Structural Notebook
Submission C

AC Hotel Philadelphia
Philadelphia, Pennsylvania

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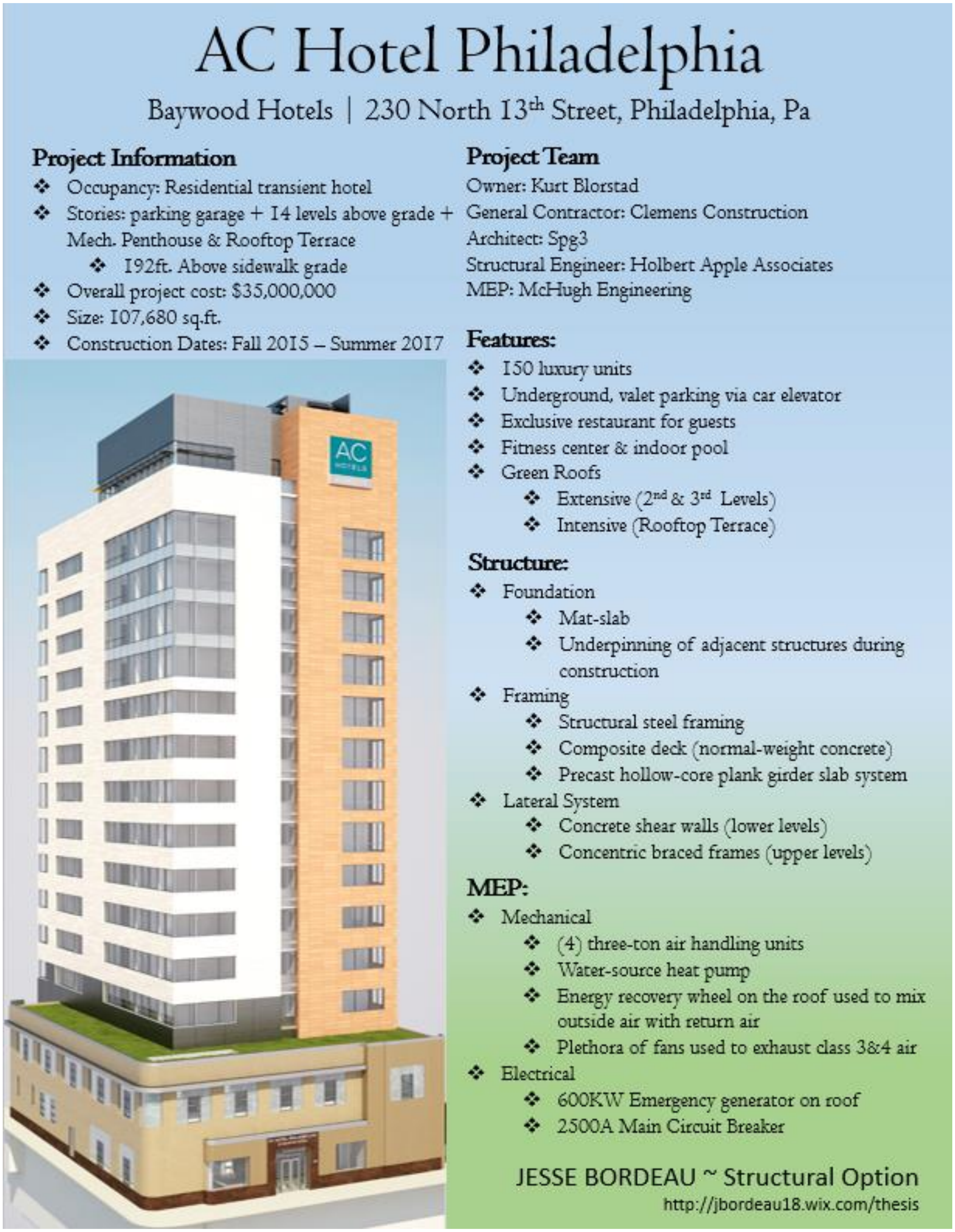
Executive Summary

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 northeast 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)

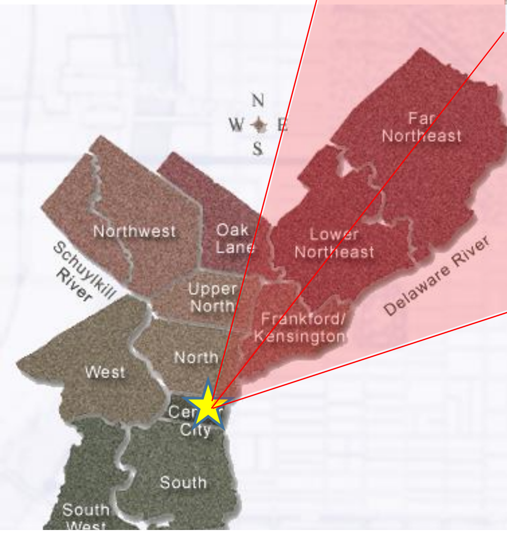


Figure 2: Map of Philadelphia (Courtesy Google Maps)

Documents used in preparation for this report

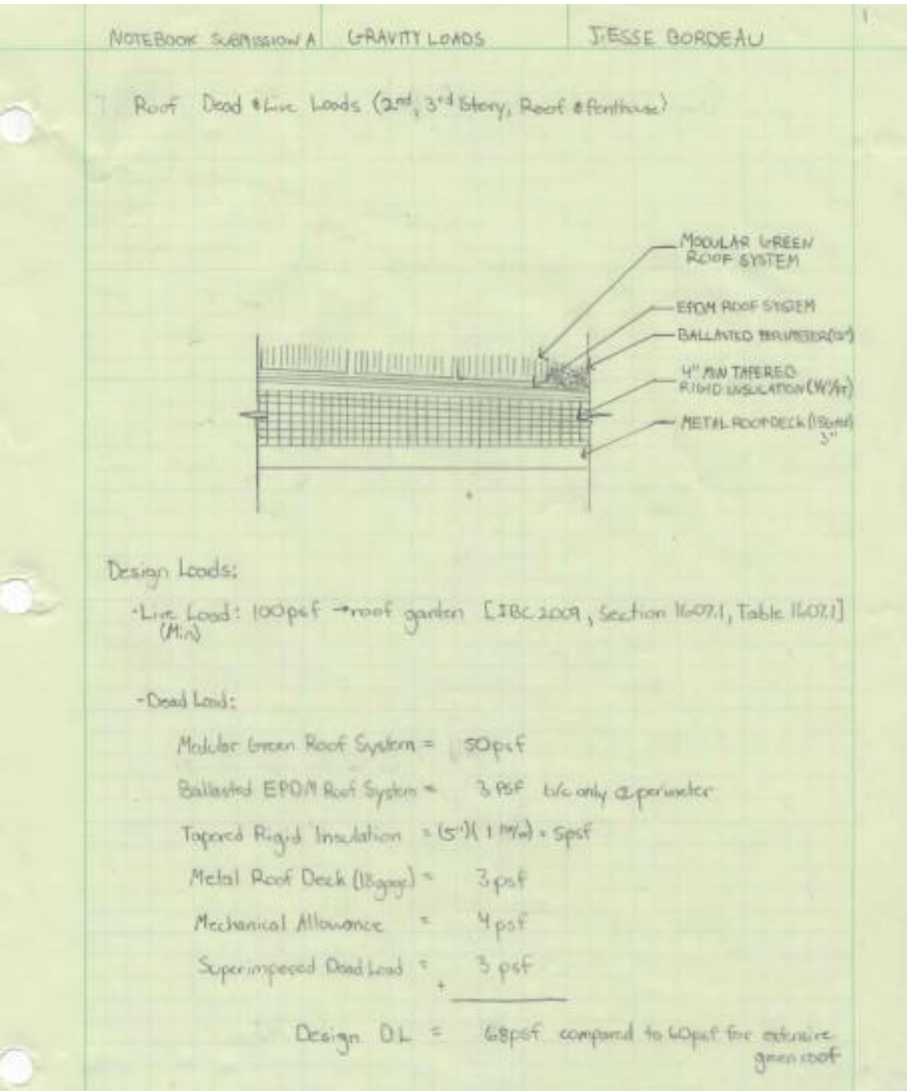
Listed below are the codes and other supporting documents which were used to determine loads, factors and other variables for this report.

- 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
    - AE530 – Computer Modeling of Buildings
    - AE 430 - Indeterminate Structures
    - AE 403 - Advanced Steel Design
- Hambro Composite Floor System Design Guide
- Girder-Slab System LRFD Version Design Guide v3.1
    - Courtesy Holbert Apple Associates
- AISC Steel Manual

Gravity Load Determination (Dead, Live & Snow)

Roof Loads

The roof load calculated below is for the extensive 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

Roof Dead & Live Loads (2nd, 3rd 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 = 3 PSF b/c only @ perimeter

Tapered Rigid Insulation = (5")(1 PSF/in) = 5psf

Metal Roof Deck (18gage) = 3psf

Mechanical Allowance = 4psf

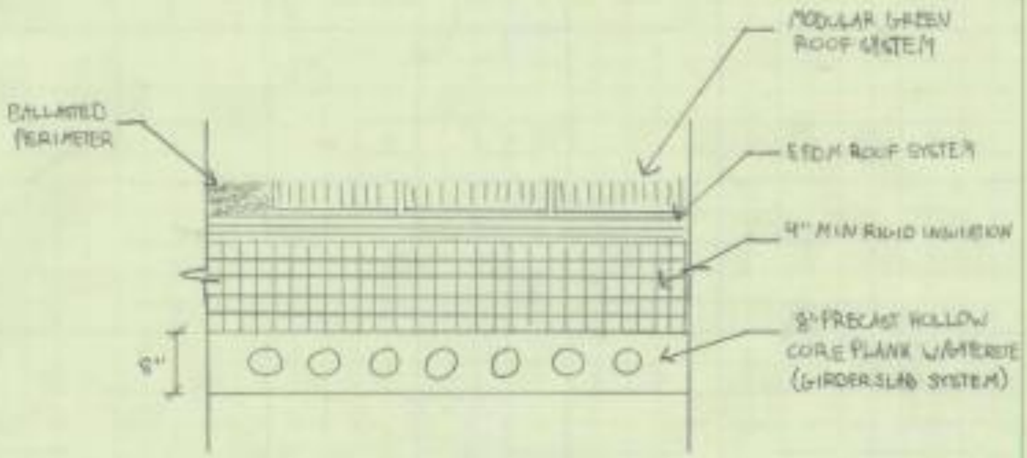
Superimposed Dead Load = + 3 psf

Design DL = 68psf compared to 60psf for extensive green roof

Note: Compared to extensive green roofs. 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 | GRAVITY CONT. | JESSE BORDEAU 1

ROOF DEAD & LIVE LOAD (PENTHOUSE)



DESIGN LOADS:

-LIVE LOAD: 100PSF

-DEAD LOAD:

MODULAR GREEN ROOF SYSTEM = 40PSF

BALLASTED EPDM ROOF SYSTEM = 3 PSF

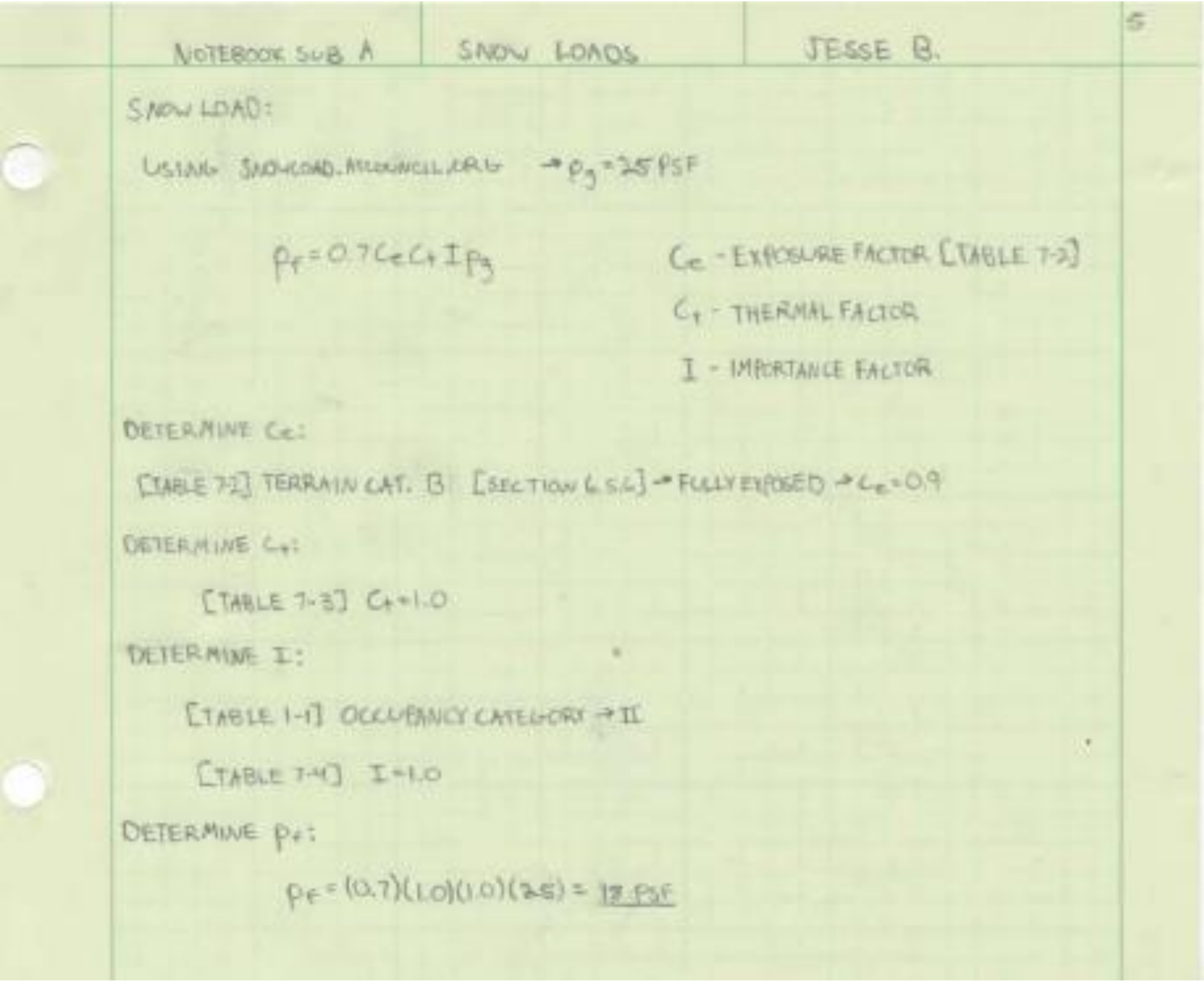
RIGID INSULATION (4" MIN) = 5PSF

8" PRECAST HOLLOW-CORE PLANK = 58+(2/12)x150= 83
2" CONCRETE TOPPING

MECHANICAL ALLOWANCE = 4 PSF

SUPERIMPOSED DL = 3 PSF

DESIGN DL = 138 PSF



SNOW LOAD:

USING SNOWLOAD.ATCOUNCIL.ORG → $p_g = 25$ 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] $C_t = 1.0$

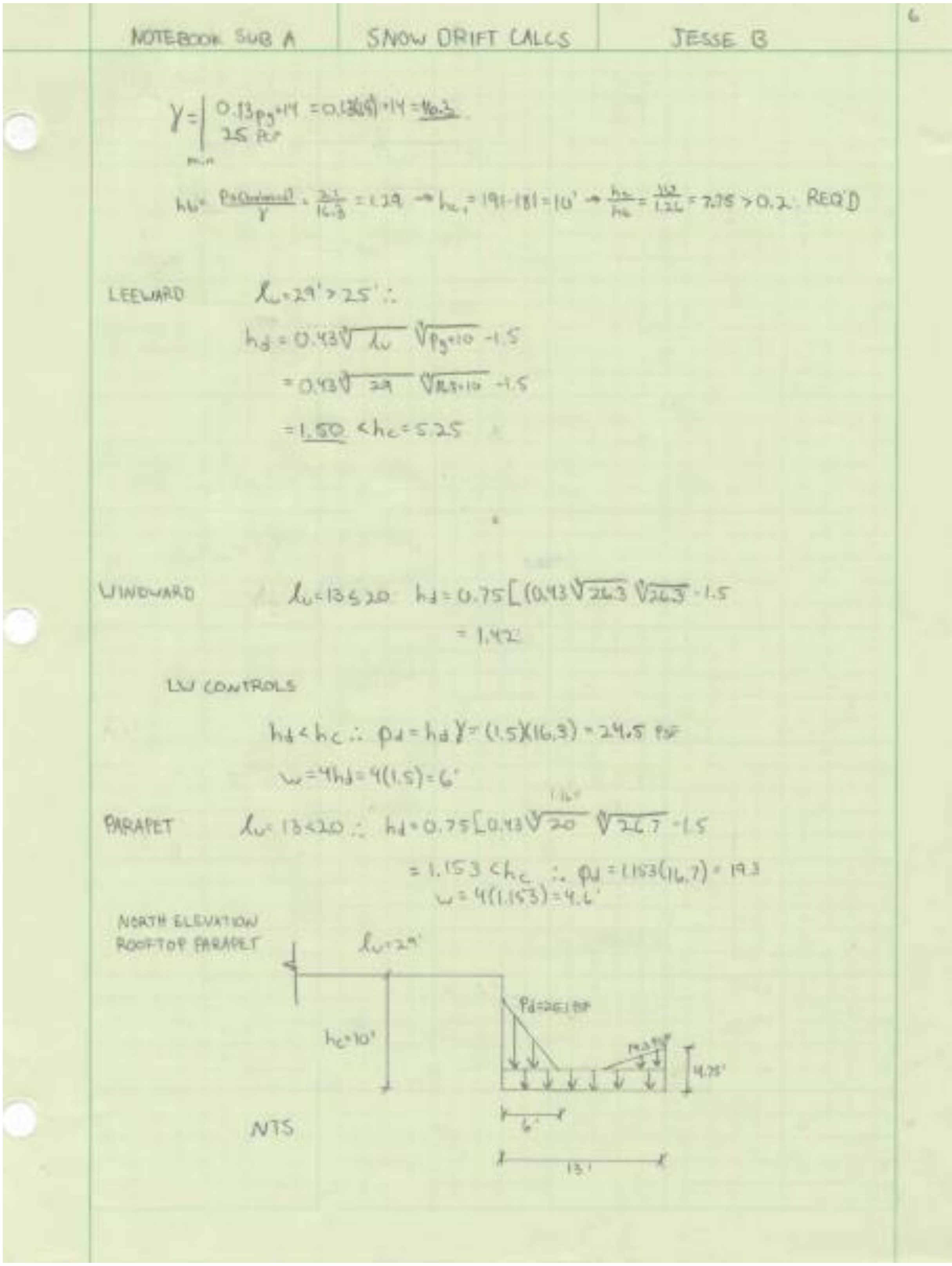
DETERMINE I:

[TABLE 1-1] OCCUPANCY CATEGORY → II

[TABLE 7-4] $I = 1.0$

DETERMINE p_f:

$$p_f = (0.7)(1.0)(1.0)(25) = \underline{18\ PSF}$$



NOTEBOOK SUB A | SNOW DRIFT CALCS | JESSE B 6

$\gamma = \left| \begin{array}{l} 0.13p_g+14 = 0.13(18)+14 = \underline{16.3} \\ 25 \text{ PCF} \end{array} \right._{min}$

$h_b = \frac{P_{d(unbalanced)}}{\gamma} = \frac{21}{16.3} = 1.29 \rightarrow h_{c_1} = 191-181 = 10' \rightarrow \frac{h_c}{h_b} = \frac{10}{1.26} = 7.75 > 0.2 \therefore$ REQ'D

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

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

$= 0.43\sqrt[3]{29}\sqrt[4]{18.5+10} - 1.5$

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

WINDWARD $\ell_u = 13 \le 20$ $h_d = 0.75[(0.43\sqrt[3]{26.3}\sqrt[4]{26.3} - 1.5$

$= 1.42$

LW CONTROLS

$h_d < h_c \therefore p_d = h_d\gamma = (1.5)(16.3) = 24.5$ PSF

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

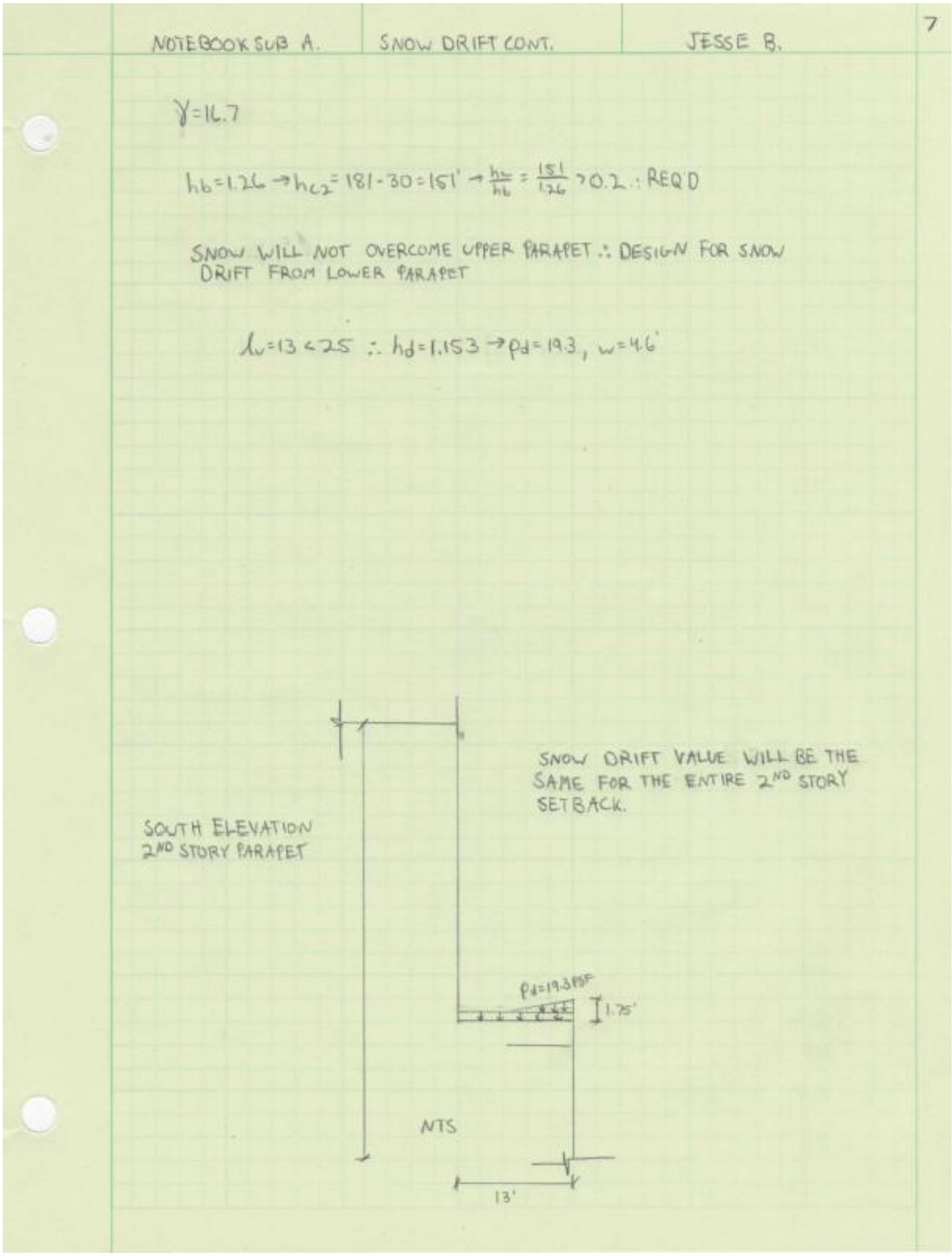
PARAPET $\ell_u = 13 < 20 \therefore$ $h_d = 0.75[0.43\overset{1.16}{\sqrt[3]{20}}\sqrt[4]{26.7} - 1.5$

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

$w = 4(1.153) = 4.6'$

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 ∴ 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

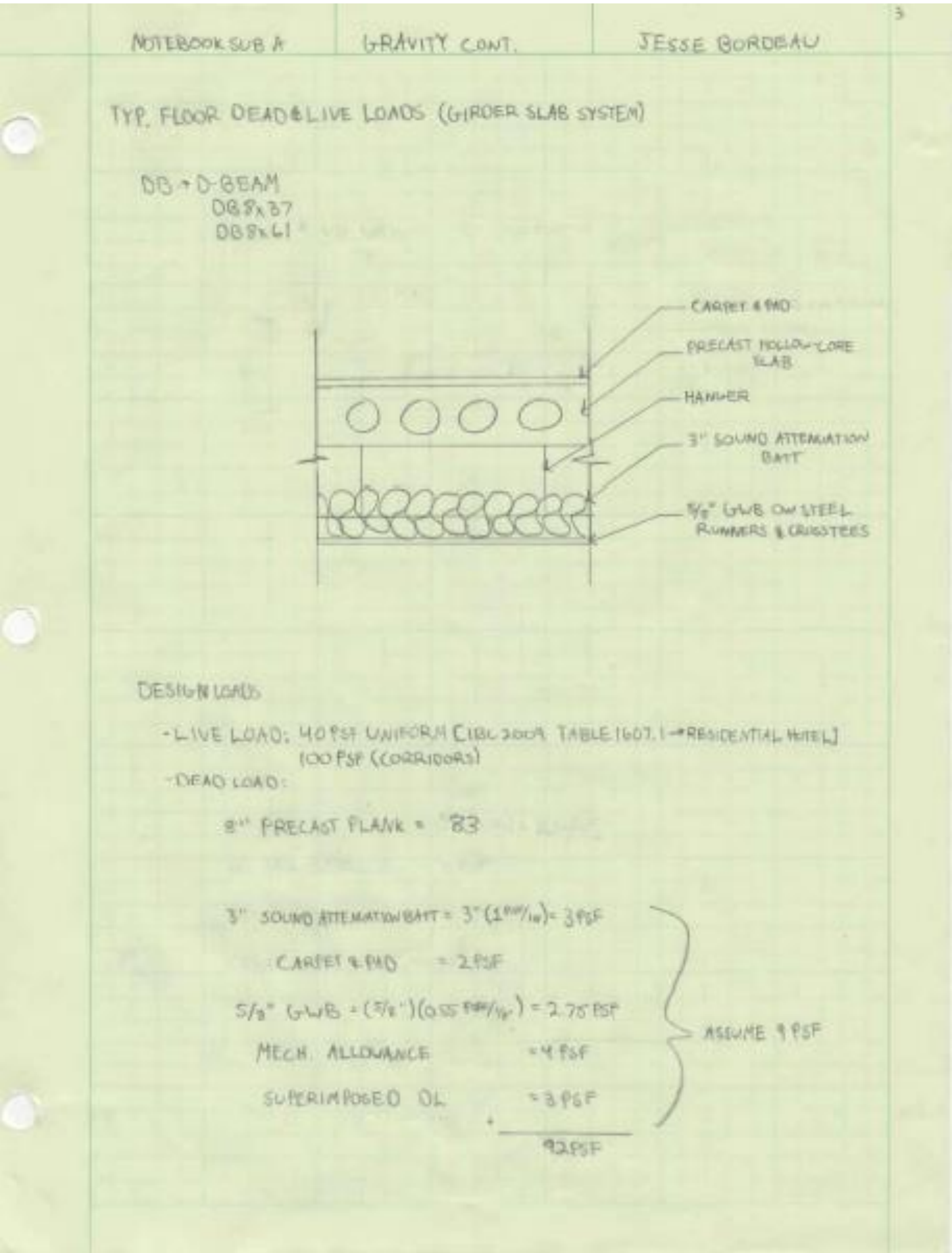
$p_d = 19.3$ PSF

1.75'

NTS

13'

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]
  100 PSF (CORRIDORS)
- DEAD LOAD:

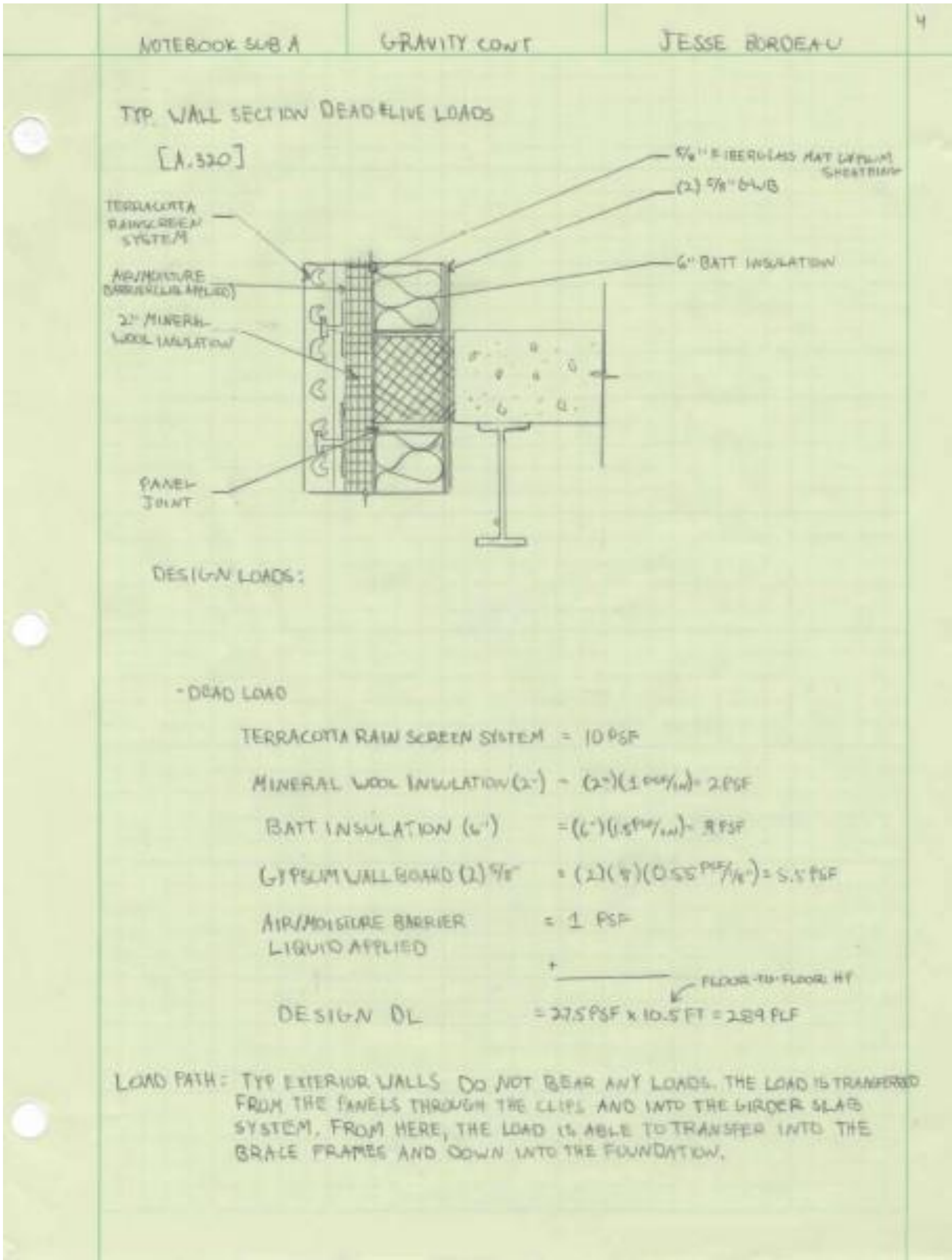
8" PRECAST PLANK = 83

3" SOUND ATTENUATION BATT = 3"(1 PSF/IN) = 3 PSF
CARPET & PAD = 2 PSF
5/8" GWB = (5/8")(0.55 PSF/1/8") = 2.75 PSF
MECH. ALLOWANCE = 4 PSF
SUPERIMPOSED DL = 3 PSF
\+ ______
92PSF

ASSUME 9 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")(1.5 PSF/IN) = 9 PSF

GYPSUM WALL BOARD (2) 5/8" = (2)(5)(0.55 PSF/⅛") = 5.5 PSF

AIR/MOISTURE BARRIER LIQUID APPLIED = 1 PSF

+

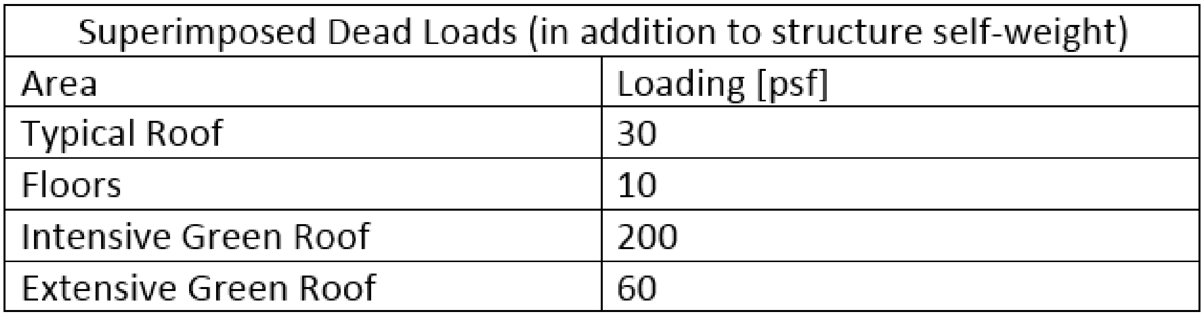
FLOOR-TO-FLOOR HT

DESIGN DL = 27.5 PSF x 10.5 FT = 289 PLF

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.

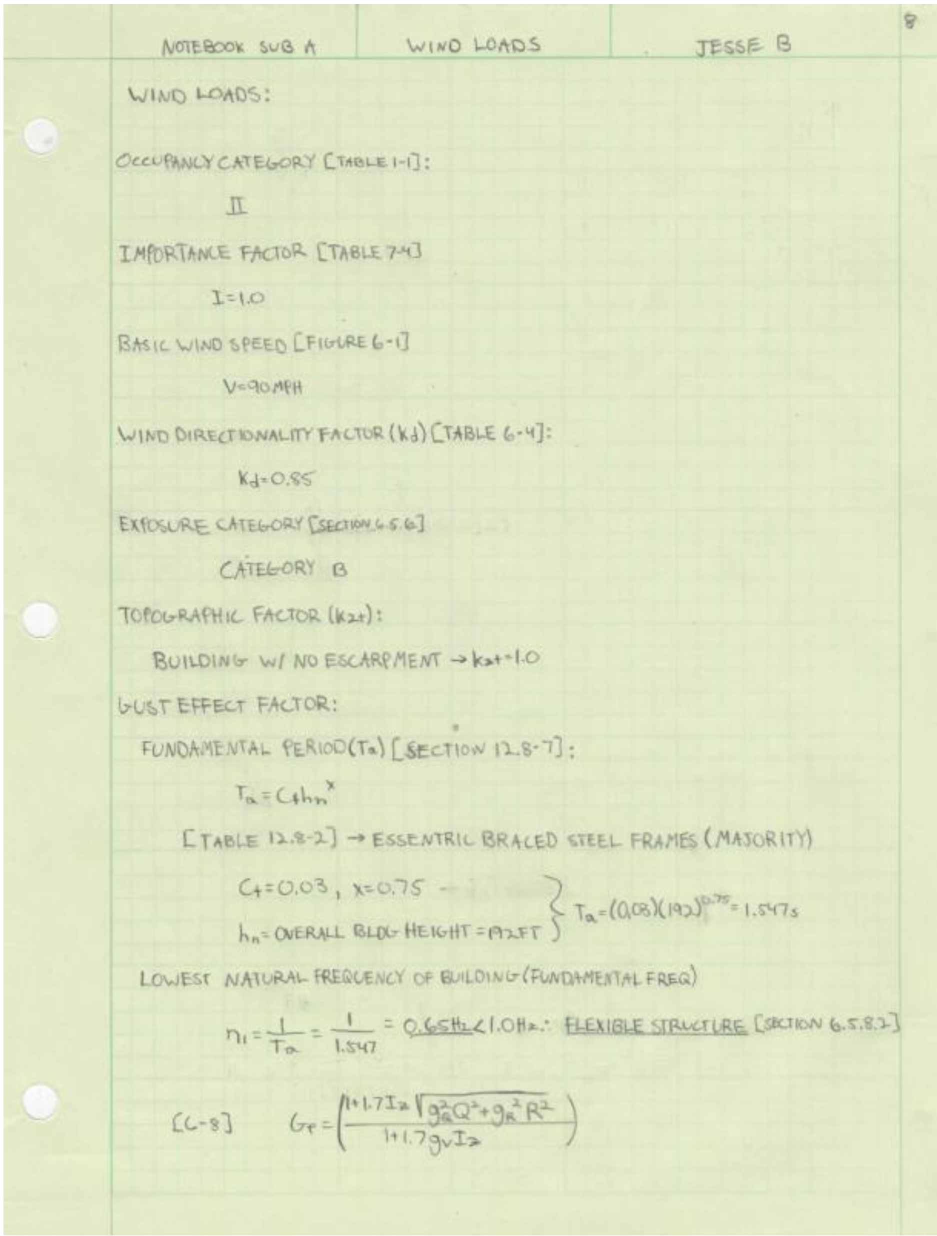
Table 1: Superimposed dead loads



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 and in Appendix A 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]:

$T_a=C_t h_n^x$

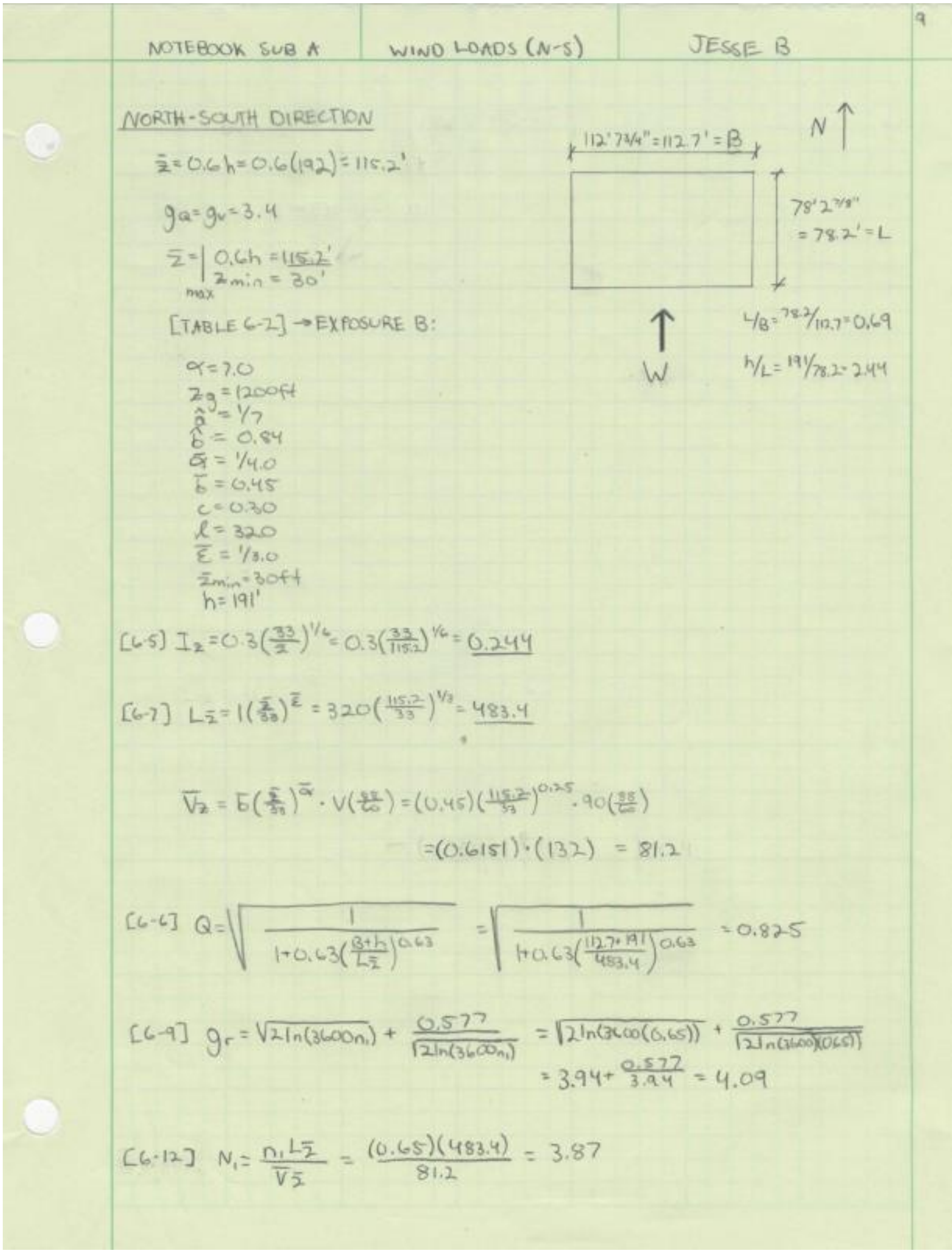
[TABLE 12.8-2] → ESSENTRIC BRACED STEEL FRAMES (MAJORITY)

$C_t=0.03$, $x=0.75$
$h_n=$ OVERALL BLDG HEIGHT $=192$ FT
$T_a=(0.03)(192)^{0.75}=1.547s$

LOWEST NATURAL FREQUENCY OF BUILDING (FUNDAMENTAL FREQ)

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

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



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

NORTH-SOUTH DIRECTION

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

$g_Q = g_v = 3.4$

$\bar{z} = \max \left| \begin{array}{l} 0.6h = \underline{115.2'} \\ z_{min} = 30' \end{array} \right.$

[TABLE 6-2] → EXPOSURE B:

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

112' 7¾" = 112.7' = B

N

78' 2⅞" = 78.2' = L

W

$L/B = 78.2/112.7 = 0.69$

$h/L = 191/78.2 = 2.44$

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

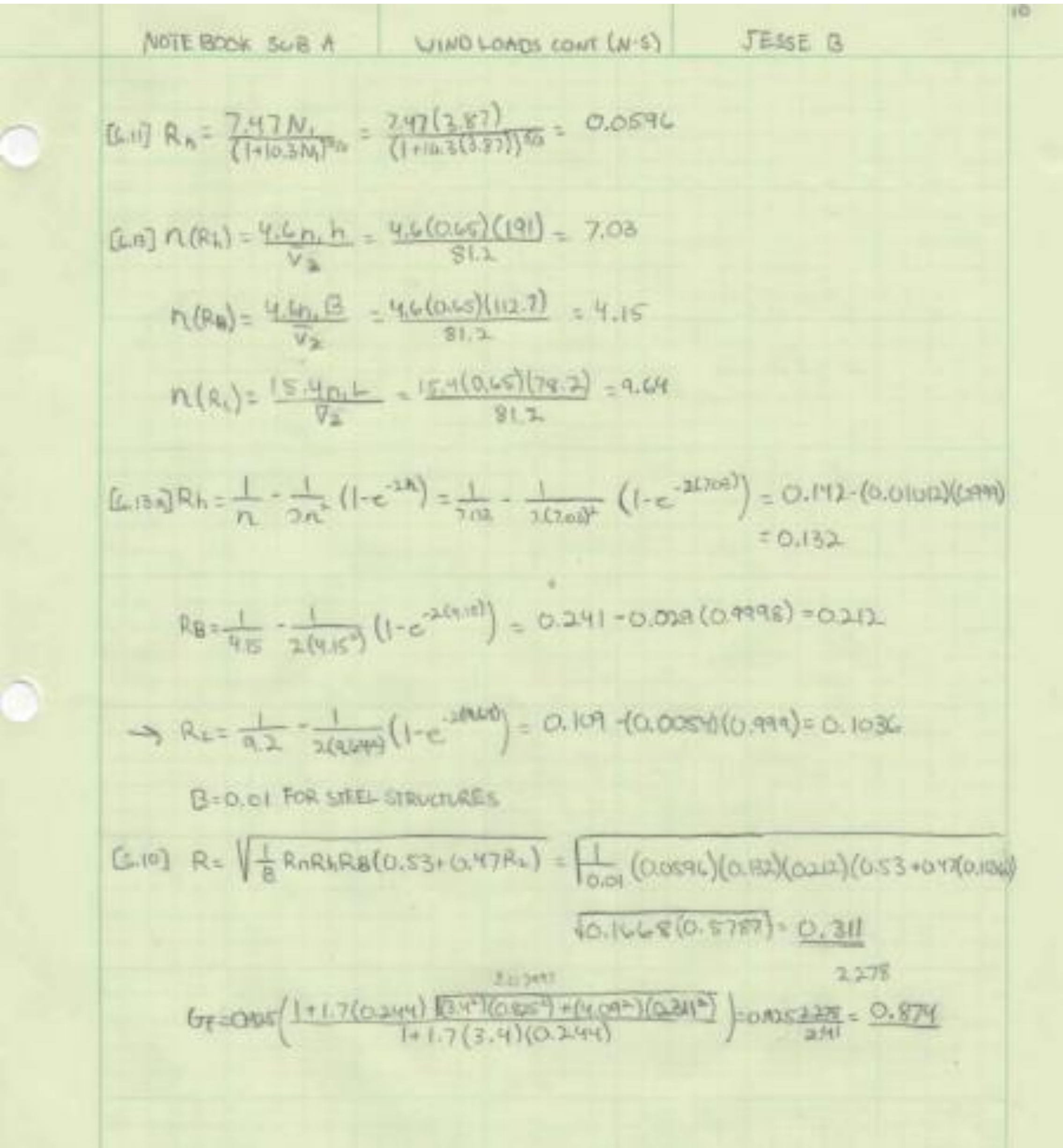
[6-7] $L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{115.2}{33}\right)^{1/3} = \underline{483.4}$

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

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

[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

10

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

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

$n(R_L) = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4(0.65)(78.2)}{81.2} = 9.64$

[6.13a] $R_h = \frac{1}{n} - \frac{1}{2n^2}(1-e^{-2n}) = \frac{1}{7.03} - \frac{1}{2(7.03)^2}\left(1-e^{-2(7.03)}\right) = 0.142-(0.01012)(0.999)$

$= 0.132$

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

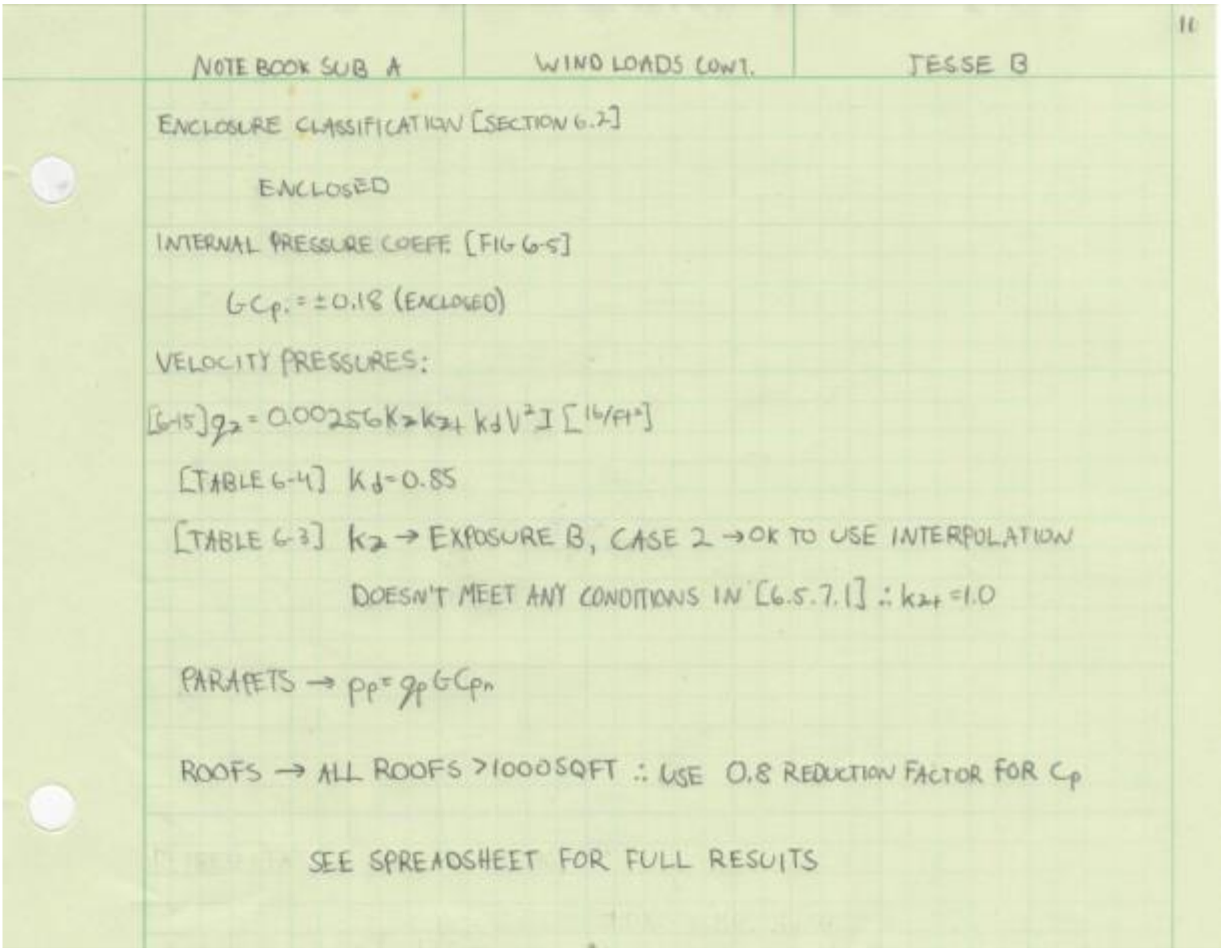
$\rightarrow R_L = \frac{1}{9.2} - \frac{1}{2(9.64^2)}\left(1-e^{-2(9.64)}\right) = 0.109 - (0.0054)(0.999) = 0.1036$

$\beta = 0.01$ FOR STEEL STRUCTURES

[6.10] $R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53+0.47R_L)} = \sqrt{\frac{1}{0.01}(0.0596)(0.132)(0.212)(0.53+0.47(0.1036))}$

$\sqrt{0.1668(0.5787)} = \underline{0.311}$

$G_f = 0.925\left(\frac{1+1.7(0.244)\overset{[illegible]}{\sqrt{(3.4^2)(0.825^2)+(4.09^2)(0.311^2)}}}{1+1.7(3.4)(0.244)}\right) = 0.925\frac{\overset{2.278}{2.278}}{2.41} = \underline{0.874}$



NOTE BOOK SUB A | WIND LOADS CONT. | JESSE B | 11

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/ft²]

[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

NOTEBOOK SUB A | WIND LOADS N-S | JESSE B

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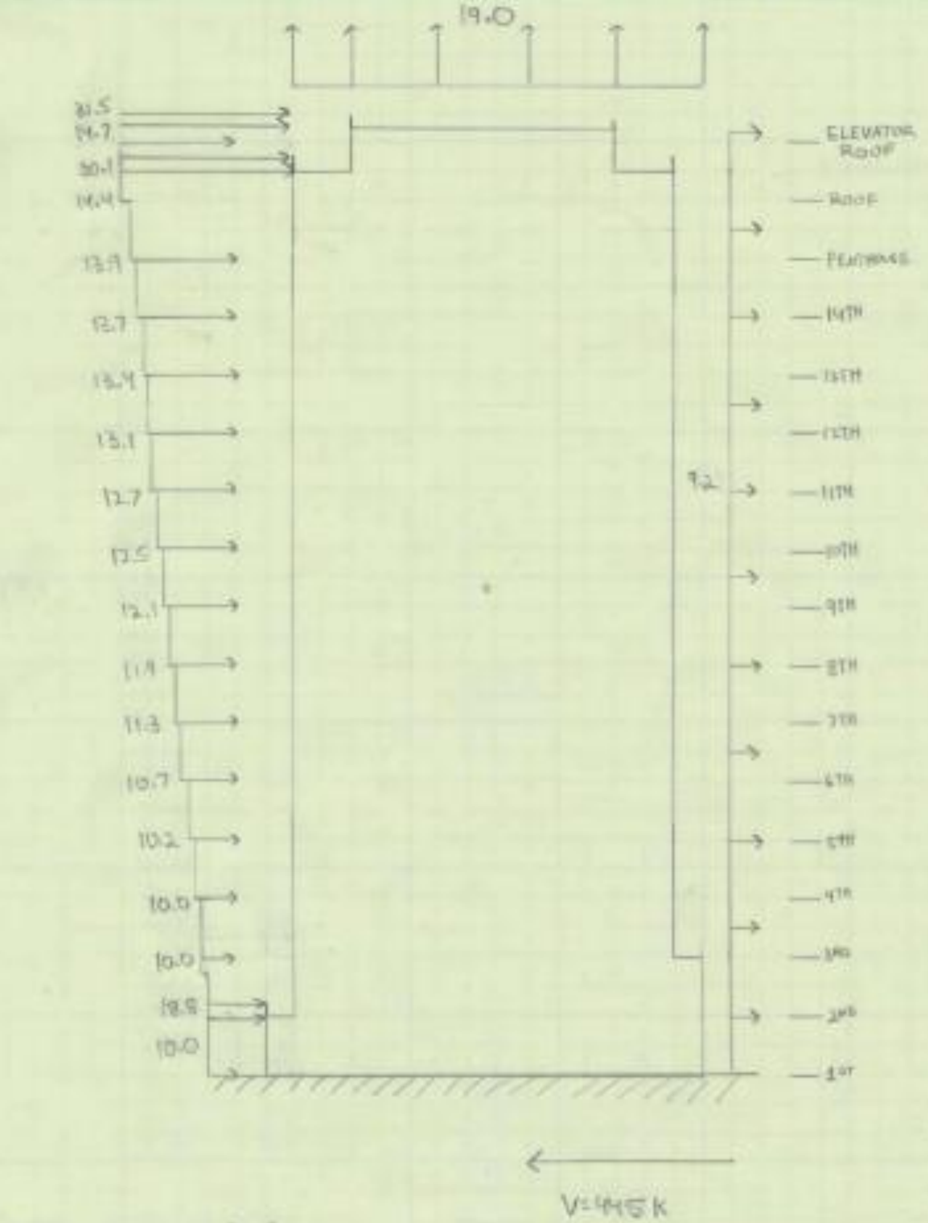


*ALL VALUES IN [PSF]
[6.14.1] ANY WIND LOAD ≤ 10PSF → 10PSF

Table 2: Wind pressures for windward, leeward and uplift displayed for the North-South direction.

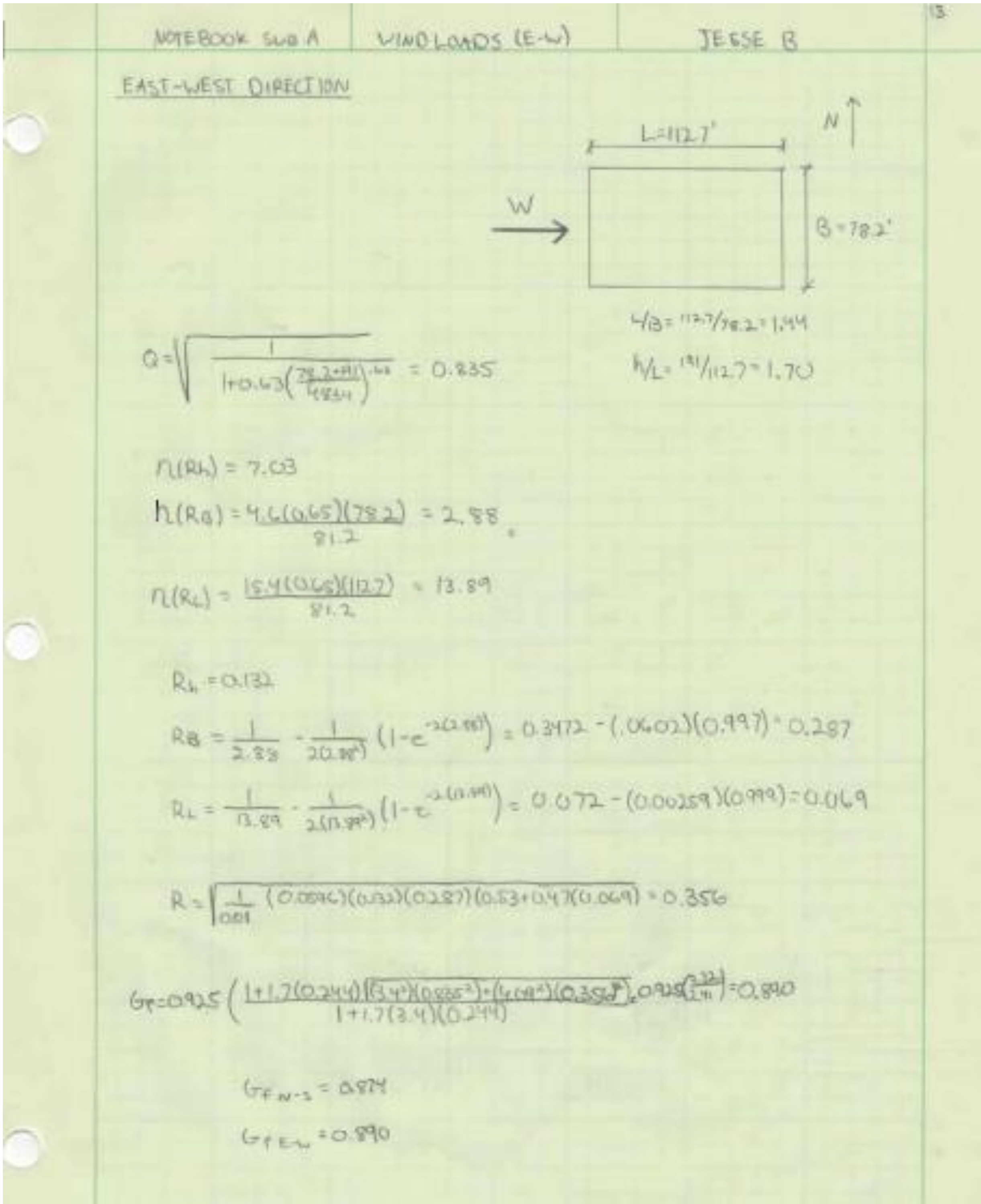
Wind Pressure Determination (N-S)									Net Pressures [psf]	
Location	Story	z [ft]	kz	qz [psf]	Cp	qzGCp [psf]	Gcpi	qhGCpi [psf]	qzGCp-qh(+Gcpi)	qzGCp-qh(-Gcpi)
Windward	1	0	0.57	10.05	0.8	7.02	0.18	3.78	3.25	10.8
	2	15.66	0.57	10.05	0.8	7.02	0.18	3.78	3.25	10.8
	3	33.75	0.72	12.74	0.8	8.91	0.18	3.78	5.13	12.7
	4	44.25	0.78	13.75	0.8	9.61	0.18	3.78	5.84	13.4
	5	54.75	0.83	14.63	0.8	10.23	0.18	3.78	6.45	14.0
	6	65.25	0.87	15.33	0.8	10.72	0.18	3.78	6.95	14.5
	7	75.75	0.91	16.09	0.8	11.25	0.18	3.78	7.48	15.0
	8	86.25	0.96	16.99	0.8	11.88	0.18	3.78	8.10	15.7
	9	96.75	0.98	17.27	0.8	12.08	0.18	3.78	8.30	15.9
	10	107.25	1.01	17.80	0.8	12.45	0.18	3.78	8.67	16.2
	11	117.75	1.03	18.21	0.8	12.73	0.18	3.78	8.96	16.5
	12	128.25	1.06	18.70	0.8	13.08	0.18	3.78	9.30	16.9
	13	138.75	1.09	19.21	0.8	13.43	0.18	3.78	9.66	17.2
	14	149.25	1.11	19.56	0.8	13.68	0.18	3.78	9.90	17.5
	Penthouse Deck	163.00	1.13	19.92	0.8	13.93	0.18	3.78	10.15	17.7
	Penthouse	163.25	1.13	19.92	0.8	13.93	0.18	3.78	10.15	17.7
	Roof	181.00	1.17	20.62	0.8	14.42	0.18	3.78	10.64	18.2
	Elevator Roof	191.02	1.19	20.97	0.8	14.67	0.18	3.78	10.89	18.4
Leeward	All	All	1.19	20.97	-0.5	-9.17	0.18	3.78	-12.94	-5.39
Side	All	All	1.19	20.92	-0.7	-12.80	0.18	3.77	-16.57	-9.03
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	-19.06	0.18	3.78	-22.84	-15.29
	(>95.5ft)	191.02	1.19	20.97	-0.7	-10.13	0.18	3.78	-13.91	-6.36

Table 3: Other factors used for wind determination (N-S direction).

V^2	8100
I	1
kd	0.85
kzt	1
G	0.874
L/B	0.69

Table 4: Wind story shears displayed for the (N-S) direction.

Forces	B	pw+pl	H	Total Force
F1	112.7	16.19	7.83	14.3
F2	112.7	16.19	16.875	30.8
F3	112.7	18.08	14.295	29.1
F4	112.7	18.78	10.5	22.2
F5	112.7	19.39	10.5	23.0
F6	112.7	19.89	10.5	23.5
F7	112.7	20.42	10.5	24.2
F8	112.7	21.05	10.5	24.9
F9	112.7	21.24	10.5	25.1
F10	112.7	21.61	10.5	25.6
F11	112.7	21.90	10.5	25.9
F12	112.7	22.24	10.5	26.3
F13	112.7	22.60	10.5	26.7
F14	112.7	22.85	12.125	31.2
FPD	112.7	23.09	7	18.2
FP	112.7	23.09	9	23.4
FR	112.7	23.58	13.885	36.9
FER	112.7	23.83	5.01	13.5
			[kip]	**444.9**



NOTEBOOK SUB A WIND LOADS (E-W) JESSE B 13

EAST-WEST DIRECTION

$L/B = 112.7/78.2 = 1.44$

$h/L = 191/112.7 = 1.70$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{78.2+191}{4854}\right)^{.63}}} = 0.835$$

$n(R_h) = 7.03$

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

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

$R_h = 0.132$

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

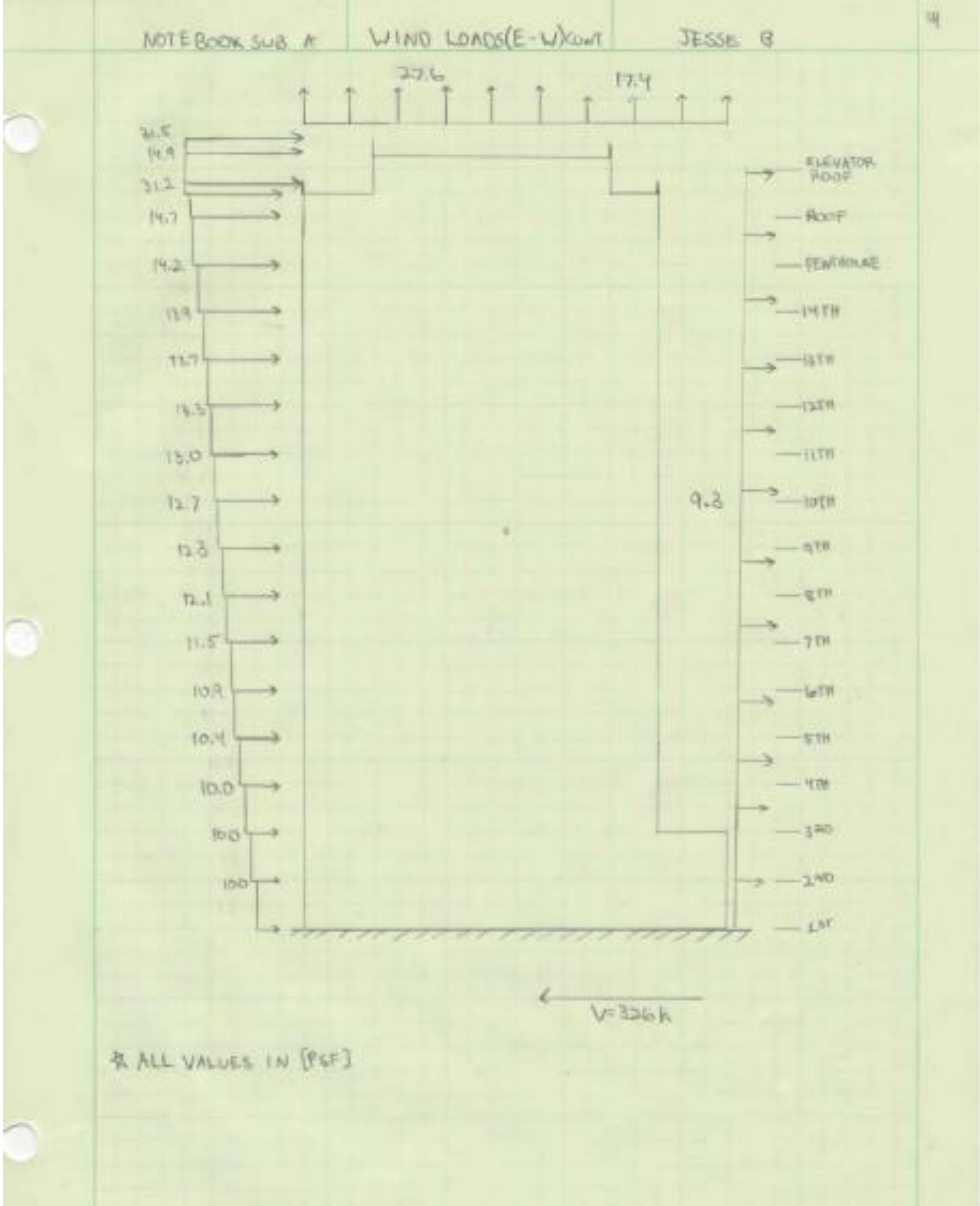
$$R_L = \frac{1}{13.89} - \frac{1}{2(13.89^2)}\left(1-e^{-2(13.89)}\right) = 0.072 - (0.00259)(0.999) = 0.069$$

$$R = \sqrt{\frac{1}{0.01}(0.0096)(0.132)(0.287)(0.53+0.47(0.069))} = 0.356$$

$$G_f = 0.925\left(\frac{1+1.7(0.244)\sqrt{(3.4^2)(0.835^2)+(4.08^2)(0.356^2)}}{1+1.7(3.4)(0.244)}\right) = 0.925\left(\frac{2.32}{2.41}\right) = 0.890$$

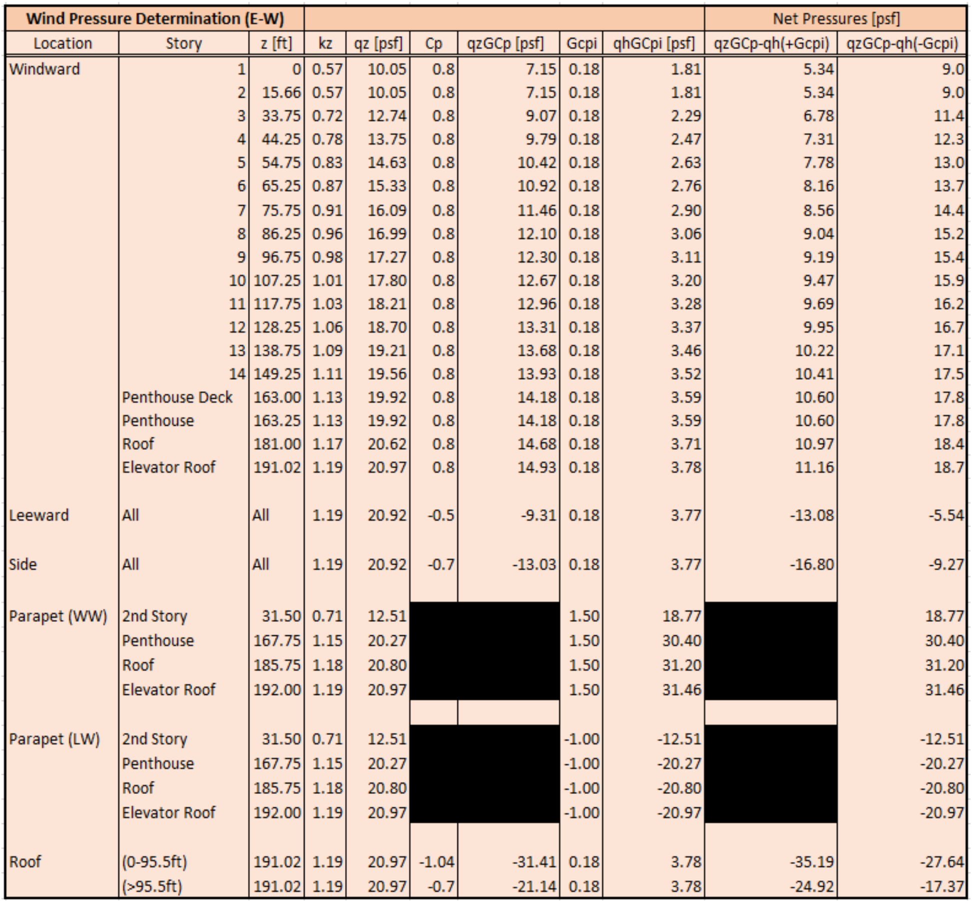
$G_{f\,N-S} = 0.874$

$G_{f\,E-W} = 0.890$



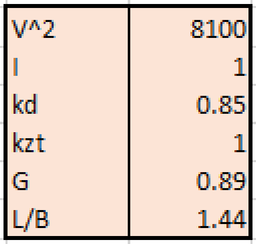
* ALL VALUES IN [PSF]

Table 5: Wind pressures for windward, leeward and uplift displayed for the North-South direction.



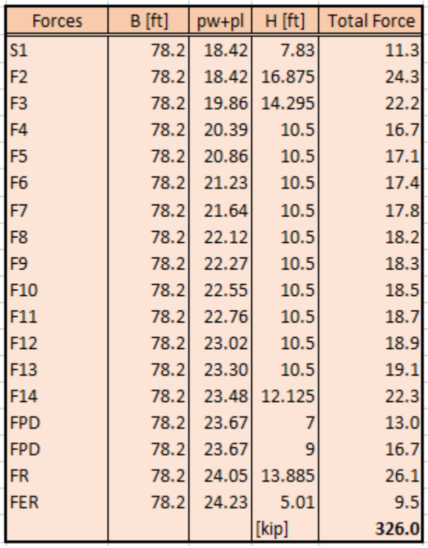
Wind Pressure Determination (E-W)									**Net Pressures [psf]**	
Location	Story	z [ft]	kz	qz [psf]	Cp	qzGCp [psf]	Gcpi	qhGCpi [psf]	qzGCp-qh(+Gcpi)	qzGCp-qh(-Gcpi)
Windward	1	0	0.57	10.05	0.8	7.15	0.18	1.81	5.34	9.0
	2	15.66	0.57	10.05	0.8	7.15	0.18	1.81	5.34	9.0
	3	33.75	0.72	12.74	0.8	9.07	0.18	2.29	6.78	11.4
	4	44.25	0.78	13.75	0.8	9.79	0.18	2.47	7.31	12.3
	5	54.75	0.83	14.63	0.8	10.42	0.18	2.63	7.78	13.0
	6	65.25	0.87	15.33	0.8	10.92	0.18	2.76	8.16	13.7
	7	75.75	0.91	16.09	0.8	11.46	0.18	2.90	8.56	14.4
	8	86.25	0.96	16.99	0.8	12.10	0.18	3.06	9.04	15.2
	9	96.75	0.98	17.27	0.8	12.30	0.18	3.11	9.19	15.4
	10	107.25	1.01	17.80	0.8	12.67	0.18	3.20	9.47	15.9
	11	117.75	1.03	18.21	0.8	12.96	0.18	3.28	9.69	16.2
	12	128.25	1.06	18.70	0.8	13.31	0.18	3.37	9.95	16.7
	13	138.75	1.09	19.21	0.8	13.68	0.18	3.46	10.22	17.1
	14	149.25	1.11	19.56	0.8	13.93	0.18	3.52	10.41	17.5
	Penthouse Deck	163.00	1.13	19.92	0.8	14.18	0.18	3.59	10.60	17.8
	Penthouse	163.25	1.13	19.92	0.8	14.18	0.18	3.59	10.60	17.8
	Roof	181.00	1.17	20.62	0.8	14.68	0.18	3.71	10.97	18.4
	Elevator Roof	191.02	1.19	20.97	0.8	14.93	0.18	3.78	11.16	18.7
Leeward	All	All	1.19	20.92	-0.5	-9.31	0.18	3.77	-13.08	-5.54
Side	All	All	1.19	20.92	-0.7	-13.03	0.18	3.77	-16.80	-9.27
Parapet (WW)	2nd Story	31.50	0.71	12.51	[redacted]	[redacted]	1.50	18.77	[redacted]	18.77
	Penthouse	167.75	1.15	20.27	[redacted]	[redacted]	1.50	30.40	[redacted]	30.40
	Roof	185.75	1.18	20.80	[redacted]	[redacted]	1.50	31.20	[redacted]	31.20
	Elevator Roof	192.00	1.19	20.97	[redacted]	[redacted]	1.50	31.46	[redacted]	31.46
Parapet (LW)	2nd Story	31.50	0.71	12.51	[redacted]	[redacted]	-1.00	-12.51	[redacted]	-12.51
	Penthouse	167.75	1.15	20.27	[redacted]	[redacted]	-1.00	-20.27	[redacted]	-20.27
	Roof	185.75	1.18	20.80	[redacted]	[redacted]	-1.00	-20.80	[redacted]	-20.80
	Elevator Roof	192.00	1.19	20.97	[redacted]	[redacted]	-1.00	-20.97	[redacted]	-20.97
Roof	(0-95.5ft)	191.02	1.19	20.97	-1.04	-31.41	0.18	3.78	-35.19	-27.64
	(>95.5ft)	191.02	1.19	20.97	-0.7	-21.14	0.18	3.78	-24.92	-17.37

Table 6: Other factors used for wind determination (E-W direction).



V^2	8100
I	1
kd	0.85
kzt	1
G	0.89
L/B	1.44

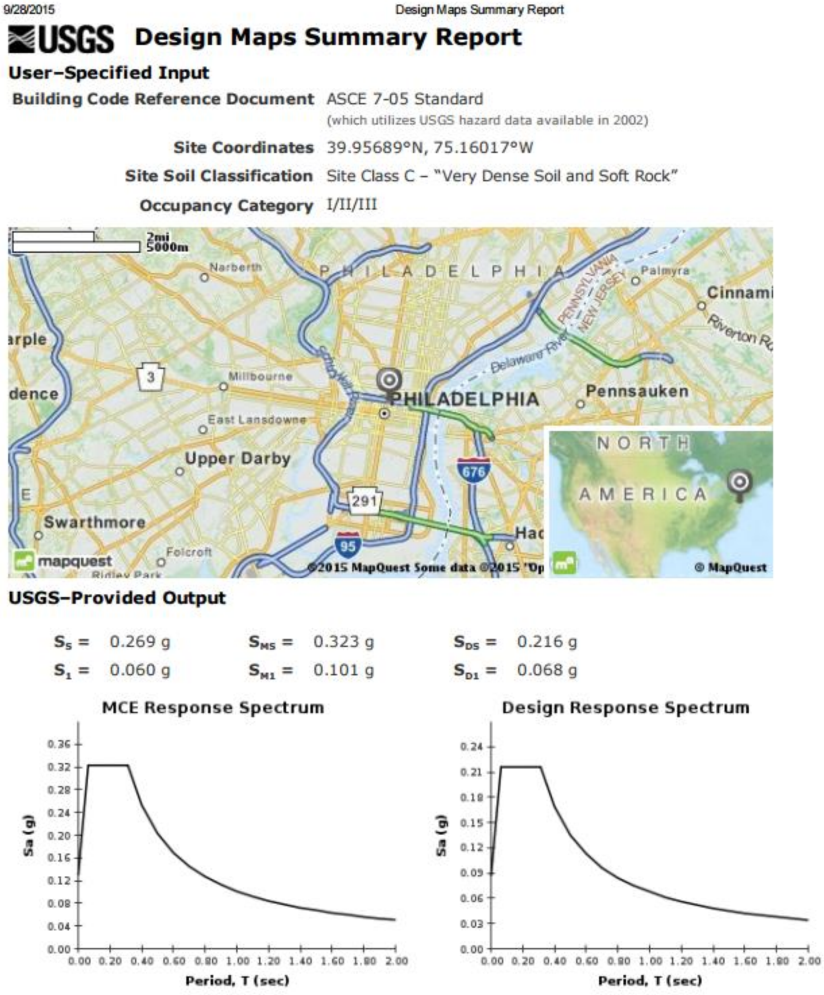
Table 7: Wind story shears displayed for the (E-W) direction.



Forces	B [ft]	pw+pl	H [ft]	Total Force
S1	78.2	18.42	7.83	11.3
F2	78.2	18.42	16.875	24.3
F3	78.2	19.86	14.295	22.2
F4	78.2	20.39	10.5	16.7
F5	78.2	20.86	10.5	17.1
F6	78.2	21.23	10.5	17.4
F7	78.2	21.64	10.5	17.8
F8	78.2	22.12	10.5	18.2
F9	78.2	22.27	10.5	18.3
F10	78.2	22.55	10.5	18.5
F11	78.2	22.76	10.5	18.7
F12	78.2	23.02	10.5	18.9
F13	78.2	23.30	10.5	19.1
F14	78.2	23.48	12.125	22.3
FPD	78.2	23.67	7	13.0
FPD	78.2	23.67	9	16.7
FR	78.2	24.05	13.885	26.1
FER	78.2	24.23	5.01	9.5
			[kip]	**326.0**

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

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

1

NOTEBOOK SUB A | SEISMIC LOADS | JESSE B

15

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

$R = 3.25$
$\Omega = 2$
$C_d = 3.0$

[TABLE 11.5-1] $I_e = 1.0$, $C_u = 1.7$ [TABLE 12.8-1]

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

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

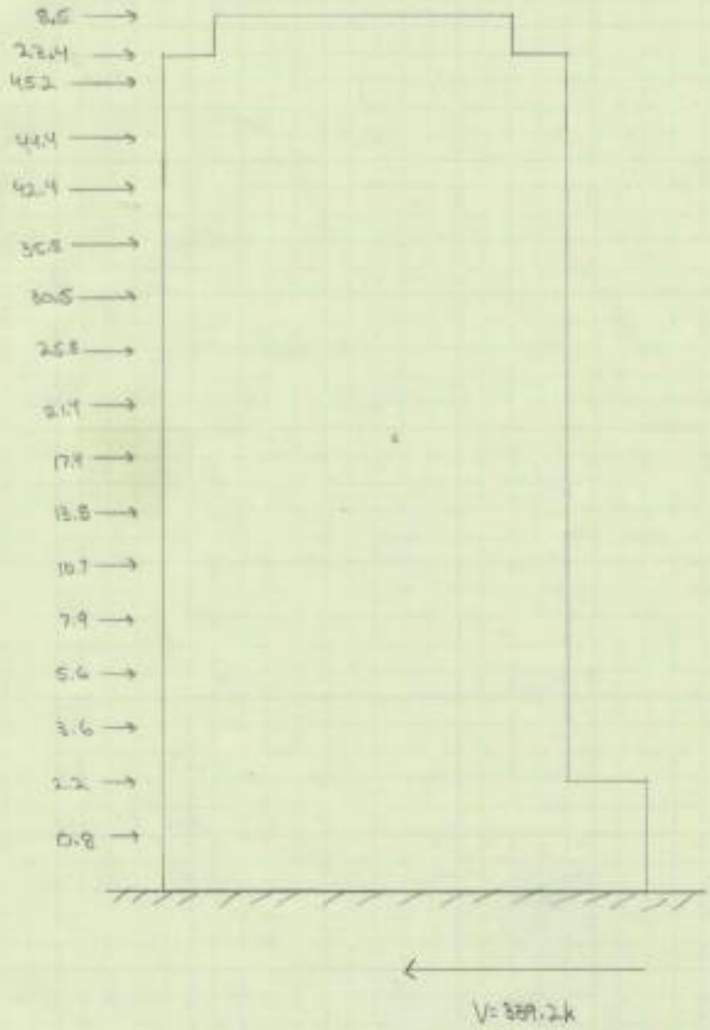
[FIGURE 22-15] $T_L = 6s \rightarrow T_a < T_L \therefore \underset{[12.8-3]}{C_s} = \frac{S_{DS}}{T\left(\frac{R}{I_e}\right)} = \frac{0.216}{1.54\left(\frac{3.25}{1}\right)} = \overset{0.029}{\underline{0.047}} > 0.01 \checkmark \overset{eqn}{[12.8-2]}$

[EQN 12.8-1] $V = C_s W = (\overset{0.029}{\cancel{0.043}})(11695) = 339.1k$

[EQN 12.8-11] $F_x = C_{vx} V$

SEE SPREADSHEET FOR COMPLETE RESULTS

NOTEBOOK SUB A | SEISMIC LOADS | JESSE B | 16



* ALL VALUES IN [k]

LOADS REMAIN THE SAME
IN BOTH DIRECTIONS

Table 6: Floor-by-floor breakdown of total building mass for AC Hotel Philadelphia.

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	95		7.83	381.8	27.5	846961.1			846961
2	9770	95		16.88	381.8	27.5	1105382	1020	44928	1151330
3	5925	95		14.3	381.8	27.5	713017.9	1909		714927
4	5925	95		10.5	381.8	27.5	673119.8			673120
5	5925	95		10.5	381.8	27.5	673119.8			673120
6	5925	95		10.5	381.8	27.5	673119.8			673120
7	5925	95		10.5	381.8	27.5	673119.8			673120
8	5925	95		10.5	381.8	27.5	673119.8			673120
9	5925	95		10.5	381.8	27.5	673119.8			673120
10	5925	95		10.5	381.8	27.5	673119.8			673120
11	5925	95		10.5	381.8	27.5	673119.8			673120
12	5925	95		10.5	381.8	27.5	673119.8			673120
13	5925	95		10.5	381.8	27.5	673119.8			673120
14	5925	95		12.125	381.8	27.5	690181.4			690181
Penthouse Deck	5925	95		7	381.8	15	602964			602964
Penthouse	5925	95		9	381.8	15	614418			614418
Roof	1250	138	3.6	13.87	328	15	245240.4	11480	2000	258720
Elevator Roof	400	68	3.6	5.01	68	15	33750.2	680	50000	84430
*included 3psf for weight of steel and extra allowances for pool, mech equip & fitness							Total Building Weight [kips]			**11695**

Table 7: Seismic story shears displayed for both orthogonal directions.

Story	hx [ft]	wx [kip]	wxhx^k	Cvx	Fx	Vx
Elevator Roof	191.0	84.4	3079641.2	0.03	8.5	8.5
Roof	181.0	258.7	8475270.7	0.07	23.4	31.9
Penthouse	163.3	614.4	16374105.6	0.13	45.2	77.1
Penthouse Deck	163.0	603.0	16021107.0	0.13	44.2	121.3
14	149.3	690.0	15370138.1	0.13	42.4	163.7
13	138.8	673.1	12958226.7	0.11	35.8	199.4
12	128.3	673.1	11071190.9	0.09	30.5	230.0
11	117.8	673.1	9332573.6	0.08	25.8	255.7
10	107.3	673.1	7742374.8	0.06	21.4	277.1
9	96.8	673.1	6300594.6	0.05	17.4	294.5
8	86.3	673.1	5007233.0	0.04	13.8	308.3
7	75.8	673.1	3862289.9	0.03	10.7	319.0
6	65.3	673.1	2865765.3	0.02	7.9	326.9
5	54.8	673.1	2017659.3	0.02	5.6	332.4
4	44.3	673.1	1317971.9	0.01	3.6	336.1
3	33.8	714.9	814315.8	0.01	2.2	338.3
2	15.7	1151.3	282339.7	0.00	0.8	339.1
1	0.0	847.0	0.0	0.00	0.0	339.1
	Σ	11694.7	122892798.1	1.0		
*k=2 b/c period is >0.5s						
V=	339.1					

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.

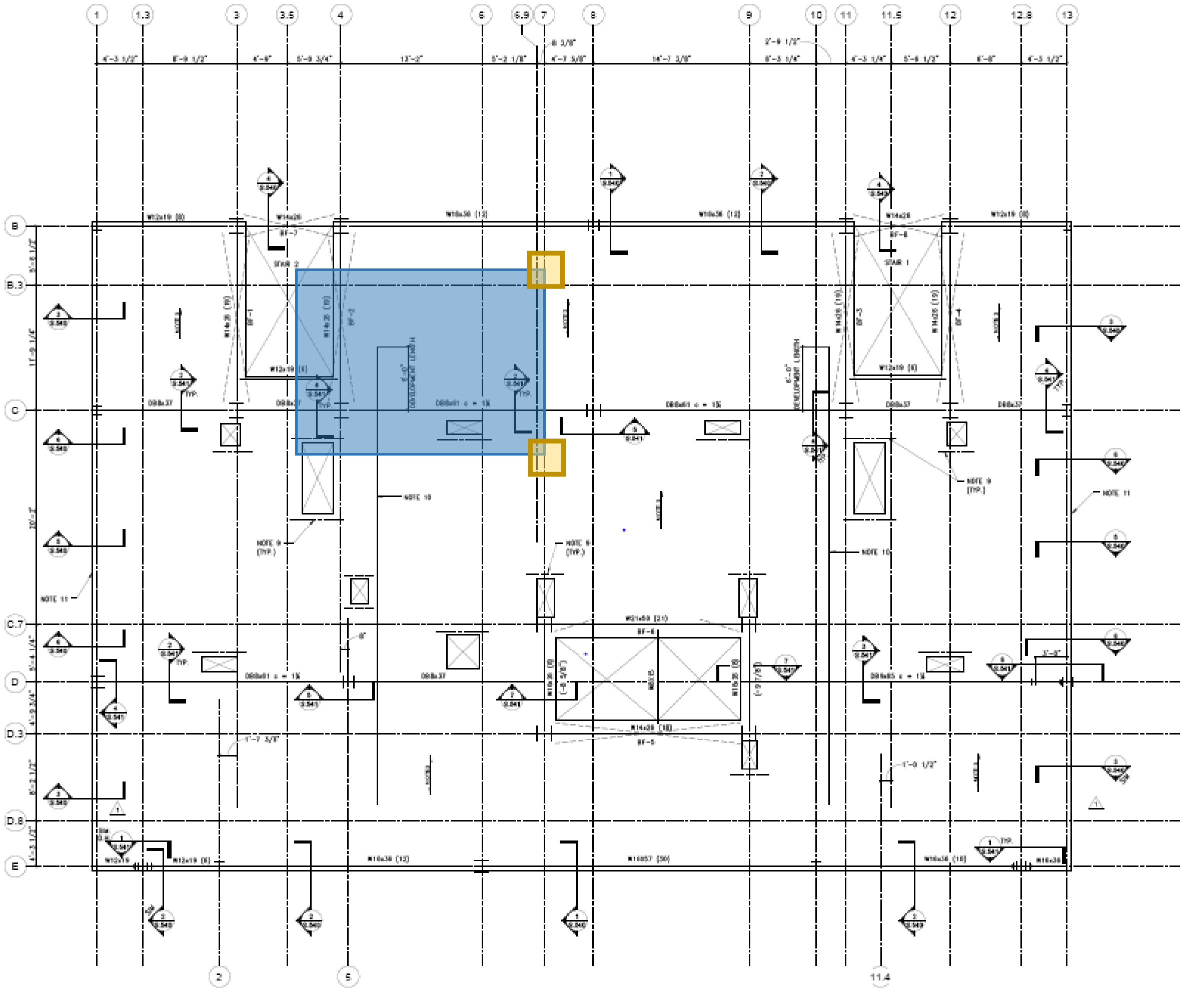


Figure 4: Typical floor for AC Hotel Philadelphia shown above (approx. 94'x63').

🟦 Typical Bay
(4-8, B-C)

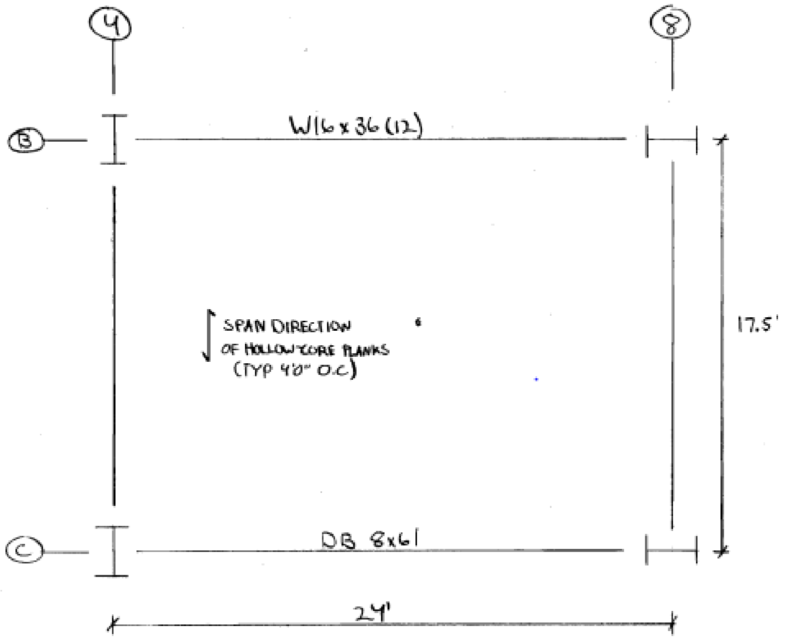
🟨 Columns under consideration
(B8 & C8)

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

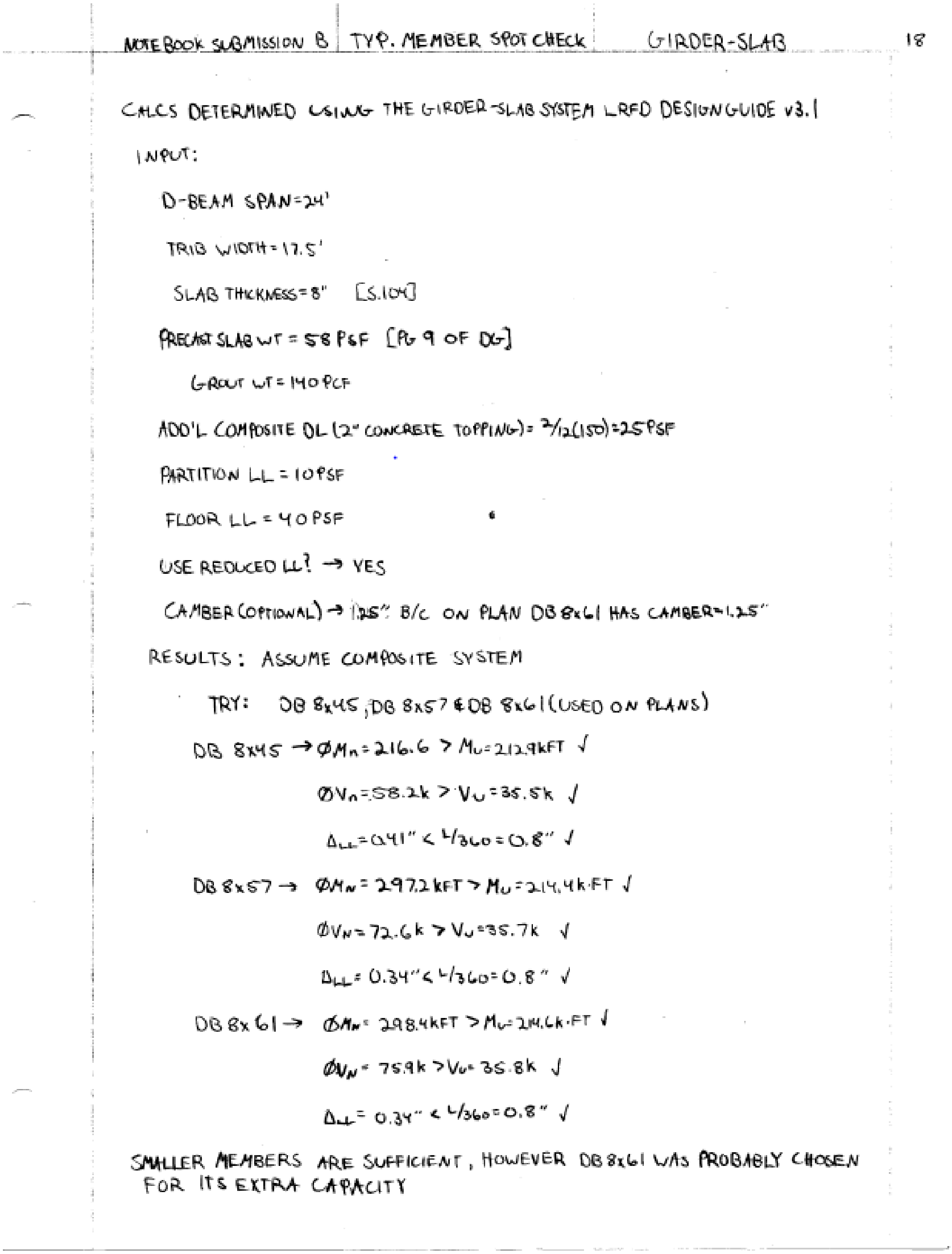
TRIB WIDTH WAS DETERMINED USING TYP. BAY SIZE = $\frac{17.5+17.5}{2}$ = 17.5'



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

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

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



NOTEBOOK SUBMISSION B | TYP. MEMBER SPOT CHECK | GIRDER-SLAB 18

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) = $\frac{2}{12}(150) = 25$ 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 ✓

$\phi V_N = 72.6k > V_u = 35.7k$ ✓

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

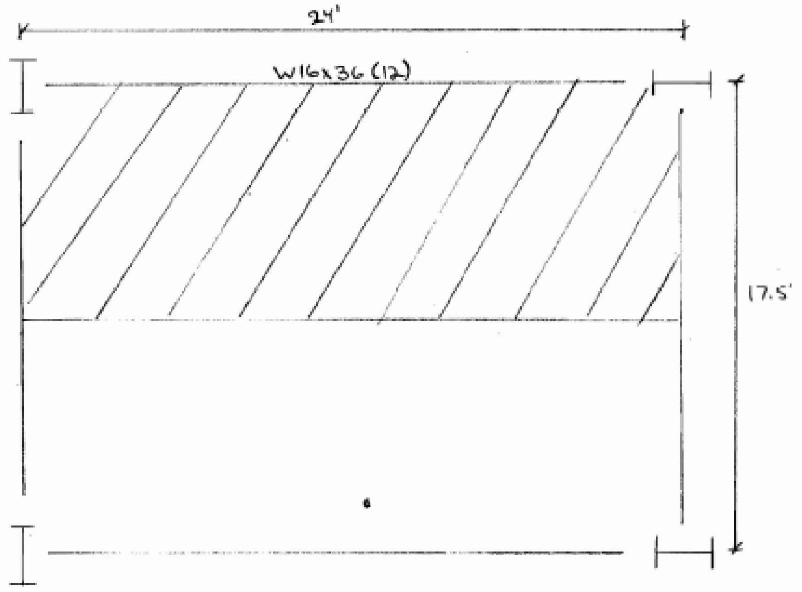
DB 8x61 → $\phi M_N = 298.4$ kFT $> M_u = 214.6$ k·FT ✓

$\phi V_N = 75.9k > V_u = 35.8k$ ✓

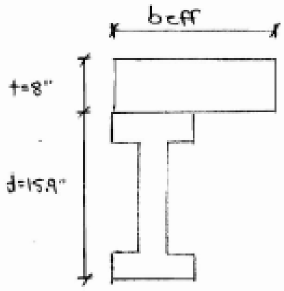
$\Delta_{LL} = 0.34" < L/360 = 0.8"$ ✓

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 = $\frac{24\times17.5}{2}$ = 210FT²



$$b_{EFF} = \min \begin{cases} SPAN/8 = \frac{24\times12}{8} = 36" \leftarrow \text{CONTROLS} \\ SPACING/2 = \frac{12.5\times12}{2} = 105" \end{cases}$$

[S.503] SHEAR STUDS → 3/4"ø, 5"

[S.001] F'c=4500PSI FOR FRAMED S.O.G.

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

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

[TABLE 1-1] FOR W16x36: d=15.9", A_s=10.6in², b_f=7.0"

DETERMINE MAX MOMENT:

$DL = 112\ PSF$

FLOOR-FLOOR ↓

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

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

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

$$\begin{aligned} W_u &= 1.2DL + 1.6LL \\ &= 1.2(112) + 1.6(100) \\ &= 294\ PSF\left(\tfrac{17.5}{2}\right)' \\ &= 2576\ PLF + 1.2(289) \\ &= 2923\ PLF = 2.92\ KLF \end{aligned}$$

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

DETERMINE MOMENT CAPACITY:

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

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

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

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

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

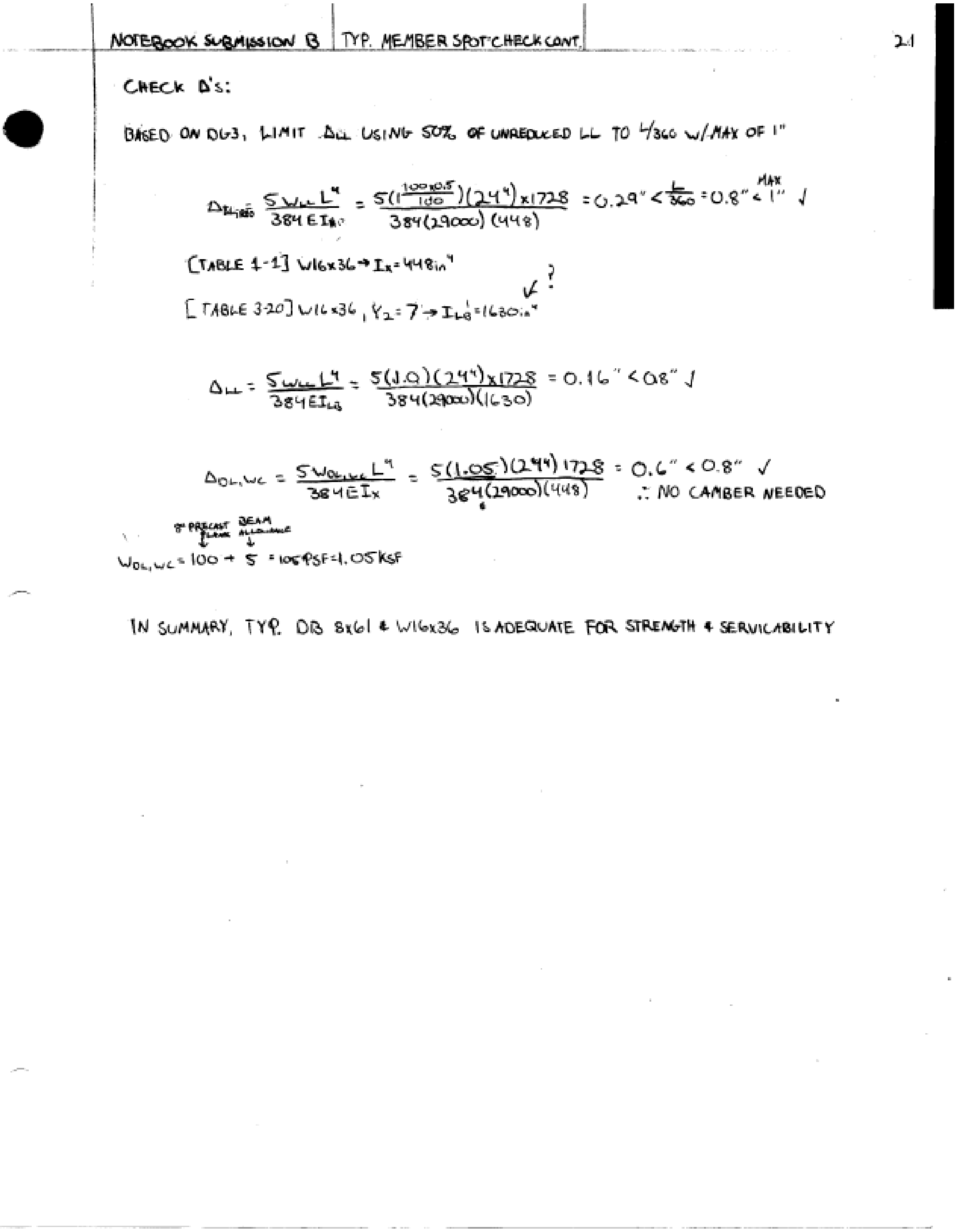
$$Y_2 = t - \frac{a}{2} = 8 - \frac{0.94}{2} = 7.53 \therefore \text{PNA IN CONCRETE}$$

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

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

$$\begin{aligned} M_N &= T_s\left(\tfrac{d}{2}\right) + \Sigma Q_N\left(t - \tfrac{a}{2}\right) - 2F_y \cdot b_f \cdot x\left(\tfrac{x}{2}\right) \\ &= 530\left(\tfrac{15.9}{2}\right) + 129(7.53) - 2(50)(7)(0.57)\left(\tfrac{0.57}{2}\right) \\ &= 423\ k\cdot FT \end{aligned}$$

$\phi M_N = 0.9(423) = 381\ k\cdot FT > 210\ k\cdot FT \therefore$ 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"

$$\Delta_{LL,50\%} = \frac{5w_{LL}L^4}{384EI_{x}} = \frac{5(1\frac{100\times0.5}{100})(24^4)\times1728}{384(29000)(448)} = 0.29" < \frac{L}{360} = 0.8" \overset{MAX}{<} 1" \checkmark$$

[TABLE 1-1] W16x36 → $I_x = 448in^4$

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

$$\Delta_{LL} = \frac{5w_{LL}L^4}{384EI_{LB}} = \frac{5(1.0)(24^4)\times1728}{384(29000)(1630)} = 0.16" < 0.8" \checkmark$$

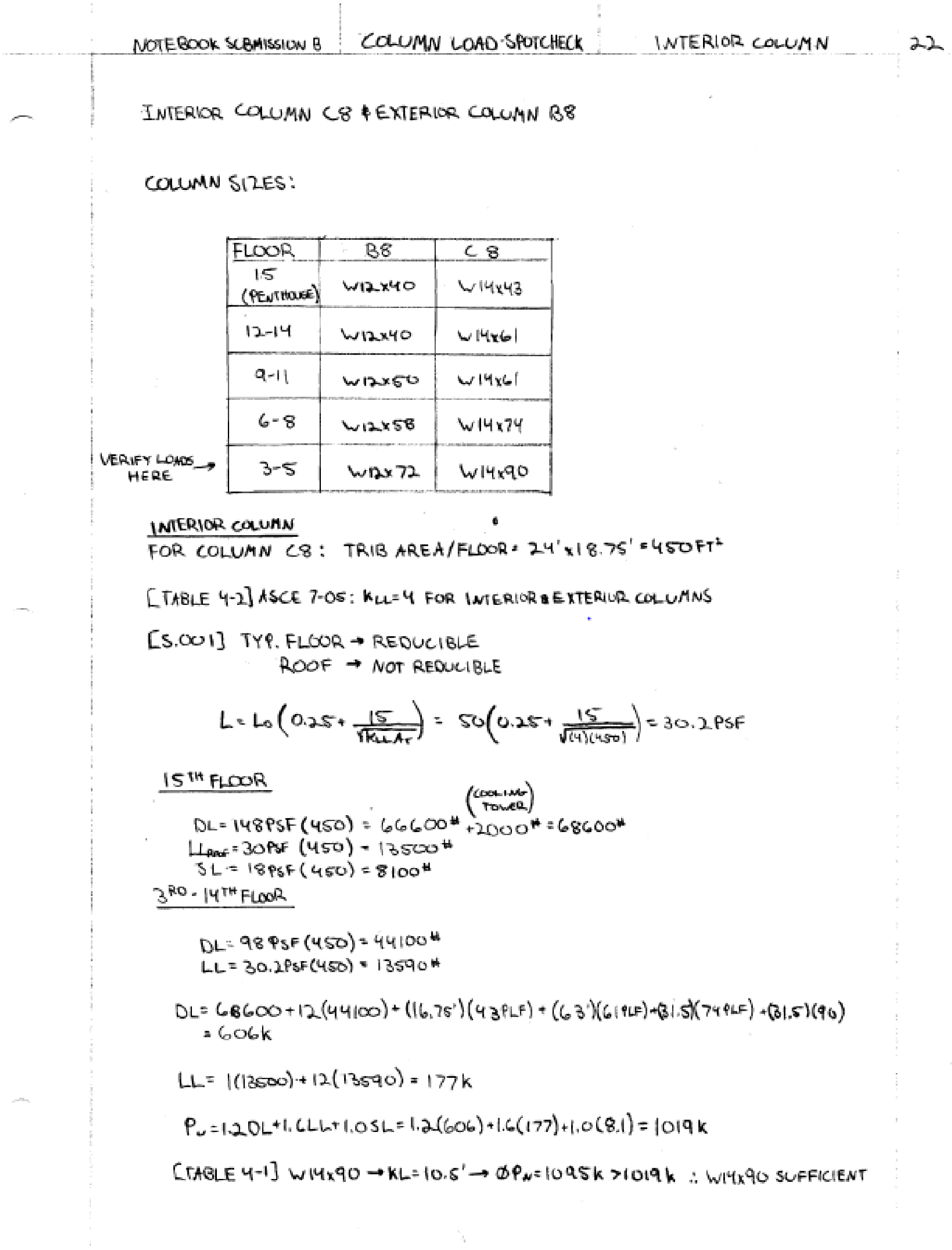
$$\Delta_{DL,WC} = \frac{5w_{DL,WC}L^4}{384EI_x} = \frac{5(1.05)(24^4)1728}{384(29000)(448)} = 0.6" < 0.8" \checkmark$$

∴ NO CAMBER NEEDED

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

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 →

INTERIOR COLUMN

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

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

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

$$L = L_0\left(0.25+\frac{15}{\sqrt{K_{LL}A_T}}\right) = 50\left(0.25+\frac{15}{\sqrt{(4)(450)}}\right) = 30.2\text{PSF}$$

15TH FLOOR

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

3RD - 14TH FLOOR

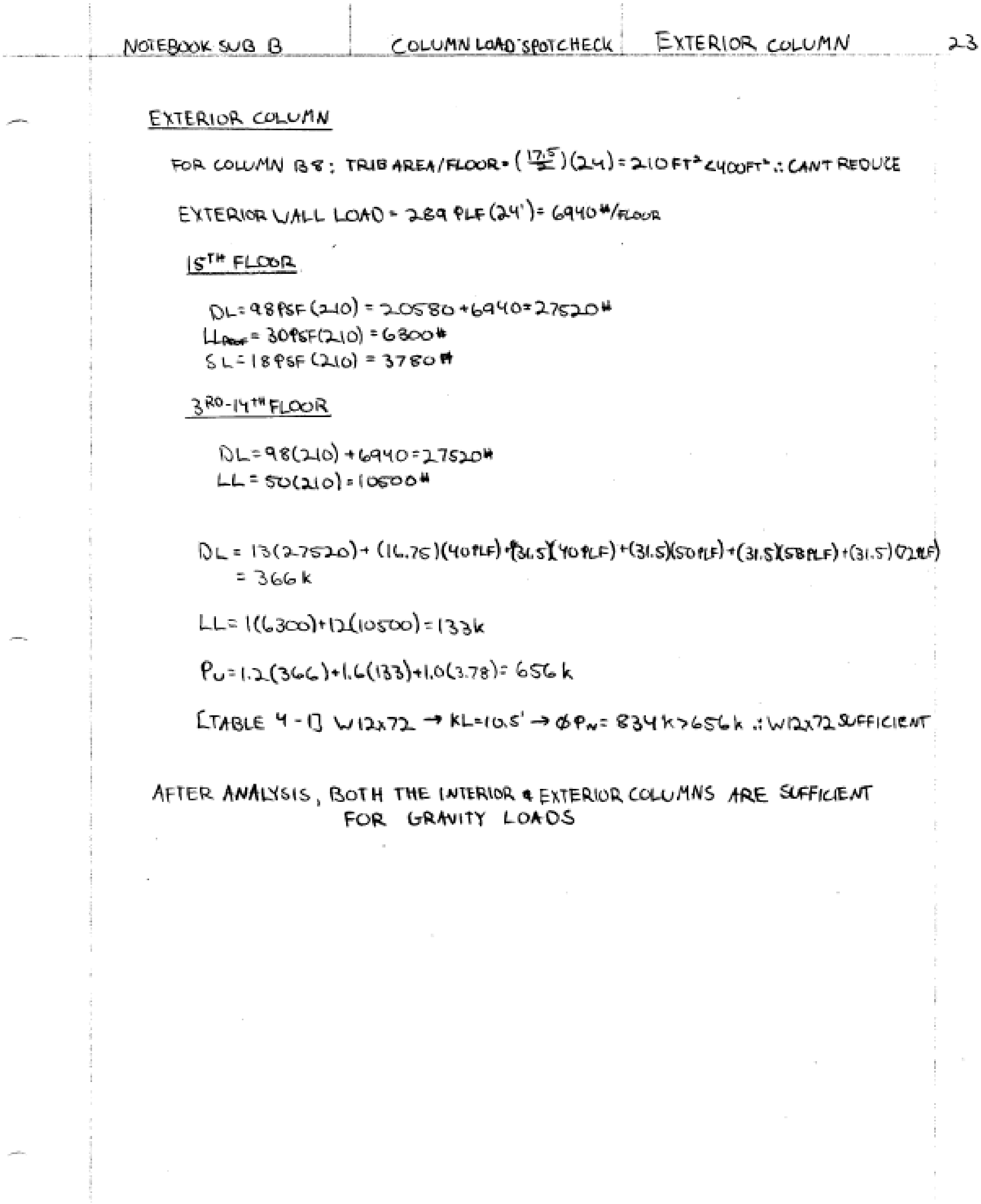
$DL = 98\text{PSF}(450) = 44100\#$
$LL = 30.2\text{PSF}(450) = 13590\#$

$DL = 68600+12(44100)+(16.75')(43\text{PLF})+(63')(61\text{PLF})+(31.5')(74\text{PLF})+(31.5')(90)$
$= 606k$

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

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

[TABLE 4-1] W14x90 → KL = 10.5' → $\phi P_n = 1095k > 1019k$ ∴ W14x90 SUFFICIENT



NOTEBOOK SUB B | COLUMN LOAD SPOTCHECK | EXTERIOR COLUMN | 23

EXTERIOR COLUMN

FOR COLUMN B8: TRIB AREA/FLOOR = $(\frac{17.5}{2})(24) = 210 FT^2 < 400 FT^2$ ∴ 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) = 133k

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

[TABLE 4-1] W12x72 → KL = 10.5' → ϕP_n = 834 k > 656 k ∴ W12x72 SUFFICIENT

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

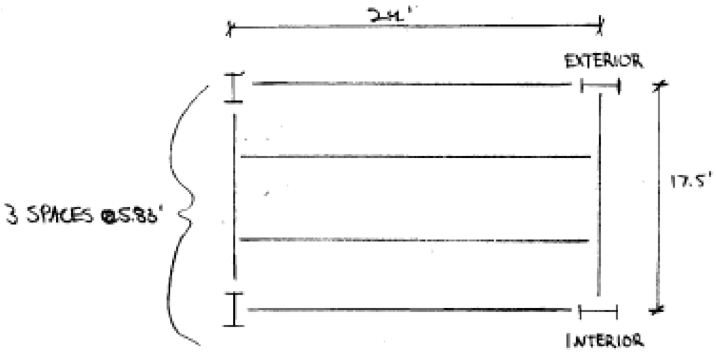
Alternative Systems

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 → $\phi M_n = 75.4\ FTK > 63.4\ FTK$ √
(ECONOMICAL)

$I_x = 103\ in^4$

CHECK Δ's:

LIMIT LIVE LOAD TO L/360 = 0.8"

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

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

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EXTERIOR : $DL_{WALL} = 289$ PLF

$W_U = 1.2(59) + 1.6(50) = 151$ PSF $\left(\frac{5.84}{2}\right) = 441$ PLF $+ 1.2(289 \text{PLF}) = 0.79$ KLF

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

CHECK MEMBER SELF-WEIGHT:

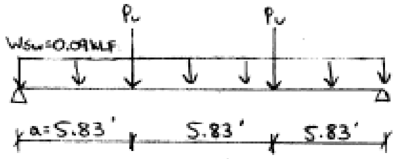
W12x16 → $\frac{16\text{ PLF}}{5.83\text{ FT}}$ = 2.8 < 5 PSF ALLOWANCE ∴ USE W12x16 @ 5.83' O.C.

SELECT GIRDER:

SPAN = 17.5'
SPACING = 24'

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

$\frac{1}{2}$ LOAD FROM ADJ BAYS



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

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

$$M_{MAX} = (P_u \cdot a) + \frac{w\ell^2}{8}$$

$$= (21)(5.83) + \frac{0.09(17.5^2)}{8}$$

$$= 126\text{ K}\cdot\text{FT}$$

TRY W12x26: I_x = 204 in^4, $\varnothing M_n$ = 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

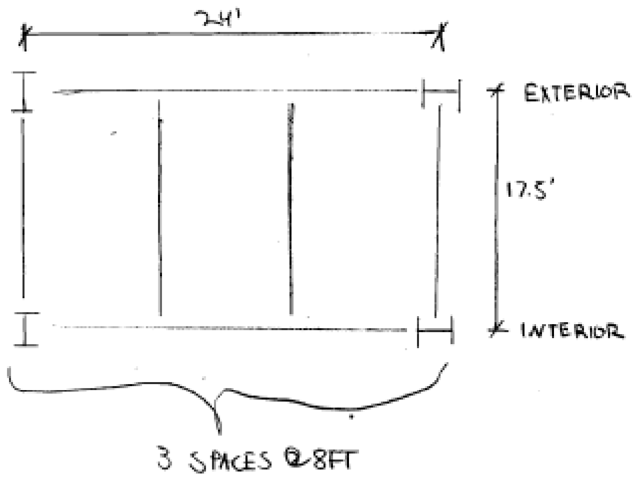
MEMBER SW: $^{26}/_{17.5}$ = 1.49 < 5 ∴ GIRDER SW IS CONSERVATIVE

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'
SPACING = 8'

BEAMS SPAN SHORTER DIRECTION

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)
$I_x = 53.8\,in^4$

$L/360 = \frac{17.5(12)}{360} = 0.583''$

CHECK Δ's:

$$\Delta_{LL} = \frac{5wL^4(172)}{384EI_x} = \frac{5(0.40)(17.5^4)(1728)}{384(29000)(53.8)} = 0.54'' < 0.58'' \therefore \text{ok} \checkmark$$

CHECK MEMBER SW:

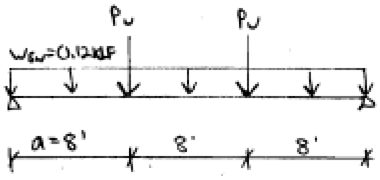
W10x12 → $\frac{12}{8} = 1.5 < 5\,\text{PSF}$ ALLOWANCE ∴ USE W10x12 @8' O.C

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

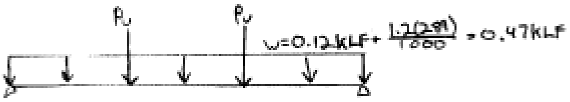
$$\Delta_{LL} = \frac{[(50)(8)(17.5)]/1000 \ (24^3)(1728)}{28(29000)(291)} = 0.71''$$

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

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

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

CHECK EXTERIOR GIRDER FOR ADD'L WALL LOAD



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

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

NOTEBOOK SUB B ALT SYSTEM 1 NONCOMPOSITE 29

DETERMINE OPTIMAL BEAM CONFIGURATION:

WEIGHT OF STEEL

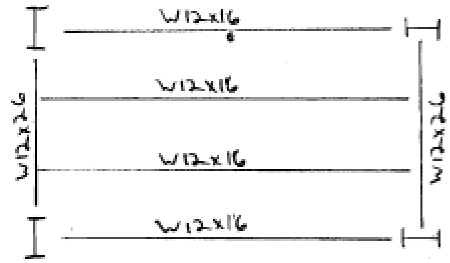
BEAMS LONG DIRECTION:

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

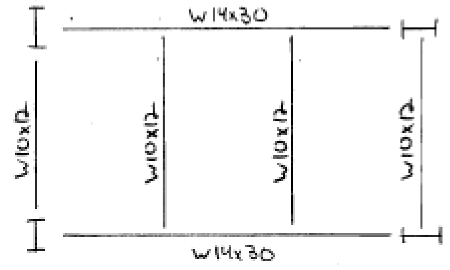
BEAMS SHORT DIRECTION:

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

OPTION 1:



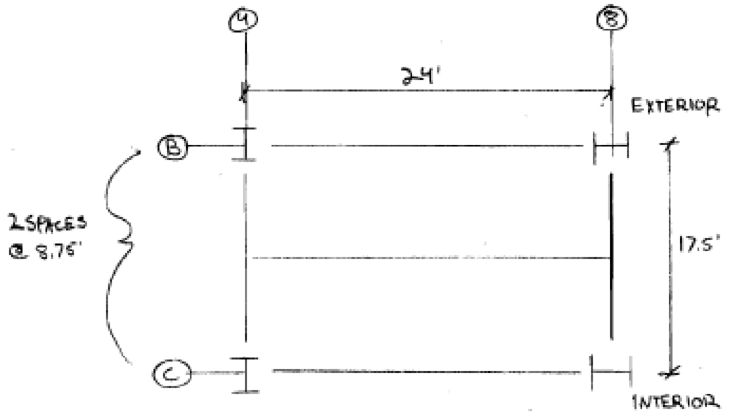
OPTION 2:



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

NOTEBOOK SUB B | ALTERNATIVE SYSTEM #2 | COMPOSITE STEEL FRAMING 30



BEAM: SPAN=24'
SPACING = 8.75'

FROM VULCRAFT CATALOG:

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

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

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

INTERIOR $W_U = 1.2DL+1.6LL = 1.2(56)+1.6(50) = 147PSF(8.75') = 1.29$ KLF

$$M_U = \frac{wL^2}{8} = \frac{(1.29)(24^2)}{8} = 93 \text{ K·FT}$$

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

[TABLE 3-21] DECK ⊥, NWC, $f'_c = 4KSI$, 3/4" Ø STUD → 1 STUD/RIB = 17.2K
2 STUD/RIB = 14.6K

NOTEBOOK SUB B ALT. SYSTEM 2 COMPOSITE 31

DETERMINE POSSIBLE BEAM SIZES: [TABLE 3-19]

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

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

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

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

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

CHECK ECONOMY

$W10x12 \rightarrow 12(24) + 14(10) = \underline{428}\#$ ∴ CONTINUE W/ W10X12

$W10x15 \rightarrow 15(24) + 10(10) = 460\#$

$W10x17 \rightarrow 17(24) + 8(10) = 488\#$

$W10x19 \rightarrow 19(24) + 10(10) = 556\#$

$W10x22 \rightarrow 22(24) + 10(10) = 628\#$

CHECK ASSUMPTION:

$$b_{eff} = \min \left| \begin{array}{l} SPAN/8 = 24(12)/8 = \underline{36} \\ SPACING/2 = 8.75(12)/2 = 52.5 \end{array} \right\} \text{DO NOT} \times 2 \text{ B/C EXTERIOR BAY}$$

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

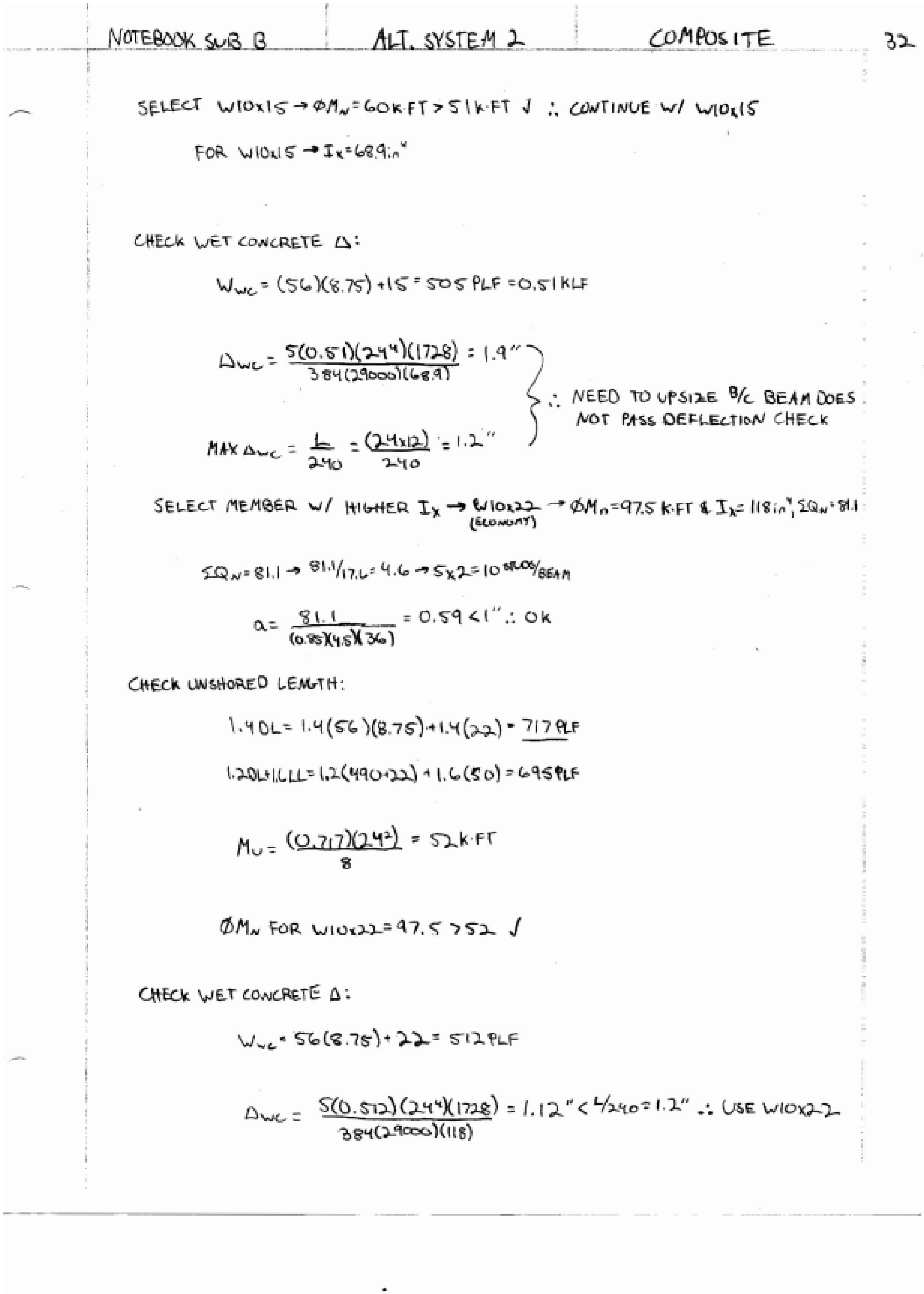
CHECK UNSHORED LENGTH:

SELECT CONTROLLING LOAD CASE

$$\left. \begin{array}{l} 1.4DL = 1.4\overbrace{(56)(8.75)}^{490} + 1.4(12) = 703 \text{ PLF} \\ 1.2DL + 1.6LL = 1.2(490 + 12) + 1.6(50) = 683 \text{ PLF} \end{array} \right\} \therefore 1.4DL \text{ CONTROLS}$$

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

∴ SIZE UP



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

SELECT W10x15 → $\phi M_N = 60\text{K·FT} > 51\text{K·FT}$ ✓ ∴ CONTINUE W/ W10x15

FOR W10x15 → $I_x = 68.9 \text{in}^4$

CHECK WET CONCRETE Δ:

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

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

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

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

SELECT MEMBER W/ HIGHER I_x → $\underset{\text{(ECONOMY)}}{\text{W10x22}}$ → $\phi M_n = 97.5 \text{ K·FT}$ & $I_x = 118 \text{in}^4$, $\Sigma Q_N = 81.1$

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

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

CHECK UNSHORED LENGTH:

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

$$1.2DL + 1.6LL = 1.2(490 + 22) + 1.6(50) = 695 \text{ PLF}$$

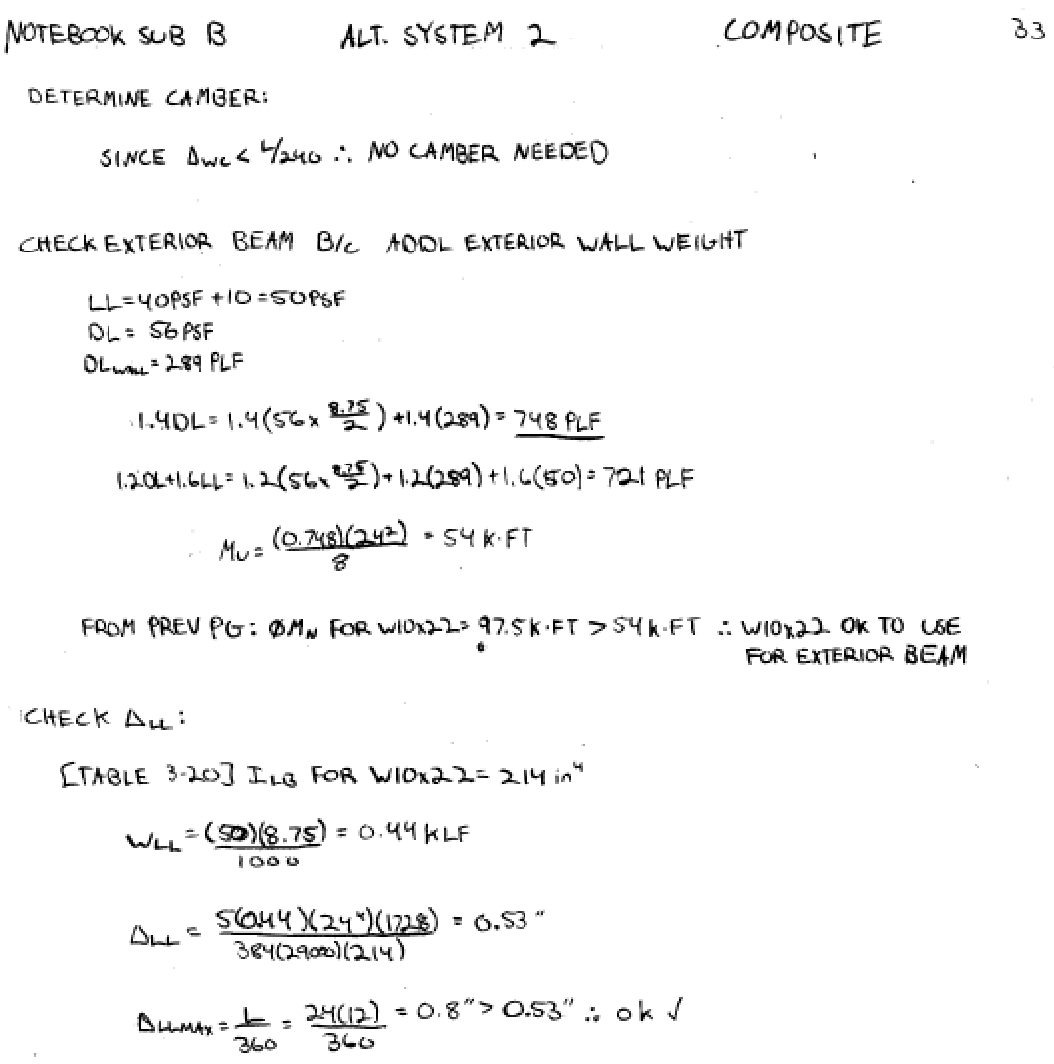
$$M_U = \frac{(0.717)(24^2)}{8} = 52 \text{ k·FT}$$

ϕM_N FOR W10x22 $= 97.5 > 52$ ✓

CHECK WET CONCRETE Δ:

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

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



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

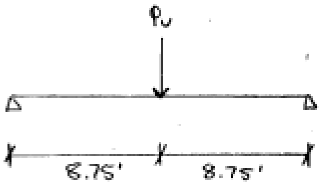
$W_{LL} = \frac{(50)(8.75)}{1000} = 0.44\ KLF$

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

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

DETERMINE GIRDER DESIGN:

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



$$P_u = (0.148)(2\times\frac{24}{2})(17.5') = 62k$$

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

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

$W14\times26 \rightarrow \Sigma Q_N = 279 \rightarrow {}^{279}/_{17.2} = 17\times2 = 34 \rightarrow \Sigma Q_N = (2)(16.5)(14.6) + 1(17.6) = 499$

$W14\times30 \rightarrow \Sigma Q_N = 183 \rightarrow {}^{183}/_{17.2} = 11\times2 = 22 \rightarrow \Sigma Q_N = (2)(5.5)(14.6) + 12(17.6) = \underline{372}$

$W14\times34 \rightarrow \Sigma Q_N = 175 \rightarrow {}^{175}/_{17.2} = 11\times2 = 22 \Rightarrow \Sigma Q_N = (2)(5.5)(14.6) + 12(17.6) = 372$

$W16\times26 \rightarrow \Sigma Q_N = 96 \rightarrow {}^{96}/_{17.2} = 6\times2 = 12$

CHECK a ASSUMPTION:

$$b_{EFF} = \min\begin{cases} \text{SPAN}/8 = \frac{17.5(12)}{8} = 26.3\times2 = \underline{52.6} \\ \text{SPACING}/2 \quad \frac{24(12)}{2} = 36\times2 = 72 \end{cases} \times 2 \text{ B/C GIRDER NOT AFFECTED BY EXTERIOR}$$

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

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}:

$W_{WC, GIRDER} = 56(17.5) + 30 = 1010 \text{ PLF}$

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

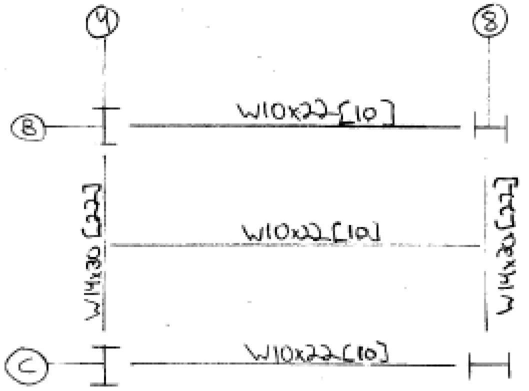
$$\Delta_{WC,MAX} = \frac{L}{240} = \frac{(17.5)(12)}{240} = 0.88'' > 0.3'' \therefore OK \checkmark$$

CHECK ECONOMY:

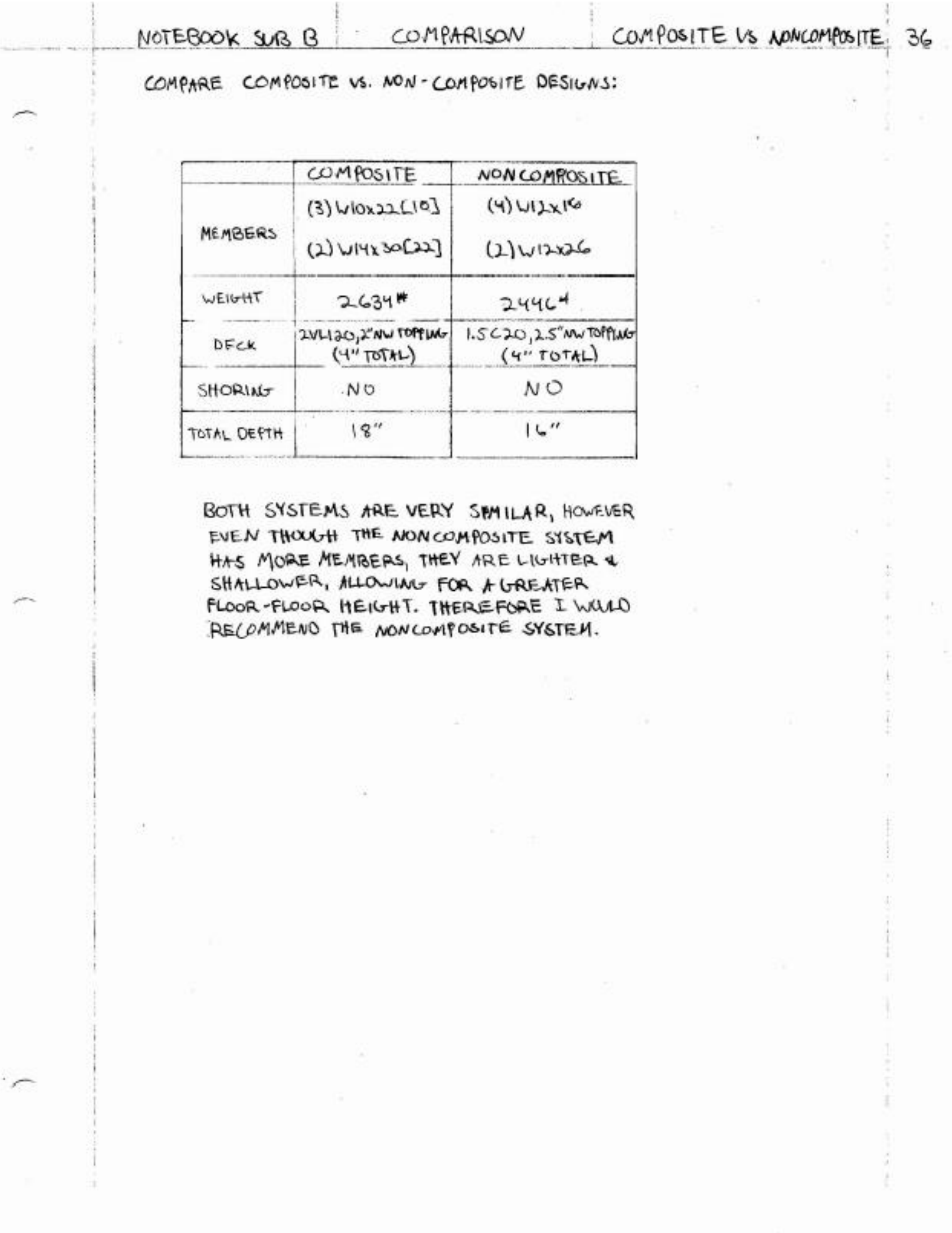
W14x26 → 26(17.5) + 34(10) = 795#

W 14x30 → 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:

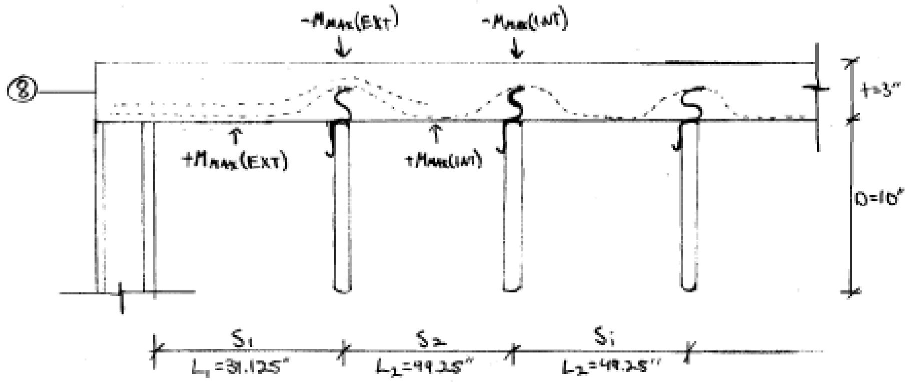
	COMPOSITE	NONCOMPOSITE
MEMBERS	(3) W10x22[10] (2) W14x30[22]	(4) W12x16 (2) W12x26
WEIGHT	2634#	2446#
DECK	2VLI20, 2" NW TOPPING (4" TOTAL)	1.5C20, 2.5" NW TOPPING (4" TOTAL)
SHORING	NO	NO
TOTAL DEPTH	18"	16"

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

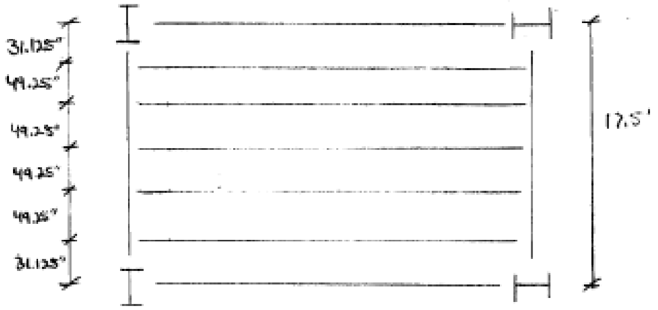
NOTEBOOK SUB B ALT. SYSTEM 3 HAMBRO D-500 37

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=40PSF, DL=65PSF

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

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

F'C=3000PSI, FY=50KSI

Δ_{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

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

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

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

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

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

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

CONCENTRATED LL REQUIREMENTS:

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

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

∴ PROVIDE SINGLE LAYER MESH THROUGHOUT B/C $S_1 = 31.125 < 48''$

DEFLECTION CHECK:

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

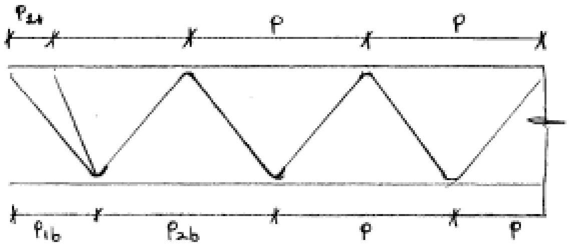
$$I_c = \frac{12(2.5^3)}{12} = 15.6\ in^4 \rightarrow \frac{\Delta}{L} = \frac{L^3}{I_c} = \frac{4.1^3}{15.6} = 4.4$$

NORMAL 7½" SLAB / 20' SPAN

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

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

SHEAR(WEB) DESIGN:



[PG 9] HAMBRO DG: WEB GEOMETRY

D=10" ∴ P_{1t} = 6@12

P_{1b} = 6@16

P_{2b} = 12

P = 20

IN SUMMARY:

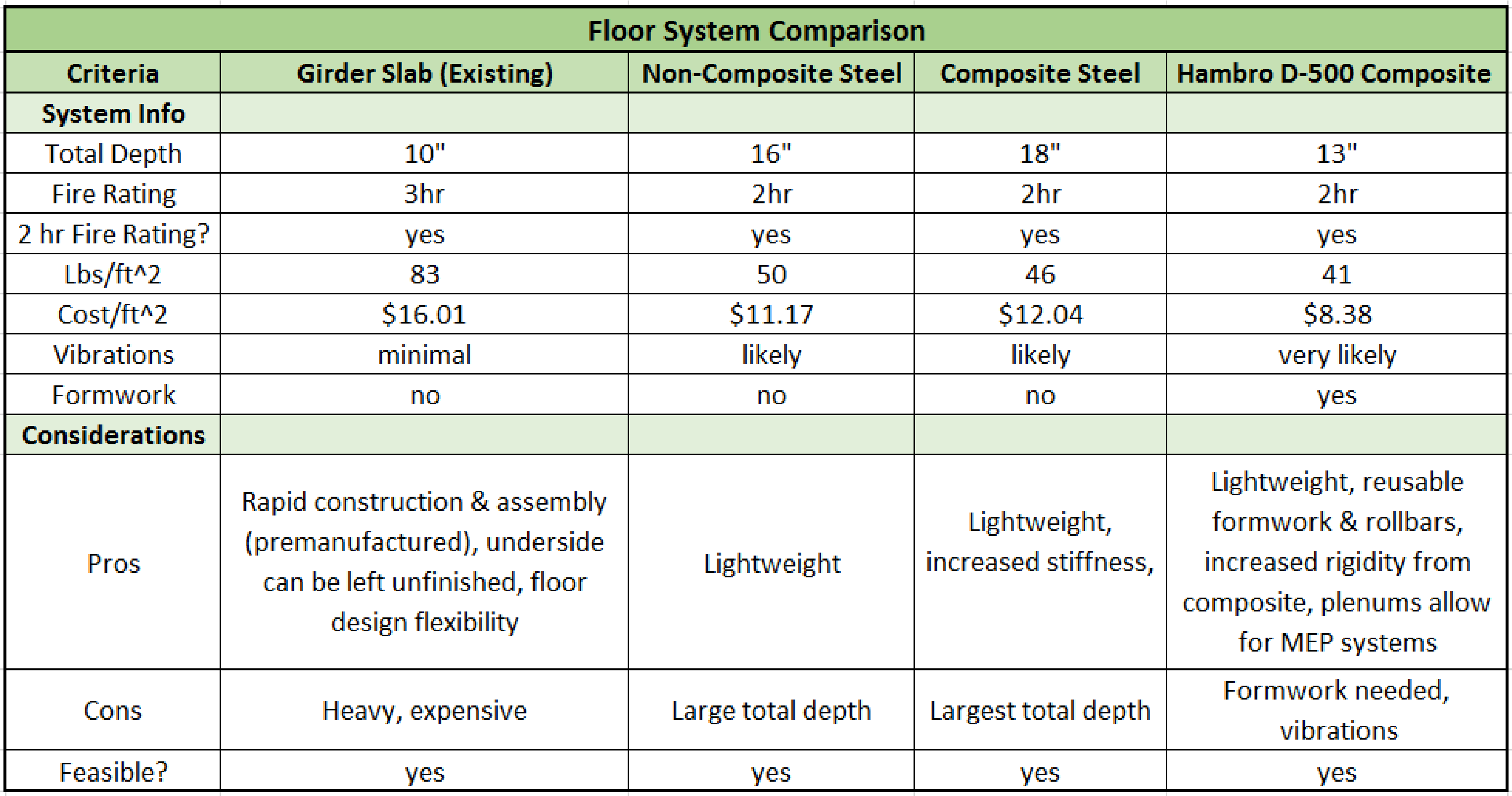
TOTAL THICKNESS = 13"

JOIST SPACING = 49¼" TYP., 31.125" FOR EXTERIOR JOISTS

PROVIDE SINGLE LAYER 6x6xW29xW2.9 MESH THROUGHOUT

System Comparison

Table 8: Several different floor systems are compared by various factors for AC Hotel Philadelphia.



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

Lateral Analysis

The scope of the analysis for Notebook Submission C includes an in-depth lateral force evaluation of 230 N. 13th St. A 3D model was created in RAM Structural System to model the lateral force resisting elements of the structure and examine how the forces are distributed to the lateral elements. Results from RAM were compared to calculations computed by hand to verify if the values make sense. Appendix B contains more relevant tables and visuals for Notebook Submission C that are not included in the report.

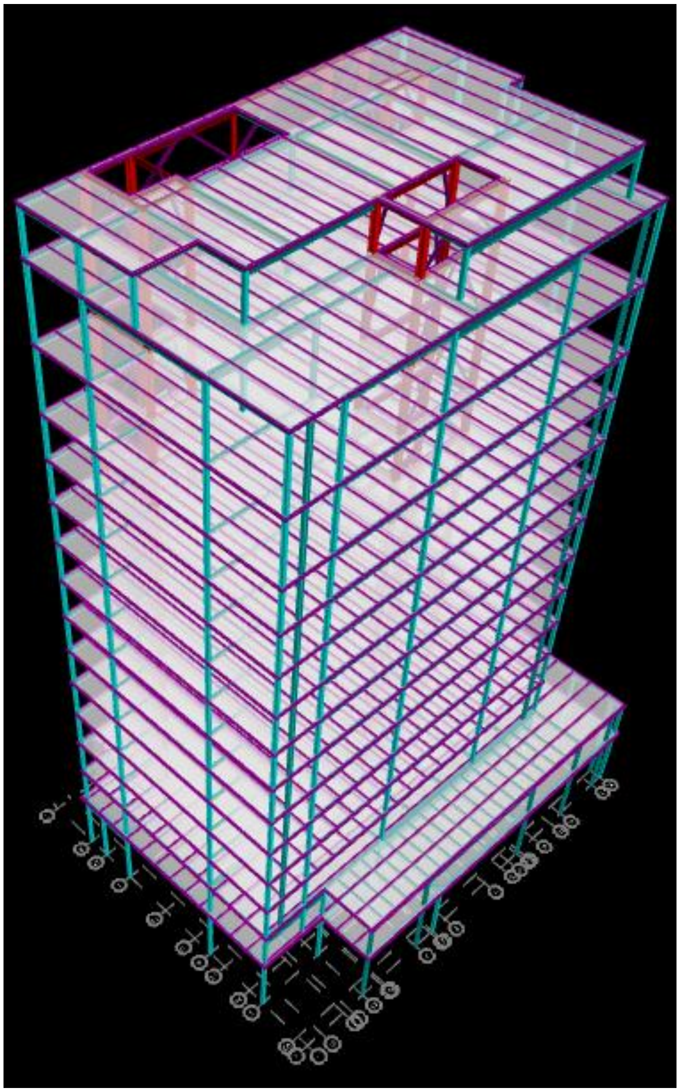


Figure 5: 3D model of AC Hotel Philadelphia created in RAM Structural System. Certain elements of the building that do not have an impact on my investigation were not modeled.

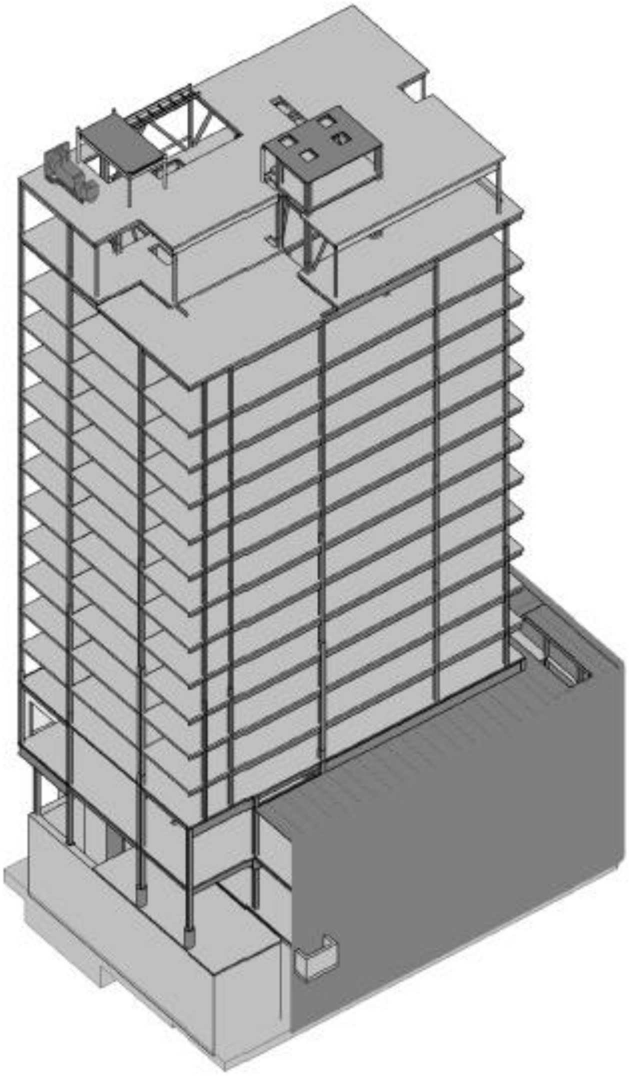


Figure 6: 3D representation of AC Hotel Philadelphia provided by Holbert Apple Associates.

Lateral Resisting Elements

The lateral system for 230 N 13th St. is made up of eight braced frames spanning 13' to 20' in both orthogonal directions. They are positioned in a configuration that keeps the COR towards the center of the structure which helps reduce eccentricity. Since braced frames are the most rigid steel lateral-resisting element, fewer frames are required which allows more spatial opportunities for the architects' floor layout.

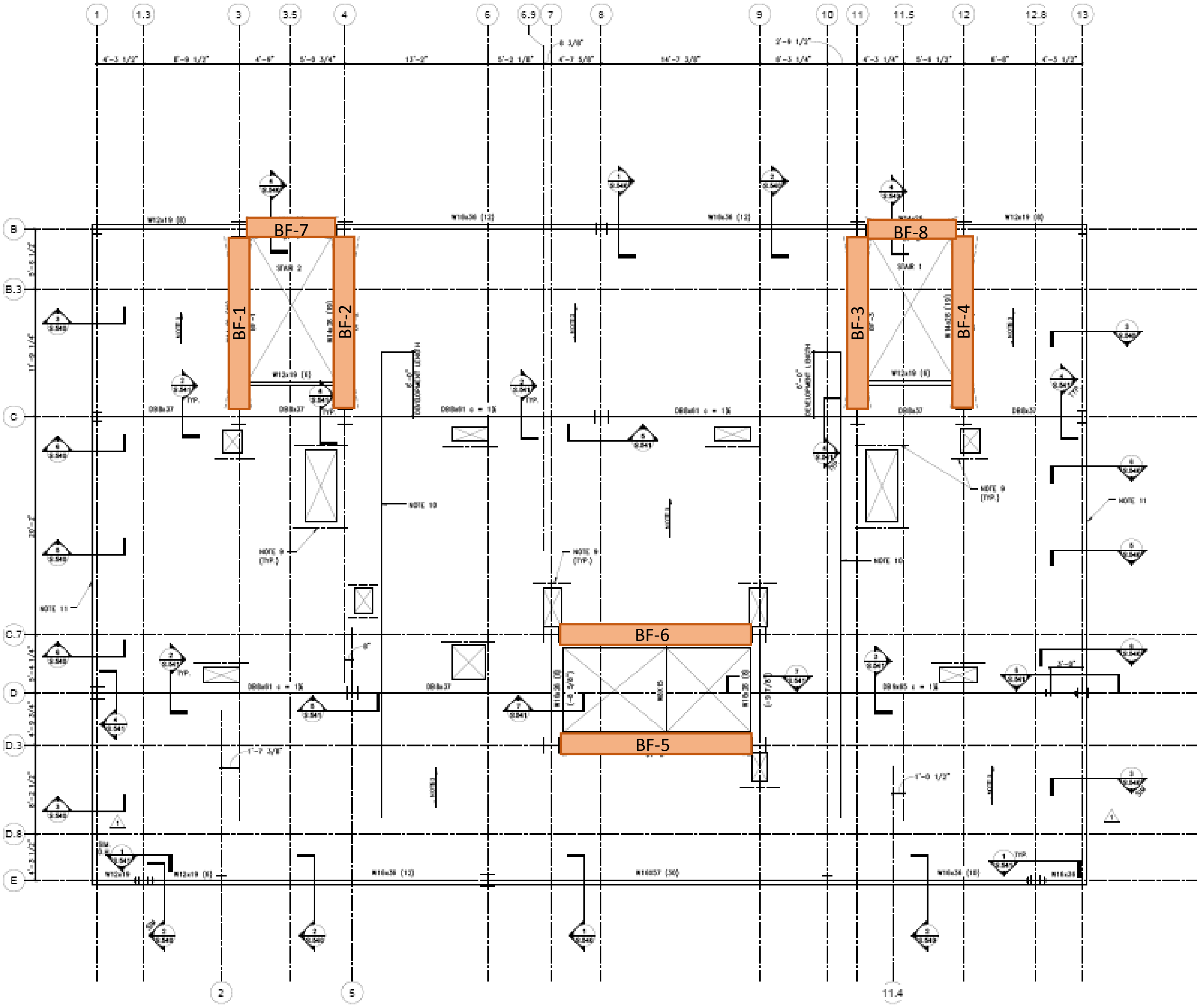


Figure 7: Typical floor plan showing all of the lateral resisting elements. BF 1-5 are concentric braced frames, BF-6 is an eccentric braced frame and BF 7-8 are regular braced frames.

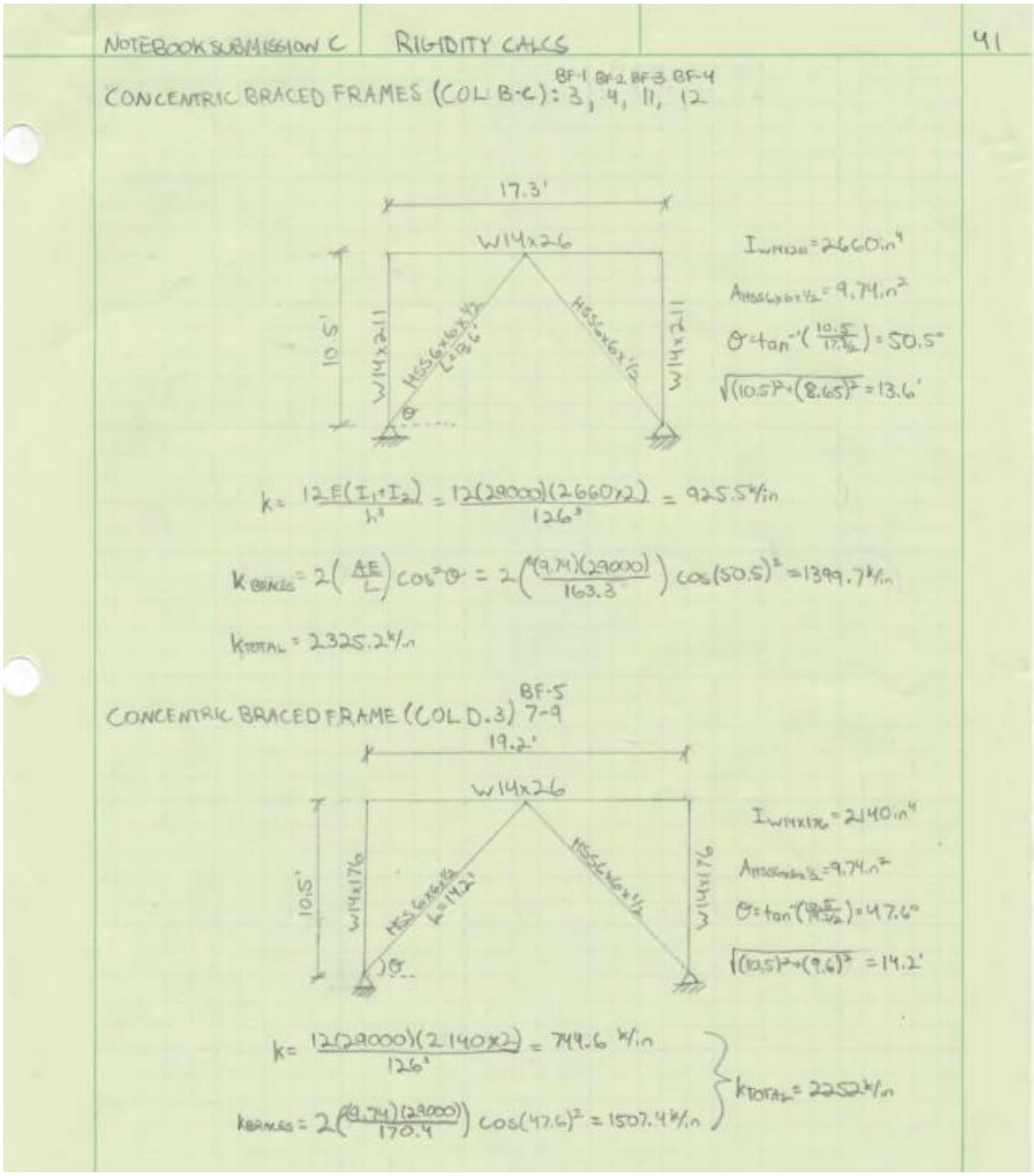
Modeling Approach/Assumptions/Constraints

To model AC Hotel Philadelphia, RAM Structural System was selected. The modeling process began with establishing the grid coordinates for the building as per the construction drawings. Once the grid was in place, the various floor plans within the building were modeled (Level 1, Level 2, Level 3, Typ. Level, Penthouse Level and Roof). Only columns supporting the braced frames were considered to be lateral columns, all others are gravity columns. To help simplify the model, only beams that had impact on the lateral resisting elements were modeled. It was also decided to not model the cellar level (beneath grade) because the amount of time it would have taken to model does not compare to the minimal amount it would have changed the results. Once the structure was modeled, member sizes were assigned to all of the lateral beams, columns and braced frames. Surface loads and line loads were then applied where appropriate, and mass dead loads were inserted in place of the mechanical equipment and any other substantial loads (i.e. pool on the second level). After the loads were imputed, the fixity of the members were applied. For beams and braces, a pinned connection was chosen for the major & minor axes, and fixed for torsion on each end. The columns are all fixed connections with the exception of the bottom of the columns at the base of the structure, where minor/major axes are pinned and torsion is fixed. As a way to reduce the amount of errors in the building model, the "integrity check" command was ran at least one time per floor. This made it much easier in the end since locating an error on an individual floor is much easier than finding an error within the whole building. To allow forces to be distributed into the resisting elements, nodes in RAM were released. It is important to keep some of the nodes connected, otherwise complications will arise within the program. While modeling, the following assumptions were also made for simplification:

- 6" slab edge overhang
- Dissymmetrical beam conversion (D-beams not available in RAM)
    - DB8x37=W8x35
    - DB8x61-W8x58
    - DB8x65=W8x67
- Only modeled (2) stair openings & (1) elevator shaft opening
- BF-6 (3'3" on both sides)
- Beams considered rigid for frame rigidity calculations
- Diaphragm for all levels considered rigid
- Extra beams/columns inserted where necessary to avoid complications within RAM (these do not affect any output numbers)
- From the Penthouse level and above, there is a curtainwall/terracotta rain screen system, so the value of 27.5psf was reduced to 15psf for the dual system
- P-delta effects are included for RAM analysis

COM & COR

The center of mass (COM) and center of rigidity (COR) were calculated both by hand and by RAM for the typical level in the building. Both the COM & COR varied from floor-floor, but the variation was minimal, so evaluating Level 4 is a reasonable approximation for the entire structure. The following diagrams illustrate the approximate rigidities of the braced frames.



NOTEBOOK SUBMISSION C RIGIDITY CALCS 41

CONCENTRIC BRACED FRAMES (COL B-C): 3, 4, 11, 12 (BF-1, BF-2, BF-3, BF-4)

$I_{W14x26} = 2660 in^4$

$A_{HSS6x6x1/2} = 9.74 in^2$

$\theta = \tan^{-1}\left(\frac{10.5}{17.3/2}\right) = 50.5°$

$\sqrt{(10.5)^2 + (8.65)^2} = 13.6'$

$$k = \frac{12E(I_1+I_2)}{h^3} = \frac{12(29000)(2660\times2)}{126^3} = 925.5 k/in$$

$$k_{BRACES} = 2\left(\frac{AE}{L}\right)\cos^2\theta = 2\left(\frac{(9.74)(29000)}{163.3}\right)\cos(50.5)^2 = 1399.7 k/in$$

$k_{TOTAL} = 2325.2 k/in$

CONCENTRIC BRACED FRAME (COL D.3) 7-9 (BF-5)

$I_{W14x176} = 2140 in^4$

$A_{HSS6x6x1/2} = 9.74 in^2$

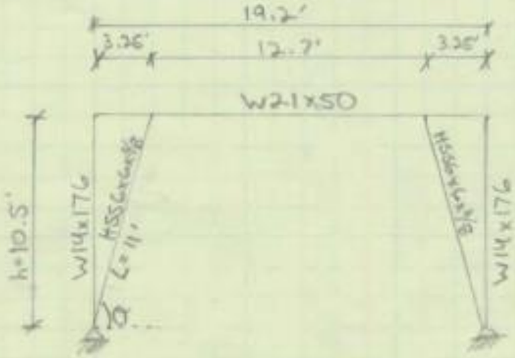
$\theta = \tan^{-1}\left(\frac{10.5}{19.2/2}\right) = 47.6°$

$\sqrt{(10.5)^2 + (9.6)^2} = 14.2'$

$$\left.\begin{aligned} k &= \frac{12(29000)(2140\times2)}{126^3} = 744.6 k/in \\ k_{BRACES} &= 2\left(\frac{(9.74)(29000)}{170.4}\right)\cos(47.6)^2 = 1507.4 k/in \end{aligned}\right\} k_{TOTAL} = 2252 k/in$$

NOTEBOOK SUBMISSION C | RIGIDITY CALCS CONT. | 42

ESSENTRIC BRACED FRAME (COL C.7) BF-6 7-9



$I_{W14x176} = 2140 in^4$

$A_{HSS6x6x3/8} = 7.58 in^2$

$L = \sqrt{(3.25)^2 + (10.5)^2} = 11'$

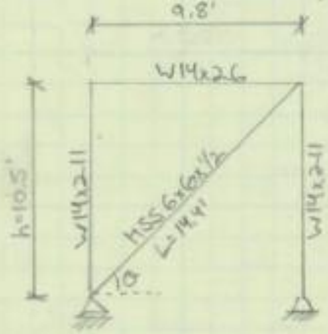
$\theta = \tan^{-1}\left(\frac{10.5}{3.25}\right) = 72.8°$

$$k = \frac{12(29000)(2 \times 2140)}{126^2} = 744.6\ k/in$$

$$k_{BRACES} = 2\left(\frac{(7.58)(29000)}{132}\right)\cos(72.8)^2 = 291.2\ k/in$$

$k_{TOTAL} = 1035.8\ k/in$

CONCENTRIC BRACED FRAMES (COL B) BF-7 3-4, BF-8 11-12



$I_{W14x211} = 2660 in^4$

$A_{HSS6x6x1/2} = 9.74 in^2$

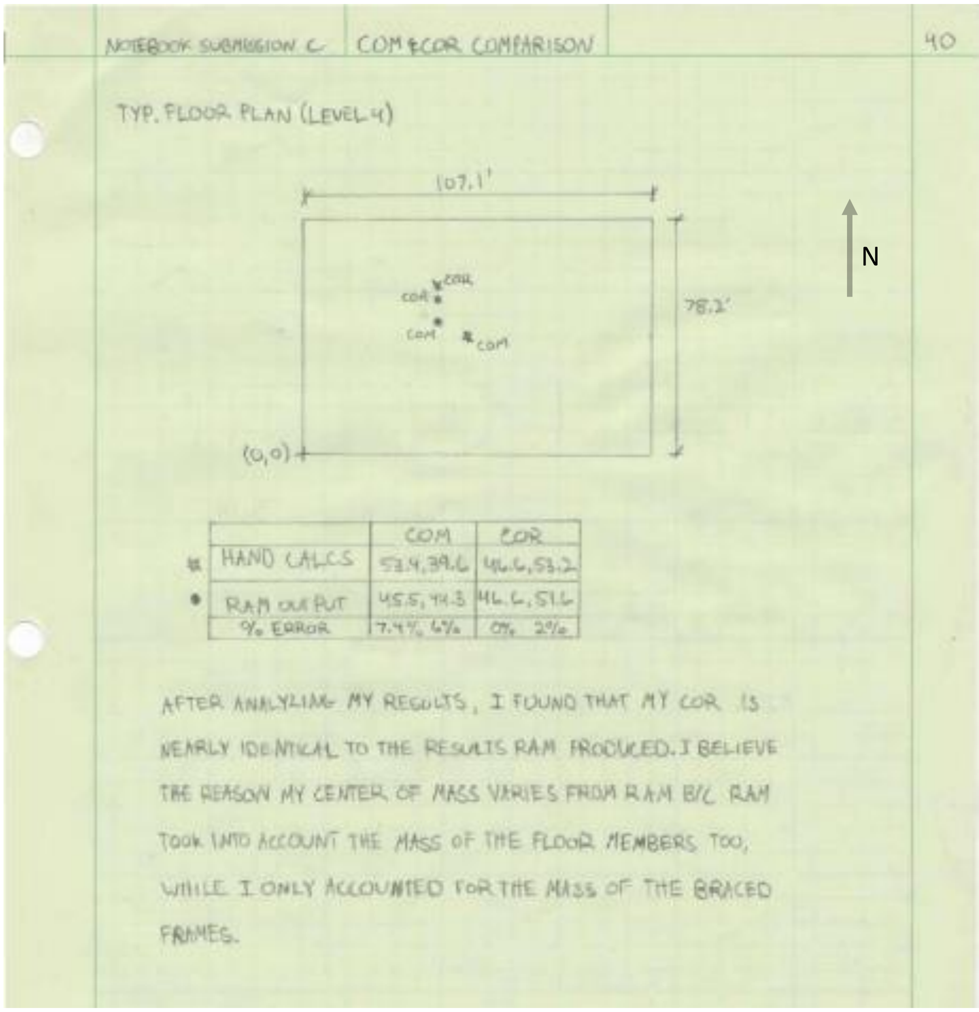
$\theta = \tan^{-1}\left(\frac{10.5}{9.8}\right) = 47°$

$L = \sqrt{(10.5)^2 + (9.8)^2} =$

$$k = \frac{12(29000)(2 \times 2660)}{126^3} = 925.5\ k/in$$

$$k_{BRACE} = \frac{(9.74)(29000)}{172.8}\cos(47)^2 = 762.1\ k/in$$

$k_{TOTAL} = 1687.6\ k/in$



NOTEBOOK SUBMISSION C | COM & COR COMPARISON | 40

TYP. FLOOR PLAN (LEVEL 4)

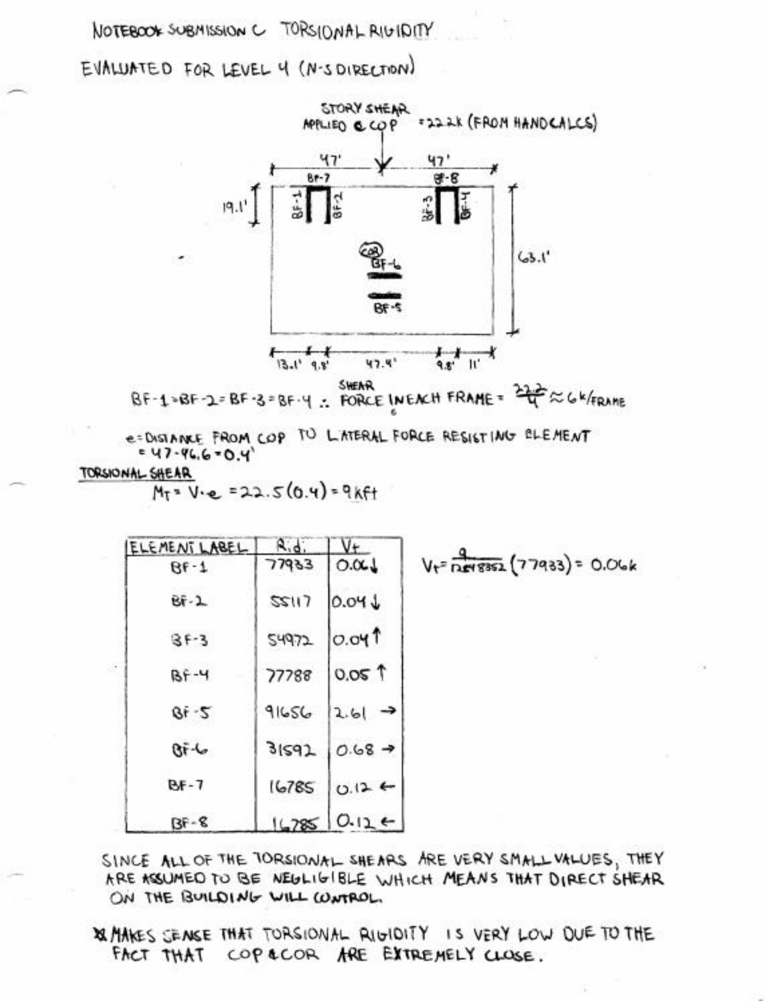
		COM	COR
*	HAND CALCS	53.4, 39.6	46.6, 53.2
●	RAM OUTPUT	45.5, 44.3	46.6, 51.6
	% ERROR	7.4%, 6%	0%, 2%

AFTER ANALYZING MY RESULTS, I FOUND THAT MY COR IS NEARLY IDENTICAL TO THE RESULTS RAM PRODUCED. I BELIEVE THE REASON MY CENTER OF MASS VARIES FROM RAM B/C RAM TOOK INTO ACCOUNT THE MASS OF THE FLOOR MEMBERS TOO, WHILE I ONLY ACCOUNTED FOR THE MASS OF THE BRACED FRAMES.

The values above seem reasonable because the COR was found to be in a location that lies in the middle of all of the LRE (Lateral Resisting Elements). The COM was found to be near the center of the floor which also makes sense because the floors do not contain any elements which would drastically move the COM from the middle. Detailed spreadsheets for COM and COR can be found in Appendix B.

Direct Shear & Torsional Rigidity

When analyzing how lateral loads find their way into buildings, it is important to keep in mind that load follows stiffness. To find the direct shear in each lateral resisting element, this proportional distribution of forces was applied and the results can be found below. Torsional shear takes into account the amount of eccentricity the building experiences. Larger eccentricities occur when the COR is not near the COP (which acts at the middle of the structure). In the specific case of AC Hotel Philadelphia, both hand calculations and RAM found the COR to be within one foot of the COP, which explains why the torsional shear is miniscule. Results for torsional shear can also be found below. Full results and calculations can be found in Appendix B.



NOTEBOOK SUBMISSION C TORSIONAL RIGIDITY

EVALUATED FOR LEVEL 4 (N-S DIRECTION)

BF-1=BF-2=BF-3=BF-4 ∴ SHEAR FORCE IN EACH FRAME = $\frac{22.2}{4} \approx 6^{k}$/FRAME

e=DISTANCE FROM COP TO LATERAL FORCE RESISTING ELEMENT
=47-46.6=0.4'

TORSIONAL SHEAR

$M_T = V \cdot e = 22.5(0.4) = 9$ kft

ELEMENT LABEL	R.d.	Vt
BF-1	77933	0.06↓
BF-2	55117	0.04↓
BF-3	54972	0.04↑
BF-4	77788	0.05↑
BF-5	91656	2.61 →
BF-6	31592	0.68 →
BF-7	16785	0.12 ←
BF-8	16785	0.12 ←

$V_t = \frac{9}{12548352}(77933) = 0.06k$

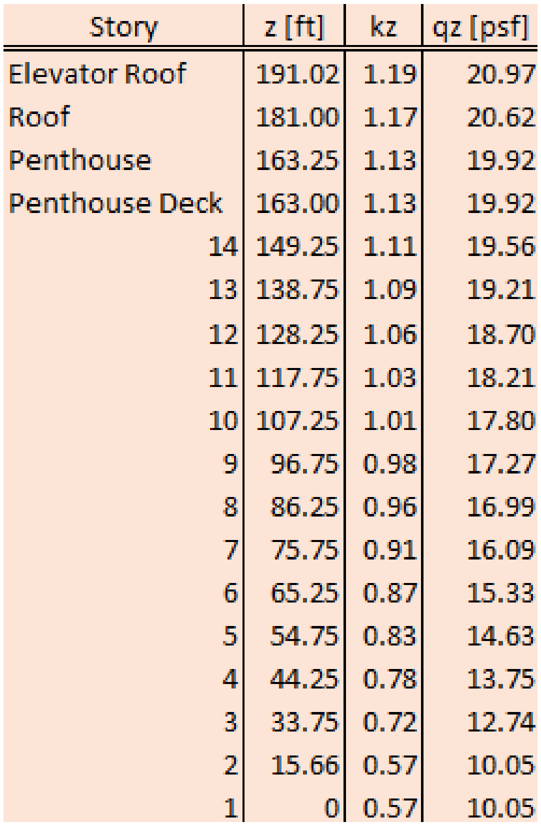
SINCE ALL OF THE TORSIONAL SHEARS ARE VERY SMALL VALUES, THEY ARE ASSUMED TO BE NEGLIGIBLE WHICH MEANS THAT DIRECT SHEAR ON THE BUILDING WILL CONTROL.

✲ MAKES SENSE THAT TORSIONAL RIGIDITY IS VERY LOW DUE TO THE FACT THAT COP & COR ARE EXTREMELY CLOSE.

Wind Load Comparison

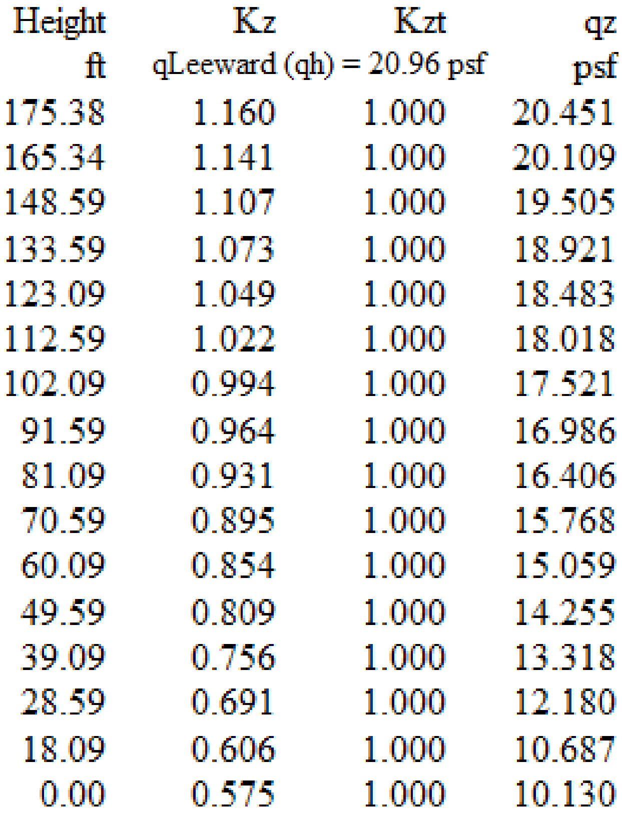
Table 9 and 10 below are tabulated wind pressure values for the various levels of AC Hotel Philadelphia. Minor differences in pressures are due to the elevation they were analyzed at. Hand calculations were analyzed at each floor and RAM uses mid-floor elevations to do so. Hand calculations include the elevator roof which was not modeled in RAM. The largest variation in calculations is in the building period calculation. Hand calculations yield a building period of 1.55s compared to RAM's calculated building period of 2.46s. The reason these values vary so greatly is the fact that in ASCE7-05, section 12.8.2, it states that it is permitted to use the approximate building period, Ta, for the fundamental period, T for hand computations, while RAM actually solves for the fundamental building period.

Table 9: Tabulated wind pressures calculated by hand for various heights



Story	z [ft]	kz	qz [psf]
Elevator Roof	191.02	1.19	20.97
Roof	181.00	1.17	20.62
Penthouse	163.25	1.13	19.92
Penthouse Deck	163.00	1.13	19.92
14	149.25	1.11	19.56
13	138.75	1.09	19.21
12	128.25	1.06	18.70
11	117.75	1.03	18.21
10	107.25	1.01	17.80
9	96.75	0.98	17.27
8	86.25	0.96	16.99
7	75.75	0.91	16.09
6	65.25	0.87	15.33
5	54.75	0.83	14.63
4	44.25	0.78	13.75
3	33.75	0.72	12.74
2	15.66	0.57	10.05
1	0	0.57	10.05

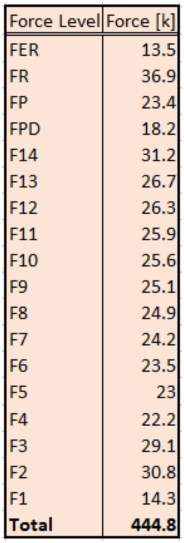
Table 10: Tabulated wind pressures calculated by RAM values for various elevations.



Height	Kz	Kzt	qz
ft	qLeeward (qh) = 20.96 psf		psf
175.38	1.160	1.000	20.451
165.34	1.141	1.000	20.109
148.59	1.107	1.000	19.505
133.59	1.073	1.000	18.921
123.09	1.049	1.000	18.483
112.59	1.022	1.000	18.018
102.09	0.994	1.000	17.521
91.59	0.964	1.000	16.986
81.09	0.931	1.000	16.406
70.59	0.895	1.000	15.768
60.09	0.854	1.000	15.059
49.59	0.809	1.000	14.255
39.09	0.756	1.000	13.318
28.59	0.691	1.000	12.180
18.09	0.606	1.000	10.687
0.00	0.575	1.000	10.130

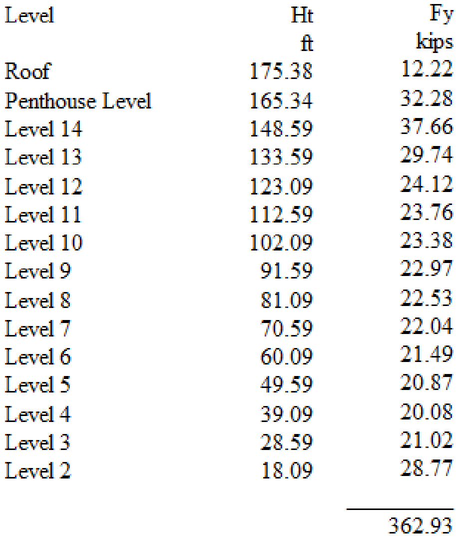
Applied Story Forces (N-S)

Table 11: Hand calculations showing the applied story forces in the North-South direction.



Force Level	Force [k]
FER	13.5
FR	36.9
FP	23.4
FPD	18.2
F14	31.2
F13	26.7
F12	26.3
F11	25.9
F10	25.6
F9	25.1
F8	24.9
F7	24.2
F6	23.5
F5	23
F4	22.2
F3	29.1
F2	30.8
F1	14.3
Total	**444.8**

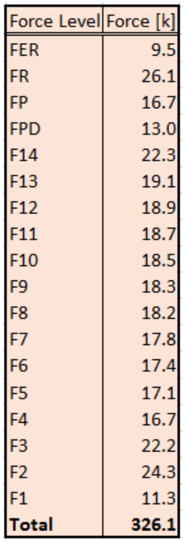
Table 12: RAM output showing the applied story forces in the North-South direction.



Level	Ht	Fy
	ft	kips
Roof	175.38	12.22
Penthouse Level	165.34	32.28
Level 14	148.59	37.66
Level 13	133.59	29.74
Level 12	123.09	24.12
Level 11	112.59	23.76
Level 10	102.09	23.38
Level 9	91.59	22.97
Level 8	81.09	22.53
Level 7	70.59	22.04
Level 6	60.09	21.49
Level 5	49.59	20.87
Level 4	39.09	20.08
Level 3	28.59	21.02
Level 2	18.09	28.77
		362.93

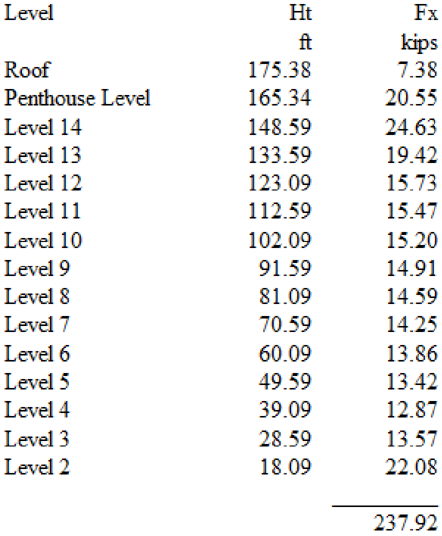
Applied Story Forces (E-W)

Table 13: Hand calculations showing the applied story forces in the North-South direction.



Force Level	Force [k]
FER	9.5
FR	26.1
FP	16.7
FPD	13.0
F14	22.3
F13	19.1
F12	18.9
F11	18.7
F10	18.5
F9	18.3
F8	18.2
F7	17.8
F6	17.4
F5	17.1
F4	16.7
F3	22.2
F2	24.3
F1	11.3
Total	**326.1**

Table 14: RAM output showing the applied story forces in the North-South direction.



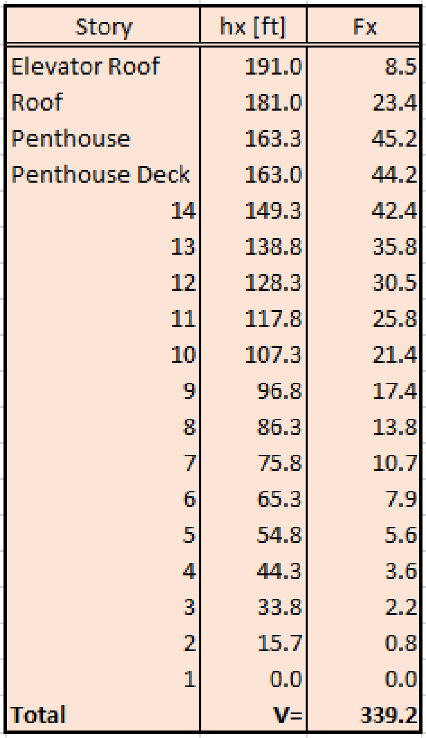
Level	Ht	Fx
	ft	kips
Roof	175.38	7.38
Penthouse Level	165.34	20.55
Level 14	148.59	24.63
Level 13	133.59	19.42
Level 12	123.09	15.73
Level 11	112.59	15.47
Level 10	102.09	15.20
Level 9	91.59	14.91
Level 8	81.09	14.59
Level 7	70.59	14.25
Level 6	60.09	13.86
Level 5	49.59	13.42
Level 4	39.09	12.87
Level 3	28.59	13.57
Level 2	18.09	22.08
		237.92

After analyzing the story forces, it is easy to see that my hand calculations yielded slightly higher forces at all elevations which results in a much larger overall force in each direction. All though my forces are off, they are proportional, therefore, the error is most likely from not distributing the pressures correctly to each floor by tributary height.

Seismic Load Comparison

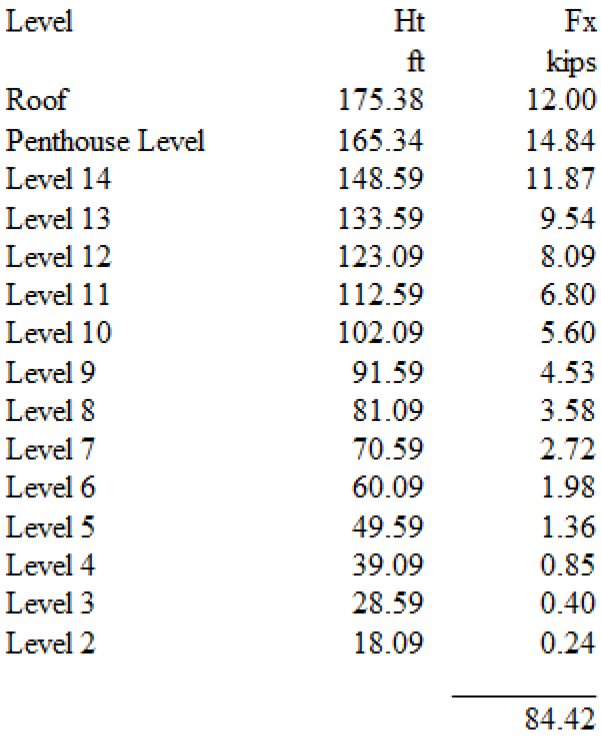
Seismic load calculations varied significantly from my hand calculations to the results RAM produced. The main reason my hand calculations found a building weight much greater (approx. 3000k) than RAM is due to the fact some members in RAM were not modeled because it was not required to have a full 3D model for the assignment. Also, as noted in the wind calculations above, the building period used for hand calculations was nearly double what RAM used. With this said, AC Hotel Philadelphia was designed with a base shear of 92k compared to 84.4k from RAM which is reasonably close. If the two changes were made in the 3D RAM model, the results in Table 15 would be much closer to what they should be.

Table 15: Hand calculations showing the applied story forces in both orthogonal directions under seismic conditions.



Story	hx [ft]	Fx
Elevator Roof	191.0	8.5
Roof	181.0	23.4
Penthouse	163.3	45.2
Penthouse Deck	163.0	44.2
14	149.3	42.4
13	138.8	35.8
12	128.3	30.5
11	117.8	25.8
10	107.3	21.4
9	96.8	17.4
8	86.3	13.8
7	75.8	10.7
6	65.3	7.9
5	54.8	5.6
4	44.3	3.6
3	33.8	2.2
2	15.7	0.8
1	0.0	0.0
Total	V=	339.2

Table 16: RAM output showing the applied story forces in both orthogonal directions under seismic conditions.



Level	Ht ft	Fx kips
Roof	175.38	12.00
Penthouse Level	165.34	14.84
Level 14	148.59	11.87
Level 13	133.59	9.54
Level 12	123.09	8.09
Level 11	112.59	6.80
Level 10	102.09	5.60
Level 9	91.59	4.53
Level 8	81.09	3.58
Level 7	70.59	2.72
Level 6	60.09	1.98
Level 5	49.59	1.36
Level 4	39.09	0.85
Level 3	28.59	0.40
Level 2	18.09	0.24
		84.42

Controlling Load Case

After analyzing both wind and seismic forces on the building, wind was determined to be the governing load case. Hand calculations show much higher values for seismic conditions, but knowing why the values vary so greatly from RAM verify that wind will control. This was also verified by looking at the overall geological location of the build site (Philadelphia, Pa), and realizing that wind would most likely control on the East coast. Therefore, the following checks will use values from wind conditions to verify the lateral systems and the members that comprise it.

Lateral System Checks

Allowable Drift

$$Allowable\ drift = \frac{h}{400} = \frac{191 * 12}{400} = 5.73"$$

$$Actual\ maximum\ drift\ (from\ RAM\ output)$$
$$= 4.74"\ @\ Roof\ Level < 5.73"\ therefore\ ok\ \surd$$

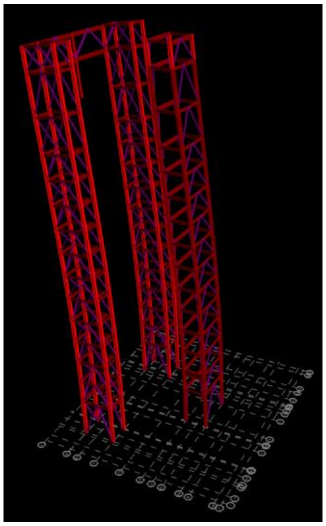
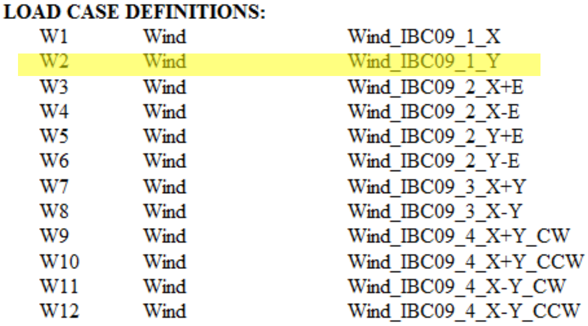


Figure 8: Deflected shape under wind conditions in the N-S direction.

In ASCE7-05, Figure 6-9, design wind load cases are presented. Of the four cases, case 1 controlled which is a “full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, each axis considered separately.” In RAM, this is equivalent to wind case 2 which analyzes the building in the N-S direction. This makes sense because the building is shallower in the N-S direction, allowing larger overall drift values. The largest drift due to seismic activity is only 1.71” at the roof level which also confirms that wind controls over seismic.

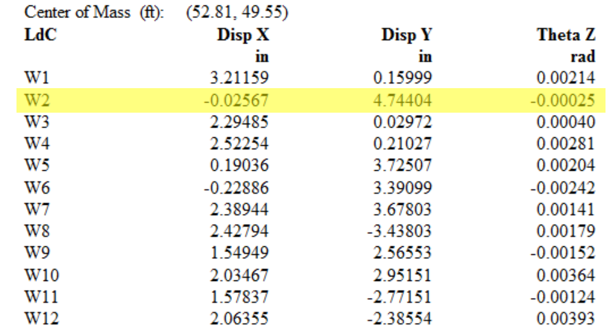
Table 18: Possible load cases considered in RAM.



LOAD CASE DEFINITIONS:

W1	Wind	Wind_IBC09_1_X
W2	Wind	Wind_IBC09_1_Y
W3	Wind	Wind_IBC09_2_X+E
W4	Wind	Wind_IBC09_2_X-E
W5	Wind	Wind_IBC09_2_Y+E
W6	Wind	Wind_IBC09_2_Y-E
W7	Wind	Wind_IBC09_3_X+Y
W8	Wind	Wind_IBC09_3_X-Y
W9	Wind	Wind_IBC09_4_X+Y_CW
W10	Wind	Wind_IBC09_4_X+Y_CCW
W11	Wind	Wind_IBC09_4_X-Y_CW
W12	Wind	Wind_IBC09_4_X-Y_CCW

Table 17: Displacements due to wind for the Roof level of AC Hotel Philadelphia.



Level: Roof, Diaph: 1

Center of Mass (ft): (52.81, 49.55)

LdC	Disp X in	Disp Y in	Theta Z rad
W1	3.21159	0.15999	0.00214
W2	-0.02567	4.74404	-0.00025
W3	2.29485	0.02972	0.00040
W4	2.52254	0.21027	0.00281
W5	0.19036	3.72507	0.00204
W6	-0.22886	3.39099	-0.00242
W7	2.38944	3.67803	0.00141
W8	2.42794	-3.43803	0.00179
W9	1.54949	2.56553	-0.00152
W10	2.03467	2.95151	0.00364
W11	1.57837	-2.77151	-0.00124
W12	2.06355	-2.38554	0.00393

To keep values comparable, inter-story drifts were evaluated for the same wind case (case 1) that controlled the design (N-S direction). Note that the actual drifts are compared to h/400 drift for all building level elevations

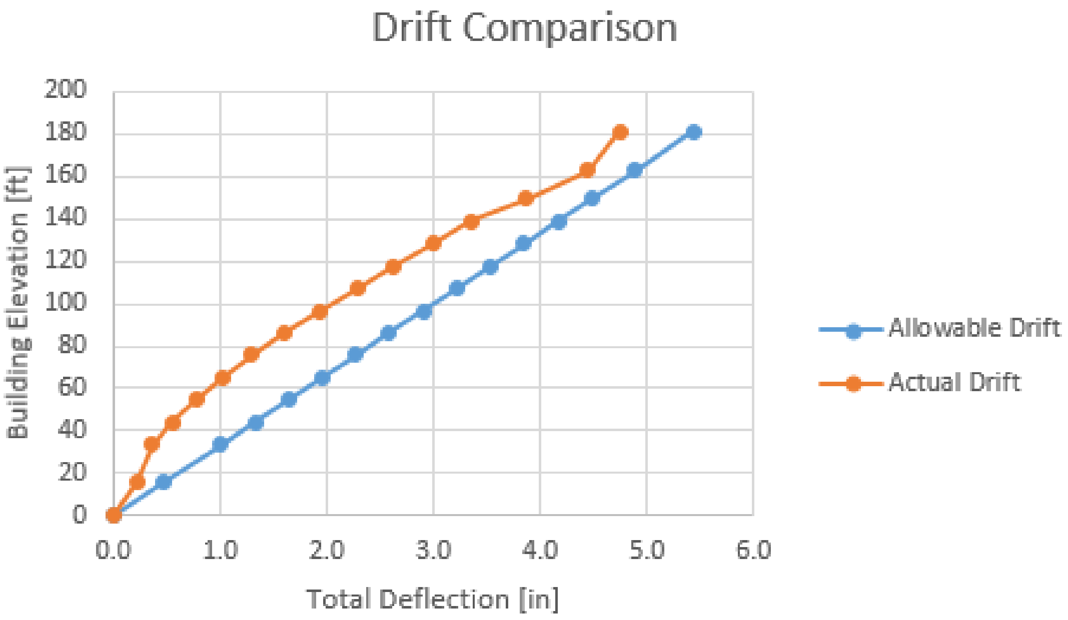


Figure 9: Actual drift compared to allowable drift (h/400) for various elevations of AC Hotel Philadelphia

Table 9: Displays whether or not each level meets the drift criteria.

Level	Elevation [ft]	Total Drift @ Particular Level [in]	Allowable Drift (h/400) [in]	Acceptable Drift?
Roof	181	4.74	5.43	yes
Penthouse	163	4.44	4.89	yes
14	149.25	3.87	4.48	yes
13	138.75	3.35	4.16	yes
12	128.25	2.99	3.85	yes
11	117.75	2.63	3.53	yes
10	107.25	2.28	3.22	yes
9	96.75	1.94	2.90	yes
8	86.25	1.61	2.59	yes
7	75.75	1.3	2.27	yes
6	65.25	1.03	1.96	yes
5	54.75	0.77	1.64	yes
4	44.25	0.55	1.33	yes
3	33.75	0.36	1.01	yes
2	15.66	0.22	0.47	yes
1	0	0	0.00	N/A

In the table above, all of the levels meet the drift requirement of H/400.

Member Spot Check for Lateral Loads

The area under consideration is a typical level (level 4). Elements under consideration are marked. BF-1 was selected because it resists forces in the N-S direction which is the axis being considered for lateral forces, making the braces in that direction more critical.

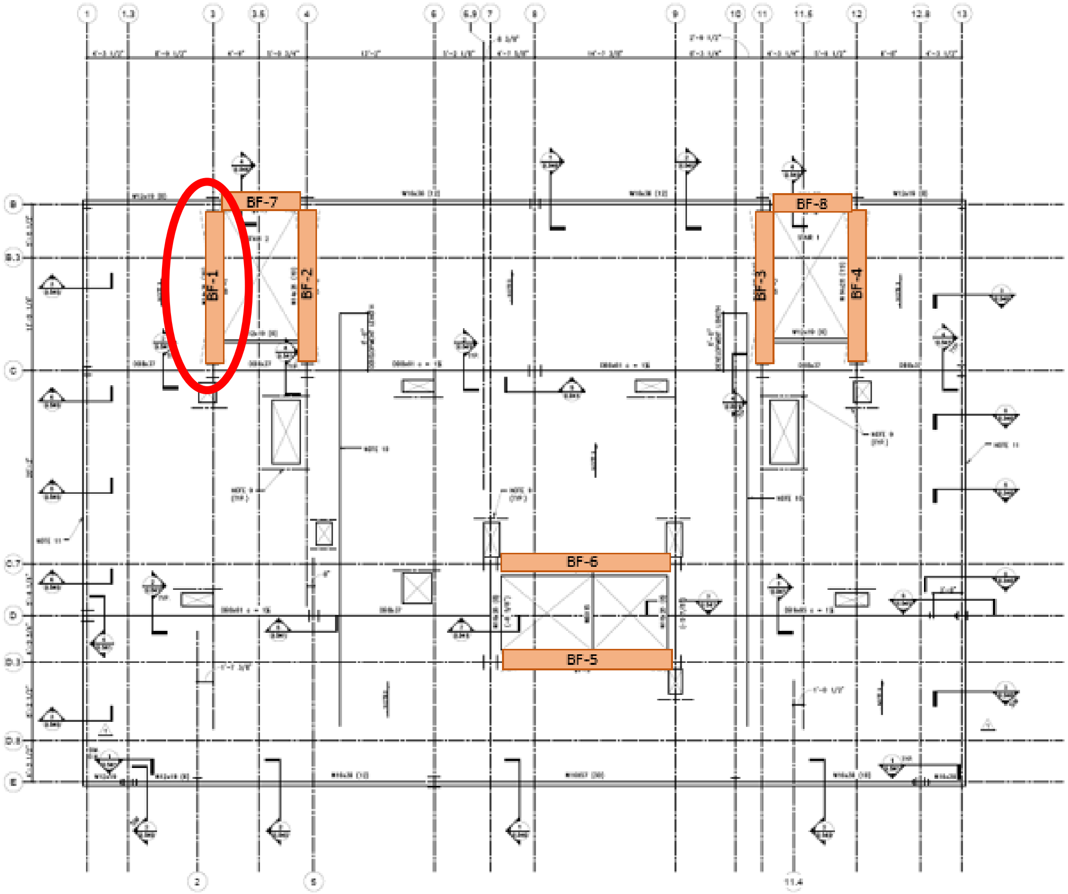


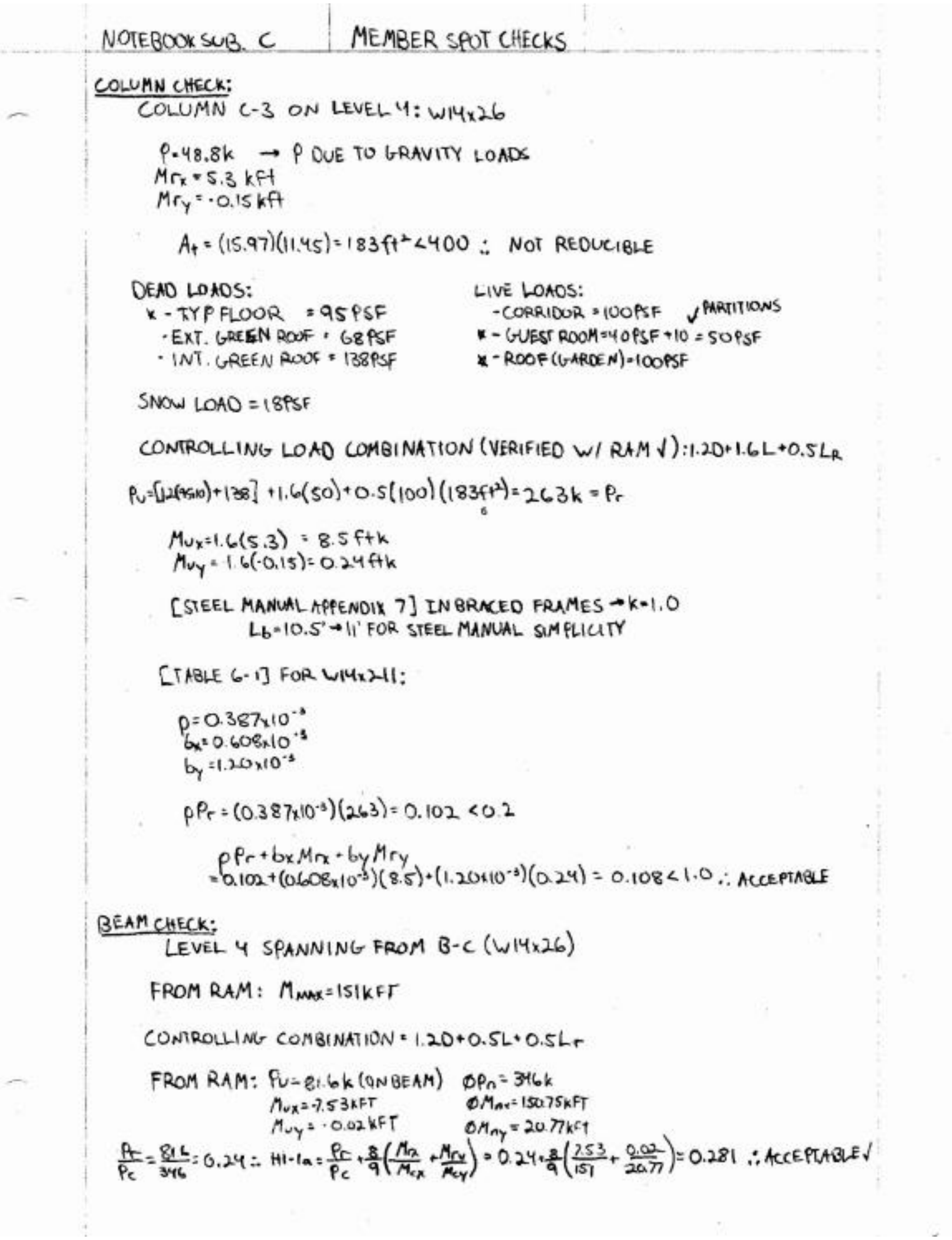
Figure 10: Typical floor plan revealing lateral elements/locations and the braced frame being studied.

BF-1 (Column line 3, spanning B-C)

Column: W14x211

Beam: W14x26

Brace: HSS 6x6x1/2



NOTEBOOK SUB. C | MEMBER SPOT CHECKS

COLUMN CHECK:

COLUMN C-3 ON LEVEL 4: W14x26

$P = 48.8k$ → P DUE TO GRAVITY LOADS
$M_{rx} = 5.3$ kft
$M_{ry} = -0.15$ kft

$A_t = (15.97)(11.45) = 183 ft^2 < 400$ ∴ NOT REDUCIBLE

DEAD LOADS:
- ✱ - TYP FLOOR = 95 PSF
- · EXT. GREEN ROOF = 68 PSF
- · INT. GREEN ROOF = 138 PSF

LIVE LOADS:
- -CORRIDOR = 100 PSF ✓ PARTITIONS
- ✱ - GUEST ROOM = 40 PSF + 10 = 50 PSF
- ✱ - ROOF (GARDEN) = 100 PSF

SNOW LOAD = 18 PSF

CONTROLLING LOAD COMBINATION (VERIFIED W/ RAM ✓): $1.2D + 1.6L + 0.5L_R$

$P_U = [1.2(95+10) + 138] + 1.6(50) + 0.5(100)(183 ft^2) = 263k = P_r$

$M_{ux} = 1.6(5.3) = 8.5$ ftk
$M_{uy} = 1.6(-0.15) = 0.24$ ftk

[STEEL MANUAL APPENDIX 7] IN BRACED FRAMES → k = 1.0
$L_b = 10.5' \rightarrow 11'$ FOR STEEL MANUAL SIMPLICITY

[TABLE 6-1] FOR W14x211:

$p = 0.387 \times 10^{-3}$
$b_x = 0.608 \times 10^{-3}$
$b_y = 1.20 \times 10^{-3}$

$pP_r = (0.387 \times 10^{-3})(263) = 0.102 < 0.2$

$pP_r + b_x M_{rx} + b_y M_{ry}$
$= 0.102 + (0.608 \times 10^{-3})(8.5) + (1.20 \times 10^{-3})(0.24) = 0.108 < 1.0$ ∴ ACCEPTABLE

BEAM CHECK:

LEVEL 4 SPANNING FROM B-C (W14x26)

FROM RAM: $M_{MAX} = 151$ KFT

CONTROLLING COMBINATION = $1.2D + 0.5L + 0.5L_r$

FROM RAM: $P_U = 81.6k$ (ON BEAM) $\phi P_n = 346k$
$M_{ux} = -7.53$ KFT $\phi M_{nx} = 150.75$ KFT
$M_{uy} = -0.02$ KFT $\phi M_{ny} = 20.77$ KFT

$\frac{P_r}{P_c} = \frac{81.6}{346} = 0.24$ ∴ H1-1a $= \frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) = 0.24 + \frac{8}{9}\left(\frac{7.53}{151} + \frac{0.02}{20.77}\right) = 0.281$ ∴ ACCEPTABLE ✓

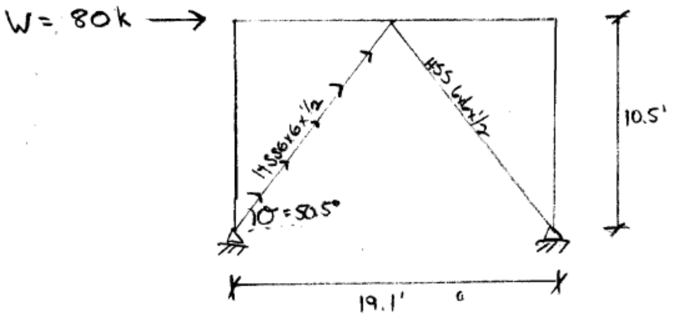
NOTEBOOK SUB C. MEMBER CHECKS CONT. 43

BRACE CHECK: HSS 6x6x1/2

[STEEL MANUAL TABLE 5-5] CONTROLLING LOAD CASE: 1.0W

FOR HSS6x6x1/2: YIELDING - $\phi P_n = 403k$
RUPTURE - $\phi P_n = 318k$

FROM RAM OUTPUT → TOTAL STORY SHEAR @ LEVEL 4 = 311.7k / 4 FRAMES = 80 k/FRAME

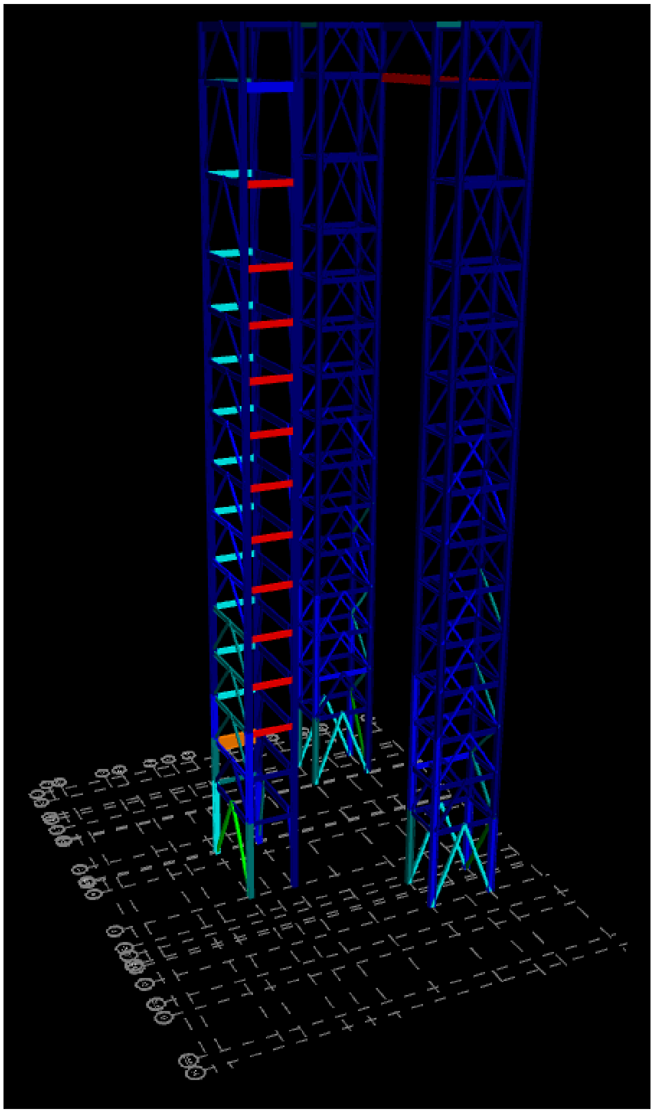


ASSUME TENSION MEMBER RESISTS FULL LOAD

FORCE IN TENSION MEMBER → $80k = F\cos\theta$
$80 = F\cos 50.5$
$F = 126$

$126k << 313k < 403k$ ∴ BRACE IS ADEQUATE

A member analysis check was run in RAM which reveals whether or not member sizes are adequate under various load combinations. The code used to check the members is AISC360-05 LRFD. The results are found below.



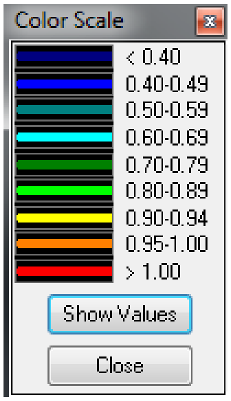
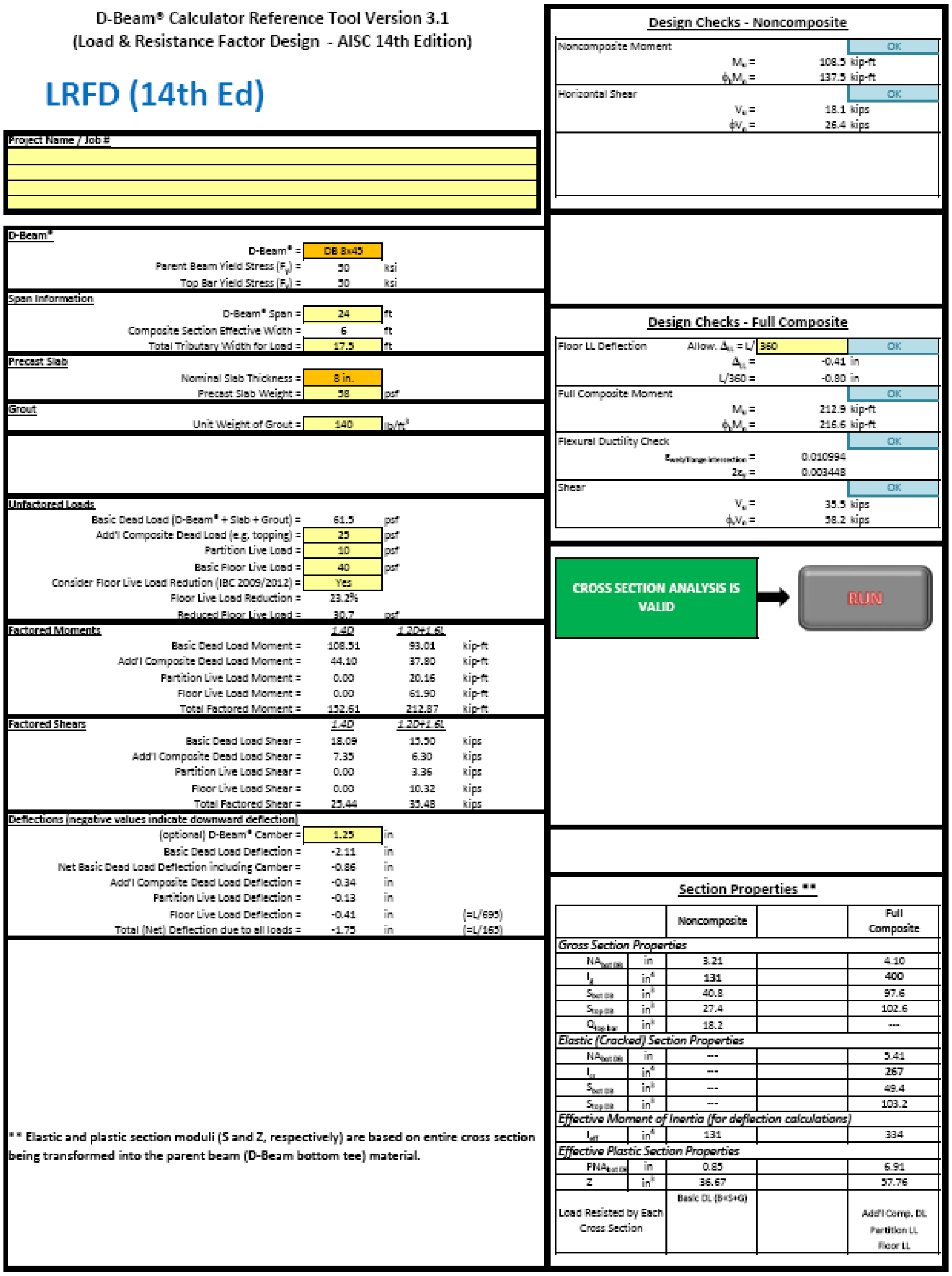


Figure 11: RAM model displaying individual member stresses. This feature allows the user to see which members are failing under loading conditions and by how much.

The color scale to the right shows the % capacity each member is experiencing under loading, with red meaning that the member is failing. With this said, it should be noted that all of the failing members are at 101% capacity, therefore, since approximations were made during the modeling process, all members are considered adequate for analysis purposes.

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/ft^3

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

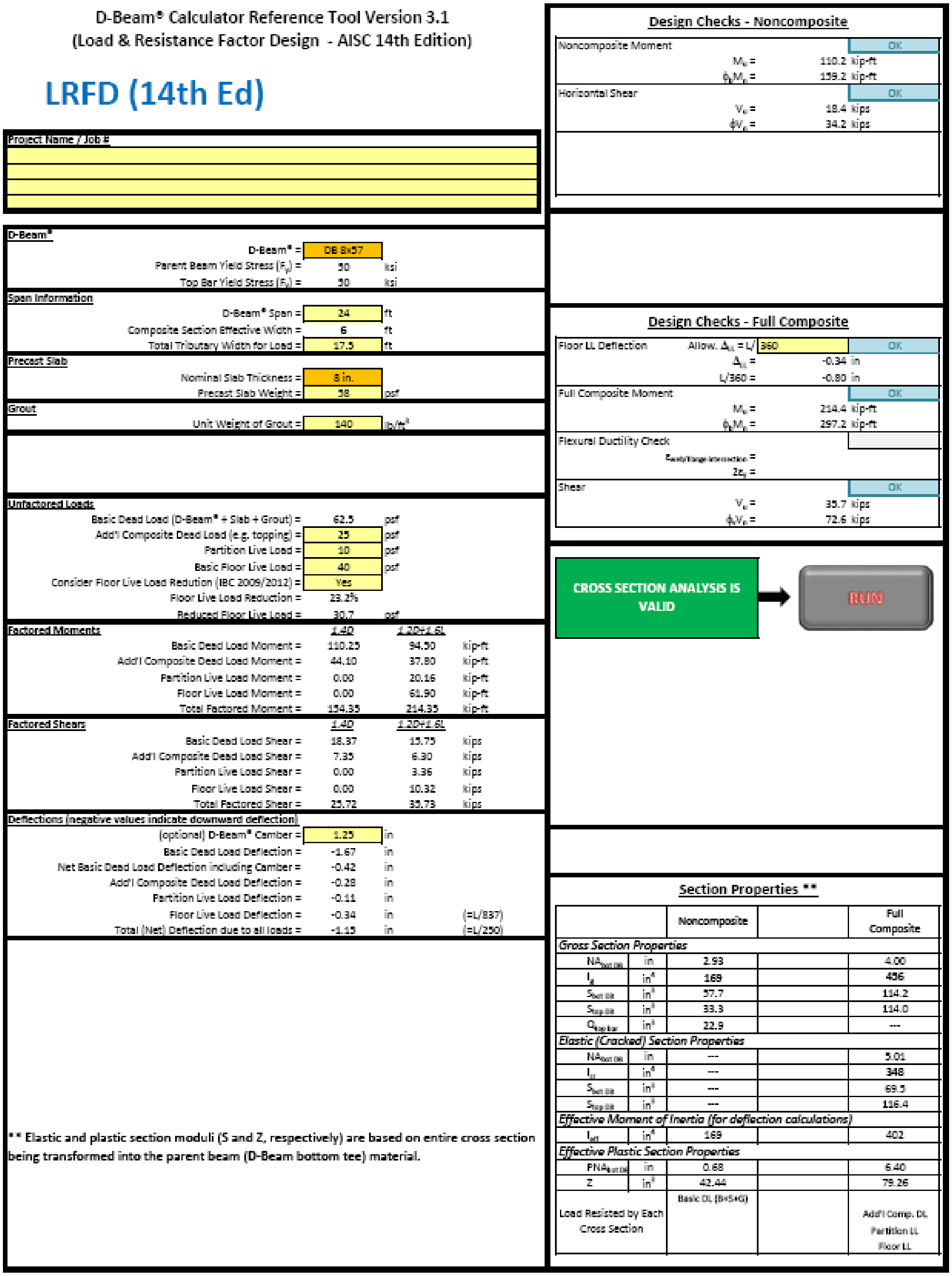
Noncomposite Moment			OK
M_u =	108.5	kip-ft	
$\phi_b M_n$ =	137.5	kip-ft	
Horizontal Shear			OK
V_u =	18.1	kips	
ϕV_n =	26.4	kips	

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	

Section Properties **

		Noncomposite		Full Composite
Gross Section Properties				
$NA_{bot\ DB}$	in	3.21		4.10
I_g	in^4	131		400
$S_{bot\ DB}$	in^3	40.8		97.6
$S_{top\ DB}$	in^3	27.4		102.6
$Q_{top\ bar}$	in^3	18.2		---
Elastic (Cracked) Section Properties				
$NA_{bot\ DB}$	in	---		5.41
I_{cr}	in^4	---		267
$S_{bot\ DB}$	in^3	---		49.4
$S_{top\ DB}$	in^3	---		103.2
Effective Moment of Inertia (for deflection calculations)				
I_{eff}	in^4	131		334
Effective Plastic Section Properties				
$PNA_{bot\ DB}$	in	0.85		6.91
Z	in^3	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/ft³

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
$\varepsilon_{web/flange\ intersection}$ =
$2\varepsilon_y$ =

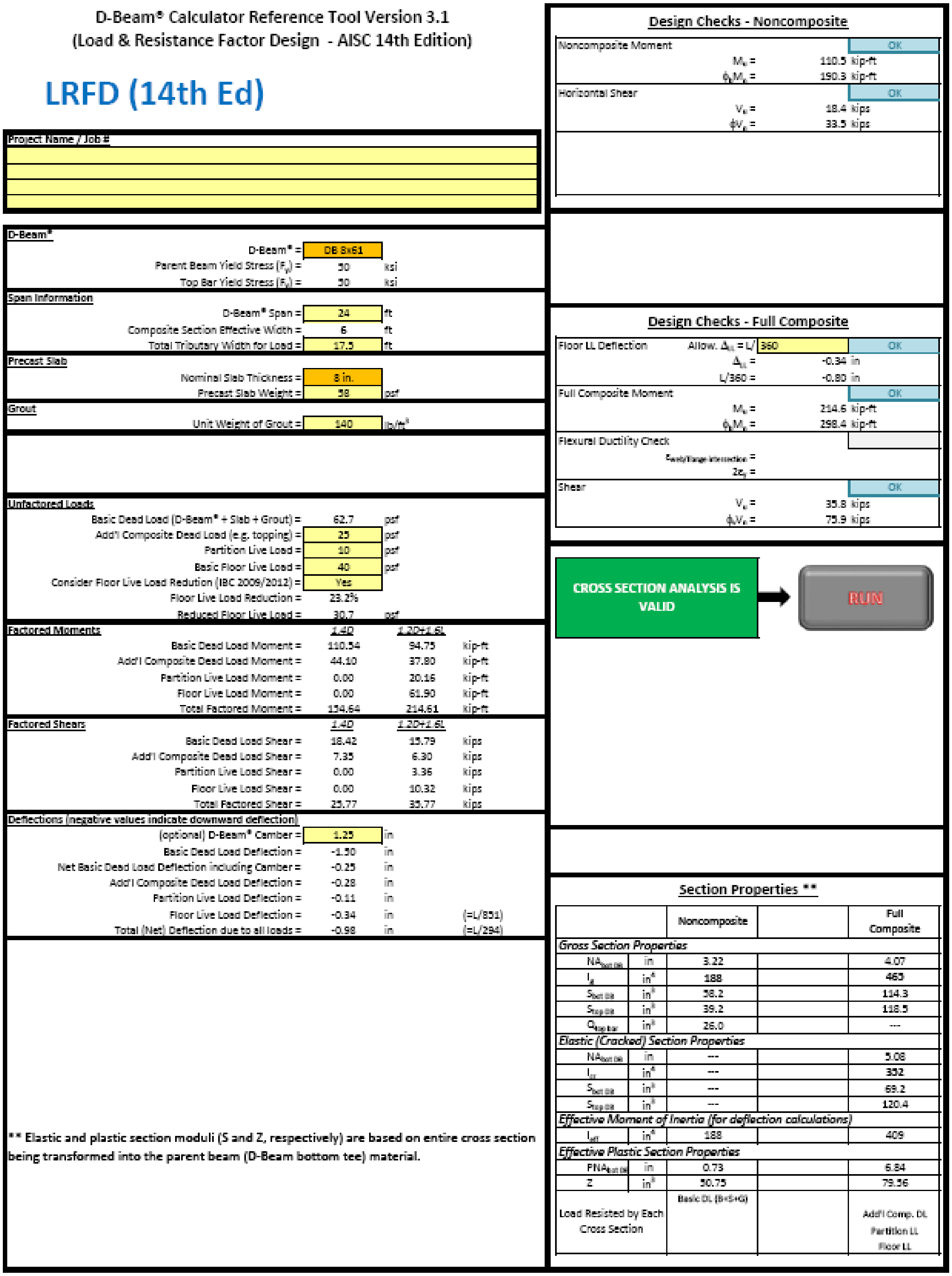
Shear OK
V_u = 35.7 kips
$\phi_v V_n$ = 72.6 kips

CROSS SECTION ANALYSIS IS VALID

RUN

Section Properties **

		Noncomposite		Full Composite
Gross Section Properties				
$NA_{bot\ DB}$	in	2.93		4.00
I_g	in⁴	169		436
$S_{bot\ DB}$	in³	57.7		114.2
$S_{top\ DB}$	in³	33.3		114.0
$Q_{top\ bar}$	in³	22.9		---
Elastic (Cracked) Section Properties				
$NA_{bot\ DB}$	in	---		5.01
I_{cr}	in⁴	---		348
$S_{bot\ DB}$	in³	---		69.5
$S_{top\ DB}$	in³	---		116.4
Effective Moment of Inertia (for deflection calculations)				
I_{eff}	in⁴	169		402
Effective Plastic Section Properties				
$PNA_{bot\ DB}$	in	0.68		6.40
Z	in³	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

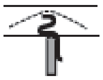
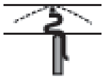
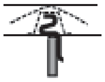
RUN

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

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

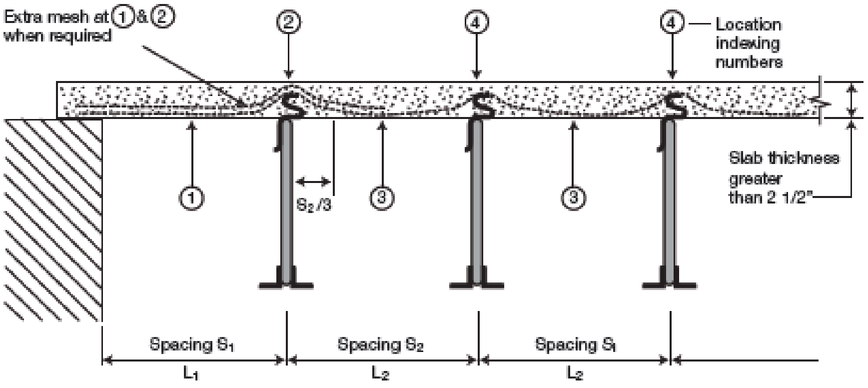
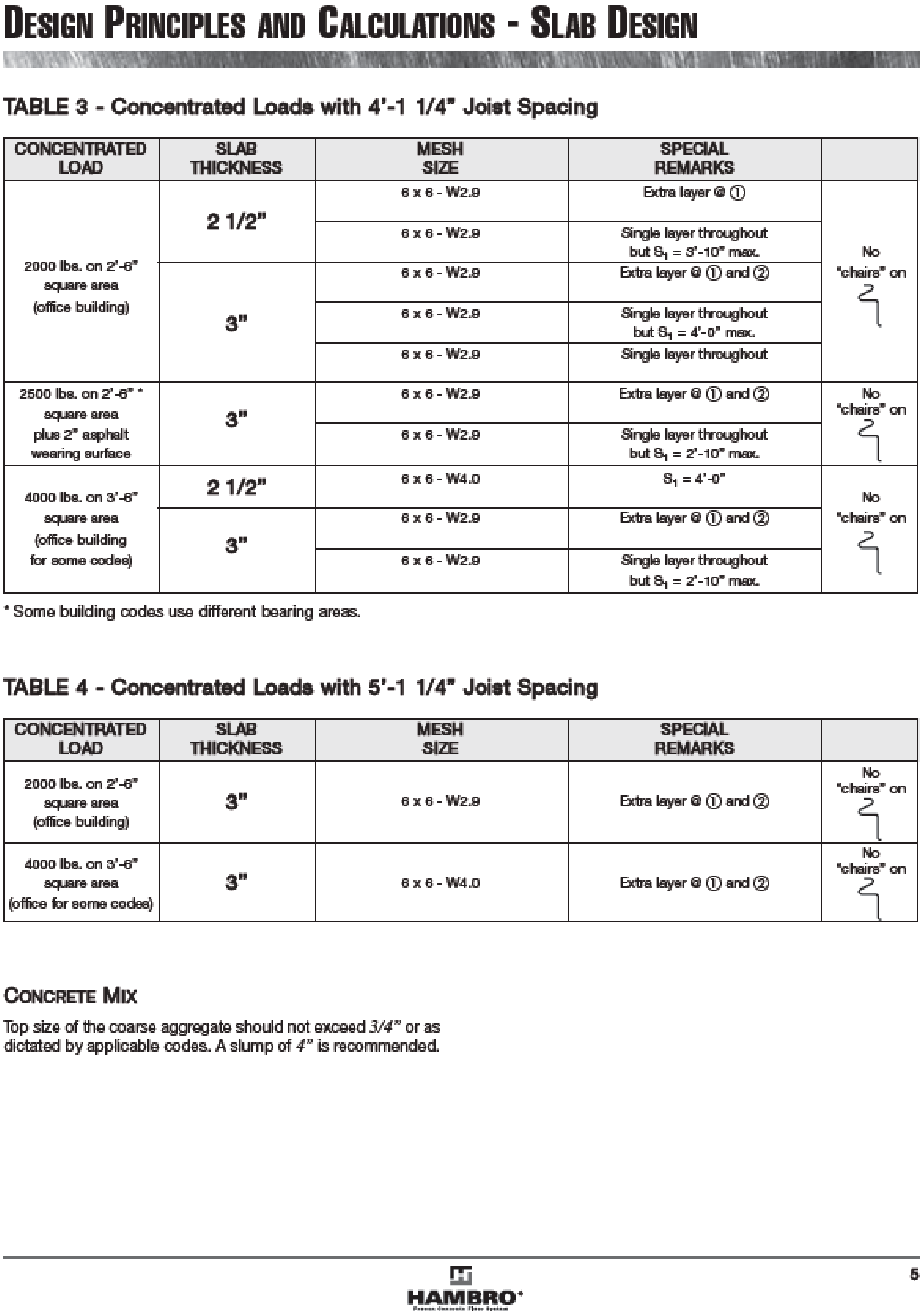


Fig. 2



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

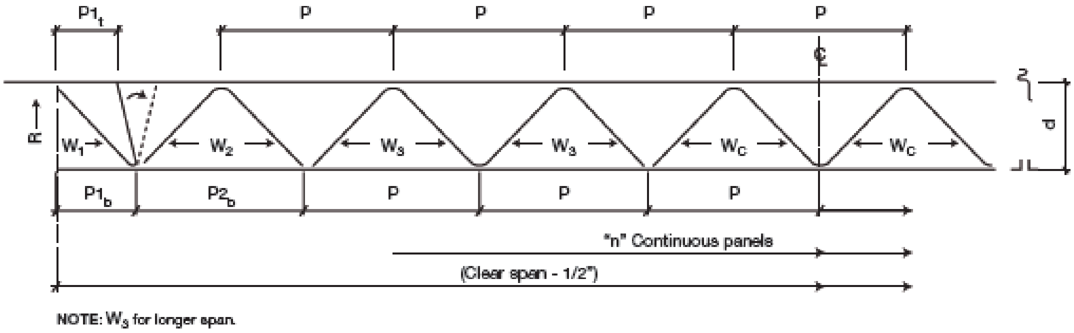
VERTICAL SHEAR (WEB DESIGN)

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.

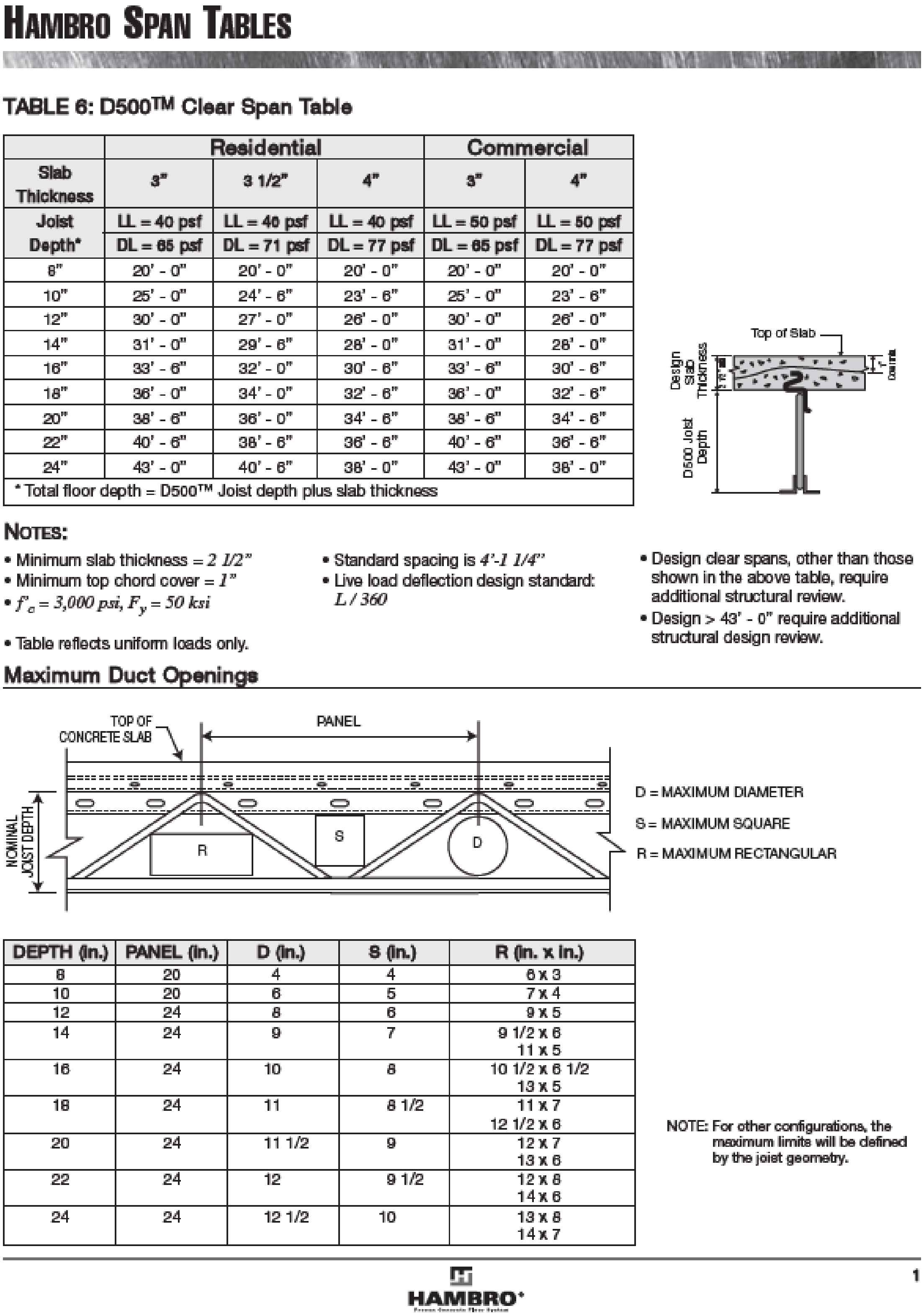
It is important to note that the web members are sized for the specified load capacity including concentrated loads where applicable. Furthermore, the webs are designed according to the latest requirements of the Steel Joist Institute.



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

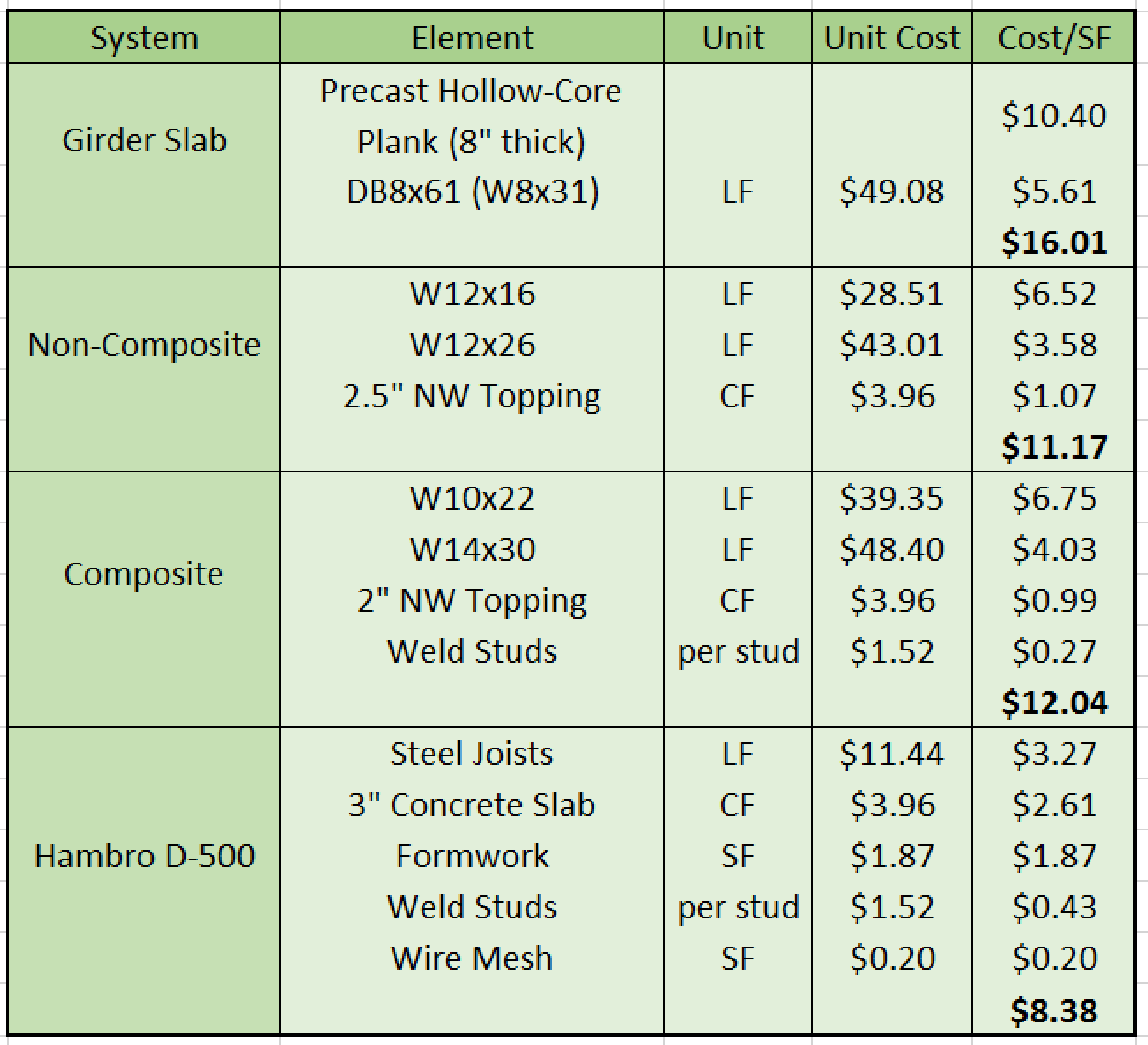
D = MAXIMUM DIAMETER
S = MAXIMUM SQUARE
R = MAXIMUM RECTANGULAR

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.

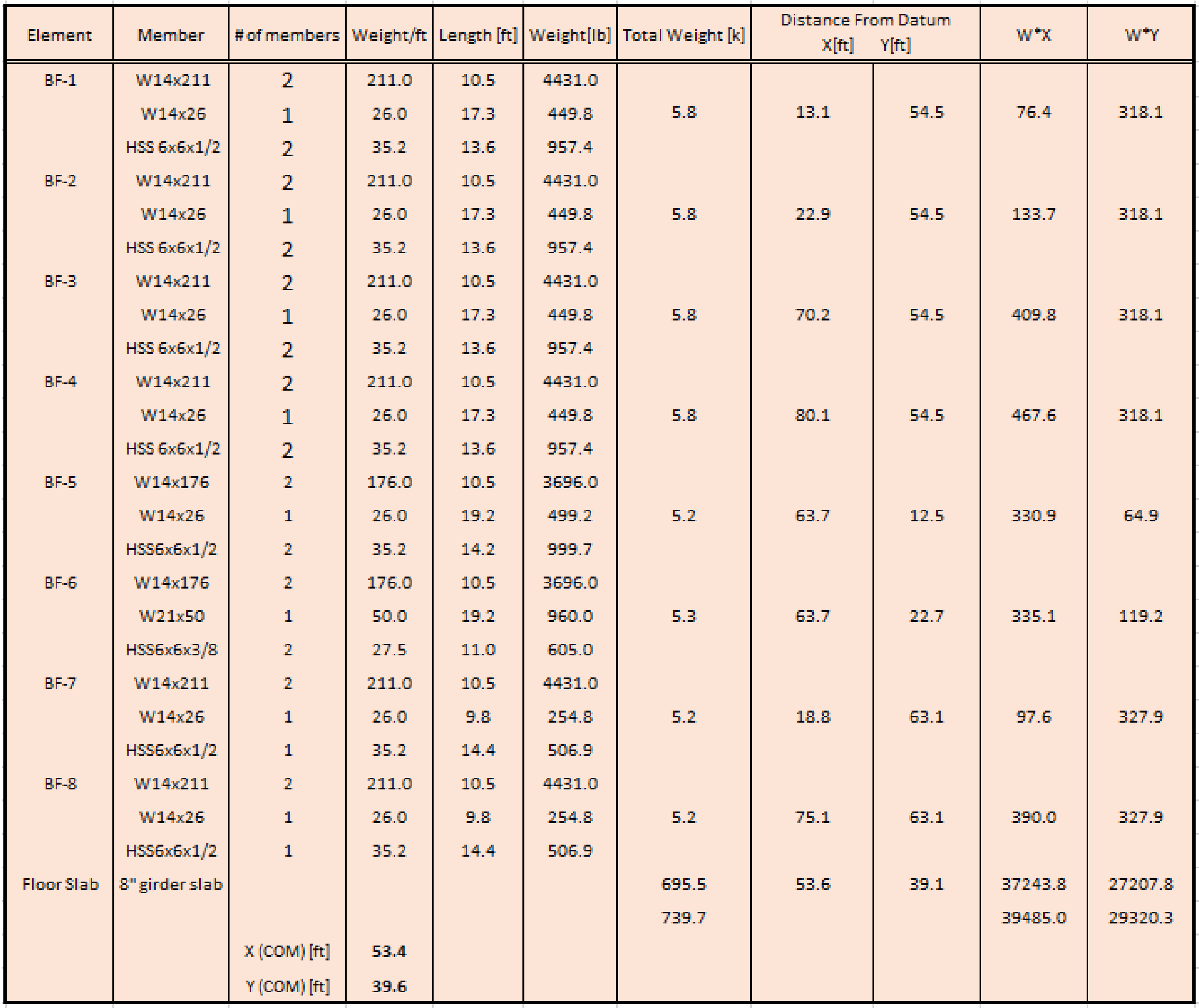
1

HAMBRO®

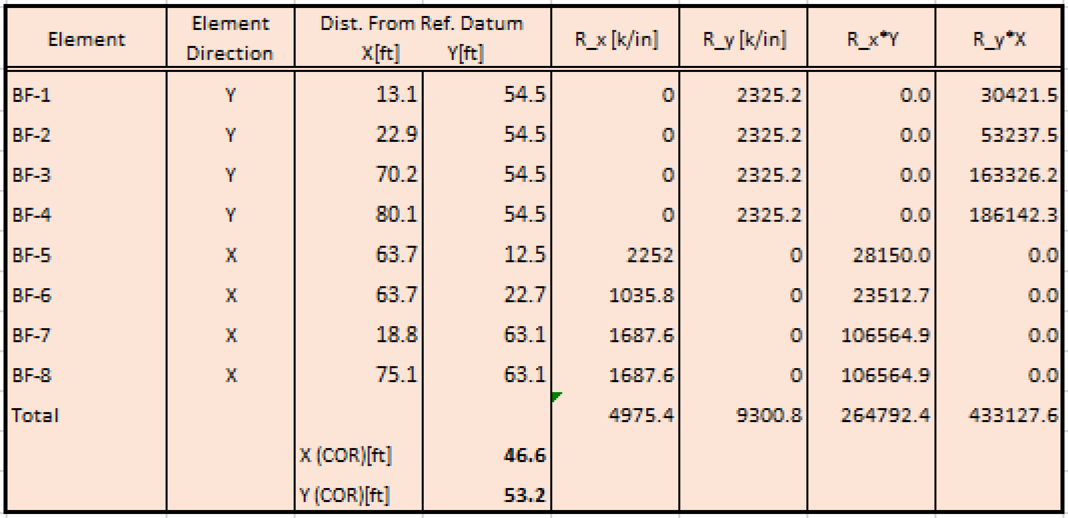


System	Element	Unit	Unit Cost	Cost/SF
Girder Slab	Precast Hollow-Core Plank (8" thick)			$10.40
	DB8x61 (W8x31)	LF	$49.08	$5.61
				$16.01
Non-Composite	W12x16	LF	$28.51	$6.52
	W12x26	LF	$43.01	$3.58
	2.5" NW Topping	CF	$3.96	$1.07
				$11.17
Composite	W10x22	LF	$39.35	$6.75
	W14x30	LF	$48.40	$4.03
	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

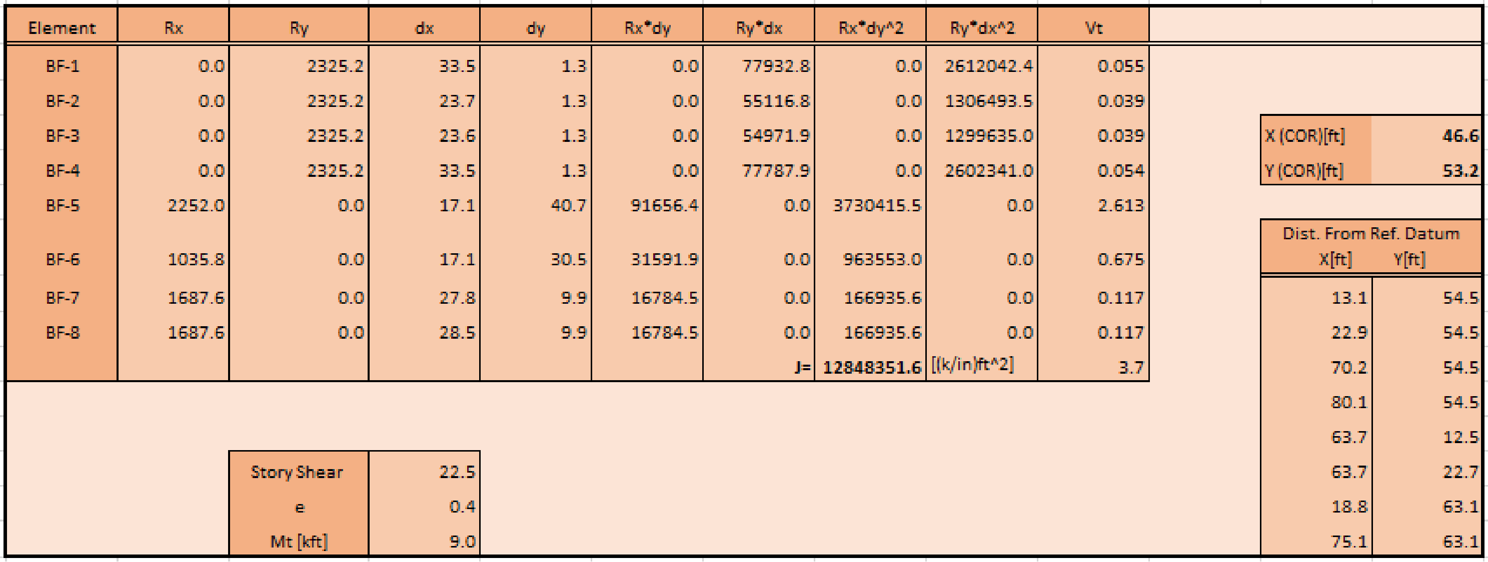
Appendix B



Element	Member	# of members	Weight/ft	Length [ft]	Weight[lb]	Total Weight [k]	Distance From Datum		W*X	W*Y
							X[ft]	Y[ft]		
BF-1	W14x211	2	211.0	10.5	4431.0	5.8	13.1	54.5	76.4	318.1
	W14x26	1	26.0	17.3	449.8					
	HSS 6x6x1/2	2	35.2	13.6	957.4					
BF-2	W14x211	2	211.0	10.5	4431.0	5.8	22.9	54.5	133.7	318.1
	W14x26	1	26.0	17.3	449.8					
	HSS 6x6x1/2	2	35.2	13.6	957.4					
BF-3	W14x211	2	211.0	10.5	4431.0	5.8	70.2	54.5	409.8	318.1
	W14x26	1	26.0	17.3	449.8					
	HSS 6x6x1/2	2	35.2	13.6	957.4					
BF-4	W14x211	2	211.0	10.5	4431.0	5.8	80.1	54.5	467.6	318.1
	W14x26	1	26.0	17.3	449.8					
	HSS 6x6x1/2	2	35.2	13.6	957.4					
BF-5	W14x176	2	176.0	10.5	3696.0	5.2	63.7	12.5	330.9	64.9
	W14x26	1	26.0	19.2	499.2					
	HSS6x6x1/2	2	35.2	14.2	999.7					
BF-6	W14x176	2	176.0	10.5	3696.0	5.3	63.7	22.7	335.1	119.2
	W21x50	1	50.0	19.2	960.0					
	HSS6x6x3/8	2	27.5	11.0	605.0					
BF-7	W14x211	2	211.0	10.5	4431.0	5.2	18.8	63.1	97.6	327.9
	W14x26	1	26.0	9.8	254.8					
	HSS6x6x1/2	1	35.2	14.4	506.9					
BF-8	W14x211	2	211.0	10.5	4431.0	5.2	75.1	63.1	390.0	327.9
	W14x26	1	26.0	9.8	254.8					
	HSS6x6x1/2	1	35.2	14.4	506.9					
Floor Slab	8" girder slab					695.5	53.6	39.1	37243.8	27207.8
						739.7			39485.0	29320.3
		X (COM) [ft]	**53.4**							
		Y (COM) [ft]	**39.6**							



Element	Element Direction	Dist. From Ref. Datum X[ft]	Y[ft]	R_x [k/in]	R_y [k/in]	R_x*Y	R_y*X
BF-1	Y	13.1	54.5	0	2325.2	0.0	30421.5
BF-2	Y	22.9	54.5	0	2325.2	0.0	53237.5
BF-3	Y	70.2	54.5	0	2325.2	0.0	163326.2
BF-4	Y	80.1	54.5	0	2325.2	0.0	186142.3
BF-5	X	63.7	12.5	2252	0	28150.0	0.0
BF-6	X	63.7	22.7	1035.8	0	23512.7	0.0
BF-7	X	18.8	63.1	1687.6	0	106564.9	0.0
BF-8	X	75.1	63.1	1687.6	0	106564.9	0.0
Total				4975.4	9300.8	264792.4	433127.6
		X (COR)[ft]	**46.6**				
		Y (COR)[ft]	**53.2**				



Element	Rx	Ry	dx	dy	Rx*dy	Ry*dx	Rx*dy^2	Ry*dx^2	Vt
BF-1	0.0	2325.2	33.5	1.3	0.0	77932.8	0.0	2612042.4	0.055
BF-2	0.0	2325.2	23.7	1.3	0.0	55116.8	0.0	1306493.5	0.039
BF-3	0.0	2325.2	23.6	1.3	0.0	54971.9	0.0	1299635.0	0.039
BF-4	0.0	2325.2	33.5	1.3	0.0	77787.9	0.0	2602341.0	0.054
BF-5	2252.0	0.0	17.1	40.7	91656.4	0.0	3730415.5	0.0	2.613
BF-6	1035.8	0.0	17.1	30.5	31591.9	0.0	963553.0	0.0	0.675
BF-7	1687.6	0.0	27.8	9.9	16784.5	0.0	166935.6	0.0	0.117
BF-8	1687.6	0.0	28.5	9.9	16784.5	0.0	166935.6	0.0	0.117
						J=	**12848351.6**	[(k/in)ft^2]	3.7

X (COR)[ft]	**46.6**
Y (COR)[ft]	**53.2**

Dist. From Ref. Datum X[ft]	Y[ft]
13.1	54.5
22.9	54.5
70.2	54.5
80.1	54.5
63.7	12.5
63.7	22.7
18.8	63.1
75.1	63.1

Story Shear	22.5
e	0.4
Mt [kft]	9.0