Appendices

Appendix A: Typical Plans

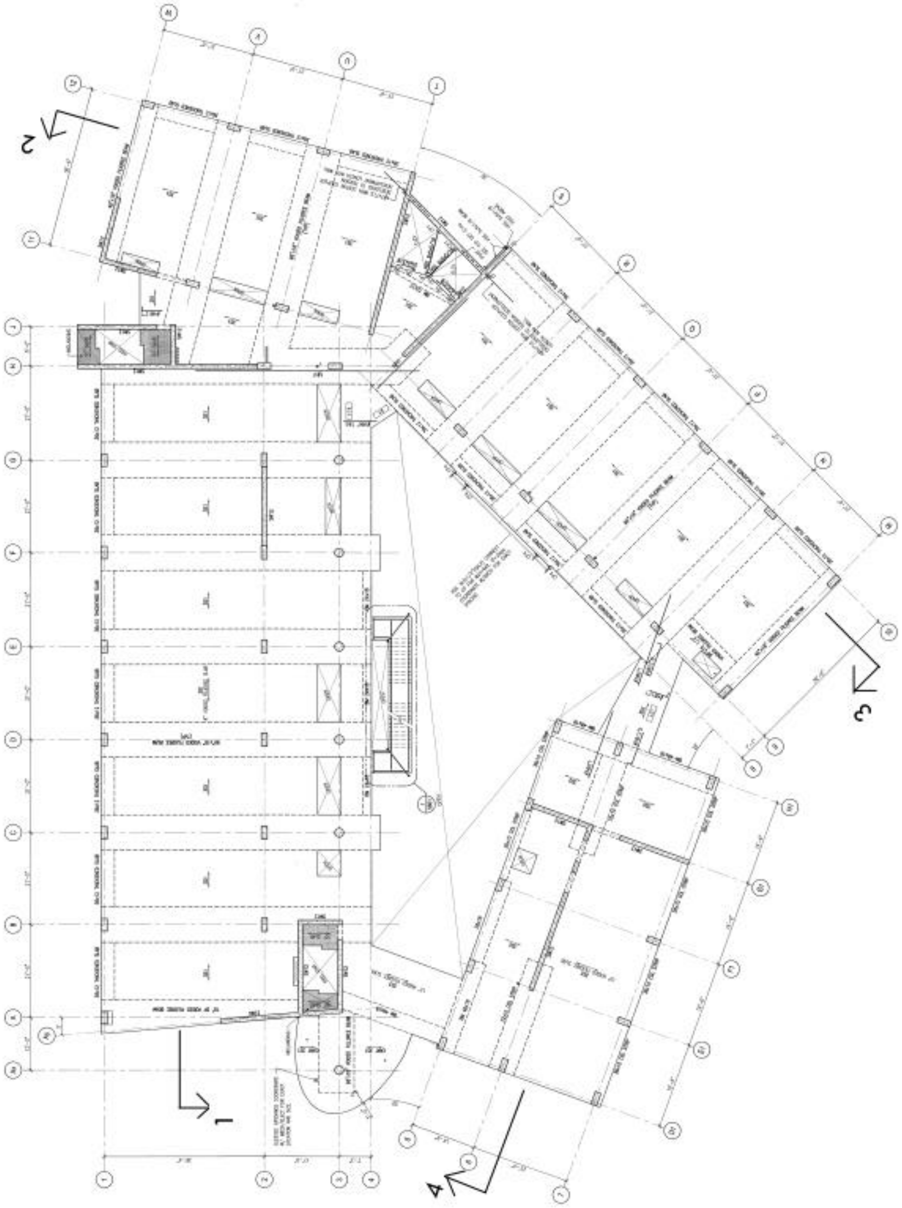


Figure A.1 Typical Floor plan, taken from S202. See following figures for sections indicated on the plan.

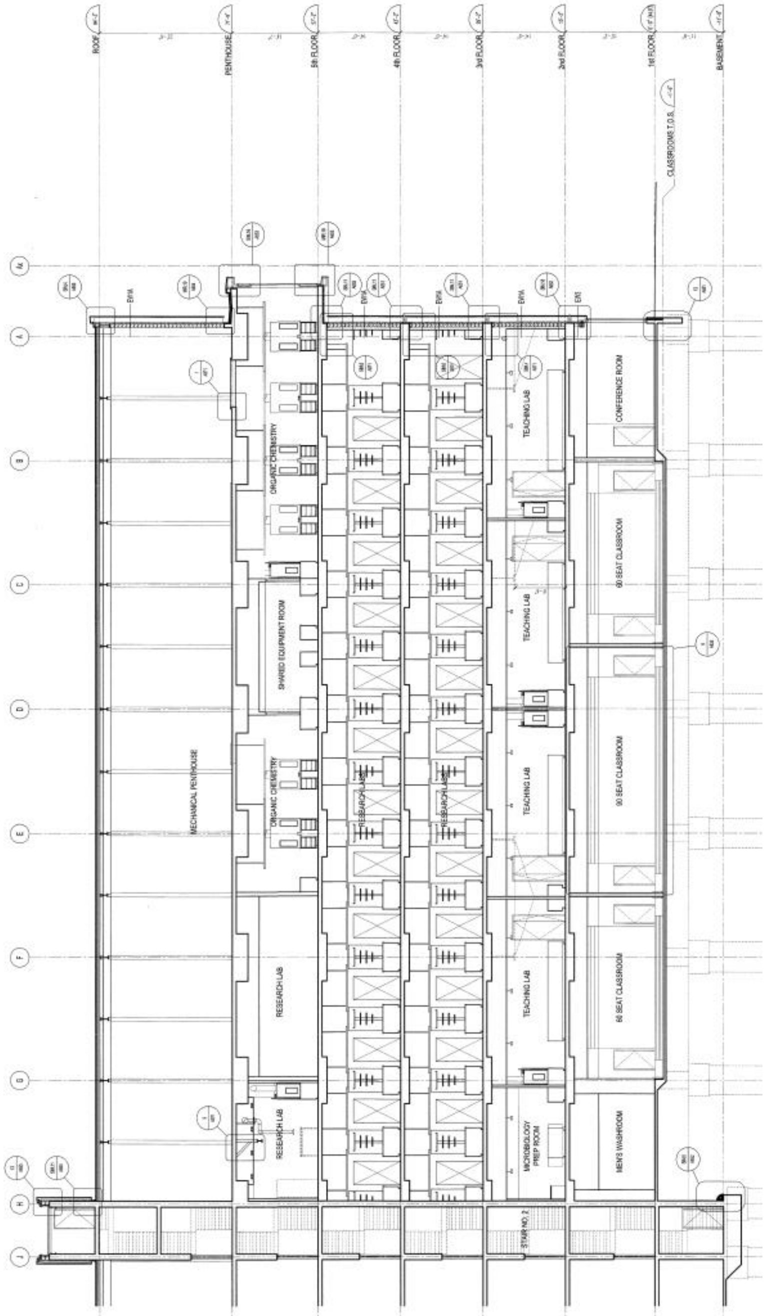


Figure A.2 Section 1 through portion of building at 0° rotation (see Figure 1), taken from 3/A401.

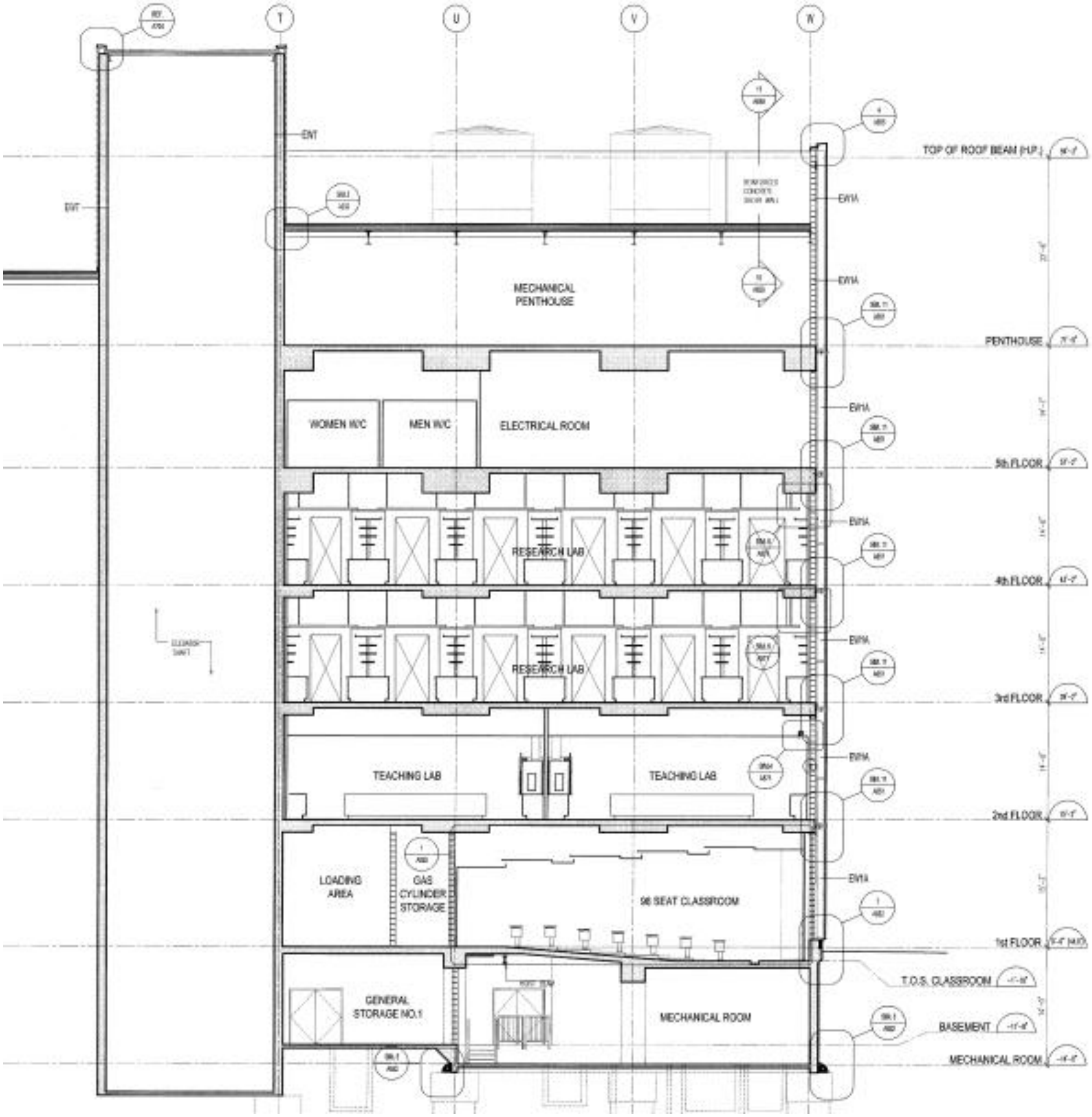


Figure A.3 Section 2 through portion of building at -15° rotation (see Figure 1), taken from 2/A402.

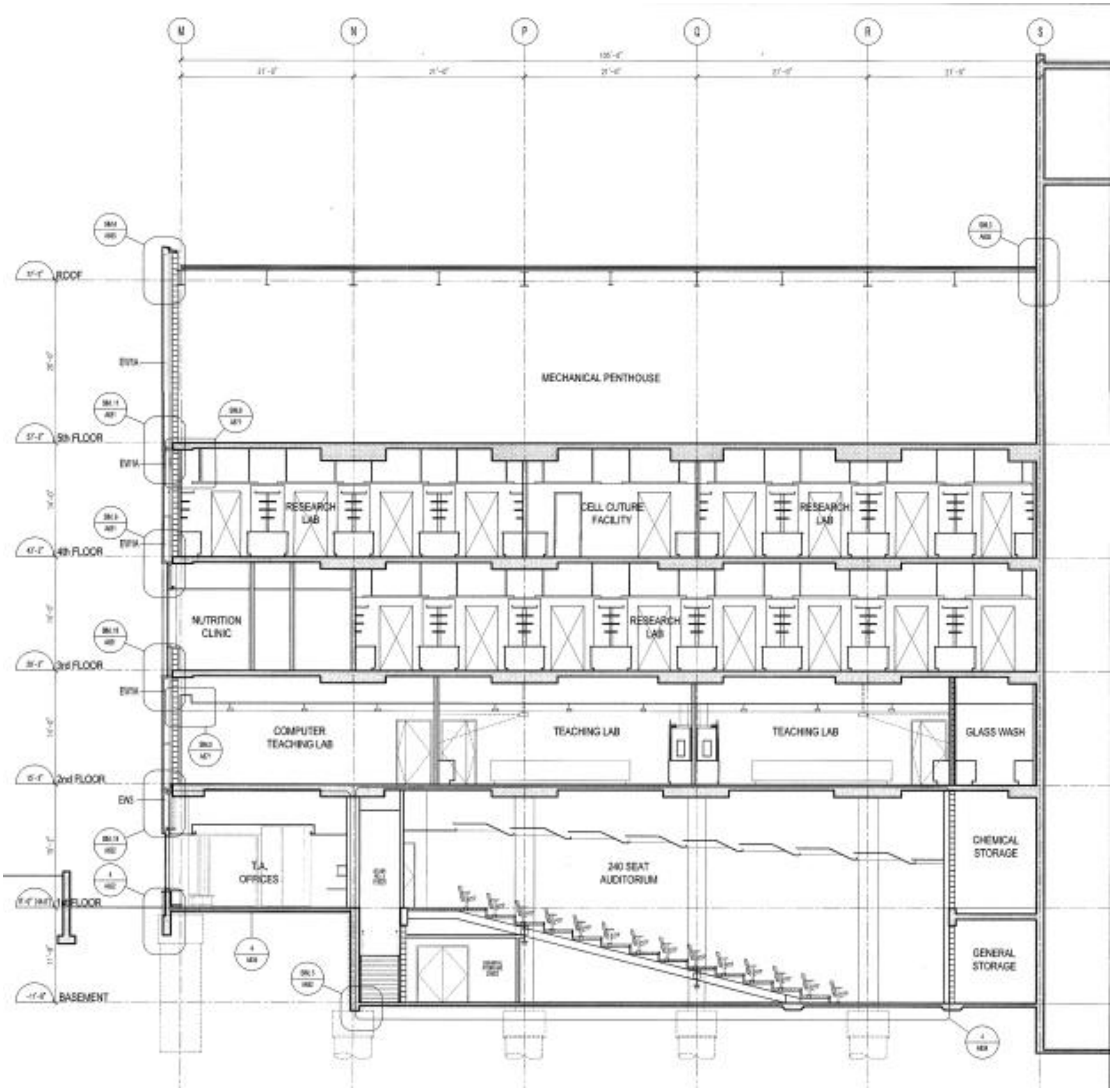


Figure A.4 Section 3 through portion of building at -45° rotation (see Figure 1), taken from 4/A402.

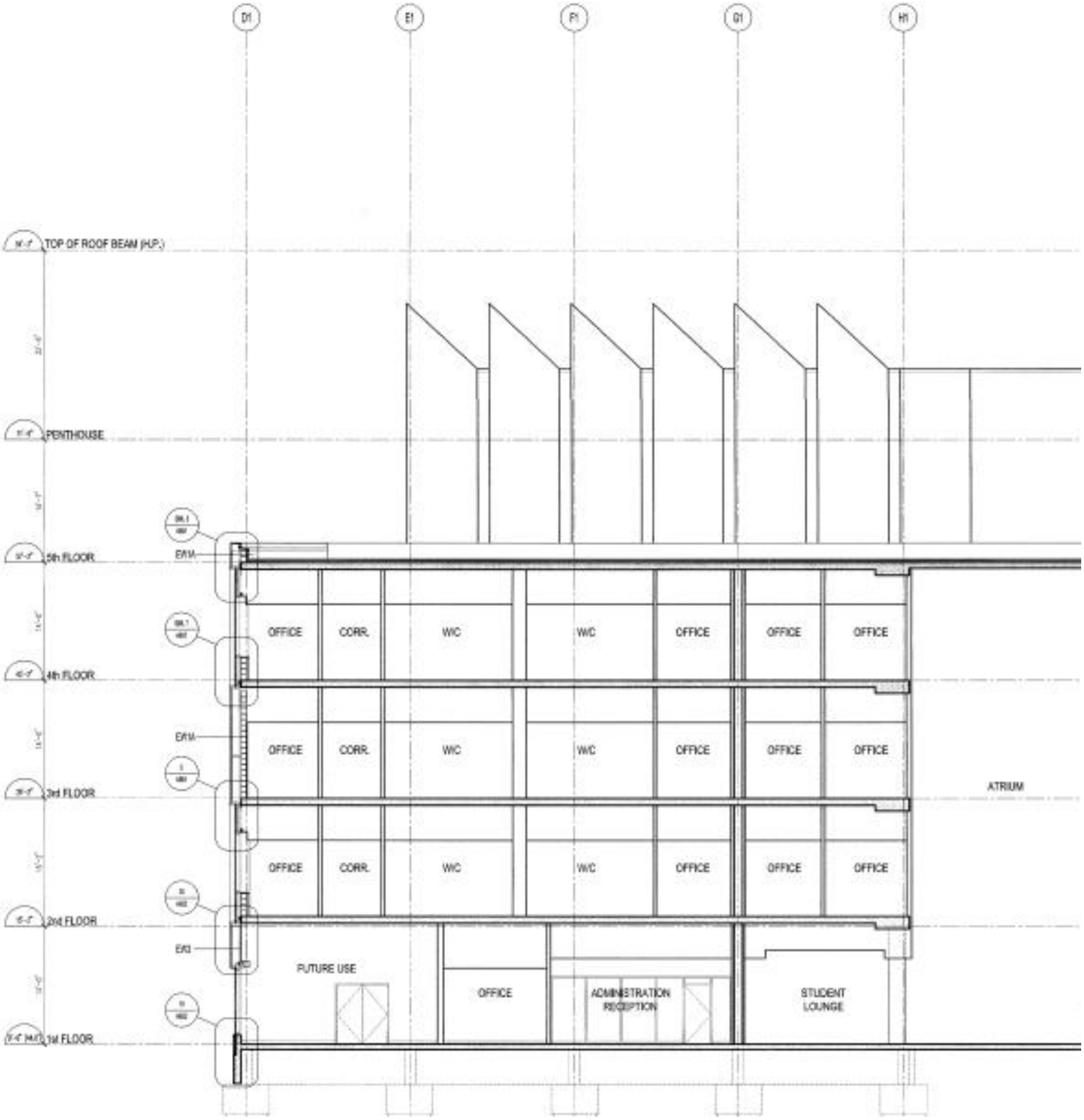
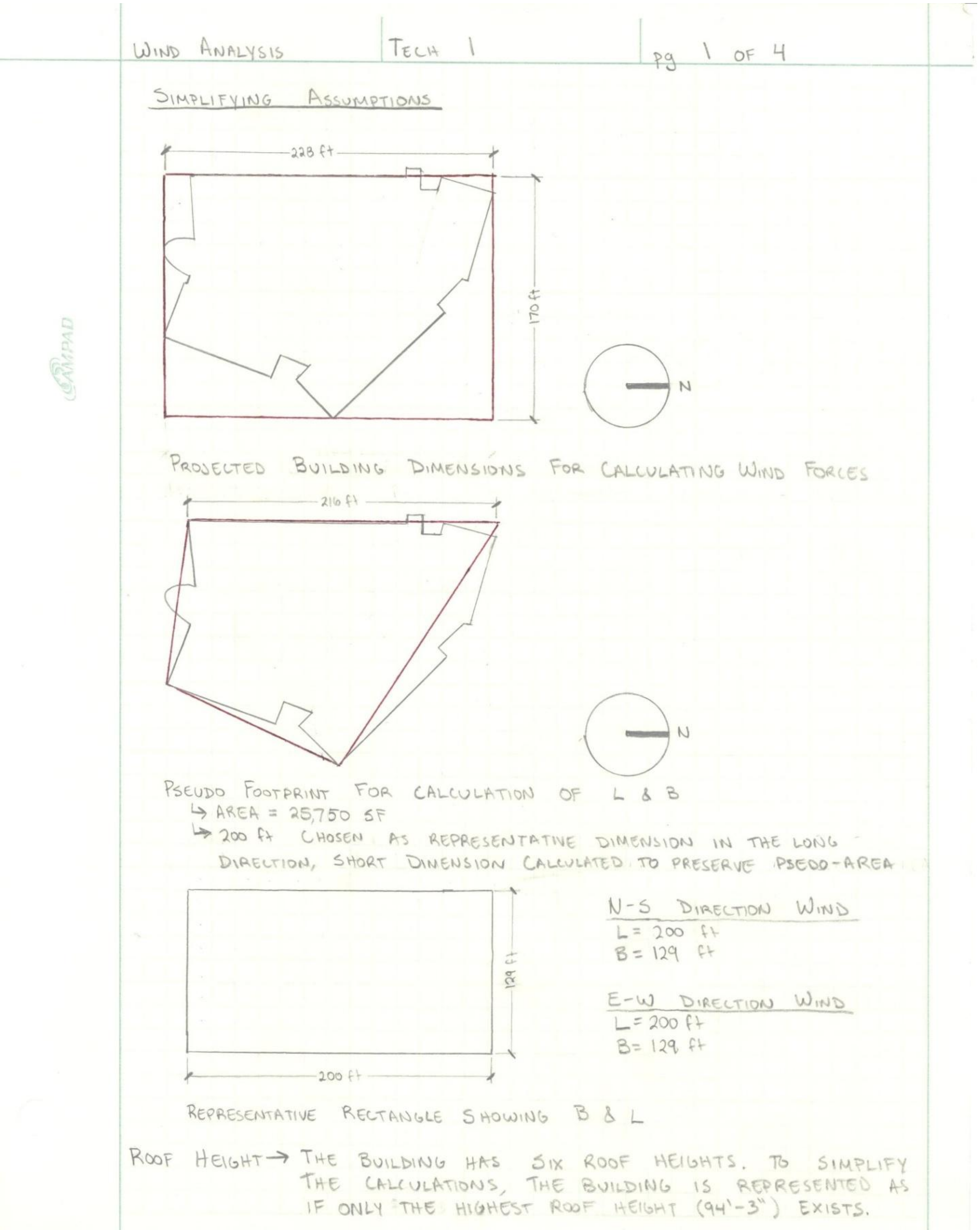


Figure A.5 Section 4 through portion of building at -20° rotation (see Figure 1), taken from 3/A403.

Appendix B: Wind Load Calculations



WIND ANALYSIS | TECH 1 | PG 1 OF 4

SIMPLIFYING ASSUMPTIONS

PROJECTED BUILDING DIMENSIONS FOR CALCULATING WIND FORCES

PSEUDO FOOTPRINT FOR CALCULATION OF L & B

↳ AREA = 25,750 SF

↳ 200 ft CHOSEN AS REPRESENTATIVE DIMENSION IN THE LONG DIRECTION, SHORT DIMENSION CALCULATED TO PRESERVE PSEDO-AREA

N-S DIRECTION WIND

L = 200 ft

B = 129 ft

E-W DIRECTION WIND

L = 200 ft

B = 129 ft

REPRESENTATIVE RECTANGLE SHOWING B & L

ROOF HEIGHT → THE BUILDING HAS SIX ROOF HEIGHTS. TO SIMPLIFY THE CALCULATIONS, THE BUILDING IS REPRESENTED AS IF ONLY THE HIGHEST ROOF HEIGHT (94'-3") EXISTS.

WIND ANALYSIS | TECH 1 | Pg 2 OF 4

USE METHOD 2 SINCE BUILDING (WITH SIMPLIFYING ASSUMPTIONS) MEETS CRITERIA OF 6.5.1 & 6.5.2

BASIC WIND SPEED → USING FIG. 6-1C, $V = 90$ mph

WIND DIRECTIONALITY FACTOR → USING TBL. 6-4, $K_d = 0.85$

OCCUPANCY CATEGORY → USING TBL 1-1, III
↳ COLLEGE FACILITY WITH MORE THAN 500 PERSON CAPACITY

IMPORTANCE FACTOR → USING TBL 6-1, $I = 1.15$

EXPOSURE CATEGORY → USING SECTION 6.5.6.3, B
↳ DUE TO URBAN SURROUNDINGS

TOPOGRAPHIC FACTOR → FROM SECTION 6.5.7.1, $K_{zt} = 1.0$

VELOCITY PRESSURE COEFFICIENTS → FROM TBL 6-3, VARIES
↳ SEE EXCEL SPREADSHEET

VELOCITY PRESSURES →

$$\left.\begin{array}{l} q_z = 0.00256\ K_z\ K_{zT}\ K_d\ V^2\ I \\ q_h = 0.00256\ K_h\ K_{zT}\ K_d\ V^2\ I \end{array}\right\rangle \text{ SEE EXCEL SPREADSHEET}$$

GUST EFFECT FACTOR

$n_1 = \frac{75}{94} = 0.798$ (LOWER BOUND FROM C6-17)

$n_1 = \frac{100}{94} = 1.064$ (AVERAGE VALUE FROM C6-18) ✱ USE FOR CALCS

BOTH VALUES ARE CLOSE TO 1.0 Hz, SO CALCULATE G_F IN THE EVENT THE BUILDING IS FLEXIBLE.

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2 \ln(3600(1.064))} + \frac{0.577}{\sqrt{2 \ln(3600(1.064))}} = 4.204$$

$\bar{z} = 0.6\,h = 0.6\,(94) = 56.4$ ft $> \bar{z}_{min} = 30$ ft OK ✓

FROM TBL. 6-2, $\bar{\alpha} = 1/4.0$, $\bar{b} = 0.45$, $c = 0.30$, $\ell = 320$ ft, & $\bar{\epsilon} = 1/3.0$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{56.4}\right)^{1/6} = 0.274$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{56.4}{33}\right)^{1/3} = 382.59$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{56.4}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 67.92$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{1.064(382.59)}{67.92} = 5.99$$

WIND ANALYSIS TECH 1 pg 3 OF 4

$$R_n = \frac{7.47\ N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47\,(5.99)}{(1+10.3(5.99))^{5/3}} = 0.045$$

$\beta = 0.010$ (ASSUMED, CONSERVATIVE FOR CONCRETE SHEAR WALLS)

NORTH-SOUTH

$h = 94$ ft
$L = 200$ ft
$B = 129$ ft

$$\eta_h = 4.6\,n_1\,{}^{h}/_{\bar{V}_{\bar z}} = 4.6(1.064)\,{}^{94}/_{67.92}$$
$$= 6.77$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$$
$$= \frac{1}{6.77} - \frac{1}{2(6.77)^2}\left(1-e^{-2(6.77)}\right) = 0.137$$

$$\eta_B = 4.6\,n_1\,{}^{B}/_{\bar{V}_{\bar z}} = 4.6(1.064)\left(\frac{129}{67.92}\right)$$
$$= 9.296$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$$
$$= \frac{1}{9.296} - \frac{1}{2(9.296)^2}\left(1-e^{-2(9.296)}\right) = 0.102$$

$$\eta_L = 15.4\,n_1\,{}^{L}/_{\bar{V}_{\bar z}} = 15.4(1.064)\left(\frac{200}{67.92}\right)$$
$$= 48.252$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$$
$$= \frac{1}{48.252} - \frac{1}{2(48.252)^2}\left(1-e^{-2(48.252)}\right) = 0.021$$

$$R = \sqrt{{}^{1}/_{\beta}\, R_n R_h R_B\,(0.53+0.47R_L)}$$
$$= \sqrt{\frac{1}{0.01}(0.045)(0.137)(0.102)[0.53+0.47(0.021)]}$$
$$= 0.184$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{129+94}{382.59}\right)^{0.63}}} = 0.831$$

$$G_f = 0.925\left(\frac{1+1.7I_{\bar z}\sqrt{g_Q^2\,Q^2+g_R^2R^2}}{1+1.7g_v I_{\bar z}}\right)$$

$$= 0.925\left(\frac{1+1.7(0.274)\sqrt{3.4^2(0.831)^2+4.204^2(0.184)^2}}{1+1.7(3.4)(0.274)}\right)$$

$$= \boxed{0.846}$$

EAST-WEST

$h = 94$ ft
$L = 129$ ft
$B = 200$ ft

$\eta_h = 6.77$ (SEE N-S DIRECTION)

$R_h = 0.137$ (SEE N-S DIRECTION)

$$\eta_B = 4.6(1.064)\left(\frac{200}{67.92}\right)$$
$$= 14.413$$

$$R_B = \frac{1}{14.413} - \frac{1}{2(14.413)^2}\left(1-e^{-2(14.413)}\right)$$
$$= 0.067$$

$$\eta_L = 15.4(1.064)\left(\frac{129}{67.92}\right)$$
$$= 31.123$$

$$R_L = \frac{1}{31.123} - \frac{1}{2(31.123)^2}\left(1-e^{-2(31.123)}\right)$$
$$= 0.032$$

$$R = \sqrt{\frac{1}{0.01}(0.045)(0.137)(0.067)[0.57+0.43(0.032)]}$$
$$= 0.150$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{200+94}{382.59}\right)^{0.63}}} = 0.808$$

$$G_f = 0.925\left(\frac{1+1.7(0.274)\sqrt{3.4^2(0.808)^2+4.204^2(0.150)^2}}{1+1.7(3.4)(0.274)}\right)$$

$$= \boxed{0.828}$$

WIND ANALYSIS | TECH 1 | pg 4 of 4

BUILDING IS FULLY ENCLOSED

SOME AREAS HAVE A PARADET → WILL BE DISREGARDED DUE TO ASSUMPTION OF UNIFORM ROOF HEIGHT

$$p_p = q_p(G C_p - GC_{pi})$$

WHERE q_p EQUALS q_z AT THE HEIGHT OF THE PARAPET

EXTERNAL PRESSURE COEFFICIENTS → FROM FIG. 6-6

WALLS → WINDWARD $C_p = 0.8$
LEEWARD C_p ⇒ INTERPOLATE BASED ON L/B VALUES
SIDE $C_p = -0.7$

ROOF → $\theta = 0°$
INTERPOLATE C_p's FOR h/L VALUES

$$\frac{h}{2} = \frac{94}{2} = 47 \text{ ft}$$
$$h = 94 \text{ ft}$$
$$2h = 188 \text{ ft}$$

ROOF AREA → $216 \times 119 = 25{,}704 \text{ ft}^2 > 1000 \text{ SF}$
↳ REDUCTION FACTOR = 0.8

INTERNAL PRESSURE COEFFICIENTS → FROM FIG. 6-5, $GC_{pi} = \pm 0.18$

<u>DESIGN WIND PRESSURES</u>

WINDWARD WALLS → $p_z = q_z G_f C_p - q_h (GC_{pi})$

$$\left.\begin{array}{l}\text{LEEWARD WALLS}\\ \text{SIDE WALLS}\\ \text{ROOF}\end{array}\right\rangle \quad p_h = q_h (G_f C_p - GC_{pi})$$

PARAPET → $p_p = q_p(G_f C_p - GC_{pi})$

Northeast USA Site

General Wind Load Design Criteria		
Design Wind Speed	90 mph	ASCE 7-05, Fig. 6-1C
Directionality Factor (K_d)	0.85	ASCE 7-05, Fig. 6-4
Importance Factor (I_w)	1.15	ASCE 7-05, Tbl. 6-1
Exposure Category	B	ASCE 7-05, Sect. 6.5.6.3
Topographic Factor (K_{zt})	1.0	ASCE 7-05, Sect. 6.5.7.1
Internal Pressure Coefficient (GC_{pi})	0.18	ASCE 7-05, Fig. 6-5

Velocity Pressure Coefficients (K_z) and Velocity Pressures (q_z)

Level	Elevation (ft)	K_z	q_z
Ground	0.00	0.570	11.55
2nd	15.17	0.572	11.59
3rd	29.17	0.693	14.05
4th	43.17	0.776	15.73
5th	57.17	0.839	17.00
Penthouse	71.75	0.897	18.18
Roof	94.25	0.972	19.70

Building Dimensions

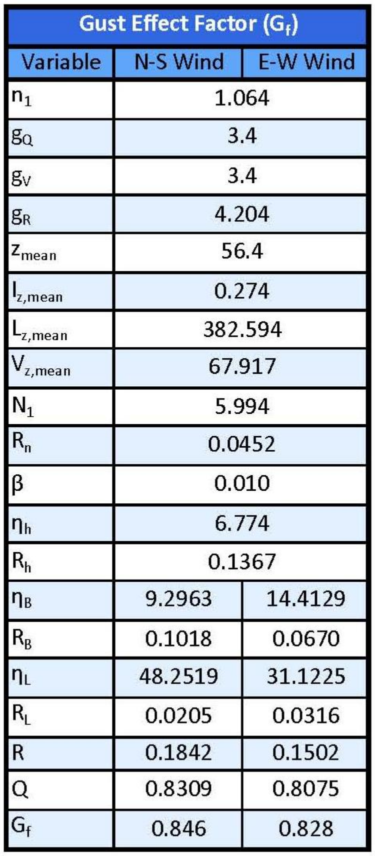
*	N-S Wind	E-W Wind
B (ft)	129	200
L (ft)	200	129
h (ft)	94	94
W (ft)	170	228

*B= normal to wind direction

L= parallel to wind direction

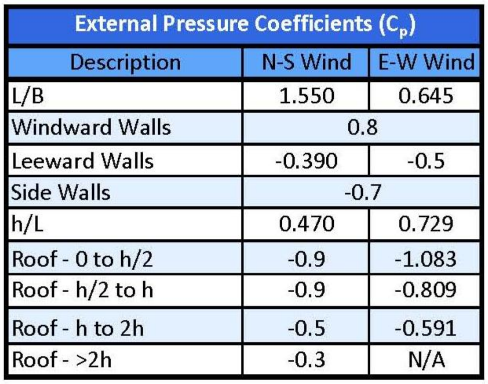
h= mean roof height

W= Length of face used to calculate wind pressures



Gust Effect Factor (G_f)

Variable	N-S Wind	E-W Wind
n_1	1.064	
g_Q	3.4	
g_V	3.4	
g_R	4.204	
z_{mean}	56.4	
$I_{z,mean}$	0.274	
$L_{z,mean}$	382.594	
$V_{z,mean}$	67.917	
N_1	5.994	
R_n	0.0452	
β	0.010	
η_h	6.774	
R_h	0.1367	
η_B	9.2963	14.4129
R_B	0.1018	0.0670
η_L	48.2519	31.1225
R_L	0.0205	0.0316
R	0.1842	0.1502
Q	0.8309	0.8075
G_f	0.846	0.828



External Pressure Coefficients (C_p)

Description	N-S Wind	E-W Wind
L/B	1.550	0.645
Windward Walls	0.8	
Leeward Walls	-0.390	-0.5
Side Walls	-0.7	
h/L	0.470	0.729
Roof - 0 to h/2	-0.9	-1.083
Roof - h/2 to h	-0.9	-0.809
Roof - h to 2h	-0.5	-0.591
Roof - >2h	-0.3	N/A

California Site

General Wind Load Design Criteria		
Design Wind Speed	85 mph	ASCE 7-05, Fig. 6-1C
Directionality Factor (K_d)	0.85	ASCE 7-05, Fig. 6-4
Importance Factor (I_w)	1.15	ASCE 7-05, Tbl. 6-1
Exposure Category	B	ASCE 7-05, Sect. 6.5.6.3
Topographic Factor (K_{zt})	1.0	ASCE 7-05, Sect. 6.5.7.1
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5th	57.17	0.839	15.16
Penthouse	71.75	0.897	16.22
Roof	94.25	0.972	17.57

Building Dimensions		
*	N-S Wind	E-W Wind
B (ft)	129	200
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h (ft)	94	94
W (ft)	170	228

*B= normal to wind direction
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h= mean roof height
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g_Q	3.4	
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g_R	4.204	
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R_n	0.0452	
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η_h	6.774	
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R_B	0.1018	0.0670
η_L	48.2519	31.1225
R_L	0.0205	0.0316
R	0.1842	0.1502
Q	0.8309	0.8075
G_f	0.846	0.828

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Description	N-S Wind	E-W Wind
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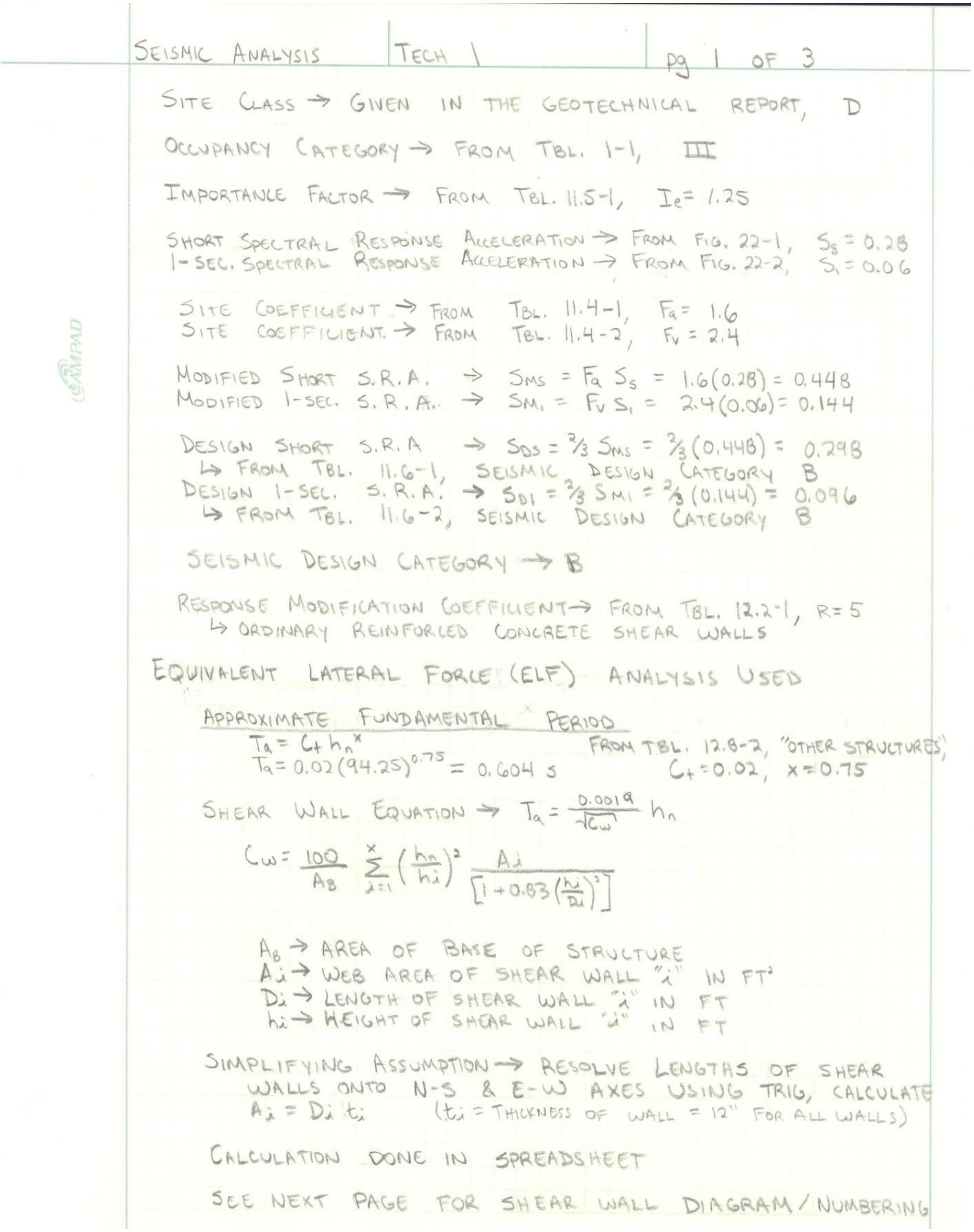
Side Walls	-0.7	
h/L	0.470	0.729
Roof - 0 to h/2	-0.9	-1.083
Roof - h/2 to h	-0.9	-0.809
Roof - h to 2h	-0.5	-0.591
Roof - >2h	-0.3	N/A

Appendix C: Seismic Load Calculations

Original Structure



SEISMIC ANALYSIS | TECH 1 | PG 1 OF 3

SITE CLASS → GIVEN IN THE GEOTECHNICAL REPORT, D

OCCUPANCY CATEGORY → FROM TBL. 1-1, III

IMPORTANCE FACTOR → FROM TBL. 11.5-1, $I_e = 1.25$

SHORT SPECTRAL RESPONSE ACCELERATION → FROM FIG. 22-1, $S_S = 0.28$
1-SEC. SPECTRAL RESPONSE ACCELERATION → FROM FIG. 22-2, $S_1 = 0.06$

SITE COEFFICIENT → FROM TBL. 11.4-1, $F_a = 1.6$
SITE COEFFICIENT → FROM TBL. 11.4-2, $F_v = 2.4$

MODIFIED SHORT S.R.A. → $S_{MS} = F_a S_s = 1.6(0.28) = 0.448$
MODIFIED 1-SEC. S.R.A. → $S_{M1} = F_v S_1 = 2.4(0.06) = 0.144$

DESIGN SHORT S.R.A → $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.448) = 0.298$
↳ FROM TBL. 11.6-1, SEISMIC DESIGN CATEGORY B
DESIGN 1-SEC. S.R.A. → $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.144) = 0.096$
↳ FROM TBL. 11.6-2, SEISMIC DESIGN CATEGORY B

SEISMIC DESIGN CATEGORY → B

RESPONSE MODIFICATION COEFFICIENT → FROM TBL. 12.2-1, R=5
↳ ORDINARY REINFORCED CONCRETE SHEAR WALLS

EQUIVALENT LATERAL FORCE (ELF) ANALYSIS USED

APPROXIMATE FUNDAMENTAL PERIOD

$T_a = C_t h_n^x$ FROM TBL. 12.8-2, "OTHER STRUCTURES",
$T_a = 0.02(94.25)^{0.75} = 0.604\ s$ $C_t = 0.02$, $x = 0.75$

SHEAR WALL EQUATION → $T_a = \frac{0.0019}{\sqrt{C_w}} h_n$

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

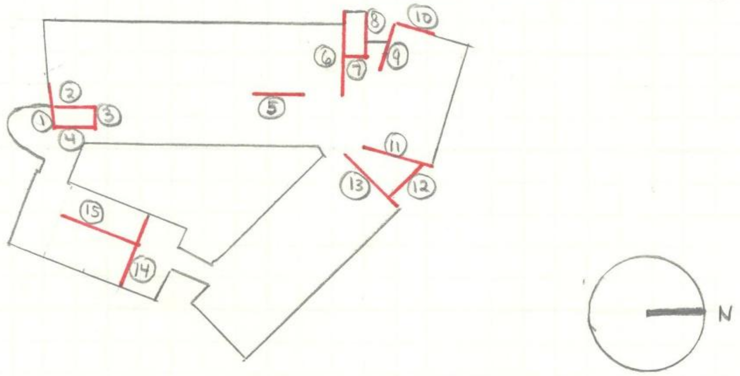
A_B → AREA OF BASE OF STRUCTURE
A_i → WEB AREA OF SHEAR WALL "i" IN FT²
D_i → LENGTH OF SHEAR WALL "i" IN FT
h_i → HEIGHT OF SHEAR WALL "i" IN FT

SIMPLIFYING ASSUMPTION → RESOLVE LENGTHS OF SHEAR WALLS ONTO N-S & E-W AXES USING TRIG, CALCULATE
$A_i = D_i t_i$ (t_i = THICKNESS OF WALL = 12" FOR ALL WALLS)

CALCULATION DONE IN SPREADSHEET

SEE NEXT PAGE FOR SHEAR WALL DIAGRAM / NUMBERING

SEISMIC ANALYSIS | TECH 1 | PG 2 OF 3



SHEAR WALL LOCATIONS & NUMBERS

PER SECT. 12.8.2, IT IS PERMITTED TO MULTIPLY T_a BY A COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD, C_u

FROM TBL 12.8-1, FOR $S_{D1} \leq 0.1$, $C_u = 1.7$

$C_u T_a$'S USED TO CALCULATE SEISMIC RESPONSE COEFFICIENTS (C_s)

$$C_{s,CALC} = \frac{S_{DS}}{(R/I_e)}$$

$$C_{s,max} = \begin{cases} \dfrac{S_{D1}}{T(R/I)} & \text{FOR } T \leq T_L \\ \dfrac{T_L S_{D1}}{T^2 (R/I)} & \text{FOR } T > T_L \end{cases}$$

$$C_{s,min} = 0.01$$

CALCULATION DONE VIA SPREADSHEET

THE BASIC SOLUTION IS CONTROLLED BY $C_{s,max}$ FOR BOTH DIRECTIONS. THIS WILL RESULT IN IDENTICAL BASE SHEARS IN BOTH DIRECTIONS, WHICH DIFFERS FROM THE STRUCTURAL DRAWINGS.

THE SPECIFIC SOLUTION IS CONTROLLED BY $C_{s,max}$ IN BOTH N-S & E-W DIRECTIONS, BUT THE TWO VALUES ARE DIFFERENT. IT WILL PRODUCE A HIGHER BASE SHEAR IN THE E-W DIRECTION, WHICH WILL MIMIC THE CONDITION NOTED IN THE STRUCTURAL DRAWINGS. BOTH C_s VALUES FROM THE SPECIFIC SOLUTION EXCEED THE C_s VALUE FROM THE BASIC SOLUTION.

USE THE SPECIFIC SOLUTION C_s VALUES TO BE CONSERVATIVE AND BECAUSE THEY SEEM MORE REALISTIC.

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BASE SHEAR

$V = C_s W$

W = WEIGHT OF BUILDING (CALCULATED IN A SPREADSHEET)
= 30,482 K

$C_{s,N\text{-}S} = 0.0308$

$C_{s,E\text{-}W} = 0.0359$

$V_{N\text{-}S} = 0.0308\ (30{,}482) = 939\ K$
↳ 955 K IN STRUCTURAL DRAWINGS → ~1.7% LOW OK ✓

$V_{E\text{-}W} = 0.359\ (30{,}482) = 1{,}095\ K$
↳ 1145 K IN STRUCTURAL DRAWINGS → ~4.4% LOW OK ✓

STORY FORCES

BASE SHEAR IS DISTRIBUTED TO EACH LEVEL BY THE EQUATION:

$F_x = C_{vx} V$

WHERE $C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ (VERTICAL DISTRIBUTION FACTOR)

w = WEIGHT OF STORY
h = HEIGHT OF STORY ABOVE GROUND
$k = 1 + \frac{T-0.5}{2}$ $(1 \leq k \leq 2)$

$K_{N\text{-}S} = 1 + \frac{0.7792 - 0.5}{2} = 1.1396$

$K_{E\text{-}W} = 1 + \frac{0.6684 - 0.5}{2} = 1.0842$

NOTE: USED $T = C_u T_a$

CALCULATION COMPLETED BY SPREADSHEET

General Seismic Design Criteria

Site Class	D	Geotechnical Report
Importance Factor (I_E)	1.25	ASCE 7-05, Tbl. 11.5-1
Short Spectral Response Acceleration (S_s)	0.28	ASCE 7-05, Fig. 22-1
1-sec. Spectral Response Acceleration (S_1)	0.06	ASCE 7-05, Fig. 22-2
Site Coefficient (F_a)	1.6	ASCE 7-05, Tbl. 11.4-1
Site Coefficient (F_v)	2.4	ASCE 7-05, Tbl. 11.4-2
Response Modification Coefficient (R)	5	ASCE 7-05, Tbl. 12.2-1
Long-Period Transition Period	6 s	ASCE 7-05, Fig. 22-15

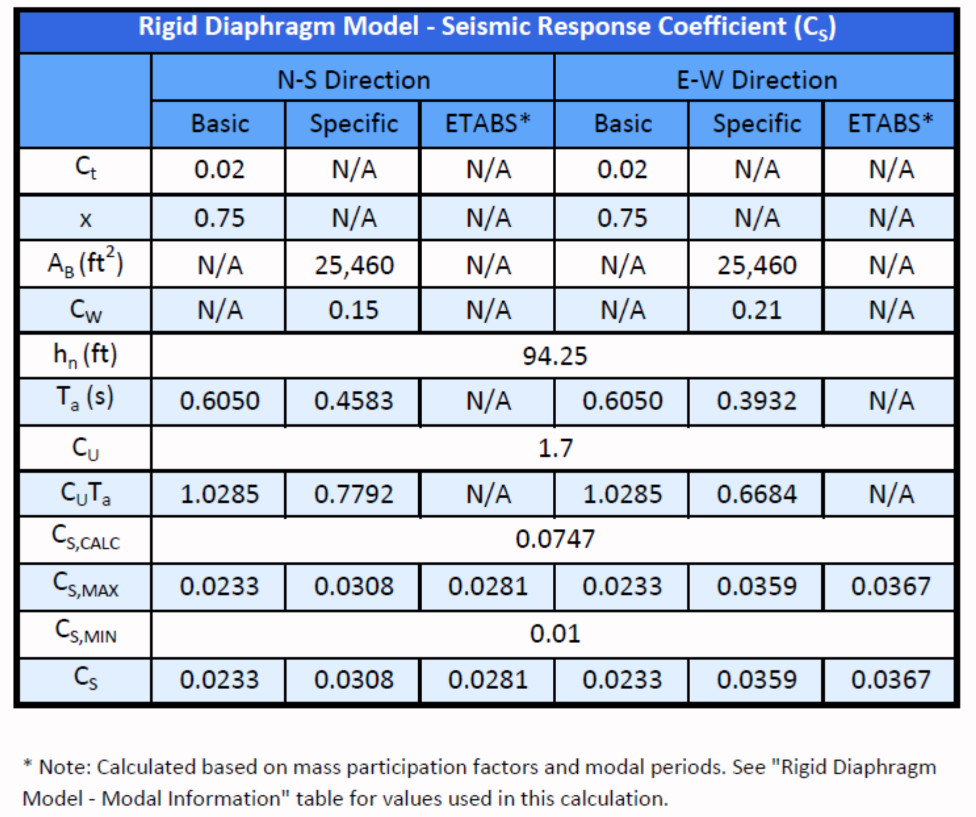
Seismic Design Parameters

Description	Value
Modified Short S.R.A. (S_{MS})	0.448
Modified 1-sec. S.R.A (S_{M1})	0.144
Design Short S.R.A. (S_{DS})	0.2987
Design 1-sec. S.R.A. (S_{D1})	0.0960
Seismic Design Category	B

Shear Wall Data

Shear Wall Number	Length (ft)	Angle with NS-axis (deg)	Height (ft)	Length in NS-Dir. (ft)	Area in NS-Dir. (ft^2)	Length in EW-Dir. (ft)	Area in EW-Dir. (ft^2)
1	40	95	94.25	3.49	3.49	39.85	39.85
2	20	0	94.25	20.00	20.00	0.00	0.00
3	8	90	94.25	0.00	0.00	8.00	8.00
4	20	0	94.25	20.00	20.00	0.00	0.00
5	18	0	71.75	18.00	18.00	0.00	0.00
6	48	90	104.25	0.00	0.00	48.00	48.00
7	8	0	104.25	8.00	8.00	0.00	0.00
8	24	90	104.25	0.00	0.00	24.00	24.00
9	18	-105	71.75	4.66	4.66	17.39	17.39
10	13	-15	71.75	12.56	12.56	3.36	3.36
11	30	-15	94.25	28.98	28.98	7.76	7.76
12	25	45	94.25	17.68	17.68	17.68	17.68
13	34	-45	85.75	24.04	24.04	24.04	24.04
14	38	-110	57.17	13.00	13.00	35.71	35.71
15	35	-20	57.17	32.89	32.89	11.97	11.97

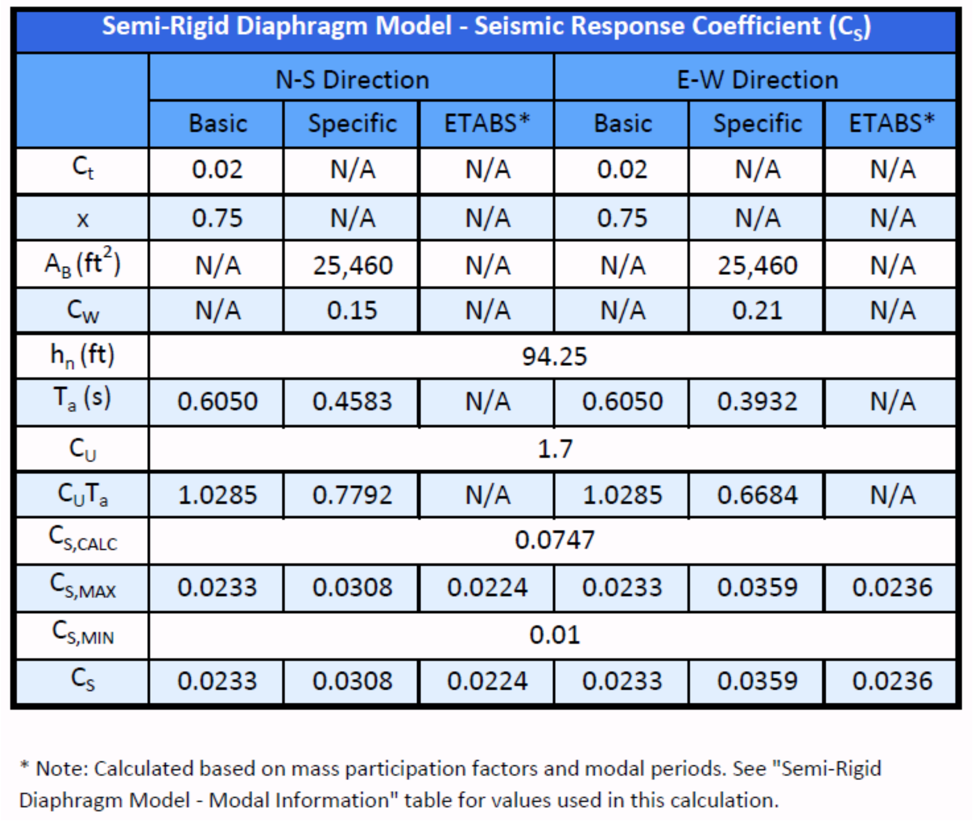
Note: "Areas" are web areas, A="Length of Wall"x"Thickness of Wall". All shear walls are 1'-0" thick



Rigid Diaphragm Model - Seismic Response Coefficient (C_S)

	N-S Direction			E-W Direction		
	Basic	Specific	ETABS*	Basic	Specific	ETABS*
C_t	0.02	N/A	N/A	0.02	N/A	N/A
x	0.75	N/A	N/A	0.75	N/A	N/A
A_B (ft^2)	N/A	25,460	N/A	N/A	25,460	N/A
C_W	N/A	0.15	N/A	N/A	0.21	N/A
h_n (ft)	94.25					
T_a (s)	0.6050	0.4583	N/A	0.6050	0.3932	N/A
C_U	1.7					
C_UT_a	1.0285	0.7792	N/A	1.0285	0.6684	N/A
$C_{S,CALC}$	0.0747					
$C_{S,MAX}$	0.0233	0.0308	0.0281	0.0233	0.0359	0.0367
$C_{S,MIN}$	0.01					
C_S	0.0233	0.0308	0.0281	0.0233	0.0359	0.0367

* Note: Calculated based on mass participation factors and modal periods. See "Rigid Diaphragm Model - Modal Information" table for values used in this calculation.



Semi-Rigid Diaphragm Model - Seismic Response Coefficient (C_S)

	N-S Direction			E-W Direction		
	Basic	Specific	ETABS*	Basic	Specific	ETABS*
C_t	0.02	N/A	N/A	0.02	N/A	N/A
x	0.75	N/A	N/A	0.75	N/A	N/A
A_B (ft^2)	N/A	25,460	N/A	N/A	25,460	N/A
C_W	N/A	0.15	N/A	N/A	0.21	N/A
h_n (ft)	94.25					
T_a (s)	0.6050	0.4583	N/A	0.6050	0.3932	N/A
C_U	1.7					
C_UT_a	1.0285	0.7792	N/A	1.0285	0.6684	N/A
$C_{S,CALC}$	0.0747					
$C_{S,MAX}$	0.0233	0.0308	0.0224	0.0233	0.0359	0.0236
$C_{S,MIN}$	0.01					
C_S	0.0233	0.0308	0.0224	0.0233	0.0359	0.0236

* Note: Calculated based on mass participation factors and modal periods. See "Semi-Rigid Diaphragm Model - Modal Information" table for values used in this calculation.

NE USA S-3 Structure

General Seismic Design Criteria		
Site Class	D	Geotechnical Report
Importance Factor (I_E)	1.25	ASCE 7-05, Tbl. 11.5-1
Short Spectral Response Acceleration (S_s)	0.28	ASCE 7-05, Fig. 22-1
1-sec. Spectral Response Acceleration (S_1)	0.06	ASCE 7-05, Fig. 22-2
Site Coefficient (F_a)	1.6	ASCE 7-05, Tbl. 11.4-1
Site Coefficient (F_v)	2.4	ASCE 7-05, Tbl. 11.4-2
Response Modification Coefficient (R)	3	ASCE 7-05, Tbl. 12.2-1
Long-Period Transition Period	6 s	ASCE 7-05, Fig. 22-15

Seismic Design Parameters	
Description	Value
Modified Short S.R.A. (S_{MS})	0.448
Modified 1-sec. S.R.A (S_{M1})	0.144
Design Short S.R.A. (S_{DS})	0.2987
Design 1-sec. S.R.A. (S_{D1})	0.0960
Seismic Design Category	B

Seismic Response Coefficient (C_S)		
	N-S Direction	E-W Direction
	Basic	Basic
C_t	0.02	0.02
x	0.75	0.75
A_B (ft^2)	N/A	N/A
C_W	N/A	N/A
h_n (ft)	94.25	
T_a (s)	0.6050	0.6050
C_U	1.7	
C_UT_a	1.0285	1.0285
$C_{S,CALC}$	0.1244	
$C_{S,MAX}$	0.0389	0.0389
$C_{S,MIN}$	0.01	
C_S	0.0389	0.0389

CA S-3 Structure

General Seismic Design Criteria		
Site Class	D	Geotechnical Report
Importance Factor (I_E)	1.25	ASCE 7-05, Tbl. 11.5-1
Short Spectral Response Acceleration (S_s)	2	ASCE 7-05, Fig. 22-1
1-sec. Spectral Response Acceleration (S_1)	0.63	ASCE 7-05, Fig. 22-2
Site Coefficient (F_a)	1	ASCE 7-05, Tbl. 11.4-1
Site Coefficient (F_v)	1.5	ASCE 7-05, Tbl. 11.4-2
Response Modification Coefficient (R)	8	ASCE 7-05, Tbl. 12.2-1
Long-Period Transition Period	8 s	ASCE 7-05, Fig. 22-15

Seismic Design Parameters	
Description	Value
Modified Short S.R.A. (S_{MS})	2
Modified 1-sec. S.R.A (S_{M1})	0.945
Design Short S.R.A. (S_{DS})	1.333
Design 1-sec. S.R.A. (S_{D1})	0.630
Seismic Design Category	D

Seismic Response Coefficient (C_S)				
	N-S Direction		E-W Direction	
	ELF	MRSA*	ELF	MRSA*
C_t	0.028	N/A	0.028	N/A
x	0.80	N/A	0.80	N/A
h_n (ft)	94.25			
T_a (s) ‡	1.0631	1.5734	1.0631	1.8947
C_U	1.4			
C_UT_a	1.4884	N/A	1.4884	N/A
$C_{S,CALC}$	0.2083			
$C_{S,MAX}$	0.0661	N/A	0.0661	N/A
$C_{S,MIN}$	0.049			
C_m	N/A	0.0304	N/A	0.0400
$C_{m,MIN}$ ¤	N/A	0.0562	N/A	0.0562
C_S	0.0661	0.0562	0.0661	0.0562

* Note: Calculated using SRSS combination of modal Cm,i values (See "CA S-3 - Modal Information" Table)

‡ Note: For "ELF" solution, this is calculated using ASCE 7-05 equation12.8-7. For "MRSA" Solution, this is calculated using SRSS combination of modal periods (See "CA S-3 - Modal Information" Table)

¤ Note: Per ASCE 7-05, Section 12.9.4, MRSA forces must be at least 85% of ELF forces

CA S-1 Design

General Seismic Design Criteria

Site Class	D	Geotechnical Report
Importance Factor (I_E)	1.25	ASCE 7-05, Tbl. 11.5-1
Short Spectral Response Acceleration (S_s)	2	ASCE 7-05, Fig. 22-1
1-sec. Spectral Response Acceleration (S_1)	0.63	ASCE 7-05, Fig. 22-2
Site Coefficient (F_a)	1	ASCE 7-05, Tbl. 11.4-1
Site Coefficient (F_v)	1.5	ASCE 7-05, Tbl. 11.4-2
Response Modification Coefficient (R)	8	ASCE 7-05, Tbl. 12.2-1
Long-Period Transition Period	8 s	ASCE 7-05, Fig. 22-15

Seismic Design Parameters

Description	Value
Modified Short S.R.A. (S_{MS})	2
Modified 1-sec. S.R.A (S_{M1})	0.945
Design Short S.R.A. (S_{DS})	1.333
Design 1-sec. S.R.A. (S_{D1})	0.630
Seismic Design Category	D

Seismic Response Coefficient (C_S)

	N-S Direction		E-W Direction	
	ELF	MRSA*	ELF	MRSA*
C_t	0.028	N/A	0.028	N/A
x	0.80	N/A	0.80	N/A
h_n (ft)	94.25			
T_a (s) ‡	1.0631	0.9871	1.0631	1.0651
C_U	1.4			
C_UT_a	1.4884	N/A	1.4884	N/A
$C_{S,CALC}$	0.2083			
$C_{S,MAX}$	0.0661	N/A	0.0661	N/A
$C_{S,MIN}$	0.049			
C_m	N/A	0.0442	N/A	0.0520
$C_{m,MIN}$ ¤	N/A	0.0562	N/A	0.0562
C_S	0.0661	0.0562	0.0661	0.0562

* Note: Calculated using SRSS combination of modal Cm,i values (See "CA S-1 - Modal Information" Table)

‡ Note: For "ELF" solution, this is calculated using ASCE 7-05 equation 12.8-7. For "MRSA" Solution, this is calculated using SRSS combination of modal periods (See "CA S-1 - Modal Information" Table)

¤ Note: Per ASCE 7-05, Section 12.9.4, MRSA forces must be at least 85% of ELF forces

Appendix D: Typical Bay Steel Gravity Design

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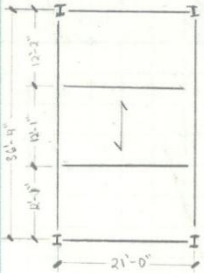
BAY: 2ND LEVEL, BAY 1 (DEPTH LIMIT: W18'S)



LOADS → SDL = 15 psf
LL = 80 psf

DECK → USE VULCRAFT 3VLI COMPOSITE DECK PER TECH 2 INVESTIGATION, USE 3½" TOPPING W/ FIREPROOFED DECK TO ACHIEVE 2 HR RATING

USING CLEAR SPAN OF 12'-6", SUPERIMPOSED LOAD OF 95 psf, & 3-SPAN CONDITION,

USE 3VLI 19 W/ 3.5" TOPPING
↳ MAX CONST. SPAN = 13'-1" OK ✓
↳ MAX ALLOWABLE LOAD = 113 psf OK ✓
↳ SELF WEIGHT = 63 psf

INFILL BEAMS → TRIBUTARY AREA: 21'-0" x 12'-1"
STUDS: 3/4" Ø, ⊥ TO RIB, WEAK POSITION ($Q_n = 17.2k$)

$$LL_R = 0.25 + \left(\frac{15}{\sqrt{\underbrace{2(21')(12.083')}_{507\ ft^2 > 400\ OK ✓}}} \right) = 0.92 \quad > 0.5 \quad OK ✓$$

$$LOAD: \left[1.2(15 + 63 + \overset{\text{ASSUMED S.W.}}{5}) + 1.6(0.92)(80)\right] 12.083\ ft = 2626\ plf$$

$$MOMENT: \frac{w l^2}{8} = \frac{2.626\ klf\,(21')^2}{8} = 145\ k\text{-}ft$$

$$I_{req,LL}: \frac{l}{360} = \frac{5 w_L l^4}{384 E I_{req,LL}} \Rightarrow I_{req,LL} = \frac{360(5)[0.92(0.08\ ksf)(12.083')](21')^3(144\ \frac{in^2}{ft^2})}{384\ (29000\ ksi)}$$
$$= 192\ in^4$$

$$I_{req,TL}: \frac{l}{240} = \frac{5 w_T l^4}{384 I_{req,TL}} \Rightarrow I_{req,TL} = \frac{240(5)[0.083 + 0.92(0.08)](12.083')(21')^3(144\ in^2/ft^2)}{384\ (29000)}$$
$$= 272\ in^4$$

$$I_{req,const}: \frac{l}{240} = \frac{5 w_{const} l^4}{384 E I_{req,beam}} \Rightarrow I_{req,beam} = \frac{240(5)[0.02\ ksf(12.083')](21')^3(144\ in^2/ft^2)}{384\ (29000\ ksi)}$$
$$= 34.7\ in^4$$

USING AISC TABLE 3-19, ASSUMING Y2 = 5.5", TRY:
- W12x14 W/ $\phi M_n = 150$ k-ft, $\Sigma Q_n = 141$ K, $I_{LB} = 309$ in⁴ & $I_{beam} = 88.6$ in⁴
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{141K}{17.2K} = 8.2 \rightarrow n = 18 \quad < 21 \quad OK ✓$
  - ↳ % COMPOSITE → 67.8 %
  - ↳ ECONOMY → 14 plf(21 ft) + 10(10) = 394

MEETS ALL REQUIREMENTS, IS RELATIVELY LIGHT → USE W12x14 W/18 STUDS

GIRDERS → TRIBUTARY AREA : 36'-4" x 21'-0"
STUDS: 3/4" Ø, // TO RIB, $w_r/h_r \geq 1.5$ ($Q_n = 21.5$ K)

$$LL_R = 0.25 + \left(\frac{15}{\sqrt{\underbrace{(2)(21)(36.33')}_{1526\ ft^2 > 400\ OK✓}}}\right) = 0.63 \quad > 0.5 \quad OK✓$$

$$LOAD: [1.2(15+63+5) + 1.6(0.63)(80)]\ 12.083'\ (21') = 45.7\ K$$

$$MOMENT: \frac{P\ell}{3} = \frac{45.7\ K\ (36.33')}{3} = 553\ K\text{-}ft$$

FROM TABLE 3-22a

$$I_{REQ,LL}: \frac{\ell}{360} = \frac{0.036\ P\ell^3}{EI_{req,LL}} \Rightarrow I_{req,LL} = \frac{360(0.036)[0.63(0.08\ ksf)](12.083')(21')(36.33')^2(144\ in^2/ft^2)}{29000\ ksi} = 1086\ in^4$$

$$I_{REQ,TL}: \frac{\ell}{240} = \frac{0.036\ P\ell^3}{EI_{req,TL}} \Rightarrow I_{req,TL} = \frac{240(0.036)[0.083 + 0.63(0.08)](12.083')(21')(36.33)^2(144\ in^2/ft^2)}{29000\ ksi} = 1917\ in^4$$

$$I_{req,CONST}: \frac{\ell}{240} = \frac{0.036\ P_{CONST}\ \ell^3}{EI_{req,beam}} \Rightarrow I_{req,beam} = \frac{240(0.036)(0.02\ ksf)(12.083')(21')(36.33)^2(144\ in^2/ft^2)}{29000\ ksi} = 287\ in^4$$

USING AISC TABLE 3-19, ASSUMING Y2 = 4.5", TRY:

- W18x50 w/ $\phi M_n = 706$ k-ft, $\Sigma Q_n = 626$ K, $I_{LB} = \overset{\text{CONTROLS}}{2030}\ in^4$ & $I_{beam} = 800\ in^4$
  - ↳ STUDS : $n = \frac{\Sigma Q_n}{Q_n} = \frac{626\ K}{21.5\ K} = 29.1 \rightarrow n = 60$ OK✓
  - ↳ % COMPOSITE → 85.4 %
  - ↳ ECONOMY → $50_{plf}(36.33\ ft) + 60(10) = 2417$
- W18x55 w/ $\phi M_n = 697$ k-ft, $\Sigma Q_n = 454$ K, $I_{LB} = \overset{\text{CONTROLS}}{1960}\ in^4$ & $I_{beam} = 890\ in^4$
  - ↳ STUDS : $n = \frac{\Sigma Q_n}{Q_n} = \frac{454\ K}{21.5K} = 21.1 \rightarrow n = 44$ OK✓
  - ↳ % COMPOSITE : 56.0 %
  - ↳ ECONOMY → $55_{plf}(36.33\ ft) + 44(10) = 2438$
- W18x60 w/ $\phi M_n = 710$ k-ft, $\Sigma Q_n = 357$ K, $I_{LB} = \overset{\text{CONTROLS}}{1930}\ in^4$ & $I_{beam} = 984\ in^4$
  - ↳ STUDS : $n = \frac{\Sigma Q_n}{Q_n} = \frac{357\ K}{21.5\ K} = 16.6 \rightarrow n = 34$ OK✓
  - ↳ % COMPOSITE → 40.5 %
  - ↳ ECONOMY → $60\ plf\ (36.33\ ft) + 34(10) = 2518$

USE W18x50 w/ 60 STUDS (MOST ECONOMICAL)

$$\text{CHECK S.W. ASSUMPTION} \rightarrow 2\left(\frac{14\ plf}{12.083\ ft}\right) + \left(\frac{50\ plf}{21\ ft}\right) = 4.7\ psf < 5\ psf$$

OK✓

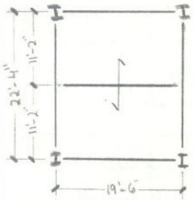
BAY: 2ND LEVEL, BAY 2 (DEPTH LIMIT: W18'S)

BAY IS 36'-6" x 21'-0", BUT OTHERWISE IDENTICAL TO BAY 1 ON THE SECOND LEVEL. THEREFORE, DESIGN OF BAY 1 WILL BE USED HERE.

BAY: 2ND LEVEL, BAY 3 (DEPTH LIMIT: W18'S)

BAY IS 36'-6" x21'-0", BUT OTHERWISE IDENTICAL TO BAY 1 ON THE SECOND LEVEL. THEREFORE, DESIGN OF BAY 1 WILL BE USED HERE.

BAY: 2ND LEVEL, BAY 4 (DEPTH LIMIT: W18'S)



LOADS → SDL = 15 psf
LL = 50 psf
PARTITIONS = 20 psf

DECK → USE VULCRAFT 3VLI COMPOSITE DECK
PER TECH 2 INVESTIGATION, USE 3½" TOPPING W/ FIREPROOFED DECK TO ACHIEVE 2 HR RATING

USING CLEAR SPAN OF 11'-6", SUPERIMPOSED LOAD OF 85 psf, & 2-SPAN CONDITION,
USE 3VLI 20 w/ 3.5" TOPPING
↳ MAX. CONST. SPAN = 11'-5" OK✓
↳ MAX. ALLOWABLE LOAD = 121 psf OK✓
↳ SELF-WEIGHT = 63 psf

INFILL BEAMS → TRIBUTARY AREA: 19'-6" x 11'-2"
STUDS: ¾" Ø, ⊥ TO RIB, WEAK POSITION (Q_n = 17.2 k)

$$LL_R = 0.25 + \left(\frac{15}{\sqrt{\underbrace{(2)(19.5')(11.17')}_{436 > 400 \text{ OK}✓}}}\right) = 0.97$$

$$\text{LOAD}: [1.2(15+63+\overset{\text{ASSUMED SELF-WEIGHT}}{5}) + 1.6(0.97)(50+20)]\,11.17' = 2326 \text{ plf}$$

$$\text{MOMENT}: \frac{w\ell^2}{8} = \frac{2.326\text{klf}\,(19.5')^2}{8} = 111 \text{ k-ft}$$

$$I_{req,LL}: \frac{\ell}{360} = \frac{5w_L\ell^4}{384EI_{req,LL}} \Rightarrow I_{req,LL} = \frac{360(5)[0.97(0.07)]11.17'(19.5')^3(144\ \text{in}^2/\text{ft}^2)}{384\ (29000\ \text{ksi})}$$
$$= 131 \text{ in}^4$$

$$I_{req,TL}: \frac{\ell}{240} = \frac{5w_T\ell^4}{384EI_{req,TL}} \Rightarrow I_{req,TL} = \frac{240(5)[0.083+0.97(0.07)]11.17'(19.5')^3(144\ \text{in}^2/\text{ft}^2)}{384\ (29000\ \text{ksi})}$$
$$= 194 \text{ in}^4$$

$I_{req,const}: \frac{l}{240} = \frac{5w_{const}\,l^4}{384 E I_{req,beam}} \Rightarrow I_{req,beam} = \frac{240(5)(0.02\text{ ksf})\,11.17'(19.5')^3(144\ \text{in}^2/\text{ft}^2)}{384\,(29000\text{ ksi})}$

$= 25.7\ \text{in}^4$

USING AISC TABLE 3-19, ASSUMING $Y2 = 5.5''$, TRY:

- $W10\times12$ w/ $\phi M_n = 121$ k-ft, $\Sigma Q_n = 135$ K, $I_{LB} = 221\ \text{in}^4$, & $I_{beam} = 53.8\ \text{in}^4$
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{135\text{ K}}{17.2\text{ K}} = 7.9 \rightarrow n = 16 < 19$ OK ✓
  - ↳ % COMPOSITE: 76.3 %
  - ↳ ECONOMY: 12 plf (19.5') + 16(10) = 394

- $W12\times14$ w/ $\phi M_n = 125$ k-ft, $\Sigma Q_n = 85.2$ K, $I_{LB} = 247\ \text{in}^4$, & $I_{beam} = 88.6\ \text{in}^4$
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{85.2\text{ K}}{17.2\text{ K}} = 5.0 \rightarrow n = 10 < 19$ OK ✓
  - ↳ % COMPOSITE: 41.0%
  - ↳ ECONOMY: 14(19.5') + 10(10) = 373 ↰ MORE ECONOMICAL

USE W12x14 w/ 10 STUDS

GIRDER → TRIBUTARY AREA: 22'-4" x 19'-6"

STUDS: 3/4" Ø, // TO RIB, $w_r/h_r \geq 1.5$ ($Q_n = 21.5$ K)

$LL_R = 0.25 + \left(\frac{15}{\sqrt{2(22.33')(19.5')}}\right) = 0.76 > 0.5$ OK ✓

871 > 400 OK ✓

LOAD: $[1.2(15+63+5) + 1.6(0.76)(50+20)]\,11.17'(19.5') = 40.2$ K

MOMENT: $\frac{Pl}{4} = \frac{40.2\text{ K}(22.33')}{4} = 235$ k-ft

FROM TABLE 3-22a

$I_{req,LL}: \frac{l}{360} = \frac{0.021 P_L l^3}{E I_{LL,req}} \Rightarrow I_{req,LL} = \frac{360(0.021)[0.76(0.07)]\,11.17'(19.5')(22.33')^2(144\ \text{in}^2/\text{ft}^2)}{29000\text{ ksi}}$

$= 217\ \text{in}^4$

$I_{req,TL}: \frac{l}{240} = \frac{0.021 P_T l^3}{E I_{req,TL}} \Rightarrow I_{req,TL} = \frac{240(0.021)[0.083 + 0.76(0.07)]\,11.17'(19.5')(22.33')^2(144\ \text{in}^2/\text{ft}^2)}{29000\text{ ksi}}$

$= 370\ \text{in}^4$

$I_{req,const}: \frac{l}{240} = \frac{0.021 P_{const}\,l^3}{E I_{req,beam}} \Rightarrow I_{req,beam} = \frac{240(0.021)(0.02)\,11.17'(19.5')(22.33')^2(144\ \text{in}^2/\text{ft}^2)}{29000\text{ ksi}}$

$= 54.4\ \text{in}^4$

USING AISC TABLE 3-19, ASSUMING $Y2 = 4.5''$, TRY:

- $W12\times22$ w/ $\phi M_n = 244$ k-ft, $\Sigma Q_n = 281$ K, $I_{LB} = 498\ \text{in}^4$, & $I_{beam} = 156\ \text{in}^4$
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{281\text{ K}}{21.5\text{ K}} = 13.1 \rightarrow n = 28$ OK ✓
  - ↳ % COMPOSITE: 86.7 %
  - ↳ ECONOMY: 22 plf (22.33') + 28(10) = 771

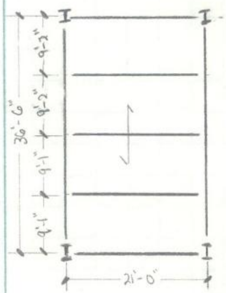
- W14x22 w/ $\phi M_n = 248$ k-ft, $\Sigma Q_n = 241$ K, $I_{LB} = 557 \text{ in}^4$; & $I_{beam} = 199 \text{ in}^4$
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{241 \text{ K}}{21.5 \text{ K}} = 11.2 \rightarrow n = 24$ OK ✓
  - ↳ % COMPOSITE: 74.2 %
  - ↳ ECONOMY: 22 plf (22.33') + 24(10) = 731

USE W14x22 W/ 24 STUDS

BAY: 5TH LEVEL, BAY 1 (DEPTH LIMIT: W18's)

BAY IS IDENTICAL TO BAY 1 ON SECOND LEVEL. THEREFORE, DESIGN OF BAY 1 FROM SECOND LEVEL WILL BE USED HERE.

BAY: 5TH LEVEL, BAY 2 (DEPTH LIMIT: W27's)



LOADS → MEP SDL = 15 psf
4" HOUSEKEEPING PAD SDL = 55 psf
LL = 150 psf

DECK → USE VULCRAFT 3VLI COMPOSITE DECK PER TECH 2 INVESTIGATION, USE 3½" TOPPING W/ FIREPROOFED DECK TO ACHIEVE 2 HR RATING

USING CLEAR SPAN OF 9'-6", SUPERIMPOSED LOAD OF 210 psf & 2-SPAN CONDITION,
USE 3VLI19 W/ 3.5" TOPPING
- ↳ MAX. CONST. SPAN = 12'-8"
- ↳ MAX. ALLOWABLE LOAD = 232 psf
- ↳ SELF-WEIGHT = 63 psf

INFILL BEAMS → TRIBUTARY AREA: 21'-0" x 9'-1"
STUDS: ¾" Ø, ⊥ TO RIB, WEAK POSITION ($Q_n = 17.2$ K)

$LL_R \rightarrow$ CANNOT REDUCE ($LL \geq 100$ psf)

$$\text{LOAD: } [1.2(15+55+63+\overset{\text{SELF-WEIGHT ASSUMED}}{10}) + 1.6(150)]\, 9.083' = 3739 \text{ plf}$$

$$\text{MOMENT: } \frac{w\ell^2}{8} = \frac{3.739 \text{ klf } (21')^2}{8} = 206 \text{ k-ft}$$

$$I_{req,LL} = \frac{\ell}{360} = \frac{5 w_L \ell^4}{384 E I_{req,LL}} \Rightarrow I_{req,LL} = \frac{360(5)(0.15)(9.083')(21')^3 \; (144 \text{ in}^2/\text{ft}^2)}{384 \; (29000 \text{ ksi})} = 294 \text{ in}^4$$

$$I_{req,TL} = \frac{\ell}{240} = \frac{5 w_T \ell^4}{384 E I_{req,TL}} \Rightarrow I_{req,TL} = \frac{240(5)(0.143+0.15)(9.083')(21')^3 \; (144 \text{ in}^2/\text{ft}^2)}{384 \; (29000 \text{ ksi})} = 383 \text{ in}^4$$

$$I_{req,\,CONST} = \frac{\ell}{240} = \frac{5 w_{CONST}\,\ell^4}{384\,E\,I_{req,\,beam}} \Rightarrow I_{req,\,beam} = \frac{240(5)(0.02)(9.083')(21')^3\,(144\ \text{in}^2/\text{ft}^2)}{384\ (29000\ \text{ksi})}$$

$$= 26.1\ \text{in}^4$$

USING AISC TABLE 3-19, ASSUMING $Y2 = 5.5''$, TRY:

- W14x22 w/ $\phi M_n = 248$ k-ft, $\Sigma Q_n = 199$ k, $I_{LB} = 577\ \text{in}^4$, & $I_{beam} = 199\ \text{in}^4$
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{199\ \text{k}}{17.2\text{k}} = 11.6 \rightarrow n = 24 > 21$ NOT OK

- W14x26 w/ $\phi M_n = 250$ k-ft, $\Sigma Q_n = 135$ k, $I_{LB} = 555\ \text{in}^4$, & $I_{beam} = 245\ \text{in}^4$
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{135\ \text{k}}{17.2\ \text{k}} = 7.8 \rightarrow n = 16 < 21$ OK ✓
  - ↳ % COMPOSITE: 35.1 %
  - ↳ ECONOMY: 26 plf (21') + 16(10) = 706

- W16x26 w/ $\phi M_n = 248$ k-ft, $\Sigma Q_n = 96$ k, $I_{LB} = 575\ \text{in}^4$, & $I_{beam} = 301\ \text{in}^4$
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{96\ \text{k}}{17.2\ \text{k}} = 5.6 \rightarrow n = 12 < 21$ OK ✓
  - ↳ % COMPOSITE: 25.0 %
  - ↳ ECONOMY: 26 plf (21') + 12(10) = 666

USE W16x26 w/ 12 STUDS

GIRDER → TRIBUTARY AREA: 36'-6" x 21'-0"
STUDS: 3/4" Ø, // TO RIB, $w_r/h_r \geq 1.5$ ($Q_n = 21.5$ k)

LL_R → CANNOT REDUCE ($LL \geq 100$ psf)

LOAD: 3.739 klf (21 ft) = 78.5 k

MOMENT: $\frac{P\ell}{2} = \frac{78.5\ \text{k}\,(36.5')}{2} = 1433\ \text{k}\cdot\text{ft}$

FROM TABLE 3-22a

$$I_{req,\,LL}: \frac{\ell}{360} = \frac{0.05\,P_L\,\ell^3}{E\,I_{req,\,LL}} \Rightarrow I_{req,\,LL} = \frac{360(0.05)(0.15)(9.083')(21')(36.5')^2\,(144\ \text{in}^2/\text{ft}^2)}{29000\ \text{ksi}}$$

$$= 3407\ \text{in}^4$$

$$I_{req,\,TL}: \frac{\ell}{240} = \frac{0.05\,P_T\,\ell^3}{E\,I_{req,\,TL}} \Rightarrow I_{req,\,TL} = \frac{240(0.05)(0.143+0.15)(9.083')(21')(36.5')^2\,(144\ \text{in}^2/\text{ft}^2)}{29000\ \text{ksi}}$$

$$= 4437\ \text{in}^4$$

$$I_{req,\,CONST}: \frac{\ell}{240} = \frac{0.05\,P_{CONST}\,\ell^3}{E\,I_{req,\,beam}} \Rightarrow I_{req,\,beam} = \frac{240(0.05)(0.02)(9.083')(21')(36.5')^2\,(144\ \text{in}^2/\text{ft}^2)}{29000\ \text{ksi}}$$

$$= 303\ \text{in}^4$$

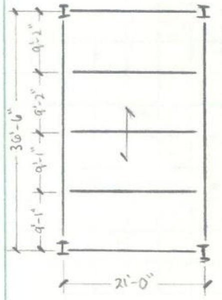
USING AISC TABLE 3-19, ASSUMING $Y2 = 4.5''$, TRY:

- W24x84 w/ $\phi M_n = 1470$ k·ft, $\Sigma Q_n = 1060$ K, $I_{LB} = 5500$ in^4, & $I_{beam} = 2370$ in^4
  ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{1060\text{ K}}{21.5\text{ K}} = 49.3 \rightarrow n = 100$ ← VERY HIGH

- W27 x 84 w/ $\phi M_n = 1490$ k·ft, $\Sigma Q_n = 758$K, $I_{LB} : 5850$ in^4, & $I_{beam} = 2850$ in^4
  ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{758\text{ K}}{21.5\text{ K}} = 35.2 \rightarrow n = 72$
  ↳ % COMPOSITE: 61.1 %
  ↳ ECONOMY: 84 plf (36.5 ft) + 72(10) = 3786

USE W27x84 w/ 72 STUDS

BAY: 5TH LEVEL, BAY 3 (DEPTH LIMIT: W24's)



LOADS → MEP SDL = 15 psf
6" HOUSEKEEPING PAD SDL = 80 psf
LL = 100 psf

DECK → USE VULCRAFT 3VLI COMPOSITE DECK
PER TECH 2 INVESTIGATION, USE 3½" TOPPING W/ FIREPROOFED DECK TO ACHIEVE 2 HR RATING

USING CLEAR SPAN OF 9'-6", SUPERIMPOSED LOAD OF 195 psf & 2-SPAN CONDITION,
USE 3VLI19 W/ 3.5" TOPPING
↳ MAX. CONST. SPAN = 12'-8"
↳ MAX ALLOWABLE LOAD = 232 psf
↳ SELF-WEIGHT = 63 psf

INFILL BEAMS → TRIBUTARY AREA: 21'-0" x 9'-1"
STUDS: ¾" Ø, ⊥ TO RIB, WEAK POSITION ($Q_n = 17.2$ K)

LLR → CANNOT BE REDUCED ($LL \geq 100$ psf)

$$\text{LOAD: } [1.2(15+80+63+\overset{\text{ASSUMED SELF-WEIGHT}}{10}) + 1.6(100)]\,9.083' = 3284 \text{ plf}$$

$$\text{MOMENT: } \frac{w\ell^2}{8} = \frac{3.284 \text{ klf } (21\text{ ft})^2}{8} = 184 \text{ k·ft}$$

$$I_{req,LL} = \frac{\ell}{360} = \frac{5w_L\ell^4}{384 E\, I_{req,LL}} \Rightarrow I_{req,LL} = \frac{360(5)(0.1)(9.083')(21')^3\,(144\text{ in}^2/\text{ft}^2)}{384\,(29000\text{ ksi})}$$
$$= 196 \text{ in}^4$$

$$I_{req,TL} = \frac{\ell}{240} = \frac{5w_T\ell^4}{384 E\, I_{req,TL}} \Rightarrow I_{req,TL} = \frac{240(5)(0.168+0.1)(9.083')(21')^3\,(144\text{ in}^2/\text{ft}^2)}{384\,(29000\text{ ksi})}$$
$$= 350 \text{ in}^4$$

$I_{req,\,CONST}: \frac{l}{240} = \frac{5 w_{CONST}\, l^4}{384 E\, I_{req,beam}} \Rightarrow I_{req,beam} = \frac{240(5)(0.02)(9.083')(21')^3 (144\ in^2/ft^2)}{384\ (29000\ ksi)}$

$= 26.1\ in^4$

USING AISC TABLE 3-19, ASSUMING $Y2 = 5.5''$, TRY:

- W12x16 w/ $\phi M_n = 192$ k·ft, $\Sigma Q_n = 209$ K, $I_{LB} = 396\ in^4$ & $I_{beam} = 103\ in^4$
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{209\ K}{17.2\ K} = 12.2 \rightarrow n = 26 > 21$ NOT OK
- W12x19 w/ $\phi M_n = 182$ k·ft, $\Sigma Q_n = 139$ K, $I_{LB} = 379\ in^4$, & $I_{beam} = 130\ in^4$
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{139\ K}{17.2\ K} = 8.1 \rightarrow n = 18 < 21$ OK ✓
  - ↳ % COMPOSITE: 49.8 %
  - ↳ ECONOMY: 19 plf (21') + 18 (10) = 579

USE W12x19 w/ 18 STUDS

GIRDER → TRIBUTARY AREA: 36'-6" x 21'-0"
STUDS: 3/4" Ø, // TO RIB, $w_r/h_r \geq 1.5$ ($Q_n = 21.5$ K)

$LL_R \rightarrow$ CANNOT REDUCE ($LL \geq 100$ psf)

LOAD: 3,284 plf (21 ft) = 69.0 K

MOMENT: $\frac{Pl}{2} = \frac{69.0\ K\ (36.5')}{2} = 1260$ k·ft

FROM TABLE 3-22a

$I_{req,\,LL}: \frac{l}{360} = \frac{0.05 P_L\, l^3}{E\, I_{req,LL}} \Rightarrow I_{req,LL} = \frac{360(0.05)(0.1)(9.083')(21')(36.5')^2 (144\ in^2/ft^2)}{29000\ ksi}$

$= 2271\ in^4$

$I_{req,\,TL}: \frac{l}{240} = \frac{0.05 P_T\, l^3}{E\, I_{req,TL}} \Rightarrow I_{req,TL} = \frac{240(0.05)(0.168+0.1)(9.083')(21')(36.5')^2 (144\ in^2/ft^2)}{29000\ ksi}$

$= 4058\ in^4$

$I_{req,\,CONST}: \frac{l}{240} = \frac{0.05 P_{CONST}\, l^3}{E\, I_{req,beam}} \Rightarrow I_{req,beam} = \frac{240(0.05)(0.02)(9.083')(21')(36.5')^2 (144\ in^2/ft^2)}{29000\ ksi}$

$= 303\ in^4$

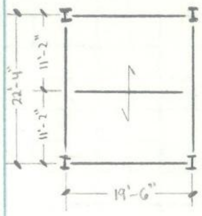
USING AISC TABLE 3-19, ASSUMING $Y2 = 4.5''$, TRY:

- W24x76 w/ $\phi M_n = 1270$ k·ft, $\Sigma Q_n = 813$ K, $I_{LB} = 4650\ in^4$, & $I_{beam} = 2100\ in^4$
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{813\ K}{21.5\ K} = 37.8 \rightarrow n = 76$ OK ✓
  - ↳ % COMPOSITE: 72.6%
  - ↳ ECONOMY: 76 plf (36.5 ft) + 76(10) = 3534

USE W24x76 w/ 76 STUDS

BAY: 5TH LEVEL, BAY 4 (DEPTH LIMIT: W14'S)

LOADS → MEP SDL = 20 psf
ROOFING SDL = 51 psf
RL = 50 psf



DECK → USE VULCRAFT 3VLI COMPOSITE DECK TO CARRY HIGH ROOF LOADS
PER TECH 2 INVESTIGATION, USE 3½" TOPPING w/ FIREPROOFED DECK TO ACHIEVE 2 HR. RATING

USING CLEAR SPAN OF 11'-6", SUPERIMPOSED LOAD OF 121 psf, & 2-SPAN CONDITION,
USE 3VLI 20 w/ 3.5" TOPPING
↳ MAX. CONST. SPAN = 11'-5"
↳ MAX. ALLOWABLE LOAD = 121 psf
↳ SELF-WEIGHT = 63 psf

INFILL BEAMS → TRIBUTARY AREA: 19'-6" x 11'-2"
STUDS: 3/4" Ø, ⊥ TO RIB, WEAK POSITION ($Q_n = 17.2$ K)

SELF-WEIGHT ASSUMED

LOAD: $[1.2(20+51+63+5)+1.6(50)]\,11.17' = 2757$ plf

MOMENT: $\dfrac{wl^2}{8} = \dfrac{2.757 \text{ klf } (21 \text{ ft})^2}{8} = 152 \text{ k}\cdot\text{ft}$

$I_{req,LL}$: $\dfrac{l}{240} = \dfrac{5 w_L l^4}{384 E I_{req,LL}} \Rightarrow I_{req,LL} = \dfrac{240(5)(0.05)(11.17')(21 \text{ ft})^3 (144 \text{ in}^2/\text{ft}^2)}{384 \ (29000 \text{ ksi})}$
$= 80.3 \text{ in}^4$

$I_{req,TL}$: $\dfrac{l}{180} = \dfrac{5 w_T l^4}{384 E I_{req,TL}} \Rightarrow I_{req,TL} = \dfrac{180(5)(0.134+0.05)(11.17')(21 \text{ ft})^3 (144 \text{ in}^2/\text{ft}^2)}{384 \ (29000 \text{ ksi})}$
$= 222 \text{ in}^4$

$I_{req,const}$: $\dfrac{l}{180} = \dfrac{5 w_{const} l^4}{384 E I_{req,beam}} \Rightarrow I_{req,beam} = \dfrac{180(5)(0.02)(11.17')(21 \text{ ft})^3 (144 \text{ in}^2/\text{ft}^2)}{384 \ (29000 \text{ ksi})}$
$= 24.1 \text{ in}^4$

USING AISC TABLE 3-19, ASSUMING Y2 = 5.5", TRY:
- W12x14 w/ $\phi M_n = 160$ k·ft, $\Sigma Q_n = 163$ K, $I_{LB} = 328 \text{ in}^4$, & $I_{beam} = 88.6 \text{ in}^4$
  ↳ STUDS: $n = \dfrac{\Sigma Q_n}{Q_n} = \dfrac{163 \text{ K}}{17.2 \text{ K}} = 9.5 \rightarrow n = 20 < 20$ OK ✓
  ↳ % COMPOSITE: 78.4%
  ↳ ECONOMY: 14 plf(21 ft) + 20 (10) = 494

USE W12x14 w/ 20 STUDS

GIRDER → TRIBUTARY AREA: 22'-4" x 19'-6"
STUDS: 3/4" Ø, // TO RIB, $w_r/h_r \geq 1.5$ ($Q_n = 21.5$ K)

LOAD: 2.757 klf (19.5') = 53.8 K

MOMENT: $\frac{P\ell}{4} = \frac{53.8\text{ K}(22.33')}{4} = 300$ K.ft

$I_{req,LL}$: $\frac{\ell}{240} = \frac{0.021 P_L \ell^3}{E I_{req,LL}} \Rightarrow I_{req,LL} = \frac{240(0.021)(0.05)(11.17')(19.5')(22.33')^2(144\ \text{in}^2/\text{ft}^2)}{29000\text{ ksi}}$
$= 136\ \text{in}^4$

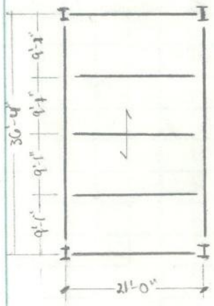
$I_{req,TL}$: $\frac{\ell}{180} = \frac{0.021 P_T \ell^3}{E I_{req,TL}} \Rightarrow I_{req,TL} = \frac{180(0.021)(0.134+0.05)(11.17)(21)(22.33)^2(144\ \text{in}^2/\text{ft}^2)}{29000\text{ ksi}}$
$= 404\ \text{in}^4$

$I_{req,CONST}$: $\frac{\ell}{180} = \frac{0.021 P_{CONST} \ell^3}{E I_{req,beam}} \Rightarrow I_{req,beam} = \frac{180(0.021)(0.02)(11.17)(21)(22.33)^2(144\ \text{in}^2/\text{ft}^2)}{29000\text{ ksi}}$
$= 43.9\ \text{in}^4$

USING AISC TABLE 3-19, ASSUMING Y2 = 5", TRY:
- W14x26 w/ $\phi M_n = 304$ k.ft, $\Sigma Q_n = 279$ K, $I_{LB} = 707\ \text{in}^4$, & $I_{beam} = 245\ \text{in}^4$
  ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{279\text{ K}}{21.5\text{ K}} = 13.0 \rightarrow n = 26$ OK ✓
  ↳ % COMPOSITE: 72.5%
  ↳ ECONOMY: 26 plf (22.33') + 26 (10) = 841

USE W14x26 w/ 26 STUDS

BAY: PENTHOUSE LEVEL, BAY 1 (DEPTH LIMIT: W27's)



LOADS → MEP SDL = 10 psf
6" HOUSEKEEPING PAD SDL = 80 psf
LL = 100 psf

BAY IS VERY SIMILAR TO 5TH LEVEL BAY 3
↳ THIS BAY IS 36'-4" x 21'-0" (INSTEAD OF 36'-6" x 21'-0")
↳ THIS BAY HAS SDL = 90 psf, LL = 100 psf (INSTEAD OF SDL = 95 psf, LL = 100 psf)

USING DESIGN OF 5TH LEVEL IS CONSERVATIVE (SLIGHTLY)

DECK → USE 3VLI19 w/ 3.5" TOPPING

INFILL BEAMS → USE W12x19 w/ 18 STUDS

GIRDER → USE W27x76 w/ 76 STUDS

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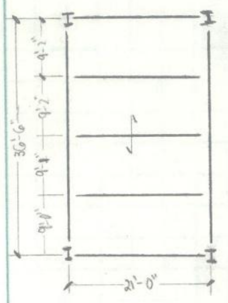
BAY: PENTHOUSE LEVEL, BAY 2 (DEPTH LIMIT: W30's)



LOADS → MEP SDL = 10 psf
6" HOUSEKEEPING PAD SDL = 80 psf
LL = 150 psf

DECK → USE VULCRAFT 3VLI COMPOSITE DECK
PER TECH 2 INVESTIGATION, USE 3½" TOPPING
AND FIREPROOFED DECK TO ACHIEVE 2 HR RATING

USING CLEAR SPAN OF 9'-6", SUPERIMPOSED LOAD
OF 240 psf, & 2-SPAN CONDITION,
USE 3VLI18 W/ 3.5" TOPPING
↳ MAX. CONST. SPAN = 13'-9" OK ✓
↳ MAX. ALLOWABLE LOAD = 264 psf OK ✓
↳ SELF-WEIGHT = 63 psf

INFILL BEAMS → TRIBUTARY AREA: 21'-0" x 9'-1"
STUDS: ¾" Ø, ⊥ TO RIB, WEAK POSITION ($Q_n = 17.2$ K)

LL_R → CANNOT REDUCE ($LL \geq 100$ psf)

$$\text{LOAD: } [1.2(10+80+\overset{\text{ASSUMED SELF-WEIGHT}}{63}+10)+1.6(150)]\,9.083' = 3957 \text{ plf}$$

$$\text{MOMENT: } \frac{w\ell^2}{8} = \frac{3.957 \text{ klf}(21')^2}{8} = 218 \text{ k}\cdot\text{ft}$$

$$I_{req,LL}: \frac{\ell}{360} = \frac{5w_L\ell^4}{384EI_{req,LL}} \Rightarrow I_{req,LL} = \frac{360(5)(0.15)(9.083')(21')^3(144\ \text{in}^2/\text{ft}^2)}{384(29000\text{ ksi})}$$
$$= 294 \text{ in}^4$$

$$I_{req,TL}: \frac{\ell}{240} = \frac{5w_T\ell^4}{384EI_{req,TL}} \Rightarrow I_{req,TL} = \frac{240(5)(0.163+0.15)(9.083')(21')^3(144\ \text{in}^2/\text{ft}^2)}{384(29000\text{ ksi})}$$
$$= 409 \text{ in}^4$$

$$I_{req,CONST}: \frac{\ell}{240} = \frac{5w_{const}\ell^4}{384EI_{req,beam}} \Rightarrow I_{req,beam} = \frac{240(5)(0.02)(9.083')(21')^3(144\ \text{in}^2/\text{ft}^2)}{384(29000\text{ ksi})}$$
$$= 26.1 \text{ in}^4$$

USING AISC TABLE 3-19 ASSUMING $Y2 = 5.5"$, TRY:

- W12x19 w/ $\phi M_n = 227$ k·ft, $\Sigma Q_n = 244$ K, $I_{LB} = 479 \text{ in}^4$ & $I_{beam} = 130 \text{ in}^4$
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{244 \text{ K}}{17.2 \text{ K}} = 14.2 \rightarrow n = 30 > 21$ NOT OK
- W14x22 w/ $\phi M_n = 230$ k·ft, $\Sigma Q_n = 157$ K, $I_{LB} = 523 \text{ in}^4$ & $I_{beam} = 199 \text{ in}^4$
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{157 \text{ K}}{17.2 \text{K}} = 9.0 \rightarrow n = 20 < 21$ OK ✓
  - ↳ % COMPOSITE: 48.3 %
  - ↳ ECONOMY: 22 plf (21') + 20(10) = 662

USE W14x22 w/ 20 STUDS

GIRDER → TRIBUTARY AREA: 36'-6" x 21'-6"
STUDS: 3/4" Ø, // TO RIB, $w_r/h_r \geq 1.5$ (Q_n = 21.5 k)

LLR → CANNOT BE REDUCED (LL ≥ 100 psf)

LOAD: 3.957 klf (21 ft) = 83.1 k

MOMENT: $\frac{Pl}{2} = \frac{83.1k\ (36.5\ ft)}{2} = 1517\ k\cdot ft$

$I_{req,LL}$: $\frac{l}{360} = \frac{0.05\ P_L\ l^3}{E\ I_{req,LL}} \Rightarrow I_{req,LL} = \frac{360(0.05)(0.15)(9.083')(21')(36.5')^2\ (144\ in^2/ft^2)}{29000\ ksi}$

$= 3407\ in^4$

$I_{req,TL}$: $\frac{l}{240} = \frac{0.05 P_T\ l^3}{E\ I_{req,TL}} \Rightarrow I_{req,TL} = \frac{240\ (0.05)\ (0.163+0.15)(9.083')(21')(36.5')^2\ (144\ in^2/ft^2)}{29000\ ksi}$

$= 2271\ in^4$

$I_{req,CONST}$: $\frac{l}{240} = \frac{0.05 P_{CONST}\ l^4}{E\ I_{req,beam}} \Rightarrow I_{req,beam} = \frac{240\ (0.05)\ (0.02)(9.083')(21')(36.5)^2\ (144\ in^2/ft^2)}{29000\ ksi}$

$= 303\ in^4$

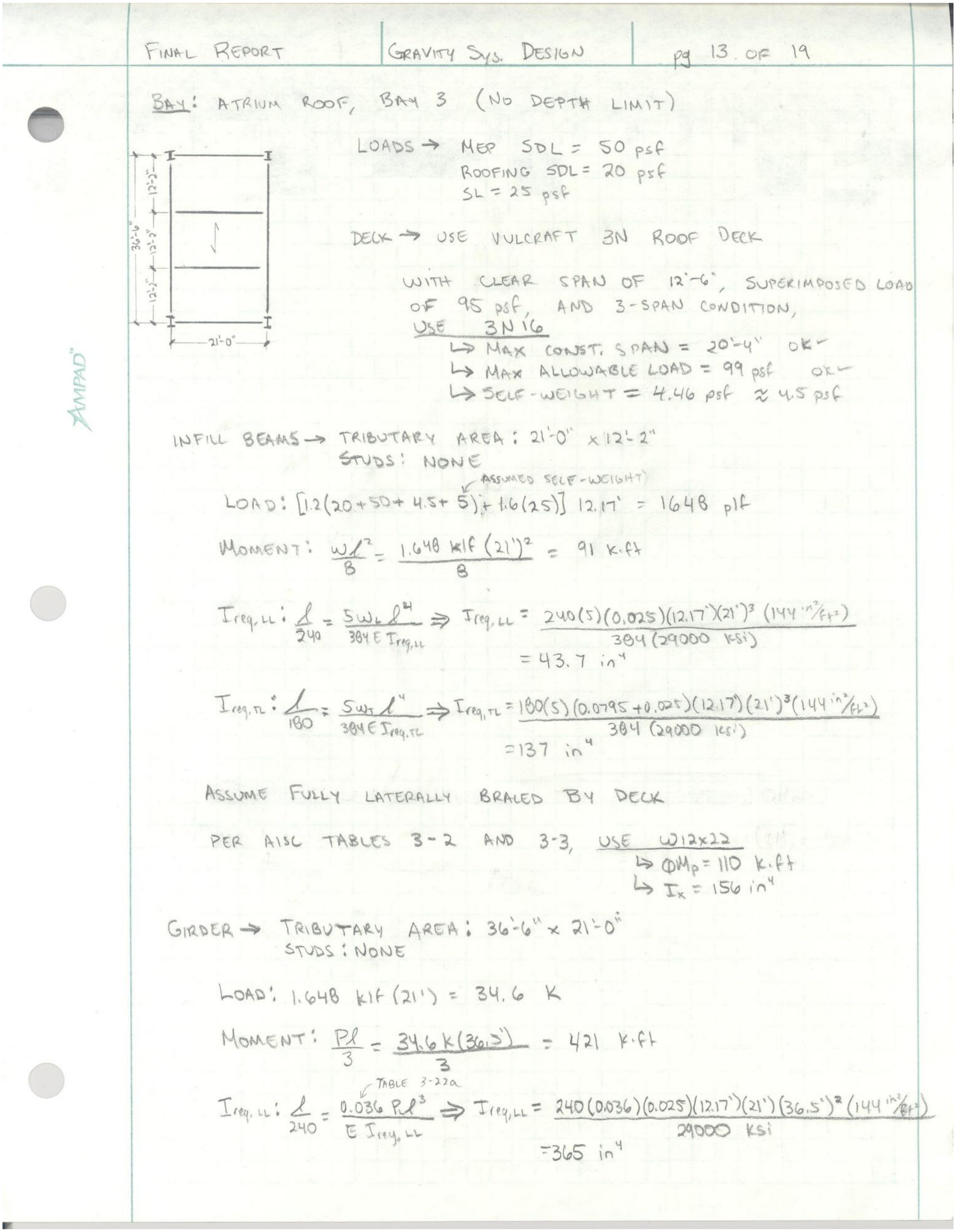
USING AISC TABLE 3-19, ASSUMING $y_2 = 4.5"$, TRY:

- W24x84 w/ ϕM_n = 1530 k.ft, ΣQ_n = 1240 k, I_{LB} = 5760 in^4, & I_{beam} = 2370 in^4
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{1240\ k}{21.5\ k} = 57.7 \rightarrow n = 116$ VERY HIGH

- W27x84 w/ ϕM_n = 1550 kft, ΣQ_n = 918 k, I_{LB} = 6210 in^4, & I_{beam} = 2850 in^4
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{918\ k}{21.5\ k} = 42.7 \rightarrow n = 86$ OK ✓
  - ↳ % COMPOSITE: 74.0 %
  - ↳ ECONOMY: 84 plf (36.5') + 86(10) = 3926

- W30x90 w/ ϕM_n = 1590 k.ft, ΣQ_n = 506 k, I_{LB} = 6320 in^4, & I_{beam} = 3610 in^4
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{506\ k}{21.5\ k} = 23.5 \rightarrow n = 48$ OK ✓
  - ↳ % COMPOSITE: 38.3 %
  - ↳ ECONOMY: 90 plf (36.5') + 48(10) = 3765

USE W30x90 w/ 48 STUDS



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BAY: ATRIUM ROOF, BAY 3 (NO DEPTH LIMIT)

LOADS → MEP SDL = 50 psf
ROOFING SDL = 20 psf
SL = 25 psf

DECK → USE VULCRAFT 3N ROOF DECK

WITH CLEAR SPAN OF 12'-6", SUPERIMPOSED LOAD OF 95 psf, AND 3-SPAN CONDITION,

USE 3N16

↳ MAX CONST. SPAN = 20'-4" OK ✓

↳ MAX ALLOWABLE LOAD = 99 psf OK ✓

↳ SELF-WEIGHT = 4.46 psf ≈ 4.5 psf

INFILL BEAMS → TRIBUTARY AREA: 21'-0" x 12'-2"
STUDS: NONE

ASSUMED SELF-WEIGHT

LOAD: $[1.2(20+50+4.5+5)+1.6(25)]\ 12.17' = 1648$ plf

MOMENT: $\frac{w\ell^2}{8} = \frac{1.648\text{ klf }(21')^2}{8} = 91$ k·ft

$I_{req,LL}: \frac{\ell}{240} = \frac{5w_L\ell^4}{384EI_{req,LL}} \Rightarrow I_{req,LL} = \frac{240(5)(0.025)(12.17')(21')^3(144\text{ in}^2/\text{ft}^2)}{384(29000\text{ ksi})}$

$= 43.7\text{ in}^4$

$I_{req,TL}: \frac{\ell}{180} = \frac{5w_T\ell^4}{384EI_{req,TL}} \Rightarrow I_{req,TL} = \frac{180(5)(0.0795+0.025)(12.17')(21')^3(144\text{ in}^2/\text{ft}^2)}{384(29000\text{ ksi})}$

$= 137\text{ in}^4$

ASSUME FULLY LATERALLY BRACED BY DECK

PER AISC TABLES 3-2 AND 3-3, USE W12x22

↳ $\phi M_p = 110$ k·ft

↳ $I_x = 156\text{ in}^4$

GIRDER → TRIBUTARY AREA: 36'-6" x 21'-0"
STUDS: NONE

LOAD: 1.648 klf (21') = 34.6 K

MOMENT: $\frac{P\ell}{3} = \frac{34.6\text{ k}(36.5')}{3} = 421$ k·ft

TABLE 3-22a

$I_{req,LL}: \frac{\ell}{240} = \frac{0.036P_L\ell^3}{EI_{req,LL}} \Rightarrow I_{req,LL} = \frac{240(0.036)(0.025)(12.17')(21')(36.5')^2(144\text{ in}^2/\text{ft}^2)}{29000\text{ ksi}}$

$= 365\text{ in}^4$

$I_{req,TL}$: $\frac{\ell}{180} = \frac{0.036 P \ell^3}{E I_{req,TL}} \Rightarrow I_{req,TL} = \frac{180(0.036)(0.0795+0.025)(12.17')(21')(36.5')^2(144 \text{ in}^2/\text{ft}^2)}{29000 \text{ ksi}}$

$= 1145 \text{ in}^4$

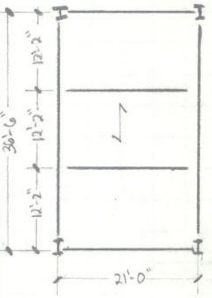
ASSUME BRACED @ 3rd POINTS BY INFILL BEAMS → $L_b = 12.17'$

PER AISC TABLES 3-3 & 3-10, USE W21 x 62

↳ $\phi M_n = 440$ k·ft

↳ $I_x = 1330 \text{ in}^4$

BAY: CHILLER ROOF, BAY 2 (DEPTH LIMIT: NONE)



LOADS → MEP SDL = 50 psf

ROOFING SDL = 20 psf

LL = 100 psf

DECK → USE VULCRAFT 3VLI COMPOSITE DECK

USING CLEAR SPAN OF 12'-6", SUPERIMPOSED LOAD OF 180 psf & 3-SPAN CONDITION,

USE 3VLI16 w/ 4.5" TOPPING

↳ MAX. CONST. SPAN = 13'-4"

↳ MAX. ALLOWABLE LOAD = 188 psf

↳ SELF-WEIGHT = 75 psf

INFILL BEAMS → TRIBUTARY AREA: 21'-0" x 12'-2"

STUDS: 3/4" ∅, ⊥ TO RIB, WEAK POSITION ($Q_n = 17.2$ k)

LL_R → CANNOT REDUCE ($LL \geq 100$ psf)

LOAD: $[1.2(50 + 20 + 75 + \overset{\text{ASSUMED SELF-WEIGHT}}{10}) + 1.6(100)]12.17' = 4211$ plf

MOMENT: $\frac{w\ell^2}{8} = \frac{4.211 \text{ klf } (21')^2}{8} = 232$ k·ft

$I_{req,LL}$: $\frac{\ell}{240} = \frac{5 w_L \ell^4}{384 E I_{req,LL}} \Rightarrow I_{req,LL} = \frac{240(5)(0.1)(12.17)(21')^3(144 \text{ in}^2/\text{ft}^2)}{384(29000 \text{ ksi})}$

$= 175 \text{ in}^4$

$I_{req,TL}$: $\frac{\ell}{180} = \frac{5 w_T \ell^4}{384 E I_{req,TL}} \Rightarrow I_{req,TL} = \frac{180(5)(0.155+0.1)(12.17)(21')^3(144 \text{ in}^2/\text{ft}^2)}{384(29000 \text{ ksi})}$

$= 335 \text{ in}^4$

$I_{req,const}$: $\frac{\ell}{180} = \frac{5 w_{const} \ell^4}{384 E I_{req,beam}} \Rightarrow I_{req,beam} = \frac{180(5)(0.02)(12.17)(21')^3(144 \text{ in}^2/\text{ft}^2)}{384(29000 \text{ ksi})}$

$= 26.2 \text{ in}^4$

USING AISC TABLE 3-19, ASSUMING Y2 = 5.5", TRY:

- W14x22 w/ $\phi M_n = 248$ k.ft, $\Sigma Q_n = 199$ k, $I_{LB} = 577 \text{ in}^4$, & $I_{beam} = 199 \text{ in}^4$

↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{199k}{17.2} = 11.6 \rightarrow n = 24 > 21$ NOT GOOD

- W14x26 w/ $\phi M_n = 250$ k·ft, $\Sigma Q_n = 135$ K, $I_{LB} = 555$ in⁴, & $I_{beam} = 245$ in⁴
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{135\text{ K}}{17.2\text{ K}} = 7.8 \rightarrow n = 16 < 21$ OK ✓
  - ↳ % COMPOSITE: 35.1 %
  - ↳ ECONOMY: 26 plf (21') + 16(10) = 706

USE W14x26 w/ 16 STUDS

GIRDER → TRIBUTARY AREA: 36'-6" x 21'-0"

STUDS: ¾" Ø, // TO RIB, $w_r/h_r \geq 1.5$ ($Q_n = 21.5$ K)

LLR → CANNOT REDUCE (LL ≥ 100 psf)

LOAD: 4.211 klf (21') = 88.4 K

$$\text{MOMENT: } \frac{P\ell}{3} = \frac{88.4\text{ K }(36.5')}{3} = 1076 \text{ K·ft}$$

TABLE 3-22a

$$I_{req,LL}: \frac{\ell}{240} = \frac{0.036 P_L \ell^3}{E I_{req,LL}} \Rightarrow I_{req,LL} = \frac{240(0.036)(0.1)(12.17')(21')(36.5')^2(144 \text{ in}^2/\text{ft}^2)}{29000 \text{ ksi}}$$

$$= 1461 \text{ in}^4$$

$$I_{req,TL}: \frac{\ell}{180} = \frac{0.036 P_T \ell^3}{E I_{req,TL}} \Rightarrow I_{req,TL} = \frac{180(0.036)(0.155+0.1)(12.17')(21')(36.5')^2(144 \text{ in}^2/\text{ft}^2)}{29000 \text{ ksi}}$$

$$= 2794 \text{ in}^4$$

$$I_{req,CONST}: \frac{\ell}{180} = \frac{0.036 P_{CONST} \ell^3}{E I_{req,beam}} \Rightarrow I_{req,beam} = \frac{180(0.036)(0.02)(12.17')(21')(36.5')^2(144 \text{ in}^2/\text{ft}^2)}{29000 \text{ ksi}}$$

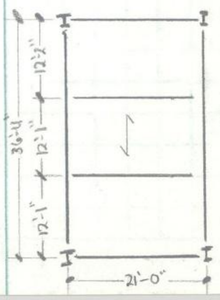
$$= 219 \text{ in}^4$$

USING AISC TABLE 3-19, ASSUMING Y2 = 4.5", TRY:

- W24x62 w/ $\phi M_n = 1080$ k·ft, $\Sigma Q_n = 807$ K, $I_{LB} = 3840$ in⁴ & $I_{beam} = 1550$ in⁴
  - ↳ STUDS: $n = \frac{\Sigma Q_n}{Q_n} = \frac{807\text{ K}}{21.5\text{ K}} = 37.5 \rightarrow n = 76$ OK ✓
  - ↳ % COMPOSITE: 88.6 %
  - ↳ ECONOMY: 62 plf (36.5') + 76(10) = 3023

USE W24x62 w/ 76 STUDS

BAY: AHU MECH. ROOM ROOF, BAY 1 (DEPTH LIMIT: NONE)

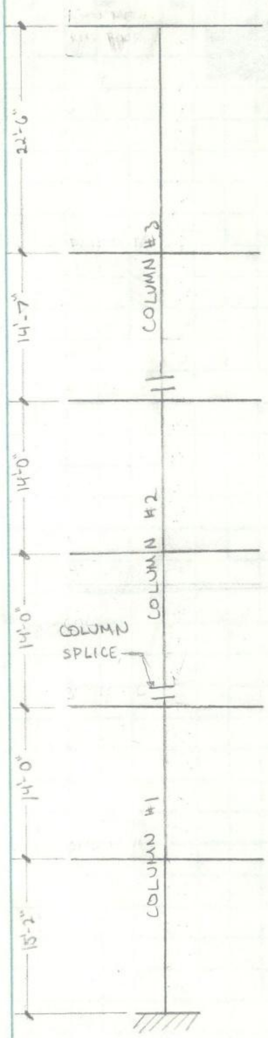


LOADS → MEP SDL = 50 psf
ROOFING SDL = 20 psf
LL = 25 psf

BAY IS 36'-4" x 21'-0" (INSTEAD OF 36'-6" x 21'-0"), BUT IS OTHERWISE IDENTICAL TO ATRIUM ROOF BAY 3.

USE DESIGN OF ATRIUM ROOF BAY 3

COLUMN C/2 → TRIBUTARY AREA: 21'-0" x 26'-8" (A_T = 560 ft²)



AHU MECH. RM. ROOF
↳ FRAMING → 5psf
↳ DECK → 5 psf
↳ SDL → 70 psf
↳ SL → 25 psf

PENTHOUSE LEVEL
↳ FRAMING → 10 psf
↳ SLAB/DECK → 63 psf
↳ SDL → 90 psf
↳ LL → 100 psf

5TH LEVEL
↳ FRAMING → 5 psf
↳ SLAB/DECK → 63 psf
↳ SDL → 15 psf
↳ LL → 80 psf

4TH LEVEL
↳ FRAMING → 5psf
↳ SLAB/DECK → 63 psf
↳ SDL → 15 psf
↳ LL → 80 psf

3rd LEVEL
↳ FRAMING → 5 psf
↳ SLAB/DECK → 63psf
↳ SDL → 15 psf
↳ LL → 80 psf

2ND LEVEL
↳ FRAMING → 5psf
↳ SLAB/DECK → 63 psf
↳ SDL → 15psf
↳ LL → 80 psf

COLUMN LOADS:

ROOF
↳ $P_D = (5+5+70)21'(26.67') = 44.8\ K$
↳ $P_S = 25\ psf(21')(26.67') = 14.0\ K$
$P_U = 1.2(44.8) + 1.6(14) = \underline{76.2\ K}$

PENTHOUSE
↳ $P_D = (10+63+90)(21')(26.67') = 91.3\ K$
↳ $P_L = 100(21')(26.67') = 56.0\ K$
$P_U = 1.2(91.3) + 1.6(56) = \underline{166\ K}$

5TH LEVEL
↳ $P_D = (5+63+15)(21')(26.67') = 46.5\ K$
↳ $LL_r = 0.25 + \left(\frac{15}{\sqrt{4(560)}}\right) = 0.57 \Rightarrow 0.6$
$P_L = 80(0.6)(560\ ft^2) = 26.9\ K$
$P_U = 1.2(46.5) + 1.6(26.9) = \underline{98.9\ K}$

4TH LEVEL
↳ $P_D = 46.5\ K$
↳ $LL_r = 0.25 + \left(\frac{15}{\sqrt{4(2)(560)}}\right) = 0.47 \Rightarrow 0.5$
$P_L = 80(0.5)(560\ ft^2) = 22.4\ K$
$P_U = 1.2(46.5) + 1.6(22.4) = \underline{91.7\ K}$

3rd LEVEL
↳ $P_D = 46.5\ K$
↳ $LL_r = 0.25 + \left(\frac{15}{\sqrt{4(3)(560)}}\right) = 0.43 \rightarrow 0.4$
$P_L = 80(0.4)(560\ ft^2) = 17.9\ K$
$P_U = 1.2(46.5) + 1.6(17.9) = \underline{84.4\ K}$

2ND LEVEL
↳ IDENTICAL TO 3rd, $P_U = \underline{84.4\ K}$

COLUMN 1

$P_U = 76.2 + 166 + 98.9 + 91.7 + 84.4 + 84.4 = 602\ K$ w/ $KL = 15.17'$

PER AISC TABLE 4-1, USE W12x65 ($\phi P_n = 662\ K$ @ 15')

COLUMN 2

$P_U = 76.2 + 166 + 98.9 + 91.7 = 433\ K$ w/ $KL = 14'$

PER AISC TABLE 4-1, USE W12x53 ($\phi P_n = 501\ K$ @ 14')

COLUMN 3

$P_U = 76.2 + 166 = 242\ K$ w/ $KL = 14'-7"$ (USE 15')
$P_U = 76.2\ K$ w/ $KL = 22'-6"$ (USE 23')
PER AISC TABLE 4-1, USE W12x40 ($\phi P_n = 280\ K$ @ 15' & $\phi P_n = 131\ K$ @ 23')

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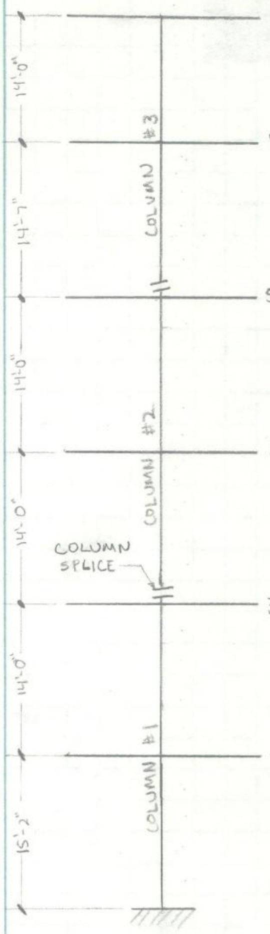
COLUMN U/2 → TRIBUTARY AREA : 21'-0" x 18'-3" (383 ft²)



CHILLER ROOF
↳ FRAMING → 10 psf
↳ SLAB/DECK → 75 psf
↳ SDL → 80 psf
↳ LL → 100 psf

PENTHOUSE
↳ FRAMING → 10 psf
↳ SLAB/DECK → 63 psf
↳ SDL → 90 psf
↳ LL → 150 psf

5TH LEVEL
↳ FRAMING → 10 psf
↳ SLAB/DECK → 63 psf
↳ SDL → 70 psf
↳ LL → 150 psf

4TH LEVEL
↳ FRAMING → 5 psf
↳ SLAB/DECK → 63 psf
↳ SDL → 15 psf
↳ LL → 80 psf

3rd LEVEL
↳ FRAMING → 5 psf
↳ SLAB/DECK → 63 psf
↳ SDL → 15 psf
↳ LL → 80 psf

2ND LEVEL
↳ FRAMING → 5 psf
↳ SLAB/DECK → 63 psf
↳ SDL → 15 psf
↳ LL → 80 psf

COLUMN LOADS:

ROOF
↳ $P_D = (10+75+80)(383\ ft^2) = 63.2K$
↳ $P_L = 100(383\ ft^2) = 38.3\ K$
$P_U = 1.2(63.2) + 1.6(38.3) = \underline{137\ K}$

PENTHOUSE
↳ $P_D = (10+63+90)(383\ ft^2) = 62.4\ K$
↳ $P_L = 150(383\ ft^2) = 57.5\ K$
$P_U = 1.2(62.4) + 1.6(57.5) = \underline{167\ K}$

5TH LEVEL
↳ $P_D = (10+63+70)(383\ ft^2) = 54.8K$
↳ $P_L = 57.5\ K$
$P_U = 1.2(54.8K) + 1.6(57.5K) = \underline{158K}$

4TH LEVEL
↳ $P_D = (5+63+15)(383\ ft^2) = 31.8K$
↳ $LL_r = 0.25 + \left(\frac{15}{\sqrt{4(383)}}\right) = 0.63$
$P_L = 80(0.63)(383\ ft^2) = 19.3\ K$
$P_U = 1.2(31.8) + 1.6(19.3) = \underline{69.0\ K}$

3rd LEVEL
↳ $P_D = 31.8\ K$
↳ $LL_r = 0.25 + \left(\frac{15}{\sqrt{4(2)(383)}}\right) = 0.52$
$P_L = 80(0.52)(383\ ft^2) = 15.9\ K$
$P_U = 1.2(31.8) + 1.6(15.9) = \underline{63.6\ K}$

2ND LEVEL
↳ $P_D = 31.8K$
↳ $LL_r = 0.25 + \left(\frac{15}{\sqrt{4(3)(383)}}\right) = 0.47$
$P_L = 80(0.47)(383\ ft^2) = 14.4\ K$
$P_U = 1.2(31.8) + 1.6(14.4) = \underline{61.2\ K}$

COLUMN 1
$P_U = 137+167+158+69.0+63.6+61.2 = 656\ K$ w/ KL = 15.17'

PER AISC TABLE 4-1, USE W12x72 ($\phi P_n = 736\ K$ @ 15')

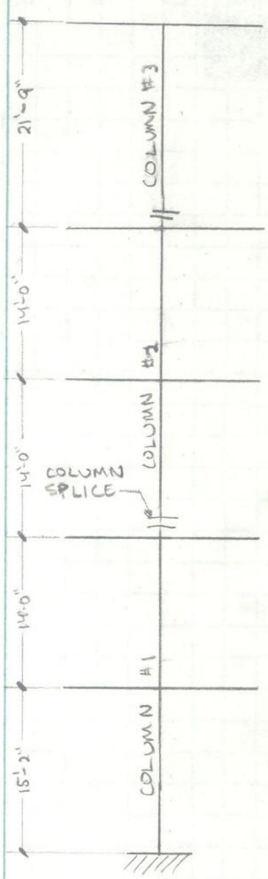
COLUMN 2
$P_U = 137+167+158+69.0 = 531\ K$ w/ KL = 14'

PER AISC TABLE 4-1, USE W12x58 ($\phi P_n = 553\ K$ @ 14')

COLUMN 3
$P_U = 137+167 = 304\ K$ w/ KL = 14'-7"

PER AISC TABLE 4-1, USE W12x45 ($\phi P_n = 317$ @ 15')

COLUMN Q/10 → TRIBUTARY AREA: 21'-0" x 18'-3" (383 ft²)



ATRIUM ROOF
- FRAMING → 5 psf
- DECK → 5 psf
- SDL → 70 psf
- SL → 25 psf

5TH LEVEL
- FRAMING → 10 psf
- SLAB/DECK → 63 psf
- SDL → 95 psf
- LL → 100 psf

4TH LEVEL
- FRAMING → 5 psf
- SLAB/DECK → 63 psf
- SDL → 15 psf
- LL → 80 psf

3RD LEVEL
- FRAMING → 5 psf
- SLAB/DECK → 63 psf
- SDL → 15 psf
- LL → 80 psf

2ND LEVEL
- FRAMING → 5 psf
- SLAB/DECK → 63 psf
- SDL → 15 psf
- LL → 80 psf

COLUMN LOADS:

ROOF
- $P_D = (5+5+70)(383^{\boxslash}) = 30.6\ \text{K}$
- $P_L = 25\,(383^{\boxslash}) = 9.6\ \text{K}$

$P_U = 1.2(30.6) + 1.6(9.6) = \underline{52.1\ \text{K}}$

5TH LEVEL
- $P_D = (10+63+95)(383^{\boxslash}) = 64.3\ \text{K}$
- $P_L = 100\,(383^{\boxslash}) = 38.3\ \text{K}$

$P_U = 1.2(64.3) + 1.6(38.3) = \underline{138\ \text{K}}$

4TH LEVEL
- $P_D = (5+63+15)(383^{\boxslash}) = 31.8\ \text{K}$
- $LL_r = 0.25 + \left(\frac{15}{\sqrt{4(383)}}\right) = 0.63$

$P_L = 80(0.63)(383) = 19.3\ \text{K}$

$P_U = 1.2(31.8) + 1.6(19.3) = \underline{69.0\ \text{K}}$

3RD LEVEL
- $P_D = 31.8\ \text{K}$
- $LL_r = 0.25 + \left(\frac{15}{\sqrt{4(2)(383)}}\right) = 0.52$

$P_L = 80(0.52)(383^{\boxslash}) = 15.9\ \text{K}$

$P_U = 1.2(31.8) + 1.6(15.9) = \underline{63.6\ \text{K}}$

2ND LEVEL
- $P_D = 31.8\ \text{K}$
- $LL_r = 0.25 + \left(\frac{15}{\sqrt{4(3)(383)}}\right) = 0.47$

$P_L = 80(0.47)(383^{\boxslash}) = 14.4\ \text{K}$

$P_U = 1.2(31.8) + 1.6(14.4) = \underline{61.2\ \text{K}}$

COLUMN 1

$P_U = 52.1 + 138 + 69 + 63.6 + 61.2 = 384\ \text{K}$ w/ $KL = 15.17'$

PER AISC TABLE 4-1, USE W10x49 ($\phi P_n = 450$ K @ 15')

COLUMN 2

$P_U = 52.1 + 138 + 69 = 259\ \text{K}$ w/ $KL = 14'$

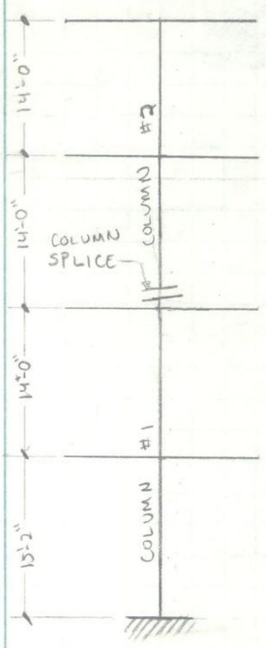
PER AISC TABLE 4-1, USE W10x39 ($\phi P_n = 305$ K @ 14')

COLUMN 3

$P_U = 52.1\ \text{K}$ w/ $KL = 21.75'$

PER AISC TABLE 4-1, USE W10x33 ($\phi P_n = 118$ K @ 22')

COLUMN F1/6 → TRIBUTARY AREA: 19'-6" x 18'-7" (362 ft²)



5TH LEVEL/ ROOF
↳ FRAMING → 5psf
↳ SLAB/DECK → 63psf
↳ SDL → 71 psf
↳ RL → 50 psf

4TH LEVEL
↳ FRAMING → 5psf
↳ SLAB/DECK → 63 psf
↳ SDL → 15 psf
↳ LL → 70 psf

3RD LEVEL
↳ FRAMING → 5 psf
↳ SLAB/DECK → 63psf
↳ SDL → 15 psf
↳ LL → 70 psf

2ND LEVEL
↳ FRAMING → 5 psf
↳ SLAB/DECK → 63 psf
↳ SDL → 15 psf
↳ LL → 70 psf

COLUMN LOADS:

ROOF
↳ $P_D = (5+63+71)(362^{\square}) = 50.3\text{ K}$
↳ $P_L = 50(362^{\square}) = 18.1\text{ K}$
$P_U = 1.2(50.3) + 1.6(18.1) = \underline{89.3\text{ K}}$

4TH LEVEL
↳ $P_D = (5+63+15)(362^{\square}) = 30.0\text{ K}$
↳ $LL_r = 0.25 + \left(\frac{15}{\sqrt{4(362)}}\right) = 0.64$
$P_L = 70(0.64)(362^{\square}) = 16.2\text{ K}$
$P_U = 1.2(30) + 1.6(16.2) = \underline{61.9\text{ K}}$

3RD LEVEL
↳ $P_D = 30.0\text{ K}$
↳ $LL_r = 0.25 + \left(\frac{15}{\sqrt{4(2)(362)}}\right) = 0.53$
$P_L = 70(0.53)(362) = 13.4\text{ K}$
$P_U = 1.2(30) + 1.6(13.4) = \underline{57.4\text{ K}}$

2ND LEVEL
↳ $P_D = 30.0\text{ K}$
↳ $LL_r = 0.25 + \left(\frac{15}{\sqrt{4(3)(362)}}\right) = 0.48$
$P_L = 70(0.48)(362) = 12.2\text{ K}$
$P_U = 1.2(30) + 1.6(12.2) = \underline{55.5\text{ K}}$

COLUMN 1

$P_U = 89.3 + 61.9 + 57.4 + 55.5\text{ K} = 264\text{ K}$ w/ $KL = 15.17'$

PER AISC TABLE 4-1, USE W10x39 ($\phi P_n = 282\text{k}$ @ 15')

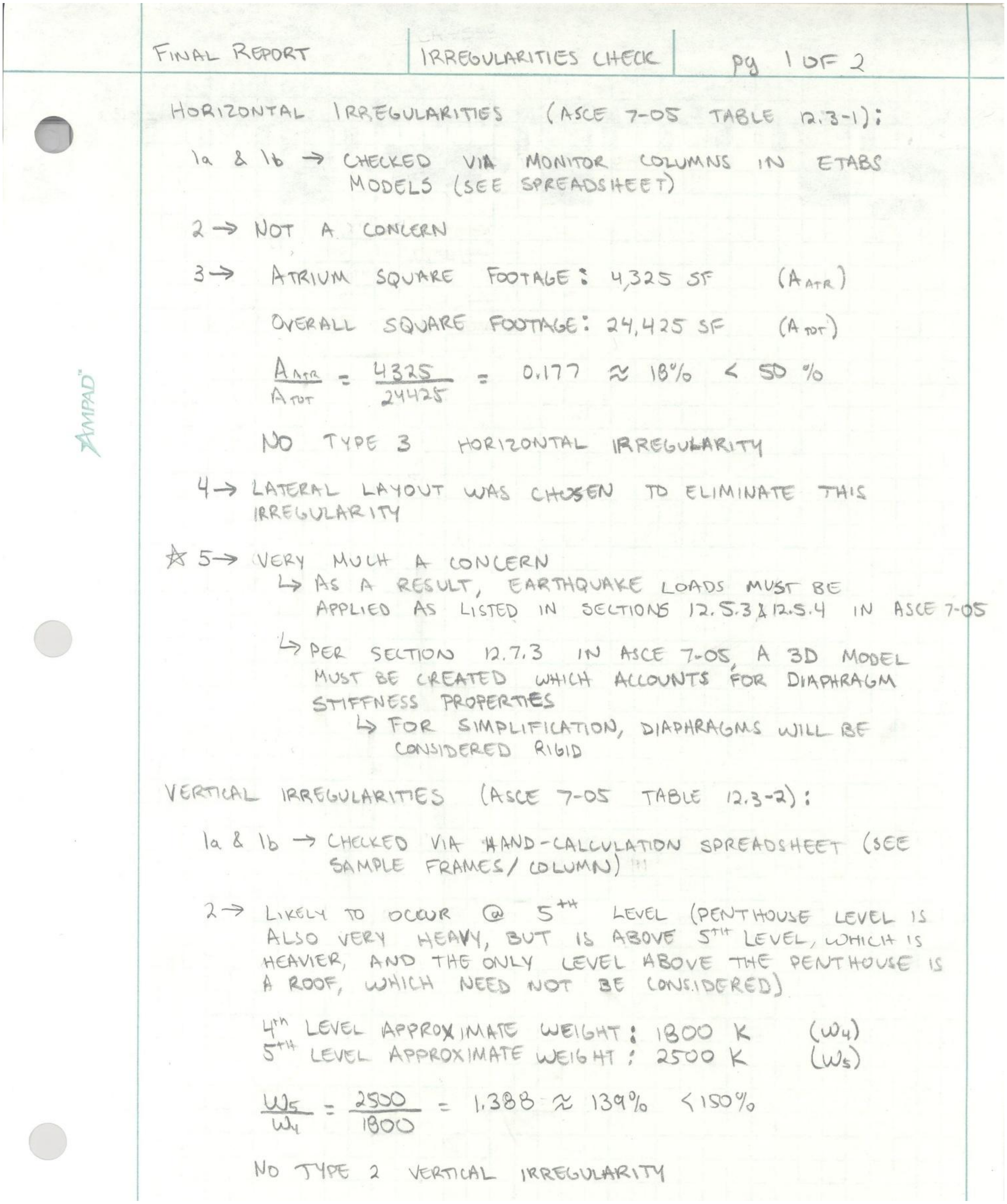
COLUMN 2

$P_U = 89.3 + 61.9 = 151\text{ K}$ w/ $KL = 14'$

PER AISC TABLE 4-1, USE W10x33 ($\phi P_n = 253\text{k}$ @ 14')

Appendix E: Moment Frame Calculations

Irregularities Check



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HORIZONTAL IRREGULARITIES (ASCE 7-05 TABLE 12.3-1):

1a & 1b → CHECKED VIA MONITOR COLUMNS IN ETABS MODELS (SEE SPREADSHEET)

2 → NOT A CONCERN

3 → ATRIUM SQUARE FOOTAGE: 4,325 SF (A_{ATR})

OVERALL SQUARE FOOTAGE: 24,425 SF (A_{TOT})

$$\frac{A_{ATR}}{A_{TOT}} = \frac{4325}{24425} = 0.177 \approx 18\% < 50\%$$

NO TYPE 3 HORIZONTAL IRREGULARITY

4 → LATERAL LAYOUT WAS CHOSEN TO ELIMINATE THIS IRREGULARITY

☆ 5 → VERY MUCH A CONCERN

↳ AS A RESULT, EARTHQUAKE LOADS MUST BE APPLIED AS LISTED IN SECTIONS 12.5.3 & 12.5.4 IN ASCE 7-05

↳ PER SECTION 12.7.3 IN ASCE 7-05, A 3D MODEL MUST BE CREATED WHICH ACCOUNTS FOR DIAPHRAGM STIFFNESS PROPERTIES

↳ FOR SIMPLIFICATION, DIAPHRAGMS WILL BE CONSIDERED RIGID

VERTICAL IRREGULARITIES (ASCE 7-05 TABLE 12.3-2):

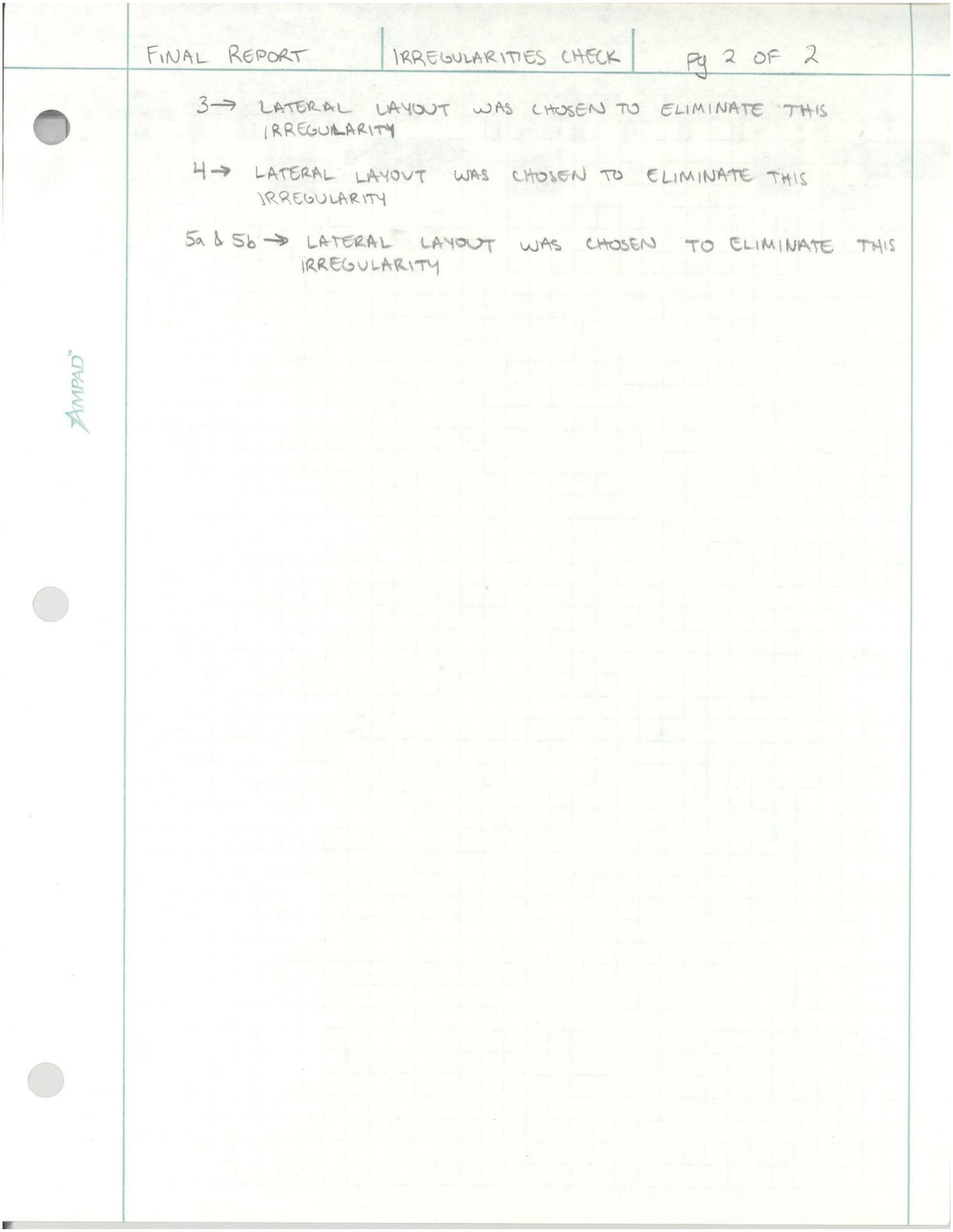
1a & 1b → CHECKED VIA HAND-CALCULATION SPREADSHEET (SEE SAMPLE FRAMES/COLUMN)

2 → LIKELY TO OCCUR @ 5TH LEVEL (PENTHOUSE LEVEL IS ALSO VERY HEAVY, BUT IS ABOVE 5TH LEVEL, WHICH IS HEAVIER, AND THE ONLY LEVEL ABOVE THE PENTHOUSE IS A ROOF, WHICH NEED NOT BE CONSIDERED)

4TH LEVEL APPROXIMATE WEIGHT: 1800 K (W_4)
5TH LEVEL APPROXIMATE WEIGHT: 2500 K (W_5)

$$\frac{W_5}{W_4} = \frac{2500}{1800} = 1.388 \approx 139\% < 150\%$$

NO TYPE 2 VERTICAL IRREGULARITY



FINAL REPORT | IRREGULARITIES CHECK | Pg 2 OF 2

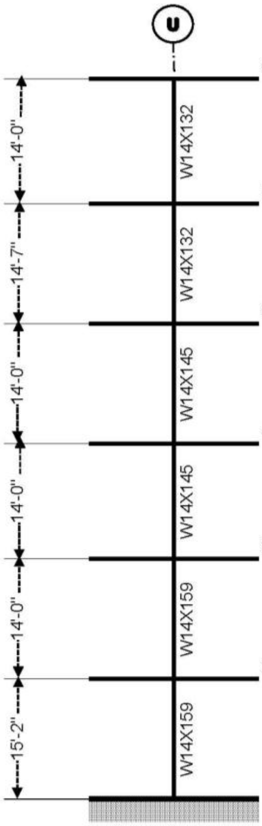
3 → LATERAL LAYOUT WAS CHOSEN TO ELIMINATE THIS IRREGULARITY

4 → LATERAL LAYOUT WAS CHOSEN TO ELIMINATE THIS IRREGULARITY

5a & 5b → LATERAL LAYOUT WAS CHOSEN TO ELIMINATE THIS IRREGULARITY

CA S-3

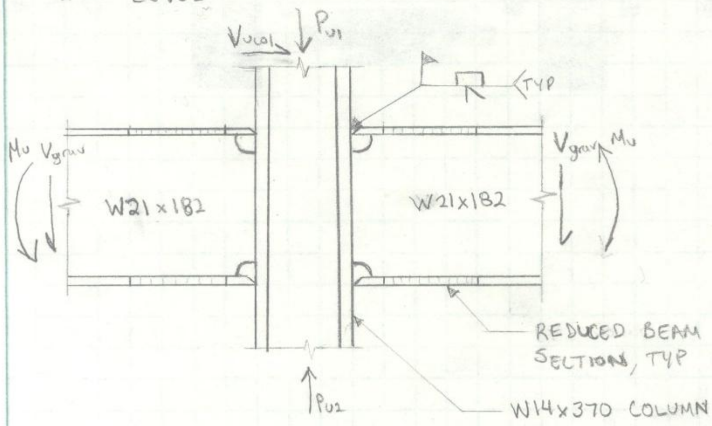
CA S-3 - Column U/12 Interaction Check



Chiller Mechanical Room Roof			
$P_{U,grav}$ (k)= 137.00	p (k^{-1}) = 6.63E-04	pP_U = 0.09	H1-1b
$M_{EQ,X}$ (k-ft)= 27.31	b_X (k-ft^{-1}) = 1.02E-03	Int. 0.08	O.K.
$M_{EQ,Y}$ (k-ft)= 0.00	b_Y (k-ft^{-1}) = 2.10E-03		
Penthouse Level			
$P_{U,grav}$ (k)= 304.00	p (k^{-1}) = 6.78E-04	pP_U = 0.21	H1-1a
$M_{EQ,X}$ (k-ft)= 100.29	b_X (k-ft^{-1}) = 1.03E-03	Int. 0.31	O.K.
$M_{EQ,Y}$ (k-ft)= 0.00	b_Y (k-ft^{-1}) = 2.10E-03		
5th Level			
$P_{U,grav}$ (k)= 462.00	p (k^{-1}) = 5.93E-04	pP_U = 0.27	H1-1a
$M_{EQ,X}$ (k-ft)= 161.41	b_X (k-ft^{-1}) = 9.12E-04	Int. 0.42	O.K.
$M_{EQ,Y}$ (k-ft)= 0.00	b_Y (k-ft^{-1}) = 1.78E-03		
4th Level			
$P_{U,grav}$ (k)= 531.00	p (k^{-1}) = 5.93E-04	pP_U = 0.31	H1-1a
$M_{EQ,X}$ (k-ft)= 192.56	b_X (k-ft^{-1}) = 9.12E-04	Int. 0.49	O.K.
$M_{EQ,Y}$ (k-ft)= 0.00	b_Y (k-ft^{-1}) = 1.78E-03		
3rd Level			
$P_{U,grav}$ (k)= 594.60	p (k^{-1}) = 5.41E-04	pP_U = 0.32	H1-1a
$M_{EQ,X}$ (k-ft)= 209.55	b_X (k-ft^{-1}) = 8.26E-04	Int. 0.49	O.K.
$M_{EQ,Y}$ (k-ft)= 0.00	b_Y (k-ft^{-1}) = 1.62E-03		
2nd Level			
$P_{U,grav}$ (k)= 655.80	p (k^{-1}) = 5.51E-04	pP_U = 0.36	H1-1a
$M_{EQ,X}$ (k-ft)= 233.20	b_X (k-ft^{-1}) = 8.31E-04	Int. 0.56	O.K.
$M_{EQ,Y}$ (k-ft)= 0.00	b_Y (k-ft^{-1}) = 1.62E-03		

FINAL REPORT | CA S-3 BM.-COL. CONN. | PG 1 OF 4

DESIGN REPRESENTATIVE CONNECTION FOR COLUMN D/2 ON 2^{ND} LEVEL



$M_u = 471\ k\cdot ft$

$V_{grav} = 57\ k$

$P_{u1} = 543\ K$

$P_{u2} = 623\ K$

$V_{u,col} = 262\ K$

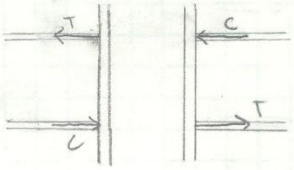
CHECK BEAM → $\phi M_p = 1790\ k\cdot ft >> M_u$ OK✓

$\phi V_n = 566\ K >> V_{grav}$ OK✓

DESIGN BEAM-TO-COLUMN CONNECTION →

PER AISC 358-05 § 5.5 & 5.6, BEAM FLANGES AND WEBS MUST BE WELDED TO COLUMN WITH COMPLETE JOINT PENETRATION GROOVE WELDS. THESE DEVELOP THE FULL STRENGTH OF THE BASE MATERIAL. SINCE THE BEAM IS OK UNDER APPLIED LOADS, THE CONNECTION WILL BE ALSO.

CHECK COLUMN-SIDE LIMIT STATES →



$$T = C = \frac{M_u}{d_b - t_{fb}} = \frac{471\ k\cdot ft\ (12\ in/ft)}{22.7\ in - 1.48\ in} = 266\ K$$

LOCAL FLANGE BENDING →

$$T_u \le \phi R_n = \phi 6.25\, t_{fc}^2\, F_{yc} = 0.9\ (6.25)(2.66\ in)^2(50\ ksi) = 1990\ K$$

$\underline{OK✓}$

LOCAL WEB YIELDING →

$$T_u, C_u \le \phi R_n = \phi\ F_{yc}\,(5K_{des,c} + t_{fb})\, t_{wc} = 1.0(50\ ksi)[5(3.26in) + 1.48](1.66in)$$
$$= 1476\ K$$

$\underline{OK✓}$

FINAL REPORT | CA S-3 BM.-COL. CONN. | pg 2 OF 4

LOCAL WEB CRIPPLING → $\frac{N}{d} = \frac{1.48}{22.7} = 0.07 < 0.2$

$$C_u < \phi R_n = \phi\, 0.8\, t_{wc}\left[1 + 3\left(\frac{t_{fb}}{d}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{E\, F_{yw}\, t_{fc}}{t_{wc}}}$$

$$\phi R_n = 0.75\,(0.8)\,(1.66\text{in})\left[1 + 3\left(\frac{1.48}{22.7}\right)\left(\frac{1.66}{2.66}\right)^{1.5}\right]\sqrt{\frac{29000\text{ ksi}\,(50\text{ ksi})\,(2.66\text{in})}{1.66\text{ in}}}$$

$$= 1{,}665\text{ K} > C_u \quad \underline{\text{OK}} ✓$$

WEB BUCKLING →

NOT APPLICABLE FOR LATERAL LOAD CASES

PANEL ZONE SHEAR YIELDING →

$$F_u = 2(266\text{ K}) - 262\text{K} = 270\text{ K}$$

$$P_r = \frac{P_{u1} + P_{u2}}{2} = \frac{543 + 623}{2} = 583\text{ K}$$

$$P_c = A_g F_y = 109\text{ in}^2(50\text{ ksi}) = 5450\text{ K}$$

$P_r < 0.75P_c = 4088\text{ K}$

$$F_u < \phi R_n = \phi\, 0.6\, F_y\, d_c\, t_{wc}\left(1 + \frac{3 b_{cf}\, t_{cf}^2}{d_b\, d_c\, t_{wc}}\right)$$

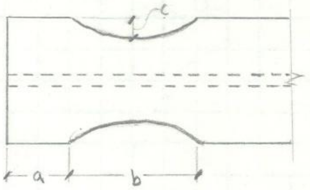
$$\phi R_n = 0.9\,(0.6)(50\text{ksi})(17.9\text{ in})(1.66\text{in})\left(1 + \frac{3\,(16.5\text{in})(2.66\text{in})^2}{22.7\text{in}(17.9\text{in})(1.66\text{in})}\right) = 1{,}219\text{ K}$$

$\underline{\text{OK}}$ ✓

NO STIFFENERS REQUIRED FOR STRENGTH DESIGN

REDUCED BEAM SECTION DESIGN →

GOVERNED BY AISC 358-05 AND AISC 341-05



TRIAL RBS DIMENSIONS

$a \approx 0.5 b_f = 0.5(12.5) = 6.25''$

$b \approx 0.65 d = 0.65(22.7) = 14.8'' \rightarrow 15''$

$c \approx 0.2 b_f = 0.2(12.5) = 2.5''$

$S_h = a + b/2 = 13.75''$

$$Z_{RBS} = Z_x - 2c\,t_{bf}\,(d - t_{bf}) = 476 - 2\,(2.5)(1.48)(22.7 - 1.48) = 318.97\text{ in}^3$$

$$M_{pr} = C_{pr}\, R_y\, F_y\, Z_{RBS} = 1.15\,(1.1)(50\text{ ksi})(318.97\text{ in}^3)(1\text{ft}/12\text{in}) = 1681\text{ k}\cdot\text{ft}$$

$$V_{RBS} = V_{grav} + \frac{2M_{pe}}{L'} = 57\text{K} + \frac{2(476\text{ in}^3)(50\text{ksi})(1.1)(1\text{ft}/12\text{in})}{21\text{ ft}} = 264\text{ K}$$

$$M_{pb}^* = M_{pr} + M_v = M_{pr} + V_{RBS}\,(S_h + d_c/2) = 1681 + 264(13.75 + 17.9/2)(1\text{ft}/12\text{in})$$
$$= 2180\text{ k}\cdot\text{ft}$$

$$\Sigma M_{pb}^* = 2(2180) = 4360\text{ k}\cdot\text{ft}$$

FINAL REPORT | CA S-3 BM.-COL. CONN. | pg 3 OF 4

$M_{pc}^{*} = Z_c \left(F_{yc} - {}^{P_u}/_{A_g}\right) = 736 \text{ in}^3 \left(50 \text{ ksi} - {}^{583}/_{109}\right)\left({}^{1\text{ft}}/_{12\text{in}}\right) = 2739 \text{ k-ft}$

$\Sigma M_{pc}^{*} = 2(2739 \text{ k.ft}) = 5478 \text{ k.ft}$

$\dfrac{\Sigma M_{pc}^{*}}{\Sigma M_{pb}^{*}} = 1.26 > 1.0$ OK ✓

$M_{pe} = Z_b R_y F_y = 476 \text{ in}^3 (1.1)(50 \text{ ksi})\left({}^{1\text{ft}}/_{12\text{in}}\right) = 2182 \text{ k.ft}$

$M_f = M_{pr} + V_{RBS} S_h = 1681 + 284\text{k}(13.75 \text{ in})\left({}^{1\text{ft}}/_{12\text{ in}}\right) = 1984 \text{ k.ft}$

$M_f < \underset{=1.0}{\underbrace{\phi_d}} M_{pe}$ OK ✓

$V_u = \dfrac{2M_{pr}}{L'} + V_{grav} = \dfrac{2(1681)}{18 \text{ ft}} + 57 = 244 \text{ K} < \phi V_n$ OK ✓

$\phi M_{p,RBS} = 0.9(319 \text{ in}^3)(50 \text{ ksi})\left({}^{1\text{ft}}/_{12\text{in}}\right) = 1196 \text{ k.ft} > M_u$ OK ✓

CHECK GEOMETRY/WEIGHT LIMITATIONS →

PER § 5.3.1 IN AISC 358-05, BEAM LIMITATIONS ARE:

- ↳ MAX SIZE = W36 > W21 OK ✓
- ↳ MAX WT. = 300 plf > 182 plf OK ✓
- ↳ $t_{f,max,b} = 1.75'' > 1.48''$ OK ✓
- ↳ $\left(\dfrac{\text{span}}{d}\right)_{min} = 7 < \dfrac{21 \times 12 - 17.9 \text{ in}}{22.7 \text{ in}} = 10.3$ OK ✓
- ↳ $b_f/2t_f = 4.22 < 0.3\sqrt{\dfrac{E}{F_y}} = 7.22$ OK ✓
- ↳ $h/t_w = 22.6 < 2.45\sqrt{E/F_y} = 59$ OK ✓

PER §5.3.2 IN AISC 358-05, COLUMN LIMITATIONS ARE:

- ↳ MAX SIZE = W36 > W14 OK ✓
- ↳ $b_f/2t_f = 3.10 < 7.22$ OK ✓
- ↳ $h/t_w = 6.89 < 59$ OK ✓

CHECK CONTINUITY PLATE REQUIREMENTS →

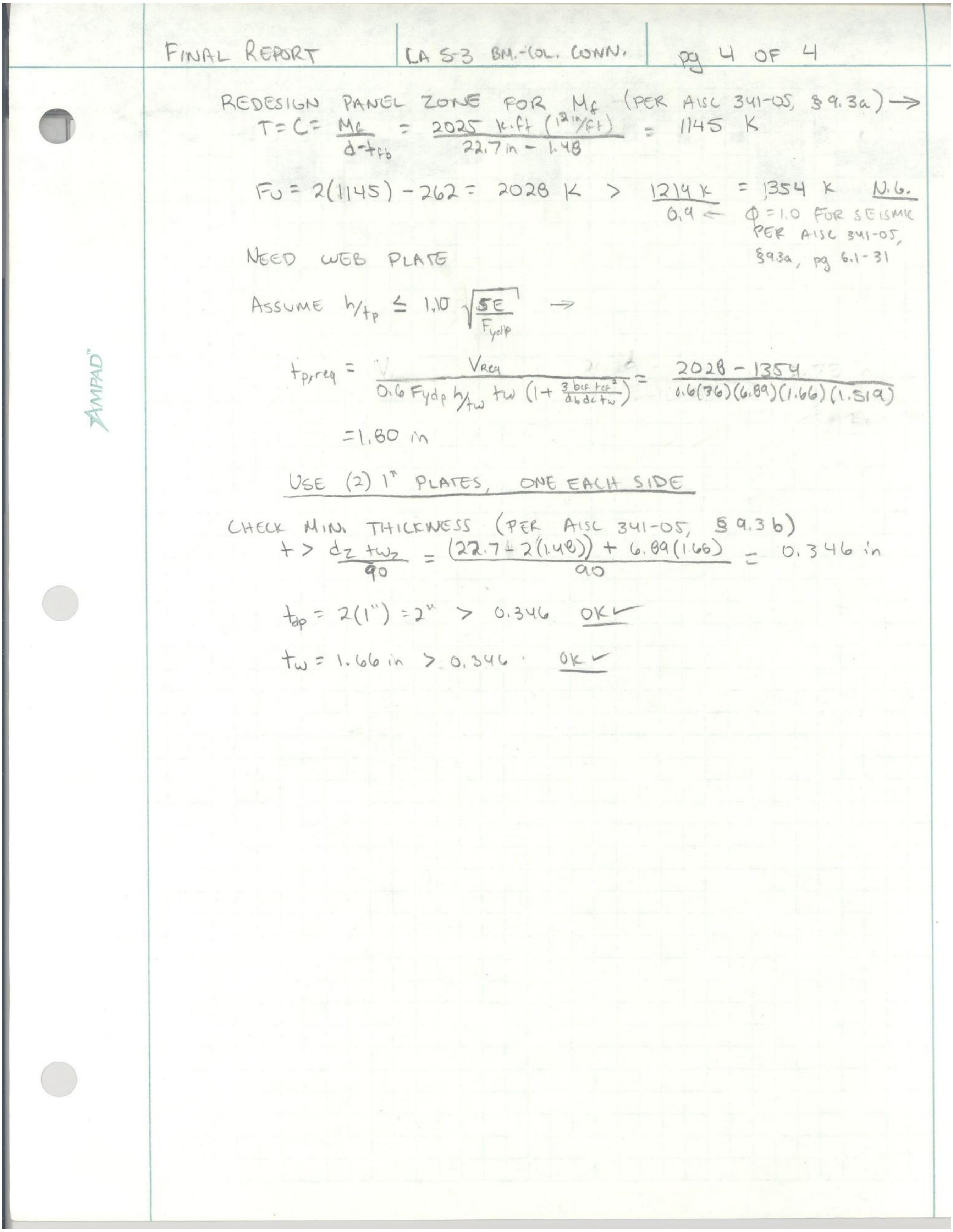
PER § 2.4.4 IN AISC 358-05, CONTINUITY PLATES NEED NOT BE PROVIDED IF

$t_{cf} = 2.66 \text{ in} \geq 0.4\sqrt{1.8\, b_{bf} t_{bf} \dfrac{F_{yb} \cancelto{1}{R_{yb}}}{\cancel{F_{yc}} R_{yc}}} = 0.4\sqrt{1.8(12.5)(1.48)} = 2.31 \text{ in}$ OK ✓

-AND-

$t_{cf} \geq \dfrac{b_{bf}}{6} = \dfrac{12.5}{6} = 2.08 \text{ in}$ OK ✓

CONTINUITY PLATES NOT REQUIRED



FINAL REPORT | CA S-3 BM.-COL. CONN. | pg 4 OF 4

REDESIGN PANEL ZONE FOR M_f (PER AISC 341-05, § 9.3a) →

$$T = C = \frac{M_f}{d - t_{fb}} = \frac{2025\ k\cdot ft\ (12\ in/ft)}{22.7\ in - 1.48} = 1145\ K$$

$$F_u = 2(1145) - 262 = 2028\ K > \frac{1214\ k}{0.9} = 1354\ K \quad \underline{N.G.}$$

0.9 ← $\phi = 1.0$ FOR SEISMIC PER AISC 341-05, §9.3a, pg 6.1-31

NEED WEB PLATE

ASSUME $h/t_p \leq 1.10\sqrt{\frac{5E}{F_{ydp}}}$ →

$$t_{p,req} = \frac{V_{req}}{0.6 F_{ydp}\, h/t_w\, t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w}\right)} = \frac{2028 - 1354}{0.6(36)(6.89)(1.66)(1.519)}$$

$= 1.80$ in

<u>USE (2) 1" PLATES, ONE EACH SIDE</u>

CHECK MIN. THICKNESS (PER AISC 341-05, § 9.3b)

$$t > \frac{d_z t_{wz}}{90} = \frac{(22.7 - 2(1.48)) + 6.89(1.66)}{90} = 0.346\ in$$

$t_{dp} = 2(1") = 2" > 0.346$ $\underline{OK}$ ✓

$t_w = 1.66$ in > 0.346 $\underline{OK}$ ✓

FINAL REPORT | CA S-3 BM. BRACING | pg 1 OF 3

PER AISC 341-05 § 9.8, BOTH FLANGES OF BEAMS MUST BE BRACED @

$$L_b = \frac{0.086 r_y E}{F_y} = \frac{0.086(3.00\text{ in})(29000\text{ ksi})(1\text{ft}/12\text{ in})}{50\text{ ksi}} = 12.47\text{ ft}$$
$$= 150\text{ in}$$

FOR A STRENGTH OF

$$P_{br} = \frac{0.02 R_y Z F_y}{h_o} = \frac{0.02(1.1)(476)(50\text{ ksi})}{22.7\text{ in} - 1.48\text{ in}} = 24.67\text{ k}$$

AND A STIFFNESS OF

$$\beta_{br} = \frac{1}{\phi}\left(\frac{10 R_y Z_x F_y}{L_b h_o}\right) = \frac{1}{0.75}\left(\frac{10(1.1)(476)(50\text{ ksi})}{126\text{ in }(22.7-1.48)}\right)$$

↖ BRACE @ MID-LENGTH = 10.5 ft = 126" < 150" OK✓

$$= 130.55\text{ k/in}$$

TRY L5x5x5/16

LENGTH → SPAN TO NEAREST BEAM → $L_x = 12'\text{-}1"$, $L_y = 22.7"$

$$L = \sqrt{145^2 + 22.7^2} = 147" = 12.23'$$

FROM TABLE 4-11 IN AISC SPECIFICATION, @ KL = 13 ft,
$\phi P_n = 27.6\text{ k} > P_{br}$ OK✓

$$\text{STIFFNESS} = \frac{AE}{L} = \frac{3.03\text{ in}^2(29000\text{ ksi})}{147\text{ in}} = 598\text{ k/in} > \beta_{br}$$
OK✓

USE (1) L5x5x5/16 @ MIDSPAN

ALSO, BRACES MUST BE PROVIDED @ RBS w/ STRENGTH OF

$$P_{br} = \frac{0.06 R_y Z_{RBS} F_y}{h_o} = \frac{0.06(1.1)(319\text{ in}^3)(50\text{ ksi})}{22.7-1.48\text{ in}} = 49.61\text{ k}$$

AND A STIFFNESS OF

$$\beta_{br} = \frac{1}{\phi}\left(\frac{10 R_y Z_{RBS} F_y}{L_b h_o}\right) = \frac{1}{0.75}\left(\frac{10(1.1)(319)(50)}{126\text{ in }(22.7-1.48)}\right) = 87.5\text{ k/in}$$

TRY L5x5x5/8

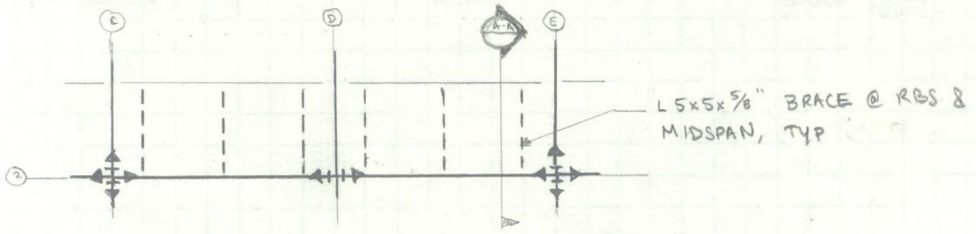
FROM TABLE 4-11 IN AISC SPEC., @ KL = 13 ft,
$\phi P_n = 51.7\text{ k} > P_{br}$ OK✓

$$\text{STIFFNESS} = \frac{5.86\text{ in}^2(29000\text{ ksi})}{147\text{ in}} = 1156\text{ k/in} > \beta_{br}$$

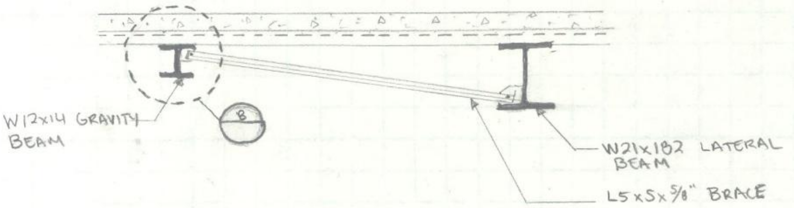
FINAL REPORT | CA S-3 BM. BRACING | pg 2 OF 3

USE (1) $L5 \times 5 \times 5/8$ @ EACH RBS

FOR CONSTRUCTABILITY, THE SAME BRACES SHOULD BE USED AT RBS & MIDSPAN



PARTIAL BRACING PLAN
SCALE: NTS



SECTION A-A
SCALE: NTS

DESIGN DETAIL "B" (BOLTS & PLATE TO CONNECT TO BEAM)
- USE 5/8" A36 PLATE TO MATCH ANGLE THICKNESS
- ONLY AT 9° ANGLE DOWN FROM HORIZONTAL
  ↳ DESIGN AS IF PERFECTLY HORIZONTAL

BOLTS REQUIRED → TRY A325-N BOLTS, 3/4" ∅

$$n = \frac{P_u}{\phi r_n} = \frac{49.61\text{ K}}{15.9\text{ K}} = 3.1 \rightarrow \underbrace{4\text{ BOLTS}}_{\text{IMPRACTICAL}}$$

TRY 1" ∅ A325N BOLTS

$$n = \frac{49.61\text{ K}}{28.3\text{ K}} = 1.75 \rightarrow \underline{2\text{ BOLTS}}$$

TENSION YIELD →

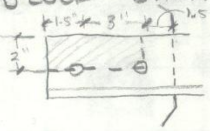
$$\phi R_n = \phi F_y A_g = 0.9(36\text{ ksi})(5.86\text{ in}^2) = 190\text{ K} > P_u\ \underline{\text{OK}}\ \checkmark$$

TENSION RUPTURE →

$$\phi R_n = \phi F_u A_e = 0.75(58\text{ ksi})\left[5.86 - 1(1'' + 1/8'')(5/8'')\right]\left(1 - \frac{1.47}{3}\right) = 114\text{ K} > P_u$$

FINAL REPORT | CA S-3 BM. BRACING | pg 3 OF 3

BLOCK SHEAR →



$R_T = F_u A_{nt} = 58 \text{ ksi} \left[\frac{5}{8}'' \left(2'' - \frac{1}{2}(1'' + \frac{1}{8}'') \right) \right] = 52.1 \text{ K}$

$Y_v = 0.6 F_y A_{gv} = 0.6(36 \text{ ksi})(\frac{5}{8}'')(4.5'') = 60.75 \text{ K} \leftarrow$

$R_v = 0.6 F_u A_{nv} = 0.6(58\text{ksi})(\frac{5}{8}'')(4.5'' - 1.5(1'' + \frac{1}{8}'')) = 61.2 \text{K}$

$\phi R_n = 0.75(52.1 + 60.75) = 84.6 \text{ K}$ OK ✓

BOLT SHEAR/BEARING/TEAR-OUT →

BOLT SHEAR → $\phi r_n = 28.3 \text{ K}$

BEARING → $\phi r_n = \phi 2.4 F_u t d_b = 0.75(2.4)(58 \text{ ksi})(\frac{5}{8}'')(1'') = 65.3 \text{ K}$

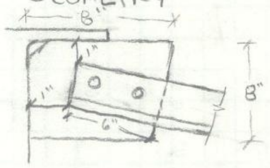
TEAR-OUT @ EDGE → $\phi r_n = \phi 1.2 F_u l_c t = 0.75(1.2)(58 \text{ksi})(\frac{5}{8}'')(1.5'' - 0.5(1'' + \frac{1}{8}''))$
$= 30.6 \text{ K}$

BOLT SHEAR CONTROLS ALL BOLTS → $\phi R_n = 2(28.3) = 56.6 \text{ K}$ OK ✓

WELD →

AVAILABLE BEAM FLANGE = $1\frac{3}{8}''$

GEOMETRY →

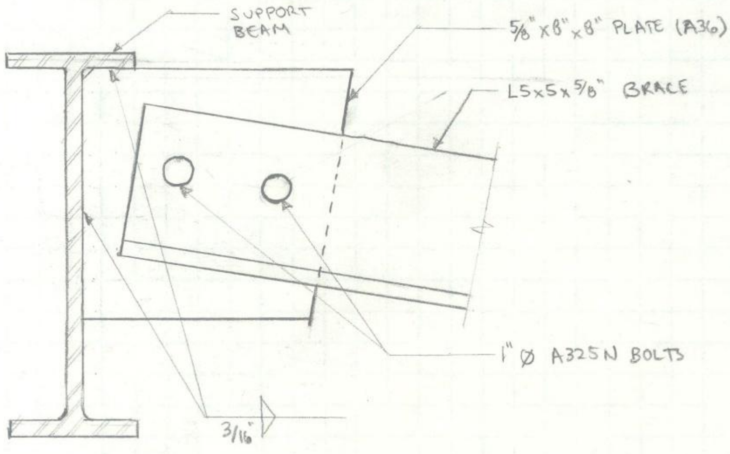


$\phi R_n = 1.392 D \left[1.5(8'' - \frac{1}{2}'') + (1.375'' - \frac{1}{2}'') \right](2) = 49.61$

FOR CLIP

$D = 1.5$

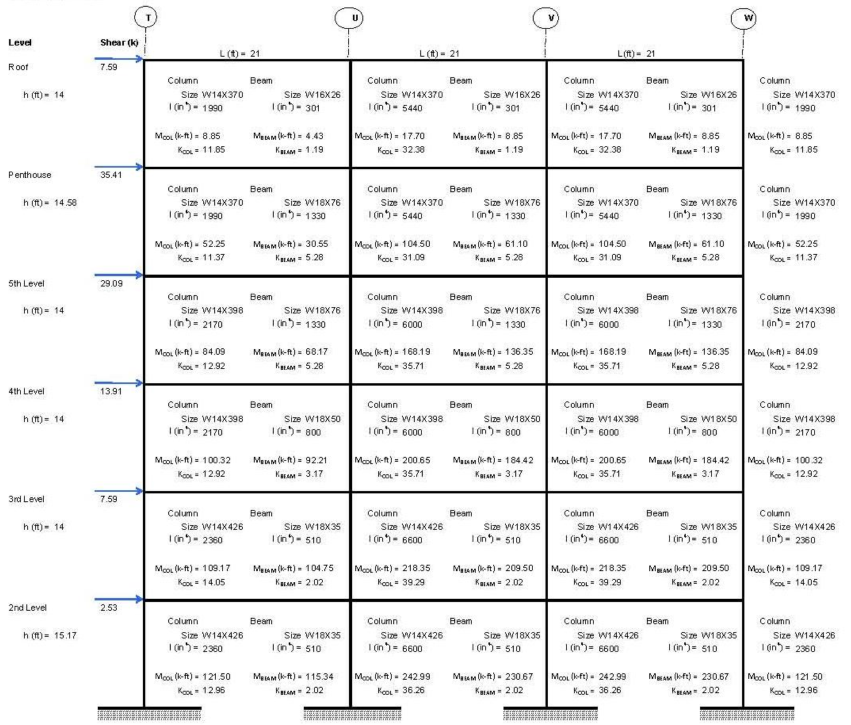
USE $\frac{3}{16}''$ FILLET WELD (MINIMUM)



DETAIL B
SCALE: NTS

CA S-1

CA S-1 - Frame #7

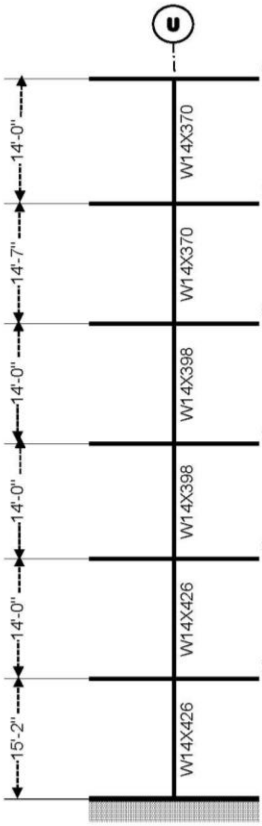


Level	Shear (k)	Column (T)	Beam (T–U)	Column (U)	Beam (U–V)	Column (V)	Beam (V–W)	Column (W)
Roof h (ft) = 14	7.59	Size W14X370 I (in^4) = 1990 M_{COL} (k-ft) = 8.85 K_{COL} = 11.85	Size W16X26 I (in^4) = 301 M_{BEAM} (k-ft) = 4.43 K_{BEAM} = 1.19	Size W14X370 I (in^4) = 5440 M_{COL} (k-ft) = 17.70 K_{COL} = 32.38	Size W16X26 I (in^4) = 301 M_{BEAM} (k-ft) = 8.85 K_{BEAM} = 1.19	Size W14X370 I (in^4) = 5440 M_{COL} (k-ft) = 17.70 K_{COL} = 32.38	Size W16X26 I (in^4) = 301 M_{BEAM} (k-ft) = 8.85 K_{BEAM} = 1.19	Size W14X370 I (in^4) = 1990 M_{COL} (k-ft) = 8.85 K_{COL} = 11.85
Penthouse h (ft) = 14.58	35.41	Size W14X370 I (in^4) = 1990 M_{COL} (k-ft) = 52.25 K_{COL} = 11.37	Size W18X76 I (in^4) = 1330 M_{BEAM} (k-ft) = 30.55 K_{BEAM} = 5.28	Size W14X370 I (in^4) = 5440 M_{COL} (k-ft) = 104.50 K_{COL} = 31.09	Size W18X76 I (in^4) = 1330 M_{BEAM} (k-ft) = 61.10 K_{BEAM} = 5.28	Size W14X370 I (in^4) = 5440 M_{COL} (k-ft) = 104.50 K_{COL} = 31.09	Size W18X76 I (in^4) = 1330 M_{BEAM} (k-ft) = 61.10 K_{BEAM} = 5.28	Size W14X370 I (in^4) = 1990 M_{COL} (k-ft) = 52.25 K_{COL} = 11.37
5th Level h (ft) = 14	29.09	Size W14X398 I (in^4) = 2170 M_{COL} (k-ft) = 84.09 K_{COL} = 12.92	Size W18X76 I (in^4) = 1330 M_{BEAM} (k-ft) = 68.17 K_{BEAM} = 5.28	Size W14X398 I (in^4) = 6000 M_{COL} (k-ft) = 168.19 K_{COL} = 35.71	Size W18X76 I (in^4) = 1330 M_{BEAM} (k-ft) = 136.35 K_{BEAM} = 5.28	Size W14X398 I (in^4) = 6000 M_{COL} (k-ft) = 168.19 K_{COL} = 35.71	Size W18X76 I (in^4) = 1330 M_{BEAM} (k-ft) = 136.35 K_{BEAM} = 5.28	Size W14X398 I (in^4) = 2170 M_{COL} (k-ft) = 84.09 K_{COL} = 12.92
4th Level h (ft) = 14	13.91	Size W14X398 I (in^4) = 2170 M_{COL} (k-ft) = 100.32 K_{COL} = 12.92	Size W18X50 I (in^4) = 800 M_{BEAM} (k-ft) = 92.21 K_{BEAM} = 3.17	Size W14X398 I (in^4) = 6000 M_{COL} (k-ft) = 200.65 K_{COL} = 35.71	Size W18X50 I (in^4) = 800 M_{BEAM} (k-ft) = 184.42 K_{BEAM} = 3.17	Size W14X398 I (in^4) = 6000 M_{COL} (k-ft) = 200.65 K_{COL} = 35.71	Size W18X50 I (in^4) = 800 M_{BEAM} (k-ft) = 184.42 K_{BEAM} = 3.17	Size W14X398 I (in^4) = 2170 M_{COL} (k-ft) = 100.32 K_{COL} = 12.92
3rd Level h (ft) = 14	7.59	Size W14X426 I (in^4) = 2360 M_{COL} (k-ft) = 109.17 K_{COL} = 14.05	Size W18X35 I (in^4) = 510 M_{BEAM} (k-ft) = 104.75 K_{BEAM} = 2.02	Size W14X426 I (in^4) = 6600 M_{COL} (k-ft) = 218.35 K_{COL} = 39.29	Size W18X35 I (in^4) = 510 M_{BEAM} (k-ft) = 209.50 K_{BEAM} = 2.02	Size W14X426 I (in^4) = 6600 M_{COL} (k-ft) = 218.35 K_{COL} = 39.29	Size W18X35 I (in^4) = 510 M_{BEAM} (k-ft) = 209.50 K_{BEAM} = 2.02	Size W14X426 I (in^4) = 2360 M_{COL} (k-ft) = 109.17 K_{COL} = 14.05
2nd Level h (ft) = 15.17	2.53	Size W14X426 I (in^4) = 2360 M_{COL} (k-ft) = 121.50 K_{COL} = 12.96	Size W18X35 I (in^4) = 510 M_{BEAM} (k-ft) = 115.34 K_{BEAM} = 2.02	Size W14X426 I (in^4) = 6600 M_{COL} (k-ft) = 242.99 K_{COL} = 36.26	Size W18X35 I (in^4) = 510 M_{BEAM} (k-ft) = 230.67 K_{BEAM} = 2.02	Size W14X426 I (in^4) = 6600 M_{COL} (k-ft) = 242.99 K_{COL} = 36.26	Size W18X35 I (in^4) = 510 M_{BEAM} (k-ft) = 230.67 K_{BEAM} = 2.02	Size W14X426 I (in^4) = 2360 M_{COL} (k-ft) = 121.50 K_{COL} = 12.96

	Soft Story Check	Beam Check	Strong Column-Weak Beam Check
K_{STORY}= 67.59 Δ_{STORY}(in) = 0.1123 h/ 1496.59 O.K.		Roof $M_{U,grav}$ (k-ft) = 232.00 M_{EQ} (k-ft) = 8.85 $M_{U,tot}$ (k-ft) = 240.85 Size W16X26 φM_n (k-ft) = 279 O.K.	Z_C (in^3)= 736 Z_B (in^3)= 44.2 Compact Z_{RBS} (in^3)~ 0.7Z_B= 30.94 ΣM_{PC}^* = [Z_C(F_Y-P_U/A_g)]= 35874.94 ΣM_B^*~ 2(1.1$R_Y Z_{RBS} F_Y$)*1.15= 4305.30 $\Sigma M_{PC}^*/\Sigma M_B^*$ = 8.33 O.K.
K_{STORY}= 151.64 Δ_{STORY}(in) = 0.2335 h/ 749.48 O.K.	O.K. O.K.	Penthouse $M_{U,grav}$ (k-ft) = 218.00 M_{EQ} (k-ft) = 61.10 $M_{U,tot}$ (k-ft) = 279.10 Size W18X76 φM_n (k-ft) = 576 O.K.	Z_C (in^3)= 736 Z_B (in^3)= 163 Compact Z_{RBS} (in^3)~ 0.7Z_B= 114.10 ΣM_{PC}^* = 2[Z_C(F_Y-P_U/A_g)]= 69494.61 ΣM_B^*~ 2(1.1$R_Y Z_{RBS} F_Y$)*1.15= 15877.02 $\Sigma M_{PC}^*/\Sigma M_B^*$ = 4.38 O.K.
K_{STORY}= 130.67 Δ_{STORY}(in) = 0.2226 *h/ 754.76 O.K.	O.K. O.K.	5th Level $M_{U,grav}$ (k-ft) = 206.00 M_{EQ} (k-ft) = 136.35 $M_{U,tot}$ (k-ft) = 342.35 Size W18X76 φM_n (k-ft) = 576 O.K.	Z_C (in^3)= 801 Z_B (in^3)= 163 Compact Z_{RBS} (in^3)~ 0.7Z_B= 114.10 ΣM_{PC}^* = 2[Z_C(F_Y-P_U/A_g)]= 73774.15 ΣM_B^*~ 2(1.1$R_Y Z_{RBS} F_Y$)*1.15= 15877.02 $\Sigma M_{PC}^*/\Sigma M_B^*$ = 4.65 O.K.
K_{STORY}= 84.96 Δ_{STORY}(in) = 0.1637 *h/ 1026.04 O.K.	1a O.K.	4th Level $M_{U,grav}$ (k-ft) = 145.00 M_{EQ} (k-ft) = 184.42 $M_{U,tot}$ (k-ft) = 329.42 Size W18X50 φM_n (k-ft) = 379 O.K.	Z_C (in^3)= 801 Z_B (in^3)= 101 Compact Z_{RBS} (in^3)~ 0.7Z_B= 70.70 ΣM_{PC}^* = 2[Z_C(F_Y-P_U/A_g)]= 72829.38 ΣM_B^*~ 2(1.1$R_Y Z_{RBS} F_Y$)*1.15= 9837.91 $\Sigma M_{PC}^*/\Sigma M_B^*$ = 7.40 O.K.
K_{STORY}= 70.83 Δ_{STORY}(in) = 0.1071 *h/ 1568.28 O.K.	1a 1b	3rd Level $M_{U,grav}$ (k-ft) = 145.00 M_{EQ} (k-ft) = 209.50 $M_{U,tot}$ (k-ft) = 354.50 Size W18X35 φM_n (k-ft) = 363 O.K.	Z_C (in^3)= 869 Z_B (in^3)= 66.5 Compact Z_{RBS} (in^3)~ 0.7Z_B= 46.55 ΣM_{PC}^* = 2[Z_C(F_Y-P_U/A_g)]= 78632.68 ΣM_B^*~ 2(1.1$R_Y Z_{RBS} F_Y$)*1.15= 6477.43 $\Sigma M_{PC}^*/\Sigma M_B^*$ = 12.14 O.K.
K_{STORY}= 60.05 Δ_{STORY}(in) = 0.0421 *h/ 4322.52 O.K.	1a 1b	2nd Level $M_{U,grav}$ (k-ft) = 145.00 M_{EQ} (k-ft) = 230.67 $M_{U,tot}$ (k-ft) = 375.67 Size W18X35 φM_n (k-ft) = 411 O.K.	Z_C (in^3)= 869 Z_B (in^3)= 66.5 Compact Z_{RBS} (in^3)~ 0.7Z_B= 46.55 ΣM_{PC}^* = 2[Z_C(F_Y-P_U/A_g)]= 77781.76 ΣM_B^*~ 2(1.1$R_Y Z_{RBS} F_Y$)*1.15= 6477.43 $\Sigma M_{PC}^*/\Sigma M_B^*$ = 12.01 O.K.

* Drift check incorporates Δ_{allow}=0.7%, Cd=5.5, I=1.25, and assumes 15% of the drift will be due to panel zones. Therefore, the acceptable ratio is h/740.

CA S-1 - Column U/12 Interaction Check



Chiller Mechanical Room Roof

$P_{U,grav}$ (k)= 137.00	p (k^{-1}) = 2.28E-04	pP_U = 0.03	H1-1b
$M_{EQ,X}$ (k-ft)= 17.70	b_X (k-ft^{-1}) = 3.22E-04	Int. 0.02	O.K.
$M_{EQ,Y}$ (k-ft)= 0.00	b_Y (k-ft^{-1}) = 6.41E-04		

Penthouse Level

$P_{U,grav}$ (k)= 304.00	p (k^{-1}) = 2.32E-04	pP_U = 0.07	H1-1b
$M_{EQ,X}$ (k-ft)= 104.50	b_X (k-ft^{-1}) = 3.22E-04	Int. 0.07	O.K.
$M_{EQ,Y}$ (k-ft)= 0.00	b_Y (k-ft^{-1}) = 6.41E-04		

5th Level

$P_{U,grav}$ (k)= 462.00	p (k^{-1}) = 2.12E-04	pP_U = 0.10	H1-1b
$M_{EQ,X}$ (k-ft)= 168.19	b_X (k-ft^{-1}) = 2.96E-04	Int. 0.10	O.K.
$M_{EQ,Y}$ (k-ft)= 0.00	b_Y (k-ft^{-1}) = 5.90E-04		

4th Level

$P_{U,grav}$ (k)= 531.00	p (k^{-1}) = 2.12E-04	pP_U = 0.11	H1-1b
$M_{EQ,X}$ (k-ft)= 200.65	b_X (k-ft^{-1}) = 2.96E-04	Int. 0.12	O.K.
$M_{EQ,Y}$ (k-ft)= 0.00	b_Y (k-ft^{-1}) = 5.90E-04		

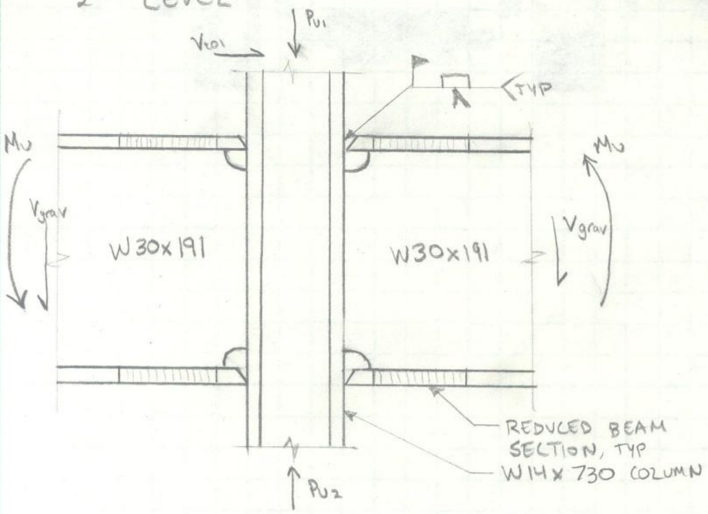
3rd Level

$P_{U,grav}$ (k)= 594.60	p (k^{-1}) = 1.98E-04	pP_U = 0.12	H1-1b
$M_{EQ,X}$ (k-ft)= 218.35	b_X (k-ft^{-1}) = 2.73E-04	Int. 0.13	O.K.
$M_{EQ,Y}$ (k-ft)= 0.00	b_Y (k-ft^{-1}) = 5.46E-04		

2nd Level

$P_{U,grav}$ (k)= 655.80	p (k^{-1}) = 2.02E-04	pP_U = 0.13	H1-1b
$M_{EQ,X}$ (k-ft)= 242.99	b_X (k-ft^{-1}) = 2.73E-04	Int. 0.14	O.K.
$M_{EQ,Y}$ (k-ft)= 0.00	b_Y (k-ft^{-1}) = 5.46E-04		

DESIGN REPRESENTATIVE CONNECTION FOR COLUMN D/2 ON 2nd LEVEL



$M_u = 431 \text{ k·ft}$

$V_{grav} = 53 \text{ K}$

$P_{u1} = 597 \text{ K}$

$P_{u2} = 688 \text{ K}$

$V_{COL} = 396 \text{ K}$

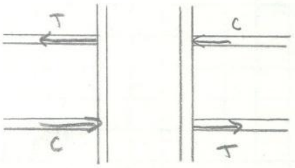
CHECK BEAM → $\phi M_p = 2530 \text{ k·ft} >> M_u$ OK ✓

$\phi V_n = 653 \text{ K} >> V_{grav}$ OK ✓

DESIGN BEAM-TO-COLUMN CONNECTION →

PER AISC 358-05 § 5.5 & 5.6, BEAM FLANGES & WEBS MUST BE WELDED TO COLUMN WITH COMPLETE JOINT PENETRATION GROOVE WELDS. THESE DEVELOP THE FULL STRENGTH OF THE BASE MATERIAL. SINCE THE BEAM IS OK UNDER APPLIED LOADS, THE CONNECTION WILL BE ALSO.

CHECK COLUMN-SIDE LIMIT STATES →



$$C = T = \frac{M_u}{d_b - t_{fb}} = \frac{431 \text{ k·ft} (12 \text{ in/ft})}{30.7 - 1.19} = 175 \text{ K}$$

LOCAL FLANGE BENDING →

$T_u \leq \phi R_n = \phi\, 6.25\, t_{fc}^2\, F_{yc} = 0.9(6.25)(4.91 \text{ in})^2(50 \text{ ksi}) = 6{,}780 \text{ K}$

OK ✓

LOCAL WEB YIELDING →

$T_u, C_u \leq \phi R_n = \phi F_{yc}(5 k_{des,c} + t_{fb}) t_{wc} = 1.0(50 \text{ ksi})[5(5.51 \text{ in}) + 1.19](3.07)$

$= 4{,}411 \text{ K}$ OK ✓

FINAL REPORT | CA S-1 BM-COL CONN. | pg 2 OF 4

LOCAL WEB CRIPPLING → $\frac{N}{d} = \frac{1.16}{30.7} = 0.04 < 0.2$

$$C_u < \phi R_n = \phi\, 0.8 t_{wc}\left[1 + 3\left(\frac{t_{fb}}{d}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_{fc}}{t_{wc}}}$$

$$\phi R_n = 0.75(0.8)(3.07)\left[1+3\left(\frac{1.16}{30.7}\right)\left(\frac{3.07}{4.91}\right)^{1.5}\right]\sqrt{\frac{24000\text{ ksi}(50\text{ ksi})(4.91)}{3.07}}$$

$$= 2{,}962\text{ K} > C_u \quad \underline{\text{OK}\checkmark}$$

WEB BUCKLING →
NOT APPLICABLE FOR LATERAL LOAD CASES

PANEL ZONE SHEAR YIELDING →

$$F_u = 2(175) - 396 = -46\text{ K}$$

$$P_r = \frac{P_{u1} + P_{u2}}{2} = \frac{597\text{ K} + 688\text{ K}}{2} = 643\text{ K}$$

$$P_c = A_g F_y = 21.5\text{ in}^2(50\text{ ksi}) = 10{,}750\text{ K}$$

$P_r < 0.75 P_c = 8{,}063$ K

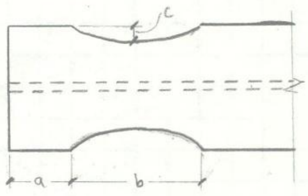
$$F_u < \phi R_n = \phi\, 0.6 F_y d_c t_{wc}\left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_{wc}}\right)$$

$$\phi R_n = 0.9(0.6)(50\text{ ksi})(22.4\text{in})(3.07\text{in})\left(1 + \frac{3(17.9\text{ in})(4.91\text{ in})^2}{30.7\text{in}(22.4\text{in})(3.07\text{in})}\right)$$

$$= 2{,}995\text{ K} > F_u$$

<u>NO STIFFENERS REQUIRED FOR STRENGTH DESIGN</u>

REDUCED BEAM SECTION DESIGN →
GOVERNED BY AISC 358-05 AND AISC 341-05



TRIAL RBS DIMENSIONS

$a \approx 0.5 b_f = 0.5(15'') = 7.75''$

$b \approx 0.65 d = 0.65(30.7'') = 20''$

$c \approx 0.2 b_f = 0.2(15'') = 3''$

$S_h = a + b/2 = 7.75 + 20/2 = 17.75''$

$$Z_{RBS} = Z_x - 2c t_{bf}(d - t_{bf}) = 675 - 2(3'')(1.19\text{in})(30.7 - 1.19) = 464\text{ in}^3$$

$$M_{pr} = C_{pr} R_y F_y Z_{RBS} = 1.15(1.1)(50\text{ ksi})(464\text{ in}^3)(1\text{ft}/12\text{ in}) = 2{,}446\text{ k}\cdot\text{ft}$$

$$V_{RBS} = V_{grav} + \frac{2M_{pe}}{L'} = 53\text{ K} + \frac{2(675\text{ in}^3)(50\text{ ksi})(1.1)(1\text{ft}/12\text{in})}{21\text{ ft}} = 348\text{ K}$$

$$M_{pb}^* = M_{pr} + M_v = M_{pr} + V_{RBS}(S_h + d_c/2) = 2446 + 348(17.75 + 22.4/2)(1\text{ft}/12\text{in})$$
$$= 3{,}286\text{ k-ft}$$

$$\Sigma M_{pb}^* = 2(3286) = 6{,}572\text{ k}\cdot\text{ft}$$

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$M^*_{pc} = Z_c(F_{yc} - P_u/A_g) = 1660\text{ in}^3\ (50\text{ ksi} - 643^k/215\text{ in}^2)(1^{ft}/12\text{ in}) = 6503\text{ k-ft}$

$\Sigma M^*_{pc} = 2(6503) = 13{,}006\text{ k}\cdot\text{ft}$

$\dfrac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} = 1.98 > 1.0$ <u>OK ✓</u>

$M_{pe} = Z_b R_y F_y = 675\text{ in}^3(50\text{ ksi})(1.1)(^{1ft}/_{12\text{ in}}) = 3{,}094\text{ k}\cdot\text{ft}$

$M_f = M_{pr} + V_{RBS}\,S_h = 2446 + 348(17.75'')(^{1ft}/_{12\text{ in}}) = 2{,}961\text{ k}\cdot\text{ft}$

$M_f < \underbrace{\phi_d}_{=1.0} M_{pe}$ <u>OK ✓</u>

$V_u = \dfrac{2M_{pr}}{L'} + V_{grav} = \dfrac{2(2446)}{16'} + 53\text{ k} = 359\text{ k} < \phi V_n$ OK ✓

$\phi M_{p,RBS} = 0.9(464\text{ in}^3)(50\text{ ksi})(^{1ft}/_{12\text{ in}}) = 1{,}740\text{ k}\cdot\text{ft} > M_u$ OK ✓

CHECK GEOMETRY/WEIGHT LIMITATIONS →

PER §5.3.1 IN AISC 358-05, BEAM LIMITATIONS ARE:

- ↳ MAX SIZE = W36 > W30 <u>OK ✓</u>
- ↳ MAX WT. = 300 plf > 191 plf <u>OK ✓</u>
- ↳ $t_{f,max} = 1.75'' > 1.19''$ <u>OK ✓</u>
- ↳ $\left(\dfrac{\text{span}}{d}\right)_{min} = 7 < \dfrac{21\times 12 - 22.4\text{ in}}{30.7\text{ in}} = 7.48$ <u>OK ✓</u>
- ↳ $b_f/2t_f = 6.35 < 0.3\sqrt{E/F_y} = 7.22$ <u>OK ✓</u>
- ↳ $h/t_w = 37.7 < 2.45\sqrt{E/F_y} = 59$ <u>OK ✓</u>

PER §5.3.2 IN AISC 358-05, COLUMN LIMITATIONS ARE:

- ↳ MAX SIZE = W36 > W14' <u>OK ✓</u>
- ↳ $b_f/2t_f = 1.82 < 7.22$ <u>OK ✓</u>
- ↳ $h/t_w = 3.71 < 59$ <u>OK ✓</u>

CHECK CONTINUITY PLATE REQUIREMENTS →

PER §2.4.4 IN AISC 358-05, CONTINUITY PLATES NEED NOT BE PROVIDED IF

$t_{cf} = 4.91\text{ in} > 0.4\sqrt{1.8\,b_{bf}\,t_{bf}\,\dfrac{F_{yb}R_{yb}}{\cancel{F_{yc}R_{yc}}}\rightarrow 1} = 0.4\sqrt{1.8(15'')(1.18'')} = 1.51\text{ in}$ <u>OK ✓</u>

-AND-

$t_{cf} \geq \dfrac{b_{bf}}{6} = \dfrac{15}{6} = 2.5\text{ in}$ OK ✓

<u>CONTINUITY PLATES NOT REQUIRED</u>

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REDESIGN PANEL ZONE FOR M_f (PER AISC 341-05 §9.3a) →

$$T = C = \frac{M_f}{d_b - t_{fb}} = \frac{2961 \text{ k·ft } (12 \text{ in/ft})}{30.7 - 1.19} = 1{,}204 \text{ K}$$

$$F_u = 2(1204) - 396 = 2012 \text{ K} >$$

$$\phi R_n = \frac{2995}{\underbrace{0.9}_{}} = 3{,}328 \text{ K} > F_u$$

$\phi = 1.0$ FOR SEISMIC PER AISC 341-05, § 9.3a, pg 6.1-31

<u>NO WEB PLATE REQUIRED</u>

CHECK COLUMN WEB THICKNESS (PER AISC 341-05, §9.3b)

$$t > \frac{d_z + w_z}{90} = \frac{(30.7'' - 2(1.19'')) + \overset{\swarrow h/t_w}{(3.71)}(3.07'')}{90} = 0.441$$

$$t_{cw} = 3.07'' > 0.441 \quad \underline{\text{OK} \checkmark}$$

FINAL REPORT | CA S-1 BM. BRACING | pg 1 OF 2

PER AISC 341-05 §9.8, BOTH FLANGES OF BEAMS MUST BE BRACED @

$$L_b = \frac{0.086\, r_y E}{F_y} = \frac{0.086\,(3.46\,in)(29000\,ksi)}{50\,ksi} = 173\ in$$
$$= 14.4\ ft$$

FOR A STRENGTH OF

$$P_{br} = \frac{0.02\, R_y Z_x F_y}{h_o} = \frac{0.02(1.1)(675\,in^3)(50\,ksi)}{30.7 - 1.14} = 25.2\ K$$

AND A STIFFNESS OF

$$\beta_{br} = \frac{1}{\phi}\left(\frac{10\, R_y Z_x F_y}{L_b h_o}\right) = \frac{1}{0.75}\left(\frac{10(1.1)(675\,in^3)(50\,ksi)}{126\,in\,(30.7'' - 1.19'')}\right) = 133.1\ K/in$$

L_b ← BRACE @ MIDLENGTH $= 10.5' = 126'' < 173$ OK ✓

TRY $L5 \times 5 \times 5/16''$

LENGTH → SPAN TO NEAREST BEAM → $L_x = 12'\text{-}1''$, $L_y = 30.7''$

$$L = \sqrt{145^2 + 30.7^2} = 148\ in = 12.4\ ft$$

FROM TABLE 4-11 IN AISC SPEC., @ KL = 13 ft

$\phi P_n = 27.4\ K > P_{br}$ OK ✓

$$\text{STIFFNESS} = \frac{AE}{L} = \frac{3.03\,in^2\,(29000\,ksi)}{148\,in} = 594\ K/in > \beta_{br}\ \text{OK ✓}$$

USE (1) $L5 \times 5 \times 5/16''$ @ MIDSPAN

ALSO, BRACES MUST BE PROVIDED @ RBS W/ STRENGTH OF

$$P_{br} = \frac{0.06\, R_y Z_{RBS} F_y}{h_o} = \frac{0.06\,(1.1)(464\,in^3)(50\,ksi)}{(30.7 - 1.19\,in)} = 51.89\ K$$

AND A STIFFNESS OF

$$\beta_{br} = \frac{1}{\phi}\left(\frac{10\, R_y Z_{RBS} F_y}{L_b h_o}\right) = \frac{1}{0.75}\left(\frac{10\,(1.1)(464\,in^3)(50\,ksi)}{126\,in\,(30.7 - 1.19)}\right) = 91.51\ K/in$$

TRY $L5 \times 5 \times 5/8''$

FROM TABLE 4-11 IN AISC SPEC. @ KL = 12.4 ft

$\phi P_n = 57.1\ K > P_{br}$ OK ✓

$$\text{STIFFNESS} = \frac{5.86\,in^2\ (29000\,ksi)}{148\,in} = 1148\ K/in > \beta_{br}\ \text{OK ✓}$$

FINAL REPORT | CA S-1 BM. BRACING | pg 2 OF 2

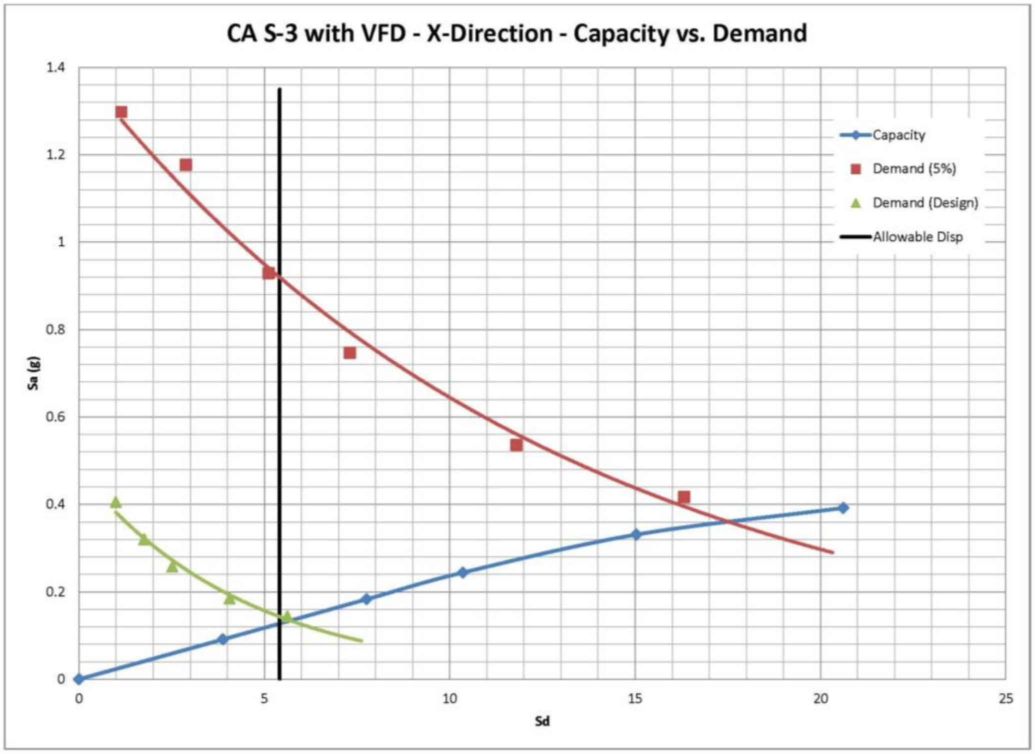
USE (1) L5x5x5/8" @ EACH RBS

FOR CONSTRUCTABILITY, THE SAME BRACES SHOULD BE USED AT RBS & MIDSPAN

* NOTE! THIS IS THE SAME DESIGN AS THE BRACE FOR THE CA S-3 SYSTEM. SEE THESE CALCULATIONS FOR ASSOCIATED SKETCHES AND THE DESIGN OF THE BOLTS/PLATE/WELD TO ATTACH THE BRACE TO THE LATERAL & SUPPORT BEAMS.

Appendix F: Viscous Fluid Damper Design

X-Direction Preliminary Sizing



CA S-3 with VFD - Y-Direction - General/Trial Properties			Check Trial vs. Calculated
Trial Damping Percentage (β)	70		
B (from ASCE 7-05 Table 18.6-1)	3.0		
C_d	5.5		
R	8		
I	1.25		
Ω_0	3		
S_{MS}	2.000		
S_{M1}	0.945		
S_{DS}	1.33		
S_{D1}	0.63		
Trial D_{1D}	6.500		O.K.
Trial D_{1M}	9.750		O.K.
Trial $T_{1D}=1.1T_1$	2.591		
Trial $T_{1M}=3.0T_1$	7.066		
D_Y	1.004		
$\mu_{D,calc}$	6.472		
$\mu_{max(T1D \le Ts)}$	2.776	Do Not Use	
$\mu_{max(T1 \ge Ts)}$	1.164	Use	
μ_D	1.164		
μ_M	9.709		
T_{1D}	2.541		O.K.
T_{1M}	7.339		O.K.
T_s	0.4725		
T_0	0.0945		
q_H	0.5		

CA S-3 with VFD - X-Direction - Modal Properties per Story

Level	Weight/ Story (w_i)	Mode 1			Mode 2			Mode 3		
		ϕ_{i1}	$w_i\phi_{i1}$	$w_i\phi_{i1}^2$	ϕ_{i2}	$w_i\phi_{i2}$	$w_i\phi_{i2}^2$	ϕ_{i3}	$w_i\phi_{i3}$	$w_i\phi_{i3}^2$
AHU Roof	818.26	1.000	818.257	818.257	1.000	818.257	818.257	1.000	818.257	818.257
Chiller Roof	354.13	0.941	333.222	313.546	0.966	341.994	330.272	1.024	362.638	371.348
Atrium Roof	569.48	0.393	223.541	87.748	0.651	370.850	241.501	3.123	1778.273	5552.887
Penthouse	2281.28	0.899	2051.752	1845.316	0.859	1959.326	1682.808	0.695	1586.225	1102.938
5th	2590.94	0.548	1419.691	777.913	0.591	1530.533	904.125	1.441	3734.093	5381.626
4th	1858.99	0.430	799.094	343.493	0.450	835.796	375.771	1.046	1943.597	2032.054
3rd	1867.80	0.267	499.374	133.512	0.288	538.499	155.253	0.649	1211.354	785.619
2nd	1884.64	0.109	204.755	22.246	0.109	204.755	22.246	0.272	512.260	139.236
Totals			6,349.686	4,342.032		6,600.011	4,530.234		11,946.698	16,183.967
Level	Weight/ Story (w_i)	Mode 4			Mode 5			Mode 6		
		ϕ_{i4}	$w_i\phi_{i4}$	$w_i\phi_{i4}^2$	ϕ_{i5}	$w_i\phi_{i5}$	$w_i\phi_{i5}^2$	ϕ_{i6}	$w_i\phi_{i6}$	$w_i\phi_{i6}^2$
AHU Roof	818.26	1.000	818.257	818.257	1.000	818.257	818.257	1.000	818.257	818.257
Chiller Roof	354.13	-3.376	-1195.394	4035.131	0.685	242.628	166.234	0.413	146.411	60.532
Atrium Roof	569.48	1.118	636.477	711.356	0.154	87.635	13.486	0.195	110.855	21.579
Penthouse	2281.28	-1.018	-2322.571	2364.608	0.171	391.196	67.083	0.113	257.405	29.044
5th	2590.94	0.656	1699.936	1115.342	-0.195	-506.212	98.903	-0.139	-360.410	50.135
4th	1858.99	0.665	1236.523	822.484	-0.341	-634.146	216.323	-0.281	-522.791	147.021
3rd	1867.80	0.516	963.480	496.999	-0.314	-587.121	184.555	-0.270	-503.550	135.755
2nd	1884.64	0.253	477.556	121.010	-0.157	-294.970	46.166	-0.133	-249.987	33.159
Totals			2,314.265	10,485.188		-482.732	1,611.006		-303.809	1,295.481
Level	Weight/ Story (w_i)	Mode 7			Mode 8			Mode 9		
		ϕ_{i7}	$w_i\phi_{i7}$	$w_i\phi_{i7}^2$	ϕ_{i8}	$w_i\phi_{i8}$	$w_i\phi_{i8}^2$	ϕ_{i9}	$w_i\phi_{i9}$	$w_i\phi_{i9}^2$
AHU Roof	818.26	1.000	818.257	818.257	1.000	818.257	818.257	1.000	818.257	818.257
Chiller Roof	354.13	0.770	272.660	209.932	-0.392	-138.718	54.338	-0.304	-107.541	32.657
Atrium Roof	569.48	-4.767	-2714.881	12942.661	-0.405	-230.580	93.361	-0.150	-85.550	12.852
Penthouse	2281.28	0.119	272.190	32.476	-0.307	-700.280	214.964	-0.298	-679.530	202.413
5th	2590.94	-0.040	-102.476	4.053	-0.046	-119.544	5.516	-0.132	-342.451	45.263
4th	1858.99	0.334	621.298	207.645	0.140	259.944	36.348	0.131	243.311	31.845
3rd	1867.80	0.406	758.447	307.979	0.218	408.032	89.137	0.284	531.076	151.002
2nd	1884.64	0.224	422.398	94.671	0.128	241.347	30.907	0.177	334.156	59.248
Totals			347.893	14,617.674		538.457	1,342.827		711.728	1,353.538

CA S-3 with VFD - X-Direction - Modal Properties per Mode

Property		Mode									SRSS
		1	2	3	4	5	6	7	8	9	
Modal Properties	T_m (s)	2.355	2.198	1.966	0.847	0.775	0.755	0.607	0.550	0.495	1.327
	$PF\%_m$	51.15%	1.01%	28.13%	0.08%	7.21%	1.06%	1.17%	0.51%	0.39%	
	$\overline{W}_m$ (k)	9285.632	9615.429	8818.826	510.799	144.649	71.248	8.280	215.915	374.246	5359.783
	Γ_m	1.462	1.457	0.738	0.221	-0.300	-0.235	0.024	0.401	0.526	
	C_{Sm}	0.039	0.046	0.052	0.120	0.131	0.135	0.168	0.185	0.206	0.027
	V_m	364.912	445.455	456.834	61.438	19.007	9.605	1.388	39.947	76.937	226.683
Damped Modal Story Forces (k)	F_{AHUm}	47.025	55.227	31.290	21.723	-32.217	-25.871	3.264	60.704	88.453	25.732
	F_{CHLRm}	19.150	23.082	13.867	-31.735	-9.553	-4.629	1.088	-10.291	-11.625	10.570
	F_{ATRm}	12.847	25.030	68.000	16.897	-3.450	-3.505	-10.829	-17.106	-9.248	20.232
	F_{PENTm}	117.913	132.241	60.656	-61.658	-15.403	-8.138	1.086	-51.952	-73.457	62.710
	F_{5m}	81.589	103.300	142.789	45.129	19.931	11.395	-0.409	-8.869	-37.019	57.956
	F_{4m}	45.923	56.410	74.322	32.826	24.968	16.529	2.478	19.285	26.302	31.507
	F_{3m}	28.699	36.345	46.321	25.578	23.117	15.921	3.025	30.271	57.409	19.707
	F_{2m}	11.767	13.820	19.588	12.678	11.614	7.904	1.685	17.905	36.122	8.207
Damped Modal Story Disp. (in)	D_{mD}	7.788	6.581	2.982	0.384	-0.783	-0.582	0.030	0.454	0.535	4.072
	δ_{CHLRm}	7.328	6.356	3.054	-1.296	-0.536	-0.241	0.023	-0.178	-0.163	3.846
	δ_{ATRm}	3.057	4.286	9.312	0.429	-0.120	-0.113	-0.142	-0.184	-0.080	3.051
	δ_{PENTm}	7.004	5.652	2.074	-0.391	-0.134	-0.066	0.004	-0.139	-0.159	3.630
	δ_{5m}	4.267	3.888	4.298	0.252	0.153	0.081	-0.001	-0.021	-0.071	2.496
	δ_{4m}	3.348	2.959	3.118	0.255	0.267	0.164	0.010	0.063	0.070	1.924
	δ_{3m}	2.082	1.897	1.934	0.198	0.246	0.157	0.012	0.099	0.152	1.196
	δ_{2m}	0.846	0.715	0.811	0.097	0.122	0.077	0.007	0.058	0.095	0.489
Damping Properties	Δ_{mD} (in)	2.023	0.993	-0.315	7.394	-1.084	-1.502	0.030	2.778	3.071	1.042
	∇_{mD} (in/s)	4.907	2.837	-1.008	54.878	-8.792	-12.494	0.311	31.711	38.953	2.618
	D_{mM}	19.115	9.872	4.473	0.576	-1.174	-0.873	0.045	0.680	0.803	9.860
	W_m (k)	971.636	1006.145	922.790	53.449	24.822	11.916	0.866	22.593	39.161	560.842
	β_I	0.05									
	β_{HD}	0.041	0	0	0	0	0	0	0	0	
	β_{HM}	0.197	0	0	0	0	0	0	0	0	
	$\beta_{Vm,req}$	0.564	0.564	0.603	0.603	0.603	0.603	0.603	0.603	0.603	
	$\Sigma W_{m,j}$	6,887.735	7,132.366	6,987.429	404.722	187.955	90.231	6.560	171.076	296.527	4,035.31
Damping Force Required (k)	$\Sigma F_{AHUm,j}$	884.45	1,083.73	2,343.04	1,053.86	-240.15	-155.07	220.75	377.17	553.96	799.79
	$\Sigma F_{CHLRm,j}$	939.95	1,122.20	2,288.08	-312.20	-350.51	-375.08	286.72	-962.87	-1,824.18	804.01
	$\Sigma F_{ATRm,j}$	2,253.16	1,664.18	750.34	942.93	-1,560.56	-796.63	-46.31	-931.52	-3,687.50	1,177.39
	$\Sigma F_{PENTm,j}$	983.39	1,261.81	3,369.72	-1,035.12	-1,400.44	-1,374.34	1,850.19	-1,228.69	-1,859.71	1,078.40
	$\Sigma F_{5m,j}$	1,614.12	1,834.58	1,625.74	1,606.22	1,229.15	1,114.79	-5,581.42	-8,174.54	-4,191.16	951.62
	$\Sigma F_{4m,j}$	2,057.56	2,410.46	2,241.05	1,584.37	703.99	551.42	660.52	2,697.32	4,232.45	1,228.43
	$\Sigma F_{3m,j}$	3,308.09	3,758.96	3,612.76	2,043.01	763.98	575.20	543.64	1,726.52	1,948.27	1,975.18
	$\Sigma F_{2m,j}$	8,140.77	9,975.06	8,620.21	4,158.98	1,534.38	1,169.08	984.96	2,945.24	3,124.32	4,821.42

CA S-3 with VFD - X-Direction - Damper Forces, 2nd Level

Total Required Damping Force per Story = ΣF_2= 4,821.42 k

Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_2/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
1	D to E	2	0	2.000	453.16	453.16
1	F to G	2	0	2.000		453.16
5	11 to 12	2	-15	1.932		469.14
10	9 to 10	2	-45	1.414		640.86
11	M to N	2	45	1.414		640.86
15	E1 to F1	2	-20	1.879		482.24

CA S-3 with VFD - X-Direction - Damper Forces, 3rd Level

Total Required Damping Force per Story = ΣF_3= 1,975.18 k

Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_3/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
1	D to E	2	0	2.000	185.64	185.64
1	F to G	2	0	2.000		185.64
5	11 to 12	2	-15	1.932		192.19
10	9 to 10	2	-45	1.414		262.54
11	M to N	2	45	1.414		262.54
15	E1 to F1	2	-20	1.879		197.56

CA S-3 with VFD - X-Direction - Damper Forces, 4th Level

Total Required Damping Force per Story = ΣF_4= 1,228.43 k

Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_4/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
1	D to E	2	0	2.000	115.46	115.46
1	F to G	2	0	2.000		115.46
5	11 to 12	2	-15	1.932		119.53
10	9 to 10	2	-45	1.414		163.28
11	M to N	2	45	1.414		163.28
15	E1 to F1	2	-20	1.879		122.87

CA S-3 with VFD - X-Direction - Damper Forces, 5th Level

Total Required Damping Force per Story = ΣF_5= 951.62 k

Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_5/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
1	D to E	2	0	2.000	89.44	89.44
1	F to G	2	0	2.000		89.44
5	11 to 12	2	-15	1.932		92.60
10	9 to 10	2	-45	1.414		126.49
11	M to N	2	45	1.414		126.49
15	E1 to F1	2	-20	1.879		95.18

CA S-3 with VFD - X-Direction - Damper Forces, Penthouse Level

Total Required Damping Force per Story = ΣF_{PENT}= 1,078.40 k

Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_{PENT}/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
1	D to E	2	0	2.000	101.36	101.36
1	F to G	2	0	2.000		101.36
5	11 to 12	2	-15	1.932		104.93
10	9 to 10	2	-45	1.414		143.34
11	M to N	2	45	1.414		143.34
15	E1 to F1	2	-20	1.879		107.86

CA S-3 with VFD - X-Direction - Damper Forces, Atrium Roof Level

Total Required Damping Force per Story = ΣF_{ATR}= 1,177.39 k

Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_{ATR}/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
1	D to E	2	0	2.000	110.66	110.66
1	F to G	2	0	2.000		110.66
5	11 to 12	2	-15	1.932		114.56
10	9 to 10	2	-45	1.414		156.50
11	M to N	2	45	1.414		156.50
15	E1 to F1	2	-20	1.879		117.76

CA S-3 with VFD - X-Direction - Damper Forces, Chiller Roof Level

Total Required Damping Force per Story = ΣF_{CHLR}= 804.01 k

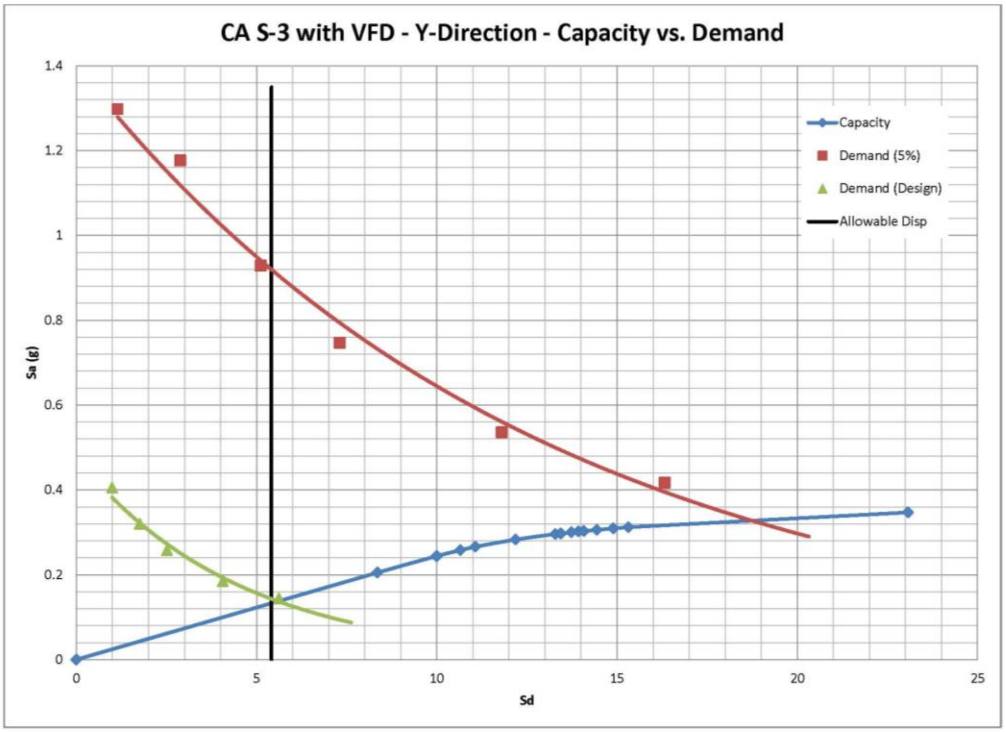
Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_{CHLR}/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
1	D to E	2	0	2.000	135.54	135.54
1	F to G	2	0	2.000		135.54
5	11 to 12	2	-15	1.932		140.32

CA S-3 with VFD - X-Direction - Damper Forces, AHU Roof Level

Total Required Damping Force per Story = ΣF_{AHU}= 799.79 k

Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_{AHU}/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
1	D to E	2	0	2.000	199.95	199.95
1	F to G	2	0	2.000		199.95

Y-Direction Preliminary Sizing



CA S-3 with VFD - Y-Direction - General/Trial Properties		Check Trial vs. Calculated
Trial Damping Percentage (β)	70	
B (from ASCE 7-05 Table 18.6-1)	3.0	
C_d	5.5	
R	8	
I	1.25	
Ω_0	3	
S_{MS}	2.000	
S_{M1}	0.945	
S_{DS}	1.33	
S_{D1}	0.63	
Trial D_{1D}	6.500	O.K.
Trial D_{1M}	9.750	O.K.
Trial $T_{1D}=1.1T_1$	2.591	
Trial $T_{1M}=3.0T_1$	7.066	
D_Y	1.004	
$\mu_{D,calc}$	6.472	
$\mu_{max(TiD \le Ts)}$	2.776	Do Not Use
$\mu_{max(T1 \ge Ts)}$	1.164	Use
μ_D	1.164	
μ_M	9.709	
T_{1D}	2.541	O.K.
T_{1M}	7.339	O.K.
T_s	0.4725	
T_0	0.0945	
q_H	0.5	

CA S-3 with VFD - Y-Direction - Modal Properties per Story

Level	Weight/ Story (w_i)	Mode 1			Mode 2			Mode 3		
		ϕ_{i1}	$w_i\phi_{i1}$	$w_i\phi_{i1}^2$	ϕ_{i2}	$w_i\phi_{i2}$	$w_i\phi_{i2}^2$	ϕ_{i3}	$w_i\phi_{i3}$	$w_i\phi_{i3}^2$
AHU Roof	818.26	1.000	818.257	818.257	1.000	818.257	818.257	1.000	818.257	818.257
Chiller Roof	354.13	13.4245	4754.024	63820.211	0.8307	294.160	244.345	-3.3957	-1202.538	4083.504
Atrium Roof	569.48	3.9424	2245.141	8851.348	0.8450	481.236	406.667	-0.1811	-103.148	18.683
Penthouse	2281.28	2.5827	5891.940	15217.313	0.8036	1833.137	1473.029	0.2207	503.479	111.118
5th	2590.94	2.0072	5200.513	10438.440	0.6681	1730.947	1156.408	0.2588	670.410	173.470
4th	1858.99	1.4676	2728.303	4004.129	0.5080	944.449	479.822	0.2237	415.939	93.064
3rd	1867.80	0.8777	1639.364	1438.866	0.3171	592.287	187.817	0.1416	264.392	37.425
2nd	1884.64	0.3381	637.252	215.474	0.1274	240.168	30.606	0.0548	103.268	5.659
Totals			23,914.795	104,804.038		6,934.643	4,796.950		1,470.059	5,341.179
Level	Weight/ Story (w_i)	Mode 4			Mode 5			Mode 6		
		ϕ_{i4}	$w_i\phi_{i4}$	$w_i\phi_{i4}^2$	ϕ_{i5}	$w_i\phi_{i5}$	$w_i\phi_{i5}^2$	ϕ_{i6}	$w_i\phi_{i6}$	$w_i\phi_{i6}^2$
AHU Roof	818.26	1.000	818.257	818.257	1.000	818.257	818.257	1.000	818.257	818.257
Chiller Roof	354.13	-0.7414	-262.560	194.667	1.1403	403.804	460.445	0.9022	319.481	288.222
Atrium Roof	569.48	0.1585	90.290	14.315	0.1915	109.036	20.877	0.2312	131.673	30.445
Penthouse	2281.28	0.0389	88.745	3.452	0.1259	287.137	36.141	0.1273	290.309	36.944
5th	2590.94	-0.1030	-266.801	27.474	-0.2299	-595.570	136.902	-0.2085	-540.219	112.637
4th	1858.99	-0.1458	-271.040	39.517	-0.3611	-671.220	242.355	-0.3413	-634.461	216.537
3rd	1867.80	-0.1239	-231.414	28.671	-0.3200	-597.696	191.263	-0.3078	-574.917	176.962
2nd	1884.64	-0.0585	-110.281	6.453	-0.1563	-294.506	46.022	-0.1488	-280.446	41.732
Totals			-144.803	1,132.808		-540.757	1,952.261		-470.321	1,721.736
Level	Weight/ Story (w_i)	Mode 7			Mode 8			Mode 9		
		ϕ_{i7}	$w_i\phi_{i7}$	$w_i\phi_{i7}^2$	ϕ_{i8}	$w_i\phi_{i8}$	$w_i\phi_{i8}^2$	ϕ_{i9}	$w_i\phi_{i9}$	$w_i\phi_{i9}^2$
AHU Roof	818.26	1.000	818.257	818.257	1.000	818.257	818.257	1.000	818.257	818.257
Chiller Roof	354.13	-17.0000	-6020.236	102344.004	7.7650	2749.823	21352.312	0.3700	131.028	48.480
Atrium Roof	569.48	16.5102	9402.219	155232.549	-5.1244	-2918.253	14954.366	-1.4614	-832.233	1216.219
Penthouse	2281.28	-2.3673	-5400.582	12785.052	-2.2581	-5151.279	11631.919	-0.2818	-642.756	181.098
5th	2590.94	-0.1020	-264.381	26.978	-0.3226	-835.786	269.608	-0.1273	-329.889	42.003
4th	1858.99	0.9592	1783.114	1710.334	1.5161	2818.470	4273.165	0.2285	424.762	97.054
3rd	1867.80	1.3878	2592.048	3597.128	2.1659	4045.464	8762.066	0.3842	717.569	275.675
2nd	1884.64	0.7755	1461.557	1133.452	1.2673	2388.368	3026.734	0.2310	435.374	100.576
Totals			4,371.996	277,647.754		3,915.065	65,088.427		722.111	2,779.363

CA S-3 with VFD - Y-Direction - Modal Properties per Mode											
	Property	Mode									SRSS
		1	2	3	4	5	6	7	8	9	
Modal Properties	T_m (s)	2.355	2.198	1.966	0.847	0.775	0.755	0.607	0.550	0.495	1.723
	PF%$_m$	1.32%	78.35%	0.02%	1.24%	1.33%	5.99%	0.02%	0.55%	3.32%	
	$\overline{W}_m$ (k)	5457.017	10024.968	404.606	18.510	149.784	128.476	68.844	235.491	187.613	7854.778
	Γ_m	0.228	1.446	0.275	-0.128	-0.277	-0.273	0.016	0.060	0.260	
	C_{Sm}	0.039	0.046	0.052	0.120	0.131	0.135	0.168	0.185	0.206	0.038
	V_m	214.453	464.427	20.959	2.226	19.681	17.321	11.538	43.569	38.569	363.888
Damped Modal Story Forces (k)	F_{AHUm}	7.338	54.800	11.666	-12.580	-29.781	-30.135	2.160	9.106	43.705	43.000
	F_{CHLRm}	42.631	19.701	-17.145	4.037	-14.697	-11.766	-15.888	30.601	6.998	15.465
	F_{ATRm}	20.133	32.229	-1.471	-1.388	-3.968	-4.849	24.814	-32.476	-44.451	25.298
	F_{PENTm}	52.835	122.769	7.178	-1.364	-10.451	-10.691	-14.253	-57.326	-34.331	96.200
	F_{5m}	46.635	115.925	9.558	4.102	21.676	19.895	-0.698	-9.301	-17.620	90.838
	F_{4m}	24.466	63.252	5.930	4.167	24.430	23.366	4.706	31.365	22.687	49.585
	F_{3m}	14.701	39.667	3.770	3.558	21.754	21.173	6.841	45.020	38.327	31.133
	F_{2m}	5.714	16.085	1.472	1.696	10.719	10.328	3.857	26.579	23.254	12.643
Damped Modal Story Disp. (in)	D_{mD}	1.215	6.530	1.112	-0.398	-0.723	-0.678	0.020	0.068	0.264	5.117
	δ_{CHLRm}	16.313	5.425	-3.776	0.295	-0.825	-0.611	-0.334	0.528	0.098	4.256
	δ_{ATRm}	4.791	5.519	-0.201	-0.063	-0.139	-0.157	0.325	-0.349	-0.387	4.324
	δ_{PENTm}	3.138	5.248	0.245	-0.016	-0.091	-0.086	-0.047	-0.154	-0.075	4.112
	δ_{5m}	2.439	4.363	0.288	0.041	0.166	0.141	-0.002	-0.022	-0.034	3.418
	δ_{4m}	1.783	3.318	0.249	0.058	0.261	0.231	0.019	0.103	0.060	2.600
	δ_{3m}	1.067	2.071	0.157	0.049	0.232	0.209	0.027	0.147	0.102	1.623
	δ_{2m}	0.411	0.832	0.061	0.023	0.113	0.101	0.015	0.086	0.061	0.652
Damping Properties	Δ_{mD} (in)	-66.430	4.866	21.506	-3.053	0.447	-0.292	1.557	-2.025	0.733	3.912
	∇_{mD} (in/s)	-161.100	13.911	68.748	-22.662	3.621	-2.428	16.106	-23.122	9.301	11.114
	D_{mM}	4.971	9.796	1.668	-0.598	-1.085	-1.017	0.029	0.102	0.397	7.675
	W_m (k)	571.015	1048.999	42.337	3.470	25.703	21.488	7.204	24.641	19.632	821.914
	β_I	0.05									
	β_{HD}	0.000	0.041	0	0	0	0	0	0	0	
	β_{HM}	0.000	0.265	0	0	0	0	0	0	0	
	$\beta_{Vm,req}$	0.603	0.564	0.603	0.603	0.603	0.603	0.603	0.603	0.603	
	$\Sigma W_{m,j}$	4,323.763	7,436.147	320.582	26.275	194.628	162.706	54.547	186.587	148.652	5,826.42
Damping Force Required (k)	$\Sigma F_{AHUm,j}$	3,558.18	1,138.68	288.31	-65.94	-269.02	-240.06	2,774.24	2,742.35	562.04	893.82
	$\Sigma F_{CHLRm,j}$	265.05	1,370.83	-84.90	88.94	-235.92	-266.10	-163.19	353.17	1,519.04	1,075.34
	$\Sigma F_{ATRm,j}$	902.53	1,347.48	-1,591.79	-415.90	-1,405.02	-1,038.26	168.03	-535.15	-384.59	1,057.89
	$\Sigma F_{PENTm,j}$	1,377.68	1,417.05	1,306.36	-1,695.04	-2,137.30	-1,886.45	-1,171.88	-1,214.47	-1,994.80	1,118.66
	$\Sigma F_{5m,j}$	1,772.72	1,704.41	1,114.25	640.35	1,170.31	1,151.37	-27,187.56	-8,501.28	-4,414.24	1,346.30
	$\Sigma F_{4m,j}$	2,424.45	2,241.30	1,288.59	452.26	745.06	703.39	2,892.29	1,808.78	2,459.79	1,758.79
	$\Sigma F_{3m,j}$	4,054.00	3,590.87	2,036.80	532.22	840.67	779.92	1,999.09	1,266.15	1,462.96	2,814.76
	$\Sigma F_{2m,j}$	10,523.14	8,935.43	5,261.74	1,126.87	1,721.51	1,613.26	3,577.31	2,163.96	2,432.95	7,003.37

CA S-3 with VFD - Y-Direction - Damper Forces, 2nd Level

Total Required Damping Force per Story = ΣF_2= 7,003.37 k

Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_2/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
3	2 to 3	2	0	2.000	658.23	658.23
4	2 to 3	2	0	2.000		658.23
7	V to W	2	15	1.932		681.45
10	9 to 10	2	-45	1.414		930.88
11	M to N	2	45	1.414		930.88
14	5 to 6	2	20	1.879		700.48

CA S-3 with VFD - Y-Direction - Damper Forces, 3rd Level

Total Required Damping Force per Story = ΣF_3= 2,814.76 k

Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_3/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
3	2 to 3	2	0	2.000	264.55	264.55
4	2 to 3	2	0	2.000		264.55
7	V to W	2	15	1.932		273.89
10	9 to 10	2	-45	1.414		374.13
11	M to N	2	45	1.414		374.13
14	5 to 6	2	20	1.879		281.53

CA S-3 with VFD - Y-Direction - Damper Forces, 4th Level

Total Required Damping Force per Story = ΣF_4= 1,758.79 k

Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_4/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
3	2 to 3	2	0	2.000	165.30	165.30
4	2 to 3	2	0	2.000		165.30
7	V to W	2	15	1.932		171.14
10	9 to 10	2	-45	1.414		233.78
11	M to N	2	45	1.414		233.78
14	5 to 6	2	20	1.879		175.91

CA S-3 with VFD - Y-Direction - Damper Forces, 5th Level

Total Required Damping Force per Story = ΣF_5= 1,346.30 k

Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_5/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
3	2 to 3	2	0	2.000	126.54	126.54
4	2 to 3	2	0	2.000		126.54
7	V to W	2	15	1.932		131.00
10	9 to 10	2	-45	1.414		178.95
11	M to N	2	45	1.414		178.95
14	5 to 6	2	20	1.879		134.66

CA S-3 with VFD - Y-Direction - Damper Forces, Penthouse Level						
Total Required Damping Force per Story = ΣF_{PENT}= 1,118.66 k						
Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_{PENT}/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
3	2 to 3	2	0	2.000	105.14	105.14
4	2 to 3	2	0	2.000		105.14
7	V to W	2	15	1.932		108.85
10	9 to 10	2	-45	1.414		148.69
11	M to N	2	45	1.414		148.69
14	5 to 6	2	20	1.879		111.89

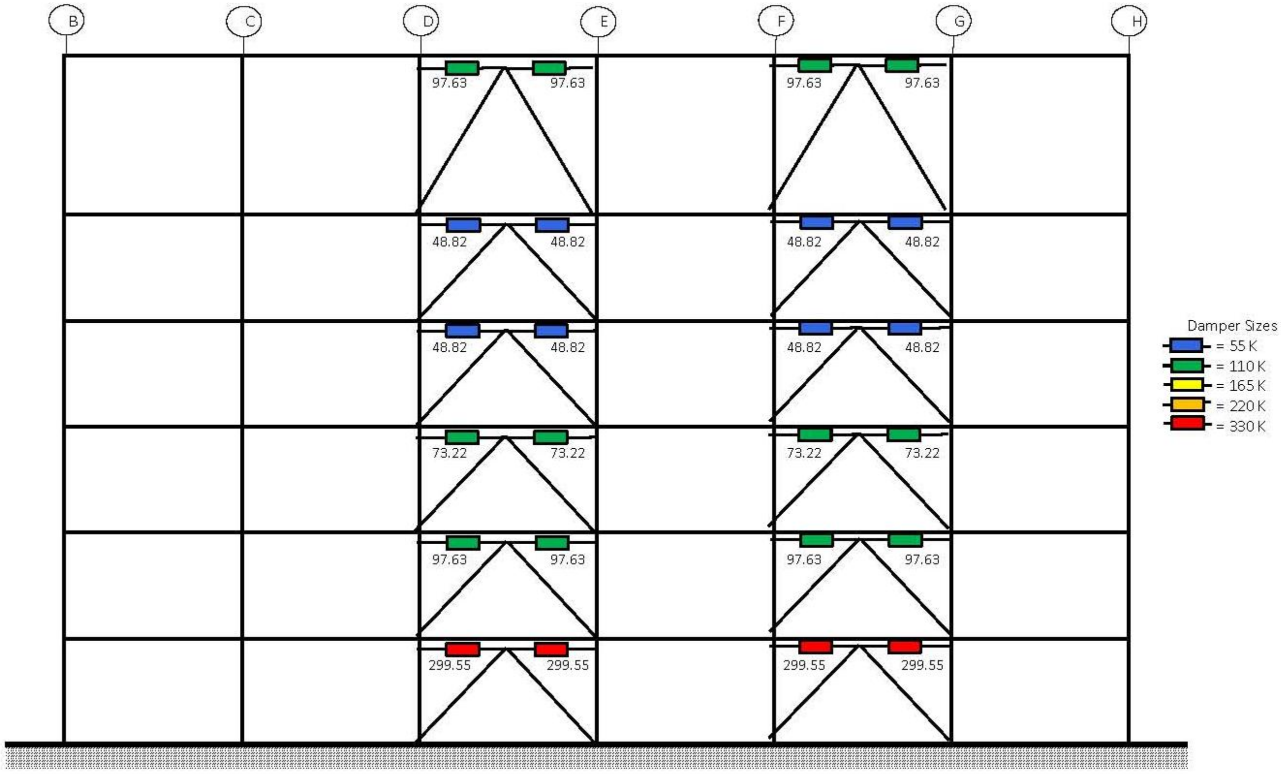
CA S-3 with VFD - Y-Direction - Damper Forces, Atrium Roof Level						
Total Required Damping Force per Story = ΣF_{ATR}= 1,057.89 k						
Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_{ATR}/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
3	2 to 3	2	0	2.000	99.43	99.43
4	2 to 3	2	0	2.000		99.43
7	V to W	2	15	1.932		102.94
10	9 to 10	2	-45	1.414		140.61
11	M to N	2	45	1.414		140.61
14	5 to 6	2	20	1.879		105.81

CA S-3 with VFD - Y-Direction - Damper Forces, Chiller Roof Level						
Total Required Damping Force per Story = ΣF_{CHLR}= 1,075.34 k						
Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_{CHLR}/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
3	2 to 3	2	0	2.000	181.28	181.28
4	2 to 3	2	0	2.000		181.28
7	V to W	2	15	1.932		187.68

CA S-3 with VFD - Y-Direction - Damper Forces, AHU Roof Level						
Total Required Damping Force per Story = ΣF_{AHU}= 893.82 k						
Frame #	Bay	# of dampers (n_i)	θ_i (deg)	N= $n_i\cos(\theta_i)$	F_{pseduo} (k) = $\Sigma F_{AHU}/\Sigma N$	F_i (k) = $F_{pseudo}/\cos(\theta_i)$
3	2 to 3	2	0	2.000	223.46	223.46
4	2 to 3	2	0	2.000		223.46

Frame Elevations and Damping Coefficient Calculations

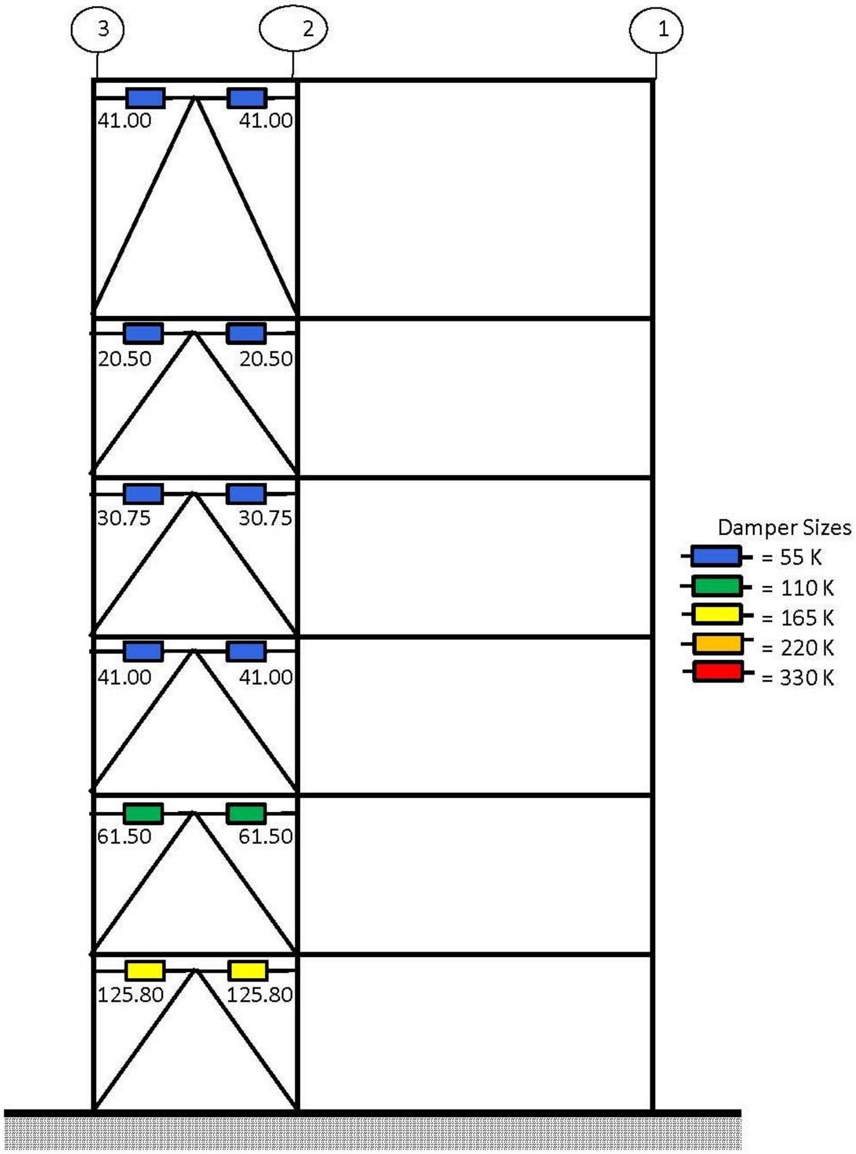
CA S-3 with VFD - Frame 1 - Actual Force Required and Damper Size



CA S-3 with VFD - Frame 1 - Required Damper Properties

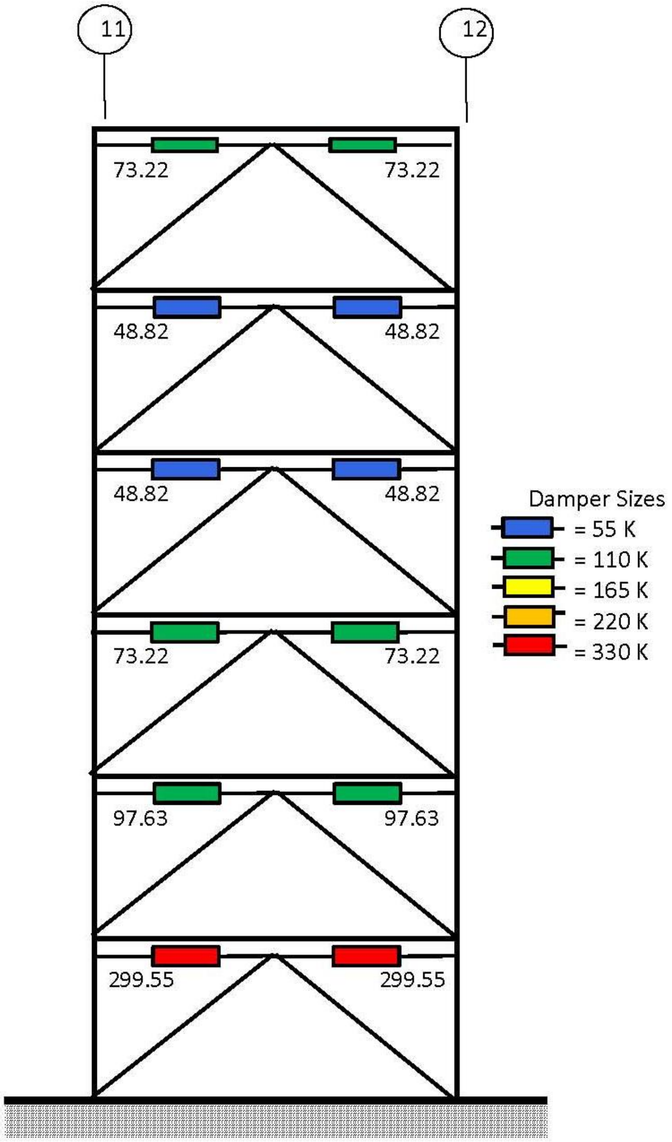
Level	C	Velocity (in/s)	α	F_{req}	Size (k)
AHU Roof	12.35	31.370	0.6	97.63	110
Chiller Roof	N/A	31.370	0.6	N/A	N/A
Atrium Roof	N/A	31.370	0.6	N/A	N/A
Penthouse	6.18	31.370	0.6	48.82	55
5th	6.18	31.370	0.6	48.82	55
4th	9.26	31.370	0.6	73.22	110
3rd	12.35	31.370	0.6	97.63	110
2nd	37.89	31.370	0.6	299.55	330

CA S-3 with VFD - Frames 3 and 4 - Actual Force Required and Damper Size



CA S-3 with VFD - Frames 3 and 4 - Required Damper Properties					
Level	C	Velocity (in/s)	α	F_{req}	Size (k)
AHU Roof	5.19	31.370	0.6	41.00	55
Chiller Roof	N/A	31.370	0.6	N/A	N/A
Atrium Roof	N/A	31.370	0.6	N/A	N/A
Penthouse	2.59	31.370	0.6	20.50	55
5th	3.89	31.370	0.6	30.75	55
4th	5.19	31.370	0.6	41.00	55
3rd	7.78	31.370	0.6	61.50	110
2nd	15.91	31.370	0.6	125.80	165

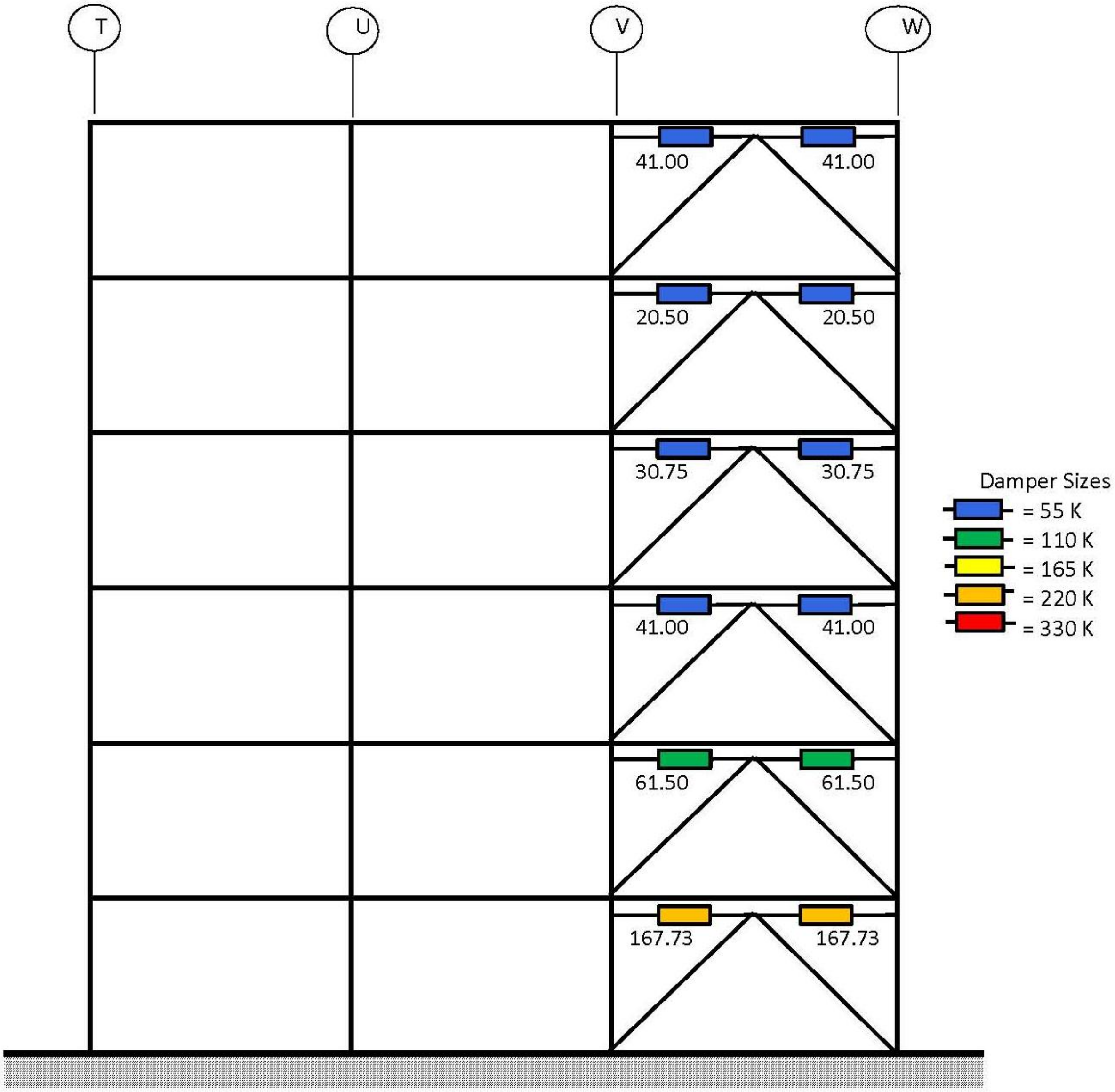
CA S-3 with VFD - Frame 5 - Actual Force Required and Damper Size



CA S-3 with VFD - Frame 5 - Required Damper Properties

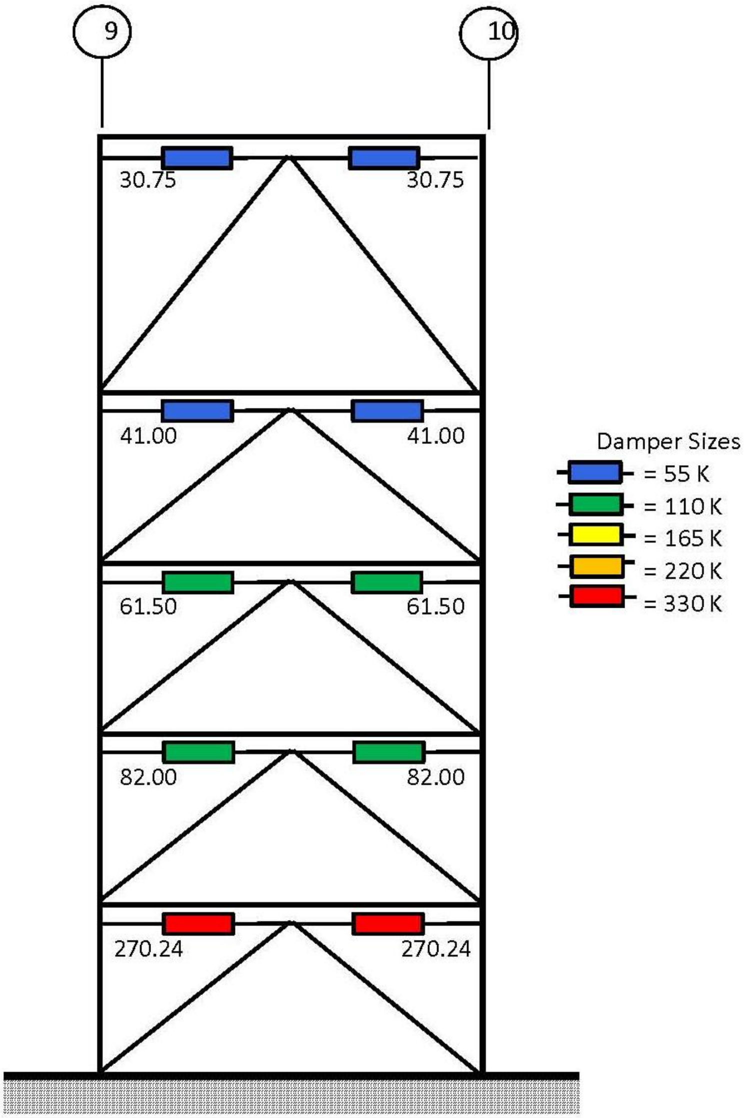
Level	C	Velocity (in/s)	α	F_{req}	Size (k)
AHU Roof	N/A	31.370	0.6	N/A	N/A
Chiller Roof	9.26	31.370	0.6	73.22	110
Atrium Roof	N/A	31.370	0.6	N/A	N/A
Penthouse	6.18	31.370	0.6	48.82	55
5th	6.18	31.370	0.6	48.82	55
4th	9.26	31.370	0.6	73.22	110
3rd	12.35	31.370	0.6	97.63	110
2nd	37.89	31.370	0.6	299.55	330

CA S-3 with VFD - Frame 7 - Actual Force Required and Damper Size



CA S-3 with VFD - Frame 7 - Required Damper Properties					
Level	C	Velocity (in/s)	α	F_{req}	Size (k)
AHU Roof	N/A	31.370	0.6	N/A	N/A
Chiller Roof	5.19	31.370	0.6	41.00	55
Atrium Roof	N/A	31.370	0.6	N/A	N/A
Penthouse	2.59	31.370	0.6	20.50	55
5th	3.89	31.370	0.6	30.75	55
4th	5.19	31.370	0.6	41.00	55
3rd	7.78	31.370	0.6	61.50	110
2nd	21.22	31.370	0.6	167.73	220

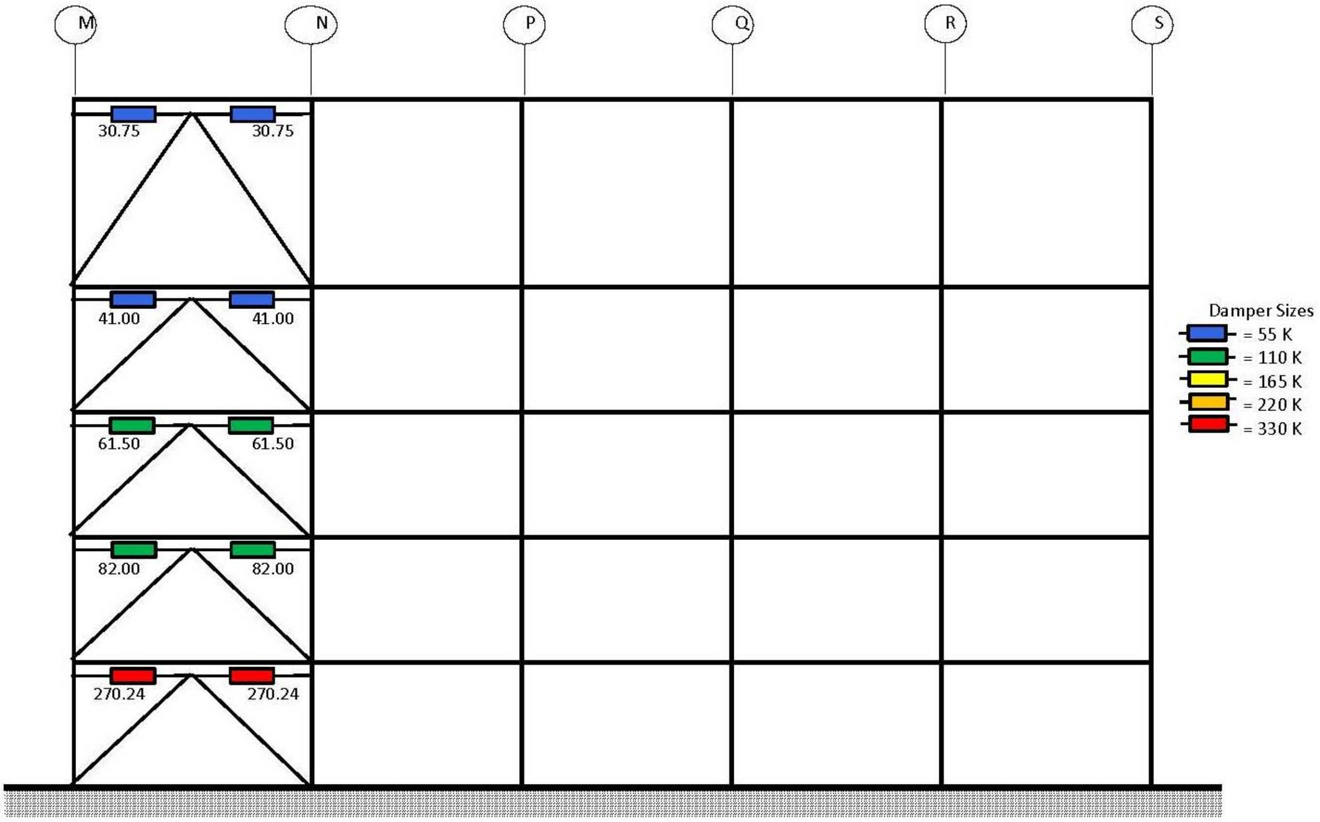
CA S-3 with VFD - Frame 10 - Actual Force Required and Damper Size



CA S-3 with VFD - Frame 10 - Required Damper Properties

Level	C	Velocity (in/s)	α	F_{req}	Size (k)
AHU Roof	N/A	31.370	0.6	N/A	N/A
Chiller Roof	N/A	31.370	0.6	N/A	N/A
Atrium Roof	3.89	31.370	0.6	30.75	55
Penthouse	N/A	31.370	0.6	N/A	N/A
5th	5.19	31.370	0.6	41.00	55
4th	7.78	31.370	0.6	61.50	110
3rd	10.37	31.370	0.6	82.00	110
2nd	34.19	31.370	0.6	270.24	330

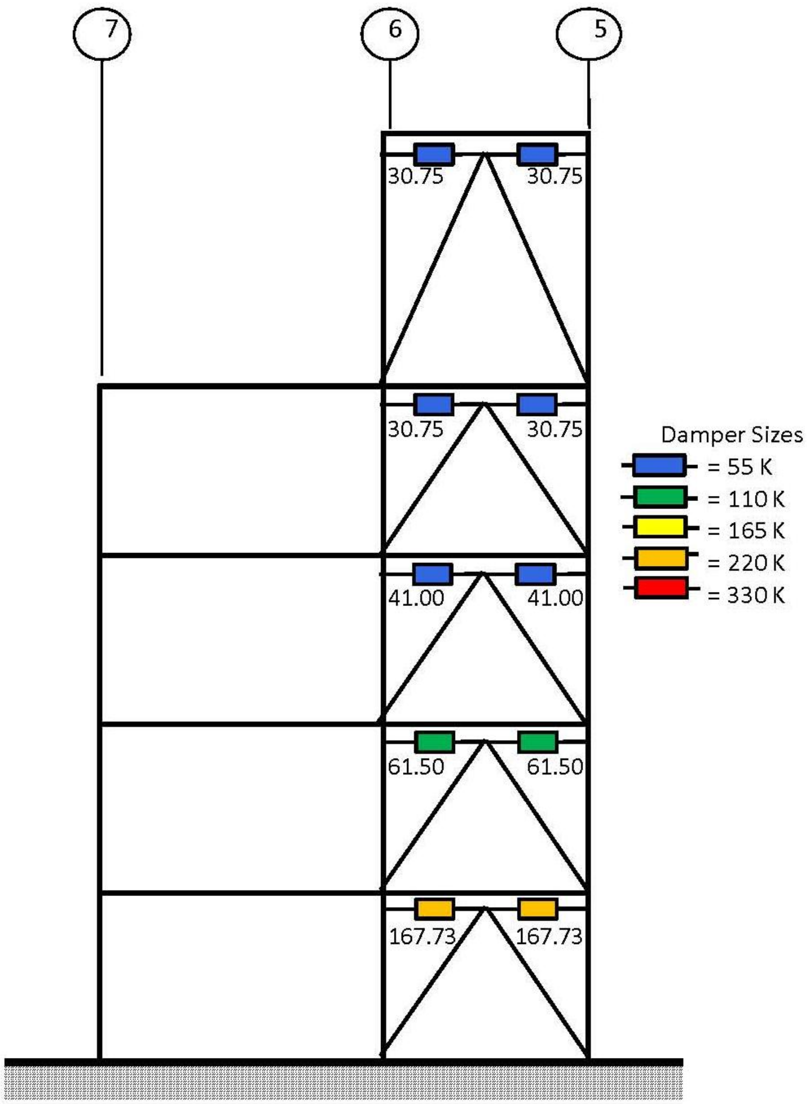
CA S-3 with VFD - Frame 11 - Actual Force Required and Damper Size



CA S-3 with VFD - Frame 11 - Required Damper Properties

Level	C	Velocity (in/s)	α	F_{req}	Size (k)
AHU Roof	N/A	31.370	0.6	N/A	N/A
Chiller Roof	N/A	31.370	0.6	N/A	N/A
Atrium Roof	3.89	31.370	0.6	30.75	55
Penthouse	N/A	31.370	0.6	N/A	N/A
5th	5.19	31.370	0.6	41.00	55
4th	7.78	31.370	0.6	61.50	110
3rd	10.37	31.370	0.6	82.00	110
2nd	34.19	31.370	0.6	270.24	330

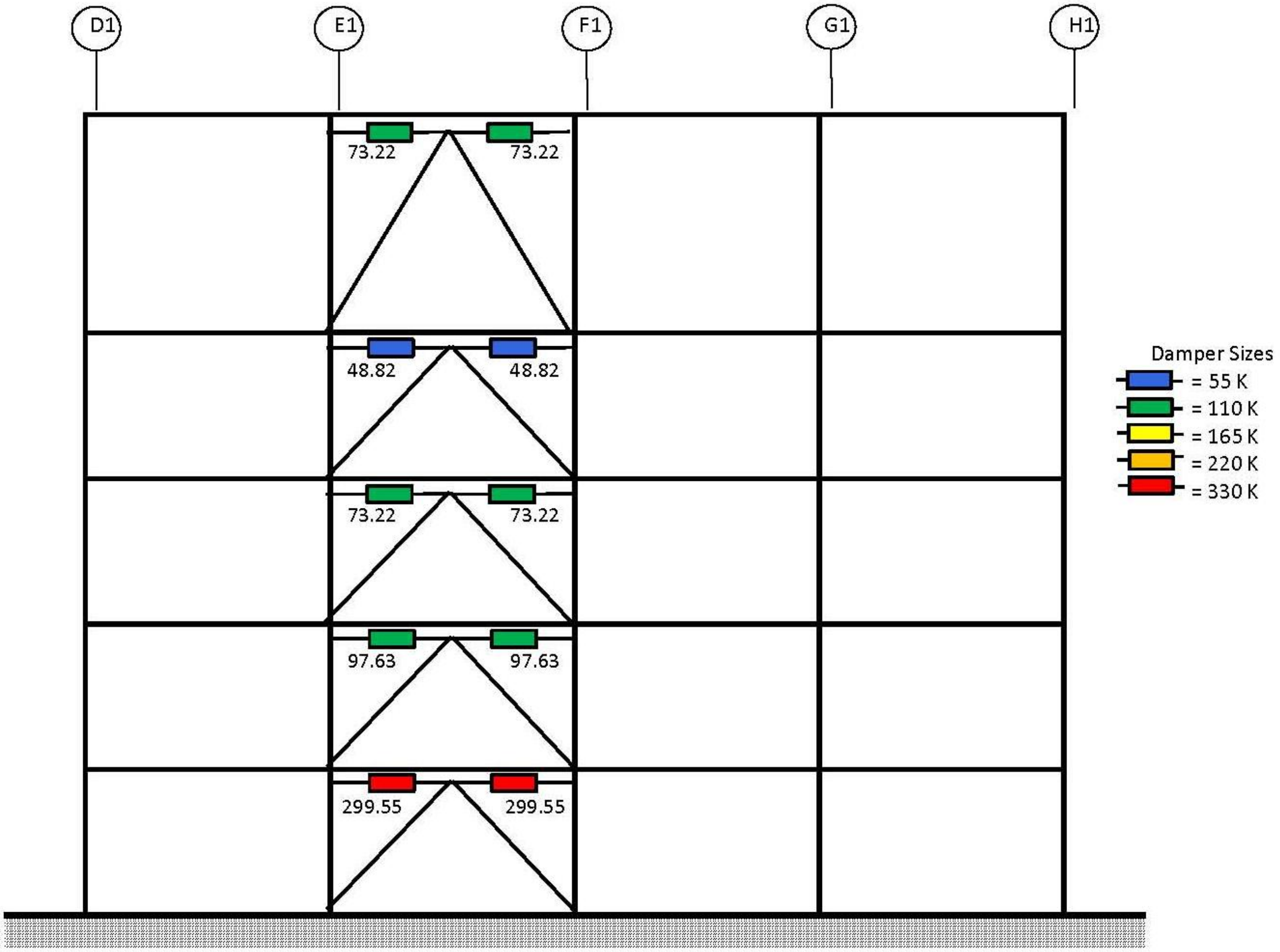
CA S-3 with VFD - Frame 14- Actual Force Required and Damper Size



CA S-3 with VFD - Frame 14 - Required Damper Properties

Level	C	Velocity (in/s)	α	F_{req}	Size (k)
AHU Roof	N/A	31.370	0.6	N/A	N/A
Chiller Roof	N/A	31.370	0.6	N/A	N/A
Atrium Roof	3.89	31.370	0.6	30.75	55
Penthouse	N/A	31.370	0.6	N/A	N/A
5th	3.89	31.370	0.6	30.75	55
4th	5.19	31.370	0.6	41.00	55
3rd	7.78	31.370	0.6	61.50	110
2nd	21.22	31.370	0.6	167.73	220

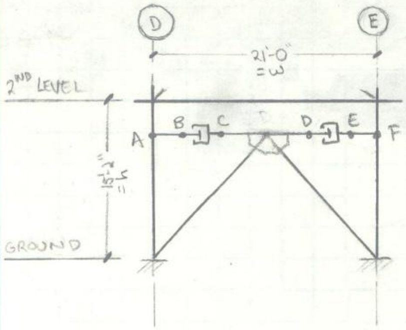
CA S-3 with VFD - Frame 15- Actual Force Required and Damper Size



CA S-3 with VFD - Frame 15 - Required Damper Properties					
Level	C	Velocity (in/s)	α	F_{req}	Size (k)
AHU Roof	N/A	31.370	0.6	N/A	N/A
Chiller Roof	N/A	31.370	0.6	N/A	N/A
Atrium Roof	9.26	31.370	0.6	73.22	110
Penthouse	N/A	31.370	0.6	N/A	N/A
5th	6.18	31.370	0.6	48.82	55
4th	9.26	31.370	0.6	73.22	110
3rd	12.35	31.370	0.6	97.63	110
2nd	37.89	31.370	0.6	299.55	330

Damper Support/Brace Connections Design

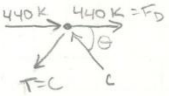
FINAL REPORT | EA 5-3 w/ VFD BRACE/DAMPER CONNECTIONS | Pg 1 OF 6



SIZE BRACES & DAMPER SUPPORTS, DESIGN CENTRAL GUSSET PLATE & CONNECTIONS, AND DESIGN DAMPER-TO-COLUMN CONNECTION FOR 1ST STORY OF DAMPER FRAME BETWEEN GRIDS D & E IN FRAME 1

DAMPERS ARE TAYLOR DEVICES FLUID VISCOUS DAMPERS WITH $\alpha = 0.6$ & $C = 181.80\ k\left(\frac{s}{in}\right)^{0.6}$, RATED AT 440 K

☆ TO ENSURE SAFETY OF BRACES/CONNECTIONS, ASSUME BOTH DAMPERS DEVELOP FULL CAPACITY. DESIGN AS THOUGH T=C



$\tan\theta = \frac{h}{\frac{1}{2}\omega}$ ∴ $\theta = \tan^{-1}\frac{2h}{\omega}$

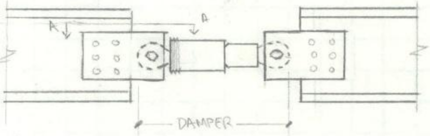
$\cancel{2}C\cos\theta = \cancel{2}F_D$ ∴ $C = \frac{F_D}{\cos\theta} = \frac{F_D}{\cos\left(\tan^{-1}\left(\frac{2h}{\omega}\right)\right)}$

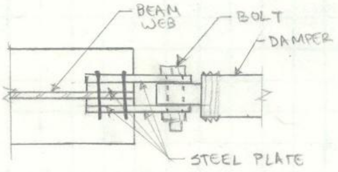
$$C = \frac{440\ K}{\cos\left(\tan^{-1}\left(\frac{2(15.17')}{21'}\right)\right)} = 773\ K$$

$$L = \sqrt{15.17^2 + 10.5^2} = 18.4\ ft$$

FROM AISC TABLE 4-1, WITH KL = 19 ft, $\boxed{\text{USE W12x96}}$

DAMPER SUPPORT →





SECTION A-A (NTS)

PER TAYLOR INFORMATION, THE DAMPER MID-STROKE LENGTH IS 62" = 5.17 ft

$\frac{21' - 2(5.17')}{4} = L_{AB} = 2.67' = 2'-8"$ ∴ $L_{CD} = 5'-4"$

$C_{CD} = 2(440\ K) = 880\ K$ @ KL = 6 ft, FROM TBL 4-1, $\boxed{\text{USE W12x72}}$

CLEVIS - TO - PLATE CONNECTION →

CLEVIS Ø = 3.5" ∴ $\phi R_n = 0.75(75\ ksi)\pi(1.69\ in)^2 \overset{\text{DOUBLE SHEAR}}{(2)} = 1006\ k$

$\boxed{\text{USE } 3\tfrac{3}{8}\text{" Ø A490X BOLT}}$

FINAL REPORT

CA S-3 w/ VFD
BRACE/DAMPER CONNECTIONS

pg 2 OF 6

PLATE THICKNESS REQUIRED:

PLATE YIELD → $\phi P_n = 0.9(9'')(2)t_{p,req}(50\text{ ksi}) = 880\text{ K} \rightarrow t_{p,req} = 1.09\text{ in}$!!

PLATE RUPTURE → $\phi P_n = 0.75(9''-3.5'')(2)t_{p,req}(65\text{ ksi}) = 880\text{ K} \rightarrow t_{p,req} = 1.64\text{ in}$

TRY (1) 1¾" THICK A992 PLATE EACH SIDE

BEARING → $\phi R_n = 0.75(2.4)(65\text{ ksi})(2)(1.75'')(3.375'') = 1382\text{ K} >$ BOLT SHEAR
∴ B.S. CONTROLS

TEAR-OUT → $\phi R_n = 0.75(1.2)(65\text{ ksi})(2)(1.75'')[6.5''-\frac{1}{2}(3.375''+\frac{1}{8}'')] = 972\text{ K} \leftarrow$ CONTROLS
$> P_u$ OK ✓

PLATE-TO-BEAM CONNECTION →

SIZE INNER PLATE FOR GEOMETRY: (MUST BE WELDED TO WEB)

$t_w = 0.43\text{ in}$
$t_{CLEVIS} = 3.5\text{ in}$

$\rangle\ t_{p,req} = \frac{3.5-0.43\text{ in}}{2} = 1.535\text{ in}$

$\boxed{\text{USE (1) 1.5'' THICK A992 ⅊ EACH SIDE}}$

BOLTS → TRY 1½" ⌀ A490 BOLTS:

$\phi R_n = 0.75(60\text{ ksi})\,\pi(0.75\text{ in})^2(2) = 159\text{ K}$

$n = \frac{P_u}{\phi R_n} = 5.53 \rightarrow \boxed{\text{USE (6) 1½'' ⌀ A490N BOLTS}}$

⅊ BEARING → $\phi R_n = 0.75(2.4)(65\text{ ksi})(2)(1.75'')(1.5'') = 614\text{ K} > \phi R_{n\,bolt}$

WEB BEARING → $\phi R_n = 0.75(2.4)(65\text{ksi})(0.43''+1.5''+1.5'')(1.5'') = 602\text{ K} > \phi R_{n\,bolt}$

TEAR-OUT @ ⅊ EDGE → $\phi R_n = 0.75(1.2)(65\text{ ksi})(2)(1.75'')[2''-\frac{1}{2}(1.5''+\frac{1}{8}'')] = 243\text{ K} > \phi R_{n\,bolt}$

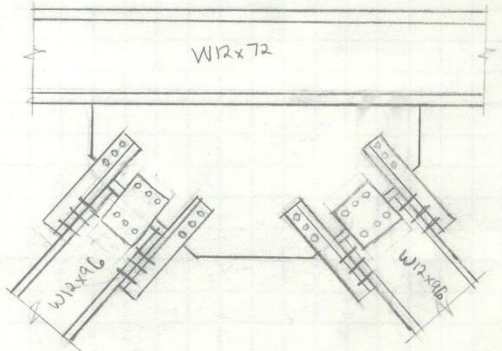
TEAR-OUT @ WEB EDGE → $\phi R_n = 0.75(1.2)(65\text{ksi})(3.43\text{ in})[l_{req}-\frac{1}{2}(1.5''+\frac{1}{8}'')] = \frac{880-3(159\text{K})}{3} = 134.3\text{K}$

$l_{req} = 1.48'' \rightarrow \boxed{\text{USE 1½''}}$

PLATE YIELD: $\phi P_n = 0.9(9'')(2)(3.5'')(50\text{ ksi}) = 2835\text{ K}$ OK ✓

PLATE RUPTURE: $\phi P_n = 0.75(2)(3.5'')(65\text{ksi})[9''-3(1.5''+\frac{1}{8}'')] = 1408\text{ K}$ OK ✓

CENTRAL GUSSET PLATE →



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NUMBER OF BOLTS REQUIRED:

TRY 1" Ø A490N BOLTS → $\phi R_n = 70.7\text{K}$ (DOUBLE SHEAR)

$n = \frac{P_u}{\phi R_n} = \frac{773\text{ K}}{70.7\text{ K}} = 10.9$ → TRY (12) 1" Ø A490N BOLTS

6 BOLTS IN WEB PLATE CONNECTION, 3 BOLTS EACH ANGLE

WEB PLATES THICKNESS:

PLATE YIELD → $\phi P_n = 0.9(9")(2)t_{p,req}(50\text{ ksi}) = 773\text{K} \rightarrow t_{p,req} = 0.95\text{ in}$

PLATE RUPTURE → $\phi P_n = 0.75(2)t_{p,req}(65\text{ksi})[9-2(1"+\frac{1}{8}")] = 773\text{ K} \rightarrow t_{p,req} = 1.17\text{in}$

TRY (1) 1¼" THICK A992 PL EACH SIDE

PL BEARING → $\phi R_n = 0.75(2.4)(65\text{ ksi})(2)(1.25")(1") = 292.5\text{K} > \phi R_{n\text{ bolt}}$

WEB BEARING → $\phi R_n = 0.75(2.4)(65\text{ksi})(0.55\text{ in})(1") = 64.35\text{ K}$

PL TEAR-OUT → $\phi R_n = 0.75(1.2)(65\text{ksi})(2)(1.25")[1.5" - \frac{1}{2}(1"+\frac{1}{8}")] = 137\text{ K} > \phi R_{n,\text{bolt}}$

WEB TEAR-OUT → $\phi R_n = 0.75(1.2)(65\text{ksi})(0.55")[3" - \frac{1}{2}(1+\frac{1}{8}")] = 78.4\text{K} > \phi R_{n,\text{bolt}}$

WEB PLATE CONNECTION STRENGTH $= 3(64.35\text{ k}) + 3(70.7\text{k}) = \underline{405.15\text{ K}}$

$\boxed{\text{USE (1) 1¼" THICK A992 PL EACH SIDE}}$

ANGLES: TRY L4x4 (A992)

ANGLE YIELD → $\phi P_n = 0.9(A_{g,req})(4)(50\text{ ksi}) = 773\text{ k} \rightarrow A_{g,req} = 4.29\text{ in}^2$

TRY L4x4x½" ($A_g = 5.52\text{ in}^2$)

ANGLE RUPTURE → $A_n = 5.52\text{ in}^2 - 1(\frac{1}{2}")(1"+\frac{1}{8}") = 4.96\text{ in}^2$
$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1.18"}{6"} = 0.803$
$A_e = UA_n = 0.803(4.96\text{ in}^2) = 3.98\text{ in}^2$

$\phi P_n = 0.75(3.98\text{ in}^2)(4)(65\text{ ksi}) = \underline{776\text{ K}}$ OK ✓

CHECKED @ BEAM SIDE (GUSSET SIDE IS SAME → ½ n, BUT THICKNESSES & $\phi R_{n,\text{bolts}}$ ARE 2x)
$\phi R_{n\text{ bolt}} = 35.3\text{k}$

ANGLE BEARING → $\phi R_n = 0.75(2.4)(65\text{ksi})(\frac{1}{2}")(1") = 58.5\text{ K} > \phi R_{n,\text{bolt}}$

FLANGE BEARING → $\phi R_n = 0.75(2.4)(65\text{ksi})(0.900")(1") = 105\text{ K} > \phi R_{n,\text{bolt}}$

ANGLE TEAR-OUT → $\phi R_n = 0.75(1.2)(65\text{ksi})(\frac{1}{2}")[1.5" - \frac{1}{2}(1"+\frac{1}{8}")] = 27.4\text{ K}$

FLANGE TEAR-OUT → $\phi R_n = 0.75(1.2)(65\text{ksi})(0.900")[1.5" - \frac{1}{2}(1"+\frac{1}{8}")] = 49.4\text{ K} > \phi R_{n\text{ bolt}}$

CLAW ANGLE CONNECTION STRENGTH $= 1(27.4) + 2(35.3) = 98\text{ K / ANGLE}$
$(98\text{ K})4 = \underline{392\text{ K}} + 405.15 = 797.15\text{K} > P_u$
OK ✓

$\boxed{\text{USE (4) L4x4x½" A992 ANGLES}}$

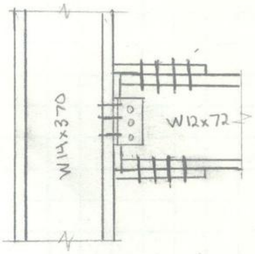
GUSSET PLATE: FOR SIMPLICITY OF CONNECTION, USE ½" THICK A992 GUSSET PLATE

↳ SEE ANGLE BEARING/TEAR-OUT FOR LIMIT STATES w/ ANGLES (x2, ½ n)

↳ SEE WEB BEARING/TEAR-OUT FOR LIMIT STATES w/ WEB PLATE

↳ GEOMETRY FOUND USING AUTOCAD

SUPPORT-TO-COLUMN MOMENT CONNECTION →



SHEAR FORCE, V → IMBALACE BETWEEN C CAPACITY & DEMAND IN BRACES

MOMENT, M → V x LENGTH OF END SUPPORT

COMPRESSION CAPACITY → $\phi P_n = 852\ K$

$V = 852\ K - 773\ K = 79\ K > \phi V_n = 158\ K$ OK✓

$M = 79K(2.67\ ft) = 211\ k\cdot ft \times 12\ in/ft = 2531\ k\text{-}in$
$> \phi M_p$ OK✓

DOUBLE-ANGLE WEB CONNECTION →
USING TABLE 10-1 IN AISC, $\boxed{\text{USE (2) A36 L}3\tfrac{1}{2}\times 3\tfrac{1}{2}\times \tfrac{5}{16}\text{ w/ (3) }\tfrac{7}{8}''\ \varnothing\text{ A325N BOLTS}}$

FLANGE PLATES

$$F_u = \frac{M}{d} = \frac{2531\ k\text{-}in}{12.3\ in} = 206\ K$$
↖CONSERVATIVE

BOLTS REQ'D → TRY $\tfrac{7}{8}''\ \varnothing$ A490N BOLTS w/ $\phi R_n = 27.1K$

$n = \frac{F_u}{\phi R_n} = 7.60$ → USE (2) ROWS, (4) BOLTS EACH

TENSION PLATE YIELD →

$\phi P_n = 0.9(36\ ksi)(8.5'')\ t_{p,req} = 206k \rightarrow t_{p,req} = 0.75\ in$

TENSION PLATE RUPTURE →

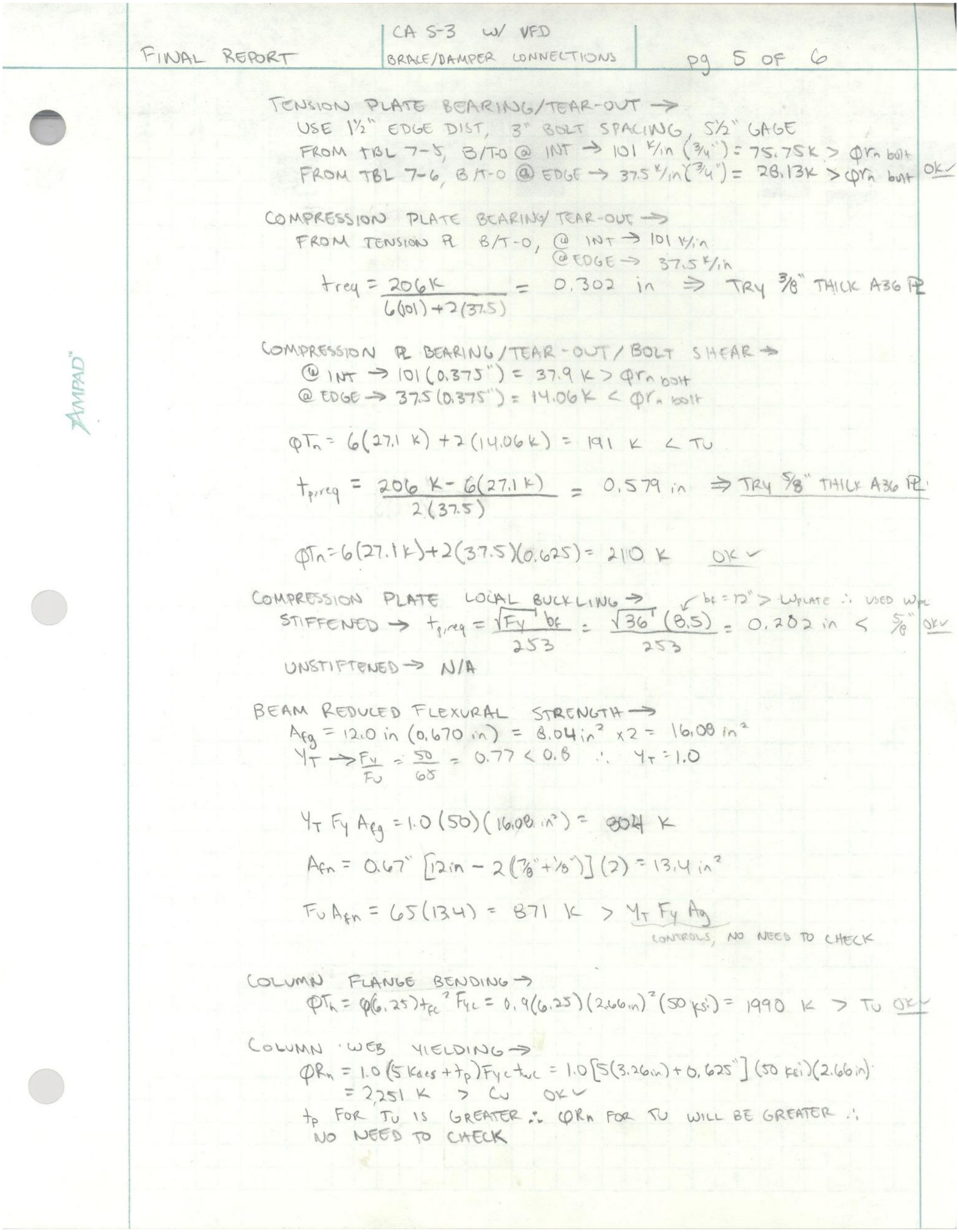
$\phi P_n = 0.75(58\ ksi)[8.5'' - 2(\tfrac{7}{8}'' + \tfrac{1}{8}'')]\ t_{p,req} = 206\ K \rightarrow t_{p,req} = 0.73\ in$

TRY ¾" THICK A36 PL

BLOCK SHEAR → $R_T = (36\ ksi)(0.75'')[3'' - 2(\tfrac{1}{2})(\tfrac{7}{8}'' + \tfrac{1}{8}'')] = 54\ K$

$Y_v = 0.6(36\ ksi)(0.75'')(10.5'')(2) = 340.2\ K$ ← CONTROLS
$R_v = 0.6(58\ ksi)(0.75'')[10.5'' - 3.5(\tfrac{7}{8}'' + \tfrac{1}{8}'')](2) = 365.4\ K$

$\phi R_n = 0.75(54 + 340.2) = 296\ K > P_u$ OK✓



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TENSION PLATE BEARING/TEAR-OUT →

USE 1½" EDGE DIST, 3" BOLT SPACING, 5½" GAGE

FROM TBL 7-5, B/T-O @ INT → 101 k/in (¾") = 75.75 k > $\phi r_{n\,bolt}$

FROM TBL 7-6, B/T-O @ EDGE → 37.5 k/in (¾") = 28.13 k > $\phi r_{n\,bolt}$ OK ✓

COMPRESSION PLATE BEARING/TEAR-OUT →

FROM TENSION PL B/T-O, @ INT → 101 k/in
@ EDGE → 37.5 k/in

$$t_{req} = \frac{206k}{6(101)+2(37.5)} = 0.302 \text{ in} \Rightarrow \text{TRY } 3/8'' \text{ THICK A36 PL}$$

COMPRESSION PL BEARING/TEAR-OUT/BOLT SHEAR →

@ INT → 101 (0.375") = 37.9 k > $\phi r_{n\,bolt}$

@ EDGE → 37.5 (0.375") = 14.06 k < $\phi r_{n\,bolt}$

$$\phi T_n = 6(27.1\text{ k}) + 2(14.06\text{ k}) = 191 \text{ k} < T_u$$

$$t_{p,req} = \frac{206\text{ k} - 6(27.1\text{ k})}{2(37.5)} = 0.579 \text{ in} \Rightarrow \underline{\text{TRY } 5/8'' \text{ THICK A36 PL}}$$

$$\phi T_n = 6(27.1\text{ k}) + 2(37.5)(0.625) = 210 \text{ k} \quad \underline{\text{OK} ✓}$$

COMPRESSION PLATE LOCAL BUCKLING →

$b_f = 12'' > w_{PLATE}$ ∴ USED w_{PL}

STIFFENED → $t_{p,req} = \frac{\sqrt{F_y}\, b_f}{253} = \frac{\sqrt{36}\,(8.5)}{253} = 0.202 \text{ in} < 5/8''$ OK ✓

UNSTIFFENED → N/A

BEAM REDUCED FLEXURAL STRENGTH →

$A_{fg} = 12.0 \text{ in } (0.670 \text{ in}) = 8.04 \text{ in}^2 \times 2 = 16.08 \text{ in}^2$

$Y_T \rightarrow \frac{F_y}{F_u} = \frac{50}{65} = 0.77 < 0.8 \quad ∴ \quad Y_T = 1.0$

$$Y_T F_y A_{fg} = 1.0(50)(16.08 \text{ in}^2) = 804 \text{ k}$$

$$A_{fn} = 0.67'' \left[12 \text{in} - 2\left(7/8'' + 1/8''\right)\right](2) = 13.4 \text{ in}^2$$

$$F_u A_{fn} = 65(13.4) = 871 \text{ k} > \underline{Y_T F_y A_g}$$

CONTROLS, NO NEED TO CHECK

COLUMN FLANGE BENDING →

$$\phi T_n = \phi(6.25) t_{fc}^2 F_{yc} = 0.9(6.25)(2.66 \text{in})^2 (50 \text{ ksi}) = 1990 \text{ k} > T_u \quad \underline{\text{OK} ✓}$$

COLUMN WEB YIELDING →

$$\phi R_n = 1.0(5k_{des} + t_p) F_{yc} t_{wc} = 1.0[5(3.26 \text{in}) + 0.625''](50 \text{ ksi})(2.66 \text{in})$$
$$= 2{,}251 \text{ k} > C_u \quad \text{OK} ✓$$

t_p FOR T_u IS GREATER ∴ ϕR_n FOR T_u WILL BE GREATER ∴ NO NEED TO CHECK

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COLUMN WEB CRIPPLING →

$$\phi R_n = \phi\,(0.8)\,t_{wc}^{2}\left[1+3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{E\,F_{yw}\,t_{fc}}{t_{wc}}}$$

$$\phi R_n = 0.75(0.8)(1.66)^2\left[1+3\left(\frac{0.625''}{17.9''}\right)\left(\frac{1.66}{2.66}\right)^{1.5}\right]\sqrt{\frac{29000\text{ ksi}\,(50\text{ ksi})(2.66)}{1.66}}$$

$$= 2650\text{ K} > C_u$$

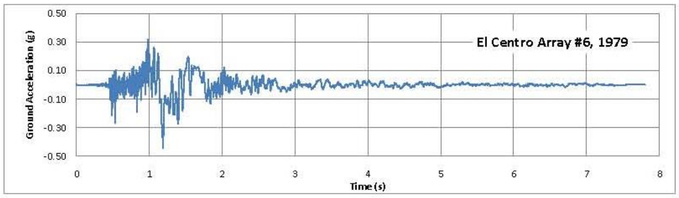
COLUMN WEB BUCKLING →
NOT AN APPLICABLE LIMIT STATE

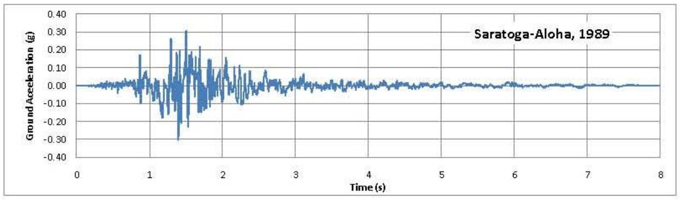
COLUMN WEB PANEL ZONE →
NOT AN APPLICABLE LIMIT STATE

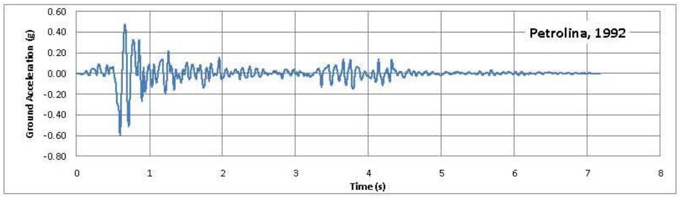
Appendix G: Earthquake Scaling for Nonlinear Analysis

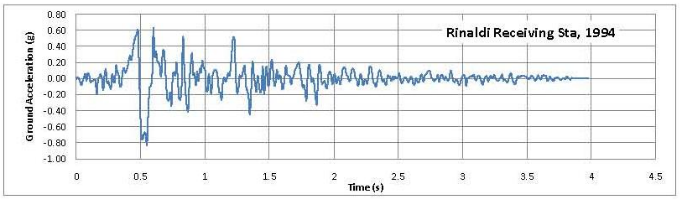
X-Direction

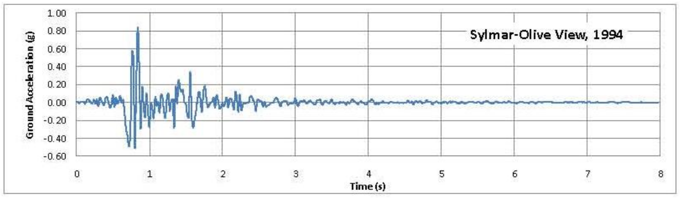
CA S-3 w VFD - X-Direction - Earthquake Ground Acceleration vs. Time Plots

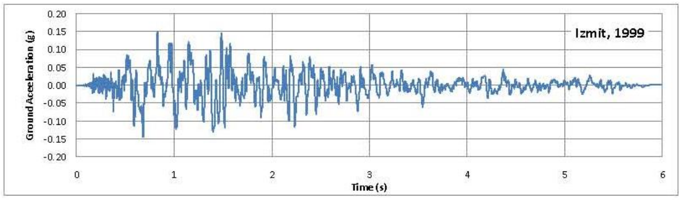


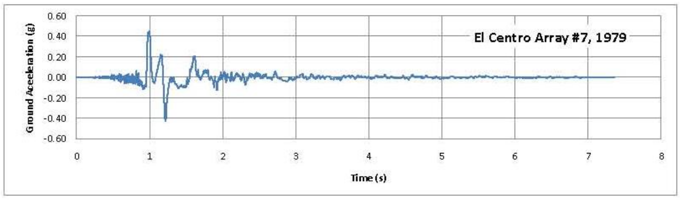












CA S-3 with VFD - X-Direction - Summary Data for Normalizing Response Accelerations							
File	**PGV (cm/sec)**	**PGA (g)**	**ΔT (sec)**			**Step 1 Factor**	**Step 2 Factor**
El Centro #6	111.84	0.4390	0.005			0.6234	0.62343
Saratoga	55.55	0.3046	0.005			1.2552	1.25516
Petrolia	81.87	0.6005	0.020			0.8516	0.85165
Rinaldi	167.05	0.8246	0.010			0.4174	0.41739
Sylmar	122.77	0.8433	0.020			0.5679	0.56793
Izmit	22.61	0.1463	0.005			3.0838	3.08378
El Centro #7	108.79	0.4475	0.005			0.6409	0.64091

Average	0.2864
Idealized Spectrum	0.286

S_{D1} = 0.63

S_{DS} = 1.33

T_0 = 0.095

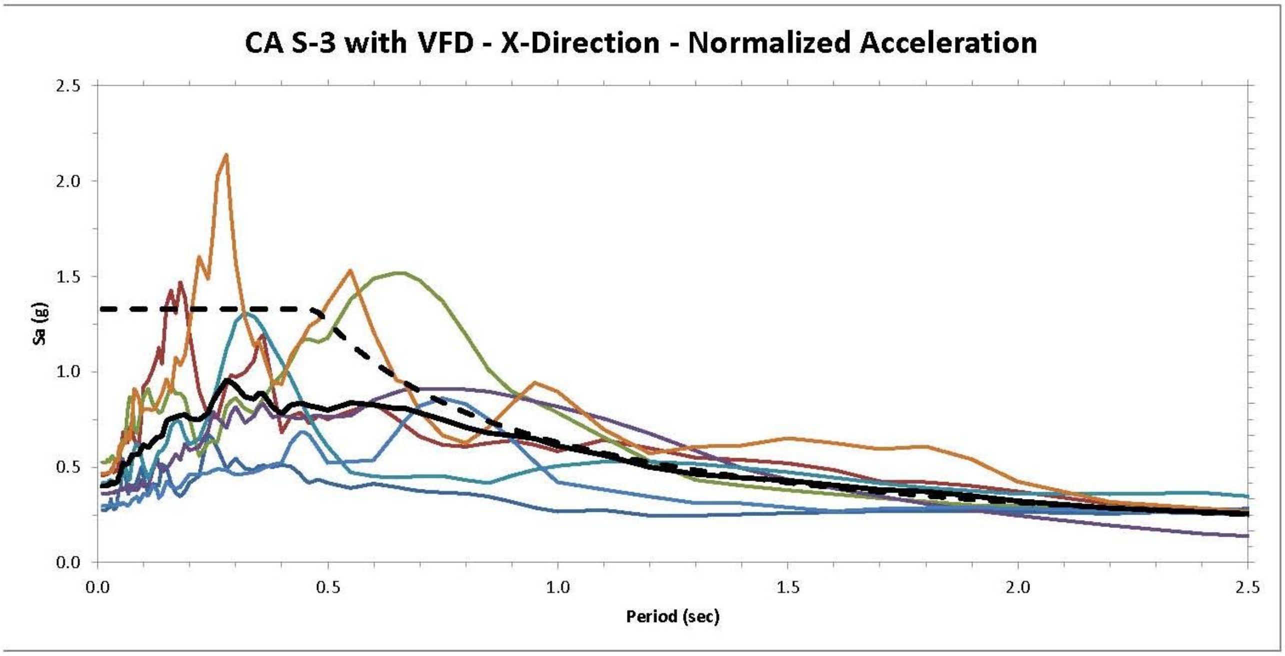
T_s = 0.474

V_g = 69.7

A_0= 0.532

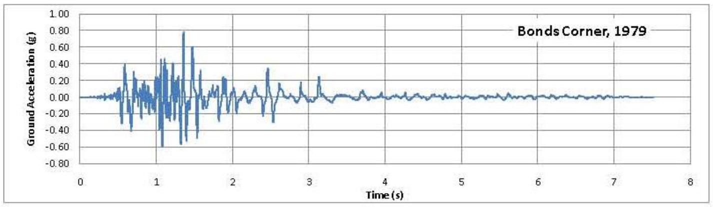
T = Cu.Tb= 2.20

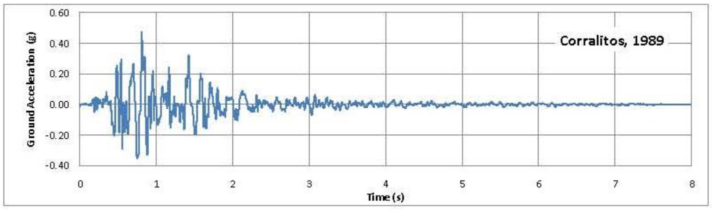
ratio= 1.00000

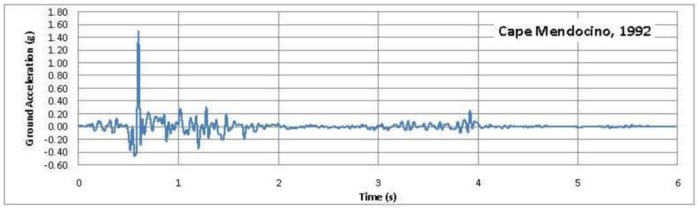


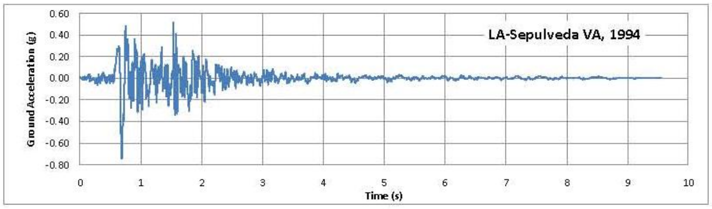
__Y-Direction__

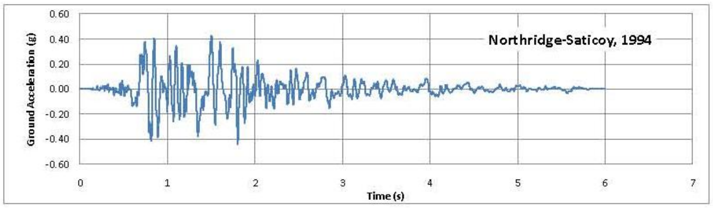
CA S-3 w VFD - Y-Direction - Earthquake Ground Acceleration vs. Time Plots

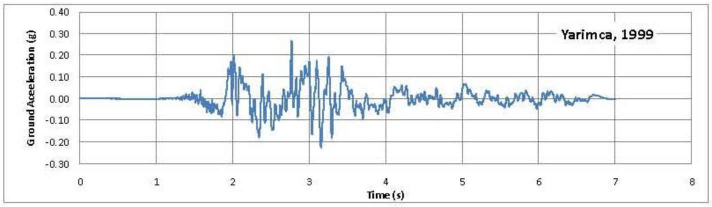


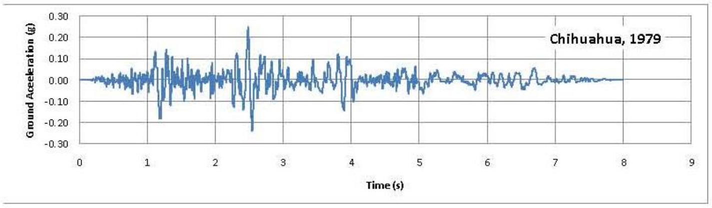












CA S-3 with VFD - Y-Direction - Summary Data for Normalizing Response Accelerations

File	PGV (cm/sec)	PGA (g)	ΔT (sec)			Step 1 Factor	Step 2 Factor
Bonds Corner	44.25	0.7711	0.005			1.5485	1.54846
Corralitos	45.43	0.4738	0.005			1.5082	1.50824
Cape Mendocino	57.61	1.4973	0.020			1.1894	1.18936
Sepulveda	63.22	0.7468	0.005			1.0838	1.08382
Saticoy	53.17	0.4346	0.010			1.2887	1.28868
Yarimca	48.18	0.2659	0.005			1.4221	1.42215
Chihuahua	30.41	0.2469	0.01			2.2532	2.25318

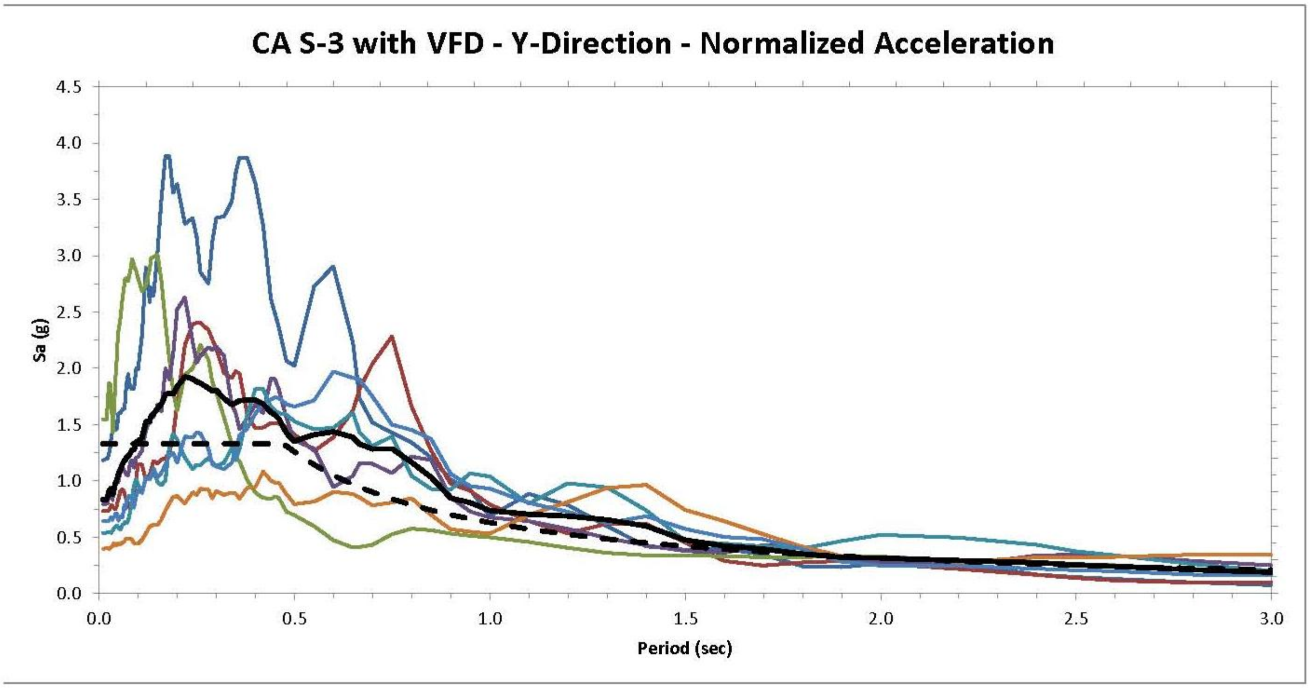
Average	0.2423
Idealized Spectrum	0.242

S_{D1} = 0.63 V_g = 68.5 ratio= 1.00000

S_{DS} = 1.33 A_0= 0.532

T_0 = 0.095 T = Cu.Tb= 2.60

T_s = 0.474



Appendix H: Construction Management Breadth

General Conditions Detailed Estimate Summary

General Conditions Cost per Month Estimate

Project Duration 23 months
Construction Duration 20 months

Personnel

Description	Quantity	Unit	Unit Rate	Cost
Vice President	100	Week	$2,500	$250,000
Project Executive	100	Week	$2,200	$220,000
Project Superintendent	100	Week	$1,925	$192,500
Assistant Superintendent	100	Week	$1,800	$180,000
Field Engineer	100	Week	$1,265	$126,500
Project Manager	100	Week	$2,075	$207,500
Project Engineer	100	Week	$1,800	$180,000
Office Engineer	100	Week	$1,265	$126,500
Project Administrator	23	Month	$800	$18,400
Safety Coordinator	100	Week	$175	$17,500
Project Scheduler	100	Week	$225	$22,500
Estimating Expenses	1	LS	$45,000	$45,000
			Total	$1,586,400

Construction Facilities/Equipment

Description	Quantity	Unit	Unit Rate	Cost
Field Office Trailer Set-Up	1	LS	$2,000	$2,000
Field Office Trailer Rental	23	Month	$425	$9,775
Field Office Trailer Removal	1	LS	$2,500	$2,500
Construction Site Fence	20	Month	$600	$12,000
Sidewalk Overhead Protection	1	LS	$1,250	$1,250
Storage Trailer	15	Month	$140	$2,100
Gang Box	20	Month	$55	$1,100
Tools/Equipment	20	Month	$650	$13,000
Fire Extinguishers	20	Month	$275	$5,500
Personal Protective Equipment	20	Month	$250	$5,000
Dumpsters	20	Month	$1,800	$36,000
Copier/Fax/Printer	23	Month	$400	$9,200
Computer/LAN Equipment	23	Month	$2,400	$55,200
Mobile Phones	23	Month	$325	$7,475
Signage	1	LS	$2,600	$2,600
			Total	$164,700

Temporary Utilities

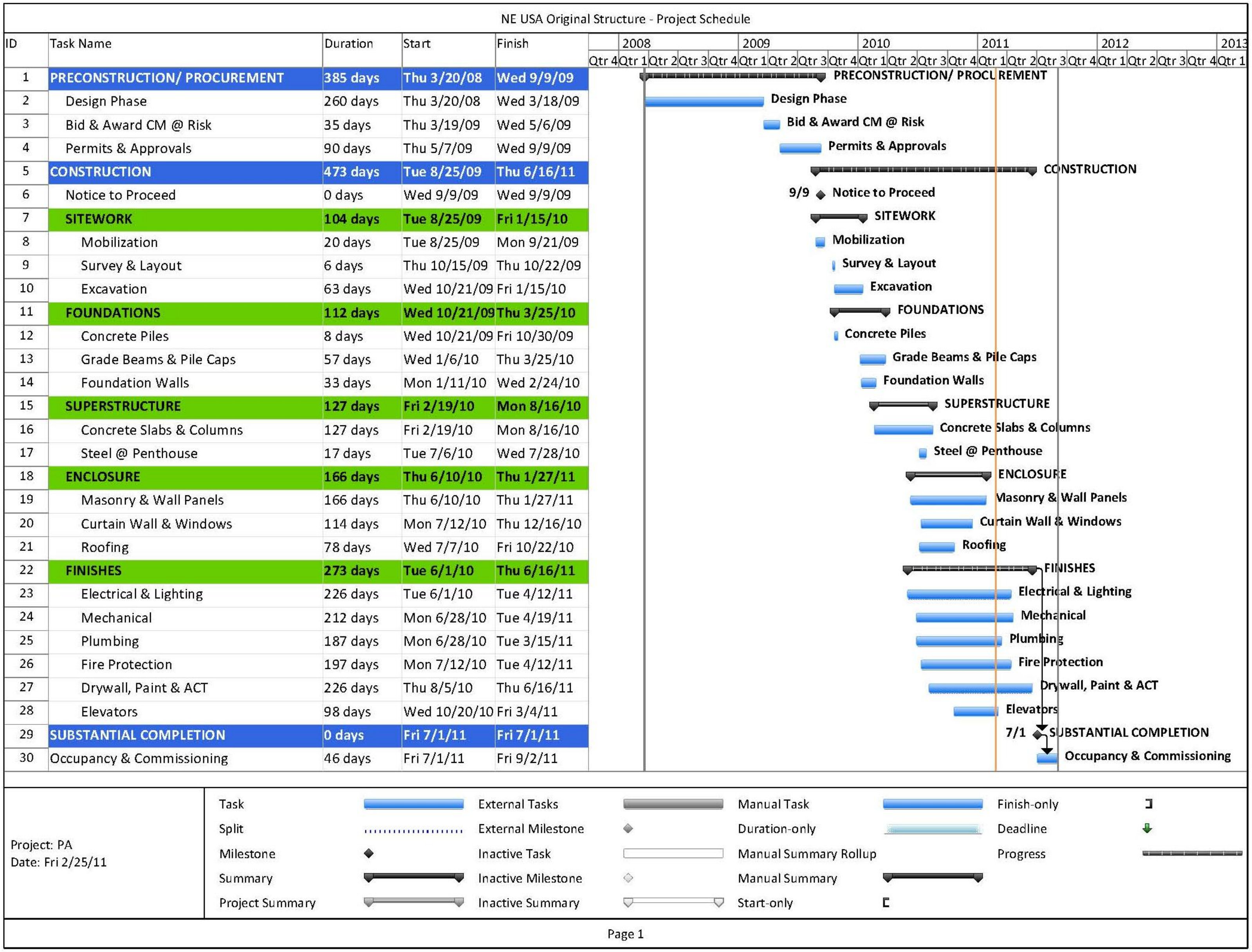
Description	Quantity	Unit	Unit Rate	Cost
Field IT/Network Set-up	1	LS	$4,250	$4,250
Temporary Power Installation	1	LS	$15,000	$15,000
Temporary Power Consumption	20	Month	$750	$15,000
Temporary Water/Sanitary Supply	1	LS	$1,500	$1,500
Temporary Toilets	23	Month	$550	$12,650
Potable Water	23	Month	$175	$4,025
			Total	$52,425

Miscellaneous Costs

Description	Quantity	Unit	Unit Rate	Cost
Progress Photographs	20	Month	$350	$7,000
Travel Expenses (Staff Vehicles)	20	Month	$3,500	$70,000
Delivery/Shipping Expenses	20	Month	$300	$6,000
Clean-Up Expenses	20	Month	$2,000	$40,000
Misc. Field Expenses	20	Month	$1,000	$20,000
Office Supplies	20	Month	$86	$1,720
Document Reproduction	1	LS	$25,000	$25,000
QC & Commissioning (0.5%)	1	LS	$250,000	$250,000
Permits (0.75%)	1	LS	$375,000	$375,000
Insurance (0.3%)	1	LS	$150,000	$150,000
Bonds (0.6%)	1	LS	$300,000	$300,000
			Total	$1,244,720

Total General Conditions Cost	$3,048,245
General Conditions Cost per Month	$132,532

Original Structure



NE USA Original Structure - Project Schedule

ID	Task Name	Duration	Start	Finish
1	**PRECONSTRUCTION/ PROCUREMENT**	**385 days**	**Thu 3/20/08**	**Wed 9/9/09**
2	Design Phase	260 days	Thu 3/20/08	Wed 3/18/09
3	Bid & Award CM @ Risk	35 days	Thu 3/19/09	Wed 5/6/09
4	Permits & Approvals	90 days	Thu 5/7/09	Wed 9/9/09
5	**CONSTRUCTION**	**473 days**	**Tue 8/25/09**	**Thu 6/16/11**
6	Notice to Proceed	0 days	Wed 9/9/09	Wed 9/9/09
7	**SITEWORK**	**104 days**	**Tue 8/25/09**	**Fri 1/15/10**
8	Mobilization	20 days	Tue 8/25/09	Mon 9/21/09
9	Survey & Layout	6 days	Thu 10/15/09	Thu 10/22/09
10	Excavation	63 days	Wed 10/21/09	Fri 1/15/10
11	**FOUNDATIONS**	**112 days**	**Wed 10/21/09**	**Thu 3/25/10**
12	Concrete Piles	8 days	Wed 10/21/09	Fri 10/30/09
13	Grade Beams & Pile Caps	57 days	Wed 1/6/10	Thu 3/25/10
14	Foundation Walls	33 days	Mon 1/11/10	Wed 2/24/10
15	**SUPERSTRUCTURE**	**127 days**	**Fri 2/19/10**	**Mon 8/16/10**
16	Concrete Slabs & Columns	127 days	Fri 2/19/10	Mon 8/16/10
17	Steel @ Penthouse	17 days	Tue 7/6/10	Wed 7/28/10
18	**ENCLOSURE**	**166 days**	**Thu 6/10/10**	**Thu 1/27/11**
19	Masonry & Wall Panels	166 days	Thu 6/10/10	Thu 1/27/11
20	Curtain Wall & Windows	114 days	Mon 7/12/10	Thu 12/16/10
21	Roofing	78 days	Wed 7/7/10	Fri 10/22/10
22	**FINISHES**	**273 days**	**Tue 6/1/10**	**Thu 6/16/11**
23	Electrical & Lighting	226 days	Tue 6/1/10	Tue 4/12/11
24	Mechanical	212 days	Mon 6/28/10	Tue 4/19/11
25	Plumbing	187 days	Mon 6/28/10	Tue 3/15/11
26	Fire Protection	197 days	Mon 7/12/10	Tue 4/12/11
27	Drywall, Paint & ACT	226 days	Thu 8/5/10	Thu 6/16/11
28	Elevators	98 days	Wed 10/20/10	Fri 3/4/11
29	**SUBSTANTIAL COMPLETION**	**0 days**	**Fri 7/1/11**	**Fri 7/1/11**
30	Occupancy & Commissioning	46 days	Fri 7/1/11	Fri 9/2/11

Project: PA
Date: Fri 2/25/11

Task		External Tasks		Manual Task		Finish-only	]
Split		External Milestone	◆	Duration-only		Deadline	⇩
Milestone	◆	Inactive Task		Manual Summary Rollup		Progress	
Summary		Inactive Milestone	◇	Manual Summary			
Project Summary		Inactive Summary		Start-only	[		

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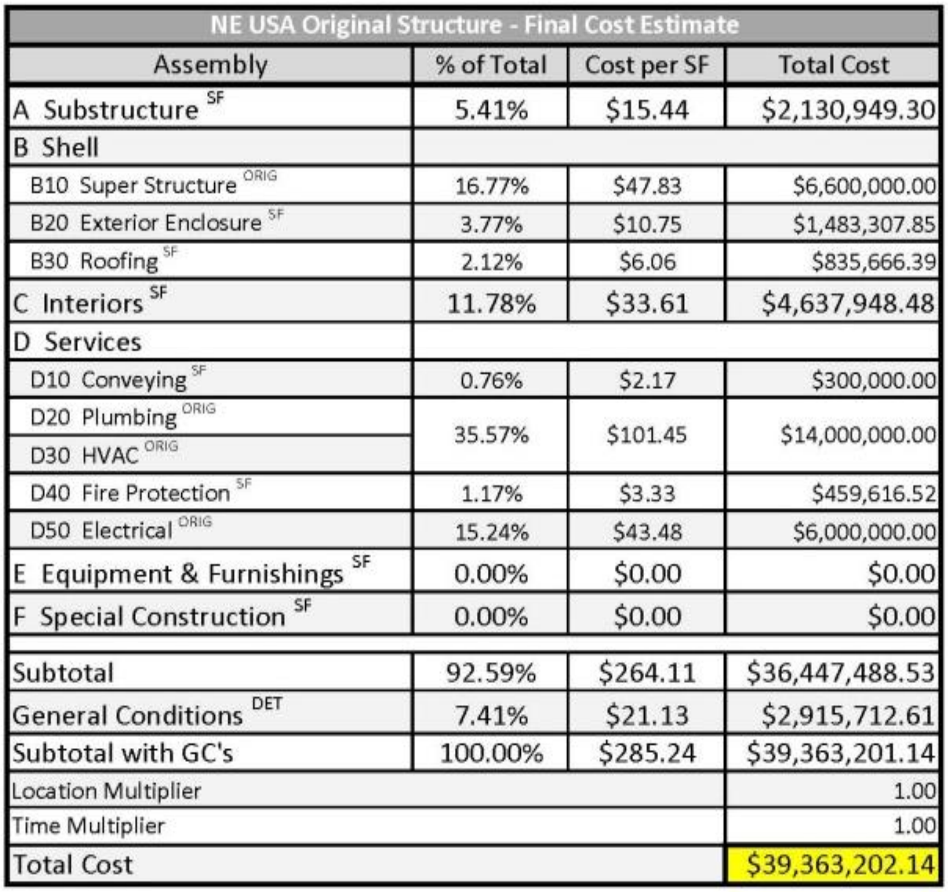


NE USA Original Structure - Final Cost Estimate

Assembly	% of Total	Cost per SF	Total Cost
A Substructure [SF]	5.41%	$15.44	$2,130,949.30
B Shell			
B10 Super Structure [ORIG]	16.77%	$47.83	$6,600,000.00
B20 Exterior Enclosure [SF]	3.77%	$10.75	$1,483,307.85
B30 Roofing [SF]	2.12%	$6.06	$835,666.39
C Interiors [SF]	11.78%	$33.61	$4,637,948.48
D Services			
D10 Conveying [SF]	0.76%	$2.17	$300,000.00
D20 Plumbing [ORIG]	35.57%	$101.45	$14,000,000.00
D30 HVAC [ORIG]			
D40 Fire Protection [SF]	1.17%	$3.33	$459,616.52
D50 Electrical [ORIG]	15.24%	$43.48	$6,000,000.00
E Equipment & Furnishings [SF]	0.00%	$0.00	$0.00
F Special Construction [SF]	0.00%	$0.00	$0.00
Subtotal	92.59%	$264.11	$36,447,488.53
General Conditions [DET]	7.41%	$21.13	$2,915,712.61
Subtotal with GC's	100.00%	$285.24	$39,363,201.14
Location Multiplier			1.00
Time Multiplier			1.00
Total Cost			$39,363,202.14

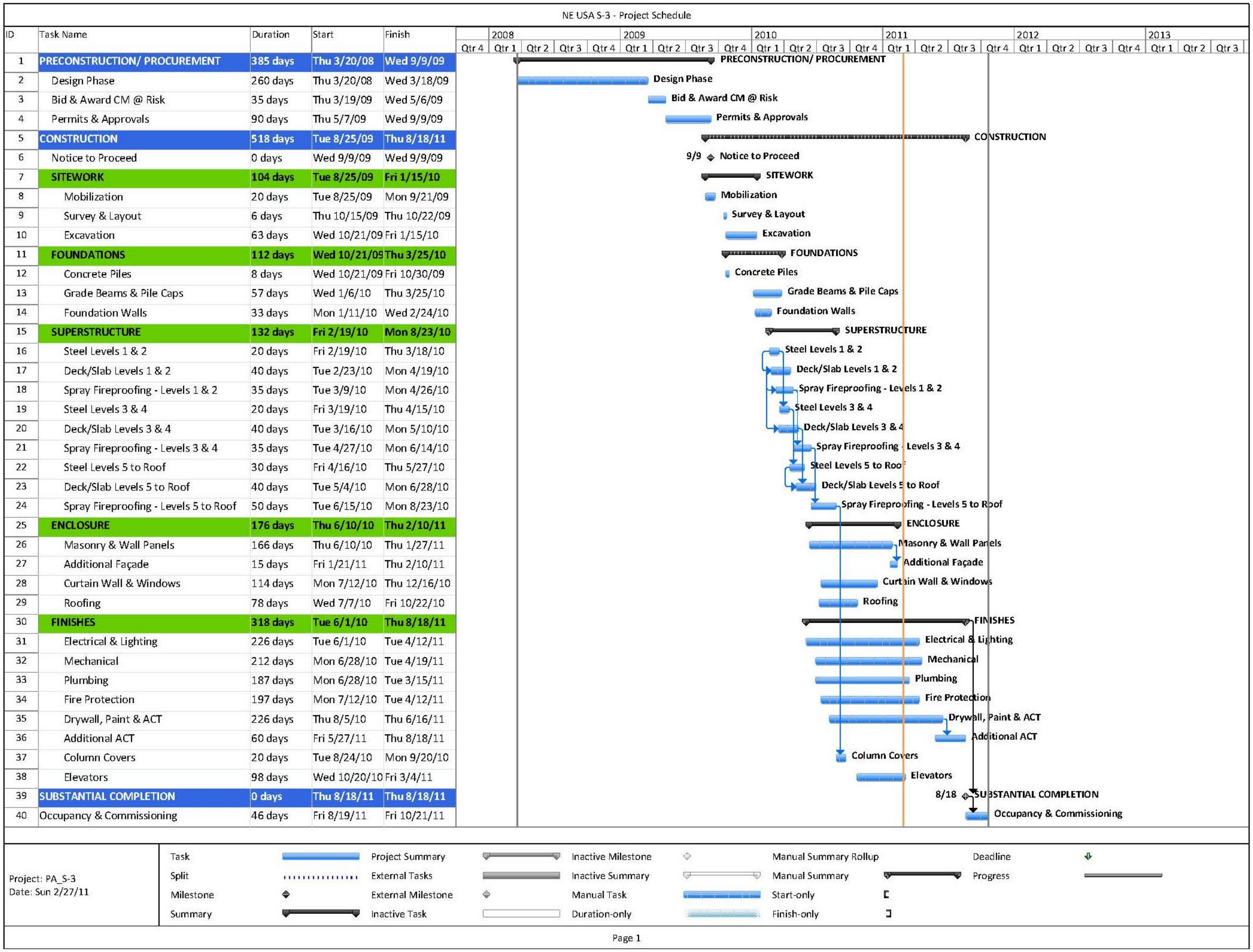
NOTES:

[SF] - Cost taken from RS Means Square Foot Estimate

[DET] - Cost taken from RS Means Detailed Estimate

[ORIG] - Cost taken from Original Building Cost Data, as provided by Turner Construction

NE USA S-3 Structure



NE USA S-3 - Project Schedule

ID	Task Name	Duration	Start	Finish
1	PRECONSTRUCTION/ PROCUREMENT	385 days	Thu 3/20/08	Wed 9/9/09
2	Design Phase	260 days	Thu 3/20/08	Wed 3/18/09
3	Bid & Award CM @ Risk	35 days	Thu 3/19/09	Wed 5/6/09
4	Permits & Approvals	90 days	Thu 5/7/09	Wed 9/9/09
5	CONSTRUCTION	518 days	Tue 8/25/09	Thu 8/18/11
6	Notice to Proceed	0 days	Wed 9/9/09	Wed 9/9/09
7	SITEWORK	104 days	Tue 8/25/09	Fri 1/15/10
8	Mobilization	20 days	Tue 8/25/09	Mon 9/21/09
9	Survey & Layout	6 days	Thu 10/15/09	Thu 10/22/09
10	Excavation	63 days	Wed 10/21/09	Fri 1/15/10
11	FOUNDATIONS	112 days	Wed 10/21/09	Thu 3/25/10
12	Concrete Piles	8 days	Wed 10/21/09	Fri 10/30/09
13	Grade Beams & Pile Caps	57 days	Wed 1/6/10	Thu 3/25/10
14	Foundation Walls	33 days	Mon 1/11/10	Wed 2/24/10
15	SUPERSTRUCTURE	132 days	Fri 2/19/10	Mon 8/23/10
16	Steel Levels 1 & 2	20 days	Fri 2/19/10	Thu 3/18/10
17	Deck/Slab Levels 1 & 2	40 days	Tue 2/23/10	Mon 4/19/10
18	Spray Fireproofing - Levels 1 & 2	35 days	Tue 3/9/10	Mon 4/26/10
19	Steel Levels 3 & 4	20 days	Fri 3/19/10	Thu 4/15/10
20	Deck/Slab Levels 3 & 4	40 days	Tue 3/16/10	Mon 5/10/10
21	Spray Fireproofing - Levels 3 & 4	35 days	Tue 4/27/10	Mon 6/14/10
22	Steel Levels 5 to Roof	30 days	Fri 4/16/10	Thu 5/27/10
23	Deck/Slab Levels 5 to Roof	40 days	Tue 5/4/10	Mon 6/28/10
24	Spray Fireproofing - Levels 5 to Roof	50 days	Tue 6/15/10	Mon 8/23/10
25	ENCLOSURE	176 days	Thu 6/10/10	Thu 2/10/11
26	Masonry & Wall Panels	166 days	Thu 6/10/10	Thu 1/27/11
27	Additional Façade	15 days	Fri 1/21/11	Thu 2/10/11
28	Curtain Wall & Windows	114 days	Mon 7/12/10	Thu 12/16/10
29	Roofing	78 days	Wed 7/7/10	Fri 10/22/10
30	FINISHES	318 days	Tue 6/1/10	Thu 8/18/11
31	Electrical & Lighting	226 days	Tue 6/1/10	Tue 4/12/11
32	Mechanical	212 days	Mon 6/28/10	Tue 4/19/11
33	Plumbing	187 days	Mon 6/28/10	Tue 3/15/11
34	Fire Protection	197 days	Mon 7/12/10	Tue 4/12/11
35	Drywall, Paint & ACT	226 days	Thu 8/5/10	Thu 6/16/11
36	Additional ACT	60 days	Fri 5/27/11	Thu 8/18/11
37	Column Covers	20 days	Tue 8/24/10	Mon 9/20/10
38	Elevators	98 days	Wed 10/20/10	Fri 3/4/11
39	SUBSTANTIAL COMPLETION	0 days	Thu 8/18/11	Thu 8/18/11
40	Occupancy & Commissioning	46 days	Fri 8/19/11	Fri 10/21/11

Project: PA_S-3
Date: Sun 2/27/11

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CA S-3 Structure

CA S-3 - Project Schedule

ID	Task Name	Duration	Start	Finish
1	**PRECONSTRUCTION/ PROCUREMENT**	**385 days**	**Thu 3/20/08**	**Wed 9/9/09**
2	Design Phase	260 days	Thu 3/20/08	Wed 3/18/09
3	Bid & Award CM @ Risk	35 days	Thu 3/19/09	Wed 5/6/09
4	Permits & Approvals	90 days	Thu 5/7/09	Wed 9/9/09
5	**CONSTRUCTION**	**518 days**	**Tue 8/25/09**	**Thu 8/18/11**
6	Notice to Proceed	0 days	Wed 9/9/09	Wed 9/9/09
7	**SITEWORK**	**104 days**	**Tue 8/25/09**	**Fri 1/15/10**
8	Mobilization	20 days	Tue 8/25/09	Mon 9/21/09
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21	Spray Fireproofing - Levels 3 & 4	35 days	Tue 4/27/10	Mon 6/14/10
22	Steel Levels 5 to Roof	35 days	Fri 4/30/10	Thu 6/17/10
23	Deck/Slab Levels 5 to Roof	40 days	Tue 5/4/10	Mon 6/28/10
24	Spray Fireproofing - Levels 5 to Roof	50 days	Tue 6/15/10	Mon 8/23/10
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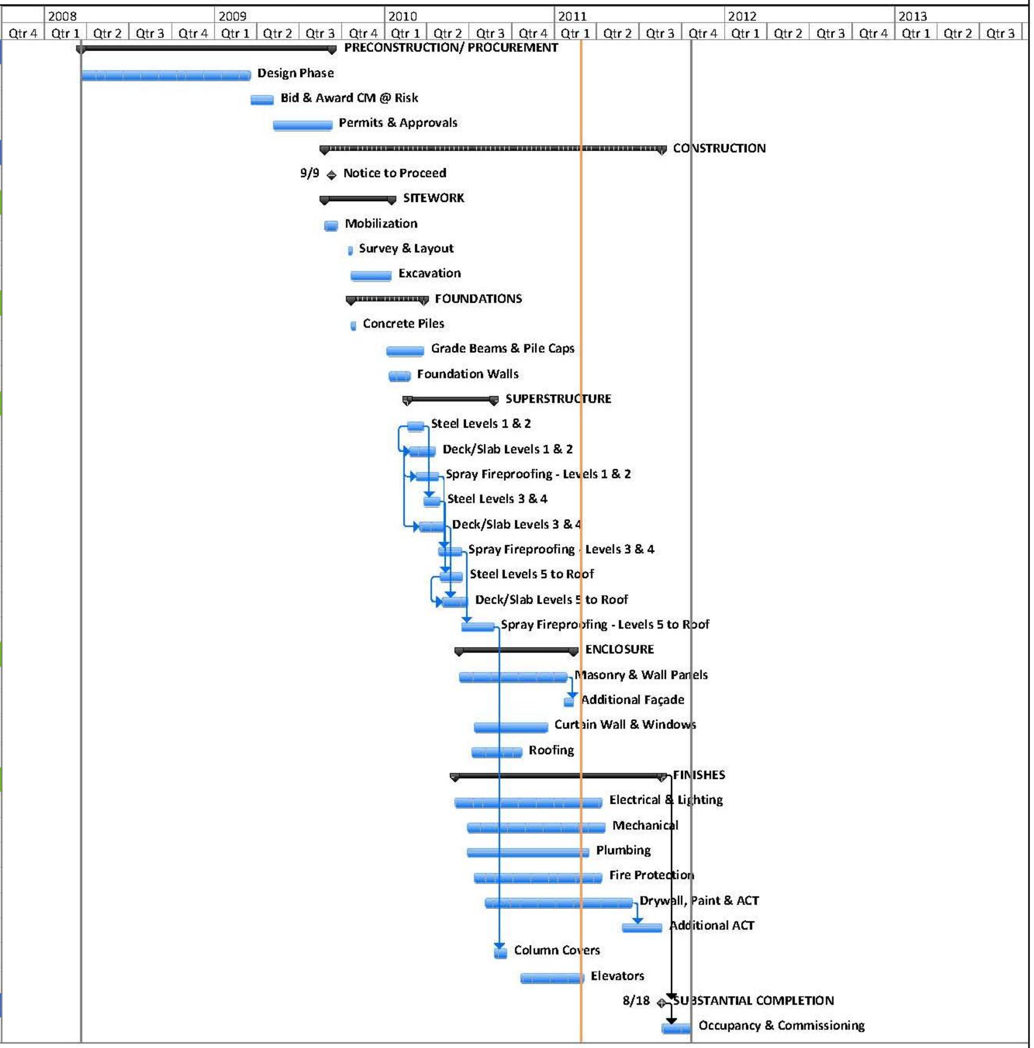
38	Elevators	98 days	Wed 10/20/10	Fri 3/4/11
39	**SUBSTANTIAL COMPLETION**	**0 days**	**Thu 8/18/11**	**Thu 8/18/11**
40	Occupancy & Commissioning	46 days	Fri 8/19/11	Fri 10/21/11



Project: CA_S-3
Date: Sun 2/27/11

Task
Split
Milestone ◆
Summary
Project Summary
External Tasks
External Milestone ◆
Inactive Task
Inactive Milestone ◇
Inactive Summary
Manual Task
Duration-only
Manual Summary Rollup
Manual Summary
Start-only [
Finish-only]
Deadline ⇩
Progress

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CA S-3 Quantity, Duration, and Cost Summary

Description	RS Means Unit Cost/Duration	Quantities/Duration/Cost per Level							
		2nd Level	3rd Level	4th Level	5th Level	Penthouse Level	Atrium Roof Level	Chiller Roof Level	AHU Roof Level
Gravity Beams									
Quantities (LF)		2,489	2,489	2,489	2,804	1,986	1,127	262	928
Duration (days)	900 LF/day	2.77	2.77	2.77	3.12	2.21	1.25	0.29	1.03
Cost	60 $/LF	$149,353.20	$149,353.20	$149,353.20	$168,257.40	$119,134.80	$67,620.00	$15,720.00	$55,686.00
Gravity Columns									
Quantities (LF)		273	252	252	252	233.28	116	87.75	110.5
Duration (days)	900 LF/day	0.30	0.28	0.28	0.28	0.26	0.13	0.10	0.12
Cost	100 $/LF	$27,306.00	$25,200.00	$25,200.00	$25,200.00	$23,328.00	$11,600.00	$8,775.00	$11,050.00
Lateral Beams									
Quantities (LF)		904	904	904	904	422	307	136	259
Duration (days)	800 LF/day	1.13	1.13	1.13	1.13	0.53	0.38	0.17	0.32
Cost	160 $/LF	$144,592.00	$144,592.00	$144,592.00	$144,592.00	$67,520.00	$49,168.00	$21,760.00	$41,440.00
Lateral Columns									
Quantities (LF)		622	574	574	574	516	244	130	111
Duration (days)	600 LF/day	1.04	0.96	0.96	0.96	0.86	0.41	0.22	0.18
Cost	300 $/LF	$186,600.00	$172,200.00	$172,200.00	$172,200.00	$154,710.00	$73,110.00	$38,940.00	$33,150.00
Welding									
Quantities (LF)		300	300	300	300	132	168	36	96
Duration (days)	12 LF/day	25.00	25.00	25.00	25.00	11.00	14.00	3.00	8.00
Cost	72.5 $/LF	$21,750.00	$21,750.00	$21,750.00	$21,750.00	$9,570.00	$12,180.00	$2,610.00	$6,960.00
Spray Fireproofing									
Quantities (SF)		36,183	36,183	36,183	37,972	23,558	15,813	4,139	18,889
Duration (days)	1250 SF/day	28.95	28.95	28.95	30.38	18.85	12.65	3.31	15.11
Cost	1.90 $/SF	$68,747.13	$68,747.13	$68,747.13	$72,147.29	$44,760.62	$30,045.38	$7,864.29	$35,889.82
Shear Studs									
Quantities (#)		1,660	1,660	1,673	2,139	1,513	0	149	0
Duration (days)	900 #/day	1.84	1.84	1.86	2.38	1.68	0.00	0.17	0.00
Cost	2.80 $/stud	$4,648.00	$4,648.00	$4,684.40	$5,989.20	$4,236.40	$0.00	$417.20	$0.00
Composite Deck									
Quantities (SF)		20,102	20,102	20,102	21,096	13,088	0	2,300	0
Duration (days)	2850 SF/day	7.05	7.05	7.05	7.40	4.59	0.00	0.81	0.00
Cost	3.21 $/SF	$64,525.82	$64,525.82	$64,525.82	$67,717.20	$42,012.16	$0.00	$7,381.40	$0.00
Roof Deck									
Quantities (SF)		0	0	0	0	0	8,785	0	10,494
Duration (days)	3600 SF/day	0.00	0.00	0.00	0.00	0.00	2.44	0.00	2.92
Cost	3.11 $/SF	$0.00	$0.00	$0.00	$0.00	$0.00	$27,321.97	$0.00	$32,636.65
Rebar									
Quantities (tons)		8.5	8.5	8.5	9.0	7.4	0.0	1.5	0.0
Duration (days)	2.9 tons/day	2.93	2.93	2.93	3.10	2.55	0.00	0.52	0.00
Cost	1900 $/tons	$16,150.00	$16,150.00	$16,150.00	$17,100.00	$14,060.00	$0.00	$2,850.00	$0.00
Slab									
Quantities (CY)		309.56	309.56	309.56	324.87	201.55	0.00	35.41	0.00
Duration (days)	110 CY/day	2.81	2.81	2.81	2.95	1.83	0.00	0.32	0.00
Cost	163 $/CY	$50,458.79	$50,458.79	$50,458.79	$52,954.43	$32,853.25	$0.00	$5,772.20	$0.00
ACT Ceiling									
Quantities (SF)		11,340	11,340	11,340	6,286	0	0	0	0
Duration (days)	380 SF/day	29.84	29.84	29.84	16.54	0.00	0.00	0.00	0.00
Cost	2.75 $/SF	$31,185.00	$31,185.00	$31,185.00	$17,286.50	$0.00	$0.00	$0.00	$0.00
Additional Façade									
Quantities (SF)		666.67	666.67	666.67	666.67	350.00	583.33	250.00	300.00
Duration (days)	300 SF/day	2.22	2.22	2.22	2.22	1.17	1.94	0.83	1.00
Cost	10 $/SF	$6,666.67	$6,666.67	$6,666.67	$6,666.67	$3,500.00	$5,833.33	$2,500.00	$3,000.00
Column Covers									
Quantities (LF)		895	895	826	826	826	749	360	218
Duration (days)	600 LF/day	1.49	1.49	1.38	1.38	1.38	1.25	0.60	0.36
Cost	60 $/LF	$53,703.60	$53,703.60	$49,560.00	$49,560.00	$49,560.00	$44,938.80	$21,582.00	$13,053.00
Steel Totals									
Quantities (LF)		4,288	4,219	4,219	4,534	3,157	1,794	616	1,408
Duration (days)		5.24	5.13	5.13	5.48	3.85	2.17	0.77	1.66
Cost		$507,851.20	$491,345.20	$491,345.20	$510,249.40	$364,692.80	$201,498.00	$85,195.00	$141,326.00
Deck/Slab Totals									
Quantities (SF)		20,102	20,102	20,102	21,096	13,088	8,785	2,300	10,494
Duration (days)		12.80	12.80	12.80	13.46	8.98	2.44	1.65	2.92
Cost		$131,134.60	$131,134.60	$131,134.60	$137,771.62	$88,925.41	$27,321.97	$16,003.60	$32,636.65
Misc. Structural Totals (Shear Studs , Welding & Fireproofing)									
Duration (days)		55.79	55.79	55.81	57.75	31.53	26.65	6.48	23.11
Cost		$95,145.13	$95,145.13	$95,181.53	$99,886.49	$58,567.02	$42,225.38	$10,891.49	$42,849.82
Misc. Architectural Totals									
Duration (days)		33.56	33.56	33.44	20.14	2.54	3.19	1.43	1.36
Cost		$91,555.27	$91,555.27	$87,411.67	$73,513.17	$53,060.00	$50,772.13	$24,082.00	$16,053.00

CA S-3 - Final Cost Estimate

Assembly	% of Total	Cost per SF	Total Cost
A Substructure [SF]	5.64%	$15.44	$2,130,949.30
B Shell			
B10 Super Structure [DET]	10.67%	$29.20	$4,029,457.85
B20 Exterior Enclosure [SF+DET]	4.04%	$11.05	$1,524,807.85
B30 Roofing [SF]	2.21%	$6.06	$835,666.39
C Interiors [SF+DET]	13.47%	$36.84	$5,084,450.98
D Services			
D10 Conveying [SF]	0.79%	$2.17	$300,000.00
D20 Plumbing [ORIG]	37.08%	$101.45	$14,000,000.00
D30 HVAC [ORIG]			
D40 Fire Protection [SF]	1.22%	$3.33	$459,616.52
D50 Electrical [ORIG]	15.89%	$43.48	$6,000,000.00
E Equipment & Furnishings [SF]	0.55%	$1.51	$208,916.60
F Special Construction [SF]	0.00%	$0.00	$0.00
Subtotal	91.58%	$250.54	$34,573,865.48
General Conditions [DET]	8.42%	$23.05	$3,180,777.39
Subtotal with GC's	100.00%	$273.58	$37,754,642.87
Location Multiplier			1.00
Time Multiplier			1.00
Total Cost			$37,754,643.87

NOTES:

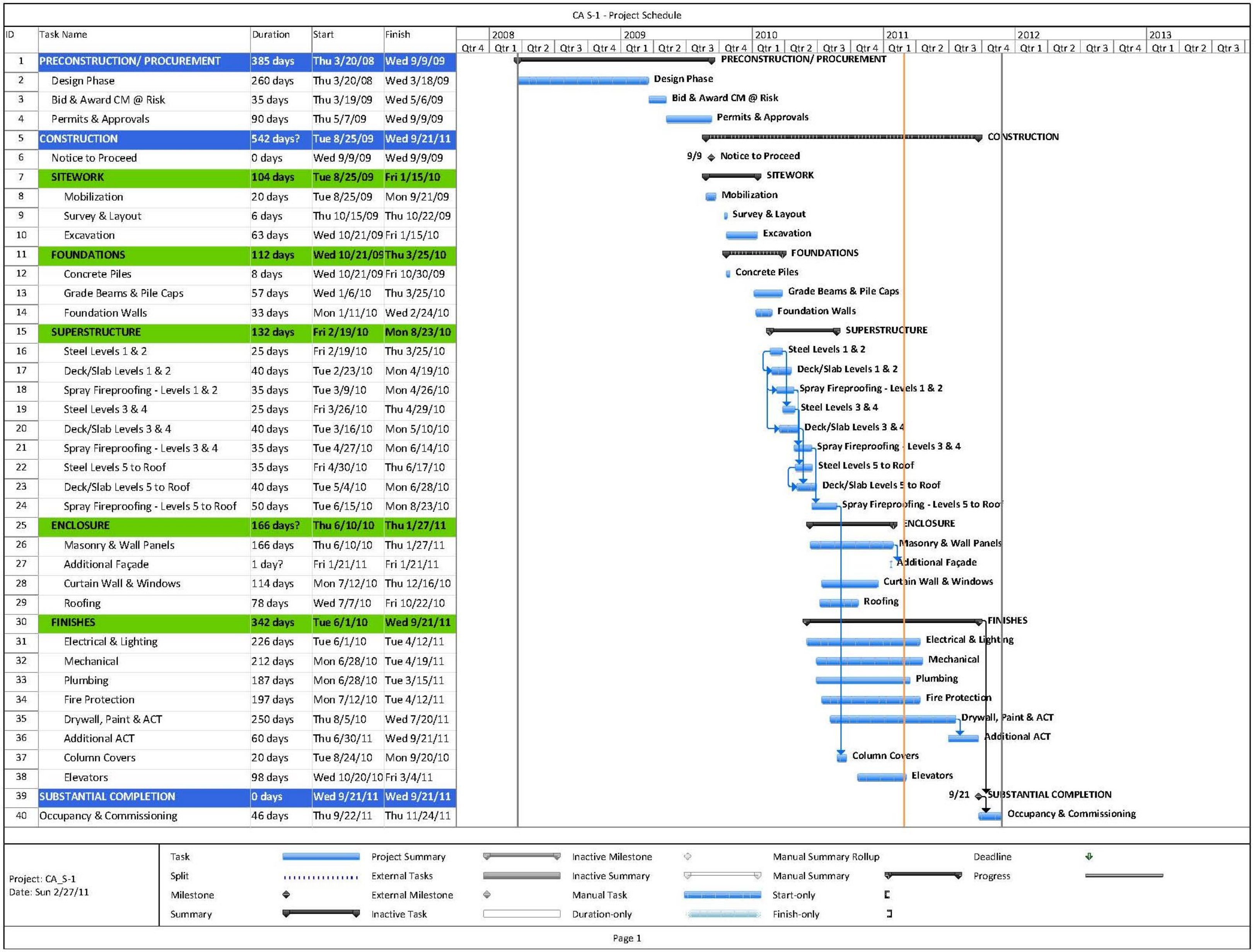
[SF] - Cost taken from RS Means Square Foot Estimate

[DET] - Cost taken from RS Means Detailed Estimate

[SF+DET] - Cost from RS Means Detailed Estimate added to RS Means Square Foot Estimate

[ORIG] - Cost taken from Original Building Cost Data, as provided by Turner Construction

CA S-1 Structure



CA S-1 - Project Schedule

ID	Task Name	Duration	Start	Finish
1	PRECONSTRUCTION/ PROCUREMENT	385 days	Thu 3/20/08	Wed 9/9/09
2	Design Phase	260 days	Thu 3/20/08	Wed 3/18/09
3	Bid & Award CM @ Risk	35 days	Thu 3/19/09	Wed 5/6/09
4	Permits & Approvals	90 days	Thu 5/7/09	Wed 9/9/09
5	CONSTRUCTION	542 days?	Tue 8/25/09	Wed 9/21/11
6	Notice to Proceed	0 days	Wed 9/9/09	Wed 9/9/09
7	SITEWORK	104 days	Tue 8/25/09	Fri 1/15/10
8	Mobilization	20 days	Tue 8/25/09	Mon 9/21/09
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11	FOUNDATIONS	112 days	Wed 10/21/09	Thu 3/25/10
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16	Steel Levels 1 & 2	25 days	Fri 2/19/10	Thu 3/25/10
17	Deck/Slab Levels 1 & 2	40 days	Tue 2/23/10	Mon 4/19/10
18	Spray Fireproofing - Levels 1 & 2	35 days	Tue 3/9/10	Mon 4/26/10
19	Steel Levels 3 & 4	25 days	Fri 3/26/10	Thu 4/29/10
20	Deck/Slab Levels 3 & 4	40 days	Tue 3/16/10	Mon 5/10/10
21	Spray Fireproofing - Levels 3 & 4	35 days	Tue 4/27/10	Mon 6/14/10
22	Steel Levels 5 to Roof	35 days	Fri 4/30/10	Thu 6/17/10
23	Deck/Slab Levels 5 to Roof	40 days	Tue 5/4/10	Mon 6/28/10
24	Spray Fireproofing - Levels 5 to Roof	50 days	Tue 6/15/10	Mon 8/23/10
25	ENCLOSURE	166 days?	Thu 6/10/10	Thu 1/27/11
26	Masonry & Wall Panels	166 days	Thu 6/10/10	Thu 1/27/11
27	Additional Façade	1 day?	Fri 1/21/11	Fri 1/21/11
28	Curtain Wall & Windows	114 days	Mon 7/12/10	Thu 12/16/10
29	Roofing	78 days	Wed 7/7/10	Fri 10/22/10
30	FINISHES	342 days	Tue 6/1/10	Wed 9/21/11
31	Electrical & Lighting	226 days	Tue 6/1/10	Tue 4/12/11
32	Mechanical	212 days	Mon 6/28/10	Tue 4/19/11
33	Plumbing	187 days	Mon 6/28/10	Tue 3/15/11
34	Fire Protection	197 days	Mon 7/12/10	Tue 4/12/11
35	Drywall, Paint & ACT	250 days	Thu 8/5/10	Wed 7/20/11
36	Additional ACT	60 days	Thu 6/30/11	Wed 9/21/11
37	Column Covers	20 days	Tue 8/24/10	Mon 9/20/10
38	Elevators	98 days	Wed 10/20/10	Fri 3/4/11
39	SUBSTANTIAL COMPLETION	0 days	Wed 9/21/11	Wed 9/21/11
40	Occupancy & Commissioning	46 days	Thu 9/22/11	Thu 11/24/11

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CA S-1 Quantity, Duration, and Cost Summary

Description	RS Means Unit Cost/Duration	Quantities/Duration/Cost per Level							
		2nd Level	3rd Level	4th Level	5th Level	Penthouse Level	Atrium Roof Level	Chiller Roof Level	AHU Roof Level
Gravity Beams									
Quantities (LF)		2,489	2,489	2,489	2,804	1,986	1,127	262	928
Duration (days)	900 LF/day	2.77	2.77	2.77	3.12	2.21	1.25	0.29	1.03
Cost	60 $/LF	$149,353.20	$149,353.20	$149,353.20	$168,257.40	$119,134.80	$67,620.00	$15,720.00	$55,686.00
Gravity Columns									
Quantities (LF)		273	252	252	252	233.28	116	87.75	110.5
Duration (days)	900 LF/day	0.30	0.28	0.28	0.28	0.26	0.13	0.10	0.12
Cost	100 $/LF	$27,306.00	$25,200.00	$25,200.00	$25,200.00	$23,328.00	$11,600.00	$8,775.00	$11,050.00
Lateral Beams									
Quantities (LF)		904	904	904	904	422	307	136	259
Duration (days)	700 LF/day	1.29	1.29	1.29	1.29	0.60	0.44	0.19	0.37
Cost	400 $/LF	$361,480.00	$361,480.00	$361,480.00	$361,480.00	$168,800.00	$122,920.00	$54,400.00	$103,600.00
Lateral Columns									
Quantities (LF)		622	574	574	574	516	244	130	111
Duration (days)	700 LF/day	0.89	0.82	0.82	0.82	0.74	0.35	0.19	0.16
Cost	600 $/LF	$373,200.00	$344,400.00	$344,400.00	$344,400.00	$309,420.00	$146,220.00	$77,880.00	$66,300.00
Welding									
Quantities (LF)		375	375	375	375	165	210	45	120
Duration (days)	12 LF/day	31.25	31.25	31.25	31.25	13.75	17.50	3.75	10.00
Cost	72.5 $/LF	$27,187.50	$27,187.50	$27,187.50	$27,187.50	$11,962.50	$15,225.00	$3,262.50	$8,700.00
Spray Fireproofing									
Quantities (SF)		36,183	36,183	36,183	37,972	23,558	15,813	4,139	18,889
Duration (days)	1250 SF/day	28.95	28.95	28.95	30.38	18.85	12.65	3.31	15.11
Cost	1.90 $/SF	$68,747.13	$68,747.13	$68,747.13	$72,147.29	$44,760.62	$30,045.38	$7,864.29	$35,889.82
Shear Studs									
Quantities (#)		1,660	1,660	1,673	2,139	1,513	0	149	0
Duration (days)	900 #/day	1.84	1.84	1.86	2.38	1.68	0.00	0.17	0.00
Cost	2.80 $/stud	$4,648.00	$4,648.00	$4,684.40	$5,989.20	$4,236.40	$0.00	$417.20	$0.00
Composite Deck									
Quantities (SF)		20,102	20,102	20,102	21,096	13,088	0	2,300	0
Duration (days)	2850 SF/day	7.05	7.05	7.05	7.40	4.59	0.00	0.81	0.00
Cost	3.21 $/SF	$64,525.82	$64,525.82	$64,525.82	$67,717.20	$42,012.16	$0.00	$7,381.40	$0.00
Roof Deck									
Quantities (SF)		0	0	0	0	0	8,785	0	10,494
Duration (days)	3600 SF/day	0.00	0.00	0.00	0.00	0.00	2.44	0.00	2.92
Cost	3.11 $/SF	$0.00	$0.00	$0.00	$0.00	$0.00	$27,321.97	$0.00	$32,636.65
Rebar									
Quantities (tons)		8.5	8.5	8.5	9.0	7.4	0.0	1.5	0.0
Duration (days)	2.9 tons/day	2.93	2.93	2.93	3.10	2.55	0.00	0.52	0.00
Cost	1900 $/tons	$16,150.00	$16,150.00	$16,150.00	$17,100.00	$14,060.00	$0.00	$2,850.00	$0.00
Slab									
Quantities (CY)		309.56	309.56	309.56	324.87	201.55	0.00	35.41	0.00
Duration (days)	110 CY/day	2.81	2.81	2.81	2.95	1.83	0.00	0.32	0.00
Cost	163 $/CY	$50,458.79	$50,458.79	$50,458.79	$52,954.43	$32,853.25	$0.00	$5,772.20	$0.00
ACT Ceiling									
Quantities (SF)		11,340	11,340	11,340	6,286	0	0	0	0
Duration (days)	380 SF/day	29.84	29.84	29.84	16.54	0.00	0.00	0.00	0.00
Cost	2.75 $/SF	$31,185.00	$31,185.00	$31,185.00	$17,286.50	$0.00	$0.00	$0.00	$0.00
Additional Façade									
Quantities (SF)		666.67	666.67	666.67	666.67	350.00	583.33	250.00	300.00
Duration (days)	300 SF/day	2.22	2.22	2.22	2.22	1.17	1.94	0.83	1.00
Cost	10 $/SF	$6,666.67	$6,666.67	$6,666.67	$6,666.67	$3,500.00	$5,833.33	$2,500.00	$3,000.00
Column Covers									
Quantities (LF)		895	895.06	826	826	826	748.98	359.7	217.55
Duration (days)	600 LF/day	1.49	1.49	1.38	1.38	1.38	1.25	0.60	0.36
Cost	60 $/LF	$53,703.60	$53,703.60	$49,560.00	$49,560.00	$49,560.00	$44,938.80	$21,582.00	$13,053.00
Steel Totals									
Quantities (LF)		4,288	4,219	4,219	4,534	3,157	1,794	616	1,408
Duration (days)		5.25	5.16	5.16	5.51	3.80	2.17	0.77	1.68
Cost		$911,339.20	$880,433.20	$880,433.20	$899,337.40	$620,682.80	$348,360.00	$156,775.00	$236,636.00
Deck/Slab Totals									
Quantities (SF)		20,102	20,102	20,102	21,096	13,088	8,785	2,300	10,494
Duration (days)		12.80	12.80	12.80	13.46	8.98	2.44	1.65	2.92
Cost		$131,134.60	$131,134.60	$131,134.60	$137,771.62	$88,925.41	$27,321.97	$16,003.60	$32,636.65
Misc. Structural Totals (Shear Studs , Welding & Fireproofing)									
Duration (days)		62.04	62.04	62.06	64.00	34.28	30.15	7.23	25.11
Cost		$100,582.63	$100,582.63	$100,619.03	$105,323.99	$60,959.52	$45,270.38	$11,543.99	$44,589.82
Misc. Architectural Totals									
Duration (days)		33.56	33.56	33.44	20.14	2.54	3.19	1.43	1.36
Cost		$91,555.27	$91,555.27	$87,411.67	$73,513.17	$53,060.00	$50,772.13	$24,082.00	$16,053.00

CA S-1 - Final Cost Estimate

Assembly	% of Total	Cost per SF	Total Cost
A Substructure ^{SF}	5.32%	$15.44	$2,130,949.30
B Shell			
B10 Super Structure ^{DET}	15.48%	$44.92	$6,199,531.85
B20 Exterior Enclosure $^{SF+DET}$	3.81%	$11.05	$1,524,807.85
B30 Roofing ^{SF}	2.09%	$6.06	$835,666.39
C Interiors $^{SF+DET}$	12.69%	$36.84	$5,084,450.98
D Services			
D10 Conveying ^{SF}	0.75%	$2.17	$300,000.00
D20 Plumbing ^{ORIG}	34.95%	$101.45	$14,000,000.00
D30 HVAC ^{ORIG}			
D40 Fire Protection ^{SF}	1.15%	$3.33	$459,616.52
D50 Electrical ^{ORIG}	14.98%	$43.48	$6,000,000.00
E Equipment & Furnishings ^{SF}	0.52%	$1.51	$208,916.60
F Special Construction ^{SF}	0.00%	$0.00	$0.00
Subtotal	91.73%	$266.26	$36,743,939.48
General Conditions ^{DET}	8.27%	$24.01	$3,313,309.78
Subtotal with GC's	100.00%	$290.27	$40,057,249.26
Location Multiplier			1.00
Time Multiplier			1.00
Total Cost			$40,057,250.26

NOTES:

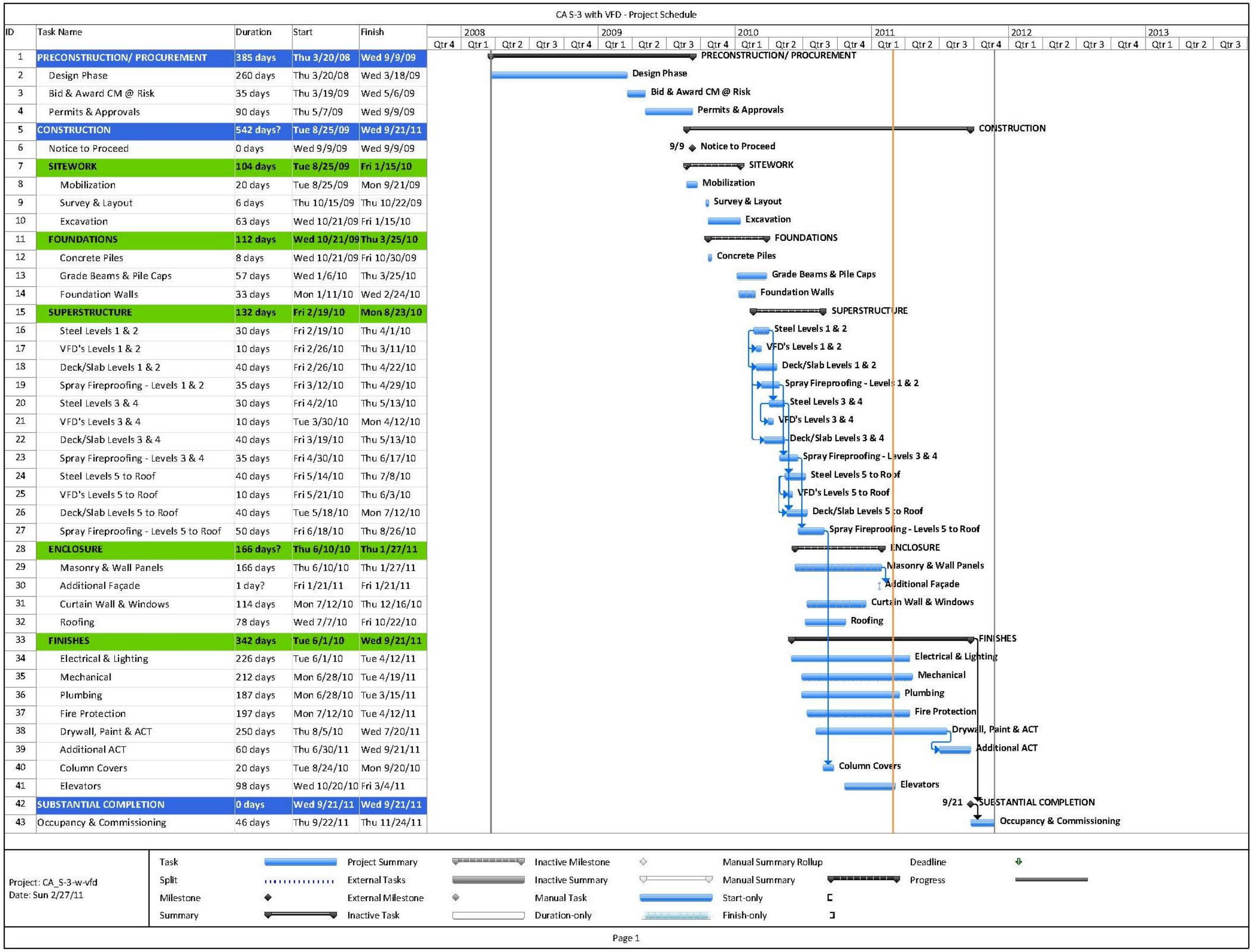
^{SF} - Cost taken from RS Means Square Foot Estimate

^{DET} - Cost taken from RS Means Detailed Estimate

$^{SF+DET}$ - Cost from RS Means Detailed Estimate added to RS Means Square Foot Estimate

^{ORIG} - Cost taken from Original Building Cost Data, as provided by Turner Construction

CA S-3 with VFD Structure



CA S-3 with VFD - Project Schedule

ID	Task Name	Duration	Start	Finish
1	PRECONSTRUCTION/ PROCUREMENT	385 days	Thu 3/20/08	Wed 9/9/09
2	Design Phase	260 days	Thu 3/20/08	Wed 3/18/09
3	Bid & Award CM @ Risk	35 days	Thu 3/19/09	Wed 5/6/09
4	Permits & Approvals	90 days	Thu 5/7/09	Wed 9/9/09
5	CONSTRUCTION	542 days?	Tue 8/25/09	Wed 9/21/11
6	Notice to Proceed	0 days	Wed 9/9/09	Wed 9/9/09
7	SITEWORK	104 days	Tue 8/25/09	Fri 1/15/10
8	Mobilization	20 days	Tue 8/25/09	Mon 9/21/09
9	Survey & Layout	6 days	Thu 10/15/09	Thu 10/22/09
10	Excavation	63 days	Wed 10/21/09	Fri 1/15/10
11	FOUNDATIONS	112 days	Wed 10/21/09	Thu 3/25/10
12	Concrete Piles	8 days	Wed 10/21/09	Fri 10/30/09
13	Grade Beams & Pile Caps	57 days	Wed 1/6/10	Thu 3/25/10
14	Foundation Walls	33 days	Mon 1/11/10	Wed 2/24/10
15	SUPERSTRUCTURE	132 days	Fri 2/19/10	Mon 8/23/10
16	Steel Levels 1 & 2	30 days	Fri 2/19/10	Thu 4/1/10
17	VFD's Levels 1 & 2	10 days	Fri 2/26/10	Thu 3/11/10
18	Deck/Slab Levels 1 & 2	40 days	Fri 2/26/10	Thu 4/22/10
19	Spray Fireproofing - Levels 1 & 2	35 days	Fri 3/12/10	Thu 4/29/10
20	Steel Levels 3 & 4	30 days	Fri 4/2/10	Thu 5/13/10
21	VFD's Levels 3 & 4	10 days	Tue 3/30/10	Mon 4/12/10
22	Deck/Slab Levels 3 & 4	40 days	Fri 3/19/10	Thu 5/13/10
23	Spray Fireproofing - Levels 3 & 4	35 days	Fri 4/30/10	Thu 6/17/10
24	Steel Levels 5 to Roof	40 days	Fri 5/14/10	Thu 7/8/10
25	VFD's Levels 5 to Roof	10 days	Fri 5/21/10	Thu 6/3/10
26	Deck/Slab Levels 5 to Roof	40 days	Tue 5/18/10	Mon 7/12/10
27	Spray Fireproofing - Levels 5 to Roof	50 days	Fri 6/18/10	Thu 8/26/10
28	ENCLOSURE	166 days?	Thu 6/10/10	Thu 1/27/11
29	Masonry & Wall Panels	166 days	Thu 6/10/10	Thu 1/27/11
30	Additional Façade	1 day?	Fri 1/21/11	Fri 1/21/11
31	Curtain Wall & Windows	114 days	Mon 7/12/10	Thu 12/16/10
32	Roofing	78 days	Wed 7/7/10	Fri 10/22/10
33	FINISHES	342 days	Tue 6/1/10	Wed 9/21/11
34	Electrical & Lighting	226 days	Tue 6/1/10	Tue 4/12/11
35	Mechanical	212 days	Mon 6/28/10	Tue 4/19/11
36	Plumbing	187 days	Mon 6/28/10	Tue 3/15/11
37	Fire Protection	197 days	Mon 7/12/10	Tue 4/12/11
38	Drywall, Paint & ACT	250 days	Thu 8/5/10	Wed 7/20/11
39	Additional ACT	60 days	Thu 6/30/11	Wed 9/21/11
40	Column Covers	20 days	Tue 8/24/10	Mon 9/20/10
41	Elevators	98 days	Wed 10/20/10	Fri 3/4/11
42	SUBSTANTIAL COMPLETION	0 days	Wed 9/21/11	Wed 9/21/11
43	Occupancy & Commissioning	46 days	Thu 9/22/11	Thu 11/24/11

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CA S-3 with VFD Quantity, Duration, and Cost Summary

Description	RS Means Unit Cost/Duration	Quantities/Duration/Cost per Level							
		2nd Level	3rd Level	4th Level	5th Level	Penthouse Level	Atrium Roof Level	Chiller Roof Level	AHU Roof Level
Gravity Beams									
Quantities (LF)		2,489	2,489	2,489	2,804	1,986	1,127	262	928
Duration (days)	900 LF/day	2.77	2.77	2.77	3.12	2.21	1.25	0.29	1.03
Cost	60 $/LF	$149,353.20	$149,353.20	$149,353.20	$168,257.40	$119,134.80	$67,620.00	$15,720.00	$55,686.00
Gravity Columns									
Quantities (LF)		273	252	252	252	233.28	116	87.75	110.5
Duration (days)	900 LF/day	0.30	0.28	0.28	0.28	0.26	0.13	0.10	0.12
Cost	100 $/LF	$27,306.00	$25,200.00	$25,200.00	$25,200.00	$23,328.00	$11,600.00	$8,775.00	$11,050.00
Lateral Beams									
Quantities (LF)		904	904	904	904	422	307	136	259
Duration (days)	800 LF/day	1.13	1.13	1.13	1.13	0.53	0.38	0.17	0.32
Cost	160 $/LF	$144,592.00	$144,592.00	$144,592.00	$144,592.00	$67,520.00	$49,168.00	$21,760.00	$41,440.00
Lateral Columns									
Quantities (LF)		622	574	574	574	516	244	130	111
Duration (days)	600 LF/day	1.04	0.96	0.96	0.96	0.86	0.41	0.22	0.18
Cost	300 $/LF	$186,600.00	$172,200.00	$172,200.00	$172,200.00	$154,710.00	$73,110.00	$38,940.00	$33,150.00
VFD Braces									
Quantities (LF)		381.80	363.40	363.40	363.40	216.60	152.80	81.00	196.00
Duration (days)	700 LF/day	0.55	0.52	0.52	0.52	0.31	0.22	0.12	0.28
Cost	150 $/LF	$57,270.00	$54,510.00	$54,510.00	$54,510.00	$32,490.00	$22,920.00	$12,150.00	$29,400.00
Viscous Fluid Dampers									
Quantities (#)		20	20	20	20	12	8	4	8
Duration (days)	4 #/day	5.00	5.00	5.00	5.00	3.00	2.00	1.00	2.00
Cost	4000 $/damper	$80,000.00	$80,000.00	$80,000.00	$80,000.00	$48,000.00	$32,000.00	$16,000.00	$32,000.00
Welding									
Quantities (LF)		300	300	300	300	132	168	36	96
Duration (days)	12 LF/day	25.00	25.00	25.00	25.00	11.00	14.00	3.00	8.00
Cost	72.5 $/LF	$21,750.00	$21,750.00	$21,750.00	$21,750.00	$9,570.00	$12,180.00	$2,610.00	$6,960.00
Spray Fireproofing									
Quantities (SF)		36,183	36,183	36,183	37,972	23,558	15,813	4,139	18,889
Duration (days)	1250 SF/day	28.95	28.95	28.95	30.38	18.85	12.65	3.31	15.11
Cost	1.90 $/SF	$68,747.13	$68,747.13	$68,747.13	$72,147.29	$44,760.62	$30,045.38	$7,864.29	$35,889.82
Shear Studs									
Quantities (#)		1,660	1,660	1,673	2,139	1,513	0	149	0
Duration (days)	900 #/day	1.84	1.84	1.86	2.38	1.68	0.00	0.17	0.00
Cost	2.80 $/stud	$4,648.00	$4,648.00	$4,684.40	$5,989.20	$4,236.40	$0.00	$417.20	$0.00
Composite Deck									
Quantities (SF)		20,102	20,102	20,102	21,096	13,088	0	2,300	0
Duration (days)	2850 SF/day	7.05	7.05	7.05	7.40	4.59	0.00	0.81	0.00
Cost	3.21 $/SF	$64,525.82	$64,525.82	$64,525.82	$67,717.20	$42,012.16	$0.00	$7,381.40	$0.00
Roof Deck									
Quantities (SF)		0	0	0	0	0	8,785	0	10,494
Duration (days)	3600 SF/day	0.00	0.00	0.00	0.00	0.00	2.44	0.00	2.92
Cost	3.11 $/SF	$0.00	$0.00	$0.00	$0.00	$0.00	$27,321.97	$0.00	$32,636.65
Rebar									
Quantities (tons)		8.5	8.5	8.5	9.0	7.4	0.0	1.5	0.0
Duration (days)	2.9 tons/day	2.93	2.93	2.93	3.10	2.55	0.00	0.52	0.00
Cost	1900 $/tons	$16,150.00	$16,150.00	$16,150.00	$17,100.00	$14,060.00	$0.00	$2,850.00	$0.00
Slab									
Quantities (CY)		309.56	309.56	309.56	324.87	201.55	0.00	35.41	0.00
Duration (days)	110 CY/day	2.81	2.81	2.81	2.95	1.83	0.00	0.32	0.00
Cost	163 $/CY	$50,458.79	$50,458.79	$50,458.79	$52,954.43	$32,853.25	$0.00	$5,772.20	$0.00
ACT Ceiling									
Quantities (SF)		11,340	11,340	11,340	6,286	0	0	0	0
Duration (days)	380 SF/day	29.84	29.84	29.84	16.54	0.00	0.00	0.00	0.00
Cost	2.75 $/SF	$31,185.00	$31,185.00	$31,185.00	$17,286.50	$0.00	$0.00	$0.00	$0.00
Additional Façade									
Quantities (SF)		666.67	666.67	666.67	666.67	350.00	583.33	250.00	300.00
Duration (days)	300 SF/day	2.22	2.22	2.22	2.22	1.17	1.94	0.83	1.00
Cost	10 $/SF	$6,666.67	$6,666.67	$6,666.67	$6,666.67	$3,500.00	$5,833.33	$2,500.00	$3,000.00
Column Covers									
Quantities (LF)		895	895	826	826	826	749	360	218
Duration (days)	600 LF/day	1.49	1.49	1.38	1.38	1.38	1.25	0.60	0.36
Cost	60 $/LF	$53,703.60	$53,703.60	$49,560.00	$49,560.00	$49,560.00	$44,938.80	$21,582.00	$13,053.00
Steel Totals									
Quantities (LF)		4,288	4,219	4,219	4,534	3,157	1,794	616	1,408
Duration (days)		5.24	5.13	5.13	5.48	3.85	2.17	0.77	1.66
Cost		$507,851.20	$491,345.20	$491,345.20	$510,249.40	$364,692.80	$201,498.00	$85,195.00	$141,326.00
Deck/Slab Totals									
Quantities (SF)		20,102	20,102	20,102	21,096	13,088	8,785	2,300	10,494
Duration (days)		12.80	12.80	12.80	13.46	8.98	2.44	1.65	2.92
Cost		$131,134.60	$131,134.60	$131,134.60	$137,771.62	$88,925.41	$27,321.97	$16,003.60	$32,636.65
Misc. Structural Totals (VFD's, Shear Studs, Welding & Fireproofing)									
Duration (days)		60.79	60.79	60.81	62.75	34.53	28.65	7.48	25.11
Cost		$175,145.13	$175,145.13	$175,181.53	$179,886.49	$106,567.02	$74,225.38	$26,891.49	$74,849.82
Misc. Architectural Totals									
Duration (days)		33.56	33.56	33.44	20.14	2.54	3.19	1.43	1.36
Cost		$91,555.27	$91,555.27	$87,411.67	$73,513.17	$53,060.00	$50,772.13	$24,082.00	$16,053.00

CA S-3 with VFD - Final Cost Estimate

Assembly	% of Total	Cost per SF	Total Cost
A Substructure [SF]	5.56%	$15.44	$2,130,949.30
B Shell			
B10 Super Structure [DET]	11.68%	$32.45	$4,477,457.85
B20 Exterior Enclosure [SF+DET]	3.98%	$11.05	$1,524,807.85
B30 Roofing [SF]	2.18%	$6.06	$835,666.39
C Interiors [SF+DET]	13.26%	$36.84	$5,084,450.98
D Services			
D10 Conveying [SF]	0.78%	$2.17	$300,000.00
D20 Plumbing [ORIG]	36.52%	$101.45	$14,000,000.00
D30 HVAC [ORIG]			
D40 Fire Protection [SF]	1.20%	$3.33	$459,616.52
D50 Electrical [ORIG]	15.65%	$43.48	$6,000,000.00
E Equipment & Furnishings [SF]	0.54%	$1.51	$208,916.60
F Special Construction [SF]	0.00%	$0.00	$0.00
Subtotal	91.36%	$253.78	$35,021,865.48
General Conditions [DET]	8.64%	$24.01	$3,313,309.78
Subtotal with GC's	100.00%	$277.79	$38,335,175.26
Location Multiplier			1.00
Time Multiplier			1.00
Total Cost			$38,335,176.26

NOTES:

[SF] - Cost taken from RS Means Square Foot Estimate

[DET] - Cost taken from RS Means Detailed Estimate

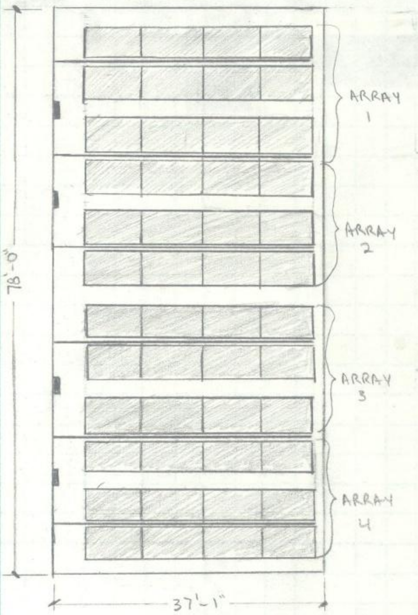
[SF+DET] - Cost from RS Means Detailed Estimate added to RS Means Square Foot Estimate

[ORIG] - Cost taken from Original Building Cost Data, as provided by Turner Construction

Appendix I: Sustainability Breadth

Photovolatic System

FINAL REPORT | PV DESIGN | pg 1 OF 2



▨ = SOLAR PANEL
▮ = INVERTER
— = CONDUIT FOR SERIES WIRING

LOCATION: OFFICE ROOF
↳ CHOSEN WITH SKETCH-UP SHADING ANALYSIS
↳ NO MECH. EQUIPMENT
↳ NO PEAK-HOUR SHADING

PANELS: LUMETA POWERPLY 400
↳ VERY EFFICIENT (MONOCRYSTAILINE Si)
↳ LAY FLAT & ADHERE (NO RACKS/ROOF PENETRATIONS)
↳ ~4 x 8'
↳ SEE APPENDIX I FOR DATA SHEET

LAYOUT: CHOSEN FOR ALLOWABLE SQUARE FOOTAGE & TO ALLOW ACCESS
↳ STRINGS OF 4 TO STAY UNDER 600V UL REQUIREMENT (5 max.)
↳ ARRAYS OF 3 STRINGS FOR EASE OF COMBINING
↳ FROM INSTALLATION INSTRUCTIONS FOR PANEL, EACH STRING HAS:

V_{mpp} = 328 V $V_{mp(NOCT)}$ = 264.5 V
I_{mpp} = 4.9 A
P_{max} = 1600 W $P_{max(NOCT)}$ = 1296.1 W
V_{oc} = 467 V I_{sc} = 5.33 A

↳ STRINGS PAIRED FOR EASE OF CONDUITING CABLES

INVERTER SIZING: USE ONE INVERTER PER ARRAY FOR EASE OF MONITORING/MAINTENANCE

SMA SUNNY BOY CHOSEN AS THE TYPE
↳ HAS BUILT-IN COMBINER & DC DISCONNECT
↳ SEE APPENDIX I FOR DATA SHEET

$$\left.\begin{array}{l} \text{ARRAY } V_{mpp} = 328\text{ V} \\ \text{ARRAY } I_{mpp} = 14.7\text{ A} \\ \text{ARRAY } I_{sc} = 16.0\text{ A} \\ \text{ARRAY } P_{max} = 3888\text{ W} \end{array}\right\}$$

CHOOSE SUNNY BOY 5000-US
↳ HAS 4 INPUTS @ DC DISCONNECT
↳ ALSO SERVES AS COMBINER

WIRE SIZING:
- PER LUMETA PRODUCT DATA, PANEL-TO-PANEL WIRING MUST BE #12 AWG OR #10 AWG
  ↳ PER "PHOTOVOLTAIC SYSTEMS" BY JAMES P. DUNLOP, TYPE SHOULD BE "USE-2" 90°C WIRE
  ↳ LESS THAN 4 CURRENT-CARRYING CONDUCTORS PER CONDUIT
    ↳ NO ADJUSTMENT
  ↳ CONDUIT INSTALLED DIRECTLY ON ROOF
    ↳ 33°C INCREASE IN AMBIENT TEMP.
    ↳ CF_{temp} = 0.71

FINAL REPORT | PV DESIGN | pg 2 OF 2

DC DISCONNECT FUSE SIZE

REQUIRED AMPS → $\frac{15\text{ A}}{0.71} = 21.2\text{ A}$

#12 AWG 90°C USE-2 WIRE IS SUFFICIENT ($I_{nom} = 30\text{ A}$)

-PER SUNNY BOY PRODUCT DATA, #10 AWG to #6 AWG PERMITTED AT INVERTER.
- ↳ SAME CONDITION AS PANEL-TO-PANEL WIRING.
- ↳ MAX. LENGTH OF CABLE IS 37'-1" + 10'-0" = 47'-1"
- ↳ VOLTAGE DROP % = $\frac{I_{op} \times R_c \times L}{V_{op}} = \frac{45\text{ A}\,(0.9989\,\frac{\Omega}{kft})\,(0.04708\text{ kft})}{264.5}$

$$= \frac{0.71\text{V}}{264.5\text{V}} = 0.27\% < 3\% \quad \text{OK}\checkmark$$

-PER SUNNY BOY PRODUCT DATA, #6 AWG 90°C WIRE IS THE MAXIMUM ALLOWABLE SIZE FOR AC WIRING
- ↳ "PHOTOVOLTAIC SYSTEMS" RECOMMENDS 75°C WIRE
- ↳ SUNNY BOY RECOMMENDS A 50 A BREAKER
  - ↳ ASSUME "EACH" ARRAY HAS ITS OWN CONDUIT → 50 A WIRE

#8 AWG 75°C THW WIRE ($I_{nom} = 50\text{ A}$)

ENERGY PRODUCED PER YEAR:

PER "PHOTOVOLTAIC SYSTEMS" pg. 34, LOS ANGELES RECIEVES A MINIMUM OF 4.5 HRS OF PEAK SUN PER DAY IN JANUARY AND 5.5 HRS IN JULY
- ↳ ASSUME 91.25 DAYS (¼ OF YEAR) RECEIVE 4.5 hrs. PEAK SUN
- ↳ ASSUME 91.25 DAYS (¼ OF YEAR) RECEIVE 5.5 hrs. PEAK SUN
- ↳ ASSUME 182.5 DAYS (½ OF YEAR) RECEIVE 5.0 hrs. PEAK SUN

$P_{DC} = 12\text{ STRINGS}\,(1.2961\,\text{kW/STRING})\,[91.25(4.5) + 91.25(5.5) + 182.5(5.0)]$

$P_{DC} = 28{,}385\text{ kWh/year}$

ASSUME 20% LOSSES FROM DC TO AC

$P_{AC} = 0.8(28385) = \underline{22{,}708\text{ kWh/year}}$

FROM PVWATTS ANALYSIS, $P_{AC} = \underline{20{,}106\text{ kWh/year}}$
- ↳ SEE SPREADSHEET

INSTALLED COST:

PER "TRACKING THE SUN" BY WISER, BARBOSE & PETERMAN pg 16, INSTALLED COST OF PHOTOVOLTAIC SYSTEMS 10-100 KW IN CALIFORNIA IS \$7.60/WATT. HOWEVER, FROM pg 20, 11% OF THE COST IS "OTHER MATERIALS" SUCH AS RACKS, MOUNTING SYSTEMS AND SEALANTS. THEREFORE, 89% OF THE COST WAS USED FOR THIS REPORT.

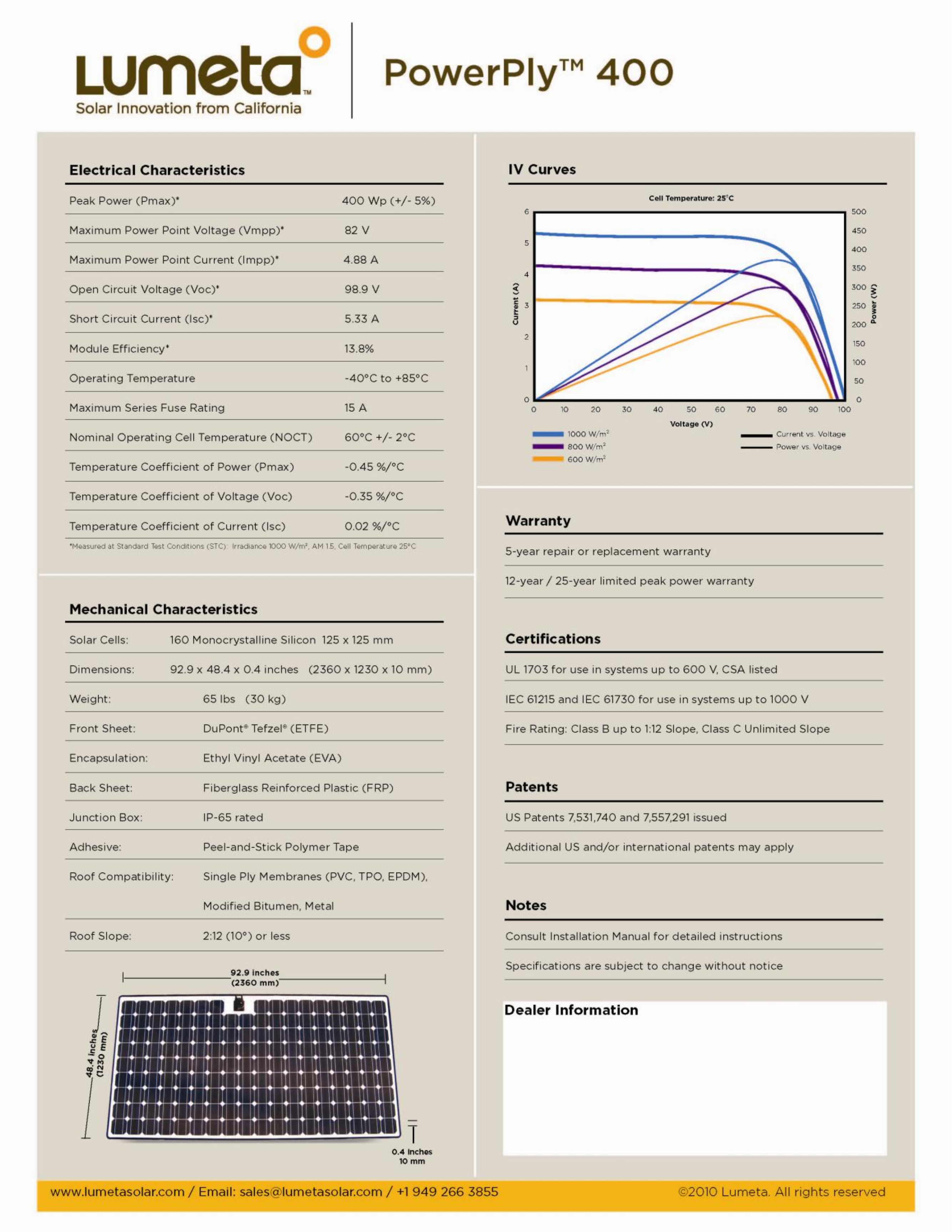
COST = 0.89 (7.60) = \$6.76/WATT × 15553.2 W = $\underline{\$105{,}201.84}$

Station Identification	
City:	Los Angeles
State:	California
Lat (deg N):	33.93
Long (deg W):	118.4
Elev (m):	32
PV System Specifications	
DC Rating:	15.6 kW
DC to AC Derate Factor:	0.77
AC Rating:	12.0 kW
Array Type: Fixed Tilt	
Array Tilt:	0
Array Azimuth:	160
Energy Specifications	
Cost of Electricity:	12.5 cents/kWh

PVWatts (Version 1) Results			
Month	Solar Radiation ($kWh/m^2/day$)	AC Energy (kWh)	Energy Value ($)
January	2.88	969	121.12
February	3.87	1,213	151.62
March	4.77	1,678	209.75
April	5.76	1,947	243.38
May	6.57	2,292	286.5
June	6.79	2,277	284.62
July	7	2,418	302.25
August	6.6	2,263	282.88
September	5.13	1,692	211.5
October	4.15	1,418	177.25
November	3.21	1,045	130.62
December	2.72	894	111.75
Year	4.96	20,106	2513.25

Cost of Energy in Los Angeles - "Rate B" Method				
Description	High Season (June-Sept.)		Low Season (Oct.-May)	
	$/kW	$/kWh	$/kW	$/kWh
Facilities Charge	$5.00			
Demand Charge				
High Peak Period	$9.00		$4.25	
Low Peak Period	$3.25			
Energy Charge				
High Peak Period		0.04679		0.04045
Low Peak Period		0.03925		0.04045
Base Period		0.01879		0.02252
ECA		0.05690		0.05690
ESA	$0.46		$0.46	
RCA	$0.96		$0.96	
Redactive Energy Charge				
High Peak Period		0.00026		0.00023
Low Peak Period		0.00017		0.00023
Base Period		0.00011		0.00014
Totals				
High Peak Period	$10.42	0.10395	$5.67	0.09758
Low Peak Period	$4.67	0.09632	$1.42	0.09758
Base Period	$1.42	0.07580	$1.42	0.07956

Cost of Energy per kWh in Los Angeles						
Description	Times/Days	Hrs/Week	High Season $/kW to $/kWh	Low Season $/kW to $/kWh	Total High Season $/kWh	Total Low Season $/kWh
High Peak Period	1P-5P/M-F	20	$0.03006	$0.00818	$0.13401	$0.10576
Low Peak Period	10A-1P/M-F	30	$0.00898	$0.00137	$0.10530	$0.09895
	5P-8P/M-F					
Base Period	8P-10A/M-F	118	$0.00069	$0.00035	$0.07649	$0.07991
	Sat, Sun					



Lumeta™
Solar Innovation from California

PowerPly™ 400

Electrical Characteristics

Peak Power (Pmax)*	400 Wp (+/- 5%)
Maximum Power Point Voltage (Vmpp)*	82 V
Maximum Power Point Current (Impp)*	4.88 A
Open Circuit Voltage (Voc)*	98.9 V
Short Circuit Current (Isc)*	5.33 A
Module Efficiency*	13.8%
Operating Temperature	-40°C to +85°C
Maximum Series Fuse Rating	15 A
Nominal Operating Cell Temperature (NOCT)	60°C +/- 2°C
Temperature Coefficient of Power (Pmax)	-0.45 %/°C
Temperature Coefficient of Voltage (Voc)	-0.35 %/°C
Temperature Coefficient of Current (Isc)	0.02 %/°C

*Measured at Standard Test Conditions (STC): Irradiance 1000 W/m², AM 1.5, Cell Temperature 25°C

IV Curves

Mechanical Characteristics

Solar Cells:	160 Monocrystalline Silicon 125 x 125 mm
Dimensions:	92.9 x 48.4 x 0.4 inches (2360 x 1230 x 10 mm)
Weight:	65 lbs (30 kg)
Front Sheet:	DuPont® Tefzel® (ETFE)
Encapsulation:	Ethyl Vinyl Acetate (EVA)
Back Sheet:	Fiberglass Reinforced Plastic (FRP)
Junction Box:	IP-65 rated
Adhesive:	Peel-and-Stick Polymer Tape
Roof Compatibility:	Single Ply Membranes (PVC, TPO, EPDM), Modified Bitumen, Metal
Roof Slope:	2:12 (10°) or less

Warranty

5-year repair or replacement warranty

12-year / 25-year limited peak power warranty

Certifications

UL 1703 for use in systems up to 600 V, CSA listed

IEC 61215 and IEC 61730 for use in systems up to 1000 V

Fire Rating: Class B up to 1:12 Slope, Class C Unlimited Slope

Patents

US Patents 7,531,740 and 7,557,291 issued

Additional US and/or international patents may apply

Notes

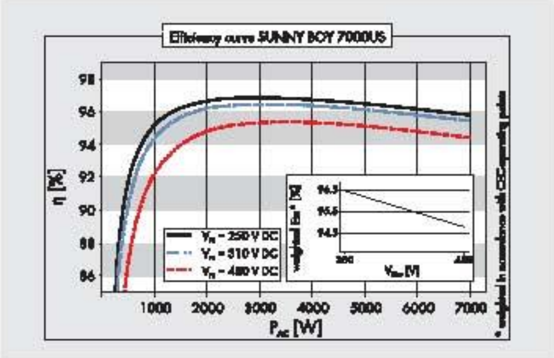
Consult Installation Manual for detailed instructions

Specifications are subject to change without notice

Dealer Information

www.lumetasolar.com / Email: sales@lumetasolar.com / +1 949 266 3855

Technical data	Sunny Boy 5000-US			Sunny Boy 6000-US			Sunny Boy 7000-US			Sunny Boy 8000-US	
	208 V AC	240 V AC	277 V AC	208 V AC	240 V AC	277 V AC	208 V AC	240 V AC	277 V AC	240 V AC	277 V AC
Input (DC)											
Max. recommended PV power (@ module STC)	6250 W			7500 W			8750 W			10000 W	
Max. DC power (@ cos φ = 1)	5300 W			6350 W			7400 W			8600 W	
Max. DC voltage	600 V			600 V			600 V			600 V	
DC nominal voltage	310 V			310 V			310 V			345 V	
MPP voltage range	250 V – 480 V			250 V – 480 V			250 V – 480 V			300 V – 480 V	
Min. DC voltage / start voltage	250 V / 300 V			250 V / 300 V			250 V / 300 V			300 V / 365 V	
Max. input current / per string (at DC disconnect)	21 A / 20 A 36 A @ combined terminal			25 A / 20 A 36 A @ combined terminal			30 A / 20 A 36 A @ combined terminal			30 A / 20 A 36 A @ combined terminal	
Number of MPP trackers / fused strings per MPP tracker	1 / 4 (DC disconnect)										
Output (AC)											
AC nominal power	5000 W			6000 W			7000 W			7680 W	8000 W
Max. AC apparent power	5000 VA			6000 VA			7000 VA			8000 VA	
Nominal AC voltage / adjustable	208 V / ●	240 V / ●	277 V / ●	208 V / ●	240 V / ●	277 V / ●	208 V / ●	240 V / ●	277 V / ●	240 V / ●	277 V / ●
AC voltage range	183 - 229 V	211 - 264 V	244 - 305 V	183 - 229 V	211 - 264 V	244 - 305 V	183 - 229 V	211 - 264 V	244 - 305 V	211 - 264 V	244 - 305 V
AC grid frequency; range	60 Hz; 59.3 – 60.5 Hz			60 Hz; 59.3 – 60.5 Hz			60 Hz; 59.3 – 60.5 Hz			60 Hz; 59.3 – 60.5 Hz	
Max. output current	24 A	21 A	18 A	29 A	25 A	22 A	34 A	29 A	25 A	32 A	
Power factor (cos φ)	1			1			1			1	
Phase conductors / connection phases	1 / 2	1 / 2	1 / 1	1 / 2	1 / 2	1 / 1	1 / 2	1 / 2	1 / 1	1 / 2	1 / 1
Harmonics	< 4%			< 4%			< 4%			< 4%	
Efficiency											
Max. efficiency	96.7%	96.8%	96.8%	96.9%	96.8%	97.0%	97.1%	96.9%	97.0%	96.3%	96.5%
CEC efficiency	95.5%	95.5%	95.5%	95.5%	95.5%	96.0%	95.5%	96.0%	96.0%	96.0%	96.0%
Protection devices											
DC reverse-polarity protection	●			●			●			●	
AC short circuit protection	●			●			●			●	
Galvanically isolated / all-pole sensitive monitoring unit	● / –			● / –			● / –			● / –	
Protection class / overvoltage category	I / III			I / III			I / III			I / III	
General data											
Dimensions (W / H / D) in mm (in)	470 / 615 / 240 (18.5 / 24 / 9)										
DC Disconnect dimensions (W / H / D) in mm (in)	187 / 297 / 190 (7 / 12 / 7.5)										
Packing dimensions (W / H / D) in mm (in)	390 / 580 / 800 (16 / 23 / 31.5)										
DC Disconnect packing dimensions (W / H / D) in mm (in)	370 / 240 / 280 (15 / 9 / 11)										
Weight / DC Disconnect weight	64 kg (141 lb) / 3.5 kg (8 lb)									66 kg (145 lb) / 3.5 kg (8 lb)	
Packing weight / DC Disconnect packing weight	67 kg (147 lb) / 4 kg (9 lb)									69 kg (152 lb) / 4 kg (9 lb)	
Operating temperature range (full power)	–25 °C ... +45 °C (–13 °F ... +113 °F)										
Noise emission (typical)	44 dB(A)			45 dB(A)			46 dB(A)			49 dB(A)	
Internal consumption at night	0.1 W			0.1 W			0.1 W			0.1 W	
Topology	LF transformer			LF transformer			LF transformer			LF transformer	
Cooling concept	OptiCool			OptiCool			OptiCool			OptiCool	
Electronics protection rating / connection area	NEMA 3R / NEMA 3R			NEMA 3R / NEMA 3R			NEMA 3R / NEMA 3R			NEMA 3R / NEMA 3R	
Features											
Display: text line / graphic	● / –			● / –			● / –			● / –	
Interfaces: RS485 / Bluetooth	○ / ○			○ / ○			○ / ○			○ / ○	
Warranty: 10 / 15 / 20 years	● / ○ / ○			● / ○ / ○			● / ○ / ○			● / ○ / ○	
Certificates and permits (more available on request)	UL1741, UL1998, IEEE 1547, FCC Part 15 (Class A & B), CSA C22.2 No. 107.1-2001										
NOTE: US inverters ship with gray lids.											
Data at nominal conditions											
● Standard features ○ Optional features – Not available											
Type designation	SB 5000US			SB 6000US			SB 7000US			SB 8000US	



Accessories


RS485 interface
485USPB-SMC-NR


Bluetooth® Piggy Back
BTPBINV-NR


Combi-Switch
DC disconnect and PV array combiner box
COMBO-SWITCH


Combiner Box
Simplify wiring for added convenience and safety
SBCB-6-3R or SBCB-6-4

Toll Free +1 888 4 SMA USA
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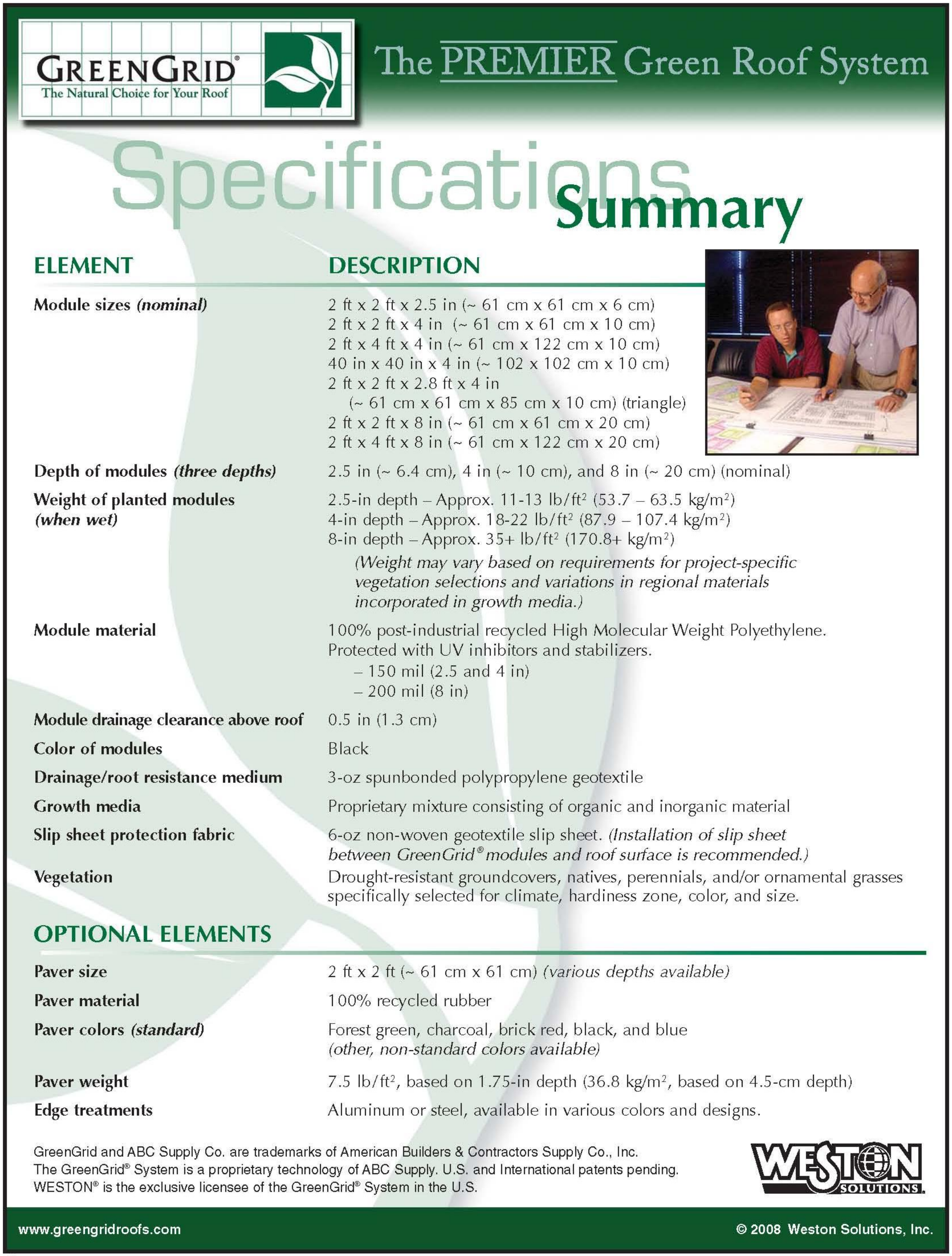
SMA America, LLC

Green Roof



GREENGRID®
The Natural Choice for Your Roof

The PREMIER Green Roof System

Specifications Summary

ELEMENT	DESCRIPTION
Module sizes *(nominal)*	2 ft x 2 ft x 2.5 in (~ 61 cm x 61 cm x 6 cm) 2 ft x 2 ft x 4 in (~ 61 cm x 61 cm x 10 cm) 2 ft x 4 ft x 4 in (~ 61 cm x 122 cm x 10 cm) 40 in x 40 in x 4 in (~ 102 x 102 cm x 10 cm) 2 ft x 2 ft x 2.8 ft x 4 in (~ 61 cm x 61 cm x 85 cm x 10 cm) (triangle) 2 ft x 2 ft x 8 in (~ 61 cm x 61 cm x 20 cm) 2 ft x 4 ft x 8 in (~ 61 cm x 122 cm x 20 cm)
Depth of modules *(three depths)*	2.5 in (~ 6.4 cm), 4 in (~ 10 cm), and 8 in (~ 20 cm) (nominal)
Weight of planted modules *(when wet)*	2.5-in depth – Approx. 11-13 lb/ft^2 (53.7 – 63.5 kg/m^2) 4-in depth – Approx. 18-22 lb/ft^2 (87.9 – 107.4 kg/m^2) 8-in depth – Approx. 35+ lb/ft^2 (170.8+ kg/m^2) *(Weight may vary based on requirements for project-specific vegetation selections and variations in regional materials incorporated in growth media.)*
Module material	100% post-industrial recycled High Molecular Weight Polyethylene. Protected with UV inhibitors and stabilizers. – 150 mil (2.5 and 4 in) – 200 mil (8 in)
Module drainage clearance above roof	0.5 in (1.3 cm)
Color of modules	Black
Drainage/root resistance medium	3-oz spunbonded polypropylene geotextile
Growth media	Proprietary mixture consisting of organic and inorganic material
Slip sheet protection fabric	6-oz non-woven geotextile slip sheet. *(Installation of slip sheet between GreenGrid® modules and roof surface is recommended.)*
Vegetation	Drought-resistant groundcovers, natives, perennials, and/or ornamental grasses specifically selected for climate, hardiness zone, color, and size.

OPTIONAL ELEMENTS

Paver size	2 ft x 2 ft (~ 61 cm x 61 cm) *(various depths available)*
Paver material	100% recycled rubber
Paver colors *(standard)*	Forest green, charcoal, brick red, black, and blue *(other, non-standard colors available)*
Paver weight	7.5 lb/ft^2, based on 1.75-in depth (36.8 kg/m^2, based on 4.5-cm depth)
Edge treatments	Aluminum or steel, available in various colors and designs.

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The GreenGrid® System is a proprietary technology of ABC Supply. U.S. and International patents pending.
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WESTON SOLUTIONS

www.greengridroofs.com

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