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September 26, 2016

Dr. Linda Hanagan
The Pennsylvania State University
212 Engineering Unit A
University Park, PA 16802

Dear Dr. Hanagan,

The following document, Technical Report III – Member Spot Check & Alternate Systems in support of my senior thesis program. This report is a detailed analysis of the gravity load resisting system of 706 Madison Avenue. Through presentation of hand calculations as well as diagrammatic sketches, inclusive of material submitted in Notebook A, this report documents a typical by representative of the existing gravity framing system of the building.

The existing gravity load resisting system in 706 Madison Avenue has been analyzed by hand calculation in the beginning of the report. To determine the most appropriate gravity system for 706 Madison Avenue, three alternate systems will be evaluated as a comparative study. Each proposed system will be presented as an individual solution, analyzed and designed in terms of applicable strength and serviceability criteria.

In addition, the report consists of an executive summary, site plan and location plan, and a brief introduction in order to provide a better understanding of the building and the purpose of this report.

Thank you for your consideration and evaluation of this report.

Sincerely,

Yong Yue

The Pennsylvania State University

Architectural Engineering Class of 2017

706 Madison Avenue | New York, USA

Member Spot Check & Alternate Systems

Structural Notebook Submission B

Submitted to: Dr. Linda Hanagan, Advisor

Prepared by: Yong Yue [Structural Option] Prepared on: October 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

1.1 Purpose

This report has been written in order to develop a detailed description of the loading conditions on 706 Madison Avenue. The loads described and analyzed in this report will serve as a foundation of technical knowledge for an investigation of the existing structural system of 706 Madison Avenue in the following reports.

1.2 Scope

The content of this report is comprised of three major sections: gravity loads, wind loads, and seismic loads. The structural loads imposed on this building are shown by hand calculations as well as graphs.

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'.

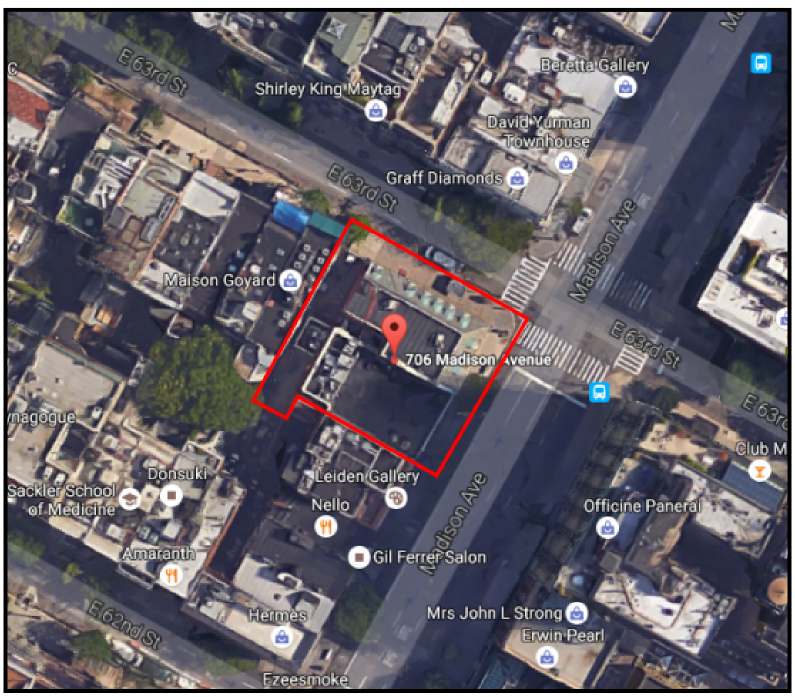


Figure 1
Site Context

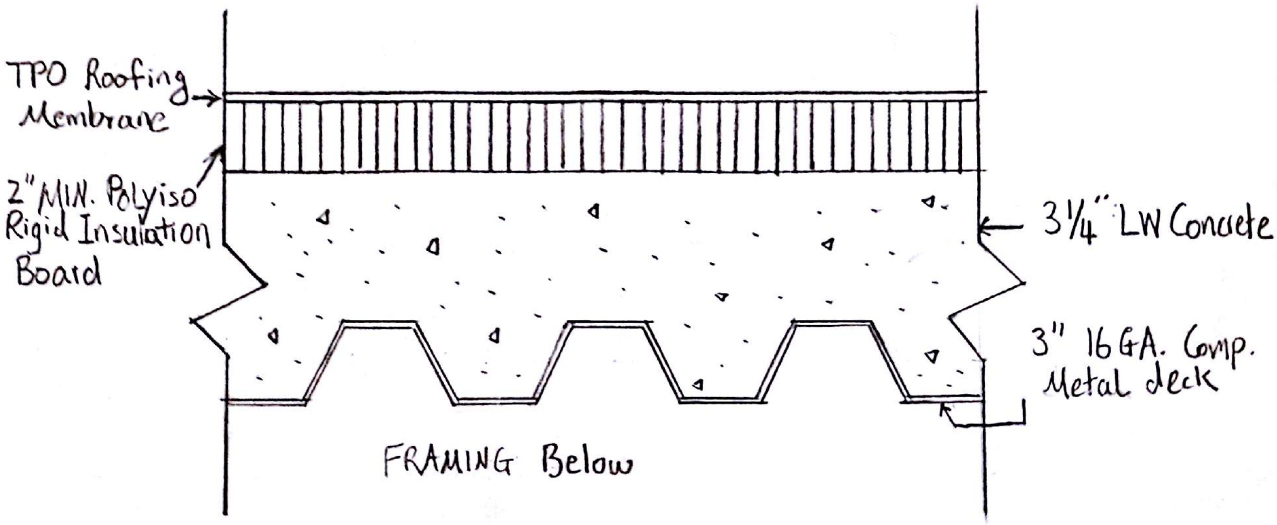
Figure 2 Site Context (Google Map)

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 constuction



- Roof Loading
  - Dead Load: (according to ASCE7-02 Table C3-1)

Roofing Membrane:	1 psf
2" Rigid Insulation:	3 psf
3¼" LW Concrete } 3" 16GA. Comp. deck }:	46 psf
Framing:	7 psf
Miscellaneous:	10 psf
	$DL_r = 67$ psf

  - Live Load:

    $LL_r = 20$ psf (according to ASCE7-02 C4.9.1 Min. Roof Live Loads)

  - Snow Load:

    Ground Snow Load; $P_g = 25$ psf (ASCE7-02 Figure 7-1)

    Exposure Factor; $C_e = 0.9$ (ASCE7-02 Table 7-2 for Terrain Category B and a fully exposed roof)

    Thermal Factor; $C_t = 1.0$ (ASCE7-02 Table 7-3)

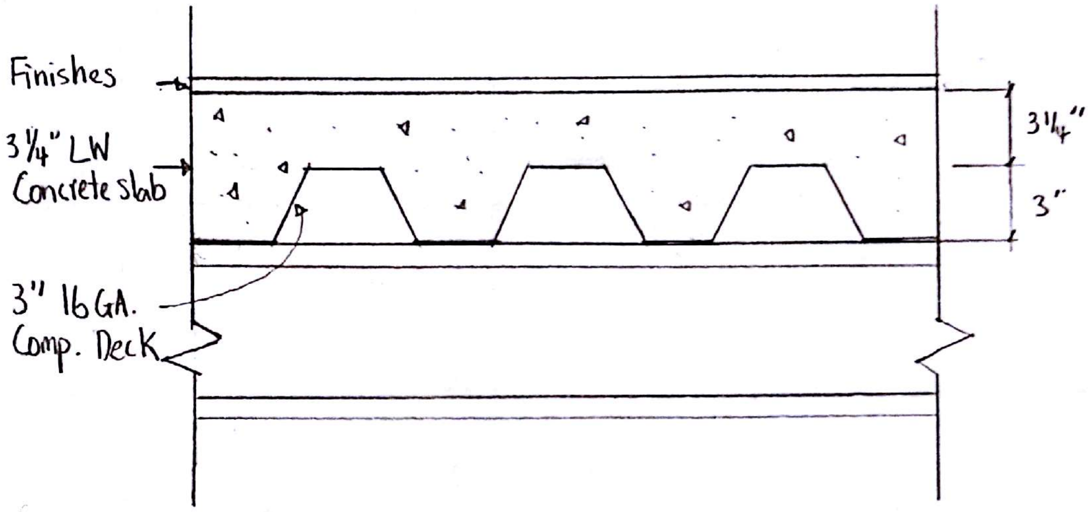
    Important Factor; $I_s = 1.0$ (ASCE7-02 Table 7-4)

    Flat Roof Snow Load; $P_f = 0.7\, C_e C_t I_s P_g$
    $= 0.7(0.9)(1)(1)(25) = 16\text{ psf} < 20\text{ psf (Min)}$

    Thus. use $P_f = 20$ 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:	10 psf

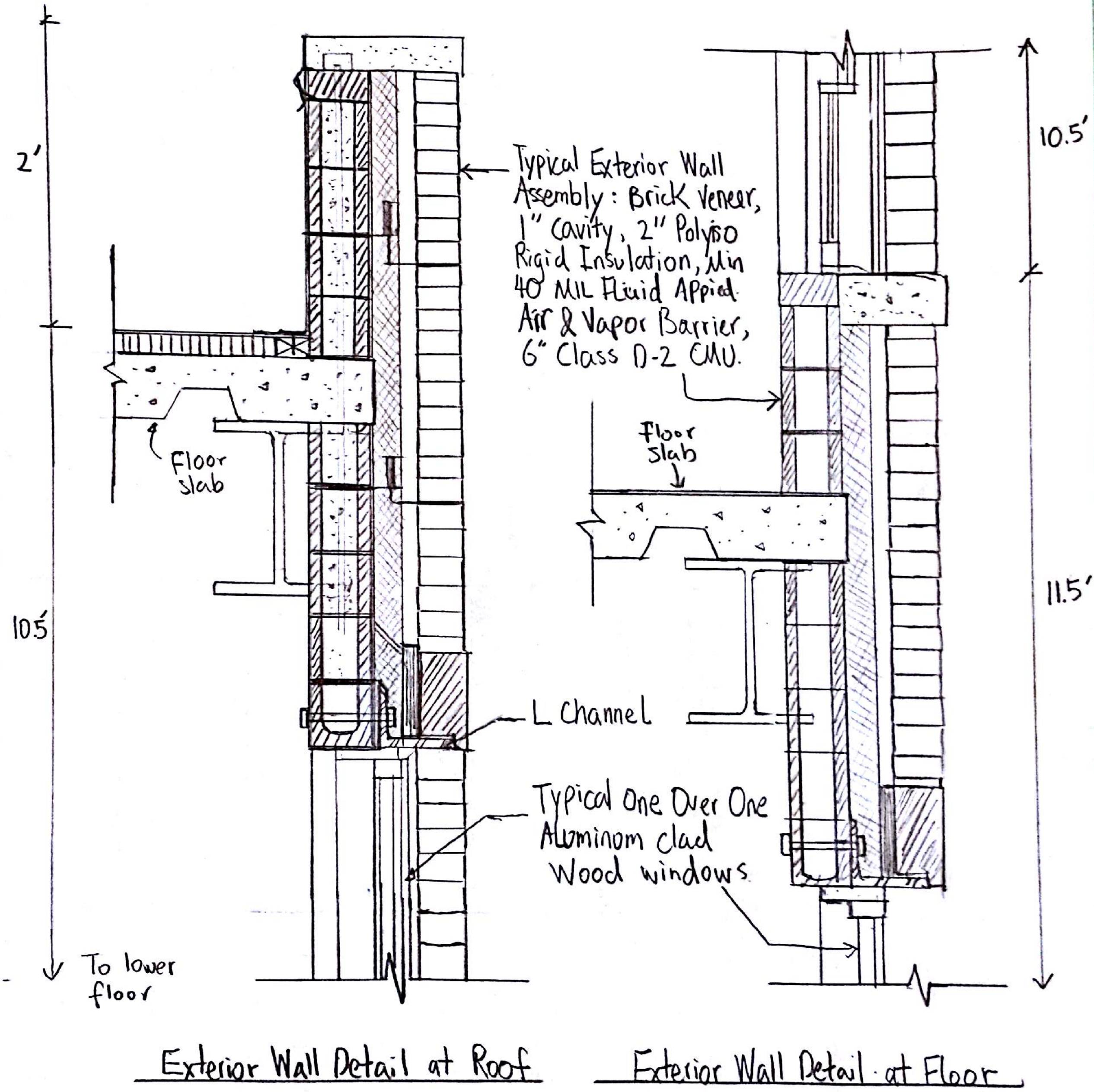
$DL_f = 66$ psf

  - Live load:

	Number Noted in Drawings	Code Minimum (ASCE7-02)
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
Side walk	600 psf	250 psf
Elevator Machine Room	125 psf	150 psf

GRAVITY LOAD (cont.)

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



- Load Path:

Wall loads act on the L channels, L channels transfer Loads into 6" Concrete masonry unit (CMU) throught 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":	1½ psf
Vapor Barrier:	½ psf
	$DL_w = 96$ psf

For Roof:

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

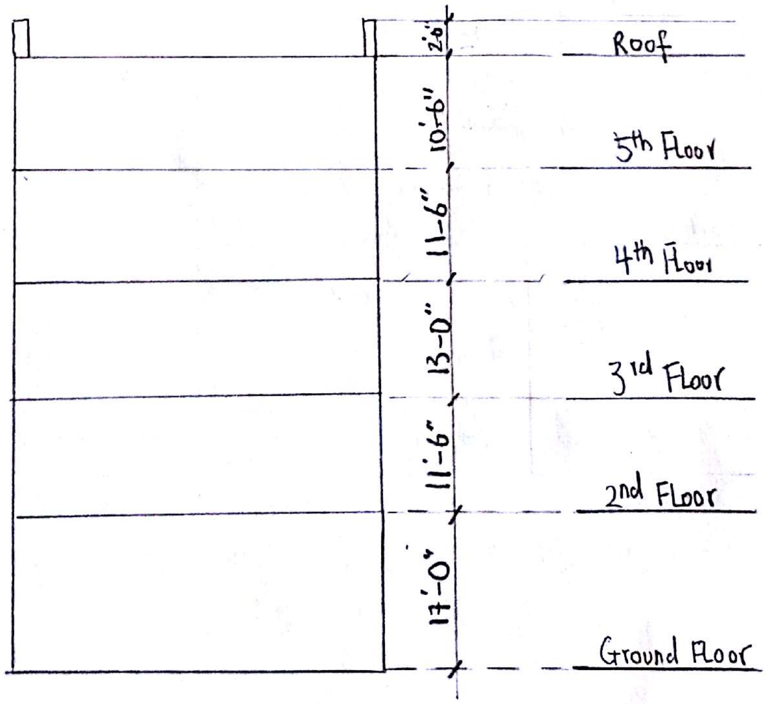
For floors:

$$W_{wall(5)} = 96\left(\frac{10.5}{2} + \frac{11.5}{2}\right) = 1{,}056 \text{ Plf}$$

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

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

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



East Elevation

2.4 Snow Load

- Drift @ Parapet (Windward drift)

$P_g = 25$ psf
$P_f = 20$ psf (From previous cals.)

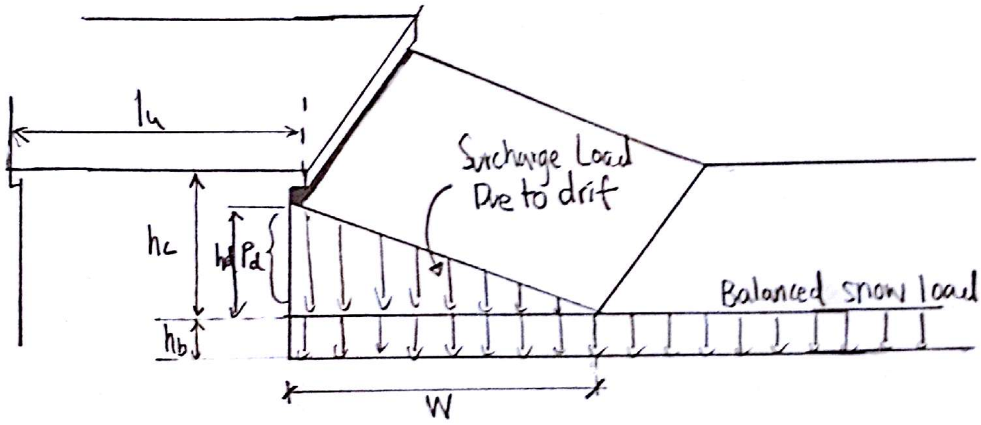
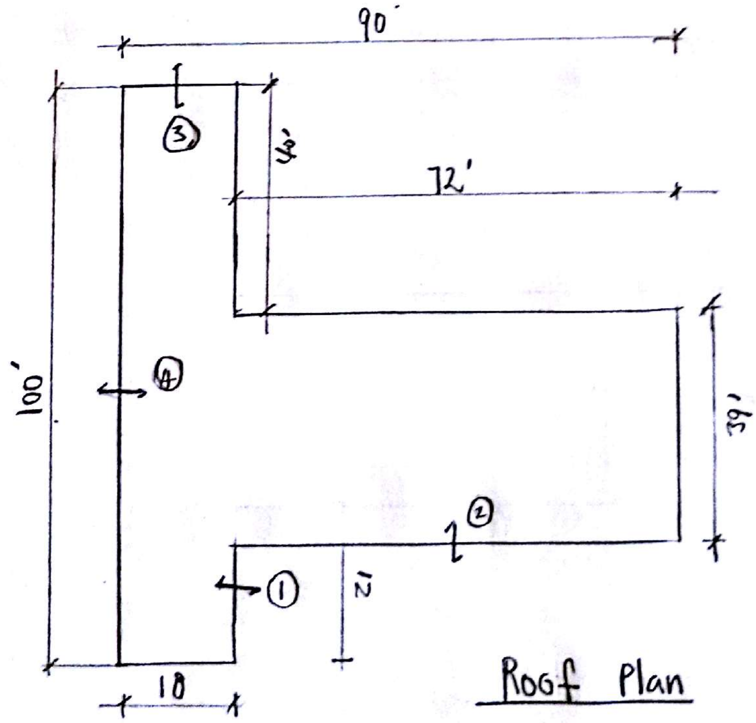


Figure 7.8 Snow Drifts on Lower Roof (ASCE 7-02)

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

$h_b = 20/17.25 = 1.16$ ft

$h_c = 2' - 1.16' = 0.84'$; $\frac{h_c}{h_b} = \frac{084}{1.16} = .72 > 0.2 \Rightarrow$ drift load must to be considered.



Roof Plan

Snow Load (cont.)

Parapet Section ①:

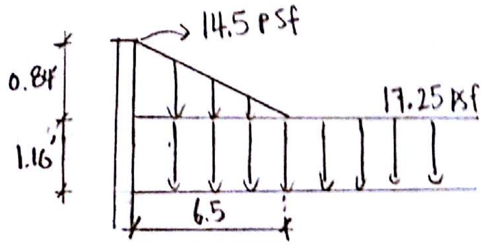
$l_u = 18' < 25 \quad \therefore$ use $l_u = 25'$

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

$W = 4h_d^2/h_c$ & $h_d = h_c$

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

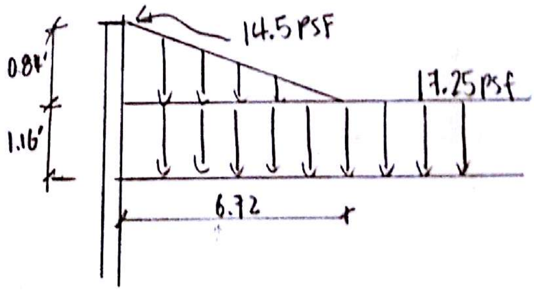
$P_d = \gamma h_d = \gamma h_c = 17.25(0.84) = 14.5$ psf



Section 1

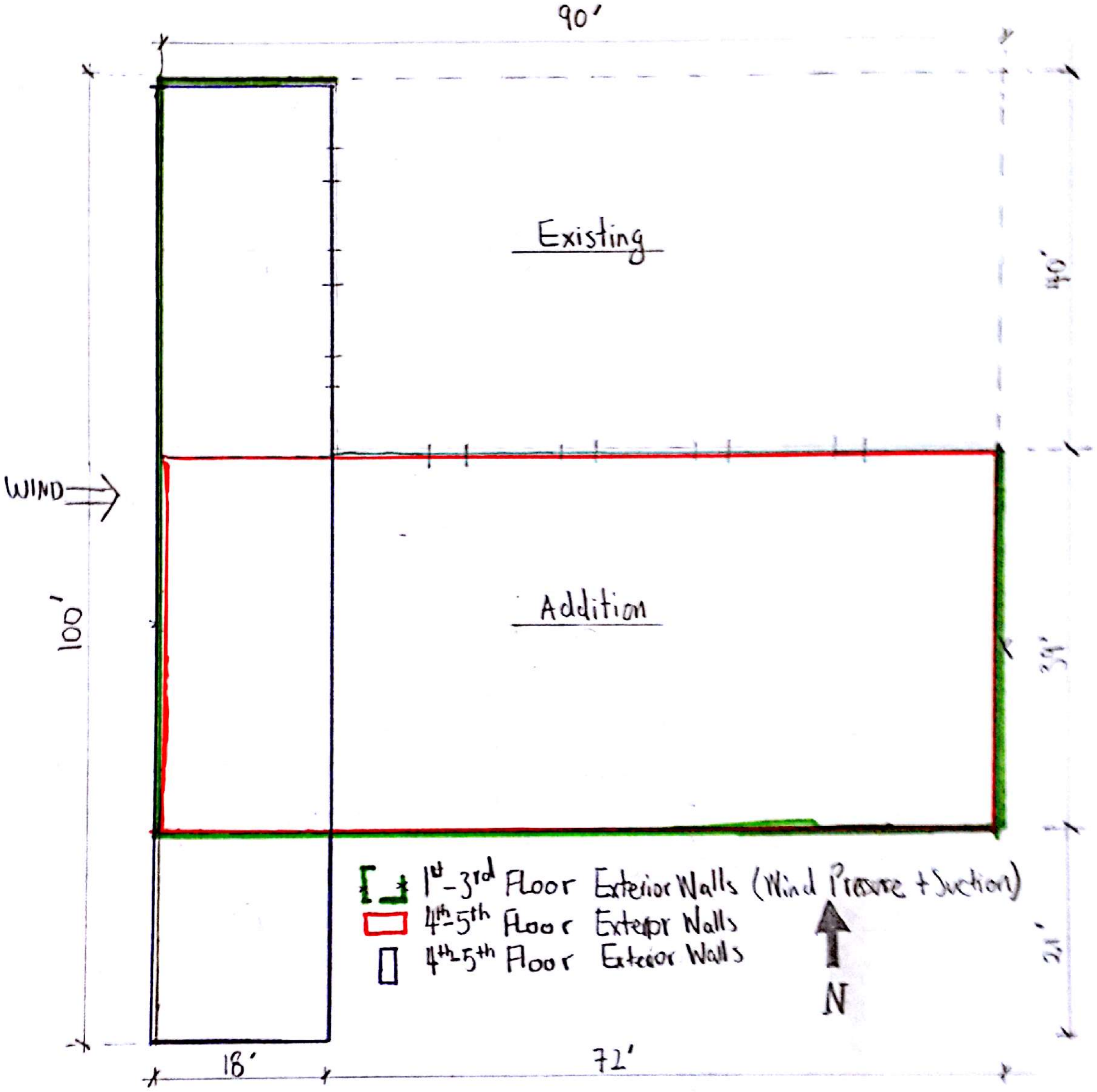
②: $h_d = 0.75\left[0.43\sqrt[3]{39}\sqrt[4]{25+10} - 1.5\right] = 1.54 > h_c$

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



$P_d = \gamma h_c = 17.25(0.84) = 14.5$ 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)
- Topograhic 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)	Story Ht (ft)	K_z	I	K_{zt}	K_d	V^2	q_z (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

* $q_z = 0.00256\, K_z K_{zt} K_d V^2 I$ (Eq. 6-15)

• Gust Effect Factor:

Since ASCE7-02 doesn't have this section.

For structure steel moment resisting frame buildings:

$n_a = 22.2/h^{0.8} = 22.2/(63.5)^{.8} = .802 < 1$ (ASCE7-10, Eq. 26.9-2)

$\therefore$ Not rigid.

$\beta = 0.01$; Conservative for steel (ASCE7-10, Structural damping)

Exposure B $\Rightarrow$ $\bar{\alpha} = 1/4$, $\bar{b} = 0.45$, $l = 320'$, $\bar{\epsilon} = 1/3$, $c = 0.3$ (ASCE7-02 Table 6-2)

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

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

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{38.1}\right)^{1/6} = .293$ (Eq. 6-5)

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

WING Load (cont.)

For ① Wind E-W →

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{79+63.5}{336}\right)^{0.63}}} = .855$$

For ② ③

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{100+63.5}{336}\right)^{0.63}}} = .845 \qquad Q = \sqrt{\frac{1}{1+0.63\left(\frac{39+63.5}{336}\right)^{.63}}} = .818$$

Eq. 6-6

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{38.1}{33}\right)^{1/4}(98)\left(\frac{88}{60}\right) = 67.0 \text{ ft/s} \quad \text{(Eq. 6-14)}$$

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)

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{.802 * 336}{67.0} = 4.02$$

$$R_n = \frac{7.47 N_1}{(1+10.3 N_1)^{5/3}} = \frac{7.47 \times 4.02}{(1+10.3 * 4.02)^{5/3}} = .058 \quad \text{(Eq. 6-11)}$$

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

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

$R_B = .203$ $\quad \eta = 4.6 n_1 B/\bar{V}_{\bar{z}} = 4.35$ ①
$R_B = .165$ $\quad \eta = 5.5$ ②
$R_B = .359$ $\quad \eta = 2.15$ ③
$R_L = .181$ $\quad \eta = 4.6 n_1 L/\bar{V}_{\bar{z}} = 4.96$ ①
$R_L = .570$ $\quad \eta = .99$ ②
$R_L = .181$ $\quad \eta = 4.96$ ③

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

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

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

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{.577}{\sqrt{2\ln(3600 n_1)}} = 4.14 \quad \text{(Eq. 6-9)}$$

WING Load (cont.)

(Eq. 6-8)

$$G_f = 0.925\left(\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right) = \left(\frac{1+1.7(.293)\sqrt{(3.4)^2(.855)^2+(4.14)^2(.421)^2}}{1+1.7(3.4)(.293)}\right) = .9232 \approx .92 \quad ①$$

$$G_f = 0.925\left(\frac{1+1.7(.293)\sqrt{(3.4)^2(.845)^2+(4.14)^2(.433)^2}}{1+1.7(3.4)(.293)}\right) = .9227 \approx .92 \quad \text{For } ②$$

$$G_f = 0.925\left(\frac{1+1.7(.293)\sqrt{(3.4)^2(.878)^2+(4.14)^2(.560)^2}}{1+1.7(3.4)(.293)}\right) = .99 \quad \text{For } ③$$

- Enclosed Building ⇒ Internal Pressure Coefficient, $GC_{pi} = \pm 0.18$ (Fig. 6-5)
- External Pressure Coefficient, C_p (Wind E-W) (Fig. 6-6)

For ① : $L/B = 1.14$; $h/L = .706$ $A = 79 * 100 = 7900\ ft^2 \Rightarrow$ Red. factor $= 0.8$

- Walls : $C_{p, windward, ew} = 0.8$
  $C_{p, leedward, ew} = -0.47$
  $C_{p, sidewall, ew} = -0.7$

1	-0.5
1.14	-0.47
2	-0.3

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

- Walls : $C_{p, windward, ew} = .8$
  $C_{p, leedward, ew} = -0.5$
  $C_{p, sidewall, ew} = -0.7$
- Roofs : $C_{p, roof, ew}(0-9) = -1.3 \times 0.8 = -1.04$
  $C_{p, roof, ew}(9-18) = -0.7$

For ③ : $L/B = 2.31$, $h/L = .706$, $A = 39 * 100 = 3900\ ft^2 \Rightarrow Rf = .8$

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

2	-0.3
2.31	
4	-0.2

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

h/L	Cp 0-h/2	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 $\quad K_z$ @ 65.5 = 0.87

$GC_{pi} = +1.5$ (Windward)
$\quad\quad = -1.0$ (leeward)

$q_p = 0.00256\,(0.87)(0.85)(1)(9604)(1) = 18.2$ psf

P_p, windward $= 18.2 * (1.5) = 27.3$

P_p, leedward $= 18.2 * (-1.0) = -18.2$

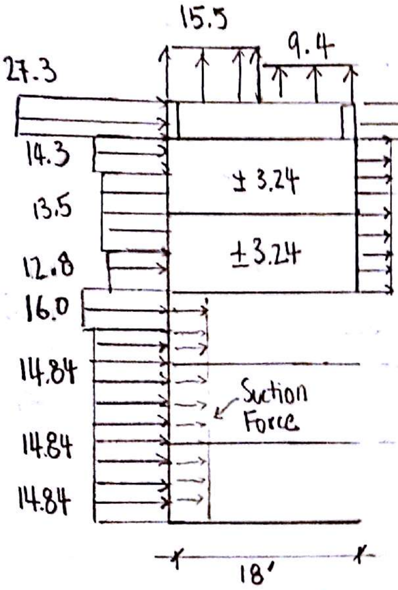
- Pressure on each surface : $P = qGC_p - q_i(GC_{pi})$ $\quad\quad$ (Eq. 6-23)

Use $G_f = 0.99$ (Conservative)

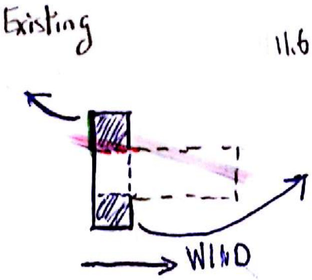
Location	z (ft)	q_z psf	C_p	q_zGC_p (psf)	GC_{pi}	q_hGC_{pi} (psf)	Net Pressure (PSF) $q_zGC_p - q_h(+GC_p)$ (psf)	$q_zGC_p - q_h(-GC_p)$ (psf)
Windward	0	14.6	0.8	11.6	0.18	3.24	8.36	14.84
	17	14.6		11.6			8.36	14.84
	28.5	14.6		11.6			8.36	14.84
	41.5	16.1		12.8			9.56	16.04
	53	17.1		13.5			10.3	16.74
	63.5	18.0		14.3			11.1	17.54
Leeward ②	ALL	18.0	-0.5	-9.0	0.18	3.24	-12.2	-5.8
③	ALL	18.0	-0.28	-5.0			-8.2	-1.8
Parapet (W)	65.5	18.2			1.5			27.3
(L)	65.5	18.2			-1.0			-18.2
Roof ② 0-9	63.5	18.0	-1.04	-18.7	0.18	3.24	-21.9	-15.5
9-18			-0.7	-12.6			-15.8	-9.4
③ 0-31.75			-0.96	-17.3			-20.5	-14.1
31.75-63.5	63.5	18.0	-.82	-14.8			-18	-11.6
63.5-90			-.58	-10.4			-13.6	-7.2

WIND LOAD (cont.)

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



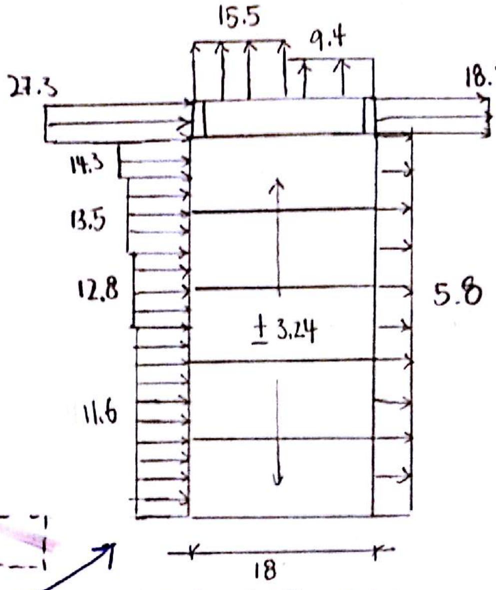
Trib. With = 40'



$$V = [14.8(17+11.5+6.5) + 16(6.5) + (12.8+5.8)(5.75) + (13.5+5.8)(5.75+5.25) + (14.3+5.8)(5.25) + (27.3+18.2)(2)] \times 40$$

$$= 45.5^K$$

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

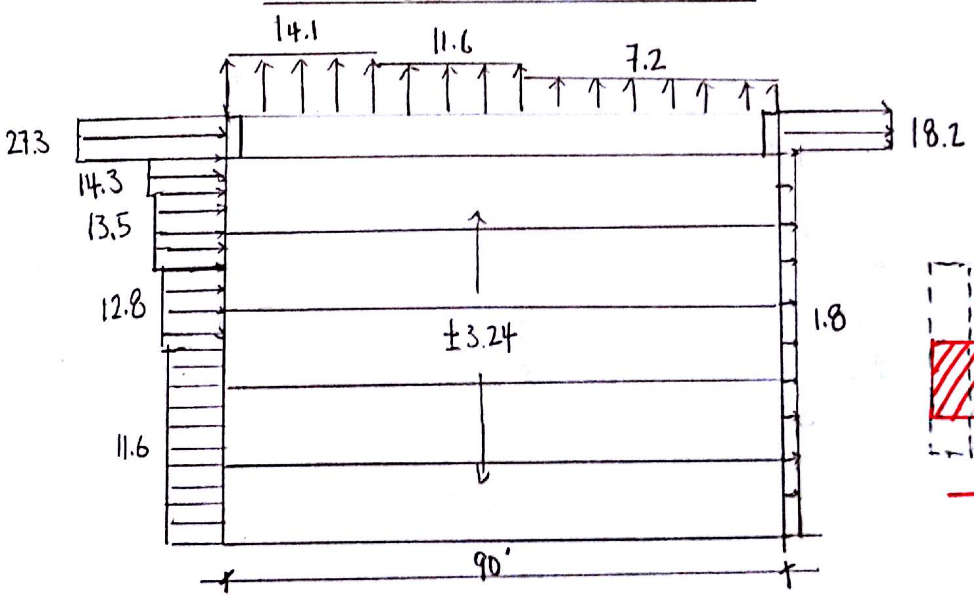


Trib. Width = 21'

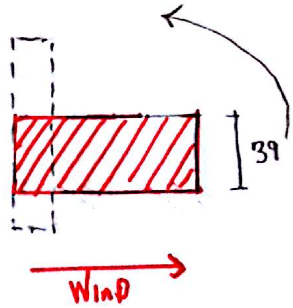
$$V = [(11.6+5.8)(17+11.5+6.5) + (12.8+5.8)(6.5+5.75) + (13.5+5.8)(5.75+5.25) + (14.3+5.8)(5.25) + (27.3+18.2)(2)] \times 21$$

$$= 26.0^K$$

③ Wind E-W



Trib. Width = 39'



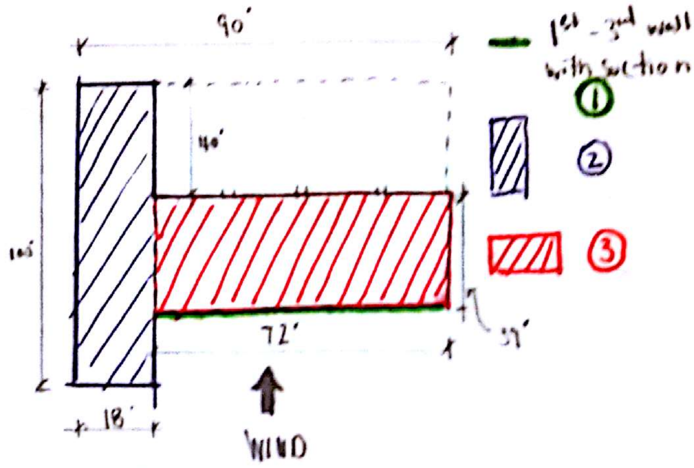
$$V = [(11.6+1.8)(17+11.5+6.5) + (12.8+1.8)(6.5+5.75) + (13.5+1.8)(5.75+5.25) + (14.3+1.8)(5.25) + (27.3+18.2)(2)] \times 39 = 38.4^K$$

* Base Shear (E-W) = 45.5 + 26 + 38.4 = 110^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:

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{72+63.5}{336}\right)^{0.63}}} = .859$$

(Eq. 6-6)

For ②:

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

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

$N_1 = 4.02$, $R_n = .058$, $\bar{V}_{\bar{z}} = 67.0'$ [from previous calcs.]

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

$R_h = .245$

$R_B = .220$ $\quad \eta = 4.6 n_1 B / \bar{V}_{\bar{z}} = 3.96$ for ① & ③

$R_B = .570$ $\quad \eta = .991$ ②

$R_L = .203$ $\quad \eta = 4.6 n_1 L / \bar{V}_{\bar{z}} = 4.35$ for ①

$R_L = .165$ $\quad = 5.50$ ②

$R_L = .359$ $\quad = 2.15$ ③

$$R(①) = \sqrt{\frac{1}{0.01}(.058)(.245)(.220)(.58 + 0.47 * .203)} = .460$$

$$R(②) = \sqrt{\frac{1}{0.01}(.058)(.245)(.57)(.58 + 0.47 * .165)} = .730$$

$$R(③) = \sqrt{\frac{1}{0.01}(.058)(.245)(.220)(.58 + 0.47 \times .359)} = .484$$

WIND Load (cont.)

- $g_r = 4.14$ (From previous calcs.)
- $G_f = 0.925\left(\frac{1+1.7(.293)\sqrt{(3.4)^2(.859)^2+(4.14)^2(.46)^2}}{1+1.7(.293)(3.4)}\right) = .9398 \approx .94$ ①

$G_f = 0.925\left(\frac{1+1.7(.293)\sqrt{(3.4)^2(.892)^2+(4.14)^2(.73)^2}}{1+1.7(3.4)(.293)}\right) = 1.076 \approx 1.08$ ② ⇐ Governs

$G_f = 0.925\left(\frac{1+1.7(.293)\sqrt{(3.4)^2(.859)^2+(4.14)^2(.484)^2}}{1+1.7(3.4)(.293)}\right) = .9478 \approx .95$ ③

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

For ①: $L/B = 1.1$, $h/L = .804$

- 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: $L/B = 5.56$, $h/L = .635$ $A = 1800 \Rightarrow R_f = 0.8$

- Walls: $C_{p,W,NS} = .8$
  $C_{p,L,NS} = -.2$
  $C_{p,S,NS} = -0.7$

- Roofs: $C_{p,roof,NS}(0-31.75) = -.94$
  $C_{p,roof,NS}(31.75-63.5) = -.85$
  $C_{p,roof,NS}(63.5-100) = -.55$

h/L	0 - h/2	h/2 - h	h - 2h
0.5	-0.9	-0.9	-0.5
0.635	-.94	-.85	-.55
1.0	-1.04	-0.7	-0.7

For ③: $L/B = .542$, $h/L = 1.63$ $A = 39 \times 72 = 2808 \Rightarrow R_f = 0.8$

- Walls: $C_{p,W,NS} = 0.8$
  $C_{p,L,NS} = -0.5$
  $C_{p,S,NS} = -0.7$

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

WIND LOAD (cont.)

- Pressure on each surface : $P = qGC_p - q_i(GC_{pi})$ (Eq. 6-23)

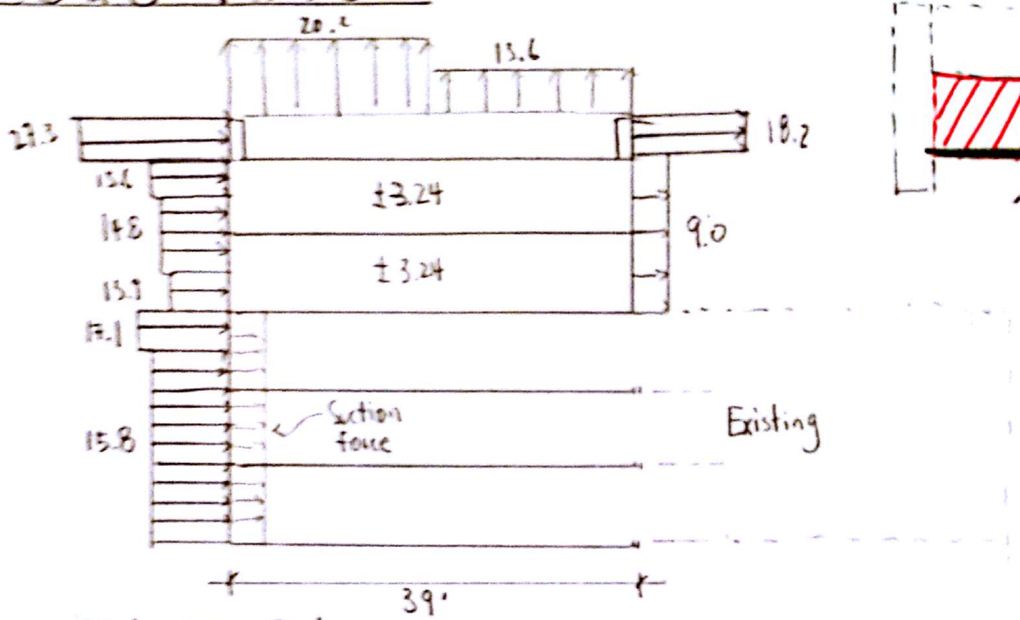
Use $G_f = 1.08$

Location	z (ft)	q_z (psf)	C_p	q_zGC_p (psf)	GC_{pi}	q_hGC_{pi} (psf)	$q_zGC_p - q_h(+GC_{pi})$ (psf)	$q_zGC_p - q_h(-GC_{pi})$ (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
Leedward								
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 2 0-31.5			-.94	-18.3			-21.5	-15.1
31.5-63.5	63.5	18.0	-.85	-16.5	0.18	3.24	-19.7	-13.3
63.5-100			-.55	-10.7			-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

Scanned by CamScanner

WIND LOAD (cont.)

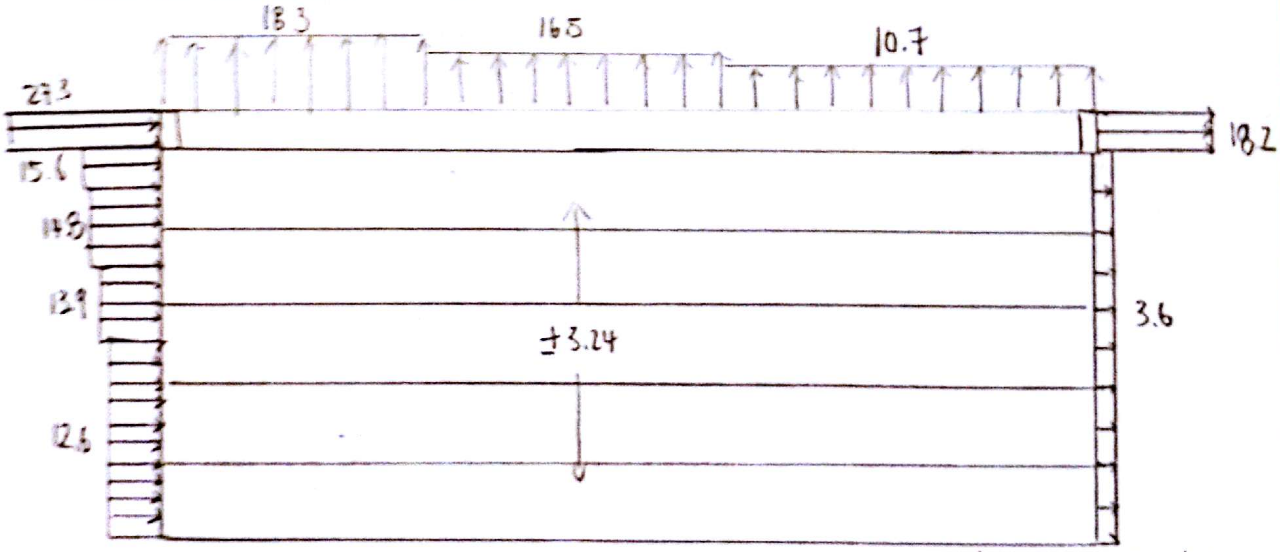
• ① & ③ Wind N-S



Trib. Width = 72'

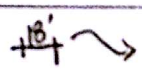
$$V = [15.8(17+11.5+6.5) + 17.1(6.5) + (13.9+9)(5.75) + (14.8+9)(5.75+5.25) + (15.6+9)(5.25) + (27.3+18.2)(2)] * 72 = 92.0^K$$

• ② Wind N-S

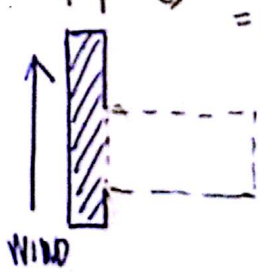


$$V = [(12.6+3.6)(17+11.5+6.5) + (13.9+3.6)(6.5+5.75) + (14.8+3.6)(5.75+5.25) + (15.6+3.6)(5.25) + (27.3+18.2)(2)] * 18$$

$$= 21.2^K$$



Trib. Width = 18'



* Base Shear (N-S) = $92 + 21.2 = 113.2^K$

[4] SEISMIC LOAD

- Structure not exempt (ASCE 7-02 §9.1.2)
- Site Class D (§ 9.4.1.2.1)
- $S_S = 0.365$ $S_{DS} = 0.367$
  $S_1 = 0.071$ $S_{D1} = 0.114$ (From USGS Design Maps Report)
- Seismic design category

  Table 9.4.2.1a, $0.33 \leq S_{DS} < 0.5 \rightarrow C$
  Table 9.4.2.1b, $0.067 \leq S_{D1} < 0.133 \rightarrow B$ ∴ SDC C.
- 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 ⇒ Seismic Use Group I (Table 9.1.3)

  ⇒ Seismic Impotant Factor, $I_e = 1.0$ (Table 9.1.4)
- Foundamental Period of the Building, Ta

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

  $C_t = 0.028$, $x = 0.8$ (Table 9.5.5.3.2)

  $T_a = 0.028\,(63.5)^{0.8} = 0.775$
- Seismic Response Coefficient, C_s

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

  $\geqslant C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.114}{.775 \times 3} = .049$ ⇐ Controls (Eq 5.5.5.2.1-2)

  ∴ $C_s = 0.049$

  check: Max $\left| \begin{array}{l} 0.044 S_{DS} I = .016 \\ 0.01 \checkmark \end{array} \right.$ $0.049 > 0.01$

  ∴ $C_s = 0.049$

SEISMIC LOAD

- Effective Total Seismic Weight

$P_f = 20\,psf < 30\,psf \Rightarrow$ Snow load in not considered (§9.5.3)

$W_{roof} = (18*100+72*39)(67+20) + 2(100+90)(696)$

$= 665.4^K$

$W_{floor} = 4(18\times100+72\times39)(100) + 2(100+90)(1056+1176+1176+1368)$

$= 3658.1^K$

$W_{Total} = W_{roof} + W_{floors}$
$= 3658.1 + 665.4$
$= 4323.5^K$

- Seismic Base Shear.

$V = C_s W = 0.049 * 4323.5 = 212^K$ (Eq. 9.5.5.2.1)

- Vertical Distribution of Seismic Forces (F_x)

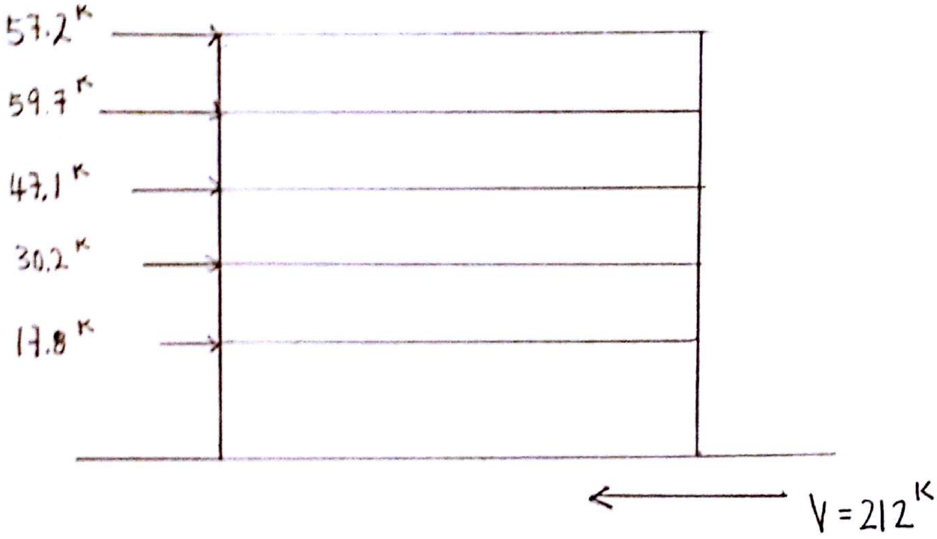
$F_x = C_{vx} \cdot V = \left[\frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}\right] V$ (Eq. 9.5.4.4)

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

Ta	K
0.5	1
0.775	1.18 ✓
2.5	2

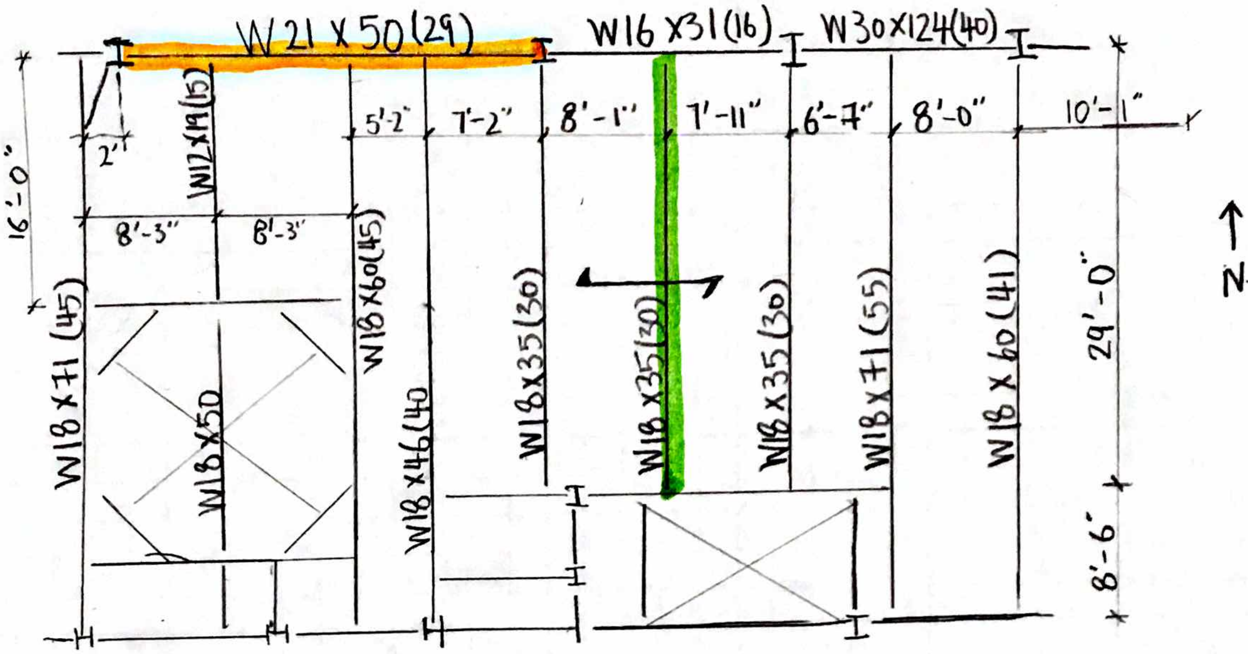
Level	hx (ft)	Wx (Kip)	$W_x h_x^k$	Cvx	Fx (Kip)	Vx (Kip)
Roof	63.5	665.4	89363.9	.270	57.2	57.2
5th	53	862.1	93369.1	.282	59.7	116.9
4th	41.5	907.7	73661.3	.222	47.1	164.0
3th	28.5	907.7	47218.0	.143	30.2	194.2
2th	17	980.6	27760.1	.084	17.8	212
	Σ	4323.5	331432.4	1		

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.
- Supperimposed Dead load:

Finishes	2 psf
Beam, Girder, Col.	8 psf
Misc	10 psf
	20 psf

- Live load : 100 psf

$W_{Total} = 100 + 23 = 123$ 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

2) Infill Beams

Dead: 20+46 = 66 psf (From Submission A)
LIVE: 100 psf

- Unshored Strength

$1.4D = 1.4(66psf) = 92.4\ psf$

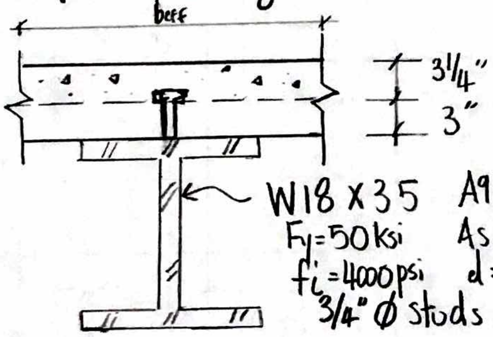
$1.2D + 1.6L = 1.2(66psf) + 1.6(20) = 112.2\ psf$

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

$M = \frac{wl^2}{8} = \frac{(898)(29)^2}{8} = 94.4\ k\cdot ft$

From Table 3-2, W18X35: $\phi M_p = 249\ K ft > 94.4\ K ft$ ∴ OK

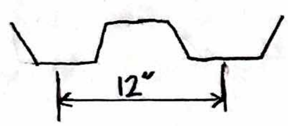
- Composite Strength



W18 X 35 A992
$F_y = 50ksi$ $A_s = 10.3 in^2$
$f'_c = 4000psi$ $d = 17.7 in$
3/4" Ø studs

$b_{eff} = min \begin{vmatrix} 29(12)/8 \\ 97/2 \end{vmatrix} + min \begin{vmatrix} 29(12)/8 \\ 95/2 \end{vmatrix}$

$= 87"$



From the drawing, # of studs = 30×2 = 60

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

From Table 3-21, $Q_n = 14.6^K$

$\Sigma Q_n = 30 * 14.6 = 438^K$

$\left.\begin{array}{l} V_{cmax} = 0.85 f'_c\, b_{eff} \cdot t = 0.85(4)(87)(3.25) = 961.4^K \\ V_{smax} = 515^K \text{ (Table 3-19)} \end{array}\right\} > 438^K$

∴ Partialy Composite

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

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

From Table 3-19,

@ $\Sigma Q_n = 388$, $Y_2 = 5.5 \Rightarrow \phi M_n = 501\ K\cdot ft$

Live Load Reduction

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

$$L = 100 \times \max \left| \begin{array}{l} 0.5 \\ 0.25 + \frac{15}{\sqrt{2 \times 232}} = 0.946 \end{array} \right. = 94.6\ psf$$

Load Combos

$1.4D = 92.4\ psf$
$1.2D + 1.6L = 1.2(66) + 1.6(94.6) = 230.6\ psf$

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

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

- check Wet Concrete Deflection:

$W_{wc} = 46(8) + 35 = .403\ plf$

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

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

- check LL Deflection:

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

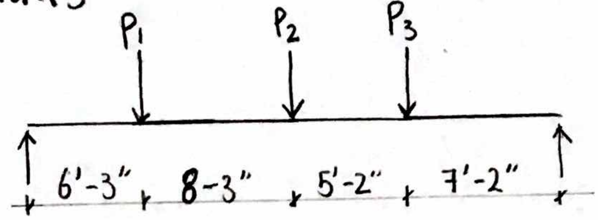
I_{LB}: $\Sigma Q_n = 438^k$
$Y_2 = 5.51''$

From Table 3-20,

@ $\Sigma_n = 388^k$, $Y_2 = 5.5 \Rightarrow I_{LB} = 1420\ in^4$ (conservative)

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

3) Girders



L = 26.83'

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

LL Reduction

$$L = 100 \times \max \begin{vmatrix} 0.5 \\ 0.25 + \frac{15}{\sqrt{2(26.83 \times 18.75)}} \end{vmatrix} = .723 = 72.3\,psf$$

P_1: $P_D = (66psf)(8.25')(16/2) = 4.36^K$
$P_L = (72.3psf)(8.25')(16/2) = 4.77^K$

$1.4D = 1.4(4.36) = 6.1^K$
$1.2D + 1.6L = 1.2(4.36) + 1.6(4.77) = 12.9^K \leftarrow$

P_2: $P_D = (66psf)(6.71)(18.75) = 8.3^K$
$P_L = (72.3psf)(6.71)(18.75) = 9.1^K$

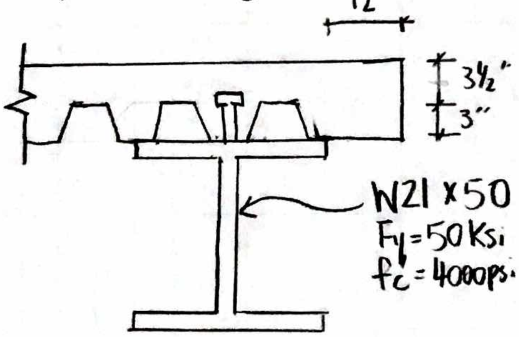
$1.2D + 1.6L = 1.2(8.3) + 1.6(9.1) = 24.5^K$

P_3: $P_D = (66psf)(6.17)(18.75) = 7.6^K$
$P_L = (72.3psf)(6.17)(18.75) = 8.4^K$

$1.2D + 1.6L = 1.2(7.6) + 1.6(8.4) = 22.6^K$

From SAP2000 Report, $M_u = 293.1$ 'K·ft (Page 7)

- Composite Strength



W21×50 A992
$F_y = 50\,ksi$ $A_s = 14.7\,in^2$
$f'_c = 4000\,psi$ $d = 20.8\,in$

$$b_{eff} = \min \begin{vmatrix} 26.83(12)/8 \\ 12" \end{vmatrix} + \begin{vmatrix} 26.83(12)/8 \\ 18.75(12) \end{vmatrix} = 52.2'$$

edge distance

\# of Stud = 29 × 2 = 58

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

3/4"ϕ Studs, 1 stud / rib, light concrete, f'_c = 4000psi
⇒ Q_n = 17.2 K (Table 3-21)

$\Sigma Q_n = 29\,(17.2) = 499^K$

$V_{c\,max} = 0.85(4)(52.2)(3.25) = 577^K$
$V_{s\,max} = 735^K$ (Table 3-19)
} > 499^K

∴ Partially Composite

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

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

$Y_2 = 6.25 - 2.81 = 3.44"$

From Table 3-19,

@ $\Sigma Q_n = 473^K$ $Y_2 = 3"$ ⇒ ϕM_n = 676 K·ft > 293.1 K·ft ∴ OK

- Unshored Strength

P_1: $P_D = 4.36^K$
$P_L = (20)(8.25')(16/2) = 1.32^K$

$1.4D = 1.4 \times 4.36 = 6.1^K$
$1.2D + 1.6L = 7.3^K$

P_2: $P_D = 8.3^K$
$P_L = (20)(6.71)(18.75) = 2.5^K$

$1.4D = 1.4 \times 8.3 = 11.6^K$
$1.2D + 1.6L = 1.2(8.3) + 1.6(2.5) = 14^K$

P_3: $P_D = 7.6^K$
$P_L = (20)(6.17)(18.75) = 2.3^K$

$1.4D = 1.4 \times 7.6 = 10.6^K$
$1.2D + 1.6L = 1.2(7.6) + 1.6(2.3) = 12.8^K$

From SAP2000 Report, M_u = 169 K·ft (Page 9)

From Table 3-2,

W21 X 50 : $\phi M_p = 413\ K\cdot ft > 169\ K\cdot ft$ $\therefore$ OK

- Check Wet Concrete Deflection

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

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

- Check LL Deflection

From SAP2000 Report, (Page 8)

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

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

7

Table: Element Forces - Frames

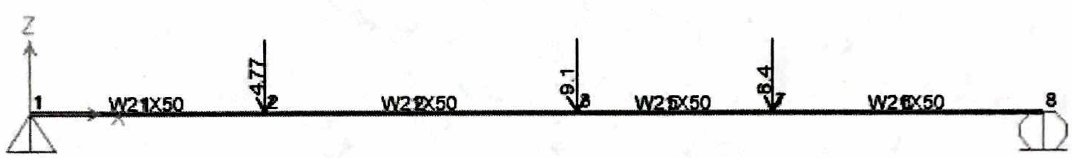
Table: Element Forces - Frames, Part 2 of 2

Frame	Station	OutputCase	M3	FrameElem	ElemStation
	ft		Kip-ft		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

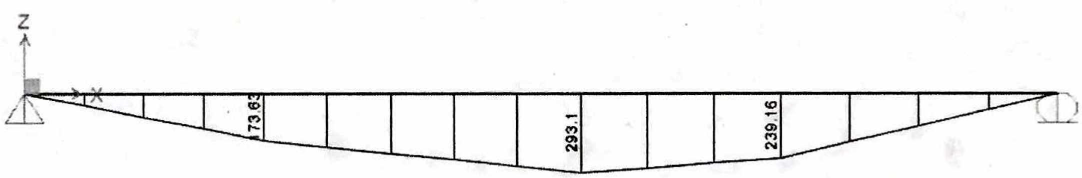
Table: Joint Displacements

Table: Joint Displacements

Joint	OutputCase	CaseType	U1	U2	U3	R1	R2	R3
			ft	ft	ft	Radians	Radians	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)

9

Table: Element Forces - Frames, Part 2 of 2

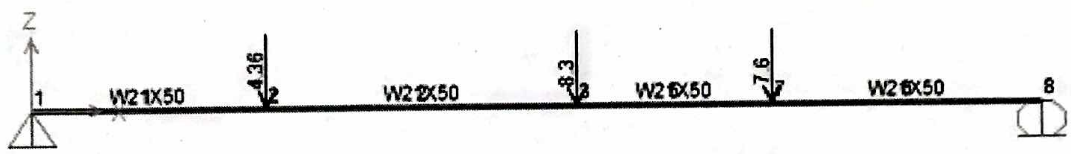
Table: Element Forces - Frames, Part 2 of 2

Frame	Station	OutputCase	M3	FrameElem	ElemStation
	ft		Kip-ft		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

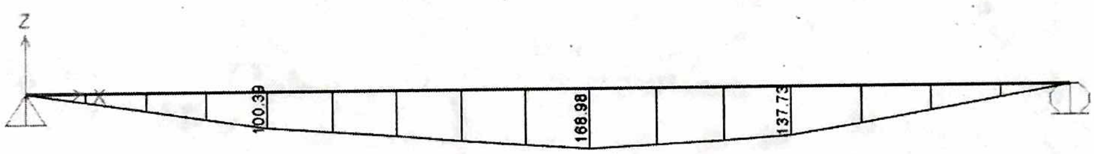
Table: Joint Displacements

Table: Joint Displacements

Joint	OutputCase	CaseType	U1	U2	U3	R1	R2	R3
			ft	ft	ft	Radians	Radians	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
$\boxed{3}$	$\boxed{\text{DEAD}}$	$\boxed{\text{LinStatic}}$	$\boxed{0}$	$\boxed{0}$	$\boxed{-0.036403}$	$\boxed{0}$	$\boxed{-0.000353}$	$\boxed{0}$
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)

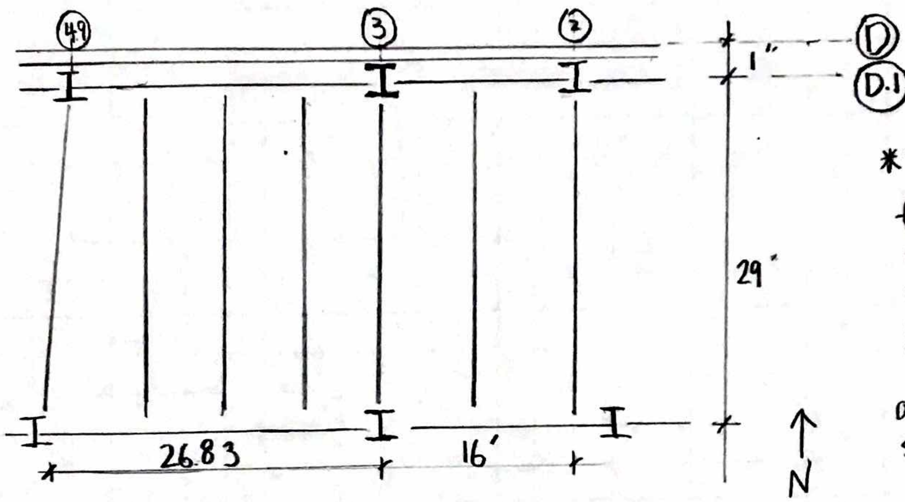


Moment Diagram (1.2D+1.6L Unshored Strength Check)

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. Celler, level 2, 5 are different because of the different floor layouts & Opening. (Influence area noted in Excel spread sheet.

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

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

Total Influence Area for the column: $\Sigma A = 6104\ ft^2$

$$\text{LL Reduction} = \max\left|\begin{array}{l} 0.4 \\ 0.25 + \frac{15}{\sqrt{6104}} \end{array}\right. = 0.44$$

➡ 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.

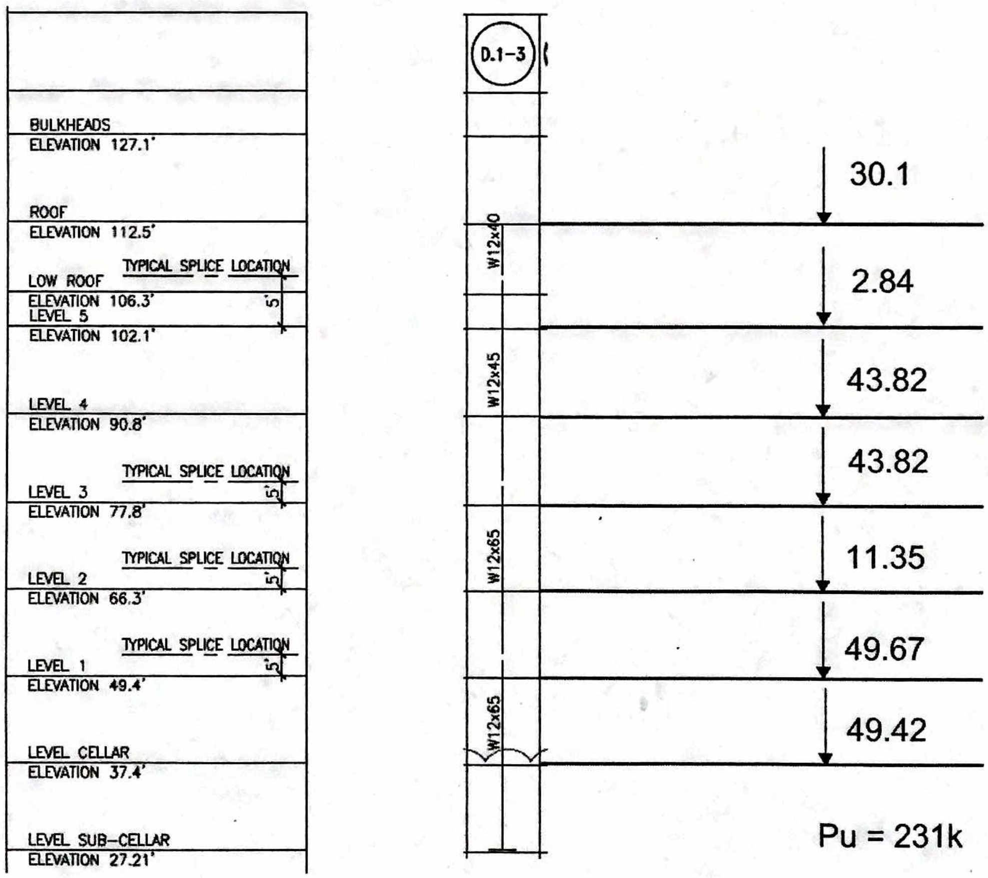
Controling Case = $1.2D + 1.6L + 0.5L_r$

$P_u = 231^K$

From Table 4-1, W12X65 @ 24": $\phi P_n = 442^K > 231^K$ ∴ OK

Interior Colum D.1-3

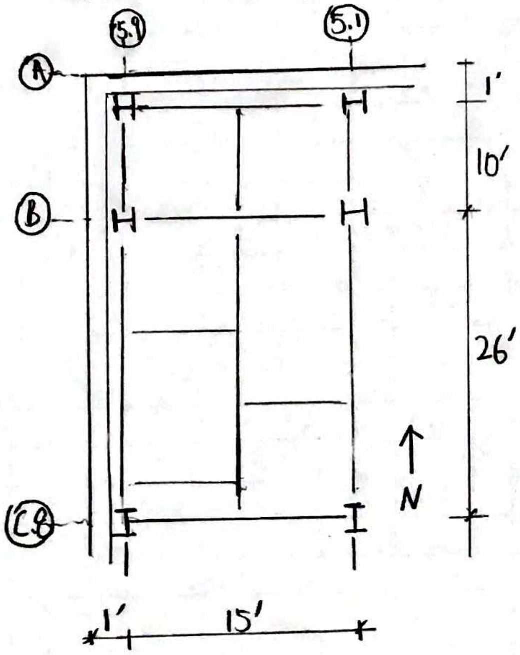
Level	Dead (psf)	Live (psf)	Tributary Area (ft^2)	Influence Area (ft^2)	Red. Live (psf)	Total Axial Load (K)		
						Dead	L or Lr	1.2D+1.6L+.5Lr
Roof	67	20	332	1285	20	22.24	6.64	30.01
Level 5	66	75	21.5	43	33	1.42	0.71	2.84
Level 4	66	75	332	1285	33	21.91	10.96	43.82
Level 3	66	75	332	1285	33	21.91	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
		∑Area:		6104			Pu:	230.94



Column B-5.9

Level 1-3 : W12X65
Level 3-5 : W10 X 60
Level 5-R : W10 X 33

Column loads



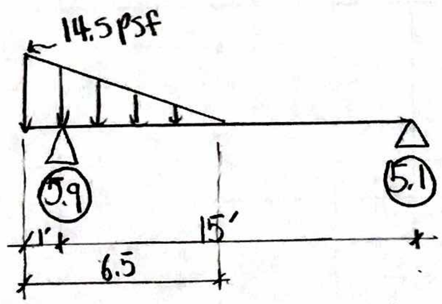
Tributary Area $= \left(\frac{10+26}{2}\right)\left(\frac{15}{2}+1\right)$
$= 153 \text{ ft}^2$

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

LL Reduction:

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

Column Snow Drift Loads:



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

$R_{(5.9)} = 36.9 \text{ Klf}$

- Snow load control for this column

Exterior Wall length = $(10+26)/2 = 18'$

Snow = 20 psf

Roof DL = 67 psf

Roof Exterior Wall load = 696 plf

Roof Drift Load = 36.9 klf

Floor DL = 66 psf

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

Floor LL = 75 psf

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

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

Floor DL:

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

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

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

Floor LL:

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

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

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

Floor: $1.2(29.1 + 31.3 + 31.3 + 34.7) + 1.6(4 \times 6.4)$

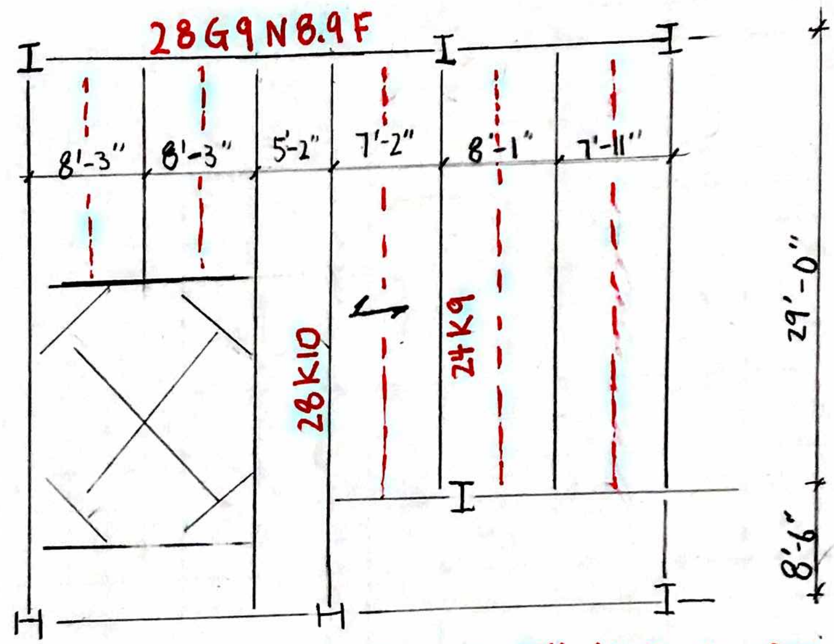
$= 162^{k}$

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

From Table 4-1.

W12x65 @ 28": $\phi P_n = 348^{k} > 191.2^{k}$ $\therefore$ OK

Alternative 1: Non-composite Steel Joist



1.0 C 20 deck with 2" LWC Topping.

1) Non-composite Decking.

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

Live load : 100 psf

Misc DL : 10 psf

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

- Max 3 span : 15-2" > 10'-1" ∴ OK
- Weight : 44 psf

Check 18 GA for total load : F_b = 36000 psf

Total load = 100 + 44 + 10 = 154 psf < 175 psf @ 10'-6" ∴ OK

Check 18GA for LL : Deflection = L/240

LL = 134 psf > 100 psf

2) Joists

3.0C18 Deck w 3" LW Conc.

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

LL. Reduction : $L = 100 \times \max\begin{vmatrix} 0.5 \\ 0.25 + \frac{15}{\sqrt{16 \times 29}} = .95 \end{vmatrix} = 95\ psf$

$W_u = [1.2(44+3+10) + 1.6(95)](8') = 1763\ plf$

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

- Recheck Decking
  SPAN = 4' continous over 3 spans
  Try 1.0C20 deck with 2" LWC Topping
  - Max 3 span : 8'-5" > 4' ∴ OK
  - Weight : 25 PSf

  check 20 GA for total load : $F_b = 36000\ psf$
  Total load = 100 + 25 + 10 = 135 PSf < 242 PSf @ 4'-6"
  ∴ OK

  check 20 GA for LL : deflection = L/240
  LL = 119 PSf > 100 PSf ∴ OK

- Recheck Joists

  not enough area for LL. Red.

  $W_{u_{tl}} = [1.2(25+3+10) + 1.6(100)](4) = 822\ plf$

  $W_{tl} = (38 + 100)(4) = 552\ plf$

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

From standard table : SJI p 54

24K9 : (d = 24", Wt = 10.3 lbs/ft)

W_{uTL} = 825 plf > 790 plf ∴ OK

W for L/360 = 436 plf

W for L/240 = 436 × 1.5 = 654 plf > 552 plf ∴ OK

10.3 plf / 4' = 2.58 psf < 3 psf ∴ allowance 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 plf

  W_{TL} = (38 + 100)(3) = 414 plf

  From standard table : SJI p

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

  W_{uTL} = 691 plf > 617 plf ∴ OK

  W for L/360 = 325 psf

  W for L/240 = 325 × 1.5 = 488 plf > 414 plf ∴ OK

  11.8'/3 = 3.9' ≈ 3' ∴ OK

- Girder : Span = 26.83', 26.83/3 = 9 spaces (From Eco. Joist Girder Table)

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

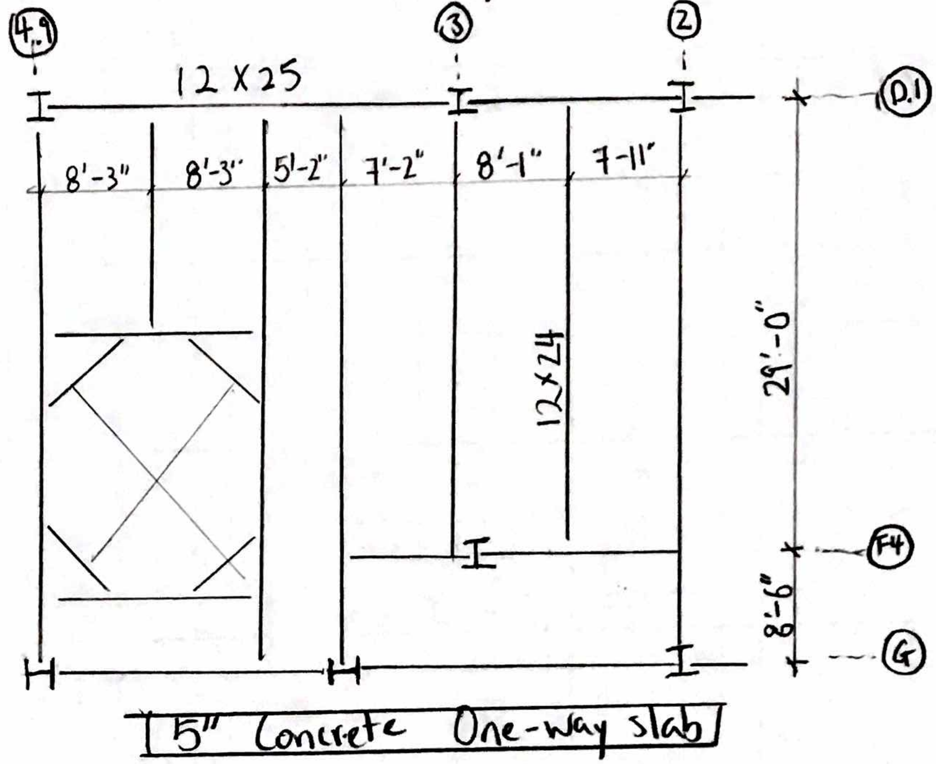
  ⇒ use 28G9N8.9F ⇒ weights approx. 27.5 plf

∴ Use

1.0 C20 deck with 2" LWC Topping 24K9 @ 4" ; 28K10 @ 3" 28G9N8.9F

(See graph)

Alternative 2: One-Way Slab



1) One-way Slab

$f'_c = 4000$ psi , $f_y = 60$ ksi

Finishes : 2psf
Supperimposed DL: 10psf
LL Red.: 94.6 PSF (From previous cals.)

· Estimate Slab thickness:

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

Use 5" slab (fire-rating 2hrs)

· Load Calculalation:

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

$LL = 94.6$ psf

- Load Combinations

$1.4D = 1.4(74.5) = 104.3$ psf
$1.2D + 1.6L = 1.2(74.5) + 1.6(94.6\text{ psf}) = 241$ psf ← controls

1' width of slab

$W_u = 241$ plf

- Max. Moment

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

- Calculate Reinforcement Required (As)

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

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

$A_s = 0.00113 \times 12'' \times 5'' = 0.069$ in²/ft

$A_{smin} = 0.0018 \times 12'' \times 5'' = 0.11$ in²/ft $>$ 0.069 in²/ft

$\therefore A_s = A_{smin} = 0.11$ in²/ft (ACI Sec 7.6.1.1)

$\therefore$ Use #3 bars with 12" ⇒ $A_s = 0.11$ in²/ft

- Moment Capacity

Assume $Cc = 0.75''$

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

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

$$\phi M_n = \phi f_y A_s \left(d - \frac{a}{2}\right)$$
$$= 0.9(60)(0.11)\left(4.06 - \frac{0.162}{2}\right)$$
$$= 23.6 \text{ k-in} = 1.97 \text{ k-ft}$$

1.97 k-ft $>$ 1.54 k-ft <u>OK</u>

- Shear Capacity (One-way shear)

$$V_u = \frac{1.15 W_u l_n}{2} = \frac{1.15(241\ \text{plf})(8')}{2} = 1108.6\ \text{lbs/ft}$$

$$V_c = 2\sqrt{f'_c}\, b_w d = 2\sqrt{4000} \times 12 \times (4.06) = 6163\ \text{lbs/ft} \quad \text{(ACI 22.5.5.1)}$$

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

- Max. Spacing

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

$$S_{max} \text{ (for crack control)} = \begin{cases} 15\left(\frac{40}{2/3(60)}\right) - 2.5 \times 0.75 = 13.13'' \\ 12\left(\frac{40}{2/3(60)}\right) = 12'' \end{cases} \quad \text{(ACI Table 24.3.2)}$$

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

$\Rightarrow$ use #3 w/ 12" spacing is <u>OK</u>.

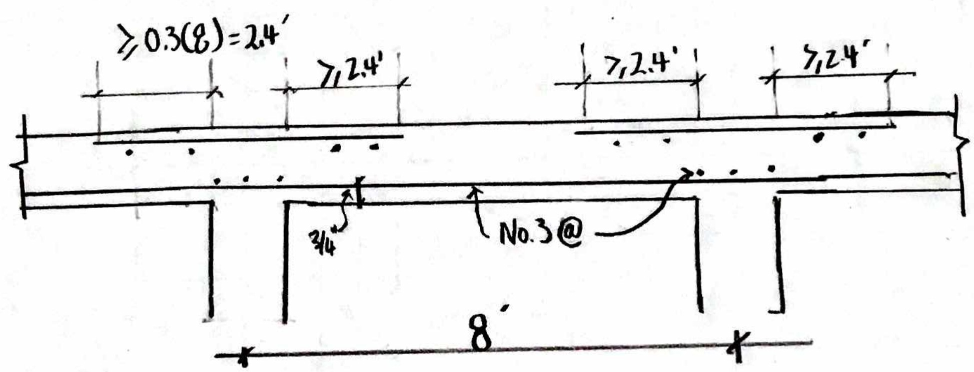
- Transverse Reinforcement (S&T)

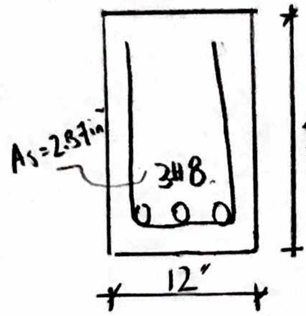
$$A_s (S\&T) = 0.0018 \times 12 \times 5 = 0.11\ ft^2/ft$$

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

$\therefore$ use #3 @ 12" $\Rightarrow A_s = 0.13\ ft^2 > 0.11\ ft^2$ <u>OK</u>

- Draw the section





• Flexural Strength

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(60)(2.37)}{0.85(4)(12)} = 3.49''$$

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

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$$
$$= 0.9(2.37)(60)\left(21.5 - \frac{3.49}{2}\right)$$
$$= 2528\ \text{K}\cdot\text{in} = 211\ \text{K}\cdot\text{ft} > M_u = 203\ \text{K}\cdot\text{ft} \quad \underline{\text{OK}}$$

Verify stain in steel

$$\varepsilon_s = \left(\frac{d-c}{c}\right)\varepsilon_{cu} = \left(\frac{21.5-4.1}{4.1}\right) * 0.003 = 0.0127 > 0.005$$
$$\Rightarrow \phi = 0.9$$

∴ Check Req. Reinforcement ($A_{s\,min}$)

$$A_{s\,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{4000}}{60000} \times 12 \times 21.5 = .81\ in^2$$

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

∴ Asmin is satisfied.

• Shear Strength

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

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

$$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{4000}\,(12)(21.5) = 32.6^K$$

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

∴ Provide min. shear reinforcement.

2) Infill Beam

$W_u = (241\ psf)(8) = 1.93\ Klf$

$M_u = \frac{(1.93)(29)^2}{8} = 203\ K\cdot ft$

- Calculate a tentative ρ

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

$M_n = \frac{M_u}{\phi} = \frac{203^{1K}}{0.9} = 256\ K\cdot ft$ (Assume $\phi = 0.9$)

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

$R = \omega f'_c (1 - 0.59\omega)$
$= .213(4)(1 - 0.59 \times 0.213) = 0.745\ Ksi$

$M_n = Rbd^2 \Rightarrow bd^2 = \frac{256 \times 12}{0.745} = 4123.5\ in^3$

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

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

$\boxed{\text{Try } 12 \times 24}$ $d = 21.5''$, $c = 2.5''$

- Req. As

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

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

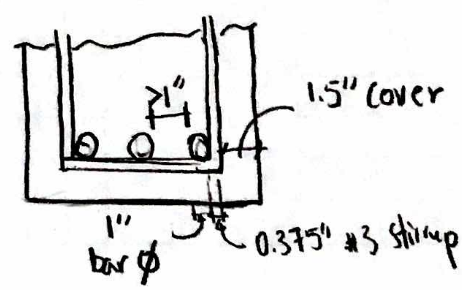
$A_s = 0.0088 \times 12 \times 21.5 = 2.28\ in^2$

$\therefore$ use 3 #8 $A_s = 2.37\ in^2$

- Minimum spacing of bars

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

$8.75'' > 12''$ OK



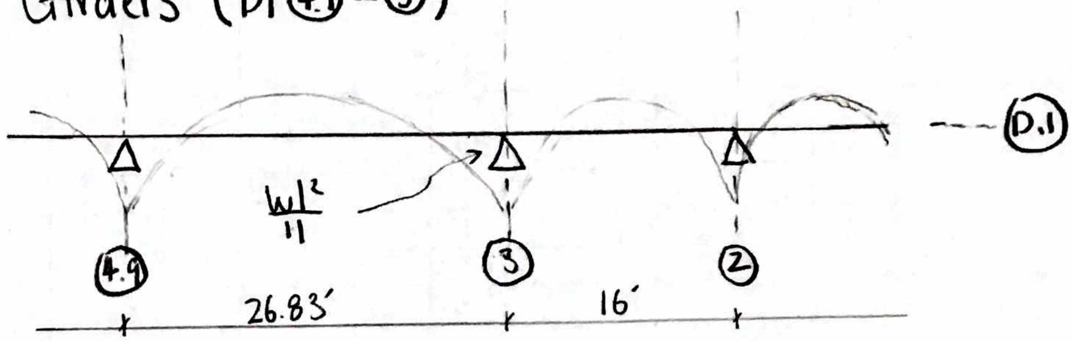
$V_s \leq 4\sqrt{f'_c}\, b_w d \Rightarrow S_{max} \leq \frac{d}{2} = 10.75''$

$\therefore$ use $S = 10''$

$$A_{s\,min} - \begin{cases} 0.75\sqrt{f'_c}\,\frac{b_w S}{f_{yt}} = \frac{0.75\sqrt{4000}\,(12)(10)}{60000} = .095\ in^2 \\ 50\,\frac{b_w S}{f_{yt}} = \frac{50\,(12)(10)}{60000} = 0.1\ in^2 \leftarrow \end{cases}$$

$\therefore$ use #3 stirrups at $10''$ ✓

3) Girders (D1 4.9 – 3)



- **Determine Girder trial Size**

- Assumption: uniform Load

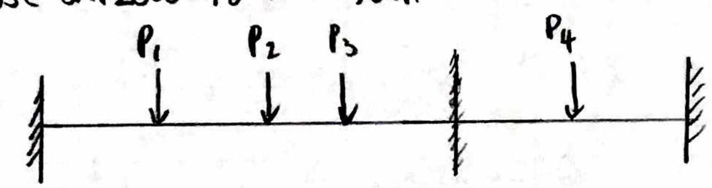
$$M_{max} = \frac{wl^2}{10} \quad \text{(To be conservative)}$$

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

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

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

- Use SAP2000 to find M_u.



$$P_1 = (241\text{ PSF})(8.25)(37.5)/2 = 37.3^k$$
$$P_2 = (241\text{ PSF})(6.71)(37.5)/2 = 30.3^k$$
$$P_3 = (241\text{ PSF})(6.17)(37.5)/2 = 27.9^k$$
$$P_4 = (241\text{ PSF})(8)(29)/2 = 28.0^k$$

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

$\therefore$ use $M_u = 270.3$ K·ft for design

Use Simplified Design method:

$$bd^2 \approx 20 M_u \quad (b \approx \tfrac{1}{2}d)$$
$$(\tfrac{1}{2}d)(d) = 20(270.3) = 5406$$
$$\Rightarrow d = 22.1''$$

$$A_s \approx \frac{M_u}{4d} \approx \frac{270.3}{4(22.5)} \approx 3.0\text{ in}^2 \checkmark$$

Design 12 X 25 in SAP2000 (see folling data)

Table: Element Forces - Frames, Part 1 of 2

Table: Element Forces - Frames, Part 1 of 2

Frame	Station	OutputCase	CaseType	P	V2	V3	T	M2
	ft			Kip	Kip	Kip	Kip-ft	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

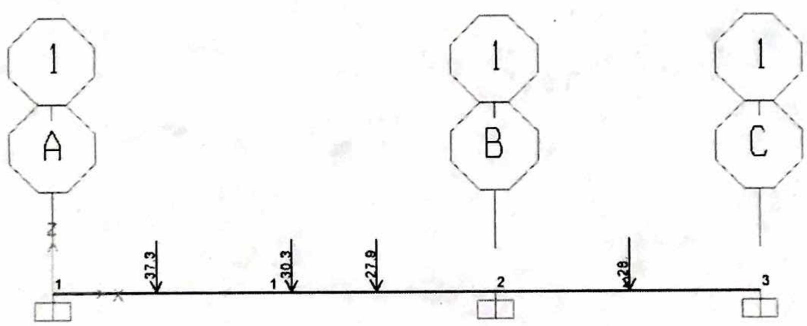
Table: Element Forces - Frames, Part 2 of 2

Table: Element Forces - Frames, Part 2 of 2

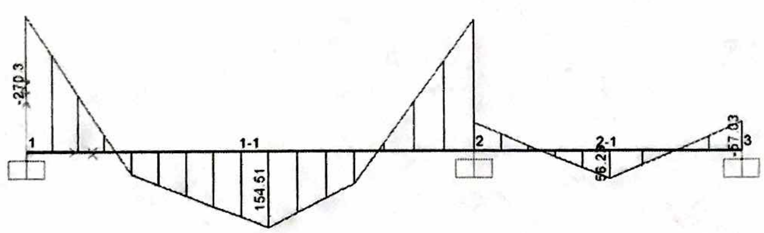
Frame	Station	OutputCase	M3	FrameElem	ElemStation
	ft		Kip-ft		ft
1	0	DEAD	-270.3	1-1	0
1	1.5625	DEAD	-191.1359	1-1	1.5625
1	3.125	DEAD	-112.0268	1-1	3.125
1	4.6875	DEAD	-32.9725	1-1	4.6875
1	6.25	DEAD	46.0268	1-1	6.25
1	6.25	DEAD	46.0268	1-1	6.25
1	7.9	DEAD	67.8455	1-1	7.9
1	9.55	DEAD	89.6029	1-1	9.55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

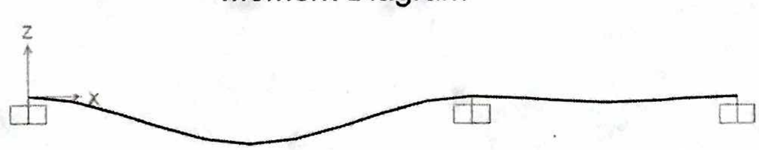
Frame	Station ft	OutputCase	M3 Kip-ft	FrameElem	ElemStation ft
1	16.2233	DEAD	124.758	1-1	16.2233
1	17.9467	DEAD	94.9414	1-1	17.9467
1	19.67	DEAD	65.058	1-1	19.67
1	19.67	DEAD	65.058	1-1	19.67
1	21.46	DEAD	-15.9932	1-1	21.46
1	23.25	DEAD	-97.1165	1-1	23.25
1	25.04	DEAD	-178.3118	1-1	25.04
1	26.83	DEAD	-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



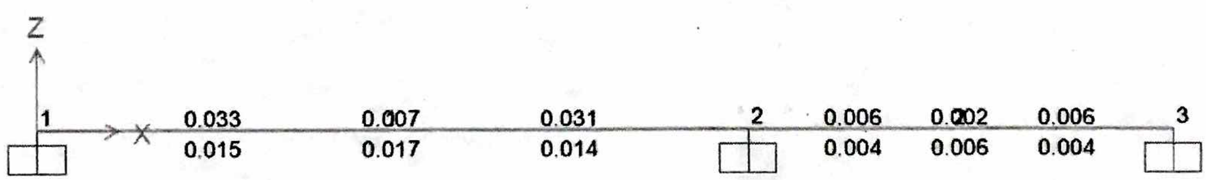
Moment Diagram



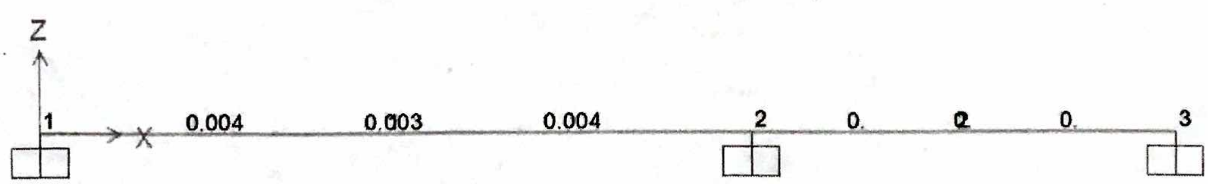
Deflection Shape

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	0.003963	DCON1	0
2	12 x 25	Beam	No Messages	12.0417	DCON1 (Sp)	0.002038	DCON1 (Sp)	0.002038	DCON1	0

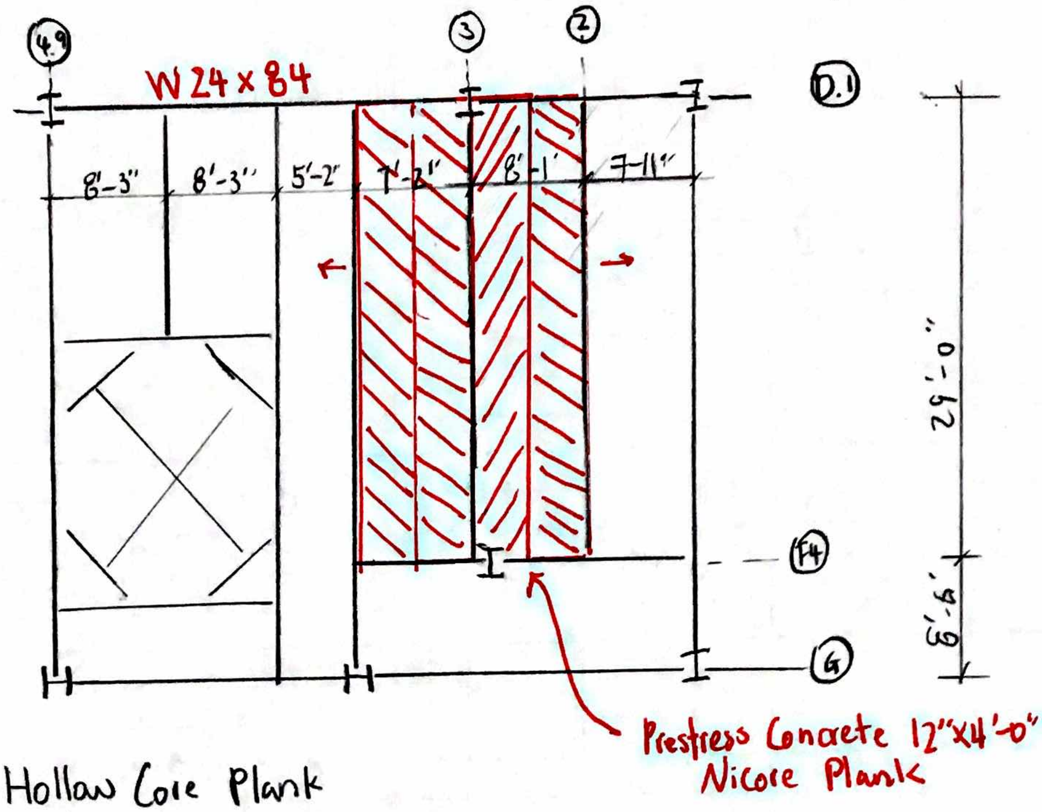


Longitudinal Reinforcement



Shear Reinforcement

Alternative 3: Hollow-Core Plank System



1) Hollow Core Plank

Dead load:

Finishes	2 psf
Framing	8 psf
Misc	10 psf
	20psf

Live Load: 100 psf

$1.2D + 1.6L = 1.2(20) + 1.6(100) = 184$ 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

2) Girders. (Non-composite steel beam)

$DL = 20\,psf + 77\,psf = 97\,psf$

$LL = 100\,psf$

LL Red. $= 72.3$ psf (From Previous calcs)

- Load Combs

$1.4D = 1.4(97) = 135.8\,psf$

$1.2D + 1.6L = 1.2(97) + 1.6(72.3) = 232.1\,psf$ ← controls

Girder { Length = 26.83 ft
spacing = $(37.5/2) + 1 = 19.75$ ft

$W_u = 19.75(232.1) = 4.58\,Klf$

$$M_u = \frac{wl^2}{8} = \frac{(4.58)(26.83)^2}{8} = 412\ K\cdot ft$$

* Detailed attachment between planks and girders to provide the fully braced condition.

- check LL deflection

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

$\Rightarrow I \geq 2059\ in^4$

From Table 3-2 Steel Manual

W24 X 84: $\phi M_p = 840 > 412$ K.ft <u>OK</u>

$I_x = 2370\ in^4 > 2059\ in^4$ <u>OK</u>

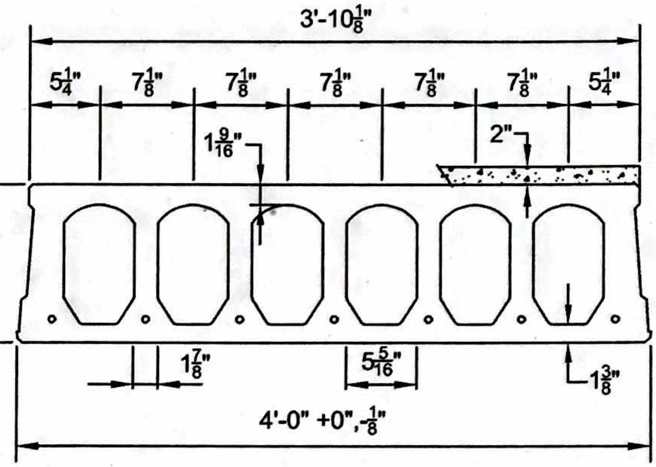
∴ use W24 X 84 (d = 24.1)

Prestressed Concrete
12"x4'-0" NiCore Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 361 in.^{2}	Precast b_w = 14.25 in.
I_c = 7840 in.^{4}	Precast S_{bcp} = 1081 in.^{3}
Y_{bcp} = 7.26 in.	Topping S_{tct} = 1644 in.^{3}
Y_{tcp} = 4.74 in.	Precast S_{tcp} = 1653 in.^{3}
Y_{tct} = 6.74 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 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

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.

	Existing Composite Steel	Alternative 1 Steel Joists	Alternative 2 One-Way Slab	Alternative 3 Hollow Core Planks
Architectural Coordination				
Depth	27	31"	30"	36"
Fire Rating	2 hr	2 hr	2 hr	2 hr
Fire Rating Type	Cementitous/Sprayed	Cementitous/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 Bracd Frame	yes	yes	no	no
Moving Forward?	N/A	YES	YES	NO

1) Weight per bay

Existing – Composite Steel

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

28.1^k

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

Alternative 1 – Steel Joists & Joist Girder

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

27.2^k

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

Alternative 2 – One-Way Slab

- Slab : $(150\text{ pcf})(5/12')(29')(16') = 29^k$
- 12X24 : $3(150\text{ pcf})\left(\frac{12 \times 24}{144}\right)(29') = 26.1^k$
- 12 X 25 : $(150\text{ pcf})\left(\frac{12 \times 25}{144}\right)(26.83) = 8.4^k$

63.5^k

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

Alternative 3 – Hollow Core Planks

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

38^k

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

2) Cost per bay

Existing – Composite Steel

use B1010 254 0800 (RS Means 2014 Assemblies)
- Bay size 25 X 20
- SDL = 75 psf
- depth = 1'-9"

Total Base Cost /SF = \$ 24.1 /SF

Alternative 1 – Steel Joist & Joist Girder

use B10 B10 2504200
- Bay Size 20X25
- SDL = 75 PSF
- depth = 26"

Total Base Cost / SF = \$ 18.43 /SF

Alternative 2 – One-Way Slab

use B10 B10 226 4600
- Bay Size 20X25
- SDL = 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 20X25
- SDL = 100 PSF
- Total depth : 30"

Total Base Cost /SF = \$ 26.5 /SF