Structural Notebook
Submission B

AC Hotel Philadelphia
Philadelphia, Pennsylvania

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Table of Contents

Executive Summary

AC Hotel Philadelphia is a 15-story residential transient hotel (including penthouse) located in the heart of downtown Philadelphia. This new hotel, owned by Baywood Hotels, will be built on top of the previous NFL Films and Warner Bros distribution center, a historic two-story building located at the corner of Florist and North 13th Street in Philadelphia.

The original two-story, 31'-0" tall building is a load bearing masonry structure. In order to properly satisfy the proposed addition, a mat foundation of varying thickness will be installed and the building will be gutted and restructured. The new construction will consist of composite steel at the bottom two levels, supporting a 12-story steel-frame structure atop, capped with a penthouse. The typical floor to floor height measures 10'6". Multiple 14" shear walls make up the lateral system until floor 3 where braced frames are utilized for architectural/spatial purposes including door and window openings.

AC Hotel Philadelphia was designed using the 2009 edition of the International Building Code and ASCE 7-05 was used to determine lateral loads on the building. The City of Philadelphia Building Code (with current amendments) and the 2014 version of "AC Hotels by Marriott Design Standards" were also used as references. The Philadelphia Historical Commission also influenced the project boundaries.

The purpose of this report is to identify the structural loads used in the design of AC Hotel Philadelphia. Gravity, wind and seismic loads are established in the following report. A code analysis was completed in order to have an accurate understanding of the design loads used for 230 North 13th Street. Codes were used in accordance to the actual design codes applied when designing the building.

AC Hotel Philadelphia

Baywood Hotels | 230 North 13th Street, Philadelphia, Pa

Project Information

- Occupancy: Residential transient hotel
- Stories: parking garage + 14 levels above grade + Mech. Penthouse & Rooftop Terrace
  - 192ft. Above sidewalk grade
- Overall project cost: $35,000,000
- Size: 107,680 sq.ft.
- Construction Dates: Fall 2015 – Summer 2017

Project Team

Owner: Kurt Blorstad
General Contractor: Clemens Construction
Architect: Spg3
Structural Engineer: Holbert Apple Associates
MEP: McHugh Engineering

Features:

- 150 luxury units
- Underground, valet parking via car elevator
- Exclusive restaurant for guests
- Fitness center & indoor pool
- Green Roofs
  - Extensive (2nd & 3rd Levels)
  - Intensive (Rooftop Terrace)

Structure:

- Foundation
  - Mat-slab
  - Underpinning of adjacent structures during construction
- Framing
  - Structural steel framing
  - Composite deck (normal-weight concrete)
  - Precast hollow-core plank girder slab system
- Lateral System
  - Concrete shear walls (lower levels)
  - Concentric braced frames (upper levels)

MEP:

- Mechanical
  - (4) three-ton air handling units
  - Water-source heat pump
  - Energy recovery wheel on the roof used to mix outside air with return air
  - Plethora of fans used to exhaust class 3&4 air
- Electrical
  - 600KW Emergency generator on roof
  - 2500A Main Circuit Breaker

JESSE BORDEAU ~ Structural Option
http://jbordeau18.wix.com/thesis

Site Location

230 North 13th St. is located in Philadelphia, Pennsylvania in proximity to the Liberty Bell, The Franklin Institute and the Eastern State Penitentiary. The site lies northwest of center city, offering dwellers a beautiful view of the Philadelphia skyline. Figures 1 and 2 clarify the exact location below.

Figure 1: Overhead view of 230 North 13th St in Philadelphia, Pa (Courtesy of Google Maps)

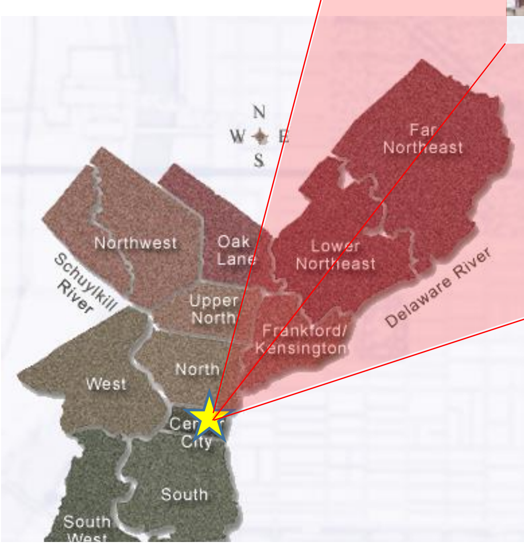


Figure 2: Map of Philadelphia (Courtesy Google Maps)

Documents used in preparation for this report

Listed below are the codes and other supporting documents which were used to determine loads for the AC Hotel Philadelphia.

- American Society of Civil Engineers
  - ASCE 7-05
- International Code Council
  - International Building Code 2009
- Construction drawings
  - Courtesy Holbert Apple Associates
- Course notes from previous semesters
- Hambro Composite Floor System Design Guide
- Girder-Slab System LRFD Version Design Guide v3.1
  - Courtesy Holbert Apple Associates

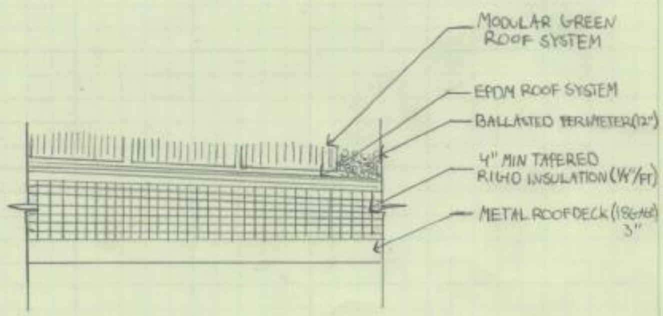
Gravity Load Determination (Dead, Live & Snow)

Roof Loads

The roof load calculated below is for the intensive green roof used in several locations around the building. Loads are compared to code minimum (IBC ch 16, Table 1607.1) within each section. Original loads, determined by professionals are located at the end of the gravity load portion of this report.

NOTEBOOK SUBMISSION A | GRAVITY LOADS | JESSE BORDEAU | 1

1. Roof Dead & Live Loads (2^{nd}, 3^{rd} Story, Roof & Penthouse)



Design Loads:

- Live Load (Min): 100psf → roof garden [IBC 2009, Section 1607.1, Table 1607.1]

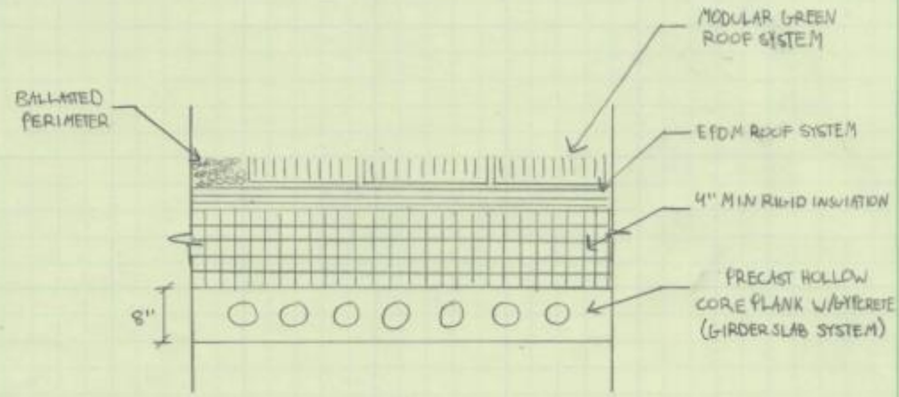
- Dead Load:

Modular Green Roof System = 50psf
Ballasted EPDM Roof System = 15psf
Tapered Rigid Insulation = (4")(1.5 psf/in) = 6psf
Metal Roof Deck (18gage) = 3psf
Mechanical Allowance = 4psf
Superimposed Dead Load = + 3 psf

Design DL = 81psf

NOTEBOOK SUB A | GRAVITY CONT. | JESSE BORDEAU 2

ROOF DEAD & LIVE LOAD (PENTHOUSE)



DESIGN LOADS:

-LIVE LOAD: 100PSF

-DEAD LOAD:

MODULAR GREEN ROOF SYSTEM = 50PSF

BALLASTED EPDM ROOF SYSTEM = 15 PSF

RIGID INSULATION (4" MIN) = 6PSF

8" PRECAST HOLLOW-CORE PLANK = $8/12(150) = 100PSF$
2" CONCRETE TOPPING

MECHANICAL ALLOWANCE = 4 PSF

SUPERIMPOSED DL = 3 PSF

DESIGN DL = 178 PSF

Note: Intensive green roofs require higher design criteria because of the possibility of human traffic over it. Modular Green Roof Systems vary in weight, therefore an average load was applied.

NOTEBOOK SUB A | SNOW LOADS | JESSE B. | 5

SNOW LOAD:

USING ASCE 7-05 [FIGURE 7-1] → $p_g = 30$ PSF

$p_f = 0.7 C_e C_t I p_g$

C_e - EXPOSURE FACTOR [TABLE 7-2]

C_t - THERMAL FACTOR

I - IMPORTANCE FACTOR

DETERMINE C_e:

[TABLE 7-2] TERRAIN CAT. B [SECTION 6.5.6] → FULLY EXPOSED → $C_e = 0.9$

DETERMINE C_t:

[TABLE 7-3] $C_t = 1.0$

DETERMINE I:

[TABLE 1-1] OCCUPANCY CATEGORY → II

[TABLE 7-4] $I = 1.0$

DETERMINE p_f:

$p_f = (0.7)(1.0)(1.0)(30) = \underline{21\ PSF}$

NOTEBOOK SUB A | SNOW DRIFT CALCS | JESSE B | 6

$\gamma = \min \begin{cases} 0.13p_g + 14 = 0.13(21)+14 = \underline{16.7} \\ 30 \text{ PCF} \end{cases}$

$h_b = \frac{p_{s(balanced)}}{\gamma} = \frac{21}{16.7} = 1.26 \rightarrow h_c = 191-181 = 10' \rightarrow \frac{h_c}{h_b} = \frac{10}{1.26} = 7.94 > 0.2 \therefore$ REQ'D

LEEWARD $\ell_u = 29' > 25' \therefore$

$h_d = 0.43\sqrt[3]{\ell_u}\sqrt[4]{p_g+10} - 1.5$

$= 0.43\sqrt[3]{29}\sqrt[4]{16.7+10} - 1.5$

$= \underline{1.50} < h_c = 5.25$

WINDWARD $\ell_u = 13 \le 20 \therefore h_d = 0.75[(0.43\sqrt[3]{26.7}\sqrt[4]{26.7} - 1.5$

$= 1.42$

LW CONTROLS

$h_d < h_c \therefore p_d = h_d\gamma = (1.5)(16.7) = 25.1$ PSF

$w = 4h_d = 4(1.5) = 6'$

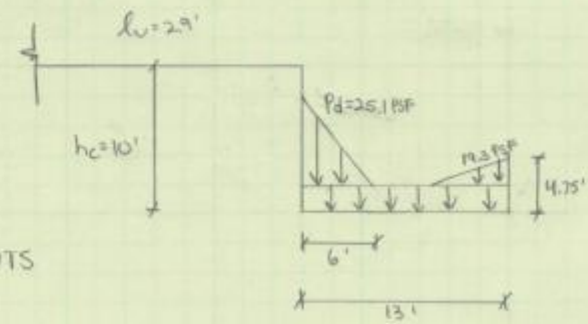
1.167

PARAPET $\ell_u = 13 < 20 \therefore h_d = 0.75[0.43\sqrt[3]{20}\sqrt[4]{26.7} - 1.5$

$= 1.153 < h_c \therefore p_d = 1.153(16.7) = 19.3$

$w = 4(1.153) = 4.6'$

NORTH ELEVATION
ROOFTOP PARAPET



NTS

$\gamma = 16.7$

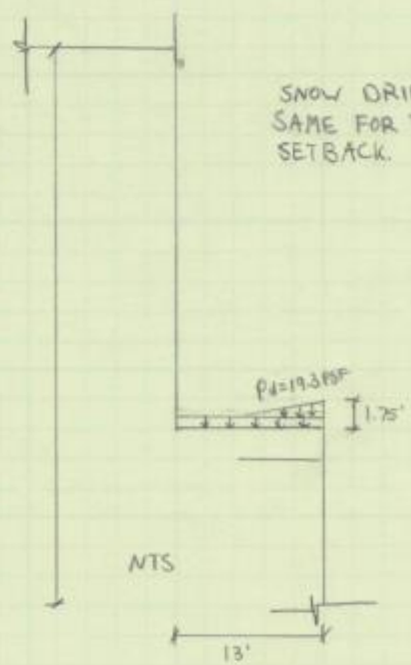
$h_b = 1.26 \rightarrow h_{c2} = 181 - 30 = 151' \rightarrow \frac{h_c}{h_b} = \frac{151}{1.26} > 0.2 \therefore$ REQ'D

SNOW WILL NOT OVERCOME UPPER PARAPET $\therefore$ DESIGN FOR SNOW DRIFT FROM LOWER PARAPET

$l_u = 13 < 25 \therefore h_d = 1.153 \rightarrow p_d = 19.3,\ w = 4.6'$

SNOW DRIFT VALUE WILL BE THE SAME FOR THE ENTIRE 2ND STORY SETBACK.

SOUTH ELEVATION
2ND STORY PARAPET

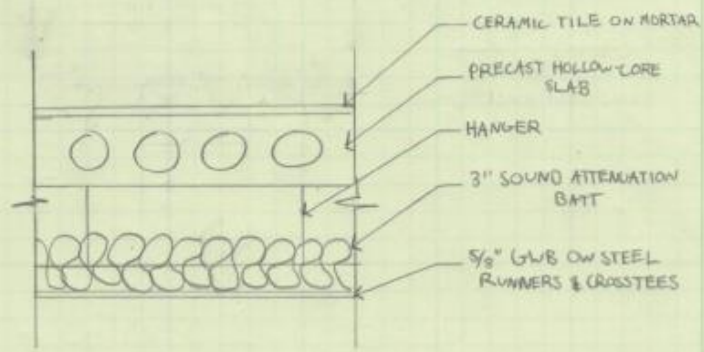


Floor Loads

NOTEBOOK SUB A | GRAVITY CONT. | JESSE BORDEAU 3

TYP. FLOOR DEAD & LIVE LOADS (GIRDER SLAB SYSTEM)

DB → D-BEAM
DB8x37
DB8x61



DESIGN LOADS

- LIVE LOAD: 40 PSF UNIFORM [IBC 2009 TABLE 1607.1 → RESIDENTIAL HOTEL]
- DEAD LOAD:

8" PRECAST PLANK = $8/12(150)$ = 100 PSF
HANGERS = 2 PSF
3" SOUND ATTENUATION BATT = $3''(1\ PSF/IN)$ = 3 PSF
CERAMIC TILE ON 1/2" MORTAR = 16 PSF
5/8" GWB = $(5/8'')(0.55\ PSF/\tfrac{1}{16}'')$ = 2.75 PSF
MECH. ALLOWANCE = 4 PSF
SUPERIMPOSED DL = 3 PSF
\+ ______
131 PSF

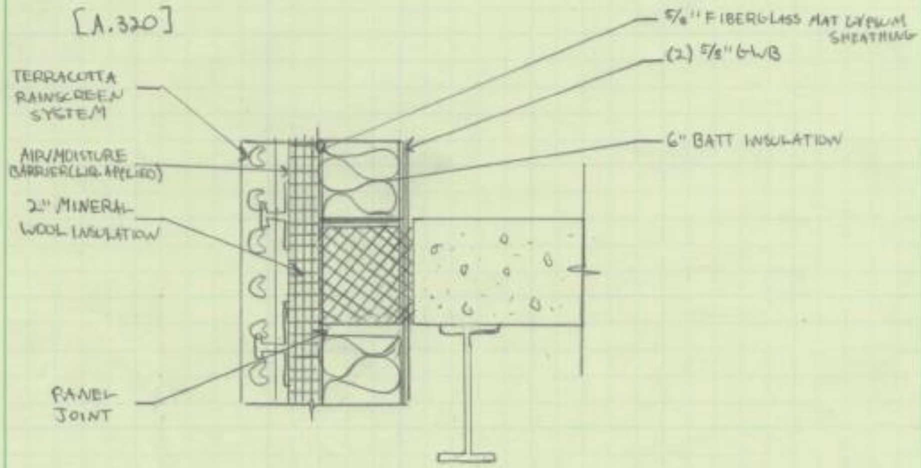
The girder-slab system is utilized to benefit construction efficiency and to reduce floor-to-floor height

Exterior Wall Loads

NOTEBOOK SUB A	GRAVITY CONT	JESSE BORDEAU	4

TYP. WALL SECTION DEAD & LIVE LOADS

[A.320]



DESIGN LOADS:

- DEAD LOAD

TERRACOTTA RAIN SCREEN SYSTEM = 10 PSF

MINERAL WOOL INSULATION (2") = (2")(1 PSF/IN) = 2 PSF

BATT INSULATION (6") = (6")(4 PSF/IN) = 24 PSF

GYPSUM WALL BOARD (2) 5/8" = (2)(5)(0.55 PSF/1/8") = 5.5 PSF

AIR/MOISTURE BARRIER LIQUID APPLIED = 1 PSF

+ ______

DESIGN DL = 43 PSF

LOAD PATH: TYP EXTERIOR WALLS DO NOT BEAR ANY LOADS. THE LOAD IS TRANSFERRED FROM THE PANELS THROUGH THE CLIPS AND INTO THE GIRDER SLAB SYSTEM. FROM HERE, THE LOAD IS ABLE TO TRANSFER INTO THE BRACE FRAMES AND DOWN INTO THE FOUNDATION.

Listed below are the dead load values used by the engineers who originally determined the loads for AC Marriott Philadelphia.

Table 1: Superimposed dead loads

Superimposed Dead Loads (in addition to structure self-weight)	
Area	Loading [psf]
Typical Roof	30
Floors	10
Intensive Green Roof	200
Extensive Green Roof	60

Wind Load Determination

The following section is the wind calculations for 230 North 13th Street using ASCE 7-05 chapter 6. Most of the calculations were determined using Microsoft Excel, therefore spreadsheets are provided. These spreadsheets can be found at the end of this section which also include base shear, along with diagrams which visually display the forces & pressures vs. building height.

NOTEBOOK SUB A | WIND LOADS | JESSE B | 8

WIND LOADS:

OCCUPANCY CATEGORY [TABLE 1-1]:

II

IMPORTANCE FACTOR [TABLE 7-4]

$I = 1.0$

BASIC WIND SPEED [FIGURE 6-1]

$V = 90$ MPH

WIND DIRECTIONALITY FACTOR (k_d) [TABLE 6-4]:

$K_d = 0.85$

EXPOSURE CATEGORY [SECTION 6.5.6]

CATEGORY B

TOPOGRAPHIC FACTOR (k_{zt}):

BUILDING W/ NO ESCARPMENT → $k_{zt} = 1.0$

GUST EFFECT FACTOR:

FUNDAMENTAL PERIOD (T_a) [SECTION 12.8-7]:

$T_a = C_t h_n^x$

[TABLE 12.8-2] → ESSENTRIC BRACED STEEL FRAMES (MAJORITY)

$$\left.\begin{array}{l} C_t = 0.03,\ x = 0.75 \\ h_n = \text{OVERALL BLDG HEIGHT} = 192\text{FT} \end{array}\right\} T_a = (0.03)(192)^{0.75} = 1.547\text{s}$$

LOWEST NATURAL FREQUENCY OF BUILDING (FUNDAMENTAL FREQ)

$n_1 = \frac{1}{T_a} = \frac{1}{1.547} = \underline{0.65\text{Hz}} < 1.0\text{Hz} \therefore$ FLEXIBLE STRUCTURE [SECTION 6.5.8.2]

[C-8] $G_f = \left(\frac{1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_z}\right)$

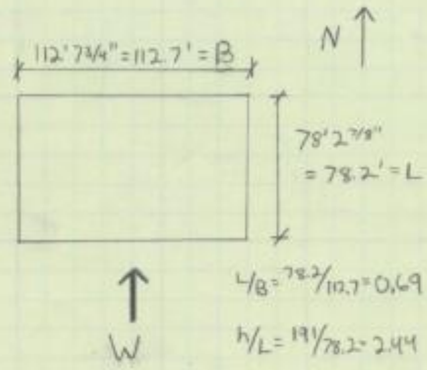
NOTEBOOK SUB A | WIND LOADS (N-S) | JESSE B | 9

NORTH-SOUTH DIRECTION

$\bar{z} = 0.6h = 0.6(192) = 115.2'$

$g_Q = g_v = 3.4$

$\bar{z} = \max \begin{cases} 0.6h = 115.2' \\ z_{min} = 30' \end{cases}$



[TABLE 6-2] → EXPOSURE B:

$\alpha = 7.0$
$z_g = 1200 ft$
$\hat{a} = 1/7$
$\hat{b} = 0.84$
$\bar{\alpha} = 1/4.0$
$\bar{b} = 0.45$
$c = 0.30$
$\ell = 320$
$\bar{\epsilon} = 1/3.0$
$z_{min} = 30 ft$
$h = 191'$

[6-5] $I_{\bar{z}} = 0.3\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{115.2}\right)^{1/6} = \underline{0.244}$

[6-7] $L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{115.2}{33}\right)^{1/3} = \underline{483.4}$

$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} \cdot V\left(\frac{88}{60}\right) = (0.45)\left(\frac{115.2}{33}\right)^{0.25} \cdot 90\left(\frac{88}{60}\right)$
$= (0.6151) \cdot (132) = 81.2$

[6-6] $Q = \sqrt{\dfrac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\dfrac{1}{1 + 0.63\left(\frac{112.7+191}{483.4}\right)^{0.63}}} = 0.825$

[6-9] $g_r = \sqrt{2\ln(3600 n_1)} + \dfrac{0.577}{\sqrt{2\ln(3600 n_1)}} = \sqrt{2\ln(3600(0.65))} + \dfrac{0.577}{\sqrt{2\ln(3600)(0.65)}}$
$= 3.94 + \frac{0.577}{3.94} = 4.09$

[6-12] $N_1 = \dfrac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \dfrac{(0.65)(483.4)}{81.2} = 3.87$

NOTEBOOK SUB A | WIND LOADS CONT (N-S) | JESSE B

[6.11] $R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(3.87)}{(1+10.3(3.87))^{5/3}} = 0.0596$

[6.13] $n(R_h) = \frac{4.6 n_1 h}{\bar{V}_z} = \frac{4.6(0.65)(191)}{81.2} = 7.03$

$n(R_B) = \frac{4.6 n_1 B}{\bar{V}_z} = \frac{4.6(0.65)(112.7)}{81.2} = 4.15$

$n(R_L) = \frac{15.4 n_1 L}{\bar{V}_z} = \frac{15.4(0.65)(78.2)}{81.2} = 9.20$

[6.13a] $R_h = \frac{1}{n} - \frac{1}{2n^2}(1-e^{-2n}) = \frac{1}{7.03} - \frac{1}{2(7.03)^2}\left(1-e^{-2(7.03)}\right) = 0.142-(0.0102)(0.999)$
$= 0.132$

$R_B = \frac{1}{4.15} - \frac{1}{2(4.15^2)}\left(1-e^{-2(4.15)}\right) = 0.241 - 0.029(0.9998) = 0.212$

$R_L = \frac{1}{9.2} - \frac{1}{2(9.2^2)}\left(1-e^{-2(9.2)}\right) = 0.109 - (0.006)(0.999) = 0.103$

$B = 0.02$

[6.10] $R = \sqrt{\frac{1}{B} R_n R_h R_B (0.53+0.47R_L)} = \sqrt{\frac{1}{0.02}(0.0596)(0.132)(0.212)(0.53+0.47(0.103))}$
$= \sqrt{0.0834(0.5784)} = 0.22$

$G_f = \left(\frac{1+1.7(0.244)\sqrt{(3.4^2)(0.825^2)+(4.09^2)(0.22^2)}}{1+1.7(3.4)(0.244)}\right) = \frac{2.22}{2.41} = 0.922$

ENCLOSURE CLASSIFICATION [SECTION 6.2]

ENCLOSED

INTERNAL PRESSURE COEFF. [FIG 6-5]

$GC_{pi} = \pm 0.18$ (ENCLOSED)

VELOCITY PRESSURES:

[6-15] $q_z = 0.00256 K_z K_{zt} K_d V^2 I$ [lb/ft^2]

[TABLE 6-4] $K_d = 0.85$

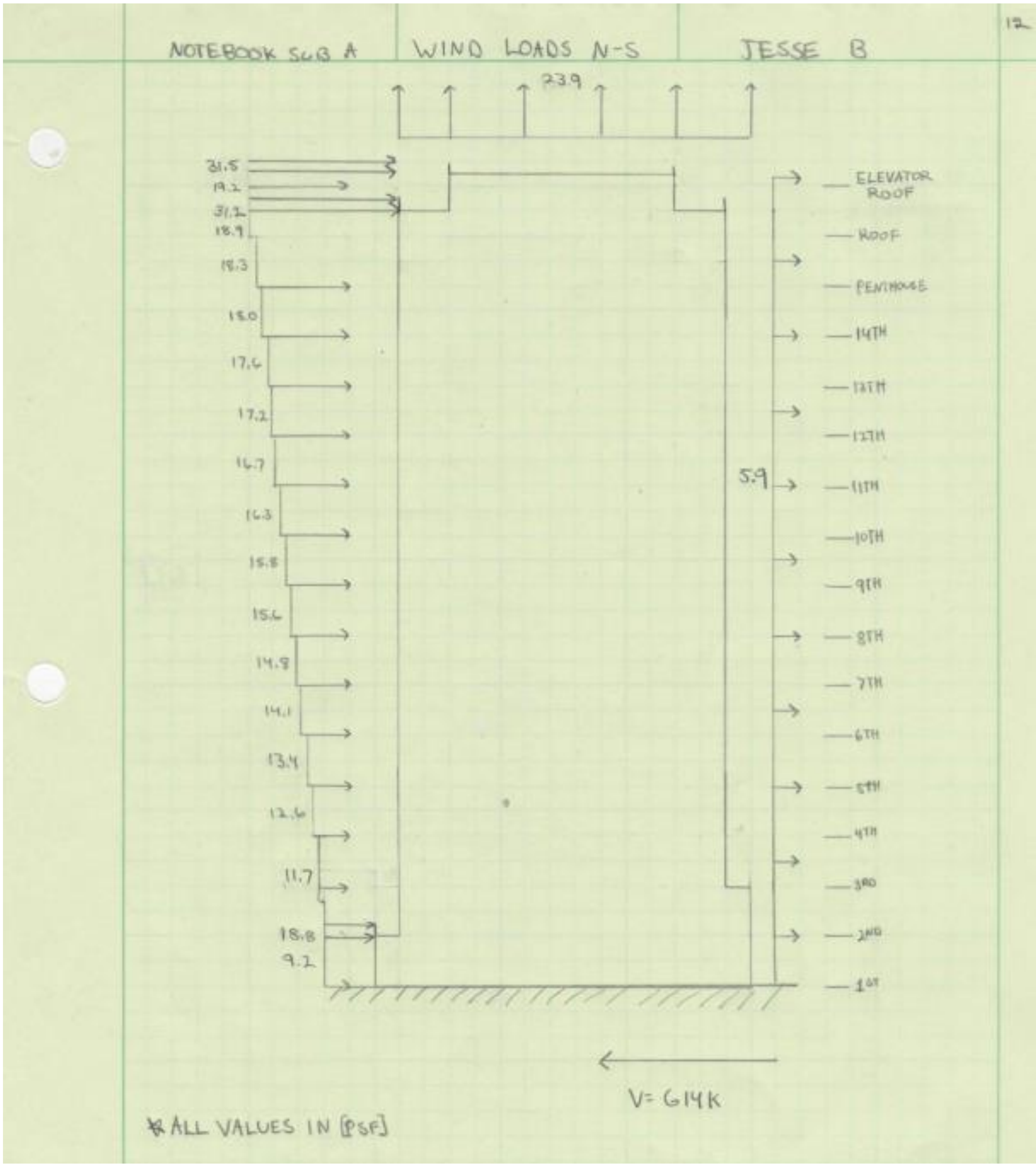
[TABLE 6-3] K_z → EXPOSURE B, CASE 2 → OK TO USE INTERPOLATION

DOESN'T MEET ANY CONDITIONS IN [6.5.7.1] ∴ $K_{zt} = 1.0$

PARAPETS → $p_p = q_p GC_{pn}$

ROOFS → ALL ROOFS >1000 SQFT ∴ USE 0.8 REDUCTION FACTOR FOR C_p

SEE SPREADSHEET FOR FULL RESULTS

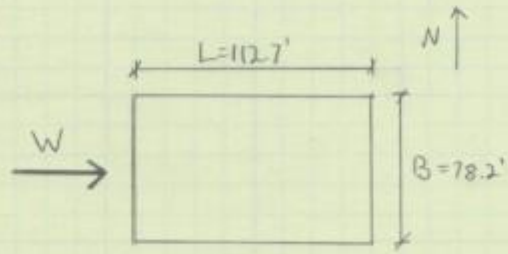

12
NOTEBOOK SUB A
WIND LOADS N-S
JESSE B
23.9
31.5
19.2
31.2
18.9
18.3
18.0
17.6
17.2
16.7
16.3
15.8
15.6
14.8
14.1
13.4
12.6
11.7
18.8
9.2
5.9
ELEVATOR ROOF
ROOF
PENTHOUSE
14TH
13TH
12TH
11TH
10TH
9TH
8TH
7TH
6TH
5TH
4TH
3RD
2ND
1ST
V= 614K
* ALL VALUES IN [PSF]

Wind Pressure Determination (N-S)									Net Pressures [psf]	
Location	Story	z [ft]	kz	qz [psf]	Cp	qzGCp [psf]	Gcpi	qhGCpi [psf]	qzGCp-qh(+Gcpi)	qzGCp-qh(-Gcpi)
Windward	1	0	0.57	10.05	0.8	7.41	0.18	1.81	5.60	9.2
	2	15.66	0.57	10.05	0.8	7.41	0.18	1.81	5.60	9.2
	3	33.75	0.72	12.74	0.8	9.40	0.18	2.29	7.11	11.7
	4	44.25	0.78	13.75	0.8	10.14	0.18	2.47	7.67	12.6
	5	54.75	0.83	14.63	0.8	10.79	0.18	2.63	8.16	13.4
	6	65.25	0.87	15.33	0.8	11.31	0.18	2.76	8.55	14.1
	7	75.75	0.91	16.09	0.8	11.87	0.18	2.90	8.97	14.8
	8	86.25	0.96	16.99	0.8	12.53	0.18	3.06	9.47	15.6
	9	96.75	0.98	17.27	0.8	12.74	0.18	3.11	9.63	15.8
	10	107.25	1.01	17.80	0.8	13.13	0.18	3.20	9.93	16.3
	11	117.75	1.03	18.21	0.8	13.43	0.18	3.28	10.15	16.7
	12	128.25	1.06	18.70	0.8	13.79	0.18	3.37	10.43	17.2
	13	138.75	1.09	19.21	0.8	14.17	0.18	3.46	10.71	17.6
	14	149.25	1.11	19.56	0.8	14.43	0.18	3.52	10.91	18.0
	Penthouse Deck	163.00	1.13	19.92	0.8	14.69	0.18	3.59	11.11	18.3
	Penthouse	163.25	1.13	19.92	0.8	14.69	0.18	3.59	11.11	18.3
	Roof	181.00	1.17	20.62	0.8	15.21	0.18	3.71	11.50	18.9
	Elevator Roof	191.02	1.19	20.97	0.8	15.47	0.18	3.78	11.70	19.2
Leeward	All	All	1.19	20.97	-0.5	-9.67	0.18	3.78	-13.44	-5.89
Side	All	All	1.19	20.92	-0.7	-13.50	0.18	3.77	-17.27	-9.74
Parapet (WW)	2nd Story	31.50	0.71	12.51			1.50	18.77		18.77
	Penthouse	167.75	1.15	20.27			1.50	30.40		30.40
	Roof	185.75	1.18	20.80			1.50	31.20		31.20
	Elevator Roof	192.00	1.19	20.97			1.50	31.46		31.46
Parapet (LW)	2nd Story	31.50	0.71	12.51			-1.00	-12.51		-12.51
	Penthouse	167.75	1.15	20.27			-1.00	-20.27		-20.27
	Roof	185.75	1.18	20.80			-1.00	-20.80		-20.80
	Elevator Roof	192.00	1.19	20.97			-1.00	-20.97		-20.97
Roof	(0-95.5ft)	191.02	1.19	20.97	-1.04	-20.11	0.18	3.78	-23.89	-16.34
	(>95.5ft)	191.02	1.19	20.97	-0.7	-10.13	0.18	3.78	-13.91	-6.36

V^2	8100
I	1
kd	0.85
kzt	1
G	0.922
L/B	0.69

Forces	B	pw+pl	H	Total Force
F1	113	22.66	7.83	19999.1
F2	113	22.66	16.88	43101.5
F3	113	25.14	14.3	40498.3
F4	113	26.06	10.5	30837.8
F5	113	26.87	10.5	31794.8
F6	113	27.52	10.5	32560.3
F7	113	28.21	10.5	33383.3
F8	113	29.04	10.5	34359.3
F9	113	29.29	10.5	34665.5
F10	113	29.78	10.5	35239.7
F11	113	30.15	10.5	35679.9
F12	113	30.60	10.5	36215.8
F13	113	31.07	10.5	36770.8
F14	113	31.40	12.13	42903.5
FPD	113	31.72	7	25024.2
FPD	113	31.72	9	32174.0
FR	113	32.37	13.89	50649.7
FER	113	32.69	5.01	18458.1
			[lb]	614315.7
			[kip]	**614.3**

EAST-WEST DIRECTION



$L/B = 112.7/78.2 = 1.44$

$h/L = 191/112.7 = 1.70$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{78.2+191}{4834}\right)^{.63}}} = 0.835$$

$$n(R_h) = 7.03$$

$$n(R_B) = \frac{4.6(0.65)(78.2)}{81.2} = 2.88$$

$$n(R_L) = \frac{15.4(0.65)(112.7)}{81.2} = 13.89$$

$$R_h = 0.132$$

$$R_B = \frac{1}{2.88} - \frac{1}{2(2.88^2)}\left(1-e^{-2(2.88)}\right) = 0.3472 - (.0602)(0.997) = 0.287$$

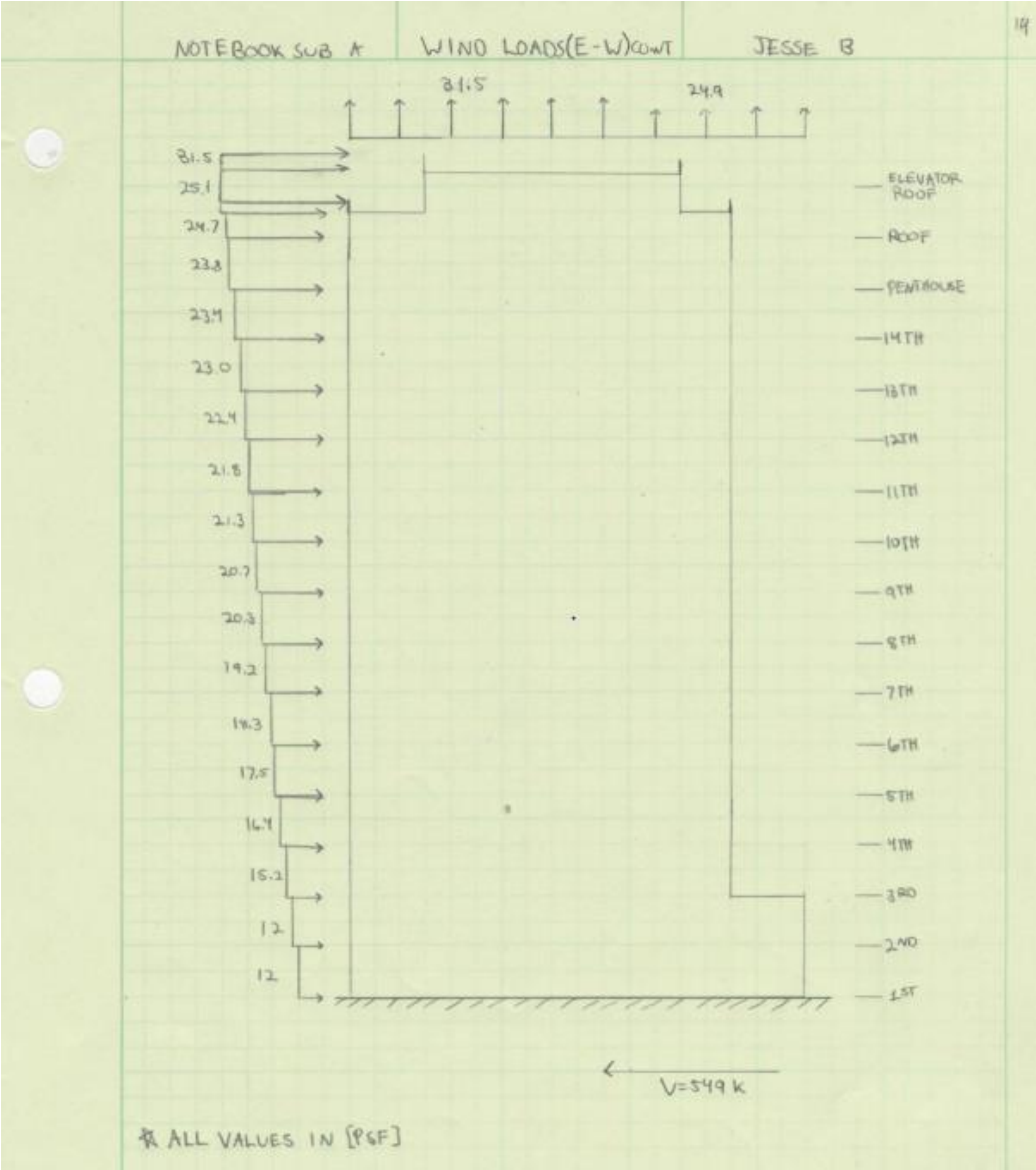
$$R_L = \frac{1}{13.89} - \frac{1}{2(13.89^2)}\left(1-e^{-2(13.89)}\right) = 0.072 - (0.00259)(0.999) = 0.069$$

$$R = \sqrt{\frac{1}{0.02}(0.0596)(0.132)(0.287)(0.53+0.47(0.069))} = 0.252$$

$$G_f = \left(\frac{1+1.7(0.244)\overbrace{\sqrt{(3.4^2)(0.835^2)+(4.09^2)(0.252^2)}}^{8.06}}{1+1.7(3.4)(0.244)}\right) = \frac{3.07}{2.41} = 1.27$$

$$G_{f_{N-S}} = 0.922$$

$$G_{f_{E-W}} = 1.27$$


NOTEBOOK SUB A
WIND LOADS(E-W)CONT
JESSE B
14
31.5
24.9
31.5
25.1
24.7
23.8
23.4
23.0
22.4
21.8
21.3
20.7
20.3
19.2
18.3
17.5
16.4
15.2
12
12
ELEVATOR ROOF
ROOF
PENTHOUSE
14TH
13TH
12TH
11TH
10TH
9TH
8TH
7TH
6TH
5TH
4TH
3RD
2ND
1ST
V=549 k
* ALL VALUES IN [PSF]

Wind Pressure Determination (E-W)									Net Pressures [psf]	
Location	Story	z [ft]	kz	qz [psf]	Cp	qzGCp [psf]	Gcpi	qhGCpi [psf]	qzGCp-qh(+Gcpi)	qzGCp-qh(-Gcpi)
Windward	1	0	0.57	10.05	0.8	10.21	0.18	1.81	8.40	12.0
	2	15.66	0.57	10.05	0.8	10.21	0.18	1.81	8.40	12.0
	3	33.75	0.72	12.74	0.8	12.95	0.18	2.29	10.65	15.2
	4	44.25	0.78	13.75	0.8	13.97	0.18	2.47	11.49	16.4
	5	54.75	0.83	14.63	0.8	14.86	0.18	2.63	12.23	17.5
	6	65.25	0.87	15.33	0.8	15.58	0.18	2.76	12.82	18.3
	7	75.75	0.91	16.09	0.8	16.35	0.18	2.90	13.45	19.2
	8	86.25	0.96	16.99	0.8	17.26	0.18	3.06	14.20	20.3
	9	96.75	0.98	17.27	0.8	17.55	0.18	3.11	14.44	20.7
	10	107.25	1.01	17.80	0.8	18.09	0.18	3.20	14.88	21.3
	11	117.75	1.03	18.21	0.8	18.50	0.18	3.28	15.22	21.8
	12	128.25	1.06	18.70	0.8	19.00	0.18	3.37	15.63	22.4
	13	138.75	1.09	19.21	0.8	19.52	0.18	3.46	16.06	23.0
	14	149.25	1.11	19.56	0.8	19.88	0.18	3.52	16.36	23.4
	Penthouse Deck	163.00	1.13	19.92	0.8	20.24	0.18	3.59	16.65	23.8
	Penthouse	163.25	1.13	19.92	0.8	20.24	0.18	3.59	16.65	23.8
	Roof	181.00	1.17	20.62	0.8	20.95	0.18	3.71	17.24	24.7
	Elevator Roof	191.02	1.19	20.97	0.8	21.31	0.18	3.78	17.53	25.1
Leeward	All	All	1.19	20.92	-0.5	-13.29	0.18	3.77	-17.05	-9.52
Side	All	All	1.19	20.92	-0.7	-18.60	0.18	3.77	-22.37	-14.83
Parapet (WW)	2nd Story	31.50	0.71	12.51			1.50	18.77		18.77
	Penthouse	167.75	1.15	20.27			1.50	30.40		30.40
	Roof	185.75	1.18	20.80			1.50	31.20		31.20
	Elevator Roof	192.00	1.19	20.97			1.50	31.46		31.46
Parapet (LW)	2nd Story	31.50	0.71	12.51			-1.00	-12.51		-12.51
	Penthouse	167.75	1.15	20.27			-1.00	-20.27		-20.27
	Roof	185.75	1.18	20.80			-1.00	-20.80		-20.80
	Elevator Roof	192.00	1.19	20.97			-1.00	-20.97		-20.97
Roof	(0-95.5ft)	191.02	1.19	20.97	-1.04	-27.70	0.18	3.78	-31.48	-23.93
	(>95.5ft)	191.02	1.19	20.97	-0.7	-21.14	0.18	3.78	-24.92	-17.37

V^2	8100
I	1
kd	0.85
kzt	1
G	1.27
L/B	1.44

Forces	B [ft]	pw+pl	H [ft]	Total Force
S1	78.2	29.07	7.83	17797.8
F2	78.2	29.07	16.875	38357.3
F3	78.2	32.29	14.295	36098.3
F4	78.2	33.49	10.5	27501.6
F5	78.2	34.55	10.5	28367.1
F6	78.2	35.39	10.5	29059.5
F7	78.2	36.30	10.5	29803.7
F8	78.2	37.37	10.5	30686.5
F9	78.2	37.71	10.5	30963.4
F10	78.2	38.34	10.5	31482.7
F11	78.2	38.83	10.5	31880.8
F12	78.2	39.42	10.5	32365.5
F13	78.2	40.03	10.5	32867.4
F14	78.2	40.45	12.125	38353.8
FPD	78.2	40.87	7	22373.2
FPD	78.2	40.87	9	28765.5
FR	78.2	41.71	13.885	45294.4
FER	78.2	42.14	5.01	16508.3
			[lb]	548527.0
			[kip]	**548.5**

Seismic Load Determination

Seismic loads are calculated in the following section using ASCE 7-05, chapters 11 &12.

9/28/2015 Design Maps Summary Report

USGS Design Maps Summary Report

User–Specified Input

Building Code Reference Document ASCE 7-05 Standard
(which utilizes USGS hazard data available in 2002)

Site Coordinates 39.95689°N, 75.16017°W

Site Soil Classification Site Class C – "Very Dense Soil and Soft Rock"

Occupancy Category I/II/III

USGS–Provided Output

S_S =	0.269 g	S_{MS} =	0.323 g	S_{DS} =	0.216 g
S_1 =	0.060 g	S_{M1} =	0.101 g	S_{D1} =	0.068 g

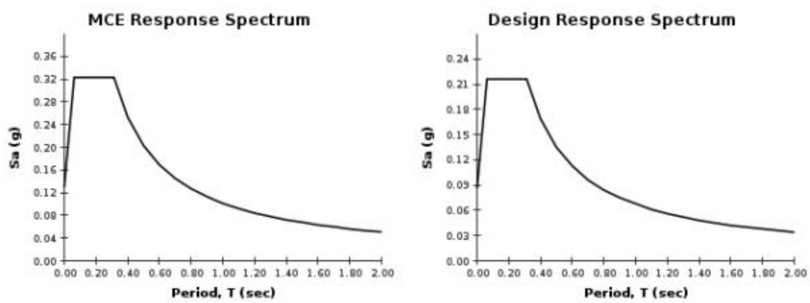


Figure 1: Seismic design criteria based on exact site location (Courtesy http://ehp2-earthquake.wr.usgs.gov)

SEISMIC LOADS

[SECTION 11.1.2] STRUCTURE NOT EXEMPT

[FROM DRAWINGS] SITE CLASS C

[SECTION 11.4.3] $S_s = 0.269g$ $S_{MS} = 0.323g$ $S_{DS} = 0.216g$
$S_1 = 0.060g$ $S_{M1} = 0.101g$ $S_{D1} = 0.068g$

[TABLE 12.6-1] SITE CLASS C, OCC. CAT II → ELF IS PERMITTED

[TABLE 12.2-1] ASSUME ESSENTRIC BRACE FRAMES TAKE MAJORITY OF LOAD ∴

ORDINARY STEEL CONCENTRICALLY BRACED FRAMES

$R = 3.25$
$\Omega = 2$
$C_d = 3.25$

[TABLE 11.5-1] $I_e = 1.0$

[EQN 12.8-7] $T_a = C_t h_n^x$ where: $C_t = 0.03$
$x = 0.75$
$h_n = 191'$

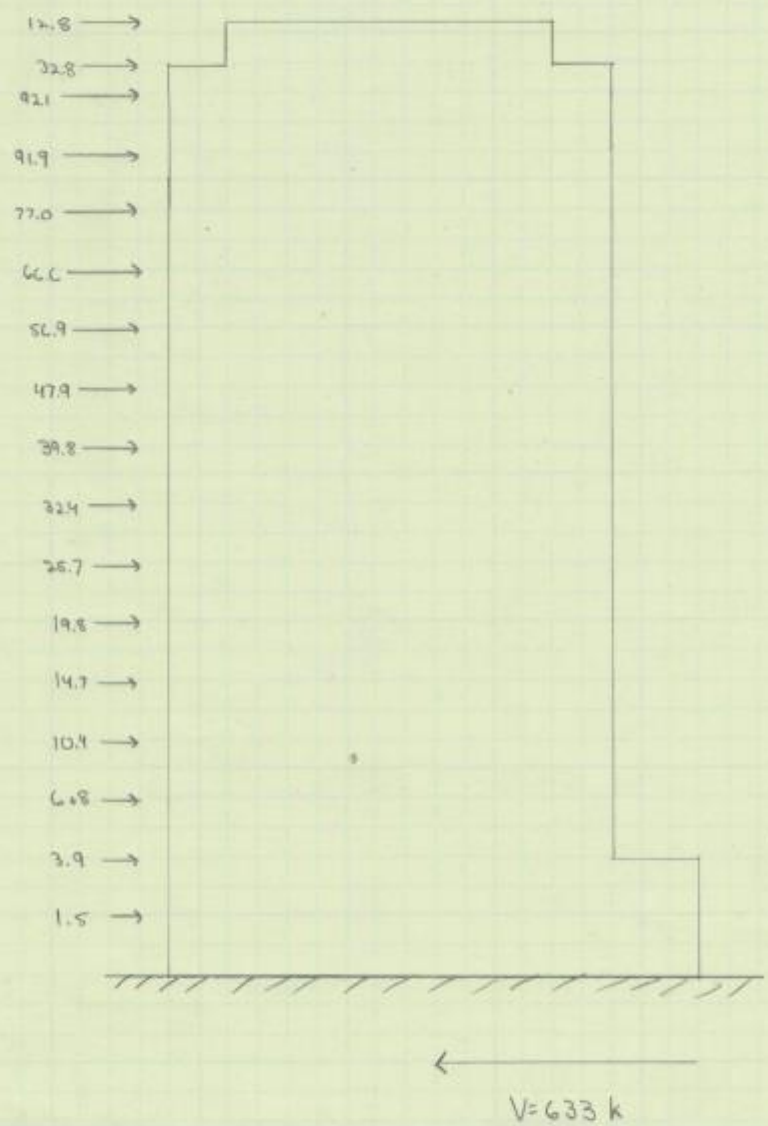
$T_a = (0.03)(191^{0.75}) = 1.54s$

[FIGURE 22-15] $T_L = 6s \rightarrow T_a < T_L \therefore C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.216}{1.54\left(\frac{3.25}{1}\right)} = 0.043 > 0.01$ √
[12.8-3]

[EQN 12.8-1] $V = C_s W = (0.043)(14728) = 633.0k$

BUILDING LOADS → SEE SPREADSHEET

NOTEBOOK SUB A | SEISMIC LOADS | JESSE B | 16



* ALL VALUES IN [k]

LOADS REMAIN THE SAME
IN BOTH DIRECTIONS

Story	Floor Area [sq.ft.)	Floor Load [psf]	Snow Load (20%) [psf]	Trib. Wall Height	Building Perimeter	Wall Load [psf]	Weight	Parapet	Mech/Misc	Total Floor Weight
1	8050	141		7.83	381.8	43	1263598			1263598
2	9770	141		16.88	381.8	43	1378296	1020	56160	1435476
3	5925	141		14.3	381.8	43	836039.9	1909		837949
4	5925	141		10.5	381.8	43	835876.5			835877
5	5925	141		10.5	381.8	43	835876.5			835877
6	5925	141		10.5	381.8	43	835876.5			835877
7	5925	141		10.5	381.8	43	835876.5			835877
8	5925	141		10.5	381.8	43	835876.5			835877
9	5925	141		10.5	381.8	43	835876.5			835877
10	5925	141		10.5	381.8	43	835876.5			835877
11	5925	141		10.5	381.8	43	835876.5			835877
12	5925	141		10.5	381.8	43	835876.5			835877
13	5925	141		10.5	381.8	43	835876.5			835877
14	5925	141		12.125	381.8	43	835946.4			835946
Penthouse Deck	5925	141		7	381.8	43	835726			835726
Penthouse	5925	141		9	381.8	43	835812			835812
Roof	1250	178	4.2	13.89	328	43	228347.3	11480	2000	241827
Elevator Roof	400	81	4.2	5.01	68	43	34295.43	680	50000	84975
*included 10 psf for weight of steel and extra allowances for pool, mech equip & fitness							Total Building Weight [kips]			**14730**

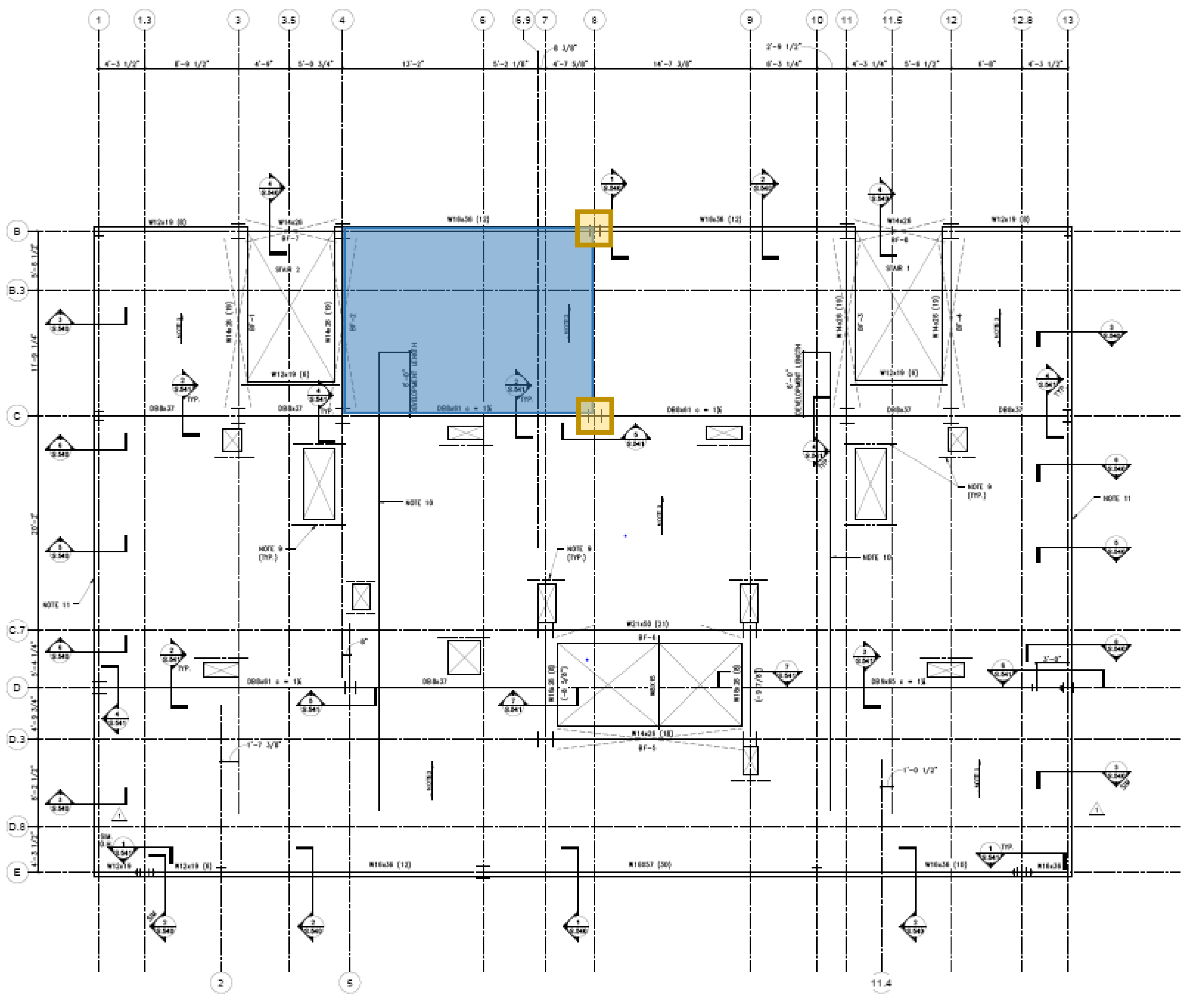
Story	hx [ft]	wx [kip]	wxhx^k	Cvx	Fx	Vx
Elevator Roof	191.0	85.0	3101534.4	0.02	12.8	12.8
Roof	181.0	241.8	7922494.3	0.05	32.8	45.6
Penthouse	163.3	835.8	22274859.9	0.15	92.1	137.8
Penthouse Deck	163.0	835.7	22204404.1	0.15	91.9	229.6
14	149.3	835.9	18621167.4	0.12	77.0	306.6
13	138.8	835.9	16091938.3	0.11	66.6	373.2
12	128.3	835.9	13748557.1	0.09	56.9	430.1
11	117.8	835.9	11589486.8	0.08	47.9	478.0
10	107.3	835.9	9614727.4	0.06	39.8	517.8
9	96.8	835.9	7824278.9	0.05	32.4	550.2
8	86.3	835.9	6218141.2	0.04	25.7	575.9
7	75.8	835.9	4796314.5	0.03	19.8	595.7
6	65.3	835.9	3558798.6	0.02	14.7	610.5
5	54.8	835.9	2505593.5	0.02	10.4	620.8
4	44.3	835.9	1636699.4	0.01	6.8	627.6
3	33.8	835.9	952116.1	0.01	3.9	631.5
2	15.7	1435.5	352029.8	0.00	1.5	633.0
1	0.0	1263.6	0.0	0.00	0.0	633.0
	Σ	14728.0	153013142.0	1.0		

*k=2 b/c period is >0.5s

V= 633.0

Typical Bay

Bays sizes vary within AC Hotel Philadelphia, therefore, an average size bay was selected for consideration. Due to the fact that the chosen bay is guest rooms, loads are based off private occupancy.



Typical Bay
(4-8, B-C)

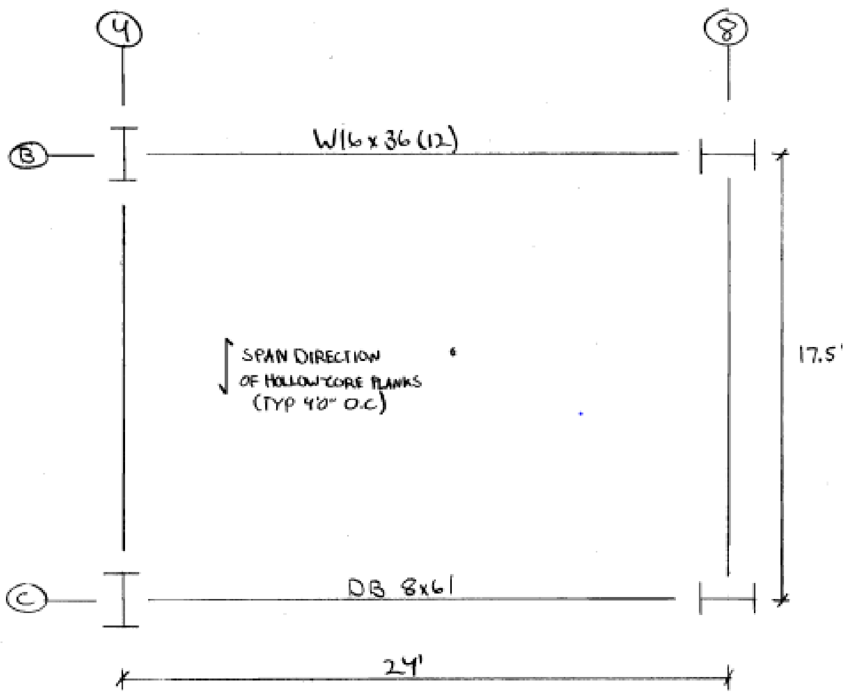
Columns under consideration
(B8 & C8)

Member Spot Check

NOTEBOOK SUBMISSION B | TYP. FLOOR BAY FRAMING | BAY BETWEEN B&C, 4 & 8 | 17

BAY SIZES VARY ∴ AN INTERMEDIATE SIZE BAY WAS CHOSEN

TRIB WIDTH WAS DETERMINED USING TYP. BAY SIZE = $\frac{17.5+17.5}{2}$ = 17.5'



DEAD LOADS

8" PRECAST PLANK w/ 2" TOPPING	=	100 PSF
SOUND ATTENUATION BATT INSULATION	=	3 PSF
MECH. ALLOWANCE	=	4 PSF
SUPERIMPOSED DL	=	3 PSF
FINISHES	=	2 PSF
		112 PSF

Live Loads

PRIVATE ROOMS & CORRIDORS SERVING THEM	=	40 PSF
PUBLIC ROOMS & CORRIDORS SERVING THEM	=	100 PSF

SELECT LL = 40 PSF B/c BAY BEING ANALYZED IS PRIVATE (GUEST ROOM)

CALCS DETERMINED USING THE GIRDER-SLAB SYSTEM LRFD DESIGN GUIDE v3.1

INPUT:

D-BEAM SPAN=24'

TRIB WIDTH=17.5'

SLAB THICKNESS=8" [S.104]

PRECAST SLAB WT = 58 PSF [PG 9 OF DG]

GROUT WT = 140 PCF

ADD'L COMPOSITE DL (2" CONCRETE TOPPING) = $2/12(150)=25$ PSF

PARTITION LL = 10 PSF

FLOOR LL = 40 PSF

USE REDUCED LL? → YES

CAMBER (OPTIONAL) → 1.25" B/C ON PLAN DB 8x61 HAS CAMBER≈1.25"

RESULTS: ASSUME COMPOSITE SYSTEM

TRY: DB 8x45, DB 8x57 & DB 8x61 (USED ON PLANS)

DB 8x45 → $\phi M_n = 216.6 > M_u = 212.9$ kFT ✓

$\phi V_n = 58.2k > V_u = 35.5k$ ✓

$\Delta_{LL} = 0.41" < L/360 = 0.8"$ ✓

DB 8x57 → $\phi M_N = 297.2$ kFT $> M_u = 214.4$ k·FT ✓

$\phi V_N = 72.6k > V_u = 35.7k$ ✓

$\Delta_{LL} = 0.34" < L/360 = 0.8"$ ✓

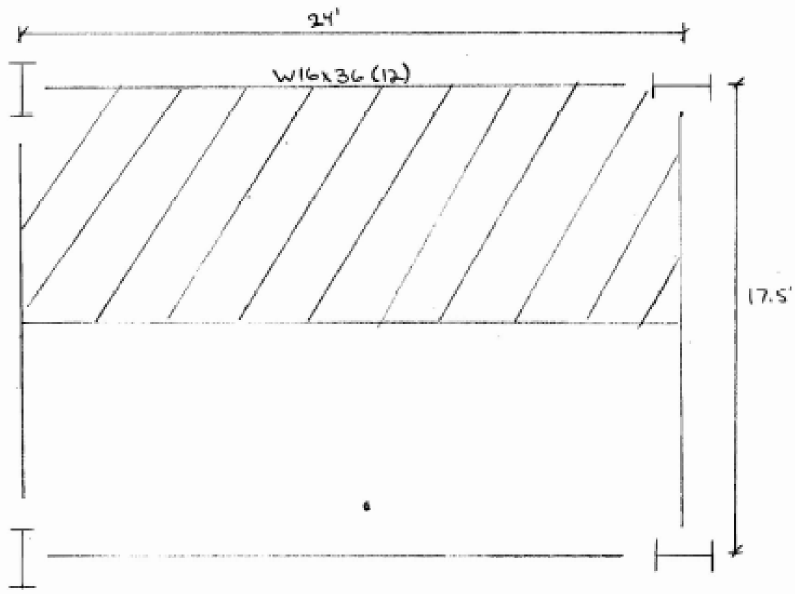
DB 8x61 → $\phi M_N = 298.4$ kFT $> M_u = 214.6$ k·FT ✓

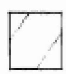
$\phi V_N = 75.9k > V_u = 35.8k$ ✓

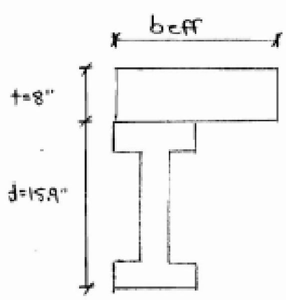
$\Delta_{LL} = 0.34" < L/360 = 0.8"$ ✓

SMALLER MEMBERS ARE SUFFICIENT, HOWEVER DB 8x61 WAS PROBABLY CHOSEN FOR ITS EXTRA CAPACITY

NOTEBOOK SUBMISSION B TYP. MEMBER SPOT CHECK WIDE-FLANGE 19



 - TRIB AREA OF W16x36 = $\frac{24 \times 17.5}{2} = 210\ FT^2$



$b_{EFF} = \min \begin{cases} SPAN/8 = \frac{24 \times 12}{8} = 36" \leftarrow \text{CONTROLS} \\ SPACING/2 = \frac{12.5 \times 12}{2} = 105" \end{cases}$

[S.503] SHEAR STUDS → 3/4"ø, 5"

[S.001] f'_c = 4500 PSI FOR FRAMED S.O.G.

[S.104] NOTE 4: COMPOSITE STEEL → UNSHORED

[TABLE 3-21] NO DECK, 3/4" ø STUD, NWC, f'_c = 4 ksi [4.5 ksi NOT IN MANUAL] → $Q_N = 21.5k$

[TABLE 1-1] FOR W16x36: $d = 15.9"$, $A_s = 10.6\ in^2$, $b_f = 7.0"$

NOTEBOOK SUBMISSION B | TYP. MEMBER SPOT CHECK CONT. | 20

DETERMINE MAX MOMENT:

$DL = 112\ PSF$

FLOOR-FLOOR

$DL_{WALL} = 27.5\ PSF \times 10.5\ FT = 289\ PLF$

$LL = 100\ PSF \rightarrow A_{TYP.BAY} = 24 \times 17.5 = 420\ FT^2$ ∴ REDUCTION ALLOWED

$L_i = \begin{cases} 100(0.5) = 50\ PSF \\ \text{MAX } 100\left(0.25 + \frac{15}{\sqrt{420}}\right) = 98\ PSF \end{cases}$ → USE 100 PSF B/C NEGLIGIBLE DIFFERENCE

$w_u = 1.2DL + 1.6LL$
$= 1.2(112) + 1.6(100)$
$= 294\ PSF(17.5/2)'$
$= 2576\ PLF + 1.2(289)$
$= 2923\ PLF = 2.92\ KLF$

$M_u = \frac{w_u l^2}{8} = \frac{(2.92)(24^2)}{8} = 210\ k\cdot FT$

DETERMINE MOMENT CAPACITY:

$\Sigma Q_N = 12$ STUDS → 6/SIDE → $6(21.5) = \underline{129k}$ CONTROLS

$T_s = A_s F_y = (10.6)(50) = 530k$

$C_c = 0.85 F'c\, b_{eff}\, t = 0.85(4.5)(36)(8) = 1102k$

$\Sigma Q_N < T_s$ & C_c ∴ PARTIALLY COMPOSITE

$a = \frac{\Sigma Q_N}{0.85 F'c\, b_{eff}} = \frac{129}{0.85(4.5)(36)} = 0.94''$

$Y_2 = t - \frac{a}{2} = 8 - \frac{0.94}{2} = 7.53$ ∴ PNA IN CONCRETE

$A_s F_y - \Sigma Q_N = 2 F_y b_f x$

$y_1 = x = \frac{A_s F_y - \Sigma Q_N}{2 F_y b_f} = \frac{530 - 129}{2(50)(7)} = 0.57''$

$M_N = T_s(d/2) + \Sigma Q_N(t - a/2) - 2F_y \cdot b_f \cdot x(x/2)$
$= 530(15.9/2) + 129(7.53) - 2(50)(7)(0.57)(0.57/2)$
$= 423\ k\cdot FT$

$\phi M_N = 0.9(423) = 381\ k\cdot FT > 210\ k\cdot FT$ ∴ W16x36 OK SO FAR → CHECK Δ

NOTEBOOK SUBMISSION B | TYP. MEMBER SPOT CHECK CONT. 21

CHECK Δ's:

BASED ON DG3, LIMIT Δ_{LL} USING 50% OF UNREDUCED LL TO L/360 W/ MAX OF 1"

$$\Delta_{LL,50\%} = \frac{5 w_{LL} L^4}{384 E I_x} = \frac{5(1 \cdot \frac{100 \times 0.5}{100})(24^4) \times 1728}{384(29000)(448)} = 0.29'' < \frac{L}{360} = 0.8'' < \overset{MAX}{1''} \checkmark$$

[TABLE 1-1] W16x36 → $I_x = 448 in^4$

[TABLE 3-20] W16x36, $Y_2 = 7 \rightarrow I_{LB}' = 1630 in^4$

$$\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I_{LB}} = \frac{5(1.0)(24^4) \times 1728}{384(29000)(1630)} = 0.16'' < 0.8'' \checkmark$$

$$\Delta_{DL,WC} = \frac{5 w_{DL,WC} L^4}{384 E I_x} = \frac{5(1.05)(24^4)1728}{384(29000)(448)} = 0.6'' < 0.8'' \checkmark$$

∴ NO CAMBER NEEDED

$$w_{DL,WC} = \underset{\text{8" PRECAST PLANK}}{100} + \underset{\text{BEAM ALLOWANCE}}{5} = 105 PSF = 1.05 KSF$$

IN SUMMARY, TYP. DB 8x61 & W16x36 IS ADEQUATE FOR STRENGTH & SERVICABILITY

Column Load Spot Check

INTERIOR COLUMN C8 & EXTERIOR COLUMN B8

COLUMN SIZES:

FLOOR	B8	C8
15 (PENTHOUSE)	W12x40	W14x43
12-14	W12x40	W14x61
9-11	W12x50	W14x61
6-8	W12x58	W14x74
3-5	W12x72	W14x90

VERIFY LOADS HERE → (3-5)

INTERIOR COLUMN

FOR COLUMN C8: TRIB AREA/FLOOR = 24'x18.75' = 450 FT²

[TABLE 4-2] ASCE 7-05: $K_{LL}=4$ FOR INTERIOR & EXTERIOR COLUMNS

[S.001] TYP. FLOOR → REDUCIBLE
ROOF → NOT REDUCIBLE

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 50\left(0.25 + \frac{15}{\sqrt{(4)(450)}}\right) = 30.2\text{ PSF}$$

15TH FLOOR

DL = 148 PSF (450) = 66600# + 2000# (COOLING TOWER) = 68600#
LL_{ROOF} = 30 PSF (450) = 13500#
SL = 18 PSF (450) = 8100#

3RD - 14TH FLOOR

DL = 98 PSF (450) = 44100#
LL = 30.2 PSF (450) = 13590#

DL = 68600 + 12(44100) + (16.75')(43 PLF) + (63')(61 PLF) + (31.5')(74 PLF) + (31.5')(90)
= 606k

LL = 1(13500) + 12(13590) = 177k

P_u = 1.2 DL + 1.6 LL + 1.0 SL = 1.2(606) + 1.6(177) + 1.0(8.1) = 1019 k

[TABLE 4-1] W14x90 → KL = 10.5' → ϕP_n = 1095 k > 1019 k ∴ W14x90 SUFFICIENT

NOTEBOOK SUB B | COLUMN LOAD SPOTCHECK | EXTERIOR COLUMN 23

EXTERIOR COLUMN

FOR COLUMN B8: TRIB AREA/FLOOR = $\left(\frac{17.5}{2}\right)(24) = 210\ FT^2 < 400\ FT^2$ ∴ CANT REDUCE

EXTERIOR WALL LOAD = 289 PLF (24') = 6940#/FLOOR

15TH FLOOR

$DL = 98\ PSF(210) = 20580 + 6940 = 27520\#$
$LL_{ROOF} = 30\ PSF(210) = 6300\#$
$SL = 18\ PSF(210) = 3780\#$

3RD-14TH FLOOR

$DL = 98(210) + 6940 = 27520\#$
$LL = 50(210) = 10500\#$

$DL = 13(27520) + (16.75)(40\ PLF) + (31.5)(40\ PLF) + (31.5)(50\ PLF) + (31.5)(58\ PLF) + (31.5)(72\ PLF)$
$= 366\ k$

$LL = 1(6300) + 12(10500) = 133\ k$

$P_u = 1.2(366) + 1.6(133) + 1.0(3.78) = 656\ k$

[TABLE 4-1] W12x72 → KL = 10.5' → $\phi P_n = 834\ k > 656\ k$ ∴ W12x72 SUFFICIENT

AFTER ANALYSIS, BOTH THE INTERIOR & EXTERIOR COLUMNS ARE SUFFICIENT FOR GRAVITY LOADS

Alternative Systems

During my analysis, three framing systems were examined:

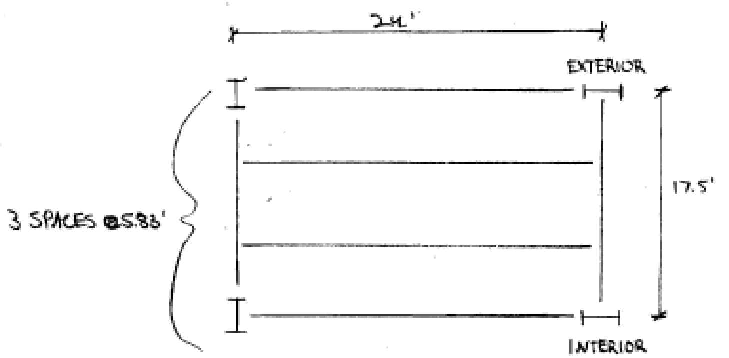
1. **Non-Composite Steel Framing**

2. **Composite Steel Framing**

3. **Hambro D-500 Composite**

Alternative System 1: Non-Composite Steel Framing

NOTEBOOK SUBMISSION B ALTERNATIVE SYSTEM #1 NON-COMPOSITE STEEL FRAMING 24



BEAMS SPAN LONGER DIMENSION

SPAN = 24'
SPACING = 5.83'

ASSUME LL = 40 PSF (BAY ANALYZED IS PRIVATE SPACE) + 10 (PARTITIONS) = 50 PSF

FROM VULCRAFT CATALOG:

TRY 1.5C20, 2.5" NWC TOPPING → 4" TOTAL

- 44 PSF
- MAX SPAN = 8'8" FOR 3 SPAN > 5.83' √

[AS PER S.001]

$$DL = DECK + DL_{SUPERIMPOSED} + SW = 44 + 10 + 5 = 59\ PSF$$

INTERIOR: $W_u = 1.2(59) + 1.6(50) = 151\ PSF(5.83') = 879\ PSF = 0.88\ KLF$

$$M_u = \frac{0.88(24^2)}{8} = 63.4\ KFT$$

[TABLE 3-2] TRY W12x16 (ECONOMICAL) → $\phi M_n = 75.4\ FTK > 63.4\ FTK$ √

$I_x = 103\ in^4$

CHECK Δ's:

LIMIT LIVE LOAD TO L/360 = 0.8"

$$\Delta_{LL} = \frac{5 w_{LL} L^4 (1728)}{384 E I_x} = \frac{5(0.292)(24^4)1728}{384(29000)(103)} = 0.73" < 0.8" \therefore \text{ADEQUATE FOR } \Delta$$

$$w_{LL} = \frac{50(5.83)}{1000} = 0.292\ KLF$$

25

EXTERIOR: $DL_{WALL} = 289\ PLF$

$W_U = 1.2(59) + 1.6(50) = 151\ PSF\left(\frac{5.83}{2}\right) = 441\ PLF + 1.2(289\ PLF) = 0.79\ KLF$

$\underset{(EXTERIOR)}{0.79\ KLF} < \underset{(INTERIOR)}{0.88\ KLF}$ ∴ W12x16 IS SUFFICIENT ✓

CHECK MEMBER SELF-WEIGHT:

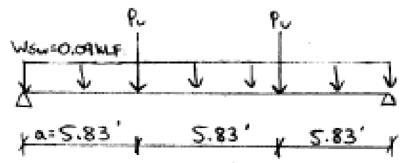
W12x16 → $\frac{16\,PLF}{5.83\,FT}$ = 2.8 < 5 PSF ALLOWANCE ∴ USE W12x16 @ 5.83' O.C.

SELECT GIRDER:

½ LOAD FROM ADJ BAYS

SPAN = 17.5'
SPACING = 24'

POINT LOAD ON GIRDER(P_u) = $\frac{1}{2}$(0.88 KLF)(24')(2) = 21 k



ASSUME GIRDER SW = 5 PSF(17.5')
= 87.5 PLF
≈ 0.09 KLF

[TABLE 3-23] CASE 9: 2 EQUAL CONCENTRATED LOADS SYMMETRICALLY PLACED]

$$M_{MAX} = (P_u \cdot a) + \frac{w\ell^2}{8}$$
$$= (21)(5.83) + \frac{0.09(17.5^2)}{8}$$
$$= 126\ K\text{-}FT$$

TRY W12x26: I_x = 204 in^4, ϕM_n = 140 K-FT > 126 k-FT ✓

CHECK Δ:

$$\Delta_{LL} = \frac{P_{LL} L^3}{28EI} = \frac{[(50)(5.83)(24)]/1000\ (17.5^3)(1728)}{28(29000)(204)} = 0.39''$$

$$\frac{L}{360} = \frac{17.5(12)}{360} = 0.583'' > 0.39''$$ ∴ GIRDER IS ADEQUATE

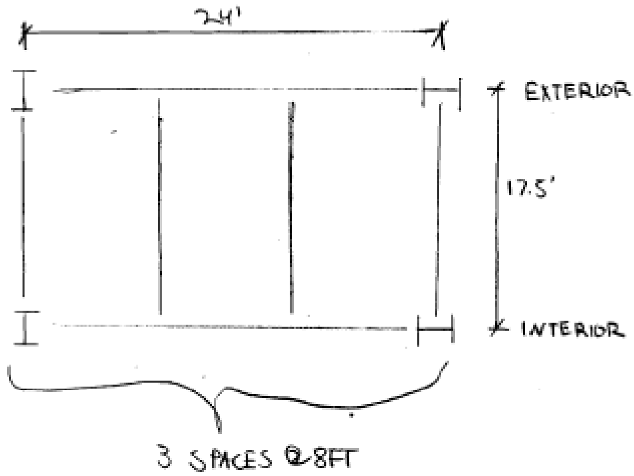
MEMBER SW: 26/17.5 = 1.49 < 5 ∴ GIRDER SW IS CONSERVATIVE

USE W12x16 BEAMS @ 5.83' O.C. & W12x26 GIRDER

* SAME DEPTHS ARE USED FOR CONNECTION PURPOSES

NOW I WILL LOOK INTO SWITCHING THE ORIENTATION OF THE BEAMS & GIRDERS IN ORDER TO DETERMINE THE OPTION THAT MINIMIZES STEEL

NOTEBOOK SUB B ALT SYSTEM #1 NON-COMPOSITE FRAMING 2-7



SPAN = 17.5' BEAMS SPAN SHORTER DIRECTION
SPACING = 8'

LL = 40PSF + 10 = 50PSF

FROM VULCRAFT CATALOG

USE SAME DECKING 1.5C20, 2.5 NWC TOPPING

$w_u = 151\text{ PSF}(8') = 1208\text{ PLF} = 1.21\text{ KLF} \rightarrow w_{LL} = \frac{(50)(8)}{1000} = 0.40$

$M_u = \frac{(1.21)(17.5^2)}{8} = 46\text{ KFT}$

[TABLE 3-2] TRY W10x12 → $\phi M_n = 97.5\text{ KFT} > 46$ √
(ECONOMY)
$I_x = 53.8\text{ in}^4$ $L/360 = \frac{17.5(12)}{360} = 0.583''$

CHECK Δ's:

$\Delta_{LL} = \frac{5 w L^4 (1728)}{384 E I_x} = \frac{5(0.40)(17.5^4)(1728)}{384(29000)(53.8)} = 0.54'' < 0.58''$ ∴ ok √

CHECK MEMBER SW:

W 10x12 → $\frac{12}{8} = 1.5 < 5\text{PSF}$ ALLOWANCE ∴ USE W10x12 @ 8' O.C.

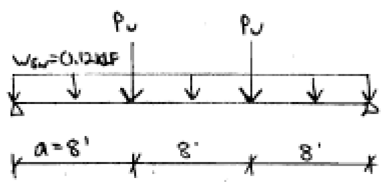
NOTEBOOK SUB B | ALT. SYSTEM 1 | Non-Composite 28

SELECT GIRDER:

SPAN = 24'
SPACING = 17.5'

POINT LOAD ON GIRDER = $\frac{1}{2}(1.21)(17.5')(2) = 21k$

ASSUME GIRDER SW = 5PSF(24) = 120PLF = 0.12KLF



[TABLE 3-23] CASE 9:

$$M_{max} = (P_u \cdot a) + \frac{wL^2}{8}$$

$$= 21(8) + \frac{(0.12)(24^2)}{8}$$

$$= 176 \text{ k-FT}$$

TRY W14x30: $I_x = 291 \text{ in}^4$, $\phi M_n = 177 \text{ k-FT} > 176 \text{ k-FT}$

$$\Delta_{LL} = \frac{[(50)(8)(17.5)]/1000 \ (24.^3)(1728)}{28(29000)(291)} = 0.71''$$

$$\frac{L}{360} = \frac{(24)(12)}{360} = 0.8'' > 0.71''$$

CHECK SW: $\frac{30}{24} = 1.25 < 5PSF$ ALLOWANCE ∴ OK

USE W10x12 BEAMS @ 8' O.C. & W14x30 GIRDERS

CHECK EXTERIOR GIRDER FOR ADD'L WALL LOAD

$w = 0.12\text{KLF} + \frac{1.2(291)}{1000} = 0.47\text{KLF}$

$P_u = \frac{1}{2}(1.08)(17.5) = 9.45k$

$M_{MAX} = 9.45(8) + \frac{(0.47)(24^2)}{8} = 110\text{KFT} < 161 \text{ k-FT}$ ∴ W14x30 IS ADEQUATE

NOTEBOOK SUB B ALT SYSTEM 1 NONCOMPOSITE 29

DETERMINE OPTIMAL BEAM CONFIGURATION:

WEIGHT OF STEEL

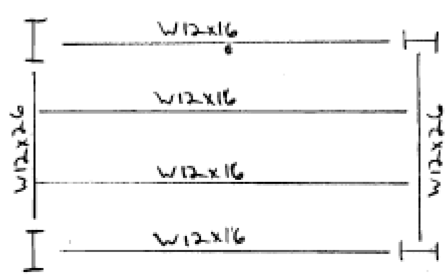
BEAMS LONG DIRECTION:

(4 BEAMS)(16#/FT)(24') + (2 GIRDERS)(26#/FT)(17.5') = 2446#

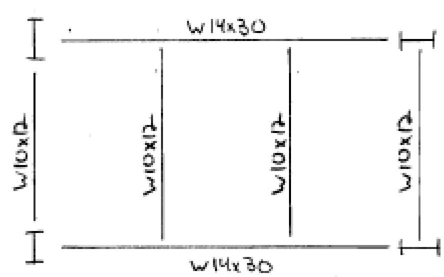
BEAMS SHORT DIRECTION:

(4 BEAMS)(12#/FT)(17.5') + (2 GIRDERS)(30#/FT)(24') = 2280#

OPTION 1:

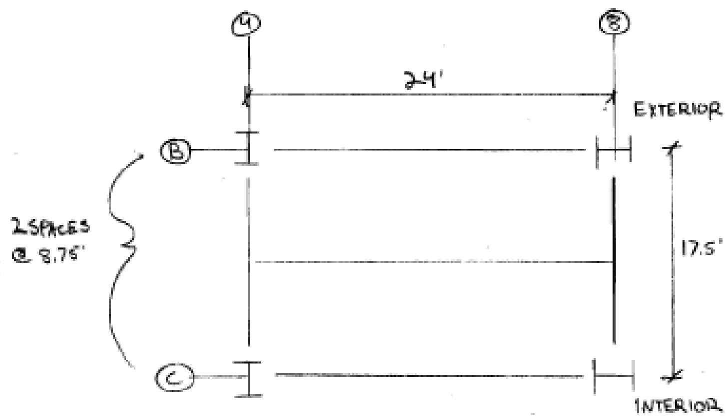


OPTION 2:



ALTHOUGH OPTION 1 IS SLIGHTLY HEAVIER IN OVERALL STEEL, BEAM & GIRDER DEPTH IS EQUAL, SIMPLIFYING CONNECTIONS ∴ CHOOSE OPTION 1.

Alternative System 2: Composite Steel Framing



BEAM: SPAN = 24'
SPACING = 8.75'

FROM VULCRAFT CATALOG:

TRY 2VLI 20, 2" NWC TOPPING → 4" TOTAL

- 39 PSF
- MAX UNSHORED CLEARSPAN (2 SPAN) = 10'10" > 8.75'

LL = 40PSF + 10 = 50
DL = DECK + $DL_{superimposed}$ + SW + FINISHES
= 39 + 10 + 5 + 2
= 56 PSF

INTERIOR $w_u = 1.2DL + 1.6LL = 1.2(56) + 1.6(50) = 147\text{PSF}(8.75') = 1.29\text{ KLF}$

$$M_u = \frac{wL^2}{8} = \frac{(1.29)(24^2)}{8} = 93\text{ k-FT}$$

ASSUMING: $a = 1" \rightarrow Y_2 = t_{total} - \frac{a}{2} = 4 - \frac{1}{2} = 3.5$

[TABLE 3-21] DECK ⊥, NWC, $f'_c = 4\text{ksi}$, 3/4" Ø STUD → 1 STUD/RIB = 17.2k
2 STUD/RIB = 14.6k

DETERMINE POSSIBLE BEAM SIZES: [TABLE 3-19]

W10x12 → $\Sigma Q_N = 115$ → $115/17.2 = 6.68$ → $7 \times 2 = 14$ STUDS/BEAM

W10x15 → $\Sigma Q_N = 83.8$ → $83.8/17.2 = 4.87$ → $5 \times 2 = 10$ STUDS/BEAM

W10x17 → $\Sigma Q_N = 62.4$ → $62.4/17.2 = 3.63$ → $4 \times 2 = 8$ STUDS/BEAM

W10x19 → $\Sigma Q_N = 70.3$ → $70.3/17.2 = 4.09$ → $5 \times 2 = 10$ STUDS/BEAM

W10x22 → $\Sigma Q_N = 81.1$ → $81.1/17.2 = 4.72$ → $5 \times 2 = 10$ STUDS/BEAM

CHECK ECONOMY

W10x12 → 12(24) + 14(10) = 428# ∴ CONTINUE w/ W10x12

W10x15 → 15(24) + 10(10) = 460#

W10x17 → 17(24) + 8(10) = 488#

W10x19 → 19(24) + 10(10) = 556#

W10x22 → 22(24) + 10(10) = 628#

CHECK ASSUMPTION:

$$b_{eff} = \min \begin{cases} \text{SPAN}/8 = 24(12)/8 = 36 \\ \text{SPACING}/2 = 8.75(12)/2 = 52.5 \end{cases} \quad \text{DO NOT} \times 2 \text{ B/C EXTERIOR BAY}$$

$$a = \frac{\Sigma Q_N}{0.85 f'_c b_{eff}} = \frac{115}{0.85(4.5)(36)} = 0.84'' < 1'' \quad \therefore Y_2 = 3.5'' \text{ IS CONSERVATIVE}$$

CHECK UNSHORED LENGTH:

SELECT CONTROLLING LOAD CASE

$$\left.\begin{aligned} 1.4DL &= 1.4(\underbrace{(56)(8.75)}_{490}) + 1.4(12) = 703 \text{ PLF} \\ 1.2DL + 1.6LL &= 1.2(490 + 12) + 1.6(50) = 683 \text{ PLF} \end{aligned}\right\} \therefore 1.4DL \text{ CONTROLS}$$

$$M_u = \frac{(0.703)(24^2)}{8} = 51 \text{ K·FT} \rightarrow \text{[TABLE 3-2] W10x12: } \phi M_n = 46.9 < 51 \text{ K·FT}$$

∴ SIZE UP

NOTEBOOK SUB B | ALT. SYSTEM 2 | COMPOSITE | 32

SELECT W10x15 → ϕM_n = 60 K·FT > 51 K·FT ✓ ∴ CONTINUE W/ W10x15

FOR W10x15 → $I_x = 68.9 in^4$

CHECK WET CONCRETE Δ:

$W_{wc} = (56)(8.75) + 15 = 505$ PLF = 0.51 KLF

$\Delta_{wc} = \frac{5(0.51)(24^4)(1728)}{384(29000)(68.9)} = 1.9''$

$MAX\ \Delta_{wc} = \frac{L}{240} = \frac{(24 \times 12)}{240} = 1.2''$

∴ NEED TO UPSIZE B/C BEAM DOES NOT PASS DEFLECTION CHECK

SELECT MEMBER W/ HIGHER I_x → W10x22 (ECONOMY) → ϕM_n = 97.5 K·FT & $I_x = 118 in^4$, $\Sigma Q_n = 81.1$

$\Sigma Q_n = 81.1 \rightarrow 81.1/17.6 = 4.6 \rightarrow 5 \times 2 = 10$ STUDS/BEAM

$a = \frac{81.1}{(0.85)(4.5)(36)} = 0.59 < 1''$ ∴ OK

CHECK UNSHORED LENGTH:

$1.4DL = 1.4(56)(8.75) + 1.4(22) = \underline{717\ PLF}$

$1.2DL + 1.6LL = 1.2(490 + 22) + 1.6(50) = 695$ PLF

$M_u = \frac{(0.717)(24^2)}{8} = 52$ K·FT

ϕM_n FOR W10x22 = 97.5 > 52 ✓

CHECK WET CONCRETE Δ:

$W_{wc} = 56(8.75) + 22 = 512$ PLF

$\Delta_{wc} = \frac{5(0.512)(24^4)(1728)}{384(29000)(118)} = 1.12'' < L/240 = 1.2''$ ∴ USE W10x22

NOTEBOOK SUB B ALT. SYSTEM 2 COMPOSITE 33

DETERMINE CAMBER:

SINCE $\Delta_{WC} < L/240$ ∴ NO CAMBER NEEDED

CHECK EXTERIOR BEAM B/C ADDL EXTERIOR WALL WEIGHT

$LL = 40PSF + 10 = 50PSF$
$DL = 56PSF$
$DL_{WALL} = 289 PLF$

$1.4DL = 1.4(56 \times \frac{8.75}{2}) + 1.4(289) = \underline{748 PLF}$

$1.2DL + 1.6LL = 1.2(56 \times \frac{8.75}{2}) + 1.2(289) + 1.6(50) = 721 PLF$

$M_U = \frac{(0.748)(24^2)}{8} = 54 k \cdot FT$

FROM PREV PG: ϕM_N FOR W10x22 = 97.5 k·FT > 54 k·FT ∴ W10x22 OK TO USE FOR EXTERIOR BEAM

CHECK Δ_{LL}:

[TABLE 3-20] I_{LB} FOR W10x22 = 214 in^4

$W_{LL} = \frac{(50)(8.75)}{1000} = 0.44 KLF$

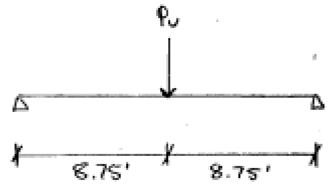
$\Delta_{LL} = \frac{5(0.44)(24^4)(1728)}{384(29000)(214)} = 0.53''$

$\Delta_{LLMAX} = \frac{L}{360} = \frac{24(12)}{360} = 0.8'' > 0.53''$ ∴ ok ✓

NOTEBOOK SUB B | ALT. SYSTEM 2 | COMPOSITE | 34

DETERMINE GIRDER DESIGN:

SPAN = 17.5'
SPACING = 24'
FROM PG 27 → w_u = 0.131 KLF



$$P_U = (0.148)(2 \times \tfrac{24}{2})(17.5') = 62\text{ k}$$

$$M_U = \frac{PL}{4} = \frac{(62)(17.5)}{4} = 270\text{ k·FT}$$

ASSUME $a = 1"$ ∴ $Y_2 = 4 - 0.5 = 3.5"$

W14x26 → $\Sigma Q_N = 279 \rightarrow 279/17.2 = 17 \times 2 = 34 \rightarrow \Sigma Q_N = (2)(16.5)(14.6) + 1(17.6) = 499$

W14x30 → $\Sigma Q_N = 183 \rightarrow 183/17.2 = 11 \times 2 = 22 \rightarrow \Sigma Q_N = (2)(5.5)(14.6) + 12(17.6) = \underline{372}$

W14x34 → $\Sigma Q_N = 175 \rightarrow 175/17.2 = 11 \times 2 = 22 \rightarrow \Sigma Q_N = (2)(5.5)(14.6) + 12(17.6) = 372$

W16x26 → $\Sigma Q_N = 96 \rightarrow 96/17.2 = 6 \times 2 = 12$

CHECK a ASSUMPTION:

$$b_{EFF} = \min \begin{cases} \text{SPAN}/8 = \frac{17.5(12)}{8} = 26.3 \times 2 = \underline{52.6} \\ \text{SPACING}/2 \quad \frac{24(12)}{2} = 36 \times 2 = 72 \end{cases}$$

x2 B/C GIRDER NOT AFFECTED BY EXTERIOR

$$a = \frac{\Sigma Q_N}{0.85 f'_c b_{EFF}} = \frac{183}{0.85(4.5)(52.6)} = 0.91" > 1" \therefore \text{ok} \checkmark$$

CHECK UNSHORED LENGTH:

$$1.4DL = 1.4\underbrace{(56)(24)}_{1344} + 1.4(26) = \underline{1918 \text{ PLF}}$$

$$1.2DL + 1.6LL = 1.2(1344 + 26) + 1.6(50) = 1724 \text{ PLF}$$

$$M_U = \frac{1.92(17.5)^2}{8} = 73.5 \text{ K·FT}$$

[TABLE 3-2] W14x30 → $\phi M_N = 177$ KFT > 73.5
$I_x = 291 \text{ in}^4$

NOTEBOOK SUB B ALT SYSTEM 2 COMPOSITE 35.

CHECK Δ_{wc}:

$W_{wc,girder} = 56(17.5) + 30 = 1010$ PLF

$$\Delta_{wc} = \frac{5(1.01)(17.5)^4(1728)}{(384)(29000)(245)} = 0.30''$$

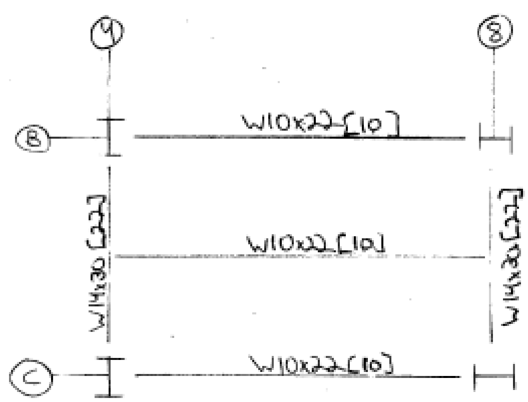
$$\Delta_{wc,max} = \frac{L}{240} = \frac{(17.5)(12)}{240} = 0.88'' > 0.3'' \therefore \text{OK} \checkmark$$

CHECK ECONOMY:

W14x36 → 26(17.5) + 34(10) = 795#

W14x30 → 30(17.5) + 22(10) = 745# ∴ SELECT W14x30 [22]

IN SUMMARY:



USE 2VLI20 DECKING, 2" NWC TOPPING (4" TOTAL)

NOTEBOOK SUB B COMPARISON COMPOSITE VS NONCOMPOSITE 36

COMPARE COMPOSITE VS. NON-COMPOSITE DESIGNS:

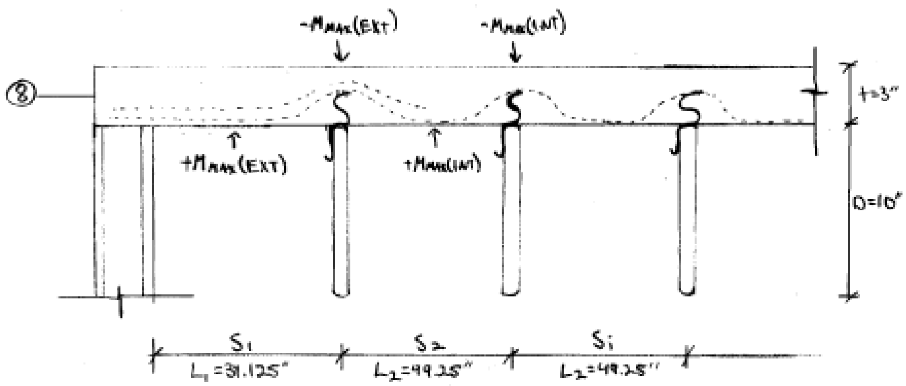
	COMPOSITE	NONCOMPOSITE
MEMBERS	(3) W10x22[10] (2) W14x30[22]	(4) W12x16 (2) W12x26
WEIGHT	2634#	2446#
DECK	2VLI20, 2" NW TOPPING (4" TOTAL)	1.5C20, 2.5" NW TOPPING (4" TOTAL)
SHORING	NO	NO
TOTAL DEPTH	18"	16"

BOTH SYSTEMS ARE VERY SIMILAR, HOWEVER EVEN THOUGH THE NONCOMPOSITE SYSTEM HAS MORE MEMBERS, THEY ARE LIGHTER & SHALLOWER, ALLOWING FOR A GREATER FLOOR-FLOOR HEIGHT. THEREFORE I WOULD RECOMMEND THE NONCOMPOSITE SYSTEM.

Alternative System 3: Hambro D-500 Composite Girder

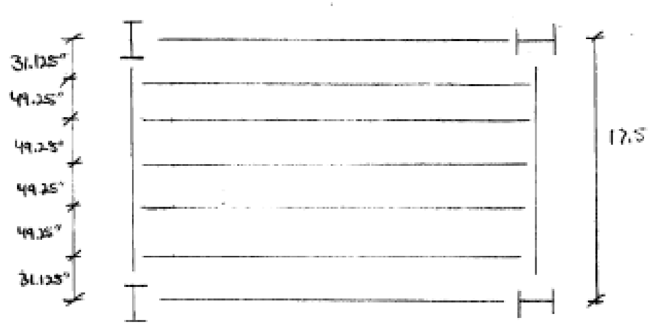
NOTEBOOK SUB B ALT. SYSTEM 3 HAMBRO D-500 37

SECTION CUT ALONG COLUMN LINE 8:



SPAN GIRDERS IN LONG DIRECTION ∴ SPAN = 24'

TYP. JOIST SPACING = 4'1¼" = 49¼" TO ACCOMODATE 48" PLYWOOD FORMS



FROM [TABLE 6]: D500 HAMBRO CLEAR SPAN TABLES:

CHOOSE RESIDENTIAL, t_s = 3" ∴ LL = 40 PSF, DL = 65 PSF

SPAN = 24' < 25' (FROM TABLE) ∴ CONSERVATIVE → JOIST DEPTH = 10"

MIN t_s = 2.5" < 3" ∴ OK ✓ ASSUME JOIST WEIGHT (RESIDENTIAL) = 1.5 PSF

$F'c$ = 3000 PSI, F_y = 50 KSI

$\Delta_{LL} = L/360$

NOTEBOOK SUB B ALT. SYSTEM 3 HAMBRO D-500 38

DETERMINE DESIGN MOMENT:

ACCORDING TO PG 2 HAMBRO DG: LOAD COMBO = 1.4DL+1.7LL

$$w = 1.4(65H.5) + 1.7(50) = 179 \text{ PSF} = 0.179 \text{ KSF}(\overset{\text{SPAN}}{24'}) = 4.3 \text{ KLF}$$

$$+M_{MAX(EXT)} = \frac{wL_1^2}{11} = \frac{(4.3)(\frac{31.125}{12})^2}{11} = 2.7 \text{ k·FT}$$

$$-M_{MAX(EXT)} = \frac{wL_1^2}{10} = \frac{(4.3)(\frac{31.125}{12})^2}{10} = 2.9 \text{ k·FT}$$

$$+M_{MAX(INT)} = \frac{wL_2^2}{16} = \frac{(4.3)(\frac{49.25}{12})^2}{16} = 4.6 \text{ k·FT}$$

$$-M_{MAX(INT)} = \frac{wL_2^2}{11} = \frac{(4.3)(\frac{49.25}{12})^2}{11} = 6.6 \text{ k·FT}$$

[TABLE 1] HAMBRO DG: $t_s = 3" \rightarrow w = 179 < 206$ (EXTERIOR) 226 (INTERIOR) $\therefore$ USE 6x6W2.9xW2.9 (d = 2.1")
(W/½" ROD SHOPWELDED TO TOP CHORD)

CONCENTRATED LL REQUIREMENTS:

[TABLE 2] HAMBRO DG: ASSUME MIN CONCENTRATED LOAD = 1000# FOR RESIDENTIAL [S.002]

[TABLE 3] HAMBRO DG: 1000# < 2000# MIN

$\therefore$ PROVIDE SINGLE LAYER MESH THROUGHOUT B/C $S_1 = 31.125 < 48"$

DEFLECTION CHECK:

[PG 2] HAMBRO DG → HAMBRO 2½" SLAB / 4'1¼" SPAN

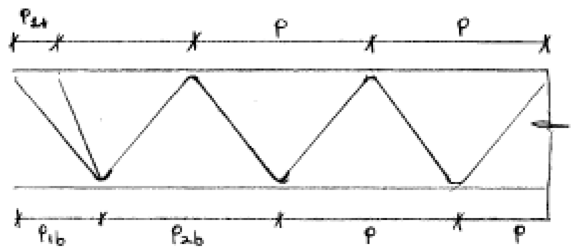
$$I_c = \frac{12(2.5^3)}{12} = 15.6 \text{ in}^4 \rightarrow \frac{\Delta}{L} = \frac{L^3}{I_c} = \frac{4.1^3}{15.6} = 4.4$$

NORMAL 7½" SLAB / 20' SPAN

$$I_c = \frac{12(7.5)^3}{12} = 422 \text{ in}^4 \rightarrow \frac{\Delta}{L} = \frac{L^3}{I_c} = \frac{20^3}{422} = 19$$

$\therefore \frac{\Delta}{L}$ HAMBRO << $\frac{\Delta}{L}$ NORMAL → DEFLECTION OK ✓

SHEAR (WEB) DESIGN:



[PG 9] HAMBRO DG: WEB GEOMETRY

$D = 10'' \therefore$ $P_{1t} = 6@12$

$P_{1b} = 6@16$

$P_{2b} = 12$

$P = 20$

IN SUMMARY:

TOTAL THICKNESS = 13"

JOIST SPACING = 49 1/4" TYP., 31.125" FOR EXTERIOR JOISTS

PROVIDE SINGLE LAYER 6x6xW2.9xW2.9 MESH THROUGHOUT

System Comparison

Floor System Comparison				
Criteria	**Girder Slab (Existing)**	**Non-Composite Steel**	**Composite Steel**	**Hambro D-500 Composite**
System Info				
Total Depth	10"	16"	18"	13"
Fire Rating	3hr	2hr	2hr	2hr
2 hr Fire Rating?	yes	yes	yes	yes
Lbs/ft^2	83	50	46	41
Cost/ft^2	$16.01	$11.17	$12.04	$8.38
Vibrations	minimal	likely	likely	very likely
Formwork	no	no	no	yes
Considerations				
Pros	Rapid construction & assembly (premanufactured), underside can be left unfinished, floor design flexibility	Lightweight	Lightweight, increased stiffness,	Lightweight, reusable formwork & rollbars, increased rigidity from composite, plenums allow for MEP systems
Cons	Heavy, expensive	Large total depth	Largest total depth	Formwork needed, vibrations
Feasible?	yes	yes	yes	yes

Appendix A

D-Beam® Calculator Reference Tool Version 3.1
(Load & Resistance Factor Design - AISC 14th Edition)

LRFD (14th Ed)

Project Name / Job #

D-Beam®

D-Beam® =	DB 8x45	
Parent Beam Yield Stress (F_y) =	50	ksi
Top Bar Yield Stress (F_y) =	50	ksi

Span Information

D-Beam® Span =	24	ft
Composite Section Effective Width =	6	ft
Total Tributary Width for Load =	17.5	ft

Precast Slab

Nominal Slab Thickness =	8 in.	
Precast Slab Weight =	58	psf

Grout

Unit Weight of Grout =	140	lb/ft³

Unfactored Loads

Basic Dead Load (D-Beam® + Slab + Grout) =	61.5	psf
Add'l Composite Dead Load (e.g. topping) =	25	psf
Partition Live Load =	10	psf
Basic Floor Live Load =	40	psf
Consider Floor Live Load Redution (IBC 2009/2012) =	Yes	
Floor Live Load Reduction =	23.2%	
Reduced Floor Live Load =	30.7	psf

Factored Moments

	1.4D	1.2D+1.6L	
Basic Dead Load Moment =	108.51	93.01	kip-ft
Add'l Composite Dead Load Moment =	44.10	37.80	kip-ft
Partition Live Load Moment =	0.00	20.16	kip-ft
Floor Live Load Moment =	0.00	61.90	kip-ft
Total Factored Moment =	152.61	212.87	kip-ft

Factored Shears

	1.4D	1.2D+1.6L	
Basic Dead Load Shear =	18.09	15.50	kips
Add'l Composite Dead Load Shear =	7.35	6.30	kips
Partition Live Load Shear =	0.00	3.36	kips
Floor Live Load Shear =	0.00	10.32	kips
Total Factored Shear =	25.44	35.48	kips

Deflections (negative values indicate downward deflection)

(optional) D-Beam® Camber =	1.25	in	
Basic Dead Load Deflection =	-2.11	in	
Net Basic Dead Load Deflection including Camber =	-0.86	in	
Add'l Composite Dead Load Deflection =	-0.34	in	
Partition Live Load Deflection =	-0.13	in	
Floor Live Load Deflection =	-0.41	in	(=L/695)
Total (Net) Deflection due to all loads =	-1.75	in	(=L/165)

** Elastic and plastic section moduli (S and Z, respectively) are based on entire cross section being transformed into the parent beam (D-Beam bottom tee) material.

Design Checks - Noncomposite

Noncomposite Moment			OK
M_u =	108.5	kip-ft	
$\phi_b M_n$ =	137.5	kip-ft	
Horizontal Shear			OK
V_u =	18.1	kips	
ϕV_n =	26.4	kips	

Design Checks - Full Composite

Floor LL Deflection	Allow. Δ_{LL} = L/ 360		OK
Δ_{LL} =	-0.41	in	
L/360 =	-0.80	in	
Full Composite Moment			OK
M_u =	212.9	kip-ft	
$\phi_b M_n$ =	216.6	kip-ft	
Flexural Ductility Check			OK
$\varepsilon_{web/flange\ intersection}$ =	0.010994		
$2\varepsilon_y$ =	0.003448		
Shear			OK
V_u =	35.5	kips	
$\phi_v V_n$ =	58.2	kips	

CROSS SECTION ANALYSIS IS VALID → RUN

Section Properties **

		Noncomposite		Full Composite
Gross Section Properties				
$NA_{bot\ DB}$	in	3.21		4.10
I_g	in⁴	131		400
$S_{bot\ DB}$	in³	40.8		97.6
$S_{top\ DB}$	in³	27.4		102.6
$Q_{top\ bar}$	in³	18.2		---
Elastic (Cracked) Section Properties				
$NA_{bot\ DB}$	in	---		5.41
I_{cr}	in⁴	---		267
$S_{bot\ DB}$	in³	---		49.4
$S_{top\ DB}$	in³	---		103.2
Effective Moment of Inertia (for deflection calculations)				
I_{eff}	in⁴	131		334
Effective Plastic Section Properties				
$PNA_{bot\ DB}$	in	0.85		6.91
Z	in³	36.67		57.76
Load Resisted by Each Cross Section		Basic DL (B+S+G)		Add'l Comp. DL Partition LL Floor LL

D-Beam® Calculator Reference Tool Version 3.1
(Load & Resistance Factor Design - AISC 14th Edition)

LRFD (14th Ed)

Project Name / Job #

D-Beam®		
D-Beam® =	DB 8x57	
Parent Beam Yield Stress (F_y) =	50	ksi
Top Bar Yield Stress (F_y) =	50	ksi
Span Information		
D-Beam® Span =	24	ft
Composite Section Effective Width =	6	ft
Total Tributary Width for Load =	17.5	ft
Precast Slab		
Nominal Slab Thickness =	8 in.	
Precast Slab Weight =	58	psf
Grout		
Unit Weight of Grout =	140	lb/ft^3

Unfactored Loads		
Basic Dead Load (D-Beam® + Slab + Grout) =	62.5	psf
Add'l Composite Dead Load (e.g. topping) =	25	psf
Partition Live Load =	10	psf
Basic Floor Live Load =	40	psf
Consider Floor Live Load Redution (IBC 2009/2012) =	Yes	
Floor Live Load Reduction =	23.2%	
Reduced Floor Live Load =	30.7	psf

Factored Moments	1.4D	1.2D+1.6L	
Basic Dead Load Moment =	110.25	94.50	kip-ft
Add'l Composite Dead Load Moment =	44.10	37.80	kip-ft
Partition Live Load Moment =	0.00	20.16	kip-ft
Floor Live Load Moment =	0.00	61.90	kip-ft
Total Factored Moment =	154.35	214.35	kip-ft

Factored Shears	1.4D	1.2D+1.6L	
Basic Dead Load Shear =	18.37	15.75	kips
Add'l Composite Dead Load Shear =	7.35	6.30	kips
Partition Live Load Shear =	0.00	3.36	kips
Floor Live Load Shear =	0.00	10.32	kips
Total Factored Shear =	25.72	35.73	kips

Deflections (negative values indicate downward deflection)			
(optional) D-Beam® Camber =	1.25	in	
Basic Dead Load Deflection =	-1.67	in	
Net Basic Dead Load Deflection including Camber =	-0.42	in	
Add'l Composite Dead Load Deflection =	-0.28	in	
Partition Live Load Deflection =	-0.11	in	
Floor Live Load Deflection =	-0.34	in	(=L/837)
Total (Net) Deflection due to all loads =	-1.15	in	(=L/250)

** Elastic and plastic section moduli (S and Z, respectively) are based on entire cross section being transformed into the parent beam (D-Beam bottom tee) material.

Design Checks - Noncomposite

Noncomposite Moment			OK
M_u =	110.2	kip-ft	
$\phi_b M_n$ =	159.2	kip-ft	
Horizontal Shear			OK
V_u =	18.4	kips	
ϕV_n =	34.2	kips	

Design Checks - Full Composite

Floor LL Deflection	Allow. Δ_{LL} = L/	360	OK
Δ_{LL} =	-0.34	in	
L/360 =	-0.80	in	
Full Composite Moment			OK
M_u =	214.4	kip-ft	
$\phi_b M_n$ =	297.2	kip-ft	
Flexural Ductility Check			
$\varepsilon_{web/flange\ intersection}$ =			
$2\varepsilon_y$ =			
Shear			OK
V_u =	35.7	kips	
$\phi_v V_n$ =	72.6	kips	

CROSS SECTION ANALYSIS IS VALID

RUN

Section Properties **

		Noncomposite		Full Composite
Gross Section Properties				
$NA_{bot\ DB}$	in	2.93		4.00
I_g	in^4	169		436
$S_{bot\ DB}$	in^3	57.7		114.2
$S_{top\ DB}$	in^3	33.3		114.0
$Q_{top\ bar}$	in^3	22.9		---
Elastic (Cracked) Section Properties				
$NA_{bot\ DB}$	in	---		5.01
I_{cr}	in^4	---		348
$S_{bot\ DB}$	in^3	---		69.5
$S_{top\ DB}$	in^3	---		116.4
Effective Moment of Inertia (for deflection calculations)				
I_{eff}	in^4	169		402
Effective Plastic Section Properties				
$PNA_{bot\ DB}$	in	0.68		6.40
Z	in^3	42.44		79.26
Load Resisted by Each Cross Section		Basic DL (B+S+G)		Add'l Comp. DL Partition LL Floor LL

D-Beam® Calculator Reference Tool Version 3.1
(Load & Resistance Factor Design - AISC 14th Edition)

LRFD (14th Ed)

Project Name / Job #

D-Beam®

D-Beam® =	DB 8x61	
Parent Beam Yield Stress (F_y) =	50	ksi
Top Bar Yield Stress (F_y) =	50	ksi

Span Information

D-Beam® Span =	24	ft
Composite Section Effective Width =	6	ft
Total Tributary Width for Load =	17.5	ft

Precast Slab

Nominal Slab Thickness =	8 in.	
Precast Slab Weight =	58	psf

Grout

Unit Weight of Grout =	140	lb/ft^3

Unfactored Loads

Basic Dead Load (D-Beam® + Slab + Grout) =	62.7	psf
Add'l Composite Dead Load (e.g. topping) =	25	psf
Partition Live Load =	10	psf
Basic Floor Live Load =	40	psf
Consider Floor Live Load Redution (IBC 2009/2012) =	Yes	
Floor Live Load Reduction =	23.2%	
Reduced Floor Live Load =	30.7	psf

Factored Moments

	1.4D	1.2D+1.6L	
Basic Dead Load Moment =	110.54	94.75	kip-ft
Add'l Composite Dead Load Moment =	44.10	37.80	kip-ft
Partition Live Load Moment =	0.00	20.16	kip-ft
Floor Live Load Moment =	0.00	61.90	kip-ft
Total Factored Moment =	154.64	214.61	kip-ft

Factored Shears

	1.4D	1.2D+1.6L	
Basic Dead Load Shear =	18.42	15.79	kips
Add'l Composite Dead Load Shear =	7.35	6.30	kips
Partition Live Load Shear =	0.00	3.36	kips
Floor Live Load Shear =	0.00	10.32	kips
Total Factored Shear =	25.77	35.77	kips

Deflections (negative values indicate downward deflection)

(optional) D-Beam® Camber =	1.25	in	
Basic Dead Load Deflection =	-1.50	in	
Net Basic Dead Load Deflection including Camber =	-0.25	in	
Add'l Composite Dead Load Deflection =	-0.28	in	
Partition Live Load Deflection =	-0.11	in	
Floor Live Load Deflection =	-0.34	in	(=L/851)
Total (Net) Deflection due to all loads =	-0.98	in	(=L/294)

** Elastic and plastic section moduli (S and Z, respectively) are based on entire cross section being transformed into the parent beam (D-Beam bottom tee) material.

Design Checks - Noncomposite

Noncomposite Moment			OK
M_u =	110.5	kip-ft	
$\phi_b M_n$ =	190.3	kip-ft	
Horizontal Shear			OK
V_u =	18.4	kips	
ϕV_n =	33.5	kips	

Design Checks - Full Composite

Floor LL Deflection	Allow. Δ_{LL} = L/ 360		OK
Δ_{LL} =	-0.34	in	
L/360 =	-0.80	in	
Full Composite Moment			OK
M_u =	214.6	kip-ft	
$\phi_b M_n$ =	298.4	kip-ft	
Flexural Ductility Check			
$\varepsilon_{web/flange\ intersection}$ =			
$2\varepsilon_y$ =			
Shear			OK
V_u =	35.8	kips	
$\phi_v V_n$ =	75.9	kips	

CROSS SECTION ANALYSIS IS VALID → RUN

Section Properties **

		Noncomposite		Full Composite
Gross Section Properties				
$NA_{bot\ DB}$	in	3.22		4.07
I_g	in^4	188		463
$S_{bot\ DB}$	in^3	58.2		114.3
$S_{top\ DB}$	in^3	39.2		118.5
$Q_{top\ bar}$	in^3	26.0		---
Elastic (Cracked) Section Properties				
$NA_{bot\ DB}$	in	---		5.08
I_{cr}	in^4	---		332
$S_{bot\ DB}$	in^3	---		69.2
$S_{top\ DB}$	in^3	---		120.4
Effective Moment of Inertia (for deflection calculations)				
I_{eff}	in^4	188		409
Effective Plastic Section Properties				
$PNA_{bot\ DB}$	in	0.73		6.84
Z	in^3	50.75		79.56
Load Resisted by Each Cross Section		Basic DL (B+S+G)		Add'l Comp. DL Partition LL Floor LL

Design Principles and Calculations - Slab Design

Table 1 - Slab Capacity Chart (Total Load in psf)

SLAB THICKNESS (t)	d	MESH SIZE F_y = 60,000 psi	4'-1 1/4" JOIST SPACING	
			Exterior	Interior
t ≥ 2 1/2" No chair	1.6"	6 x 6 W2.0 x W2.0	114	123
		6 x 6 W2.0 x W2.9	157	172
		6 x 6 W4.0 x W4.0	210	230
t ≥ 3" with 1/2" Rod (shop welded to top chord)	2.1"	6 x 6 W2.9 x W2.9	206	226
		6 x 6 W4.0 x W4.0	279	306
t ≥ 3 1/2" with 2 1/2" Chair	2.6"	6 x 6 W2.9 x W2.9	256	280
		6 x 6 W4.0 x W4.0	347	380

Note: Slab capacities are based on mesh over joists raised as indicated.

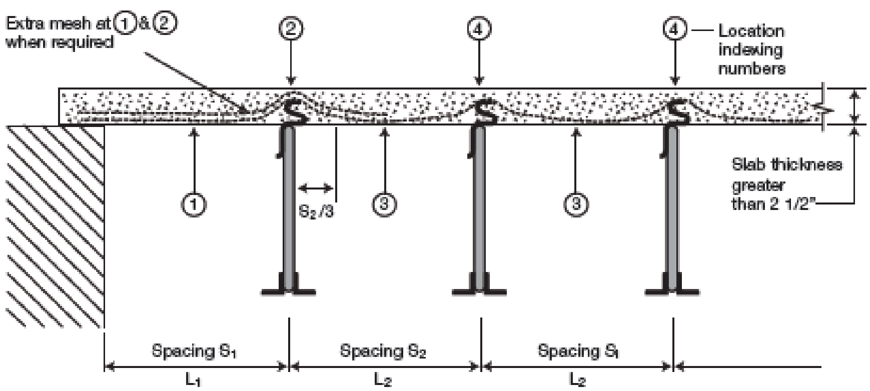


Fig. 2

Design Principles and Calculations - Slab Design

TABLE 3 - Concentrated Loads with 4'-1 1/4" Joist Spacing

CONCENTRATED LOAD	SLAB THICKNESS	MESH SIZE	SPECIAL REMARKS	
2000 lbs. on 2'-6" square area (office building)	2 1/2"	6 x 6 - W2.9	Extra layer @ ①	No "chairs" on
		6 x 6 - W2.9	Single layer throughout but S_1 = 3'-10" max.	
	3"	6 x 6 - W2.9	Extra layer @ ① and ②	
		6 x 6 - W2.9	Single layer throughout but S_1 = 4'-0" max.	
		6 x 6 - W2.9	Single layer throughout	
2500 lbs. on 2'-6" * square area plus 2" asphalt wearing surface	3"	6 x 6 - W2.9	Extra layer @ ① and ②	No "chairs" on
		6 x 6 - W2.9	Single layer throughout but S_1 = 2'-10" max.	
4000 lbs. on 3'-6" square area (office building for some codes)	2 1/2"	6 x 6 - W4.0	S_1 = 4'-0"	No "chairs" on
	3"	6 x 6 - W2.9	Extra layer @ ① and ②	
		6 x 6 - W2.9	Single layer throughout but S_1 = 2'-10" max.	

* Some building codes use different bearing areas.

TABLE 4 - Concentrated Loads with 5'-1 1/4" Joist Spacing

CONCENTRATED LOAD	SLAB THICKNESS	MESH SIZE	SPECIAL REMARKS	
2000 lbs. on 2'-6" square area (office building)	3"	6 x 6 - W2.9	Extra layer @ ① and ②	No "chairs" on
4000 lbs. on 3'-6" square area (office for some codes)	3"	6 x 6 - W4.0	Extra layer @ ① and ②	No "chairs" on

Concrete Mix

Top size of the coarse aggregate should not exceed *3/4"* or as dictated by applicable codes. A slump of *4"* is recommended.

Design Principles and Calculations - Web Design

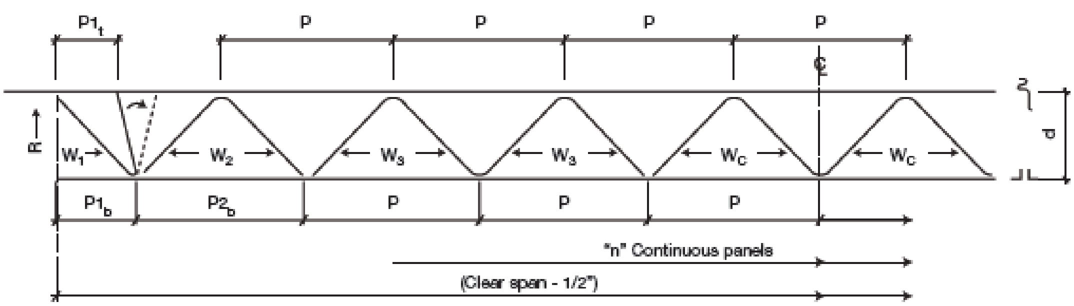
Vertical Shear (Web Design)

The vertical shear forces are assumed to be carried entirely by the web member, forces being calculated using the conventional pin jointed truss analysis method. These assumptions result in calculated bar forces which have been shown by tests to be as much as 15% higher than the actual values because the slab, acting compositely with section, is stiff enough to transmit some load directly to the support. This is particularly true of web members at the joist ends - those which are subjected to the highest vertical shear.

Effective Length of Compression Diagonal

With the web member forces calculated as below, the bar sections are sized to prevent failure in either axial tension or axial compression using conventional working stress design procedures. As per AISC specifications fig. 7 is used as a reference in determining the effective length, k_l, of the compression diagonals.

It is important to note that the web members are sized for the specified load capacity including concentrated loads where applicable. Furthermore, the webs are designed according to the latest requirements of the Steel Joist Institute.



NOTE: W_3 for longer span.

Fig. 7
D500™ and MD2000® Geometry

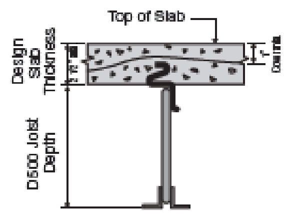
WEB GEOMETRY (in.)				
NOM. DEPTH "d"	$P1_t$	$P1_b$	$P2_b$	P
8, 10	6 @ 12	6 @ 16	12	20
12	10 @ 16	10 @ 21	16	24
14, 16	15 @ 24	15 @ 32	20	24
18, 20, 22, 24	19 @ 24	19 @ 32	24	24

Hambro Span Tables

TABLE 6: D500™ Clear Span Table

	Residential			Commercial	
Slab Thickness	3"	3 1/2"	4"	3"	4"
Joist Depth*	LL = 40 psf DL = 65 psf	LL = 40 psf DL = 71 psf	LL = 40 psf DL = 77 psf	LL = 50 psf DL = 65 psf	LL = 50 psf DL = 77 psf
8"	20' - 0"	20' - 0"	20' - 0"	20' - 0"	20' - 0"
10"	25' - 0"	24' - 6"	23' - 6"	25' - 0"	23' - 6"
12"	30' - 0"	27' - 0"	26' - 0"	30' - 0"	26' - 0"
14"	31' - 0"	29' - 6"	28' - 0"	31' - 0"	28' - 0"
16"	33' - 6"	32' - 0"	30' - 6"	33' - 6"	30' - 6"
18"	36' - 0"	34' - 0"	32' - 6"	36' - 0"	32' - 6"
20"	38' - 6"	36' - 0"	34' - 6"	38' - 6"	34' - 6"
22"	40' - 6"	38' - 6"	36' - 6"	40' - 6"	36' - 6"
24"	43' - 0"	40' - 6"	38' - 0"	43' - 0"	38' - 0"

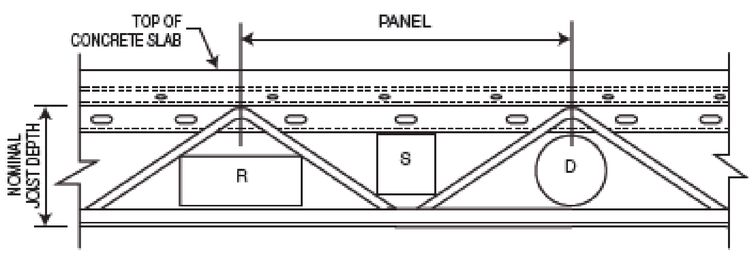
* Total floor depth = D500™ Joist depth plus slab thickness



Notes:

- Minimum slab thickness = *2 1/2"*
- Minimum top chord cover = *1"*
- $f'_c = 3{,}000$ psi, $F_y = 50$ ksi
- Table reflects uniform loads only.
- Standard spacing is *4'-1 1/4"*
- Live load deflection design standard: *L / 360*
- Design clear spans, other than those shown in the above table, require additional structural review.
- Design > 43' - 0" require additional structural design review.

Maximum Duct Openings



D = MAXIMUM DIAMETER
S = MAXIMUM SQUARE
R = MAXIMUM RECTANGULAR

DEPTH (in.)	PANEL (in.)	D (in.)	S (in.)	R (in. x in.)
8	20	4	4	6 x 3
10	20	6	5	7 x 4
12	24	8	6	9 x 5
14	24	9	7	9 1/2 x 6 11 x 5
16	24	10	8	10 1/2 x 6 1/2 13 x 5
18	24	11	8 1/2	11 x 7 12 1/2 x 6
20	24	11 1/2	9	12 x 7 13 x 6
22	24	12	9 1/2	12 x 8 14 x 6
24	24	12 1/2	10	13 x 8 14 x 7

NOTE: For other configurations, the maximum limits will be defined by the joist geometry.

System	Element	Unit	Unit Cost	Cost/SF
Girder Slab	Precast Hollow-Core Plank (8" thick)			$10.40
	DB8x61 (W8x31)	LF	$49.08	$5.61
				$16.01
Non-Composite	W12x16	LF	$28.51	$6.52
	W12x26	LF	$43.01	$3.58
	2.5" NW Topping	CF	$3.96	$1.07
				$11.17
Composite	W10x22	LF	$39.35	$6.75
	W14x30	LF	$48.40	$4.03
	2" NW Topping	CF	$3.96	$0.99
	Weld Studs	per stud	$1.52	$0.27
				$12.04
Hambro D-500	Steel Joists	LF	$11.44	$3.27
	3" Concrete Slab	CF	$3.96	$2.61
	Formwork	SF	$1.87	$1.87
	Weld Studs	per stud	$1.52	$0.43
	Wire Mesh	SF	$0.20	$0.20
				$8.38