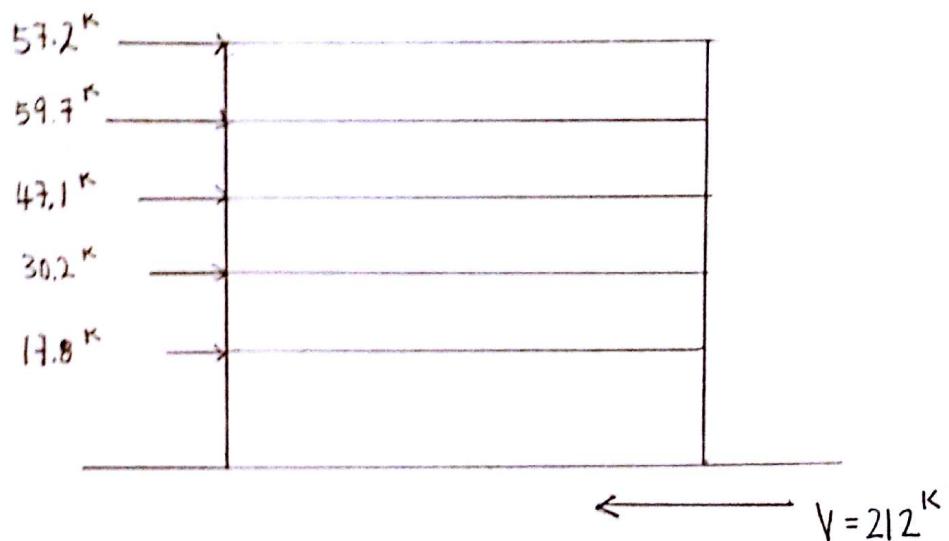
$$F_x = C_{vX} \cdot V = \left[\frac{W_x h_x^k}{\sum_{i=1}^n W_i h_i^k} \right] V \quad (\text{Eq. 9.5.4.4})$$

$$V_x = \sum_{i=x}^n F_i \quad [\text{Eq. 9.5.5.5}]$$

Ta	K
0.5	1
0.75	1.18
2.5	2

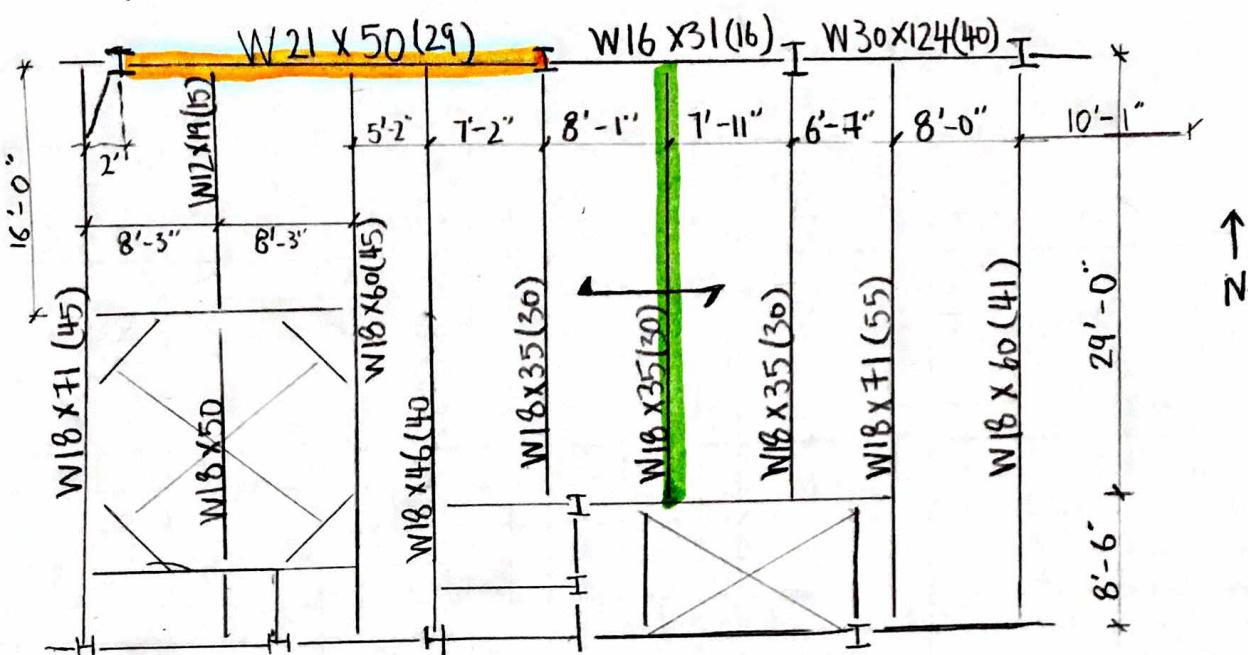
Level	h_x (ft)	W_x (kip)	$W \times h_x^k$	C_{vX}	F_x (kip)	V_x (kip)
Roof	63.5	665.4	89363.9	.270	57.2	57.2
5 th	53	862.1	93369.1	.282	59.7	116.9
4 th	41.5	907.7	73661.3	.222	47.1	164.0
3 rd	28.5	907.7	47278.0	.143	30.2	194.2
2 nd	17	980.6	27760.1	.084	17.8	212
	Σ	4323.5	331432.4	1		

SEISMIC LOAD



Seismic loading vs. height

Composite Steel:



* This building doesn't really have a typical bay, so the critical infill beam and the girder have been chosen to evaluate the floor framing for gravity loads.

1) Composite Decking:

3 1/4" LW CONCRETE OVER 3"-16 GA. METAL Deck

- 2 hr fire-rating reqd.
- Superimposed Dead Load:

Finishes	2 psf
Beam, Girder, Col.	8 psf
Misc	10 psf
<hr/>	
	20 psf

- Live load: 100 psf

$$W_{\text{Total}} = 100 + 23 = 123 \text{ psf}$$

From Vulcraft Steel Roof and Floor Deck catalog, Appendix 1.

- Max 3 SPAN Unshored = 15'-10" > 10'-1" ∴ OK
- @ 10'-5". SDL = 254 psf > 123 psf ∴ OK
- Slab Weight = 46 psf

Notebook B Existing Typical Bay

2) Infill Beams

$$\text{Dead: } 20 + 46 = 66 \text{ PSF}$$

LIVE: 100 PSF

(From Submission A)

- Unshored Strength

$$1.4D = 1.4(66 \text{ PSF}) = 92.4 \text{ PSF}$$

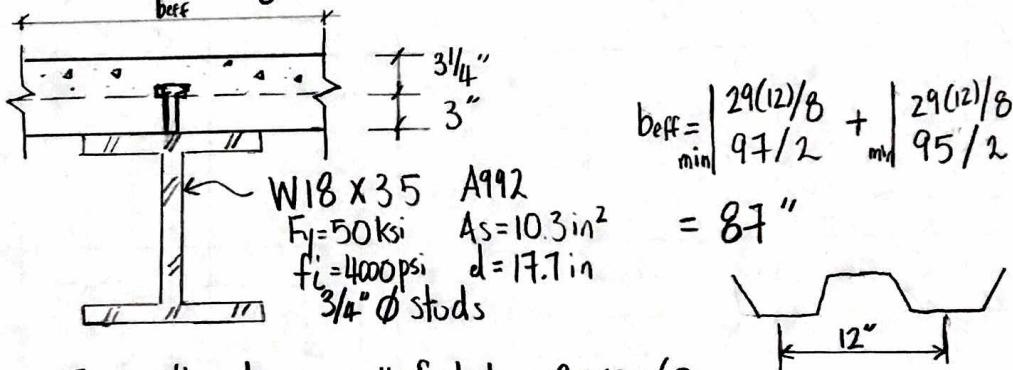
$$1.2D + 1.6L = 1.2(66 \text{ PSF}) + 1.6(20) = 112.2 \text{ PSF}$$

$$W = \left(\frac{8'-1'' + 7'-11''}{2}\right)(112.2 \text{ PSF}) = 898 \text{ plf}$$

$$M = \frac{Wl^2}{8} = \frac{(898)(24)}{8} = 94.4 \text{ k-ft}$$

From Table 3-2, W18 X 35; $\phi M_p = 249 \text{ k-ft} > 94.4 \text{ k-ft} \therefore \text{OK}$

- Composite Strength



From the drawing, # of studs = $30 \times 2 = 60$

$$60 \times 1' / 29' = 2.0 \Rightarrow 2 \text{ studs/rib}$$

From Table 3-21, $Q_n = 14.6 \text{ k}$

$$\sum Q_n = 30 * 14.6 = 438 \text{ k}$$

$$\begin{aligned} V_{cm} &= 0.85 f'_c b_{eff} \cdot t = 0.85(4)(87)(3.25) = 961.4 \text{ k} \\ V_{sm} &= 515 \text{ k} \quad (\text{Table 3-19}) \end{aligned} \} > 438 \text{ k}$$

\therefore Partially Composite

$$\sum Q_n = 0.85 f'_c b_{eff} a$$

$$\Rightarrow a = \frac{438}{0.85(4)(87)} = 1.48'' \Rightarrow Y_2 = 6.25 - \frac{1.48}{2} = 5.51''$$

From Table 3-19,

$$\textcircled{a} \sum Q_n = 388, Y_2 = 5.5 \Rightarrow \phi M_n = 501 \text{ k-ft}$$

Live Load Reduction

$$K_{LL} = 2$$

$$A_T = 8(29) = 232 \text{ ft}^2$$

$$L = 100 \times \left| \frac{0.5}{\max(0.25 + \frac{15}{\sqrt{2 \times 232}})} \right| = 0.946 = 94.6 \text{ psf}$$

Load Combos

$$1.4D = 92.4 \text{ psf}$$

$$1.2D + 1.6L = 1.2(66) + 1.6(94.6) = 230.6 \text{ psf}$$

$$W = (230.6 \text{ psf})(8') = 1.84 \text{ klf}$$

$$M_u = \frac{Wl^2}{8} = \frac{(1.84)(29)^2}{8} = 194 \text{ k-ft} < \phi M_n = 501 \text{ kft} \therefore \text{OK}$$

- Check Wet Concrete Deflection:

$$W_w = 46(8) + 35 = .403 \text{ plf}$$

$$\Delta_{wc} = \frac{5(0.403)(29)^4(1728)}{384(29000)(510)} = 0.434''$$

$$\Delta_{wc \max} = \frac{29(12)}{360} = .97'' > 0.403 \therefore \text{OK}$$

- Check LL Deflection:

$$W_u = (94.6 \text{ psf})(8') = 756.8 \text{ plf}$$

$$I_{LB}: \quad \sum Q_n = 438 \text{ k}$$

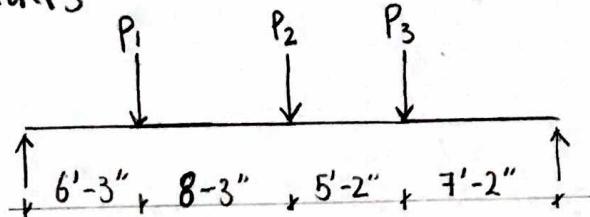
$$Y_2 = 5.51''$$

From Table 3-20,

$$\textcircled{a} \quad \sum n = 388 \text{ k}, Y_2 = 5.5 \Rightarrow I_{LB} = 1420 \text{ in}^4 \text{ (conservative)}$$

$$\Delta_L = \frac{5(.757)(29)^4(1728)}{384(29000)(1420)} = 0.293 < 0.97 = \frac{L}{360}$$

3) Girders



$$L = 26.83'$$

$$S = (29 + 8.5)/2 = 18.75'$$

LL Reduction

$$L = 100 \times \max \left| \frac{0.5}{0.25 + \frac{15}{\sqrt{2(26.83 \times 18.75)}}} \right| = .723 = 72.3 \text{ psf}$$

$$P_1: P_0 = (66 \text{ psf})(8.25')(16/2) = 4.36 \text{ k}$$

$$P_L = (72.3 \text{ psf})(8.25')(16/2) = 4.77 \text{ k}$$

$$1.4D = 1.4(4.36) = 6.1 \text{ k}$$

$$1.2D + 1.6L = 1.2(4.36) + 1.6(4.77) = 12.9 \text{ k} \leftarrow$$

$$P_2: P_0 = (66 \text{ psf})(6.71)(18.75) = 8.3 \text{ k}$$

$$P_L = (72.3 \text{ psf})(6.71)(18.75) = 9.1 \text{ k}$$

$$1.2D + 1.6L = 1.2(8.3) + 1.6(9.1) = 24.5 \text{ k}$$

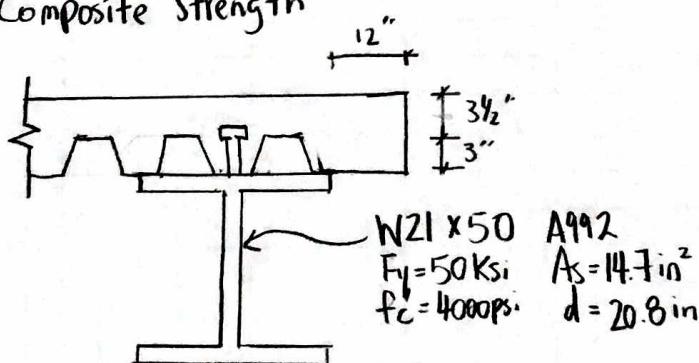
$$P_3: P_0 = (66 \text{ psf})(6.17)(18.75) = 7.6 \text{ k}$$

$$P_L = (72.3 \text{ psf})(6.17)(18.75) = 8.4 \text{ k}$$

$$1.2D + 1.6L = 1.2(7.6) + 1.6(8.4) = 22.6 \text{ k}$$

From SAP2000 Report, $M_u = 293.1 \text{ k}\cdot\text{ft}$ (Page 7)

- Composite Strength



$$b_{eff} = \min \left| \frac{26.83(12)/8}{12''} \right| + \left| \frac{26.83(12)/8}{18.75(12)} \right| = 52.2'$$

edge distance

Notebook B Existing Typical Bay

$$\# \text{ of Stud} = 29 \times 2 = 58$$

$$\text{Spacing} = 26.83' / 58 = 0.462' = 5.55'' < \left| \begin{array}{l} 8(6.25) = 50'' \\ 36'' \end{array} \right\} \therefore \text{OK}$$

$3/4'' \phi$ studs, 1 stud/rib, light concrete, $f'_c = 11000 \text{ psi}$
 $\Rightarrow Q_n = 17.2 \text{ K}$ (Table 3-21)

$$\sum Q_n = 29 (17.2) = 499 \text{ K}$$

$$\left. \begin{array}{l} V_{c\max} = 0.85(4)(52.2)(3.25) = 577 \text{ K} \\ V_{s\max} = 735 \text{ K} \quad (\text{Table 3-19}) \end{array} \right\} > 499 \text{ K}$$

\therefore Partially Composite

$$\sum Q_n = 0.85 f'_c b_{eff} a$$

$$a = \frac{499}{0.85(4)(52.2)} = 2.81''$$

$$Y_2 = 6.25 - 2.81 = 3.44''$$

From Table 3-19,

$$\textcircled{a} \sum Q_n = 473 \text{ K } Y_2 = 3'' \Rightarrow \phi M_n = 676 \text{ K-ft} > 293.1 \text{ K-ft} \therefore \text{OK}$$

- Unshored Strength

$$\begin{aligned} P_1: \quad P_o &= 4.36 \text{ K} \\ P_L &= (20)(8.25')(1^b/2) = 1.32 \text{ K} \end{aligned}$$

$$\begin{aligned} 1.4D &= 1.4 \times 4.36 = 6.1 \text{ K} \\ 1.2D + 1.6L &= 7.3 \text{ K} \end{aligned}$$

$$\begin{aligned} P_2: \quad P_o &= 8.3 \text{ K} \\ P_L &= (20)(6.71)(18.75) = 2.5 \text{ K} \end{aligned}$$

$$\begin{aligned} 1.4D &= 1.4 \times 8.3 = 11.6 \text{ K} \\ 1.2D + 1.6L &= 1.2(8.3) + 1.6(2.5) = 14 \text{ K} \end{aligned}$$

$$\begin{aligned} P_3: \quad P_o &= 7.6 \text{ K} \\ P_L &= (20)(6.7)(18.75) = 2.3 \text{ K} \end{aligned}$$

$$\begin{aligned} 1.4D &= 1.4 \times 7.6 = 10.6 \text{ K} \\ 1.2D + 1.6L &= 1.2(7.6) + 1.6(2.3) = 12.8 \text{ K} \end{aligned}$$

From SAP2000 Report, $M_n = 169 \text{ K-ft}$ (Page 9)

From Table 3-2,

$$W21 \times 50: \phi M_p = 413 \text{ k-ft} > 169 \text{ k-ft} \therefore \text{OK}$$

- Check Wet Concrete Deflection

From SAP2000 Report, $\Delta = 0.0364' = 0.437''$ (Page 10)

$$\frac{26.83(12)}{360} = .894 > 0.437 \therefore \text{OK}$$

- Check LL Deflection

From SAP2000 Report, (Page 8)

$$\Delta = 0.0381' = .457 < .894 \therefore \text{OK}$$

* The value from SAP2000 Report is the deflection for noncomposite beam, which is more conservative.

Table: Element Forces - Frames

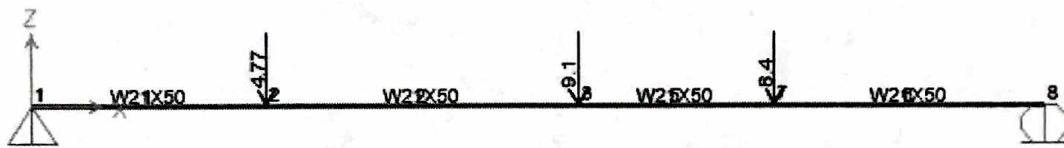
Table: Element Forces - Frames, Part 2 of 2

Frame	Station ft	OutputCase	M3	FrameElem	ElemStation ft
			Kip-ft		
1	0	LIVE	-5.684E-14	1-1	0
1	1.5625	LIVE	15.7579	1-1	1.5625
1	3.125	LIVE	31.5158	1-1	3.125
1	4.6875	LIVE	47.2737	1-1	4.6875
1	6.25	LIVE	63.0316	1-1	6.25
1	0	1.2D+1.6L	1.137E-13	1-1	0
1	1.5625	1.2D+1.6L	43.6275	1-1	1.5625
1	3.125	1.2D+1.6L	87.1084	1-1	3.125
1	4.6875	1.2D+1.6L	130.4427	1-1	4.6875
1	6.25	1.2D+1.6L	173.6305	1-1	6.25
2	0	LIVE	63.0316	2-1	0
2	1.65	LIVE	71.8015	2-1	1.65
2	3.3	LIVE	80.5713	2-1	3.3
2	4.95	LIVE	89.3412	2-1	4.95
2	6.6	LIVE	98.111	2-1	6.6
2	8.25	LIVE	106.8809	2-1	8.25
2	0	1.2D+1.6L	173.6305	2-1	0
2	1.65	1.2D+1.6L	197.8522	2-1	1.65
2	3.3	1.2D+1.6L	221.9104	2-1	3.3
2	4.95	1.2D+1.6L	245.8052	2-1	4.95
2	6.6	1.2D+1.6L	269.5366	2-1	6.6
2	8.25	1.2D+1.6L	293.1046	2-1	8.25
5	0	LIVE	106.8809	5-1	0
5	1.7222	LIVE	100.3624	5-1	1.7222
5	3.4444	LIVE	93.8439	5-1	3.4444
5	5.1667	LIVE	87.3254	5-1	5.1667
5	0	1.2D+1.6L	293.1046	5-1	0
5	1.7222	1.2D+1.6L	275.301	5-1	1.7222
5	3.4444	1.2D+1.6L	257.3194	5-1	3.4444
5	5.1667	1.2D+1.6L	239.1597	5-1	5.1667
6	0	LIVE	87.3254	6-1	0
6	1.7917	LIVE	65.494	6-1	1.7917
6	3.5833	LIVE	43.6627	6-1	3.5833
6	5.375	LIVE	21.8313	6-1	5.375
6	7.1667	LIVE	-2.132E-14	6-1	7.1667
6	0	1.2D+1.6L	239.1597	6-1	0
6	1.7917	1.2D+1.6L	179.6588	6-1	1.7917
6	3.5833	1.2D+1.6L	119.9652	6-1	3.5833
6	5.375	1.2D+1.6L	60.0789	6-1	5.375
6	7.1667	1.2D+1.6L	1.189E-14	6-1	7.1667

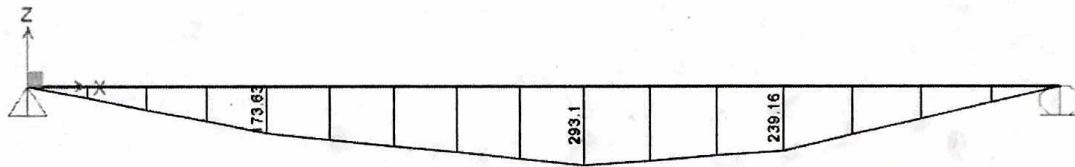
Table: Joint Displacements

Table: Joint Displacements

Joint	OutputCase	CaseType	U1 ft	U2 ft	U3 ft	R1 Radians	R2 Radians	R3 Radians
1	LIVE	LinStatic	0	0	0	0	0.004171	0
1	1.2D+1.6L	LinStatic	0	0	0	0	0.011475	0
2	LIVE	LinStatic	0	0	-0.024716	0	0.003177	0
2	1.2D+1.6L	LinStatic	0	0	-0.067964	0	0.008731	0
3	LIVE	LinStatic	0	0	-0.038093	0	-0.000359	0
3	1.2D+1.6L	LinStatic	0	0	-0.104632	0	-0.000999	0
7	LIVE	LinStatic	0	0	-0.029254	0	-0.002891	0
7	1.2D+1.6L	LinStatic	0	0	-0.080319	0	-0.007941	0
8	LIVE	LinStatic	0	0	0	0	-0.00447	0
8	1.2D+1.6L	LinStatic	0	0	0	0	-0.012275	0



Live Load (Live Load Deflection Check)



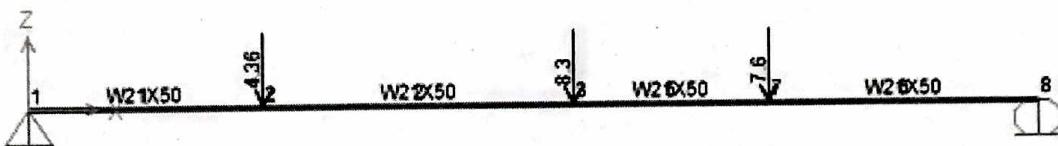
Moment Diagram (1.2D + 1.6L Composite Strength Check)

Table: Element Forces - Frames, Part 2 of 2**Table: Element Forces - Frames, Part 2 of 2**

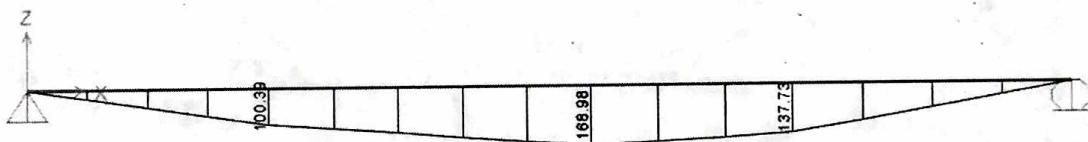
Frame	Station ft	OutputCase	M3 Kip-ft	FrameElem	ElemStation ft
1	0	DEAD	5.684E-14	1-1	0
1	1.5625	DEAD	15.3457	1-1	1.5625
1	3.125	DEAD	30.5692	1-1	3.125
1	4.6875	DEAD	45.6706	1-1	4.6875
1	6.25	DEAD	60.6499	1-1	6.25
1	0	1.2D+1.6L	0	1-1	0
1	1.5625	1.2D+1.6L	25.3162	1-1	1.5625
1	3.125	1.2D+1.6L	50.4858	1-1	3.125
1	4.6875	1.2D+1.6L	75.509	1-1	4.6875
1	6.25	1.2D+1.6L	100.3855	1-1	6.25
2	0	DEAD	60.6499	2-1	0
2	1.65	DEAD	69.1415	2-1	1.65
2	3.3	DEAD	77.4969	2-1	3.3
2	4.95	DEAD	85.7161	2-1	4.95
2	6.6	DEAD	93.7991	2-1	6.6
2	8.25	DEAD	101.746	2-1	8.25
2	0	1.2D+1.6L	100.3855	2-1	0
2	1.65	1.2D+1.6L	114.4313	2-1	1.65
2	3.3	1.2D+1.6L	128.3136	2-1	3.3
2	4.95	1.2D+1.6L	142.0326	2-1	4.95
2	6.6	1.2D+1.6L	155.5881	2-1	6.6
2	8.25	1.2D+1.6L	168.9802	2-1	8.25
5	0	DEAD	101.746	5-1	0
5	1.7222	DEAD	95.601	5-1	1.7222
5	3.4444	DEAD	89.3076	5-1	3.4444
5	5.1667	DEAD	82.8659	5-1	5.1667
5	0	1.2D+1.6L	168.9802	5-1	0
5	1.7222	1.2D+1.6L	158.7419	5-1	1.7222
5	3.4444	1.2D+1.6L	148.3256	5-1	3.4444
5	5.1667	1.2D+1.6L	137.7313	5-1	5.1667
6	0	DEAD	82.8659	6-1	0
6	1.7917	DEAD	62.3903	6-1	1.7917
6	3.5833	DEAD	41.7541	6-1	3.5833
6	5.375	DEAD	20.9573	6-1	5.375
6	7.1667	DEAD	1.939E-14	6-1	7.1667
6	0	1.2D+1.6L	137.7313	6-1	0
6	1.7917	1.2D+1.6L	103.5875	6-1	1.7917
6	3.5833	1.2D+1.6L	69.251	6-1	3.5833
6	5.375	1.2D+1.6L	34.7219	6-1	5.375
6	7.1667	1.2D+1.6L	-9.423E-15	6-1	7.1667

Table: Joint Displacements**Table: Joint Displacements**

Joint	OutputCase	CaseType	U1 ft	U2 ft	U3 ft	R1 Radians	R2 Radians	R3 Radians
1	DEAD	LinStatic	0	0	0	0	0.004	0
1	1.2D+1.6L	LinStatic	0	0	0	0	0.006629	0
2	DEAD	LinStatic	0	0	-0.023682	0	0.003039	0
2	1.2D+1.6L	LinStatic	0	0	-0.039254	0	0.00504	0
3	DEAD	LinStatic	0	0	-0.036403	0	-0.000353	0
3	1.2D+1.6L	LinStatic	0	0	-0.060387	0	-0.000581	0
7	DEAD	LinStatic	0	0	-0.027927	0	-0.002763	0
7	1.2D+1.6L	LinStatic	0	0	-0.04634	0	-0.004583	0
8	DEAD	LinStatic	0	0	0	0	-0.004269	0
8	1.2D+1.6L	LinStatic	0	0	0	0	-0.007083	0



Dead Load (Wet Concrete Deflection Check)

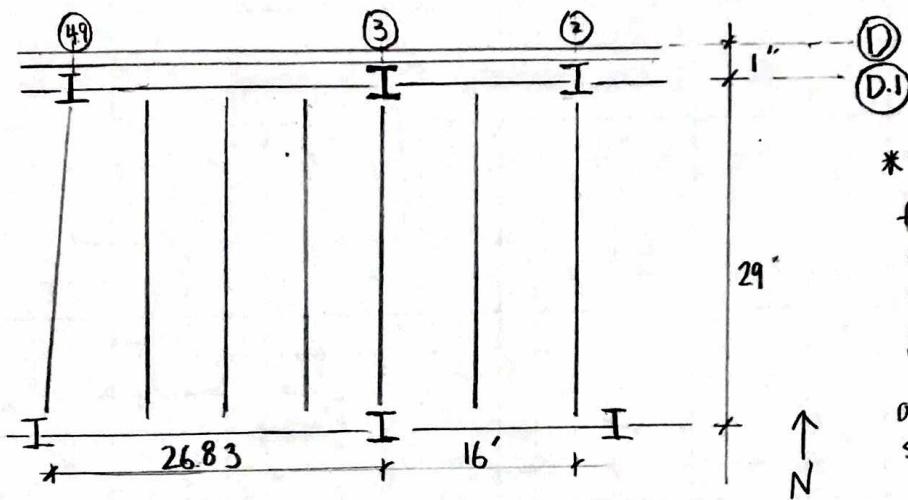


Moment Diagram (1.2D+1.6L Unshored Strength Check)

Column D.1-3

2 lower storage levels: W12x65
 Level 1-3: W12x65
 Level 3-5: W12x45
 Level 5-Roof: W12x40

Column Loads



* Typical floor plane for Level 1, 3, 4 and R. Cellar, level 2, 5 are different because of the different floor layouts & Opening. (Influence area noted in Excel spread sheet..)

$$\text{Tributary Area} = \left(\frac{26.83' + 16'}{2} \right) \left(\frac{29}{2} + 1' \right) = 332 \text{ ft}^2$$

$$\text{Influence Area} = (26.83' + 16') (29 + 1') = 1285 \text{ ft}^2$$

$$\text{Total Influence Area for the column: } \sum A = 6104 \text{ ft}^2$$

$$\text{LL Reduction} = \min \left\{ 0.4, 0.25 + \frac{15}{\sqrt{6104}} \right\} = 0.44$$

→ See following excel. Note that roof (LL=20psf) and cellar floor (LL=125 psf) are not reduced.

Snow loads don't control for this column.

Selfweights are included in dead load.

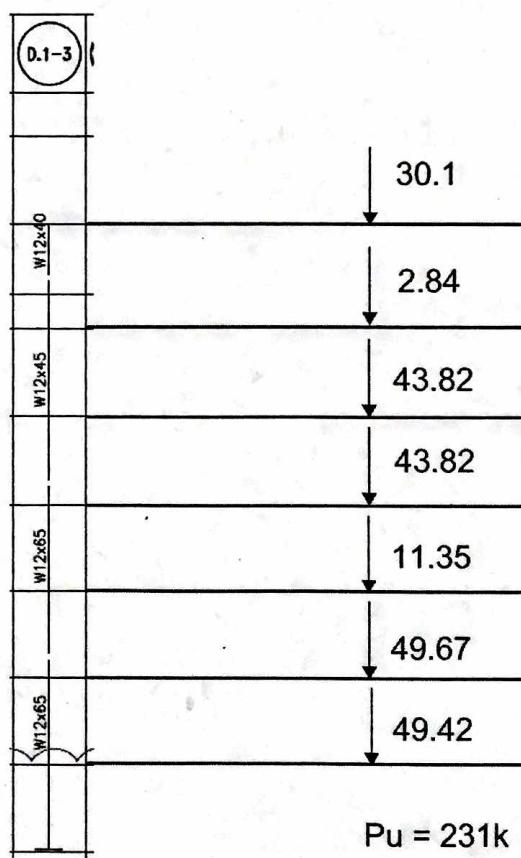
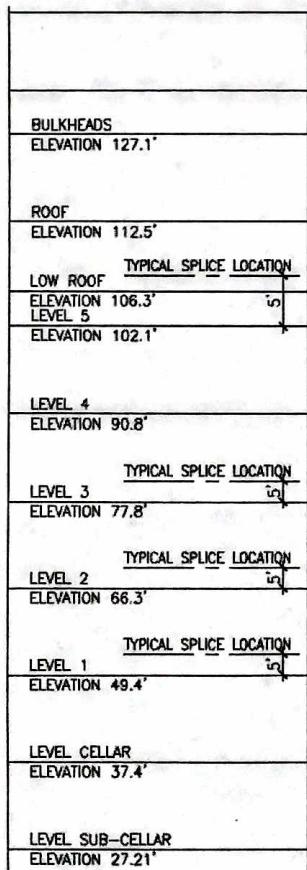
Controlling Case = 1.2D + 1.6L + 0.5LR

$$P_n = 231 \text{ k}$$

From Table 4-1, W12x65 @ 24": $\phi P_n = 442 \text{ k} > 231 \text{ k}; \text{OK}$

Interior Column D.1-3

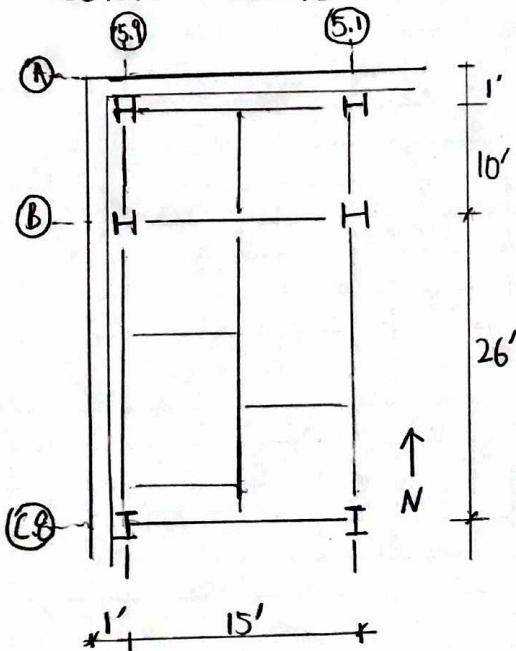
Level	Dead (psf)	Live (psf)	Tributary Area (ft^2)	Influence Area (ft^2)	Red. Live (psf)	Total Axial Load (K)		
						Dead	L or Lr	1.2D+1.6L+.5Lr
Roof	67	20	332	1285	20	22.24	6.64	30.01
Level 5	66	75	21.5	43	33	1.42	0.71	2.84
Level 4	66	75	332	1285	33	21.91	10.96	43.82
Level 3	66	75	332	1285	33	21.91	10.96	43.82
Level 2	66	75	86	300	33	5.68	2.84	11.35
Level 1	66	100	332	1285	44	21.91	14.61	49.67
Cellar	66	125	177	621	125	11.68	22.13	49.42
Σ Area:				6104		Pu:		230.94



Column B-5.9

Level 1-3 : W12 X 65
 Level 3-5 : W10 X 60
 Level 5-R : W10 X 33

Column loads



$$\text{Tributary Area} = \left(\frac{10+26}{2}\right)\left(\frac{15}{2} + 1\right) = 153 \text{ ft}^2$$

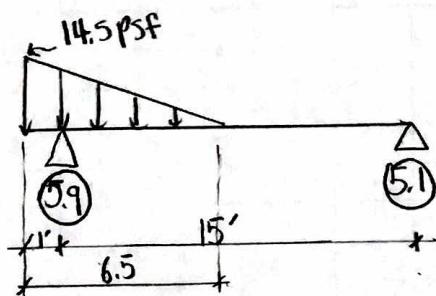
Influence Area:

$$= (10+26)(15+1) = 576 \text{ ft}^2$$

LL Reduction:

$$= \begin{cases} 0.4 \\ \min(0.25 + \frac{15}{\sqrt{4(576)}}) \end{cases} = .56$$

Column Snow Drift Loads:



$$\sum M(5.1) : \frac{[14.5(6.5)/2](15 - 6.5/3)}{15} = R_A$$

$$R_{5.1} = 36.9 \text{ Klf}$$

- Snow load control for this column

$$\text{Exterior Wall length} = (10 + 26)/2 = 18'$$

$$\text{Snow} = 20 \text{ PSF}$$

$$\text{Roof DL} = 67 \text{ PSF}$$

$$\text{Roof Exterior Wall load} = 696 \text{ PI f}$$

$$\text{Roof Drift Load} = 36.9 \text{ Klf}$$

$$\text{Floor DL} = 66 \text{ PSF}$$

$$\text{Floor Ext Wall load} = \begin{cases} \text{Level 5} = 1056 \text{ PI f} \\ \text{Level 4} = 1176 \text{ PI f} \\ \text{Level 3} = 1176 \text{ PI f} \\ \text{Level 2} = 1368 \text{ PI f} \end{cases}$$

$$\text{Floor LL} = 75 \text{ PSF}$$

$$\text{Roof DL} = [67(153) + 696(18)]/1000 = 22.8^k$$

$$\text{Roof SL} = [20(153) + 36.9(18)]/1000 = 3.72^k$$

Floor DL:

$$\text{Level 5: } [66(153) + 1056(18)]/1000 = 29.1^k$$

$$\text{Level 3\&4: } [66(153) + 1176(18)]/1000 = 31.3^k$$

$$\text{Level 2: } [66(153) + 1368(18)]/1000 = 34.7^k$$

Floor LL:

$$= 75(0.56)(153)/1000 = 6.4^k/\text{floor}$$

Total Load: $1.2D + 1.6L + 0.5S$ (control)

$$\text{Roof: } 1.2(22.8) + 0.5(3.72) = 29.2^k$$

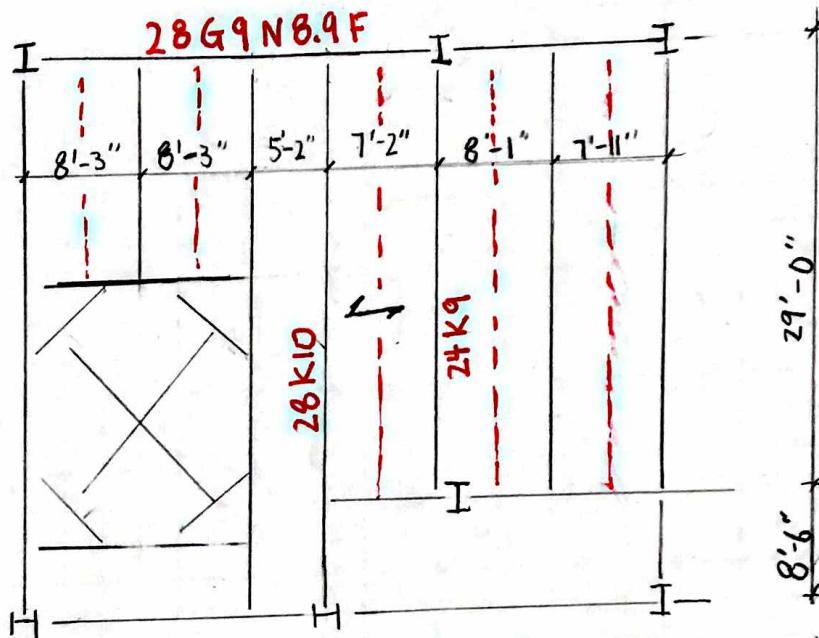
$$\begin{aligned} \text{Floor: } & 1.2(29.1 + 31.3 + 31.3 + 34.7) + 1.6(4 \times 6.4) \\ & = 162^k \end{aligned}$$

$$P_u = 29.2 + 162 = 191.2^k$$

From Table 4-1.

$$W12 \times 65 @ 28": \phi P_n = 348^k > 191.2^k \therefore \text{OK}$$

Alternative 1: Non-composite Steel Joist



1) Non-composite Decking.

Deck Span: 10'-1" (MAX) continuous over 3 spans.

Live Load: 100 psf

Misc PL: 10 psf

Try 3C 18 deck with 3" LWC Topping (Valcraft catalog)

- Max 3 span: 15-2" > 10'-1" ∴ OK

- Weight: 44 psf

Check 18GA for total load: $F_b = 36000 \text{ psf}$

$$\text{Total load} = 100 + 44 + 10 = 154 \text{ psf} < 175 \text{ psf} @ 10'-6" \\ \therefore \text{OK}$$

Check 18GA for LL: Deflection = $L/240$

$$\text{LL} = 134 \text{ psf} > 100 \text{ psf}$$

2) Joists

3.0C18 Deck w 3" LW Conc.

Slab/deck	44 PSF
Joists	3 PSF
misc DL	10 PSF
LL	100 PSF

$$\text{LL. Reduction: } L = 100 \times \max \left| \frac{0.5}{0.25 + \frac{15}{\sqrt{16 \times 29}}} \right| = .95 = 95 \text{ PSF}$$

$$W_u = [1.2(44+3+10) + 1.6(95)](8') = 1763 \text{ Plf}$$

- * No applicable steel joists can carry this much load
⇒ decrease spacing to 4'

• Recheck Decking

SPAN = 4' continuous over 3 spans

Try 1.0C20 deck with 2" LWL Topping

- Max 3 span: 8'-5" > 4' ∴ OK
- Weight: 25 PSF

check 20 GA for total load: $F_b = 36000 \text{ PSF}$

$$\text{Total load} = 100 + 25 + 10 = 135 \text{ PSF} < 242 \text{ PSF} @ 4'-6" \therefore \text{OK}$$

check 20 GA for LL: deflection = $\frac{L}{240}$

$$\text{LL} = 119 \text{ PSF} > 100 \text{ PSF} \therefore \text{OK}$$

• Recheck Joists not enough area for LL. Red.

$$W_{u1} = [1.2(25+3+10) + 1.6(100)](4) = 822 \text{ Plf}$$

$$W_{t1} = (38 + 100)(4) = 552 \text{ Plf}$$

$$\Delta_{tl} \leq L/240$$

From standard table : SJI p 54

24K9 : (d=24", Wt = 103 lbs/ft)

$$W_{utl} = 825 \text{ Plf} > 790 \text{ Plf} \therefore \text{OK}$$

$$W_{for} L/360 = 436 \text{ Plf}$$

$$W_{for} L/240 = 436 \times 1.5 = 654 \text{ Plf} > 552 \text{ Plf} \therefore \text{OK}$$

$$10.3 \text{ Plf}/4' = 2.58 \text{ PSF} < 3 \text{ PSF} \therefore \text{allowable OK}$$

3) Joist Girders

Since the girder and the joist are not in the same bay, so determine the joist in the girder's bay

• Joist : Span = 29 + 8.5 = 37.5'

change spacing to 3"

$$W_{utl} = [1.2(38) + 1.6(100)](3') = 617 \text{ Plf}$$

$$W_{for} = (38 + 100)(3) = 414 \text{ Plf}$$

From standard table : SJI p

28K10 : (d=28", Wt = 11.8 Plf)

$$W_{utl} = 691 \text{ Plf} > 617 \text{ Plf} \therefore \text{OK}$$

$$W_{for} L/360 = 325 \text{ PSF}$$

$$W_{for} L/240 = 325 \times 1.5 = 488 \text{ Plf} > 414 \text{ Plf} \therefore \text{OK}$$

$$11.8'/3 = 3.9' \approx 3' \therefore \text{OK}$$

• Girder : Span = 26.83', $26.83/3 = 9$ spaces (From ECO Girder Table)

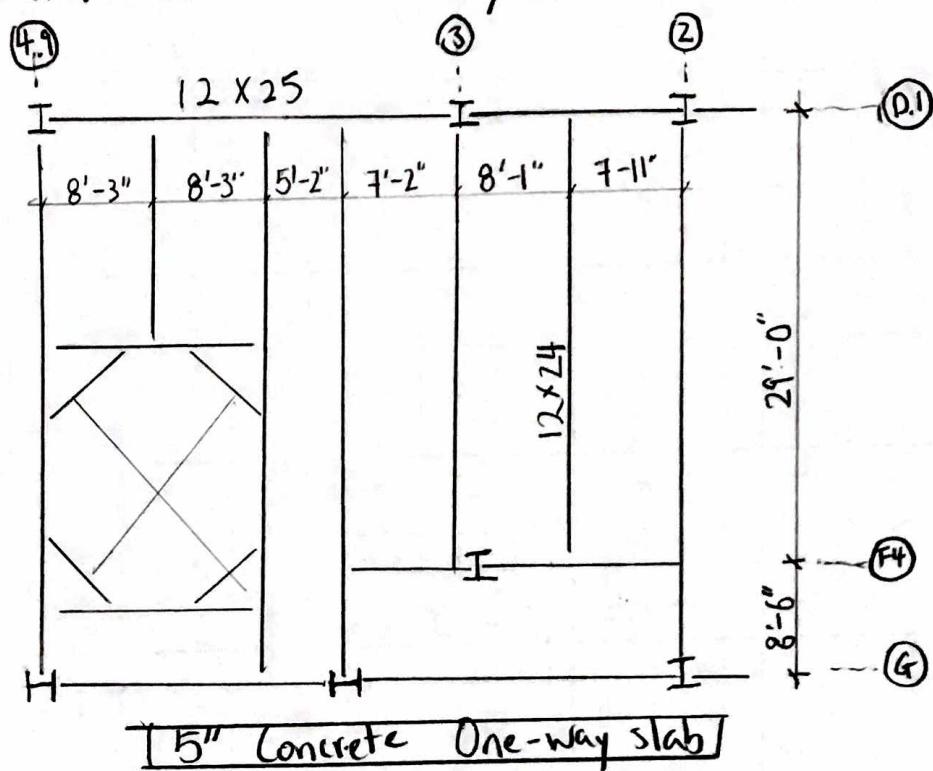
$$P_n = 617 \left(\frac{29}{2}\right) / 1000 = 8.9^k$$

\Rightarrow Use 28G9N8.9F \Rightarrow weights approx. 27.5 Plf

\therefore Use I.O C20 deck with 2" LWL Topping
24K9 @ 4"; 28K10 @ 3"
28G9N8.9F

(See graph)

Alternative 2: One-Way Slab



1) One-way Slab

$$f'_c = 4000 \text{ psi}, f_y = 60 \text{ ksi}$$

Finishes: 2 psf
 Superimposed DL: 10 psf
 LL Red.: 94.6 psf (From previous calcs.)

Estimate Slab thickness:

$$\text{Interior bay} \Rightarrow \frac{l}{28} = \frac{8 \times 12}{28} = 3.4 \quad (\text{Table 7.3.1.1 ACI})$$

use 5" slab (fire-rating 2 hrs)

Load calculation:

$$DL = \frac{5}{12} \times 150 + 10 + 2 = 74.5 \text{ psf}$$

$$LL = 94.6 \text{ psf}$$

- Load Combinations

$$1.4D = 1.4(74.5) = 104.3 \text{ psf}$$

$$1.2D + 1.6L = 1.2(74.5) + 1.6(94.6 \text{ psf}) = 241 \text{ psf} \leftarrow \text{controls}$$

1' width of slab

$$W_n = 241 \text{ Pif}$$

- Max. Moment

$$M_u = \frac{W_n l n^2}{10} = \frac{(241)(8)^2}{10} = 1.54 \text{ k-ft / ft}$$

- Calculate Reinforcement Required (A_s)

$$R = \frac{M_u}{\phi b d^2} = \frac{(1.54 \text{ k-ft})(12)}{0.9(12)(5)^2} = .068$$

$$\beta = \frac{0.85(4)}{60} \left[1 - \sqrt{1 - \frac{2(0.068)}{0.85(4)}} \right] = 0.00113$$

$$A_s = 0.00113 \times 12'' \times 5'' = 0.069 \text{ in}^2/\text{ft}$$

$$A_{smin} = 0.0018 \times 12'' \times 5'' = 0.11 \text{ in}^2/\text{ft} > 0.069 \text{ in}^2/\text{ft}$$

$$\therefore A_s = A_{smin} = 0.11 \text{ in}^2/\text{ft} \quad (\text{ACI Sec 7.6.1.1})$$

$$\therefore \text{Use #3 bars with } 12'' \Rightarrow A_s = 0.11 \text{ in}^2/\text{ft}$$

- Moment Capacity

Assume $c = 0.75''$

$$d = 5'' - 0.75'' - \frac{0.375''}{2} = 4.06''$$

$$a = \frac{A_s f_y}{0.85 f_{ib}} = \frac{(0.11 \text{ in}^2)(60)}{0.85(4)(12 \text{ in})} = 0.162''$$

$$\phi M_n = \phi f_y A_s \left(d - \frac{a}{2} \right)$$

$$= 0.9(60)(0.11)(4.06 - \frac{0.162}{2})$$

$$= 23.6 \text{ k-in} = 1.97 \text{ k-ft}$$

$$1.97 \text{ k-ft} > 1.54 \text{ k-ft} \quad \underline{\text{OK}}$$

Notebook B One-Way Slab

43

- Shear Capacity (One-way shear)

$$V_u = \frac{1.15 W_{uln}}{2} = \frac{1.15(241 \text{ Pf})(8')}{2} = 1108.6 \text{ lbs/ft}$$

$$V_c = 2\sqrt{f_c b w d} = 2\sqrt{4060} \times 12 \times (4.06) = 6163 \text{ lbs/ft} \quad (\text{ACI 22.5.5.1})$$

$$\phi V_c = 0.75 \times 6163 = 4622 \text{ lbs/ft} > 1108.6 \text{ lbs/ft} \quad \underline{\text{OK}}$$

- Max. Spacing

$$S_{max} = 3t = 3 \times 5 = 15'' < 18'' \text{ use } 15'' \quad (\text{ACI 7.7.2.3})$$

$$S_{max} = \begin{cases} 15 \left(\frac{40}{73(60)} \right) - 2.5 \times 0.75 = 13.13'' & (\text{ACI Table 24.3.2}) \\ (\text{for crack control}) \quad 12 \left(\frac{40}{73(60)} \right) = 12'' \end{cases}$$

$$\therefore \text{actual } S_{max} = 12''$$

\Rightarrow use #3 w/ 12" spacing is OK.

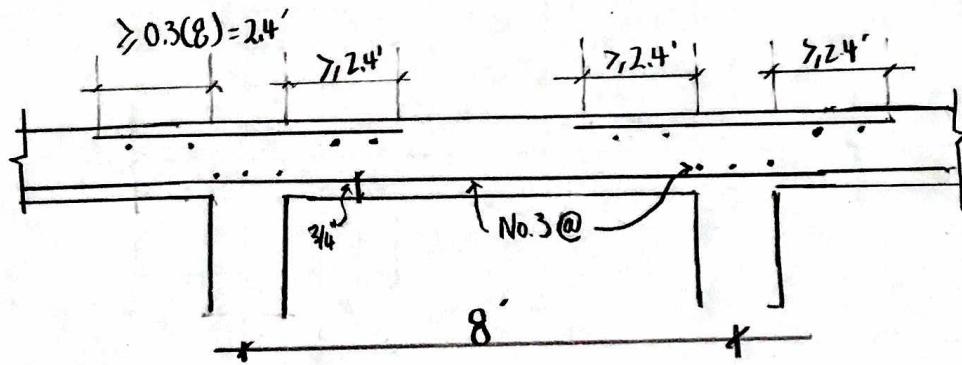
- Transverse Reinforcement (S&T)

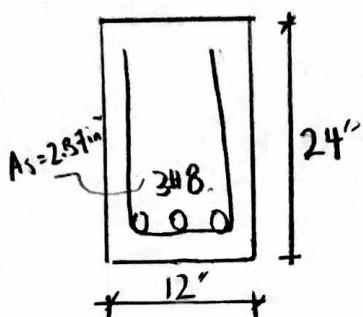
$$A_s (\text{S&T}) = 0.0018 \times 12 \times 5 = 0.11 \text{ ft}^2 / \text{ft}$$

$$S_{max} = 5t = 5 \times 5 = 25'' > 18'' \quad (\text{ACI 7.7.2.4})$$

\therefore use #3 @ 12" $\Rightarrow A_s = 0.13 \text{ ft}^2 > 0.11 \text{ ft}^2 \quad \underline{\text{OK}}$

- Draw the section





• Flexural Strength

$$a = \frac{A_s f_y}{0.85 f_i b} = \frac{(60)(237)}{0.85(4)(12)} = 3.49''$$

$$c = a/\beta_1 = 4.1''$$

$$\phi M_n = \phi A_s f_y (d - \frac{a}{2})$$

$$= 0.9(237)(60) \left(21.5 - \frac{3.49}{2}\right)$$

$$= 2528 \text{ K-in} = 211 \text{ K-ft} > M_u = 203 \text{ K-ft} \quad \text{OK}$$

Verify strain in steel

$$\epsilon_s = \left(\frac{d-L}{L}\right) \epsilon_{eu} = \left(\frac{21.5-4.1}{4.1}\right) * 0.003 = 0.0127 > 0.005$$

$$\Rightarrow \phi = 0.9$$

∴ Check Reg. Reinforcement (A_{smin})

$$A_{smin} = \frac{3\sqrt{f_i}}{f_y} b w d = \frac{3\sqrt{4000}}{60000} \times 12 \times 21.5 = .81 \text{ in}^2$$

$$\text{but not less than } \frac{200 b w d}{f_y} = \frac{200 \times 12 \times 21.5}{60000} = .86 \text{ in}^2$$

∴ A_{smin} is satisfied.

• Shear Strength

$$V_u = \frac{1.93 \times 29}{2} = 28.0^k$$

$$V_u @ d = 28 - \frac{28 \times 21.5}{14.5(12)} = 24.5^k$$

$$V_c = 2\lambda \sqrt{f_i} b w d = 2(1) \sqrt{4000} (12)(21.5) = 32.6^k$$

$$\phi V_c = 0.75(32.6) = 24.5^k \approx V_u$$

∴ Provide min. shear reinforcement.

2) Infill Beam

$$W_u = (241 \text{ psf})(8) = 1.93 \text{ klf}$$

$$M_u = \frac{(1.93)(29)^2}{8} = 203 \text{ k-ft}$$

• Calculate a tentative ρ

$$\rho = \frac{0.25 f'_c \beta_1}{f_y} = \frac{0.25(4)(0.85)}{60} = 0.0142$$

$$M_u = \frac{\mu_y}{\phi} = \frac{203 \text{ k}}{0.9} = 256 \text{ k-ft} \quad (\text{Assume } \phi = 0.9)$$

$$w = \frac{\rho f_y}{f'_c} = 0.0142 \times \frac{60}{4} = .213$$

$$R = w f'_c (1 - 0.59 w) \\ = .213(4)(1 - 0.59 \times 0.213) = 0.745 \text{ ksi}$$

$$M_u = R b d^2 \Rightarrow b d^2 = \frac{256 \times 12}{0.745} = 4123.5 \text{ in}^3$$

Potential configurations ($b \approx d/2$)

$$4123.5 = \left(\frac{d}{2}\right)(d)^2 \Rightarrow d = 20.2, h \approx d + 2.5 = 22.7''$$

Try 12 x 24 $d = 21.5'', c = 2.5''$

• Req. As

$$R = \frac{\mu_y}{\phi b d^2} = \frac{203 \times 12}{0.9(12)(21.5)^2} = .488 \text{ ksi}$$

$$\rho = \frac{0.85(4)}{(60)} \left[1 - \sqrt{1 - \frac{2(488)}{0.85(4)}} \right] = 0.0088$$

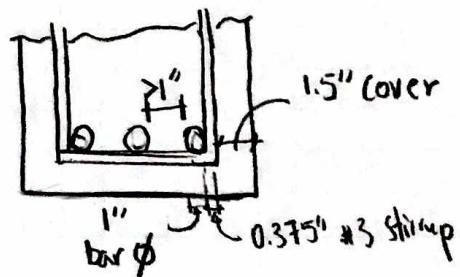
$$A_s = 0.0088 \times 12 \times 21.5 = 2.28 \text{ in}^2$$

$$\therefore \text{use } 3 \# 8 \quad A_s = 2.37 \text{ in}^2$$

• Minimum spacing of bars

$$5 + 2(0.375 + 1.5) = 8.75''$$

$$8.75'' > 12'' \quad \underline{\text{OK}}$$



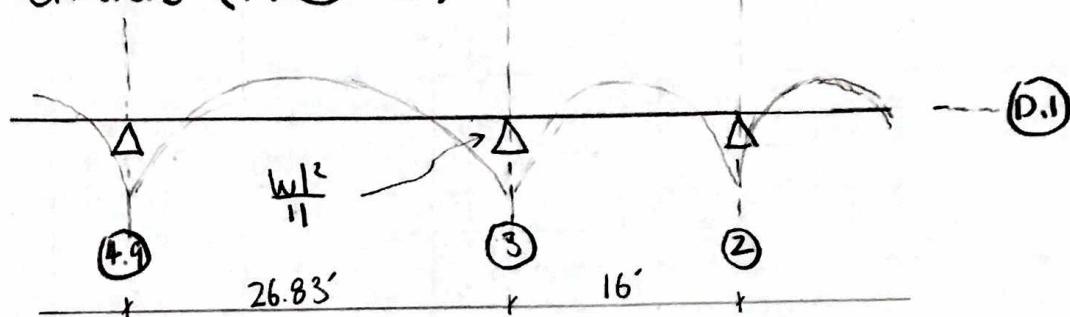
$$V_s \leq 4\sqrt{f'_c} bw \Rightarrow S_{max} \leq \frac{d}{2} = 10.75"$$

∴ use $S = 10"$

$$A_{smin} = \begin{cases} 0.75 \sqrt{f'_c} \frac{bwS}{f_y t} = \frac{0.75 \sqrt{4000} (12)(10)}{60000} = 0.95 \text{ in}^2 \\ 50 \frac{bwS}{f_y t} = \frac{50(12)(10)}{60000} = 0.1 \text{ in}^2 \end{cases}$$

∴ use #3 stirrups at 10" ✓

3) Girders (D1 ④.9 - ③)

Determine Girder trial Size

- Assumption: Uniform Load

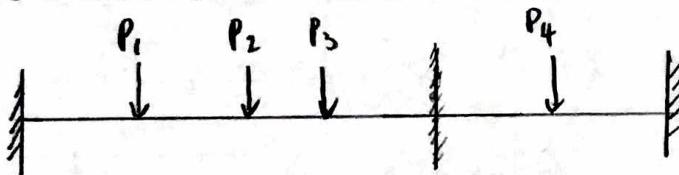
$$M_{\max} = \frac{wl^2}{10} \quad (\text{To be conservative})$$

$$\text{SPACING} = \frac{29}{2} + 1 = 15.5'$$

$$W_u = (241 \text{ PSF})(15.5) = 3.74 \text{ ksf}$$

$$M_u(③) = \frac{wl^2}{10} = \frac{(3.74)(26.8+16)/2]^2}{10} = 172 \text{ K-ft}$$

- Use SAP2000 to find M_u .



$$P_1 = (241 \text{ PSF})(8.25)(37.5)/2 = 37.3 \text{ k}$$

$$P_2 = (241 \text{ PSF})(6.71)(37.5)/2 = 30.3 \text{ k}$$

$$P_3 = (241 \text{ PSF})(6.17)(37.5)/2 = 27.9 \text{ k}$$

$$P_4 = (241 \text{ PSF})(8)(29)/2 = 28.0 \text{ k}$$

- From SAP2000 Data (in following pages): $M_u = 270.3 \text{ K-ft} > 172 \text{ K-ft}$

- $\therefore M_u = 270.3 \text{ K-ft}$ for design

- Use Simplified Design method:

$$bd^2 \approx 20 M_u \quad (b \approx l/2d)$$

$$(\frac{1}{2}d)(d) = 20(270.3) = 5406$$

$$\Rightarrow d = 22.1''$$

$$A_s \propto \frac{M_u}{4d}$$

$$\approx \frac{270.3}{4(22.5)}$$

$$\approx 3.0 \text{ in}^2$$

Design 12 X 25 in SAP2000 (See following data) ✓

Table: Element Forces - Frames, Part 1 of 2**Table: Element Forces - Frames, Part 1 of 2**

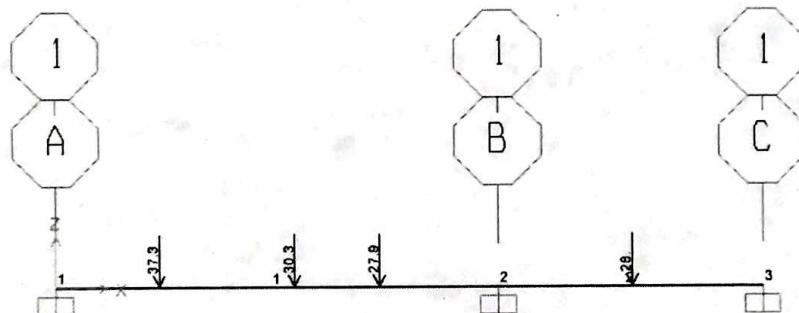
Frame	Station ft	OutputCase	CaseType	P Kip	V2 Kip	V3 Kip	T Kip-ft	M2 Kip-ft
1	0	DEAD	LinStatic	0	-50.683	0	0	0
1	1.5625	DEAD	LinStatic	0	-50.647	0	0	0
1	3.125	DEAD	LinStatic	0	-50.612	0	0	0
1	4.6875	DEAD	LinStatic	0	-50.577	0	0	0
1	6.25	DEAD	LinStatic	0	-50.542	0	0	0
1	6.25	DEAD	LinStatic	0	-13.242	0	0	0
1	7.9	DEAD	LinStatic	0	-13.205	0	0	0
1	9.55	DEAD	LinStatic	0	-13.168	0	0	0
1	11.2	DEAD	LinStatic	0	-13.131	0	0	0
1	12.85	DEAD	LinStatic	0	-13.094	0	0	0
1	14.5	DEAD	LinStatic	0	-13.056	0	0	0
1	14.5	DEAD	LinStatic	0	17.244	0	0	0
1	16.2233	DEAD	LinStatic	0	17.282	0	0	0
1	17.9467	DEAD	LinStatic	0	17.321	0	0	0
1	19.67	DEAD	LinStatic	0	17.36	0	0	0
1	19.67	DEAD	LinStatic	0	45.26	0	0	0
1	21.46	DEAD	LinStatic	0	45.3	0	0	0
1	23.25	DEAD	LinStatic	0	45.34	0	0	0
1	25.04	DEAD	LinStatic	0	45.381	0	0	0
1	26.83	DEAD	LinStatic	0	45.421	0	0	0
2	0	DEAD	LinStatic	0	-13.964	0	0	0
2	1.6167	DEAD	LinStatic	0	-13.928	0	0	0
2	3.2333	DEAD	LinStatic	0	-13.892	0	0	0
2	4.85	DEAD	LinStatic	0	-13.855	0	0	0
2	6.4667	DEAD	LinStatic	0	-13.819	0	0	0
2	8.0833	DEAD	LinStatic	0	-13.782	0	0	0
2	8.0833	DEAD	LinStatic	0	14.218	0	0	0
2	10.0625	DEAD	LinStatic	0	14.262	0	0	0
2	12.0417	DEAD	LinStatic	0	14.307	0	0	0
2	14.0208	DEAD	LinStatic	0	14.351	0	0	0
2	16	DEAD	LinStatic	0	14.396	0	0	0

Table: Element Forces - Frames, Part 2 of 2**Table: Element Forces - Frames, Part 2 of 2**

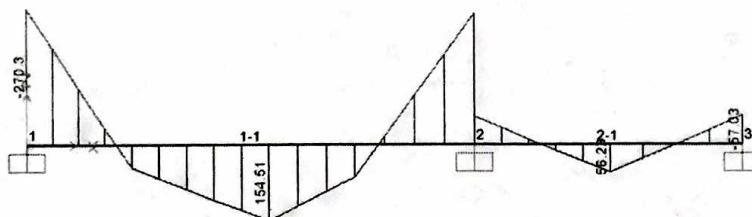
Frame	Station ft	OutputCase	M3 Kip-ft	FrameElem	ElemStation ft
1	0	DEAD	-270.3	1-1	0
1	1.5625	DEAD	-191.1359	1-1	1.5625
1	3.125	DEAD	-112.0268	1-1	3.125
1	4.6875	DEAD	-32.9725	1-1	4.6875
1	6.25	DEAD	46.0268	1-1	6.25
1	6.25	DEAD	46.0268	1-1	6.25
1	7.9	DEAD	67.8455	1-1	7.9
1	9.55	DEAD	89.6029	1-1	9.55
1	11.2	DEAD	111.2991	1-1	11.2
1	12.85	DEAD	132.9341	1-1	12.85
1	14.5	DEAD	154.5078	1-1	14.5
1	14.5	DEAD	154.5078	1-1	14.5

Table: Element Forces - Frames, Part 2 of 2

Frame	Station ft	OutputCase	M3 Kip-ft	FrameElem	ElemStation ft
1	16.2233	DEAD	124.758	1-1	16.2233
1	17.9467	DEAD	94.9414	1-1	17.9467
1	19.67	DEAD	65.058	1-1	19.67
1	19.67	DEAD	65.058	1-1	19.67
1	21.46	DEAD	-15.9932	1-1	21.46
1	23.25	DEAD	-97.1165	1-1	23.25
1	25.04	DEAD	-178.3118	1-1	25.04
1	26.83	DEAD	-259.5791	1-1	26.83
2	0	DEAD	-55.9154	2-1	0
2	1.6167	DEAD	-33.3691	2-1	1.6167
2	3.2333	DEAD	-10.8817	2-1	3.2333
2	4.85	DEAD	11.547	2-1	4.85
2	6.4667	DEAD	33.9169	2-1	6.4667
2	8.0833	DEAD	56.2279	2-1	8.0833
2	8.0833	DEAD	56.2279	2-1	8.0833
2	10.0625	DEAD	28.0451	2-1	10.0625
2	12.0417	DEAD	-0.2259	2-1	12.0417
2	14.0208	DEAD	-28.585	2-1	14.0208
2	16	DEAD	-57.0322	2-1	16



Loading Configuration



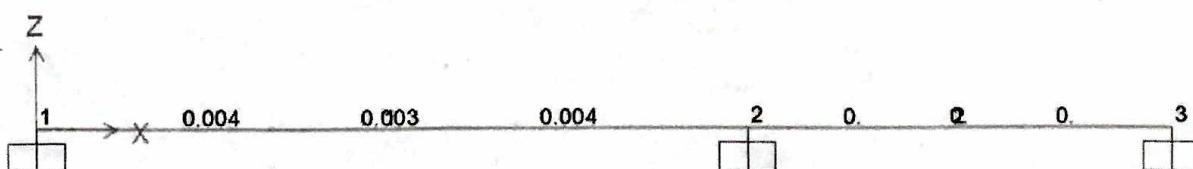
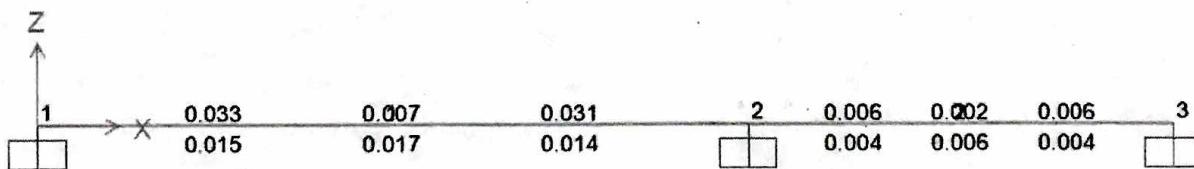
Moment Diagram



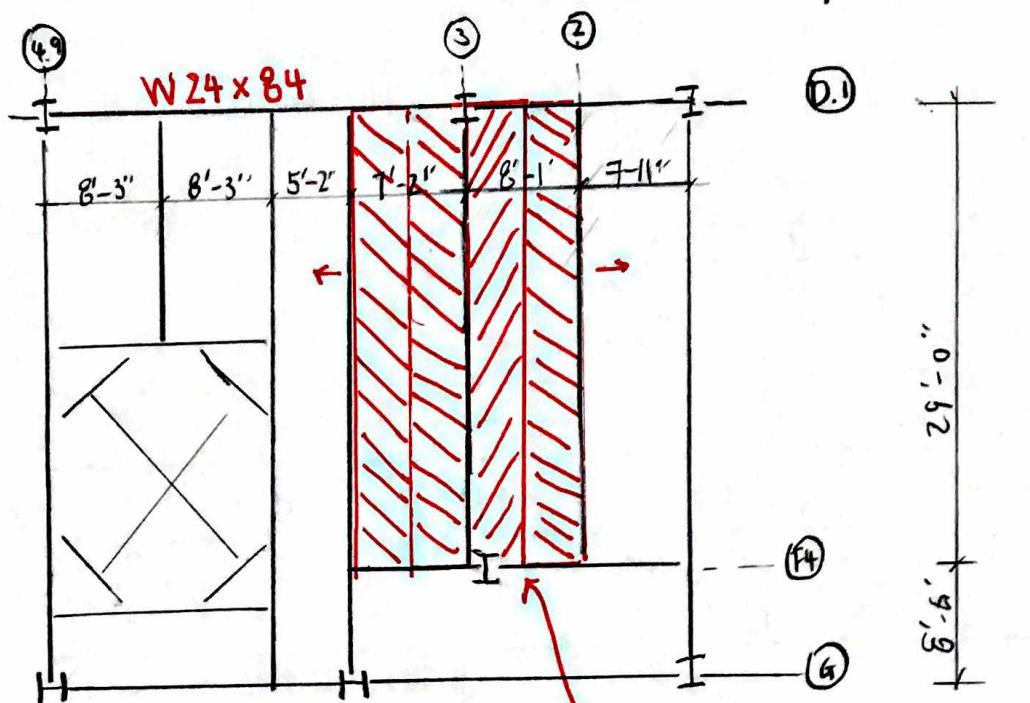
Deflection Shape

Girder Design:

Frame Text	DesignSect Text	DesignType Text	Status Text	Location ft	FTopCombo Text	FTopArea ft2	FBotCombo Text	FBotArea ft2	VCombo Text	VRebar ft2/ft
1	12 x 25	Beam	No Messages	0	DCON1	0.032637	DCON1 (Sp)	0.01485	DCON1	0.00418
1	12 x 25	Beam	No Messages	26.83	DCON1	0.031168	DCON1 (Sp)	0.014254	DCON1	0.00357
1	12 x 25	Beam	No Messages	1.5625	DCON1	0.021687	DCON1 (Sp)	0.007146	DCON1	0.00412
1	12 x 25	Beam	No Messages	25.04	DCON1	0.020063	DCON1 (Sp)	0.007146	DCON1	0.00351
1	12 x 25	Beam	No Messages	3.125	DCON1	0.012082	DCON1 (Sp)	0.007146	DCON1	0.00407
1	12 x 25	Beam	No Messages	23.25	DCON1	0.010341	DCON1 (Sp)	0.007146	DCON1	0.00344
1	12 x 25	Beam	No Messages	4.6875	DCON1 (Sp)	0.007146	DCON1 (Sp)	0.007146	DCON1	0.00401
1	12 x 25	Beam	No Messages	6.25	DCON1 (Sp)	0.007146	DCON1 (Sp)	0.007146	DCON1	0.00395
1	12 x 25	Beam	No Messages	6.25	DCON1 (Sp)	0.007146	DCON1 (Sp)	0.007146	DCON1	0
1	12 x 25	Beam	No Messages	7.9	DCON1 (Sp)	0.007146	DCON1	0.007169	DCON1	0
1	12 x 25	Beam	No Messages	9.55	DCON1 (Sp)	0.007146	DCON1	0.009671	DCON1	0
1	12 x 25	Beam	No Messages	11.2	DCON1 (Sp)	0.007146	DCON1	0.012149	DCON1	0
1	12 x 25	Beam	No Messages	12.85	DCON1 (Sp)	0.007146	DCON1	0.014598	DCON1	0
1	12 x 25	Beam	No Messages	14.5	DCON1 (Sp)	0.007146	DCON1	0.017018	DCON1	0
1	12 x 25	Beam	No Messages	14.5	DCON1 (Sp)	0.007146	DCON1	0.017018	DCON1	0
1	12 x 25	Beam	No Messages	16.2233	DCON1 (Sp)	0.007146	DCON1	0.013572	DCON1	0
1	12 x 25	Beam	No Messages	17.9467	DCON1 (Sp)	0.007146	DCON1	0.01015	DCON1	3.916E-05
1	12 x 25	Beam	No Messages	19.67	DCON1 (Sp)	0.007146	DCON1 (Sp)	0.007146	DCON1	0.0001
1	12 x 25	Beam	No Messages	19.67	DCON1 (Sp)	0.007146	DCON1 (Sp)	0.007146	DCON1	0.00332
1	12 x 25	Beam	No Messages	21.46	DCON1 (Sp)	0.007146	DCON1 (Sp)	0.007146	DCON1	0.00338
2	12 x 25	Beam	No Messages	16	DCON1	0.006254	DCON1 (Sp)	0.004106	DCON1	0
2	12 x 25	Beam	No Messages	0	DCON1	0.00625	DCON1 (Sp)	0.004033	DCON1	0
2	12 x 25	Beam	No Messages	1.6167	DCON1	0.004711	DCON1 (Sp)	0.002038	DCON1	0
2	12 x 25	Beam	No Messages	14.0208	DCON1	0.003993	DCON1 (Sp)	0.002038	DCON1	0
2	12 x 25	Beam	No Messages	3.2333	DCON1 (Sp)	0.002038	DCON1 (Sp)	0.002038	DCON1	0
2	12 x 25	Beam	No Messages	4.85	DCON1 (Sp)	0.002038	DCON1 (Sp)	0.002038	DCON1	0
2	12 x 25	Beam	No Messages	6.4667	DCON1 (Sp)	0.002038	DCON1	0.004775	DCON1	0
2	12 x 25	Beam	No Messages	8.0833	DCON1 (Sp)	0.002038	DCON1	0.00625	DCON1	0
2	12 x 25	Beam	No Messages	8.0833	DCON1 (Sp)	0.002038	DCON1	0.00625	DCON1	0
2	12 x 25	Beam	No Messages	10.0625	DCON1 (Sp)	0.002038	DCON1	0.003983	DCON1	0
2	12 x 25	Beam	No Messages	12.0417	DCON1 (Sp)	0.002038	DCON1 (Sp)	0.002038	DCON1	0



Alternative 3: Hollow-Core Plank System



1) Hollow Core Plank

Dead load:

Finishes	2 psf
Framing	8 psf
Misc	10 psf
	20 psf

Live Load: 100 psf

$$1.2D + 1.6L = 1.2(20) + 1.6(100) = 184 \text{ PSF}$$

Using NITTERHOUSE Concrete Products

- Select Prestressed Concrete 12"x4'-0" Nicore Plank

2 Hr Fire Resistance Rating w/ 2" Topping.

- Precast Wt. = 77 PSF

@ 29' - can support 217 psf factored load > 184 psf

OK

Notebook B

Hollow core
plank system

2) Girders. (Non-composite steel beam)

$$DL = 20 \text{ PSF} + 77 \text{ PSF} = 97 \text{ PSF}$$

$$LL = 100 \text{ PSF}$$

$$LL \text{ Red.} = 72.3 \text{ PSF} \text{ (From Previous calc.)}$$

- Load Combs

$$1.4D = 1.4(97) = 135.8 \text{ PSF}$$

$$1.2D + 1.6L = 1.2(97) + 1.6(72.3) = 232.1 \text{ PSF} \leftarrow \text{controls}$$

Girder length = 26.83 ft
 Spacing = $(37.5/2) + 1 = 19.75 \text{ ft}$

$$W_n = 19.75(232.1) = 4.58 \text{ Kif}$$

$$M_n = \frac{Wl^2}{8} = \frac{(4.58)(26.83)^2}{8} = 412 \text{ K-ft}$$

- * Detailed attachment between planks and girders to provide the fully braced condition.
- check LL deflection

$$\Delta_{ll} = \frac{5(4.58)(26.83)^4(1728)}{384(29000)(I)} \leq \frac{L}{360} = \frac{26.83 \times 12}{360} = 0.894$$

$$\Rightarrow I \geq 2059 \text{ in}^4$$

From Table 3-2 Steel Manual

$$W24 \times 84: \phi M_p = 840 > 412 \text{ Kft} \quad \underline{\text{OK}}$$

$$I_x = 2370 \text{ in}^4 > 2059 \text{ in}^4 \quad \underline{\text{OK}}$$

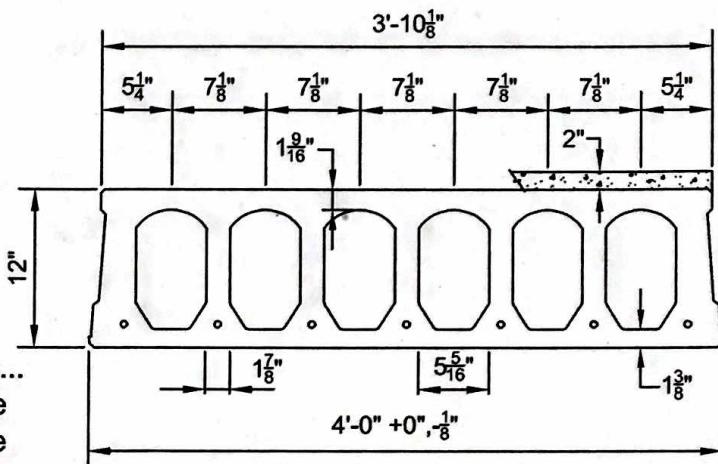
\therefore use W24 x 84 ($d = 24.1$)

Prestressed Concrete 12"x4'-0" NiCore Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES Composite Section

$A_c = 361 \text{ in.}^2$ Precast $b_w = 14.25 \text{ in.}$
 $I_c = 7840 \text{ in.}^4$ Precast $S_{bcp} = 1081 \text{ in.}^3$
 $Y_{bcp} = 7.26 \text{ in.}$ Topping $S_{tct} = 1644 \text{ in.}^3$
 $Y_{tcp} = 4.74 \text{ in.}$ Precast $S_{tcp} = 1653 \text{ in.}^3$
 $Y_{tct} = 6.74 \text{ in.}$ Precast Wt. = 308 PLF
 Precast Wt. = 77.00 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3800 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
 - 6-1/2"Ø, 270K = 205.4 k-ft at 60% jacking force
 - 7-1/2"Ø, 270K = 235.4 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f_c} = 775 \text{ PSI}$
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. All load values are controlled by ultimate flexural strength or fire endurance limits.
14. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.
15. At 2 hours the calculated strand temperature is 790 degrees Farenheit @ 49% of yield strength.

SAFE SUPERIMPOSED SERVICE LOADS												IBC 2012 & ACI 318-11 (1.2 D + 1.6 L)											
Strand Pattern		SPAN (FEET)																					
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44			
6 - 1/2"Ø	LOAD (PSF)	242	217	194	174	156	140	125	111	99	87	77	68	59	51	43	36	29	23	18			
7 - 1/2"Ø	LOAD (PSF)	295	266	240	217	196	177	160	144	130	117	105	94	84	74	65	57	50	43	36			

NITTERHOUSE
CONCRETE  PRODUCTS

2655 Molly Pitcher Hwy. South, Box 2013

Chambersburg, PA 17202-9203

717-267-4505 Fax 717-267-4518

03/25/14

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

12F2.0T

	Existing	Alternative 1	Alternative 2	Alternative 3
	Composite Steel	Steel Joists	One-Way Slab	Hollow Core Planks
Architectural Coordination				
Depth	27	31"	30"	36"
Fire Rating	2 hr	2 hr	2 hr	2 hr
Fire Rating Type	Cementitious/Sprayed	Cementitious/Sprayed	None	None
Construction Statistics				
Cost	\$24.10	\$18.43	\$19.90	\$26.50
Durability	Acceptable	Acceptable	High	High
Structural Considerations				
Weight	60.6 psf	58.6 psf	136.9 psf	81.9 psf
Servicability	Vibration	Vibrations	N/A	N/A
Lateral Systems				
Concrete Shear Wall	yes	no	yes	yes
Steel Moment Frame	yes	yes	no	no
Steel Braced Frame	yes	yes	no	no
Moving Forward?	N/A	YES	YES	NO

1) Weight per bay

Existing - Composite Steel

- Deck/slab : $(46 \text{ psf})(29')(16') = 21.3 \text{ k}$
- Beams : $3(35 \text{ psf})(29) = 3.05 \text{ k}$
- Girders : $(50 \text{ psf})(26.83 \text{ psf}) = 1.34 \text{ k}$
- Studs : $3(60 \times 10) + (58 \times 10) = \frac{2.38 \text{ k}}{28.1 \text{ k}}$

$$\frac{28.1 \times 1000}{29 \times 16} = \boxed{60.6 \text{ psf}}$$

Alternative 1 - Steel Joists & Joist Girder

- Slab/deck : $(25 \text{ psf})(29)(16") = 11.6 \text{ k}$
- Joists : $5(10.3 \text{ plf})(29) = 14.9 \text{ k}$
- Girders : $(27.5 \text{ plf})(26.8') = \frac{74 \text{ k}}{27.2 \text{ k}}$

$$\frac{27.2 \times 1000}{29 \times 16} = \boxed{58.6 \text{ psf}}$$

Alternative 2 - One-Way Slab

- Slab : $(150 \text{pcf})(\frac{5}{12})(29')(16') = 29 \text{ k}$
- 12x24 : $3(150 \text{pcf})(\frac{18 \times 24}{144})(29') = 26.1 \text{ k}$
- 12x25 : $(150 \text{pcf})(\frac{12 \times 25}{144})(26.83) = \frac{8.4 \text{ k}}{63.5 \text{ k}}$

$$\frac{63.5 \times 1000}{29 \times 16} = \boxed{136.9 \text{ psf}}$$

Alternative 3 - Hollow Core Planks

- Hollow Core : $(77 \text{ psf})(29)(16') = 35.7 \text{ k}$
- Girder : $(84 \text{ plf})(26.83) = \frac{2.3 \text{ k}}{38 \text{ k}}$

$$\frac{38 \times 1000}{29 \times 16} = \boxed{81.9 \text{ psf}}$$

Notebook B Comparisons

2) Cost per bay

Existing - Composite Steel

use B1010 254 0800 (RSMeans 2014 Assemblies)

- Bay size 25 x 20
- SDL = 75 psf
- depth = 1' - 9"

Total Base Cost / SF = \$ 24.1 / SF

Alternative 1 - Steel Joist & Tors Girder

use B10 B10 250 4200

- Bay Size 20 x 25
- SDL = 75 PSF
- depth = 26 "

Total Base Cost / SF = \$ 18.43 / SF

Alternative 2 - One-Way Slab

use B10 B10 226 4600

- Bay Size 20 x 25
- SPL = 200 PSF
- Min col. size 18 "
- Rib Depth : 12 "

Total Base Cost / SF = \$ 19.9 / SF

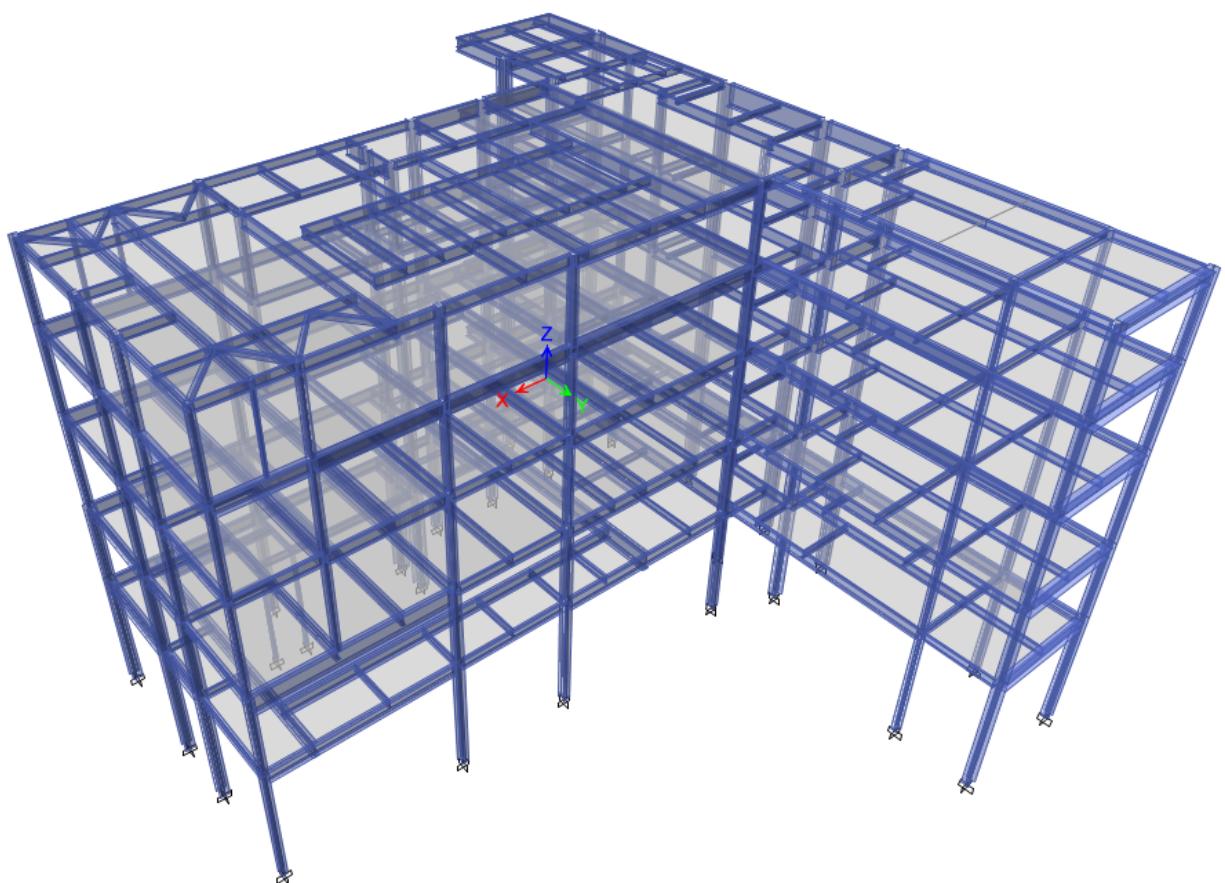
Alternative 3 - Hollow Core

use B1010 238 5200

- Bay Size 20 x 25
- SDL = 100 PSF
- Total depth : 30 "

Total Base Cost / SF = \$ 26.5 / SF

Lateral System Analysis Study



Modeling Information for Lateral Load Analysis

- **Typical Floor Plan of Lateral Resisting Elements**

The lateral force-resisting system of the building is steel moment frame system. As shown in the Figure 1, the typical floor framing includes four moment frames (1, 2, 3, 4) in x-direction and three moment frames (5, 6, 7) in y-direction. The detail of the moment frames are shown in the Figure 2, including frame sections and dimensions. The orange zones indicates the story that will be analyzed in this report.

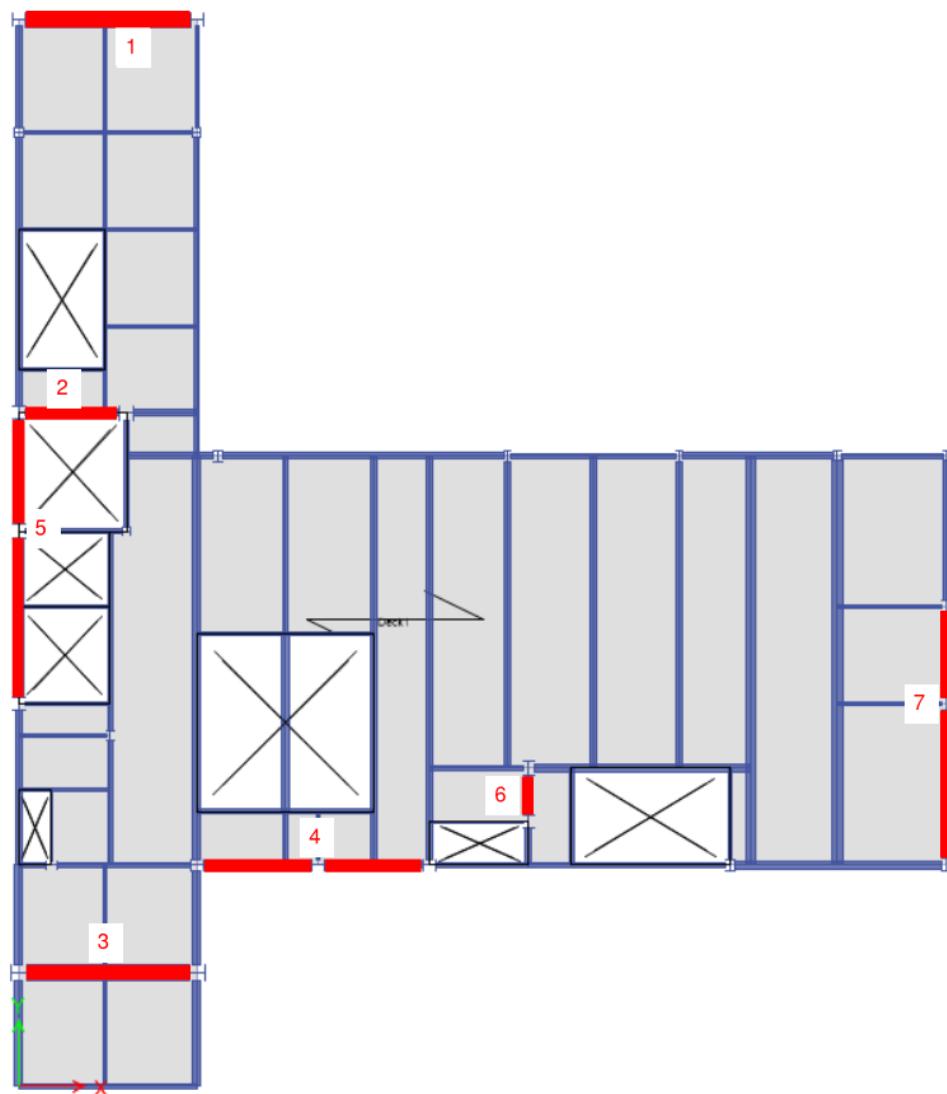


Figure 1. Typical Floor Plan of Lateral Resisting Elements (Fourth Floor Framing)

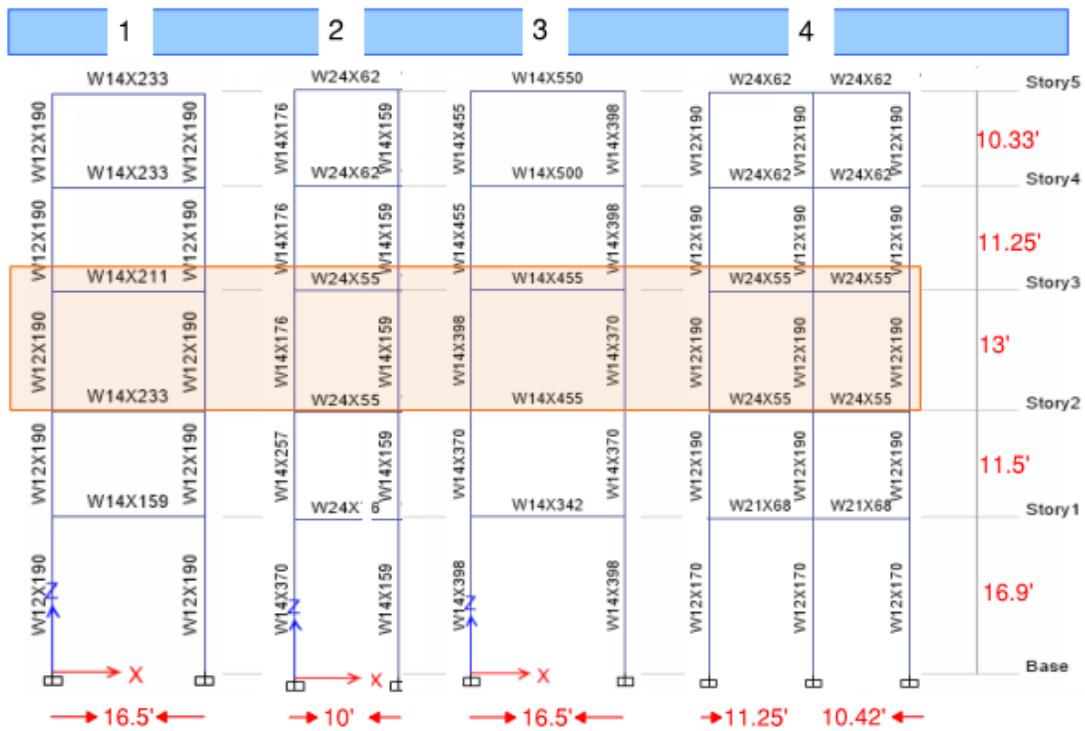


Figure2. Moment Frames details

- **Summary of Modeling Approach**

A full 3D model is created using ETAB for analyzing the lateral force-resisting system of the building. In order to run a lateral analysis, both lateral and gravity systems are modeled and existing member sizes were assigned. Model will run all applicable load combination cases to find the controlling cases for the member strengths and drifts. The following Table 1 shows all the assumptions that have been made in the model:

Element	Assumptions
Diaphragms	<ul style="list-style-type: none"> • Rigid diaphragm for all floors • Include selfweight of the deck • Wall Weights applied to perimeter beams • Roof live Loads included, snow load omitted • No storage loads or partitions provided
Ordinary Moment Frames	<ul style="list-style-type: none"> • Beams are fixed-fixed • Columns are fixed-fixed
Others	<ul style="list-style-type: none"> • All other beams are pinned-pinned

Table 1. ETAB Model Assumptions

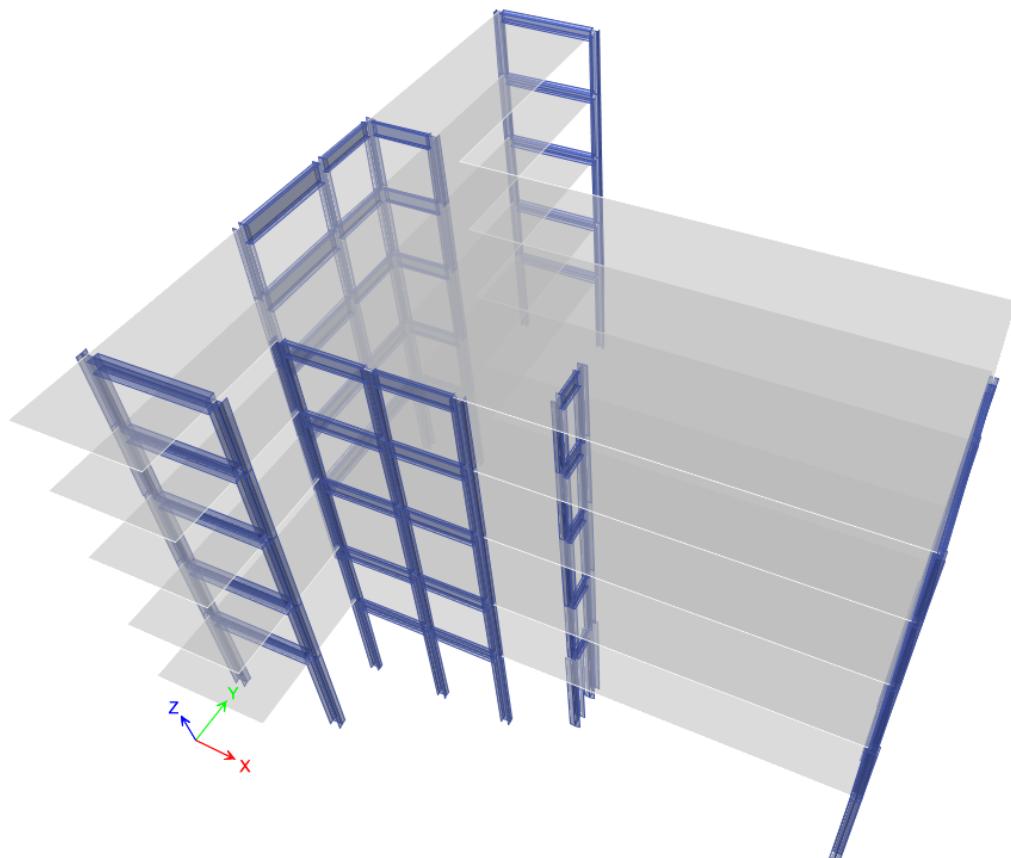


Figure3. Lateral Force-Resisting Elements 3D View

Model Validation

- **Center of Mass**

Center of mass is calculated by computer analysis and verified by manual calculation. Table 2 shows the results of center of mass from ETAB report and Table 3 indicates the COM which calculated by hand. (See detailed manual calculation for COM in Appendix A)

Story	Diaphragm	Mass X lb-s ² /ft	Mass Y lb-s ² /ft	XCM ft	YCM ft
Story5	Rigid	18560.49	18560.49	32.6541	43.9976
Story4	Rigid	18664.67	18664.67	26.8543	44.8081
Story3	Rigid	25114.9	25114.9	33.3829	44.3564
Story2	Rigid	25135.17	25135.17	33.1723	44.1149
Story1	Rigid	25049.1	25049.1	30.4372	44.978

Table 2. Computer analysis of COM (@ 4th Floor)

Center of Mass (Fourth floor)				
COM (X)=	$\frac{\sum W*x}{\sum W}$	=	$\frac{24784534}{750953.1}$	= 33.00
COM (Y)=	$\frac{\sum W*y}{\sum W}$	=	$\frac{31037695}{750953.1}$	= 41.33

Table 3. Manual Calculation of COM (@ 4th Floor)

- **Comparison of COM:**

Table 4 shows the different between of center of mass calculated by hand and computer; it is found to be acceptable since the % of error is less than 7% in both directions. Figure 4 provides a clear picture for audiences to understand where the center of mass is and what difference of COM determined by the two methods.

Comparison Of COM						
	Manual	Computer	Difference	% Error	% of Dimension	Check
COM (X)	33	33.38	0.38	1.14	0.44	Ok
COM (Y)	41.33	44.36	3.03	6.83	3.06	Ok

Table 4. Comparison of COM

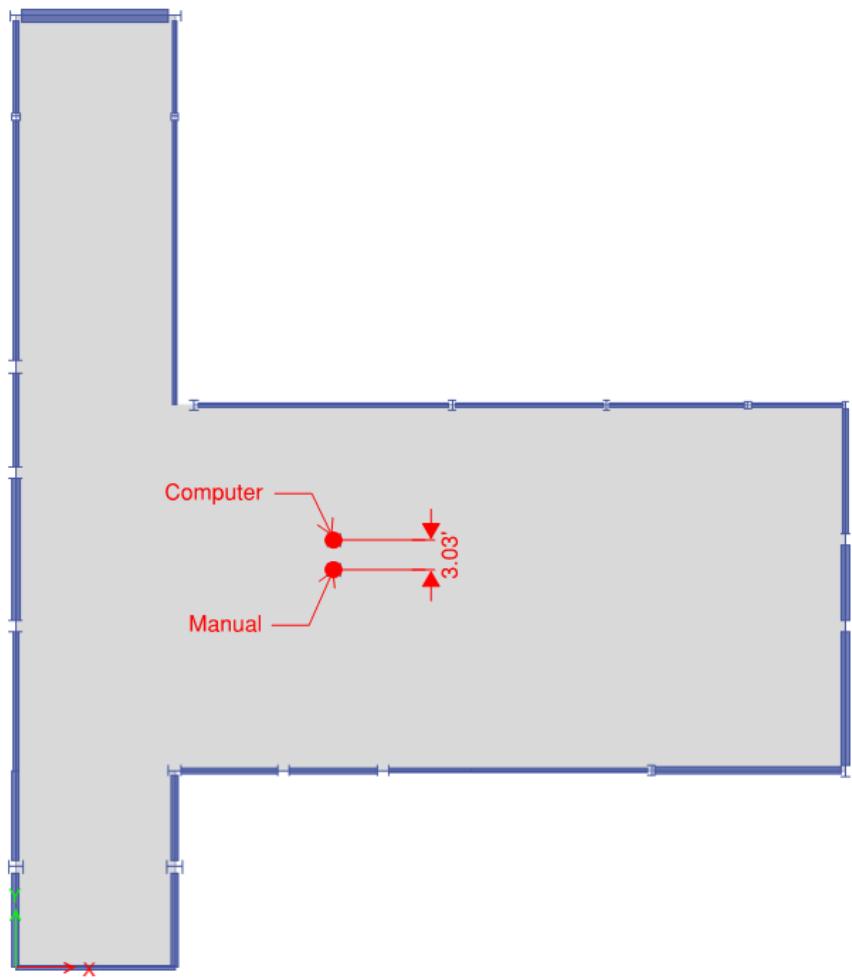


Figure 4. Comparison of COM

- **Center of Rigidity**

Center of rigidity is also calculated by computer analysis and verified by manual calculation. Table 6 shows the center of rigidity by manual calculation and Table 7 shows the COR from ETAB reports.

Story shear analysis is used to determine the horizontal stiffness of the building. The horizontal stiffness of a rigid frame is governed mainly by the bending resistance of the girders, the columns and their connections. The following equation is used to estimate the shear stiffness of each lateral force-resisting member: (Figure 5)

The shear stiffness of a story k_v is given as:

$$K_v = \sum_{\text{all columns}} K_{vc}$$

$$K_{vc} = \frac{12EI_c}{h^3} \left[\frac{1}{1 + \frac{I_c}{h \sum_{i=1}^{\text{floor}} \frac{I_b}{b}} + \frac{I_c}{h \sum_{i=1}^{\text{floor}} \frac{I_b}{b}}} \right]$$

Figure 5. Story Shear Equation of Calculate Stiffness

Calculation of Member Stiffness (Fourth floor)									
Element	Col. #	E (ksi)	Ic (in^4)	Ib3 (in^4)	Ib2 (in^4)	hc (ft)	Lb (ft)	K (k/in)	kt (k/in)
1	Col.1	29000	1890	2660	3010	13	16.5	8.1	16.2
	Col.2	29000	1890	2660	3010	13	16.5	8.1	
2	Col.1	29000	2140	1350	1350	13	10	6.5	12.6
	Col.2	29000	838	1350	1350	13	10	6.2	
3	Col.1	29000	6000	7190	7190	13	16.5	20.8	41.6
	Col.2	29000	5440	7190	7190	13	16.5	20.7	
4	Col.1	29000	1890	1350	1350	13	11.3	5.8	23.5
	Col.2	29000	1890			13	10.8	11.6	
	Col.3	29000	1890	1350	1350	13	10.4	6.2	
5	Col.1	29000	2140	5900	4470	13	16	14.6	46.2
	Col.2	29000	2140			13	13.5	23.0	
	Col.3	29000	2140	1550	2850	13	11	8.7	
6	Col.1	29000	1890	1350	1350	13	5	12.4	24.9
	Col.2	29000	1890	1350	1350	13	5	12.4	
7	Col.1	29000	722	722	722	13	15	2.3	11.9
	Col.2	29000	722			13	12	5.8	
	Col.3	29000	722	722	722	13	9	3.8	

Table 5. Horizontal Stiffness of Each Member @ 4th Floor

Center of Rigidity (Fourth floor)							
Element Table	Element Direction	Dist. From Ref. Datum X (ft)	Y (ft)	Rx (k/in)	Ry (k/in)	Rx*Y	Ry*x
1	X	8.25	99	16.2	0	1604.04	0
2	X	5	62.5	12.6	0	790.41	0
3	X	8.25	10.5	41.6	0	436.32	0
4	X	27.34	20.5	23.5	0	482.30	0
5	Y	0	43.5	0	46.2	0	0
6	Y	47.34	27	0	24.9	0	1178.12
7	Y	86.18	32.5	0	11.9	0	1023.56
				Σ	93.9	83.0	3313.07
							2201.68
$COR(X) = \frac{\sum Ry^*X}{\sum Ry} = \frac{2201.684}{83.0} = 26.53$							
$COR(Y) = \frac{\sum Rx^*Y}{\sum Rx} = \frac{3313.069}{93.9} = 35.27$							

Table 6. Manual Calculation of COR (@4th Floor)

Story	Diaphragm	Mass X lb-s ² /ft	Mass Y lb-s ² /ft	XCM ft	YCM ft	Cumulative X lb-s ² /ft	Cumulative Y lb-s ² /ft	XCCM ft	YCCM ft	XCR ft	YCR ft
Story5	Rigid	18560.49	18560.49	32.6541	43.9976	18560.49	18560.49	32.6541	43.9976	22.6269	34.9385
Story4	Rigid	18664.67	18664.67	26.8543	44.8081	37225.16	37225.16	29.7461	44.404	23.8529	35.1613
Story3	Rigid	25114.9	25114.9	33.3829	44.3564	62340.05	62340.05	31.2112	44.3148	24.8758	35.4993
Story2	Rigid	25135.17	25135.17	33.1723	44.1149	87475.23	87475.23	31.7747	44.3072	26.3544	35.8567
Story1	Rigid	25049.1	25049.1	30.4372	44.978	112524.33	112524.33	31.477	44.4566	27.61	36.0639

Table 7. Computer Analysis of COR (@ 4th Floor)

- Comparison of COM:

Table 8 shows the difference between center of rigidity calculated by hand and computer; it is found to be acceptable since the % of error is less than 7% in both directions. Figure 6 provides a clear picture for audiences to understand where the center of rigidity is and what the difference of COM determined two methods.

Comparison Of COR						
	Manual	Computer	Difference	% Error	% of Dimension	Check
COR (X)	26.53	24.88	1.65	6.63	1.91	Ok
COR (Y)	35.27	35.5	0.23	0.65	0.23	Ok

Table 8. Comparison of COR

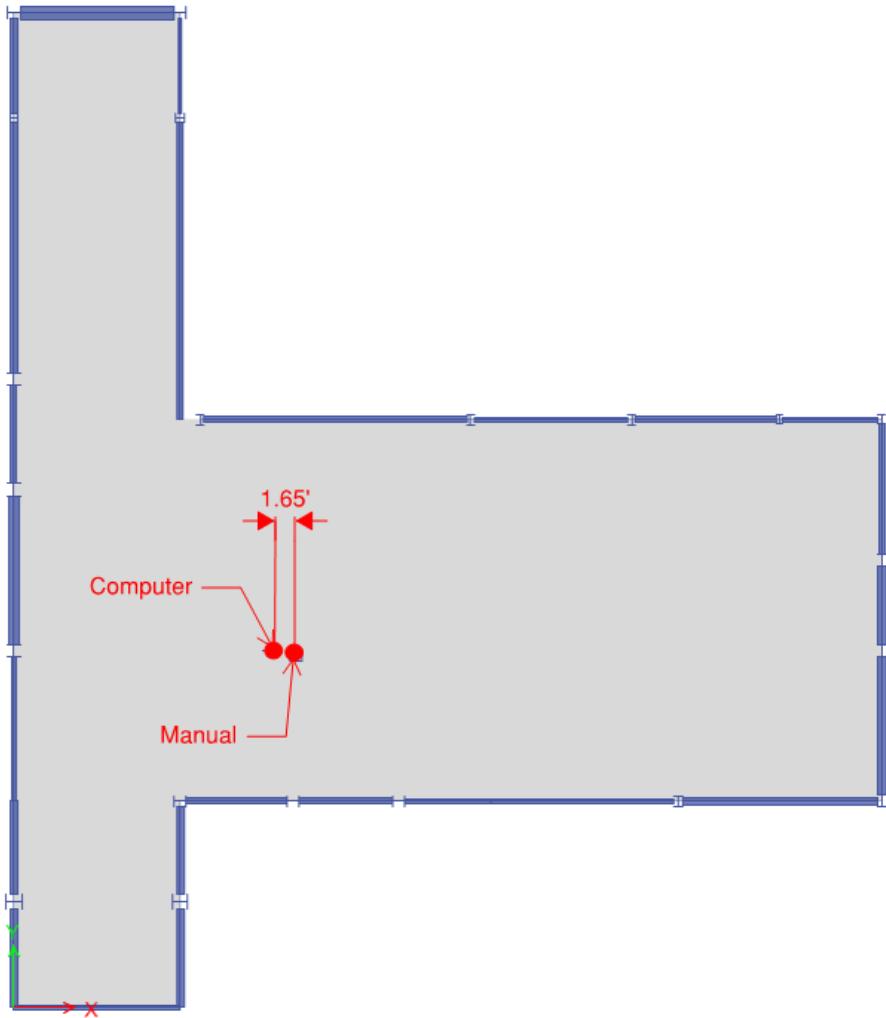


Figure 6. Comparison of COR

- **Wind and Seismic Load Check**

In order to verify the computer model, wind loads determined by computer model is compared by the wind load calculated in Notebook Submission A. Table 9 shows the comparison of wind loads in both x and y direction. Table 10 shows the comparison of seismic loads in both x and y direction. The largest percentage of difference in Table 9 and Table 10 is less 10%, which is acceptable. (See hand calculation of wind and seismic loads in Structural Notebook A and computer analysis in Appendix A)

Wind Load Comparison in X-direction						
Story	Manual Calculation (Notebook A)			ETAB2015 Model		% of Error
	Wind Load (psf)	Story Height (ft)	Dimension (ft)	Wind Load (k)	Wind Load (k)	
Parapet	45.5	2	99	9.01	21.35	5.95
Roof	21.3	5.25	99	11.07		
4	20.5	11	99	22.32	22.38	0.25
3	19.8	12.25	99	24.01	24.1	0.36
2	18.6	12.25	99	22.56	22.99	1.88
1	18.6	14.25	99	26.24	24.65	6.06

Note: (1) Due to the irregularity of my building, the leeward wind load is 9 psf at part 2 and 5 psf at part 3 (showed in E-S Wind calculation section, Notebook A. An average value 7 psf is used as a leeward load applying on the whole building (E-W) to determined the wind load (psf) at first column above. (2) See Appendix B for detailed wind load information of ETAB Model

Wind Load Comparison in Y-direction						
Story	Manual Calculation (Notebook A)			ETAB2015 Model		% of Error
	Wind Load (psf)	Story Height (ft)	Dimension (ft)	Wind Load (k)	Wind Load (k)	
Parapet	45.5	2	86.2	7.84	18.37	4.83
Roof	21.3	5.25	86.2	9.64		
4	20.5	11	86.2	19.44	19.02	2.15
3	19.8	12.25	86.2	20.91	20.46	2.14
2	18.6	12.25	86.2	19.64	19.49	0.77
1	18.6	14.25	86.2	22.85	20.85	8.74

Note: (1) Same assumption as X - direction (2) Wind load adjusted since 1 is used for gust factor in computer model; however, gust factor is 1.08 in the hand calculation in the Notebood A. (3) See Appendix B for detailed wind load information of ETAB Model

Table 9. Comparison of Wind Loads

Seismic Load Comparison (X & Y direction)			
Story	Manual Calculation	ETAB Model	% of Error
	Seismic Load (k)	Seismic Load (k)	
Roof	49.4	50.4	1.98
4	38.4	41.3	7.02
3	39.5	42.3	6.62
2	25.4	27.6	7.97
1	13.8	15.2	9.21
Base Shear	166.5	176.8	5.83

Note: (1) Seismic base shear calculated by SGH is 164 k. So it's good
(2) See Appendix B for detailed seismic laod information of ETAB Model

Table 10. Comparison of Seismic Loads

- **Equilibrium Check**

The base reactions are checked to verify the computer model. Theoretically, for the wind load case 1, when the wind acts in the X direction (E-W) of the building, the sum of the base reactions of the lateral columns in the X direction should be equal to the base shear due to the wind load, and the column reactions in the Y direction should be added up to 0.

However, it's shown in the Table 11 that they are not perfectly equal. The difference in X direction is 7.6 k and the difference in Y direction is 0.8 k. Since columns are all assigned fixed-fixed; columns rather than lateral columns are still taking some small forces from the wind load.

Equilibrium Check @the base of the building			
Member	Joint Label	FX kips	FY kips
1	1	-9.24	-0.28
	31	-9.24	-0.12
2	9	-9.45	-3.24
	46	-8.30	-0.23
3	15	-15.78	-1.65
	84	-15.75	-0.67
4	82	-5.13	-0.12
	83	-6.37	-0.02
	70	-5.14	0.07
	5	-4.74	-4.24
	10	-3.93	-3.33
	12	-6.44	3.75
6	74	-6.21	3.75
	77	-0.97	1.74
7	55	-0.92	2.11
	56	-0.28	1.68
	58	Σ	-107.90
			-0.80
Base Shear		115.49	0.00
Difference		7.59	0.80

All values are from
Joint Reactions of
ETAB report data,
which is attached in
Appendix B

See Appendix B for the
Base Reactions

Table 11. Equilibrium Check

- Lateral Load Distribution to Individual Members

Wind Case:

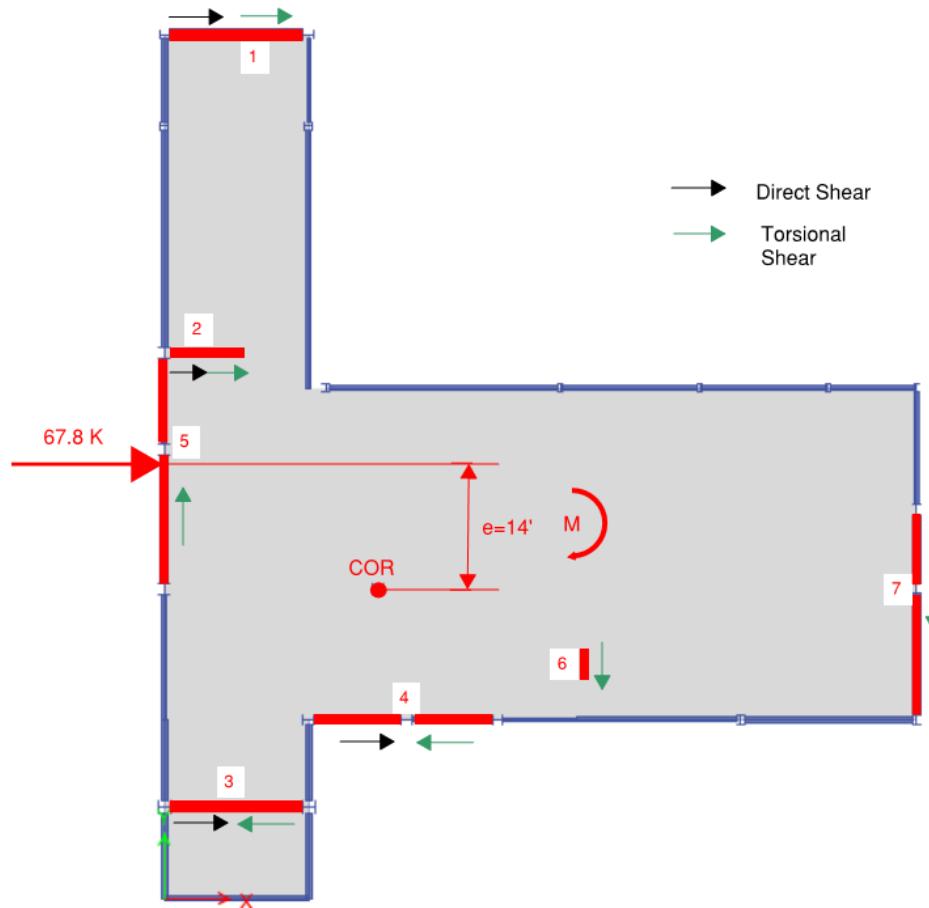


Figure 7. Configuration of Shear Forces Distribution

◆ Torsional Rigidity:

Torsional Rigidity								
Element	Rx	Ry	dx	dy	Rx*dy	Ry*dx	Rx*dy^2	Ry*dx^2
1	16.2	/	/	63.5	1028.86	/	65332.3	/
2	12.6	/	/	27	341.46	/	9219.4	/
3	41.6	/	/	25	1038.85	/	25971.2	/
4	23.5	/	/	15	352.90	/	5293.5	/
5	/	46.23	24.88	/	/	1150.209	/	28617.21
6	/	24.89	22.46	/	/	558.95	/	12554.0
7	/	11.88	61.3	/	/	728.06	/	44630.2
						J=Σ	191617.7	

Table 12. Torsional Rigidity

✚ Direct Shear Into Moment Frames (Vd)

Direct Shear Into MFs			
Element	Rx (k/in)	Direct Shear Vd (k)	
1	16.2	11.70	→
2	12.6	9.13	→
3	41.6	29.99	→
4	23.5	16.98	→
ΣRX	93.9		

Eq:

$$V = 67.8 \text{ k}$$

$$V_d = \frac{R_i}{\sum R_X} * V$$

Table 13. Direct Shear Into Moment Frames (Vd)

✚ Torsional Shear Into Moment Frames (Vt)

Torsional Shear Into MFs			
Element	Ridi	Torsional Shear Vt (k)	
1	1028.86	5.10	→
2	341.46	1.69	→
3	1038.85	5.15	←
4	352.90	1.75	←
5	1150.21	5.70	↑
6	558.95	2.77	↓
7	728.06	3.61	↓

Eq:

$$M_t = V * e = 949.2 \text{ k}$$

$$V_t = \frac{M_t}{J} * (Ridi)$$

Equilibrium Check

$$\sum V_x = 67.69 \text{ Ok}$$

$$\sum V_y = -0.68 \text{ Ok}$$

Table 14. Torsional Shear Into Moment Frames (Vt)

✚ Total Shear Into Moment Frames (V):

Total Shear Into MFs			
Element	Manual	Model	% of Error
1	16.79	17.89	6.15
2	10.82	8.47	21.69
3	24.85	24.36	1.96
4	15.23	13.73	9.88
5	5.70	3.88	31.85
6	2.77	1.80	34.99
7	3.61	3.51	2.62

Note: The errors are caused by the gravity columns taking some small shear forces as well the model. See the Figure 8 in the following page for the detailed shear forces of each lateral load-resisting element from ETAB Model

Table 15.Total Shear Into Moment Frames (V)

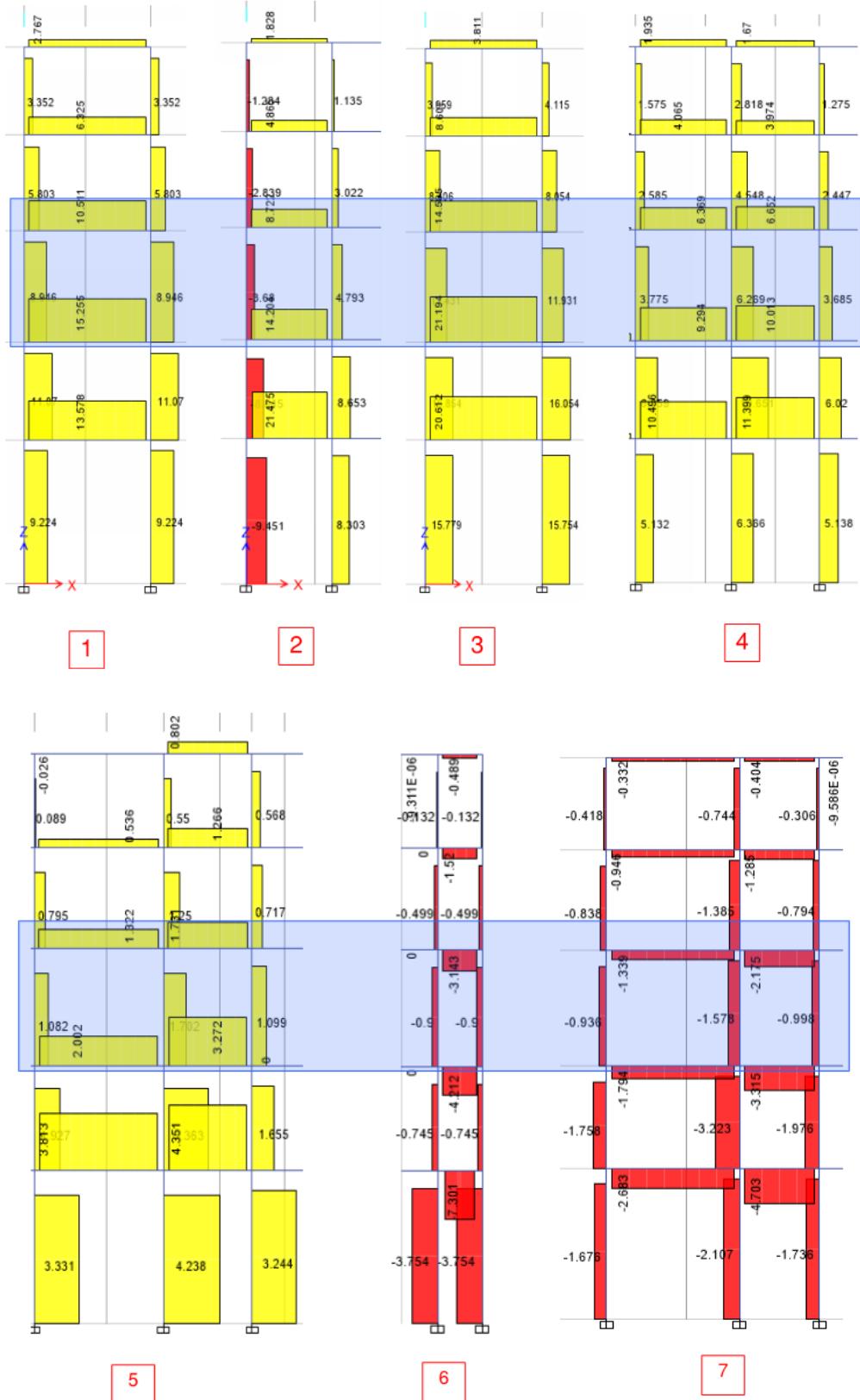


Figure 8. Detailed Shear Forces in Each Lateral Load-Resisting Element (due to wind)

Seismic Load

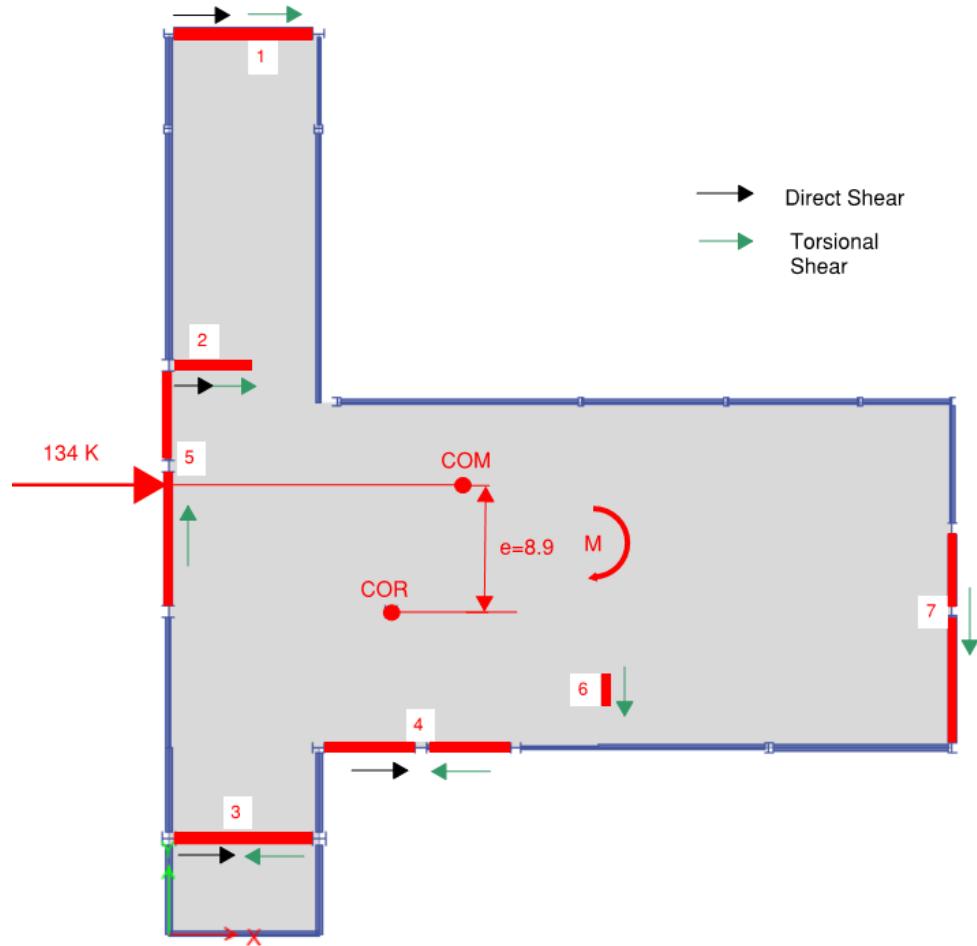


Figure 9. Configuration of Shear Forces Distribution

■ Torsional Rigidity (J):

Torsional Rigidity								
Element	Rx	Ry	dx	dy	Rx*dy	Ry*dx	Rx*dy^2	Ry*dx^2
1	16.2	/	/	63.5	1028.86	/	65332.3	/
2	12.6	/	/	27	341.46	/	9219.4	/
3	41.6	/	/	25	1038.85	/	25971.2	/
4	23.5	/	/	15	352.90	/	5293.5	/
5	/	46.23	24.88	/	/	1150.209	/	28617.21
6	/	24.89	22.46	/	/	558.95	/	12554.0
7	/	11.88	61.3	/	/	728.06	/	44630.2
						$J=\sum$	191617.7	

Table 16. Torsional Rigidity

► Direct Shear Into Moment Frames (Vd):

Direct Shear Into MFs				Eq:
Element	Rx (k/in)	Direct Shear Vd (k)	→	
1	16.2	23.11	→	V = 134 k
2	12.6	18.04	→	Vd = $\frac{R_i}{\sum R_x} * V$
3	41.6	59.28	→	
4	23.5	33.56	→	
$\sum R_x$	93.9			

Table 17. Direct Shear Into Moment Frames (Vd)

► Torsional Shear Into Moment Frames (Vt):

1. Accidental Eccentricity:

Where diaphragms are not flexible, the design shall include the torsional moment (M_t) (kip or kN) resulting from the location of the structure masses plus the accidental torsional moments (M_{ta}) (kip or kN) caused by assumed displacement of the mass each way from its actual location by a distance equal to **5%** of the dimension of the structure perpendicular to the direction of the applied forces. (S. 9.5.5.5.2 ASCE 7-02)

$$e_{acc} = 5\% L = 0.05 (99') = 4.95'$$

2. Critical Shear case:

Use $e + e_{acc}$ for V1 & V2 since the shear due to Torsion will be added to the direct shear in moment frame 1 and 2; use $e - e_{acc}$ for V3 & V4 since the shear due to Torsion will be subtracted from direct shear in moment frame 3 and 4.

Torsional Shear Into MFs							Eq:
Element	Ridi	Critical Moment	Torsional	Shear Vt			
		V * (e+eacc)	V * (e-eacc)	V * e			
1	1028.86	1855.9	/	/	9.96	→	$M_t = V * e = 1192.6$
2	341.46	1855.9	/	/	3.31	→	$M_t = V * (e + eacc) = 1855.9$
3	1038.85	/	529.3		2.87	←	$M_t = V * (e - eacc) = 529.3$
4	352.90	/	529.3		0.97	←	$Vt = \frac{M_t}{J}$
5	1150.21	/	/	1192.6	7.16	↑	Equilibrium Check
6	558.95	/	/	1192.6	3.48	↓	$\sum Vx = 143.43$ ok
7	728.06	/	/	1192.6	4.53	↓	$\sum Vy = -0.85$ ok

Table 18. Torsional Shear Into Moment Frames (Vt)

3. Plan Irregularity Requirements

a. Re-entrant Irregularity

$$(99-58.5)(86.2-16.5)/(99 \times 86.2) = 0.33 > 0.15$$

So, re-entrant Irregularity exists.

Design forces determined shall be increased for connections of diaphragms to vertical elements and to collectors to the vertical elements. (S.9.5.2.6.4.3 ASCE 7-2)

b. Torsional Irregularity

$$V = R * \delta$$

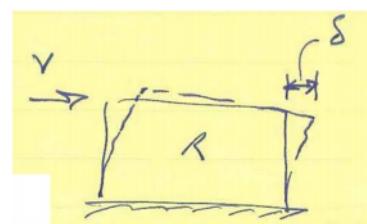
$$\delta(1) = V_1/R_1 = 33.08/16.2 = 2.0" \text{ (Max.)}$$

$$\delta(3) = V_1/R_1 = 56.41/41.6 = 1.36"$$

$$\delta(\text{avg}) = (1.36 + 2.0) / 2 = 1.68$$

$$\delta(\text{Max})/\delta(\text{avg}) = 2.0 / 1.68 = 1.19 < 1.2$$

Resulting Displacements



So, torsional Irregularity doesn't exist.

$$A_x = \left(\frac{\delta_{\max}}{1.2\delta_{\text{avg}}} \right)^2 \quad (\text{Eq. 9.5.5.5.2})$$

Doesn't apply.

>Total Shear Into Moment Frames (V)

Total Shear Into MFs			
Element	Manual	Model	% of Error
1	33.08	30.84	6.77
2	21.35	15.76	26.17
3	56.41	52.04	7.75
4	32.59	28.44	12.72
5	7.16	4.81	32.85
6	3.48	2.38	31.47
7	4.53	4.67	2.89

in

Note: The errors are caused by the gravity columns taking some small shear forces as well in the model. See Figure 10 the following page for the detailed shear forces of the lateral load-resisting elements from ETAB Model

Table 19.Total Shear Into Moment Frames (V)

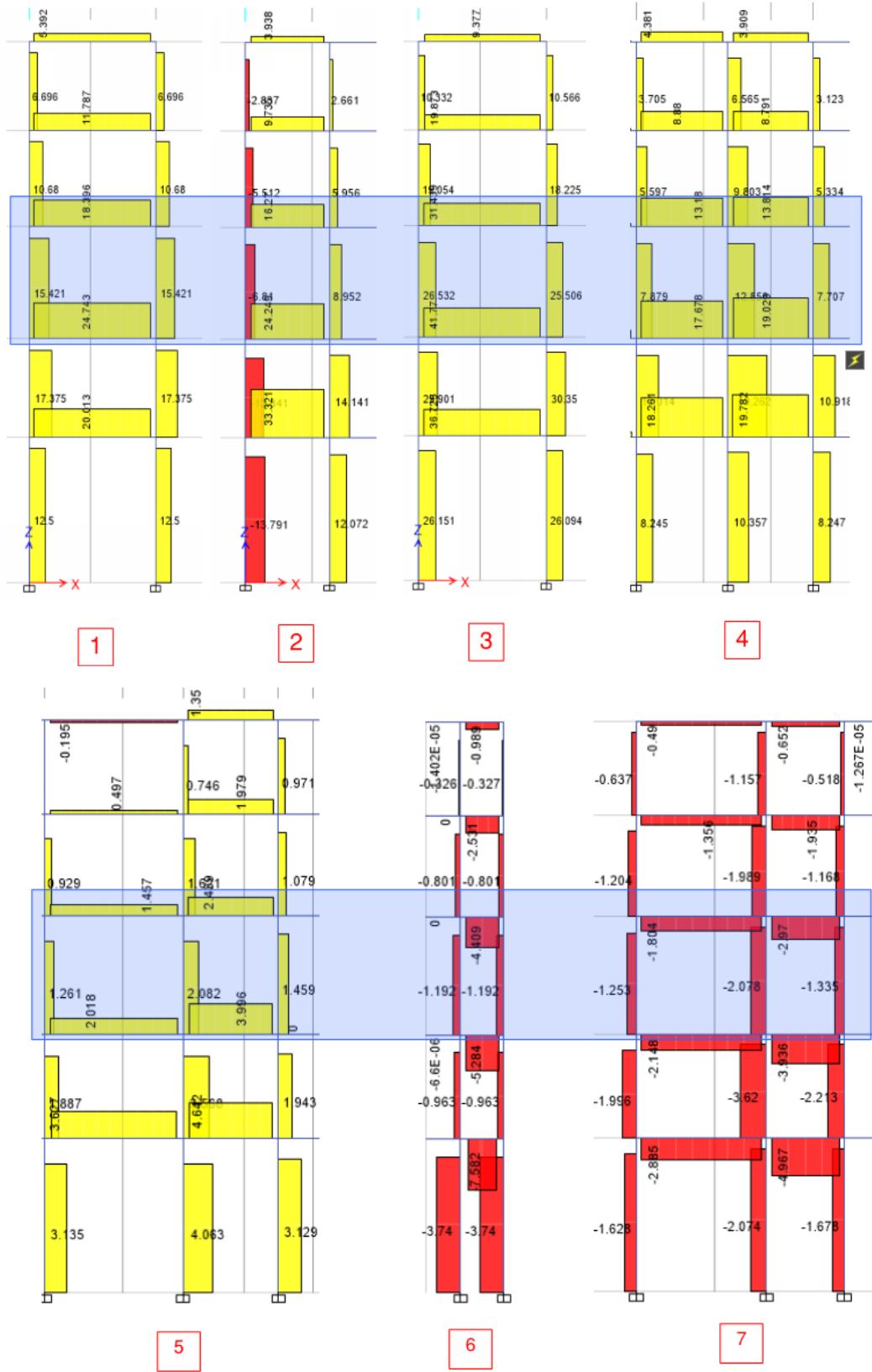


Figure 10. Detailed Shear Forces in Each Lateral Load-Resisting Element (due to Seismic)

Lateral System Checks

- Drift/Story Drift

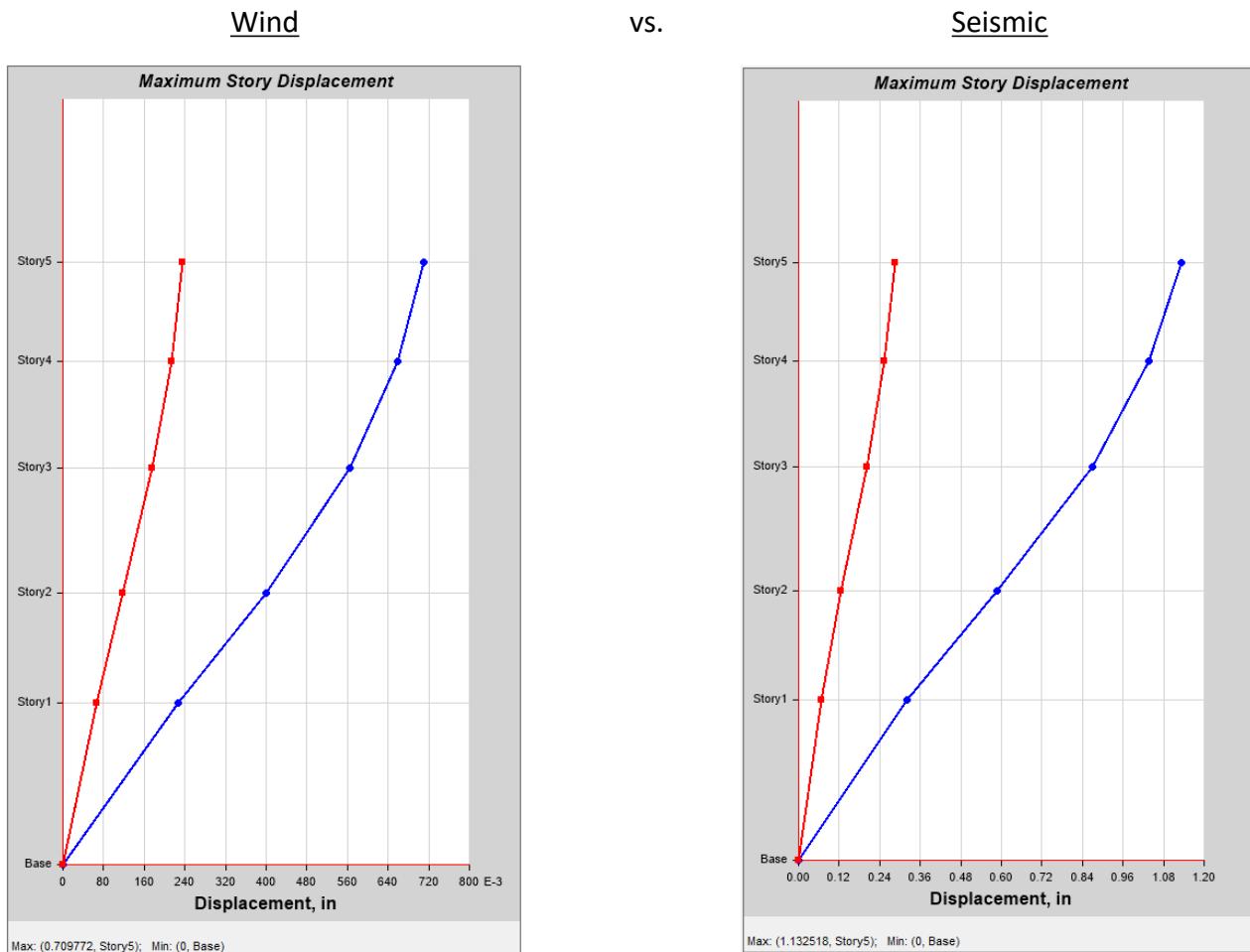


Figure 11. Wind & Seismic Drift

From graphs above, it's able to conclude that the seismic load controls the drift of the building.

- Code or Industry Acceptable Values
 - Wind: H/400 (Industry)
 - Seismic: 0.020hsx (ASCE 7 - 02)

Drift Comparison				
	Model	Code	Industry	Good ?
Wind	0.7	/	1.905	Good
Seismic	0.26	0.26		Okay
Note: Story shear for seismic is calculated on story 3, which is $0.85 - 0.59 = 0.26"$ ($hsx = 13'$)				

Table 20. Drift Comparison with Codes/Industry Acceptable Values

- Strength and overturning

In this section, moment frame #3 was chosen to be checked and compared by three different load combinations base on its strength, overturning and impact on the foundations. The following graphs indicates the axial forces, in-plane shears and moment in the columns for three different load combinations.

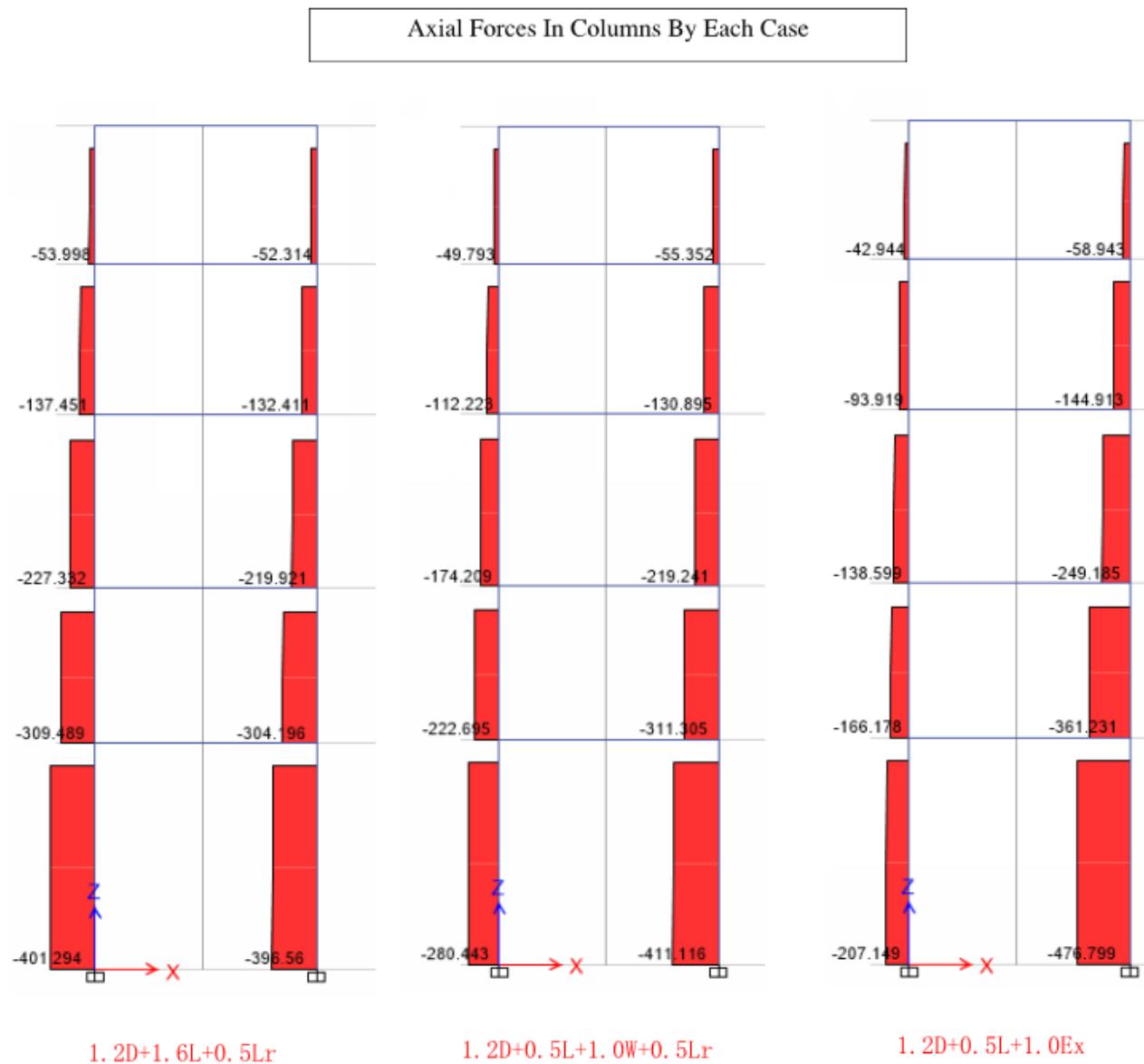


Figure 12. Axial Forces in Columns by Each Load Combination (MF#3)

In-Plane Shear In Columns By Each Case

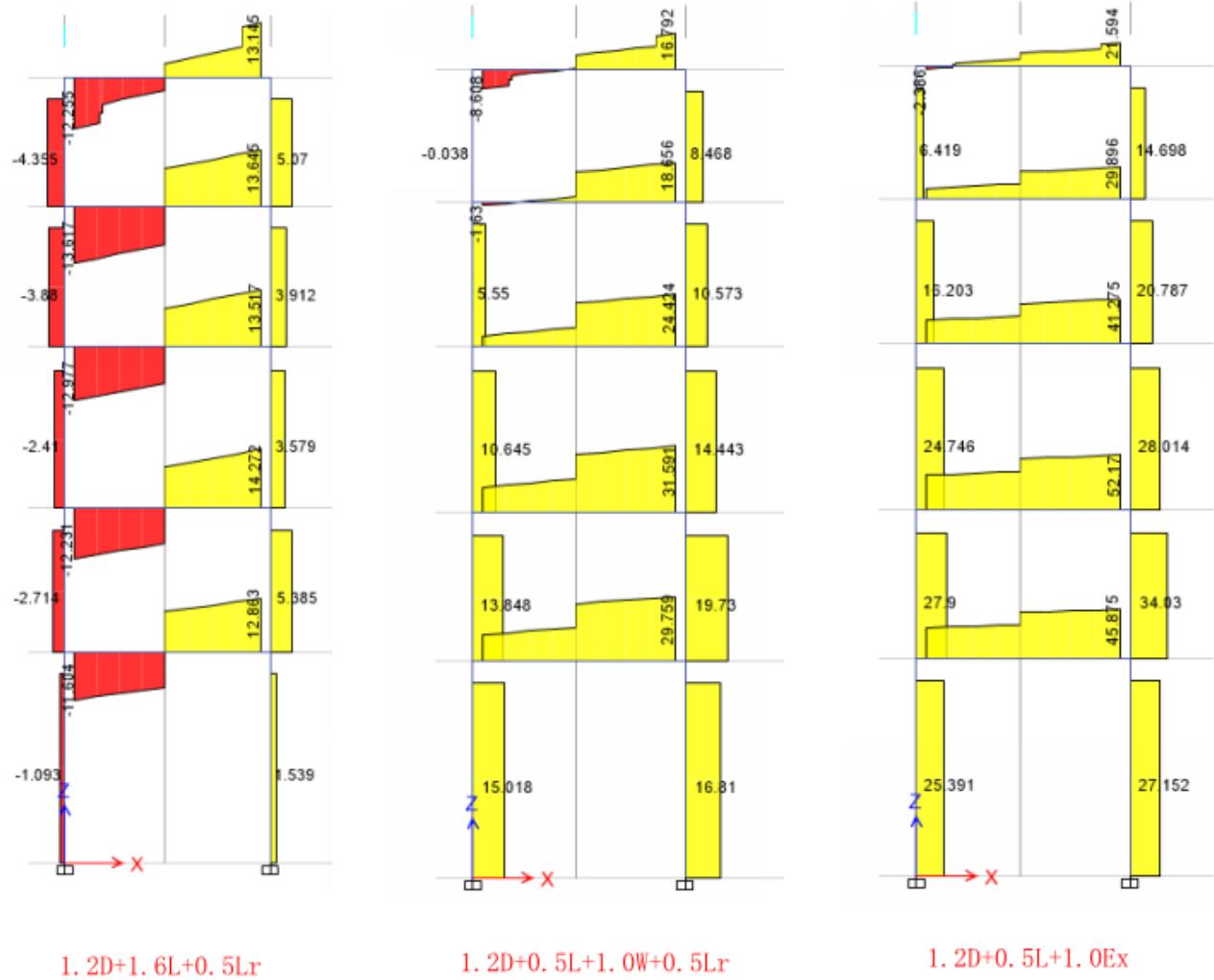


Figure 13. In-Plane Shear Forces in Columns by Each Load Combination (MF#3)

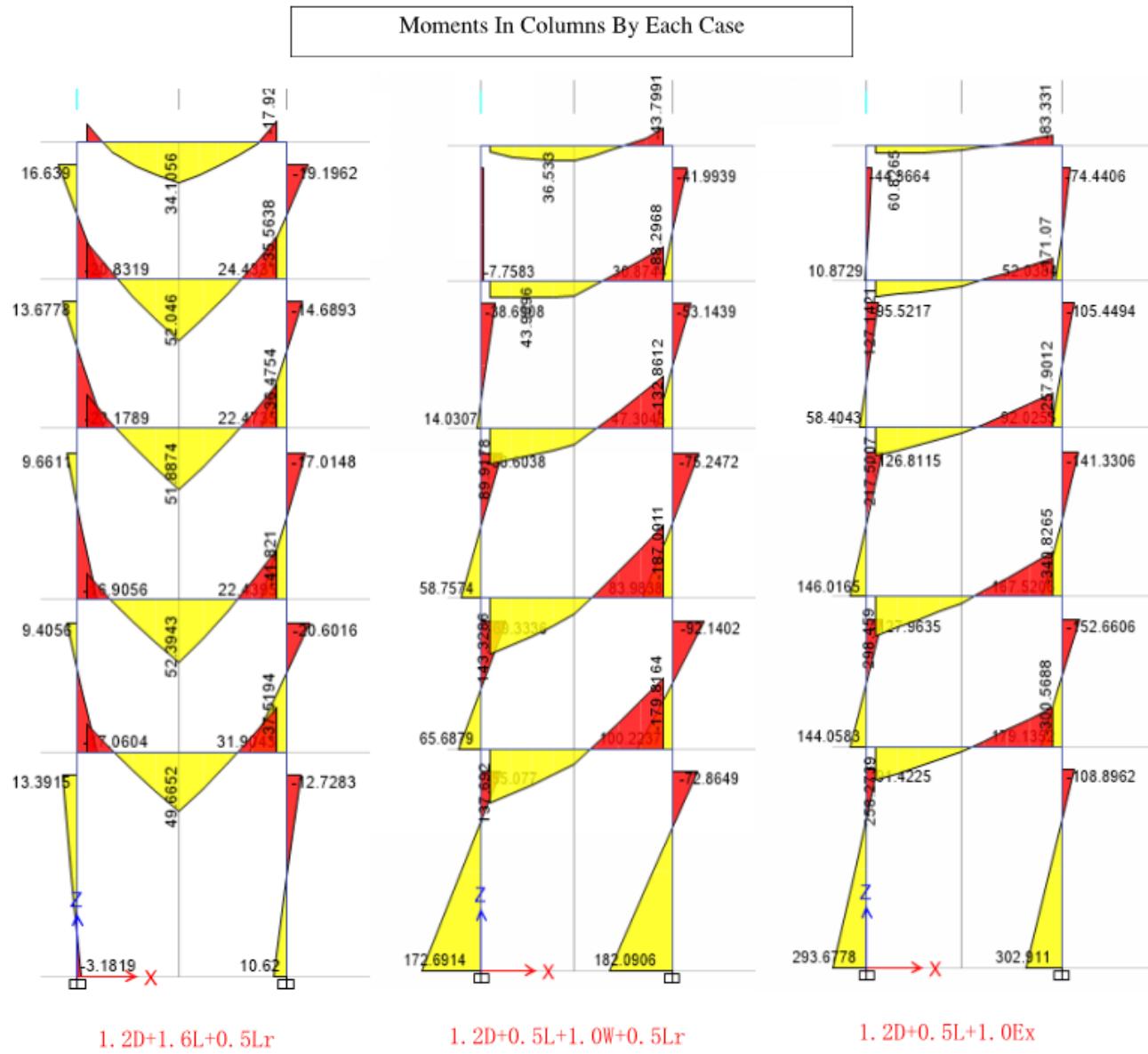


Figure 14. Moments in Columns by Each Load Combination (MF#3)

From the graphs above, it's able to conclude that Load Comb. 3 (1.2D + 0.5L +1.0Ex) controls both strength and overturning moment at the base of moment frame #3. Therefore, shear and gravity forces from Load Comb. 3 will be used to verify the capacity and size of the moment frame #3 in the next section. (P_δ Effect)

Member Spot Checks for Lateral Loads

- The important element of lateral force-resisting system will be verified in this section is moment frame #3, which is circled in Figure 15. The elevation and the forces acting on the moment frame #3 are shown in the Figure 16. (Forces are found in the previous section)

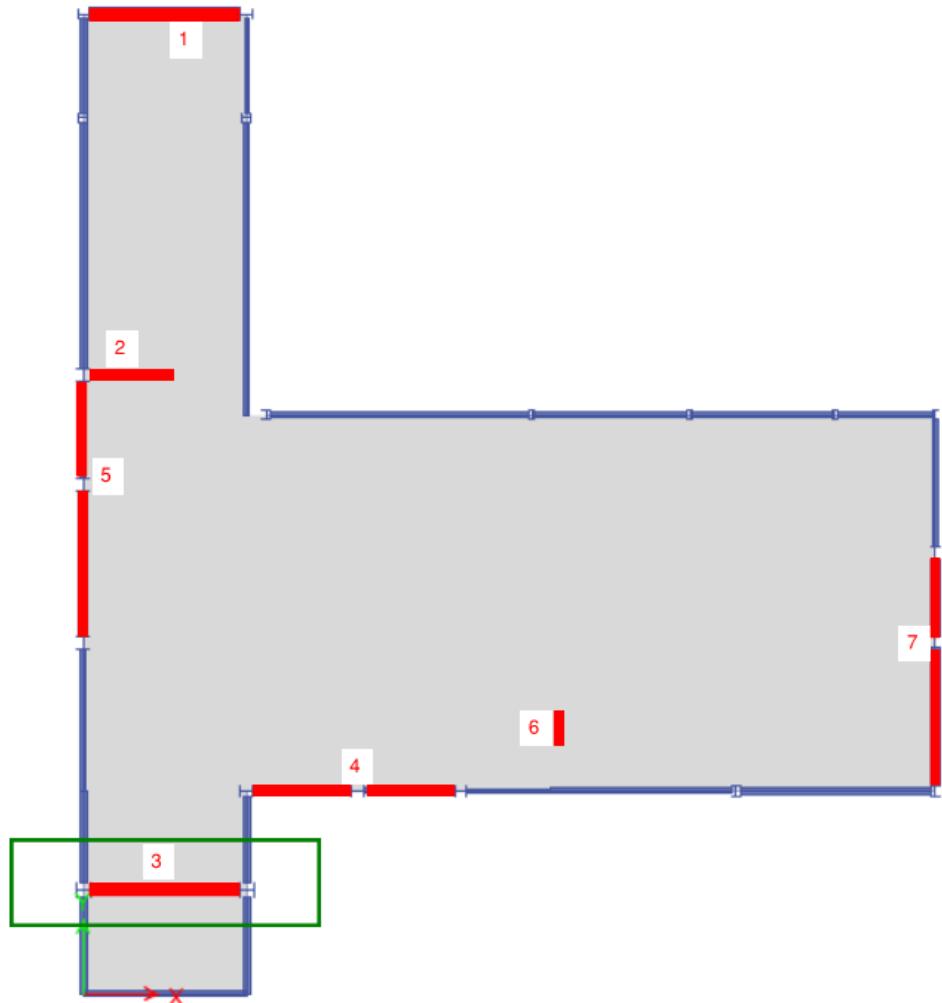


Figure 15. Selected Moment Fame # 3 @4th Floor (circled in green)

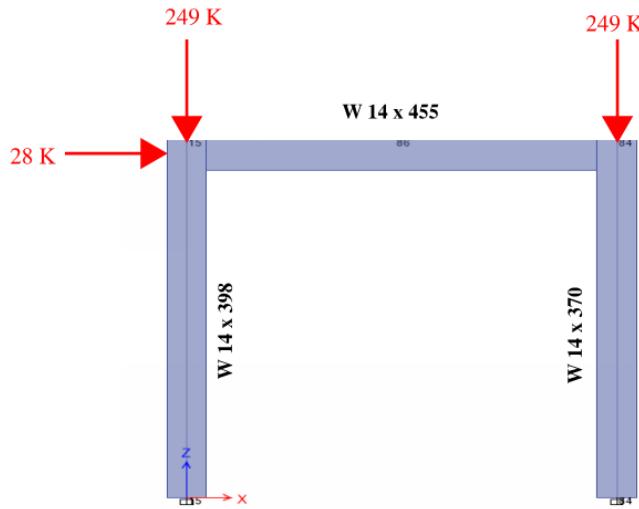


Figure 16. Elevation of Moment Frame #3 @ 4th Floor

- **Verification**

P_{story} , P_{mf} & R_m

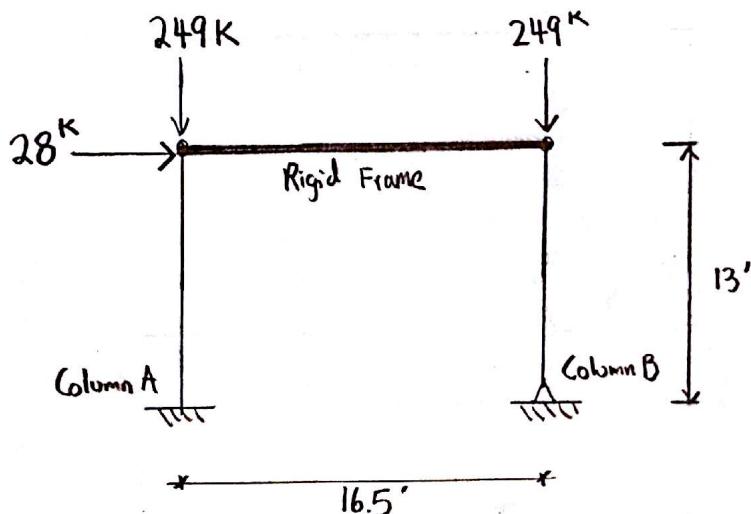
Column Load Summary for 4th floor		Column Load Summary for 4th floor	
Column	$P (k) (1.2D + 0.5L + 1.0Ex)$	Column	$P (k) (1.2D + 0.5L + 1.0Ex)$
A2	138.6	3L	109.6
A5	103.5	4G	119.1
A7	58.7	G(34)	39.3
A9	43.6	5B	55.2
A13	113.4	5L	68.5
A14	44.8	6L	41.0
C2	249.2	7B	56.3
C13	136.8	8C	202.3
C14	114.7	8G	150.6
3(AB)	50.6	8I	112.5
3C	78.6	8K	107.6
3(DE)	81.2	8L	56.3
3F	103.6	9B	80.3
3J	147.5	$P_{story} =$	2663.0
		$p_{mf} =$	1474.0
		R_m	0.92

Table 21. Column Load Summary @ 4th Floor

See hand calculation attached.

Member Spot Checks for Lateral Load

- Columns are checked using the Direct Analysis Method and the Effect Length Method.
- Controlling Load Combination: $1.2D + 0.5L + 1.0E_x$



Info.

Column A: W14x398, $\phi P_n = 4780^k$, $I_x = 6000 \text{ in}^4$

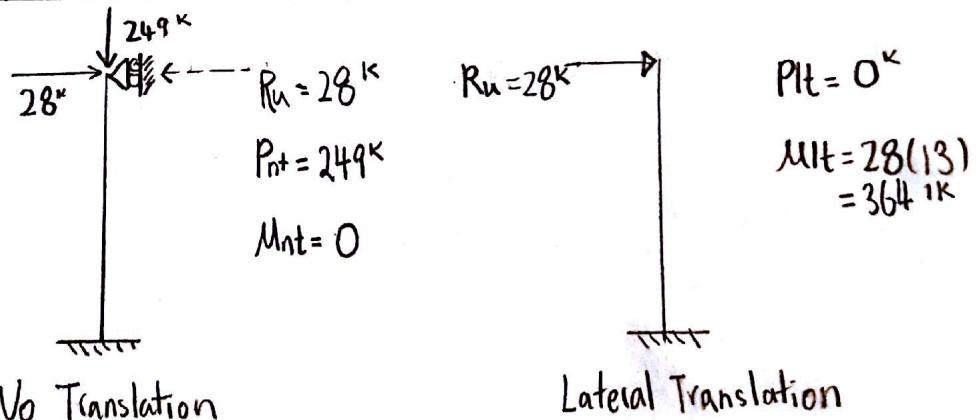
Column B: W14x370, $\phi P_n = 4450^k$, $I_x = 5440 \text{ in}^4$

Determine Lateral Load: (Assume $B_2 < 1.7$ & $P_r/P_y < 0.5$)

$$H = \frac{28^k}{\text{mod } N_i} = 0.002 \text{ and } Y_i = 0.002(1.0)(249 + 249) = 1.0^k$$

$$\therefore H = 28^k$$

Determine Moments and Axial loads:



Check Limitations DAM:

No Limitation

Check Limitations ELM:

- Columns are vertical: OK
- $B_2 \leq 1.5$, assume, then conform

Determine B_2 DAM:

$$R_m = 0.92 \text{ (From the table above)}$$

$$\begin{aligned} \Delta_H &= \frac{P l^3}{3EI} \\ &= \frac{28(13)^3(1728)}{3(29000)(10.8)(6000)} \\ &= .25 \end{aligned}$$

$$\begin{aligned} P_{shay} &= R_m \frac{H_L}{\Delta H} \\ &= 0.92 \left(\frac{28(13)(12)}{0.25} \right) \\ &= 16072 \end{aligned}$$

$$\begin{aligned} B_2 &= \frac{1}{1 - \frac{P_{shay}}{P_{stony}}} \\ &= \frac{1}{1 - \frac{2663}{16072}} \\ &= 1.20 \end{aligned}$$

$$\begin{aligned} \Delta_H &= \frac{P l^3}{3EI} \\ &= \frac{28(13)^3(1728)}{3(29000)(6000)} \\ &= .20 \end{aligned}$$

$$\begin{aligned} P_{shay} &= \frac{R_m H_L}{\Delta H} \\ &= 0.92 \left(\frac{28(13)(12)}{.2} \right) \\ &= 20093 \end{aligned}$$

$$\begin{aligned} B_2 &= \frac{1}{1 - \frac{P_{shay}}{P_{stony}}} \\ &= \frac{1}{1 - \frac{2663}{20093}} \\ &= 1.15 < 1.5 \text{ method OK} \\ &> 1.1 \therefore K+1.0 \end{aligned}$$

Determine Required Axial Load and Moment:

$$\begin{aligned} P_r &= P_{nt} + B_2 P_{lt} \\ &= 249 + 1.2(0) = 249 \text{ kN} \end{aligned}$$

$$\begin{aligned} M_r &= B_1 M_{nt} + B_2 M_{lt} \\ &= B_1(0) + 1.2(364) = 436.8 \text{ kNm} \end{aligned}$$

$$P_r = 249 + 1.1(0) = 249$$

$$\begin{aligned} M_r &= B_1(0) + 1.15(364) \\ &= 418.6 \text{ kNm} \end{aligned}$$

Check: $\alpha P_r \leq 0.5 P_y$
 $249 \leq 0.5(117)(50) = 1170$
 $\therefore 0.8 EI$ assumption OK

Determine Available Strength: (DAM & ELM)

$$K = 1$$

$$KL = 13; P_c = 4780^k \text{ (Table 4-1)}$$

$$M_c = 3000 \text{ (Table 3-2)}$$

$$L_p = 15.2' > 13'$$

$$K_x = 2.0 \quad k_y = 1.0$$

$$k^* = 2.0 \sqrt{1 + \frac{2663 - 1474}{1474}} \\ = 2.69$$

$$\frac{k^* L_x}{T_x / T_1} = \frac{2.69(13)}{1.66} = 21.1 > k_y l_y = 13$$

$$P_c = 4000^k @ 22'$$

Calculate Interaction: (DAM & ELM)

$$\frac{P_r}{P_c} = \frac{249}{4780} = 0.05 < 0.2$$

$$\frac{0.05}{2} + \left(\frac{436.8}{3000}\right) = .17 < 1.0 \quad \frac{0.06}{2} + \frac{418.6}{3000} = .17 < 1.0$$

- The columns are over designed for the stability. However, it might be designed to meet drift requirements.

So, check the Drift Controlled Case.

$$H/\Delta H_{wind}(x) = 67.8^k / (L/192) = 13017.6/L$$

$$H/\Delta \text{seismic}(x) = 134^k / (L/120) = 16080/L \rightarrow \text{critical}$$

$$P_{\text{story}} = 0.92 \left(\frac{16080}{L} \right) L = 14793.6$$

$$B_2 = \frac{1}{1 - \frac{2663}{14793.6}} = 1.22$$

→ B_2 doesn't change that much. Therefore, the columns are still over designed.

Appendix A: 1.Manual Calculation of Center of Rigidity

Center of Mass (Fourth floor)							
ELEMENT LABEL	LENGTH (ft)	LINEAR W (plf)	W (k)	DIS. FROM REF. DATUM		W*X	W*Y
				X	Y		
W16x31	16.5	31	511.5	8.25	0	4219.875	0
W24x68	10.5	68	714	0	5.25	0	3748.5
W12x19	10.5	19	199.5	8.25	5.25	1645.875	1047.375
W24x68	10.5	68	714	16.5	5.25	11781	3748.5
W14x455	16.5	455	7507.5	8.25	10.5	61936.88	78828.75
W24x68	10	68	680	0	10.25	0	6970
W12x19	10	19	190	8.26	10.25	1569.4	1947.5
W24x68	10	68	680	16.5	10.25	11220	6970
W18x46	3	46	138	1.5	20.5	207	2829
W18x40	13.5	40	540	9.75	20.5	5265	11070
W24x55	11.25	55	618.75	22.125	20.5	13689.84	12684.38
W24x55	10.42	55	573.1	32.96	20.5	18889.38	11748.55
W16x31	9.17	31	284.27	42.755	20.5	12153.96	5827.535
W18x46	18.67	46	858.82	57.175	20.5	49103.03	17605.81
W21x93	20.17	93	1875.81	76.595	20.5	143677.7	38454.11
W14X34	15	34	510	0	28	0	14280
C10x15.3	7	15.3	107.1	3	24	321.3	2570.4
C10x15.3	8.5	15.3	130.05	4.25	27.5	552.7125	3576.375
W12x19	12	19	228	8.5	26.5	1938	6042
W12x19	8.5	19	161.5	4.25	32.5	686.375	5248.75
W33x118	16	118	1888	0	43.5	0	82128
HSS8x4x3/8	8.5	27.48	233.58	4.25	35.5	992.715	8292.09
HSS8x4x1/2	8.5	35.24	299.54	4.25	44.5	1273.045	13329.53
HSS8x4x1/2	10	35.24	352.4	5	51.5	1762	18148.6
W16x26	19	26	494	8.5	42	4199	20748
W18x71	38	71	2698	16.5	39.5	44517	106571
W12x19	16.5	19	313.5	24.75	50.25	7759.125	15753.38
W18x50	16.5	50	825	24.75	34.5	20418.75	28462.5
W12X19	5	19	95	24.75	23	2351.25	2185
W12X19	5	19	95	27.75	23	2636.25	2185
W18x65	38	65	2470	33	39.5	81510	97565
W18X50	38	50	1900	38.17	39.5	72523	75050
W18x35	29	35	1015	45.33	44	46009.95	44660
W16x31	9.17	31	284.27	42.75	29.5	12152.54	8385.965
C10x15.3	9.17	15.3	140.301	42.75	24.5	5997.868	3437.375
W24x55	5	55	275	51.34	27	14118.5	7425
W24x55	4	55	220	51.34	22.5	11294.8	4950
W18x35	29	35	1015	53.33	44	54129.95	44660

W18x35	29	35	1015	61.33	44	62249.95	44660
W18x55	18.67	55	1026.85	57.67	29	59218.44	29778.65
W12x19	9	19	171	51.33	14.5	8777.43	2479.5
W12x19	9	19	171	66	14.5	11286	2479.5
W18x119	38	119	4522	68	29	307496	131138
W18x119	38	119	4522	76	29	343672	131138
W12x19	10.17	19	193.23	81.1	44.5	15670.95	8598.735
W12x19	10.17	19	193.23	81.1	35.5	15670.95	6859.665
W14x68	15	68	1020	86.18	55	87903.6	56100
W14x68	9	68	612	86.18	40	52742.16	24480
W14x34	14	34	476	86.18	28	41021.68	13328
W24x62	11	62	682	0	57	0	38874
W18x35	11	35	385	10	57	3850	21945
W18x35	8.5	35	297.5	14.25	58.5	4239.375	17403.75
W24x55	26.84	55	1476.2	31.92	58.5	47120.3	86357.7
W16x31	16	31	496	53.34	58.5	26456.64	29016
W18x40	14.67	40	586.8	68.68	58.5	40301.42	34327.8
W16x31	10.17	31	315.27	81.1	58.5	25568.4	18443.3
W24x55	10	55	550	5	62.5	2750	34375
W12X30	6.5	30	195	13.25	62.5	2583.75	12187.5
W21x62	26	62	1612	0	75.5	0	121706
W12x19	8	19	152	4	66.5	608	10108
W12x19	17	19	323	8	73	2584	23579
W12x19	8.5	19	161.5	12.25	70.5	1978.375	11385.75
W18x40	30	40	1200	16.5	73.5	19800	88200
W14x22	16.5	22	363	8.25	88.5	2994.75	32125.5
W12x19	9	19	171	8	84	1368	14364
W12x40	10.5	40	420	0	93.75	0	39375
W12x19	10.5	19	199.5	8	93.75	1596	18703.13
W12x19	10.5	19	199.5	16.5	93.75	3291.75	18703.13
W14x211	16.5	211	3481.5	8.25	99	28722.38	344668.5
W14x398	6.5	398	2587	0	10.5	0	27163.5
W14x370	6.5	370	2405	16.5	10.5	39682.5	25252.5
W14x455	5.63	455	2559.375	0	10.5	0	26873.44
W14x398	5.63	398	2238.75	16.5	10.5	36939.38	23506.88
W12x40	12.13	40	485	1.5	20.5	727.5	9942.5
W12x190	12.13	190	2303.75	16.5	20.5	38011.88	47226.88
W12x190	12.13	190	2303.75	27.25	20.5	62777.19	47226.88
W12x190	12.13	190	2303.75	38.17	20.5	87934.14	47226.88
W12x53	12.13	53	642.625	66	20.5	42413.25	13173.81
W14x68	12.13	68	824.5	86.18	20.5	71055.41	16902.25
W12x190	12.13	190	2303.75	47.42	23.5	109243.8	54138.13
W12x190	12.13	190	2303.75	47.42	29.5	109243.8	67960.63
W10x54	5.63	54	303.75	68.01	29.5	20658.04	8960.625

W14x176	12.13	176	2134	0	35.5	0	75757
W14x176	12.13	176	2134	0	51.5	0	109901
W14x176	12.13	176	2134	0	62.5	0	133375
W10x60	12.13	60	727.5	0	88.5	0	64383.75
W12x190	12.13	190	2303.75	0	99	0	228071.3
W10x49	12.13	49	594.125	16.5	88.5	9803.063	52580.06
W12x190	12.13	190	2303.75	16.5	99	38011.88	228071.3
W14x159	12.13	159	1927.875	10	62.5	19278.75	120492.2
W10x33	12.13	33	400.125	10	51.5	4001.25	20606.44
W12x53	12.13	53	642.625	18.5	58.5	11888.56	37593.56
W12x45	12.13	45	545.625	45.34	58.5	24738.64	31919.06
W12x45	12.13	45	545.625	61.34	58.5	33468.64	31919.06
W10x33	12.13	33	400.125	76.01	58.5	30413.5	23407.31
W10x39	12.13	39	472.875	86.18	58.5	40752.37	27663.19
W14x68	12.13	68	824.5	86.18	35.5	71055.41	29269.75
W14x68	12.13	68	824.5	86.18	44.5	71055.41	36690.25
W1	99	1180	116820	0	49.5	0	5782590
W2	16.5	1180	19470	8.25	99	160627.5	1927530
W3	40.5	1180	47790	16.5	78.75	788535	3763463
W4	69.68	1180	82222.4	51.34	58.5	4221298	4810010
W5	38	1180	44840	86.18	39.5	3864311	1771180
W6	69.68	1180	82222.4	51.35	20.5	4222120	1685559
W7	20.5	1180	24190	16.5	10.25	399135	247947.5
W8	16.5	1180	19470	8.25	0	160627.5	0
Deck	Area (ft^2)	Weight (psf)					
D 1	1633.5	61	99643.5	8.25	49.5	822058.9	822058.9
D2	2647.84	61	161518.2	51.345	39.5	8293154	8293154
Opening 1	104	-61	-6344	4	73	-25376	-463112
Opening 2	100	-61	-6100	5	57	-30500	-347700
Opening 3	59.5	-61	-3629.5	4.25	48	-15425.4	-174216
Opening 4	76.5	-61	-4666.5	4.25	40	-19832.6	-186660
Opening 5	21	-61	-1281	1.5	24	-1921.5	-30744
Opening 6	272.25	-61	-16607.3	24.75	33.75	-411029	-560495
Opening 7	33.67	-61	-2053.87	42.75	22.5	-87802.9	-46212.1
Opening 8	132.06	-61	-8055.66	58.67	25	-472626	-201392
	$\sum W$	750953.1				24784534	31037695
						$\sum Wx$	$\sum Wy$

COM (X)=	$\frac{\sum W \cdot x}{\sum W}$	=	$\frac{24784534}{750953.1}$	=	33.00
COM (Y)=	$\frac{\sum W \cdot y}{\sum W}$	=	$\frac{31037695}{750953.1}$	=	41.33

2. Base Reaction

Table 5.1 - Base Reactions

Load Case/Combo	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft	X ft	Y ft	Z ft
Seismic X 1	-176.892	0	0	1.016E-05	-8146.6885	7849.6807	0	0	0
Seismic X 2	-176.892	0	0	1.523E-05	-8146.6885	8725.2948	0	0	0
Seismic X 3	-176.892	0	0	5.101E-06	-8146.6885	6974.0666	0	0	0
Seismic Y 1	0	-176.892	0	8146.6885	-1.589E-06	-5547.8355	0	0	0
Seismic Y 2	0	-176.892	0	8146.6885	-2.086E-06	-6310.062	0	0	0
Seismic Y 3	0	-176.892	0	8146.6885	-1.091E-06	-4785.609	0	0	0
Wind 1	-115.468	0	0	8.619E-06	-4591.9604	5715.6526	0	0	0
Wind 2	0	-98.187	0	3912.6132	-1.37E-06	-4230.8752	0	0	0
Wind 3	-86.601	0	0	0	-3443.9703	3000.7176	0	0	0
Wind 4	-86.601	0	0	1.288E-05	-3443.9703	5572.7613	0	0	0
Wind 5	0	-73.64	0	2934.4599	-1.561E-06	-4125.1033	0	0	0
Wind 6	0	-73.64	0	2934.4599	0	-2221.2095	0	0	0
Wind 7	-86.601	75.386	0	-2997.9936	-3443.9703	7535.1408	0	0	0
Wind 8	-84.595	-73.64	0	2934.4599	-3370.9855	1014.2864	0	0	0
Wind 9	-65.008	56.59	0	-2250.4938	-2585.2737	3959.4653	0	0	0
Wind 10	-65.008	56.59	0	-2250.4938	-2585.2737	7353.2927	0	0	0
Wind 11	-63.502	-55.279	0	2202.8012	-2530.4864	-896.2159	0	0	0
Wind 12	-63.502	-55.279	0	2202.8012	-2530.4864	2418.9979	0	0	0

3. Centers of Mass and Rigidity

Table 5.2 - Centers of Mass and Rigidity

Story	Diaphragm	Mass X lb-s ² /ft	Mass Y lb-s ² /ft	XCM ft	YCM ft	Cumulative X lb-s ² /ft	Cumulative Y lb-s ² /ft	XCCM ft	YCCM ft	XCR ft	YCR ft
Story5	Rigid	18560.49	18560.49	32.6541	43.9976	18560.49	18560.49	32.6541	43.9976	22.6269	34.9385
Story4	Rigid	18664.67	18664.67	26.8543	44.8081	37225.16	37225.16	29.7461	44.404	23.8529	35.1613
Story3	Rigid	25114.9	25114.9	33.3829	44.3564	62340.05	62340.05	31.2112	44.3848	24.8758	35.4993
Story2	Rigid	25135.17	25135.17	33.1723	44.1149	87475.23	87475.23	31.7747	44.3072	26.3544	35.8567
Story1	Rigid	25049.1	25049.1	30.4372	44.978	112524.33	112524.33	31.477	44.4566	27.61	36.0639

4. ASCE 7-02 Auto Wind Load Calculation

This calculation presents the automatically generated lateral wind loads for load pattern Wind according to ASCE 7-02, as calculated by ETABS.

Exposure Parameters

Exposure From = Diaphragms

Exposure Category = B

Wind Direction = 0 degrees

Basic Wind Speed, V [ASCE 6.5.4] $V = 98 \text{ mph}$

Windward Coefficient, $C_{p,wind}$ [ASCE 6.5.11.2.1] $C_{q,wind} = 0.8$

Leeward Coefficient, $C_{p,lee}$ [ASCE 6.5.11.2.1] $C_{q,lee} = 0.5$

Wind Case = All Cases

Top Story = Story5

Bottom Story = Base

Include Parapet = Yes, Parapet Height = 2

Factors and Coefficients

Gradient Height, z_g [ASCE Table 6-2] $z_g = 900$

Empirical Exponent, α [ASCE Table 6-2] $\alpha = 9.5$

Velocity Pressure Exposure Coefficient, K_z [ASCE Table 6-3]
$$K_z = 2.01 \left(\frac{z}{z_g} \right)^{\frac{15}{z_g}} \text{ for } 15 \text{ ft} \leq z \leq z_g$$

$$K_z = 2.01 \left(\frac{z}{z_g} \right)^{\frac{15}{z_g}} \text{ for } z > 15 \text{ ft}$$

Topographical Factor, K_{zt} [ASCE 6.5.7.2] $K_{zt} = 1$

Directionality Factor, K_d [ASCE 6.5.4.4] $K_d = 0.85$

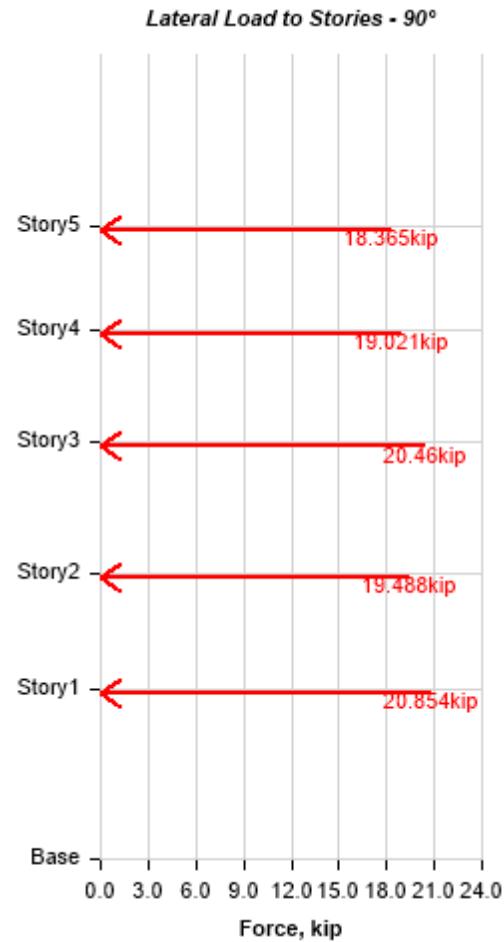
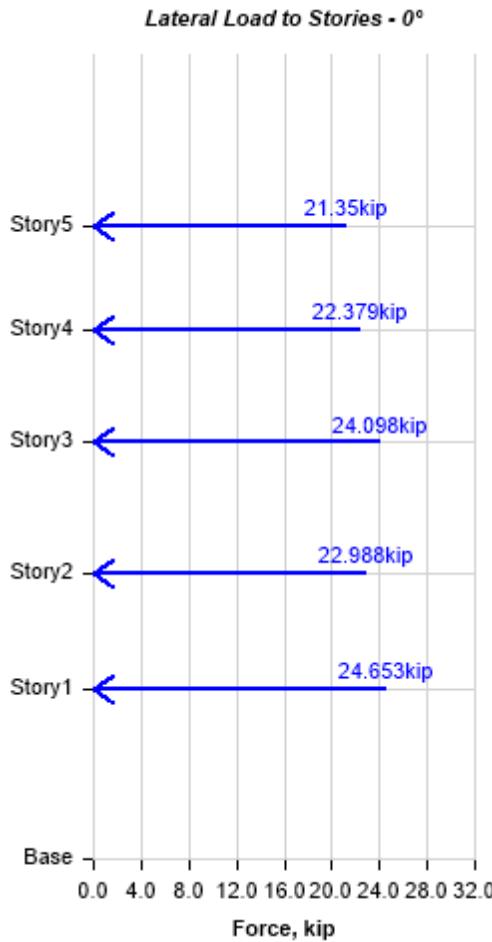
Importance Factor, I [ASCE 6.5.5] $I = 1$

Gust Effect Factor, G [ASCE 6.5.8] $G = 0.92$

Lateral Loading

Velocity Pressure, q_z [ASCE 6.5.10 Eq. 6-15] $q_z = 0.00256 K_z K_{zt} K_d V^2 I$

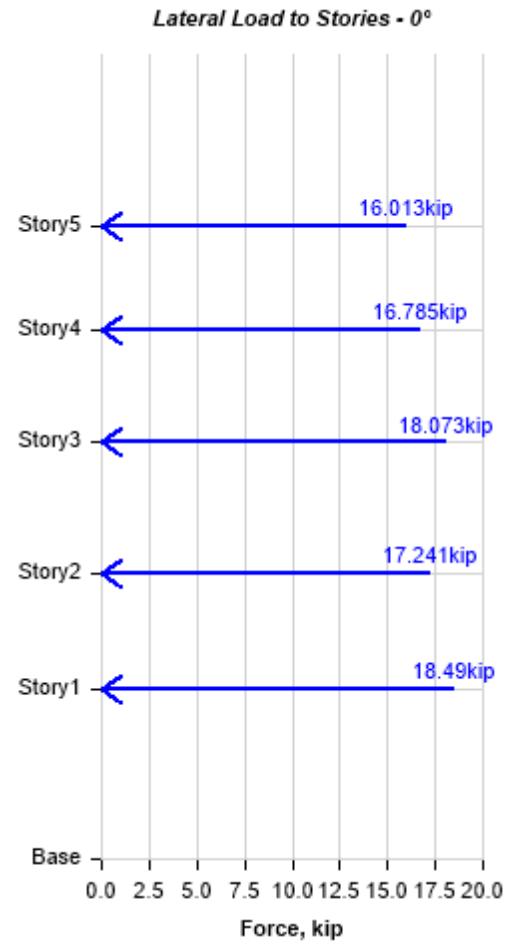
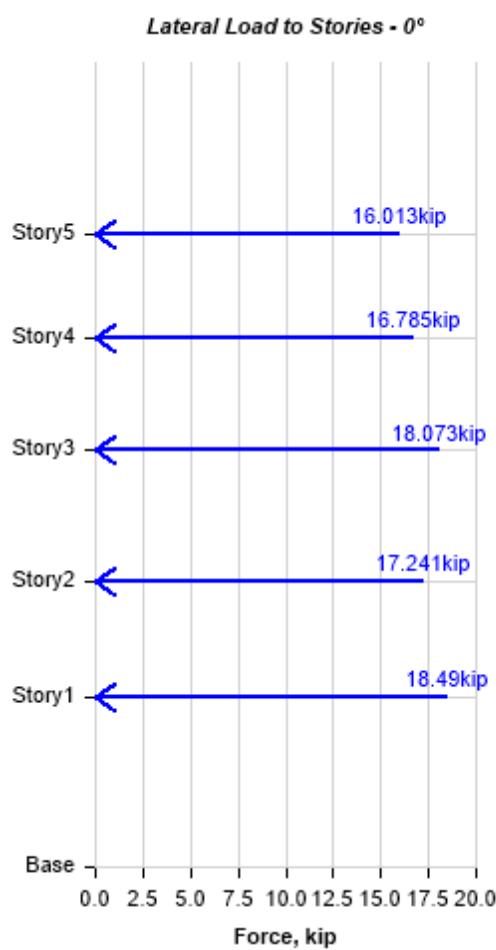
Design Wind Pressure, p [ASCE 6.5.12.2.1 Eq. 6-17] $p = q G C_{p,wind} + q_h (G C_{p,lee})$

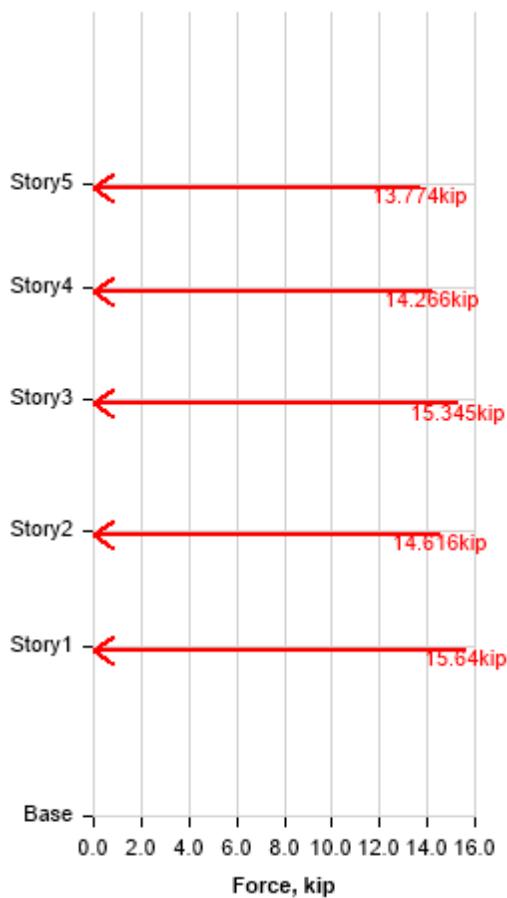
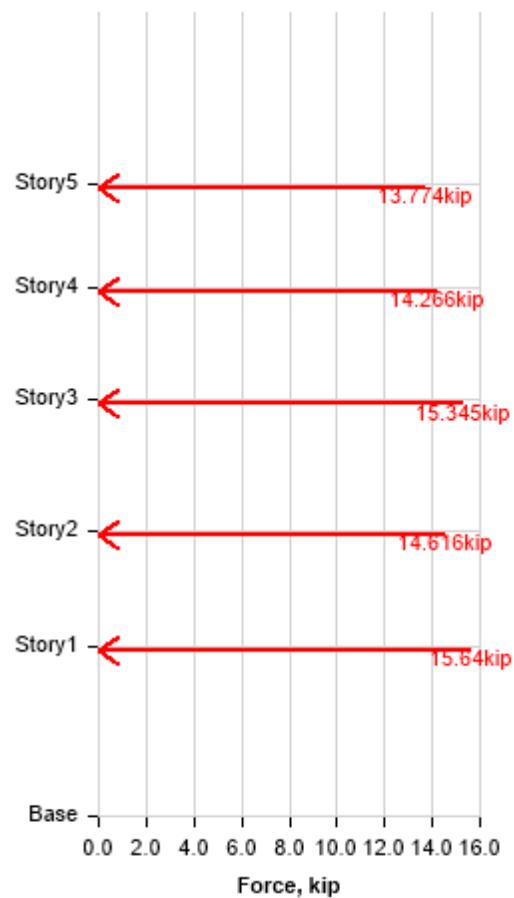
Applied Story Forces**Wind Load Case 1:**

Story	Elevation ft	X-Dir kip	Y-Dir kip
Story5	62.9967	21.35	0
Story4	52.6667	22.379	0
Story3	41.4167	24.098	0
Story2	28.4167	22.988	0
Story1	16.9167	24.653	0
Base	0	0	0

Story	Elevation ft	X-Dir kip	Y-Dir kip
Story5	62.9967	0	18.365
Story4	52.6667	0	19.021
Story3	41.4167	0	20.46
Story2	28.4167	0	19.488
Story1	16.9167	0	20.854
Base	0	0	0

Wind Load Case 2:

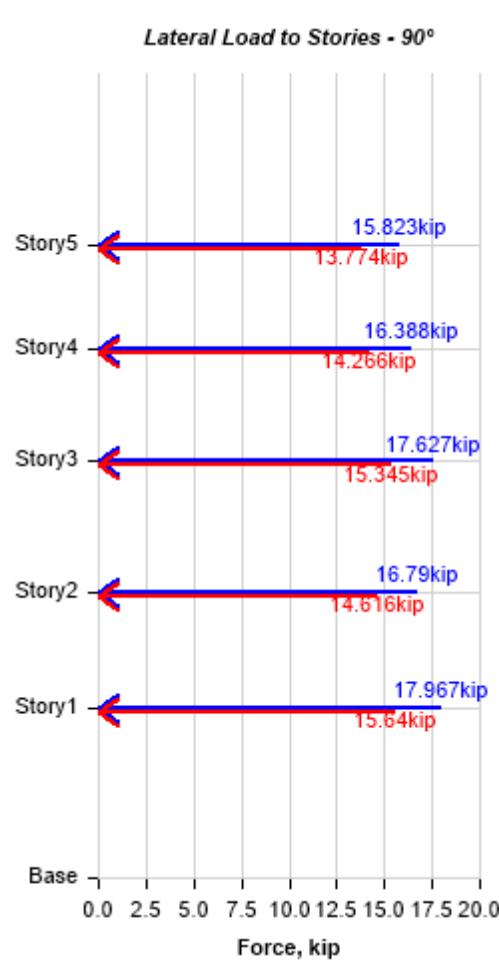
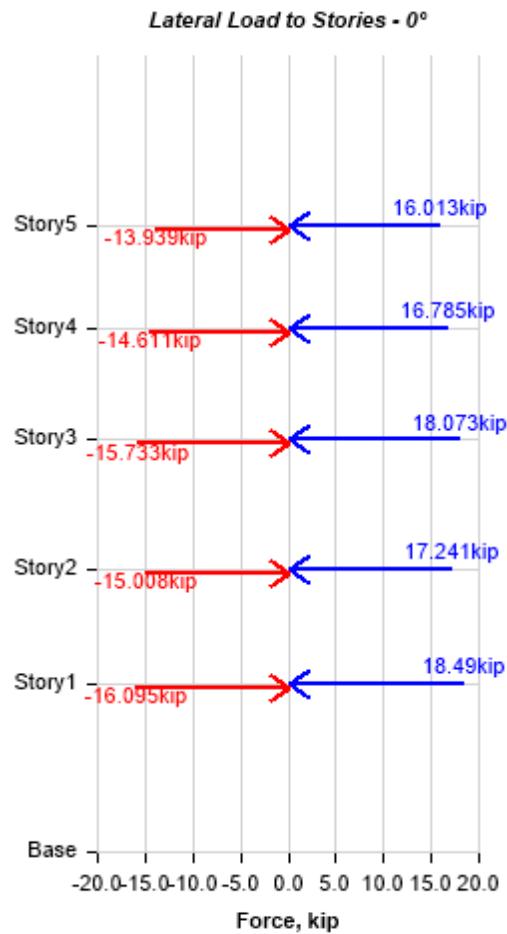


Lateral Load to Stories - 90°*Lateral Load to Stories - 90°*

Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Story 5	62.9967	0	13.774
Story 4	52.6667	0	14.266
Story 3	41.4167	0	15.345
Story 2	28.4167	0	14.616
Story 1	16.9167	0	15.64
Base	0	0	0

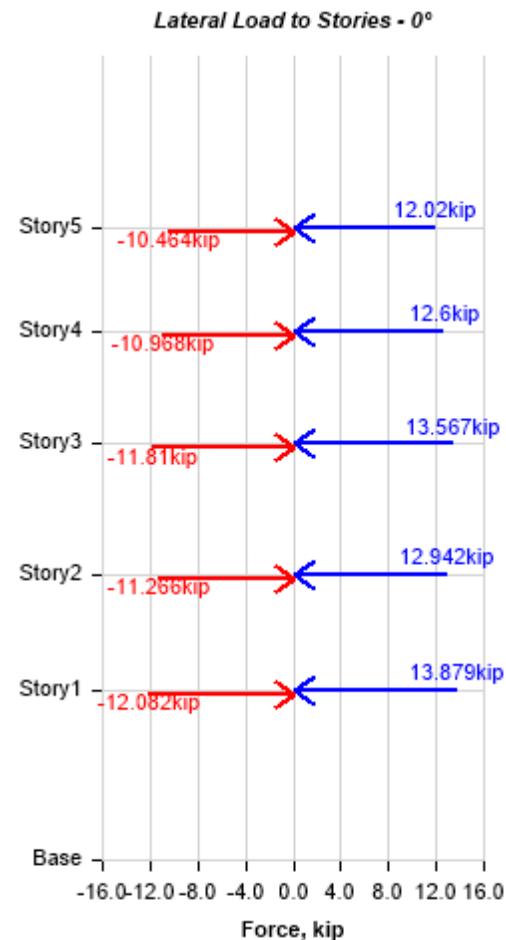
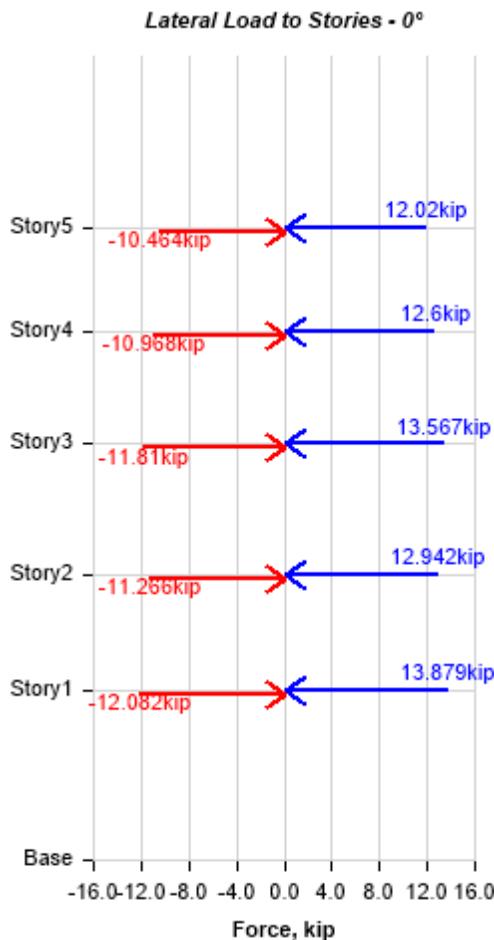
Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Story 5	62.9967	0	13.774
Story 4	52.6667	0	14.266
Story 3	41.4167	0	15.345
Story 2	28.4167	0	14.616
Story 1	16.9167	0	15.64
Base	0	0	0

Wind Load Case 3:

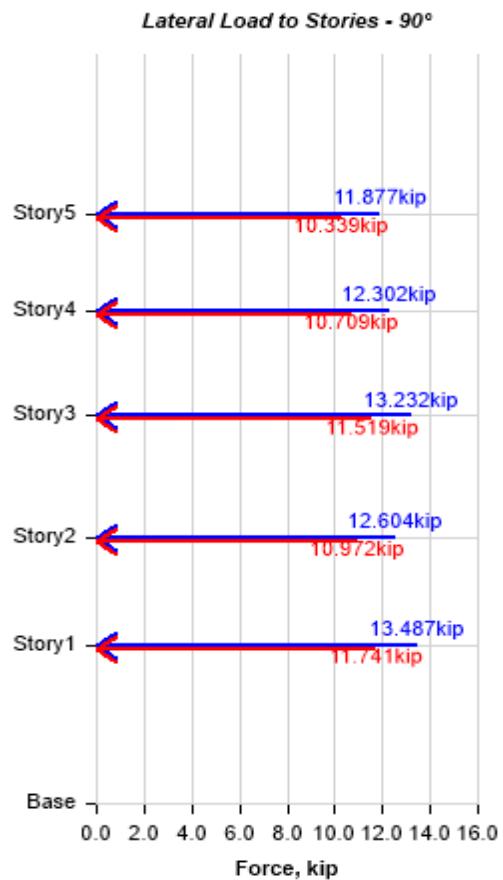


Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Story5	62.9967	16.013	-13.939
Story4	52.6667	16.785	-14.611
Story3	41.4167	18.073	-15.733
Story2	28.4167	17.241	-15.008
Story1	16.9167	18.49	-16.095
Base	0	0	0

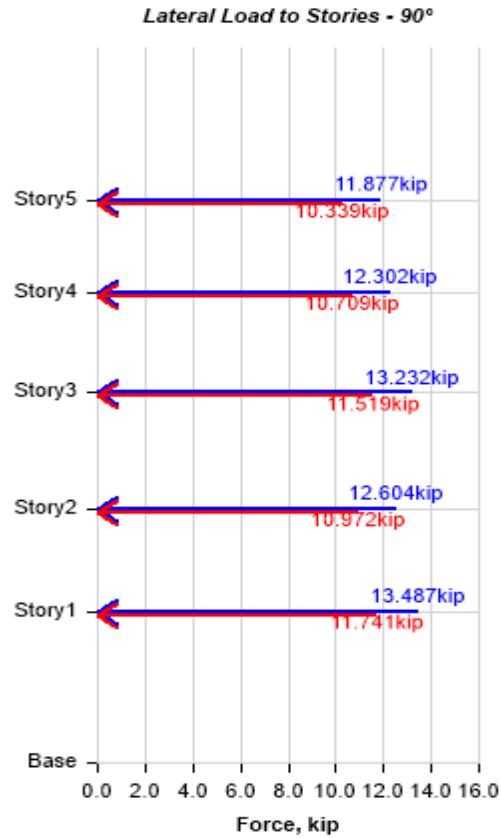
Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Story5	62.9967	15.823	13.774
Story4	52.6667	16.388	14.266
Story3	41.4167	17.627	15.345
Story2	28.4167	16.79	14.616
Story1	16.9167	17.967	15.64
Base	0	0	0



Wind Load Case 4:



Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Story5	62.9967	11.877	10.339
Story4	52.6667	12.302	10.709
Story3	41.4167	13.232	11.519
Story2	28.4167	12.604	10.972
Story1	16.9167	13.487	11.741
Base	0	0	0



Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Story5	62.9967	11.877	10.339
Story4	52.6667	12.302	10.709
Story3	41.4167	13.232	11.519
Story2	28.4167	12.604	10.972
Story1	16.9167	13.487	11.741
Base	0	0	0

5. ASCE 7-02 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern Seismic X according to ASCE 7-02, as calculated by ETABS.

Direction and Eccentricity

Direction = Multiple

Eccentricity Ratio = 5% for all diaphragms

Structural Period

Period Calculation Method = User Specified

User Period $T = 0.775 \text{ sec}$

Factors and Coefficients

Seismic Group = I

Response Modification Factor, R $R = 3$

Response Acceleration, S_s $S_s = 0.365g$

Response Acceleration, S_1 $S_1 = 0.071g$

Site Class = D - Stiff Soil

Site Coefficient, F_a $F_a = 1.508$

Site Coefficient, F_v $F_v = 2.4$

Seismic Response

MCE Spectral Response Acceleration, S_{MS} $S_{MS} = F_a S_s$ $S_{MS} = 0.55042g$

MCE Spectral Response Acceleration, S_{M1} $S_{M1} = F_v S_1$ $S_{M1} = 0.1704g$

Design Spectral Response Acceleration, S_{DS} $S_{DS} = \frac{2}{3} S_{MS}$ $S_{DS} = 0.366947g$

Design Spectral Response Acceleration, S_{D1} $S_{D1} = \frac{2}{3} S_{M1}$ $S_{D1} = 0.1136g$

Equivalent Lateral Forces

Seismic Response Coefficient, C_s [ASCE Eq. 9.5.5.2.1-1]

[ASCE Eq. 9.5.5.2.1-2]

[ASCE Eq. 9.5.5.2.1-3]

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

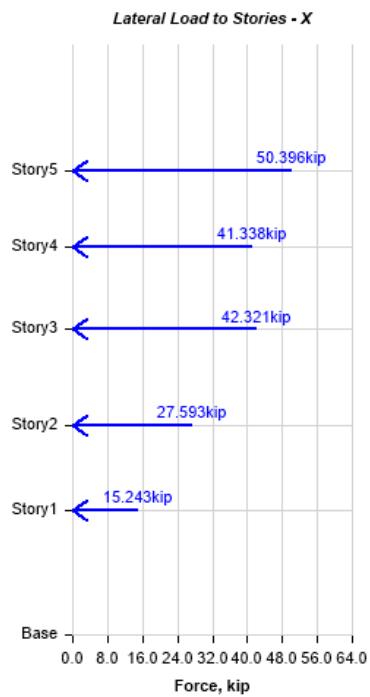
$$C_{s,max} = \frac{S_{D1}}{T\left(\frac{R}{I}\right)}$$

$$C_{s,min} = 0.044 S_{DS} I = 0.016146$$

$$C_{s,min} \leq C_s \leq C_{s,max}$$

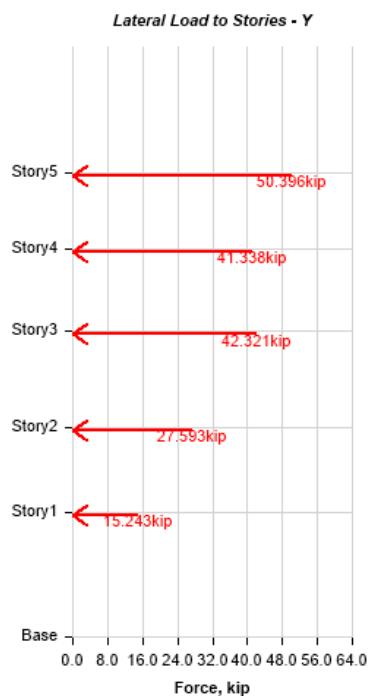
Calculated Base Shear

Direction	Period Used (sec)	C_s	W (kip)	V (kip)
X	0.775	0.04886	3620.3634	176.8917
X + Ecc. Y	0.775	0.04886	3620.3634	176.8917
X - Ecc. Y	0.775	0.04886	3620.3634	176.8917

Applied Story Forces

Story	Elevation	X-Dir	Y-Dir
	ft	kip	kip
Story 5	62.9967	50.396	0
Story 4	52.6667	41.338	0
Story 3	41.4167	42.321	0
Story 2	28.4167	27.593	0
Story 1	16.9167	15.243	0
Base	0	0	0

Direction	Period Used (sec)	C_s	W (kip)	V (kip)
Y	0.775	0.04886	3620.3634	176.8917
Y + Ecc. X	0.775	0.04886	3620.3634	176.8917
Y - Ecc. X	0.775	0.04886	3620.3634	176.8917

Applied Story Forces

Story	Elevation ft	X-Dir kip	Y-Dir kip
Story5	62.9967	0	50.396
Story4	52.6667	0	41.338
Story3	41.4167	0	42.321
Story2	28.4167	0	27.593
Story1	16.9167	0	15.243
Base	0	0	0

Appendix B:

1. Base Reaction

Table 5.1 - Base Reactions

Load Case/Combo	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft	X ft	Y ft	Z ft
Seismic X 1	-176.892	0	0	1.016E-05	-8146.6885	7849.6807	0	0	0
Seismic X 2	-176.892	0	0	1.523E-05	-8146.6885	8725.2948	0	0	0
Seismic X 3	-176.892	0	0	5.101E-06	-8146.6885	6974.0666	0	0	0
Seismic Y 1	0	-176.892	0	8146.6885	-1.589E-06	-5547.8355	0	0	0
Seismic Y 2	0	-176.892	0	8146.6885	-2.086E-06	-6310.062	0	0	0
Seismic Y 3	0	-176.892	0	8146.6885	-1.091E-06	-4785.609	0	0	0
Wind 1	-115.468	0	0	8.619E-06	-4591.9604	5715.6526	0	0	0
Wind 2	0	-98.187	0	3912.6132	-1.37E-06	-4230.8752	0	0	0
Wind 3	-86.601	0	0	0	-3443.9703	3000.7176	0	0	0
Wind 4	-86.601	0	0	1.288E-05	-3443.9703	5572.7613	0	0	0
Wind 5	0	-73.64	0	2934.4599	-1.561E-06	-4125.1033	0	0	0
Wind 6	0	-73.64	0	2934.4599	0	-2221.2095	0	0	0
Wind 7	-86.601	75.386	0	-2997.9936	-3443.9703	7535.1408	0	0	0
Wind 8	-84.595	-73.64	0	2934.4599	-3370.9855	1014.2864	0	0	0
Wind 9	-65.008	56.59	0	-2250.4938	-2585.2737	3959.4653	0	0	0
Wind 10	-65.008	56.59	0	-2250.4938	-2585.2737	7353.2927	0	0	0
Wind 11	-63.502	-55.279	0	2202.8012	-2530.4864	-896.2159	0	0	0
Wind 12	-63.502	-55.279	0	2202.8012	-2530.4864	2418.9979	0	0	0

2. Point Results

Table 5.6 - Joint Reactions

Story	Joint Label	Unique Name	Load Case/Combo	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft
Base	1	581	Wind 1	-9.224	-0.279	-48.436	3.5004	-99.9587	0.0202
Base	1	581	Wind 2	2.622	-0.496	20.015	8.5039	29.5265	-0.0201
Base	1	581	Wind 3	-4.614	0.02	-24.261	-0.3267	-50.1537	-0.0014
Base	1	581	Wind 4	-9.222	-0.439	-48.393	5.5773	-99.7843	0.0317
Base	1	581	Wind 5	3.671	-0.202	23.97	4.1926	40.5165	-0.0273
Base	1	581	Wind 6	0.261	-0.542	6.053	8.5632	3.7733	-0.0028
Base	1	581	Wind 7	-8.931	0.172	-51.644	-3.9034	-97.6345	0.0306
Base	1	581	Wind 8	-4.791	-0.577	-20.579	8.9426	-51.1013	-0.0003
Base	1	581	Wind 9	-3.664	0.432	-22.846	-6.8254	-40.5471	0.0011
Base	1	581	Wind 10	-9.744	-0.174	-54.689	0.965	-106.0349	0.0448
Base	1	581	Wind 11	-0.627	-0.137	0.152	2.9076	-6.37	-0.0215
Base	1	581	Wind 12	-6.566	-0.729	-31.047	10.5181	-70.3502	0.0211
Base	5	874	Wind 1	-1.584	-0.082	-7.444E-05	1.0343	-22.0454	0.0009
Base	5	874	Wind 2	0.369	-0.146	3.652E-05	2.5094	5.6382	-0.0009
Base	5	874	Wind 3	-0.835	0.006	-4.802E-05	-0.0967	-11.7067	-0.0001
Base	5	874	Wind 4	-1.541	-0.13	-6.364E-05	1.6482	-21.3614	0.0014
Base	5	874	Wind 5	0.538	-0.059	3.32E-05	1.2362	7.8022	-0.0012
Base	5	874	Wind 6	0.016	-0.16	2.159E-05	2.5279	0.655	-0.0001
Base	5	874	Wind 7	-1.472	0.05	-8.377E-05	-1.1508	-20.8625	0.0014
Base	5	874	Wind 8	-0.883	-0.17	-2.732E-05	2.6398	-11.9242	-1.299E-05
Base	5	874	Wind 9	-0.639	0.127	-5.258E-05	-2.0151	-9.2911	0.0001

Story	Joint Label	Unique Name	Load Case/Combo	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft
Base	5	874	Wind 10	-1.57	-0.052	-7.319E-05	0.2873	-22.0305	0.002
Base	5	874	Wind 11	-0.208	-0.04	-1.04E-05	0.857	-2.7285	-0.001
Base	5	874	Wind 12	-1.117	-0.215	-3.062E-05	3.1062	-15.1737	0.0009
Base	9	877	Wind 1	-9.451	-3.244	-39.672	34.9092	-96.5562	0.0919
Base	9	877	Wind 2	1.366	-7.401	50.442	91.6938	14.6731	-0.0915
Base	9	877	Wind 3	-5.881	0.219	-30.514	-3.1105	-60.1555	-0.0064
Base	9	877	Wind 4	-8.296	-5.084	-28.993	55.4743	-84.6788	0.1442
Base	9	877	Wind 5	1.919	-3.589	37.269	47.0837	20.083	-0.1244
Base	9	877	Wind 6	0.131	-7.514	38.393	90.457	1.9266	-0.0129
Base	9	877	Wind 7	-8.138	3.251	-68.374	-44.2023	-83.6802	0.1391
Base	9	877	Wind 8	-5.9	-7.928	8.695	94.3499	-59.7514	-0.0013
Base	9	877	Wind 9	-4.515	5.938	-52.33	-71.8327	-46.6367	0.0051
Base	9	877	Wind 10	-7.702	-1.058	-50.323	5.4704	-78.9952	0.2038
Base	9	877	Wind 11	-2.872	-2.534	5.548	33.0628	-29.0457	-0.098
Base	9	877	Wind 12	-5.986	-9.368	7.506	108.5878	-60.661	0.096
Base	10	898	Wind 1	-4.743	-4.238	-3.782	40.1288	-67.4989	0.0919
Base	10	898	Wind 2	0.42	-11	4.136	110.5775	6.6142	-0.0915
Base	10	898	Wind 3	-3.174	0.361	-2.659	-3.8606	-45.2413	-0.0064
Base	10	898	Wind 4	-3.941	-6.719	-3.014	64.0537	-56.0071	0.1442
Base	10	898	Wind 5	0.599	-5.629	3.233	57.7917	8.9457	-0.1244
Base	10	898	Wind 6	0.031	-10.871	2.971	108.0745	0.9756	-0.0129
Base	10	898	Wind 7	-3.88	5.265	-5.996	-54.7746	-55.7013	0.1391
Base	10	898	Wind 8	-3.16	-11.355	0.322	112.3381	-44.4986	-0.0013
Base	10	898	Wind 9	-2.406	8.624	-4.267	-85.9242	-34.7103	0.0051
Base	10	898	Wind 10	-3.419	-0.719	-4.735	3.6892	-48.9158	0.2038
Base	10	898	Wind 11	-1.877	-3.96	0.47	40.5503	-26.4645	-0.098
Base	10	898	Wind 12	-2.867	-13.088	0.014	128.1068	-40.3428	0.096
Base	12	785	Wind 1	-3.928	-3.331	-7.71	33.6021	-55.5552	0.0737
Base	12	785	Wind 2	-0.004	-8.446	-46.176	91.951	0.1956	-0.0734
Base	12	785	Wind 3	-2.957	0.323	1.784	-3.4662	-41.8015	-0.0051
Base	12	785	Wind 4	-2.934	-5.32	-13.348	53.8695	-41.5313	0.1156
Base	12	785	Wind 5	-0.011	-4.245	-29.018	47.7385	0.0468	-0.0997
Base	12	785	Wind 6	0.006	-8.423	-40.245	90.1879	0.2466	-0.0104
Base	12	785	Wind 7	-2.943	3.986	29.564	-45.3761	-41.816	0.1116
Base	12	785	Wind 8	-2.88	-8.775	-40.293	93.5847	-40.5605	-0.0011
Base	12	785	Wind 9	-2.225	6.715	32.176	-71.8897	-31.5681	0.0041
Base	12	785	Wind 10	-2.194	-0.731	12.209	3.7651	-31.2116	0.1634
Base	12	785	Wind 11	-2.177	-2.95	-20.472	33.2928	-30.6214	-0.0786
Base	12	785	Wind 12	-2.147	-10.224	-40.021	107.2091	-30.2735	0.077
Base	15	904	Wind 1	-15.779	-1.651	-72.568	16.3467	-175.1437	0.113
Base	15	904	Wind 2	-3.03	-2.806	-29.12	36.7372	-34.1856	-0.1126
Base	15	904	Wind 3	-14.605	-0.19	-64.658	0.2756	-161.7502	-0.0078
Base	15	904	Wind 4	-9.064	-2.287	-44.194	24.2445	-100.9653	0.1773
Base	15	904	Wind 5	-4.324	-1.328	-29.438	18.6807	-48.1396	-0.153
Base	15	904	Wind 6	-0.222	-2.88	-14.243	36.4251	-3.1388	-0.0159
Base	15	904	Wind 7	-9.507	0.917	-32.138	-15.942	-105.1129	0.1711
Base	15	904	Wind 8	-13.833	-3.314	-75.155	39.5308	-153.9823	-0.0016
Base	15	904	Wind 9	-10.793	2.071	-37.626	-27.7807	-119.0077	0.0063
Base	15	904	Wind 10	-3.481	-0.695	-10.624	3.8465	-38.8017	0.2506
Base	15	904	Wind 11	-13.955	-1.136	-69.646	14.2254	-154.7688	-0.1206
Base	15	904	Wind 12	-6.813	-3.839	-43.188	45.1235	-76.4099	0.1181
Base	31	582	Wind 1	-9.224	-0.122	48.436	1.3841	-99.959	0.0202
Base	31	582	Wind 2	2.622	-0.632	-20.015	10.4955	29.5265	-0.0201
Base	31	582	Wind 3	-4.614	0.008	24.261	-0.1727	-50.1539	-0.0014

Story	Joint Label	Unique Name	Load Case/Combo	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft
Base	31	582	Wind 4	-9.222	-0.191	48.393	2.2489	-99.7845	0.0317
Base	31	582	Wind 5	3.671	-0.4	-23.97	6.9753	40.5165	-0.0273
Base	31	582	Wind 6	0.261	-0.548	-6.053	8.768	3.7733	-0.0028
Base	31	582	Wind 7	-8.931	0.394	51.644	-7.0197	-97.6348	0.0306
Base	31	582	Wind 8	-4.791	-0.563	20.579	8.8857	-51.1015	-0.0003
Base	31	582	Wind 9	-3.664	0.427	22.846	-6.8672	-40.5472	0.0011
Base	31	582	Wind 10	-9.744	0.164	54.689	-3.6718	-106.0351	0.0448
Base	31	582	Wind 11	-0.627	-0.294	-0.152	5.1094	-6.37	-0.0215
Base	31	582	Wind 12	-6.566	-0.551	31.047	8.231	-70.3504	0.0211
Base	32	873	Wind 1	-0.822	-0.019	0.0001041	0.2204	-11.3809	0.0006
Base	32	873	Wind 2	0.193	-0.101	0.00048	1.6686	2.9126	-0.0006
Base	32	873	Wind 3	-0.433	0.001	3.07E-05	-0.0274	-6.04	-3.983E-05
Base	32	873	Wind 4	-0.8	-0.03	0.0001255	0.358	-11.0313	0.0009
Base	32	873	Wind 5	0.28	-0.064	0.0003248	1.1088	4.0319	-0.0008
Base	32	873	Wind 6	0.009	-0.087	0.0003952	1.3942	0.3369	-0.0001
Base	32	873	Wind 7	-0.764	0.063	-0.0002892	-1.1158	-10.7717	0.0009
Base	32	873	Wind 8	-0.457	-0.09	0.0004365	1.4129	-6.1545	-8.284E-06
Base	32	873	Wind 9	-0.331	0.068	-0.0002796	-1.0919	-4.7929	3.222E-05
Base	32	873	Wind 10	-0.816	0.026	-0.0001546	-0.5833	-11.379	0.0013
Base	32	873	Wind 11	-0.107	-0.047	0.0002664	0.8122	-1.403	-0.0006
Base	32	873	Wind 12	-0.58	-0.088	0.0003889	1.3091	-7.837	0.0006
Base	46	876	Wind 1	-8.303	-0.232	51.088	2.806	-85.6334	0.0082
Base	46	876	Wind 2	1.191	-0.731	-8.313	12.3018	13.0131	-0.0081
Base	46	876	Wind 3	-5.164	0.016	31.327	-0.2958	-53.3429	-0.0006
Base	46	876	Wind 4	-7.29	-0.365	45.305	4.5048	-75.1073	0.0128
Base	46	876	Wind 5	1.681	-0.407	-11.422	7.4494	17.8167	-0.011
Base	46	876	Wind 6	0.106	-0.69	-1.048	11.0033	1.703	-0.0011
Base	46	876	Wind 7	-7.142	0.388	44.681	-7.34	-74.214	0.0123
Base	46	876	Wind 8	-5.19	-0.719	31.286	11.2822	-52.9918	-0.0001
Base	46	876	Wind 9	-3.958	0.543	24.319	-8.6772	-41.3508	0.0005
Base	46	876	Wind 10	-6.764	0.04	42.763	-2.3427	-70.0691	0.0181
Base	46	876	Wind 11	-2.525	-0.294	14.453	5.375	-25.7499	-0.0087
Base	46	876	Wind 12	-5.266	-0.785	32.518	11.5633	-53.8084	0.0085
Base	47	899	Wind 1	-0.666	-0.03	0.082	0.355	-9.2492	0.0006
Base	47	899	Wind 2	0.058	-0.092	0.318	1.545	0.9038	-0.0006
Base	47	899	Wind 3	-0.446	0.002	0.033	-0.0374	-6.1991	-3.983E-05
Base	47	899	Wind 4	-0.553	-0.047	0.089	0.5699	-7.6747	0.0009
Base	47	899	Wind 5	0.084	-0.051	0.218	0.9339	1.2241	-0.0008
Base	47	899	Wind 6	0.004	-0.087	0.259	1.3835	0.1317	-0.0001
Base	47	899	Wind 7	-0.544	0.049	-0.182	-0.92	-7.6307	0.0009
Base	47	899	Wind 8	-0.444	-0.091	0.298	1.4188	-6.0991	-8.284E-06
Base	47	899	Wind 9	-0.338	0.069	-0.173	-1.0913	-4.7546	3.222E-05
Base	47	899	Wind 10	-0.48	0.004	-0.1	-0.2899	-6.7016	0.0013
Base	47	899	Wind 11	-0.264	-0.037	0.188	0.6736	-3.6273	-0.0006
Base	47	899	Wind 12	-0.403	-0.1	0.26	1.4565	-5.5294	0.0006
Base	48	875	Wind 1	-0.518	-0.099	0.008	1.0905	-7.0968	0.0012
Base	48	875	Wind 2	0.061	-0.621	-0.003	10.5523	0.9351	-0.0012
Base	48	875	Wind 3	-0.33	0.007	0.005	-0.1534	-4.5356	-0.0001
Base	48	875	Wind 4	-0.447	-0.154	0.007	1.7892	-6.1096	0.0019
Base	48	875	Wind 5	0.089	-0.406	-0.003	7.1952	1.2839	-0.0016
Base	48	875	Wind 6	0.003	-0.525	-0.001	8.6333	0.1187	-0.0002
Base	48	875	Wind 7	-0.435	0.403	0.008	-7.2836	-6.0405	0.0018
Base	48	875	Wind 8	-0.333	-0.538	0.004	8.7131	-4.4985	-1.746E-05
Base	48	875	Wind 9	-0.25	0.409	0.004	-6.7491	-3.4959	0.0001

Story	Joint Label	Unique Name	Load Case/Combo	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft
Base	48	875	Wind 10	-0.404	0.196	0.008	-4.1859	-5.5728	0.0027
Base	48	875	Wind 11	-0.175	-0.3	0.001	5.2886	-2.3624	-0.0013
Base	48	875	Wind 12	-0.325	-0.508	0.004	7.7927	-4.3913	0.0013
Base	51	916	Wind 1	-0.462	0.129	0	-2.0184	-6.3342	0.0009
Base	51	916	Wind 2	0.054	-0.742	0.0002248	12.2437	0.8325	-0.0009
Base	51	916	Wind 3	-0.295	-0.011	4.075E-05	0.0822	-4.0493	-0.0001
Base	51	916	Wind 4	-0.399	0.205	-3.757E-05	-3.1099	-5.452	0.0014
Base	51	916	Wind 5	0.079	-0.636	0.0001977	10.3643	1.1435	-0.0012
Base	51	916	Wind 6	0.002	-0.476	0.0001395	8.0013	0.1052	-0.0001
Base	51	916	Wind 7	-0.388	0.667	-0.0001706	-10.9139	-5.3898	0.0014
Base	51	916	Wind 8	-0.298	-0.462	0.0001702	7.7038	-4.0166	-1.299E-05
Base	51	916	Wind 9	-0.223	0.358	-7.636E-05	-6.0867	-3.1205	0.0001
Base	51	916	Wind 10	-0.36	0.643	-0.0001797	-10.2987	-4.9713	0.002
Base	51	916	Wind 11	-0.157	-0.486	0.0001784	7.8403	-2.1111	-0.001
Base	51	916	Wind 12	-0.29	-0.207	7.719E-05	3.7257	-3.9191	0.0009
Base	52	915	Wind 1	-0.461	0.259	-0.065	-3.8034	-6.3304	0.0009
Base	52	915	Wind 2	0.054	-0.854	0.022	13.9301	0.8317	-0.0009
Base	52	915	Wind 3	-0.294	-0.022	-0.037	0.2128	-4.0469	-0.0001
Base	52	915	Wind 4	-0.398	0.41	-0.06	-5.9178	-5.4486	0.0014
Base	52	915	Wind 5	0.079	-0.8	0.025	12.7167	1.1426	-0.0012
Base	52	915	Wind 6	0.002	-0.481	0.008	8.1784	0.105	-0.0001
Base	52	915	Wind 7	-0.388	0.85	-0.065	-13.5473	-5.3863	0.0014
Base	52	915	Wind 8	-0.297	-0.451	-0.031	7.6608	-4.0144	-1.299E-05
Base	52	915	Wind 9	-0.223	0.353	-0.034	-6.1248	-3.1185	0.0001
Base	52	915	Wind 10	-0.359	0.923	-0.064	-14.2141	-4.9681	0.002
Base	52	915	Wind 11	-0.156	-0.616	-0.009	9.7019	-2.1101	-0.001
Base	52	915	Wind 12	-0.29	-0.06	-0.038	1.7995	-3.9169	0.0009
Base	54	913	Wind 1	-0.356	0.346	0.094	-4.9126	-4.882	0.0015
Base	54	913	Wind 2	0.042	-0.771	-0.017	12.345	0.6419	-0.0015
Base	54	913	Wind 3	-0.227	-0.029	0.061	0.3127	-3.1206	-0.0001
Base	54	913	Wind 4	-0.307	0.547	0.081	-7.6817	-4.2023	0.0023
Base	54	913	Wind 5	0.061	-0.791	-0.02	12.2177	0.8818	-0.002
Base	54	913	Wind 6	0.002	-0.365	-0.005	6.2997	0.0811	-0.0002
Base	54	913	Wind 7	-0.299	0.851	0.084	-13.1624	-4.1543	0.0022
Base	54	913	Wind 8	-0.229	-0.325	0.057	5.6593	-3.0955	-2.122E-05
Base	54	913	Wind 9	-0.172	0.26	0.05	-4.6062	-2.4049	0.0001
Base	54	913	Wind 10	-0.277	1.019	0.076	-15.1549	-3.8322	0.0033
Base	54	913	Wind 11	-0.121	-0.615	0.03	9.4007	-1.6265	-0.0016
Base	54	913	Wind 12	-0.224	0.127	0.055	-0.9042	-3.0208	0.0015
Base	55	912	Wind 1	-0.974	1.736	-11.882	-17.7097	-13.3786	0.0022
Base	55	912	Wind 2	0.048	-4.4	37.277	46.2034	0.775	-0.0022
Base	55	912	Wind 3	-0.685	-0.112	-0.193	1.0329	-9.4199	-0.0002
Base	55	912	Wind 4	-0.776	2.715	-17.629	-27.5975	-10.6479	0.0035
Base	55	912	Wind 5	0.069	-4.346	34.427	45.2511	1.0357	-0.003
Base	55	912	Wind 6	0.003	-2.253	21.489	24.054	0.1267	-0.0003
Base	55	912	Wind 7	-0.767	4.68	-37.456	-48.7484	-10.6288	0.0034
Base	55	912	Wind 8	-0.677	-2.028	19.231	21.6749	-9.2211	-3.2E-05
Base	55	912	Wind 9	-0.517	1.648	-16.614	-17.7048	-7.1685	0.0001
Base	55	912	Wind 10	-0.636	5.378	-39.62	-55.4828	-8.7889	0.0049
Base	55	912	Wind 11	-0.45	-3.344	25.7	34.7256	-6.1305	-0.0024
Base	55	912	Wind 12	-0.567	0.3	3.172	-2.1844	-7.7135	0.0023
Base	56	911	Wind 1	-0.916	2.107	4.788	-19.6755	-12.5947	0.0022
Base	56	911	Wind 2	-0.002	-5.488	-14.986	51.9676	0.0371	-0.0022
Base	56	911	Wind 3	-0.69	-0.125	0.061	1.1014	-9.4779	-0.0002

Story	Joint Label	Unique Name	Load Case/Combo	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft
Base	56	911	Wind 4	-0.684	3.285	7.121	-30.6146	-9.4142	0.0035
Base	56	911	Wind 5	-0.004	-5.378	-13.859	50.7171	0.0042	-0.003
Base	56	911	Wind 6	0.001	-2.854	-8.621	27.2342	0.0514	-0.0003
Base	56	911	Wind 7	-0.686	5.793	15.068	-54.644	-9.4744	0.0034
Base	56	911	Wind 8	-0.672	-2.572	-7.724	24.5566	-9.2003	-3.2E-05
Base	56	911	Wind 9	-0.519	2.099	6.653	-20.0947	-7.1542	0.0001
Base	56	911	Wind 10	-0.511	6.598	15.969	-61.9441	-7.07	0.0049
Base	56	911	Wind 11	-0.509	-4.129	-10.358	38.879	-6.9474	-0.0024
Base	56	911	Wind 12	-0.501	0.267	-1.237	-2.0114	-6.8653	0.0023
Base	58	910	Wind 1	-0.82	1.676	7.118	-17.3951	-11.2892	0.0022
Base	58	910	Wind 2	-0.085	-4.245	-22.286	45.3862	-1.1918	-0.0022
Base	58	910	Wind 3	-0.698	-0.105	0.153	0.9981	-9.5746	-0.0002
Base	58	910	Wind 4	-0.532	2.62	10.524	-27.0908	-7.3592	0.0035
Base	58	910	Wind 5	-0.125	-4.193	-20.562	44.438	-1.7138	-0.003
Base	58	910	Wind 6	-0.002	-2.175	-12.866	23.6413	-0.0739	-0.0003
Base	58	910	Wind 7	-0.55	4.517	22.402	-47.8838	-7.5517	0.0034
Base	58	910	Wind 8	-0.664	-1.956	-11.486	21.2922	-9.1655	-3.2E-05
Base	58	910	Wind 9	-0.522	1.593	9.974	-17.413	-7.1305	0.0001
Base	58	910	Wind 10	-0.303	5.188	23.658	-54.4766	-4.2072	0.0049
Base	58	910	Wind 11	-0.605	-3.224	-15.322	34.0898	-8.308	-0.0024
Base	58	910	Wind 12	-0.392	0.288	-1.922	-2.1231	-5.4524	0.0023
Base	60	909	Wind 1	-0.441	0.368	0.073	-5.369	-6.0769	0.0016
Base	60	909	Wind 2	-0.046	-1.1	0.003	17.9059	-0.6415	-0.0016
Base	60	909	Wind 3	-0.376	-0.031	0.056	0.3117	-5.154	-0.0001
Base	60	909	Wind 4	-0.286	0.583	0.053	-8.3651	-3.9614	0.0025
Base	60	909	Wind 5	-0.067	-1.052	0.003	16.641	-0.9225	-0.0022
Base	60	909	Wind 6	-0.001	-0.598	0.002	10.2179	-0.0397	-0.0002
Base	60	909	Wind 7	-0.296	1.121	0.052	-17.7739	-4.0651	0.0024
Base	60	909	Wind 8	-0.358	-0.555	0.056	9.4956	-4.9337	-2.288E-05
Base	60	909	Wind 9	-0.281	0.437	0.041	-7.6177	-3.8383	0.0001
Base	60	909	Wind 10	-0.163	1.247	0.037	-19.0668	-2.2647	0.0035
Base	60	909	Wind 11	-0.326	-0.812	0.044	12.7202	-4.4721	-0.0017
Base	60	909	Wind 12	-0.211	-0.022	0.04	1.5358	-2.935	0.0017
Base	70	906	Wind 1	-5.138	0.074	33.708	-1.2226	-55.1359	0.0147
Base	70	906	Wind 2	-0.542	-0.713	3.869	11.5214	-5.8556	-0.0147
Base	70	906	Wind 3	-4.359	-0.007	28.449	0.0281	-46.7123	-0.001
Base	70	906	Wind 4	-3.348	0.118	22.112	-1.862	-35.9916	0.0231
Base	70	906	Wind 5	-0.781	-0.581	5.253	9.3406	-8.36	-0.0199
Base	70	906	Wind 6	-0.033	-0.489	0.549	7.9414	-0.4233	-0.0021
Base	70	906	Wind 7	-3.437	0.603	22.319	-9.7625	-36.8562	0.0223
Base	70	906	Wind 8	-4.171	-0.481	27.664	7.7452	-44.7943	-0.0002
Base	70	906	Wind 9	-3.247	0.37	20.935	-6.0814	-34.7398	0.0008
Base	70	906	Wind 10	-1.913	0.535	12.574	-8.5754	-20.5937	0.0327
Base	70	906	Wind 11	-3.782	-0.442	24.861	7.0322	-40.5357	-0.0157
Base	70	906	Wind 12	-2.48	-0.281	16.671	4.5959	-26.7156	0.0154
Base	74	908	Wind 1	-6.44	3.754	-16.79	-49.0992	-101.4643	0.3596
Base	74	908	Wind 2	-0.241	-21.091	97.002	279.703	-3.706	-0.3583
Base	74	908	Wind 3	-5.076	-0.149	-1.199	1.3811	-79.9133	-0.0249
Base	74	908	Wind 4	-4.584	5.781	-23.986	-75.0299	-72.2832	0.5643
Base	74	908	Wind 5	-0.363	-18.013	81.204	238.066	-5.604	-0.4869
Base	74	908	Wind 6	0.001	-13.624	64.299	181.4884	0.0451	-0.0506
Base	74	908	Wind 7	-4.644	19.008	-86.89	-251.5056	-73.2524	0.5447
Base	74	908	Wind 8	-4.9	-13.067	60.421	173.792	-77.1379	-0.0052
Base	74	908	Wind 9	-3.811	10.356	-50.192	-138.3845	-60.0221	0.0201

Story	Joint Label	Unique Name	Load Case/Combo	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft
Base	74	908	Wind 10	-3.162	18.18	-80.26	-239.2092	-49.9541	0.7976
Base	74	908	Wind 11	-3.995	-13.631	60.074	179.7186	-62.8232	-0.3837
Base	74	908	Wind 12	-3.361	-5.987	30.638	81.2011	-52.9865	0.376
Base	77	907	Wind 1	-6.206	3.754	16.665	-49.0991	-97.8228	0.3596
Base	77	907	Wind 2	-0.462	-21.091	-97.016	279.7025	-7.2664	-0.3583
Base	77	907	Wind 3	-5.093	-0.149	1.1	1.3811	-80.1681	-0.0249
Base	77	907	Wind 4	-4.217	5.781	23.897	-75.0298	-66.5661	0.5643
Base	77	907	Wind 5	-0.67	-18.013	-81.218	238.0657	-10.4852	-0.4869
Base	77	907	Wind 6	-0.022	-13.624	-64.306	181.4881	-0.4144	-0.0506
Base	77	907	Wind 7	-4.3	19.008	86.807	-251.5052	-67.7884	0.5447
Base	77	907	Wind 8	-4.894	-13.067	-60.523	173.7917	-77.1396	-0.0052
Base	77	907	Wind 9	-3.806	10.356	50.122	-138.3843	-59.8604	0.0201
Base	77	907	Wind 10	-2.65	18.18	80.204	-239.2089	-41.9125	0.7976
Base	77	907	Wind 11	-4.238	-13.631	-60.158	179.7184	-66.6741	-0.3837
Base	77	907	Wind 12	-3.11	-5.987	-30.708	81.2009	-49.1381	0.376
Base	79	10	Wind 1	-0.019	0.005	-8.811E-05	-0.0912	-0.318	0.0087
Base	79	10	Wind 2	-0.001	-0.019	5.418E-05	0.3227	-0.0114	-0.0087
Base	79	10	Wind 3	-0.015	-0.0002324	-5.615E-05	0.0039	-0.2503	-0.0006
Base	79	10	Wind 4	-0.013	0.008	-7.601E-05	-0.1407	-0.2267	0.0136
Base	79	10	Wind 5	-0.001	-0.017	4.8E-05	0.2956	-0.0173	-0.0118
Base	79	10	Wind 6	1.29E-05	-0.011	3.328E-05	0.1885	0.0002	-0.0012
Base	79	10	Wind 7	-0.014	0.019	-0.0001076	-0.3161	-0.2298	0.0132
Base	79	10	Wind 8	-0.014	-0.01	-2.394E-05	0.1752	-0.2416	-0.0001
Base	79	10	Wind 9	-0.011	0.008	-6.77E-05	-0.1418	-0.1881	0.0005
Base	79	10	Wind 10	-0.009	0.02	-9.391E-05	-0.3327	-0.1569	0.0193
Base	79	10	Wind 11	-0.012	-0.013	-5.156E-06	0.2248	-0.1966	-0.0093
Base	79	10	Wind 12	-0.01	-0.002	-3.079E-05	0.0382	-0.1662	0.0091
Base	82	902	Wind 1	-5.132	-0.122	-24.16	1.2963	-55.1068	0.0147
Base	82	902	Wind 2	-0.573	-0.626	14.269	9.6103	-6.0214	-0.0147
Base	82	902	Wind 3	-4.354	0.003	-20.83	-0.1306	-46.6853	-0.001
Base	82	902	Wind 4	-3.345	-0.186	-15.41	2.0751	-35.9749	0.0231
Base	82	902	Wind 5	-0.803	-0.399	8.691	6.3913	-8.4806	-0.0199
Base	82	902	Wind 6	-0.056	-0.539	12.713	8.0242	-0.5515	-0.0021
Base	82	902	Wind 7	-3.409	0.389	-29.04	-6.406	-36.7075	0.0223
Base	82	902	Wind 8	-4.19	-0.558	-7.047	8.1575	-44.8977	-0.0002
Base	82	902	Wind 9	-3.225	0.417	-25.376	-6.264	-34.6213	0.0008
Base	82	902	Wind 10	-1.894	0.167	-18.224	-3.3535	-20.4889	0.0327
Base	82	902	Wind 11	-3.795	-0.297	-8.792	4.7019	-40.6066	-0.0157
Base	82	902	Wind 12	-2.495	-0.541	-1.788	7.5453	-26.7998	0.0154
Base	83	905	Wind 1	-6.366	-0.015	-1.549	-0.0418	-61.742	0.0147
Base	83	905	Wind 2	-0.703	-0.637	1.734	10.4134	-6.7189	-0.0147
Base	83	905	Wind 3	-5.394	-0.0004116	-1.342	-0.0579	-52.2765	-0.001
Base	83	905	Wind 4	-4.156	-0.022	-0.982	-0.0048	-40.3366	0.0231
Base	83	905	Wind 5	-0.985	-0.47	1.167	7.7904	-9.4591	-0.0199
Base	83	905	Wind 6	-0.069	-0.486	1.434	7.8297	-0.6193	-0.0021
Base	83	905	Wind 7	-4.235	0.479	-2.488	-8.0263	-41.1486	0.0223
Base	83	905	Wind 8	-5.192	-0.489	0.163	7.7793	-50.285	-0.0002
Base	83	905	Wind 9	-3.996	0.373	-2.105	-6.0602	-38.7663	0.0008
Base	83	905	Wind 10	-2.363	0.345	-1.63	-5.99	-23.0115	0.0327
Base	83	905	Wind 11	-4.695	-0.353	-0.11	5.8054	-45.4436	-0.0157
Base	83	905	Wind 12	-3.1	-0.381	0.355	5.874	-30.051	0.0154
Base	84	903	Wind 1	-15.754	-0.668	65.19	6.3099	-175.0105	0.113
Base	84	903	Wind 2	-3.023	-3.749	-3.208	46.5388	-34.1458	-0.1126
Base	84	903	Wind 3	-14.581	-0.044	61.208	-0.2207	-161.6235	-0.0078

Story	Joint Label	Unique Name	Load Case/Combo	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft
Base	84	903	Wind 4	-9.05	-0.958	36.576	9.6855	-100.8922	0.1773
Base	84	903	Wind 5	-4.314	-2.473	6.738	31.2374	-48.0898	-0.153
Base	84	903	Wind 6	-0.22	-3.15	-11.549	38.5707	-3.1288	-0.0159
Base	84	903	Wind 7	-9.494	2.378	51.347	-30.9931	-105.0435	0.1711
Base	84	903	Wind 8	-13.809	-3.302	45.49	39.5282	-153.8544	-0.0016
Base	84	903	Wind 9	-10.776	2.388	54.796	-29.8012	-118.9203	0.0063
Base	84	903	Wind 10	-3.478	1.182	22.294	-16.7299	-38.7849	0.2506
Base	84	903	Wind 11	-13.93	-1.889	50.069	23.2878	-154.6383	-0.1206
Base	84	903	Wind 12	-6.801	-3.067	18.226	36.0571	-76.3485	0.1181
Base	87	901	Wind 1	-1.721	-0.083	-0.635	1.03	-19.2462	0.0012
Base	87	901	Wind 2	-0.389	-0.169	12.052	2.915	-3.1631	-0.0012
Base	87	901	Wind 3	-1.399	0.006	-2.803	-0.0994	-15.9728	-0.0001
Base	87	901	Wind 4	-1.183	-0.13	1.85	1.6444	-12.8965	0.0019
Base	87	901	Wind 5	-0.371	-0.076	7.312	1.5408	-3.5109	-0.0016
Base	87	901	Wind 6	-0.212	-0.177	10.766	2.8317	-1.2337	-0.0002
Base	87	901	Wind 7	-0.993	0.068	-9.701	-1.4657	-12.0066	0.0018
Base	87	901	Wind 8	-1.553	-0.187	8.572	2.9409	-16.475	-1.746E-05
Base	87	901	Wind 9	-0.887	0.141	-10.353	-2.2507	-11.0426	0.0001
Base	87	901	Wind 10	-0.603	-0.039	-4.212	0.0503	-6.9834	0.0027
Base	87	901	Wind 11	-1.304	-0.053	3.428	1.0837	-14.3498	-0.0013
Base	87	901	Wind 12	-1.027	-0.228	9.442	3.3315	-10.3846	0.0013
Base	29	67	Wind 1	-0.019	0.006	0	-0.1016	-0.3193	0.0087
Base	29	67	Wind 2	-0.001	-0.02	0	0.333	-0.0115	-0.0087
Base	29	67	Wind 3	-0.015	-0.0002758	0	0.0047	-0.2514	-0.0006
Base	29	67	Wind 4	-0.014	0.009	0	-0.157	-0.2276	0.0136
Base	29	67	Wind 5	-0.001	-0.018	0	0.3096	-0.0175	-0.0118
Base	29	67	Wind 6	5.935E-06	-0.011	0	0.1899	0.0002	-0.0012
Base	29	67	Wind 7	-0.014	0.02	0	-0.3317	-0.2306	0.0132
Base	29	67	Wind 8	-0.014	-0.01	0	0.1753	-0.2427	-0.0001
Base	29	67	Wind 9	-0.011	0.008	0	-0.1423	-0.1888	0.0005
Base	29	67	Wind 10	-0.009	0.021	0	-0.3557	-0.1574	0.0193
Base	29	67	Wind 11	-0.012	-0.014	0	0.2358	-0.1975	-0.0093
Base	29	67	Wind 12	-0.01	-0.002	0	0.0273	-0.1668	0.0091