Appendix A

Seismic Calculations

BUILDING LOCATION

Zip code: 92592

*USING USGS SOFTWARE

→ $S_s = 2.026\,g$ → max

→ $S_1 = 0.761\,g$

SITE CLASS – B

•IBC 2006

– OCCUPANCY CLASS IV → HOSPITAL

$S_1 \geq 0.75$

↓

SDC F

•PERMITTED ANALYTICAL PROCEDURES

– NO DAMPING SYSTEM

$S_{MS} = F_a S_s = 2.026\,g$

$S_{M1} = F_v S_1 = 0.761\,g$

$F_v = 1.0$, $F_a = 1.0$ } ASCE TABLE 11.4

$S_{DS} = \frac{2(S_{MS})}{3} = \frac{2(2.026)}{3} = 1.351$

$S_{D1} = \frac{2(S_{M1})}{3} = \frac{2(.761)}{3} = 0.507$

→ EQUIVALENT LATERAL FORCE PROCEDURE

$R = 3.25$ → regular steel concentric bracing

$I = 1.5$

$T_a = C_t h_n^x$

$T_a = 0.02(107)^{0.75}$

$T_a = 0.665$

$C_t = 0.02$, $x = 0.75$ } concentric frames

$h_n = 107'$

$T = C_u T_a$ $\qquad C_u = 1.4$

$T = 1.4(0.665) = 0.932$

From Figure 22-15 in ASCE

$T_L = 8$

$T < T_L$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.507}{0.932\left(\frac{3.25}{1.5}\right)} = 0.251$$

$$C_s > \frac{0.5 S_1}{\left(\frac{R}{I}\right)} = \frac{0.5(0.761)}{\left(\frac{3.25}{1.5}\right)} = 0.176$$

$$C_s = 0.176$$

EFFECTIVE SEISMIC WEIGHT

AREA CALCULATIONS:

LEVEL 1 = 26,500 sq.ft
LEVEL 2-6 = 26,500 sq.ft
} BED TOWER

Level 1:

Slab	48 psf		× 26,500 ft²	= 1272 k
Framing				= 115 k
Ext. Wall	35 psf × 800' PERIMETER		× 18'	= 504 k
Partition	20 psf		× 26,500 ft²	= 530 k
Columns		70 PLF	× 18' × 72	= 91 k
Misc	10 psf		× 26,500 ft²	= 265 k
			TOTAL	= 2777 k

Level 2-6:

Slab	48 psf		× 26500	= 1272 k
Framing				= 115 k
Ext Wall	35 psf × 800'		× 13.5'	= 378 k
Partition	20 psf		× 26500	= 530 k
Columns		70 PLF	× 13.5' × 72	= 68 k
Misc	10 psf		× 26500	= 265 k
			TOTAL	= 2628 k

ROOF:

Roofing:	17 psf	x	26500	= 451^k
Framing:	10 psf	x	26500	= 265^k
			Total	= 716^k

WEIGHT TOTAL = $16{,}633^k$

$$V = C_s W = 0.176(16633) = 2927.4^k$$

$$k = 0.75 + 0.5(0.932) = 1.216$$

SHEAR FORCE AT EACH LEVEL

$$C_x = \frac{w_x h_x^k}{w_i h_i^k}$$

$$C_1 = 0$$

$$C_2 = \frac{(2628)(18)^{1.22}}{(2628)(18)^{1.22} + (2628)(31.5)^{1.22} + (2628)(45)^{1.22} + (2628)(58.5)^{1.22} + (2628)(72)^{1.22} + (716)(87.5)^{1.22}}$$

1.54×10^6

$C_2 = 0.057$	$V = 167^k$
$C_3 = 0.113$	$V = 331^k$
$C_4 = 0.175$	$V = 512^k$
$C_5 = 0.24$	$V = 703^k$
$C_6 = 0.309$	$V = 905^k$
$C_R = 0.107$	$V = 313^k$

Wind Load Calculations

Gust Effect Coefficients										
Rigid Building										
IZ	QN-S	QE-W	BN-S (ft)	BE-W (ft)	gQ	h(ft)	c	Є bar	z(ft)	gV
0.179	0.769	0.805	564	353	3.4	97	0.2	1/5.0	64.2	3.4

N-S Wind Forces					
Floor	Height	Tributary Height	Story Force	Story Shear	Overturning Moment
	(ft)	(ft)	(kips)	(kips)	(ft-kips)
1	0.0	0.0	0.0	504.3	0.0
2	18.0	18.0	279.0	504.3	2511.3
3	31.5	13.5	30.6	225.3	757.4
4	45.0	13.5	32.3	194.7	1235.5
5	58.5	13.5	33.6	162.4	1738.8
6	72.0	13.5	34.5	128.7	2251.1
roof	87.3	15.3	40.8	94.3	3249.7
ridge	107.0	19.7	53.5	53.5	5197.5
		Total	504.3		16941.3

E-W Wind Forces					
Floor	Height	Tributary Height	Story Force	Story Shear	Overturning Moment
	(ft)	(ft)	(kips)	(kips)	(ft-kips)
1	0.0	0.0	0.0	1213.5	0.0
2	18.0	18.0	177.1	1213.5	1593.9
3	31.5	13.5	140.3	1036.4	3472.4
4	45.0	13.5	150.7	896.1	5764.3
5	58.5	13.5	154.5	745.4	7995.4
6	72.0	13.5	158.2	591.0	10322.6
roof	87.3	15.3	187.1	432.7	14902.5
ridge	107.0	19.7	245.6	245.6	23860.0
		Total	1213.5		67911.1

Pressure: $p = q\,GC_p - q_i(GC_{pi})$

Windward: $p_z = q_z G_f C_p - q_h(GC_{pi})$

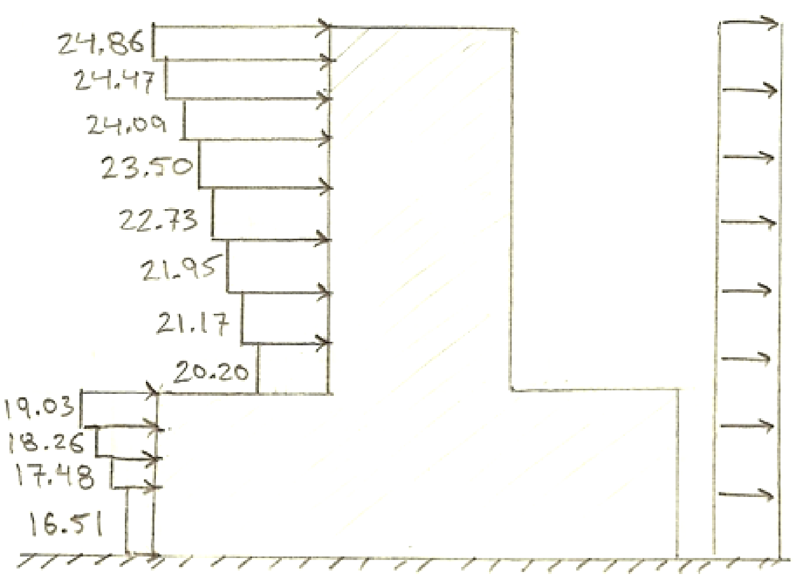
Leeward: $p_h = q_h G C_p - q_h (GC_{pi})$

WIND FROM N-S

WIND LOAD

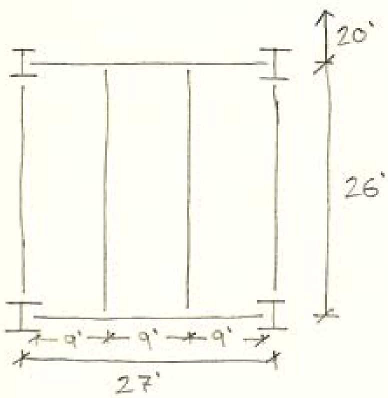
WINDWARD (PSF)

LEE-WARD (PSF)



Appendix B

Floor Framing and Column Calculations



Light-Weight Conc

110 PCF $f'_c = 5$ ksi

2" LOK-FLOOR, 19 Ga DECK w/ 4" SLAB, 48 PSF

DL : 40 PSF

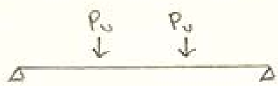
LL : 100 PSF

Girder Design : Trib Area : $\left(\frac{26' + 20'}{2}\right) \times 27' = 621\ ft^2$

Influence Area : $2A_T = 1242\ ft^2$

$LL = L_o\left(.25 + \frac{15}{\sqrt{1242}}\right) = L_o(.676)$

$LL = 67.6$ PSF



P_u : DEAD : $40 \times 9' \times \left(\frac{26+20}{2}\right) = 8.2^k$

LIVE : $67.6 \times 9' \times \left(\frac{26+20}{2}\right) = 13.9^k$

STRENGTH : $1.2D + 1.6L = 32.1^k = P_u$

$M_u = P_u \times 9' = 288.7^{'k}$

DEFLECTION :

$\Delta_{LL} \leq \frac{27 \times 12}{360} = 0.9" = \frac{13.9 \times (27 \times 12)^3}{28(29000) I} \Rightarrow I_{req} = 646.9\ in^4\ (LB)$

PRE-COMPOSITE DL $\quad \Delta_T \leq \frac{27 \times 12}{240} = 1.35" = \frac{22.1 \times (27 \times 12)^3}{28(29000) I} \Rightarrow I_{req} = 685.7\ in^4\ (LB)$

$(48)(9)\left(\frac{26+20}{2}\right) = 9.9^k \quad \Delta_{pc} \leq \frac{27 \times 12}{360} = 0.9" = \frac{9.9 \times (27 \times 12)^3}{28(29000) I} \Rightarrow I_{req} = 460.8\ in^4 (I_x)$

TRY W18×35

$Y_2 = 6" - \frac{2"}{2} = 5" \Rightarrow$ USE PNA 7

$\phi M_n = 365^{'k} \quad I_{LB} = 906\ in^4 \quad I_x = 510\ in^4 \quad \therefore$ OK

$2Q_n = 129^k \quad a = \frac{129}{.85 \times 5 \times 81"} = 0.375" < 2" \quad \therefore$ OK

$Q_n = 18.3^k$ FOR $\frac{3}{4}"\ \phi$ STUDS DECK PARALLEL

$129/18.3 \Rightarrow 8$ STUDS $\Rightarrow 16$ STUDS

BEAM DESIGN: TRIB AREA: 9' × 26' = 234 ft²
INFLUENE: $2A_T$ = 468 ft²

$LL = L_o(.25 + \frac{15}{\sqrt{468}}) = 0.94 LL = 94.3$ psf

w_u = DL: 40 × 9 = 0.36 k/ft
LL: 94.3 × 9 = 0.85 k/ft

26'

STRENGTH: $1.2D + 1.6L = 1.79^{k}/ft$

$M_u = \frac{1.79(26)^2}{8} = 151.4^{1k}$

DEFLECTION:

$\Delta_{LL} \le \frac{(26 \times 12)}{360} = 0.87" = \frac{5(\frac{.85}{12})(26 \times 12)^4}{384(29000)(I)} \Rightarrow I_{req} = 346.4 \text{ in}^4$

$\Delta_T \le \frac{(26 \times 12)}{240} = 1.3" = \frac{5(\frac{1.21}{12})(26 \times 12)^4}{384(29000)(I)} \Rightarrow I_{req} = 330.0 \text{ in}^4$

$48 \text{ psf} \times 9' = 0.43 \Rightarrow \Delta_{pc} \le \frac{(26 \times 12)}{360} = 0.87" = \frac{5(\frac{.43}{12})(26 \times 12)^4}{384(29000)(I)} \Rightarrow I_{req} = 175.2 \text{ in}^4$

TRY W14×26

$Y_2 = 6" - \frac{2"}{2} = 5" \Rightarrow$ PNA 7

$\phi M_n = 224^{1k}$ $I_{LB} = 465 \text{ in}^4$ $I_x = 245 \text{ in}^4$ ∴ OK

$2 Q_n = 96.1^k$ $a = \frac{96.1}{.85 \times 5 \times 90} = 0.25 < 2"$ ∴ OK

$Q_n = 17.2^k$ FOR ¾" Ø STUDS, DECK PERP

$\frac{96.1^k}{17.2^k} \Rightarrow 6$ STUDS $\Rightarrow$ 12 STUDS

Interior Column Calculation

Interior Column Floor 1

$H = 18'$, 5 Floors $A_T = 27' \times 26' = 702\ ft^2$

Loads:

Live : $100\ psf\ (3510) = 351^k$

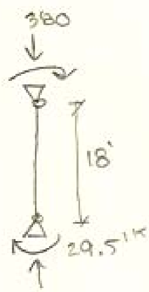
Dead : $40\ psf\ (3510) = 140^k$

$A_T = 5(702) = 3510\ ft^2$

$A_I = 4A_T = 14040\ ft^2$

$L = L_0\left(25 + \frac{15}{\sqrt{A_I}}\right) = 351\left(25 + \frac{15}{\sqrt{14040}}\right) = 132^k$

$1.2D + 1.6L = 1.2(140) + 1.6(132) = 380^k$



$P_u = 380^k$

WIND: $27\ psf\ (27') = 729\ plf$

$M = \frac{wL^2}{8} = \frac{729(18)^2}{8} = 29.5'^k$

$P_{eff} = P_u + \left(\frac{24}{d}\right) M_u$

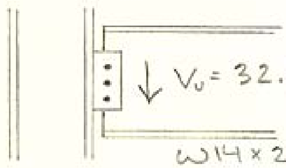
$= 380 + \left(\frac{24}{12}\right) 29.5 = 439^k$

$KL = 18'$

$\boxed{\text{TRY W12} \times 65}$

Shear Connection Calculation

SHEAR CONNECTION – TYPICAL
(Girder – Column)



$\downarrow V_u = 32.1^k$ → From Floor Calcs

W14×26

$t_w = .255$ T – 11.625 in.

USE $7/8"\ \phi$ A325-N BOLTS : $\phi r_n = 21.6^k$

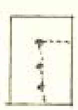
$n = \frac{32.1}{21.6} = 1.5$ ⇒ USE 3 BOLTS

– PLATE THICKNESS
· MAX : $t_{plate} \leq \frac{d_b}{2} + 1/16" = 0.5"$

TRY 1/4" PLATE

– BOLT BEARING :
· PLATE $\phi r_n = 0.75 \times 2.4 \cdot (58)(0.875)(0.25)$
$= 22.8^k > 21.6^k$ ∴ BOLTS CONTROL

– BLOCK SHEAR :



· BEAM NOT COPED
· PLATE : TABLE 9-3 :
SHEAR YIELD : $121^k/in \times 1/4" = 30.3^k$
SHEAR RUPTURE : $131^k/in \times 1/4" = 32.8^k$
TENSION RUPTURE : $43.5^k/in \times 1/4" = \underline{10.9^k}$
$\phi R_n = 41.2^k$
$> 32.1^k$ ∴ OK

· SHEAR YIELD : $\phi R_n = 1.0(0.6 F_y) A_g$
$\phi R_n = 1.0(0.6 \times 36)(9 \times 1/4)$
$\phi R_n = 48.6^k > 32.1^k$ ∴ OK

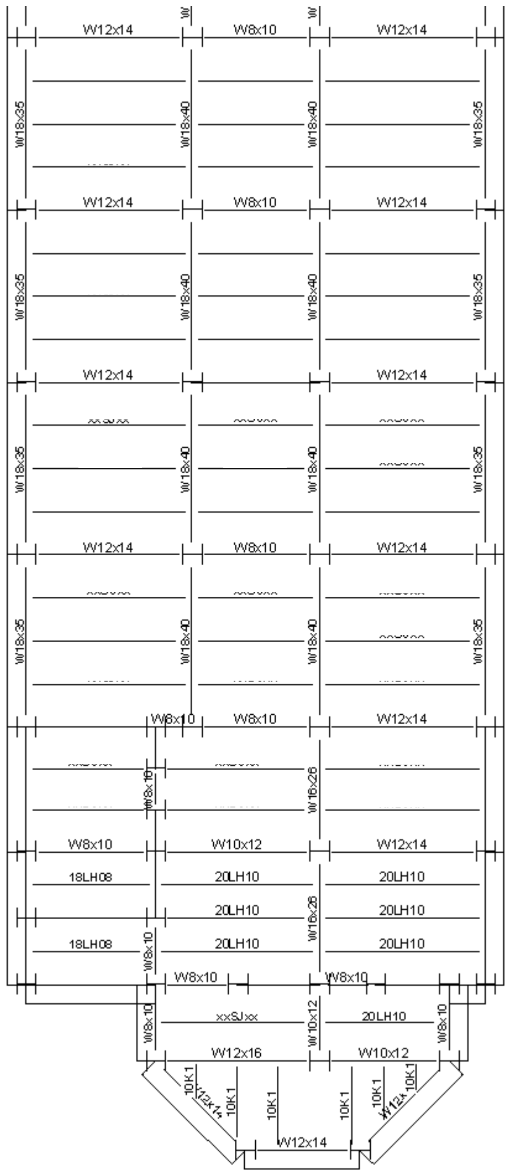
· SHEAR RUPTURE : $\phi R_n = 0.75(0.6 \times F_u) A_n$
$= 0.75(0.6 \times 58)[9" - 3(7/8 + 1/8)](1/4)$
$= 39.2^k > 32.1^k$ ∴ OK

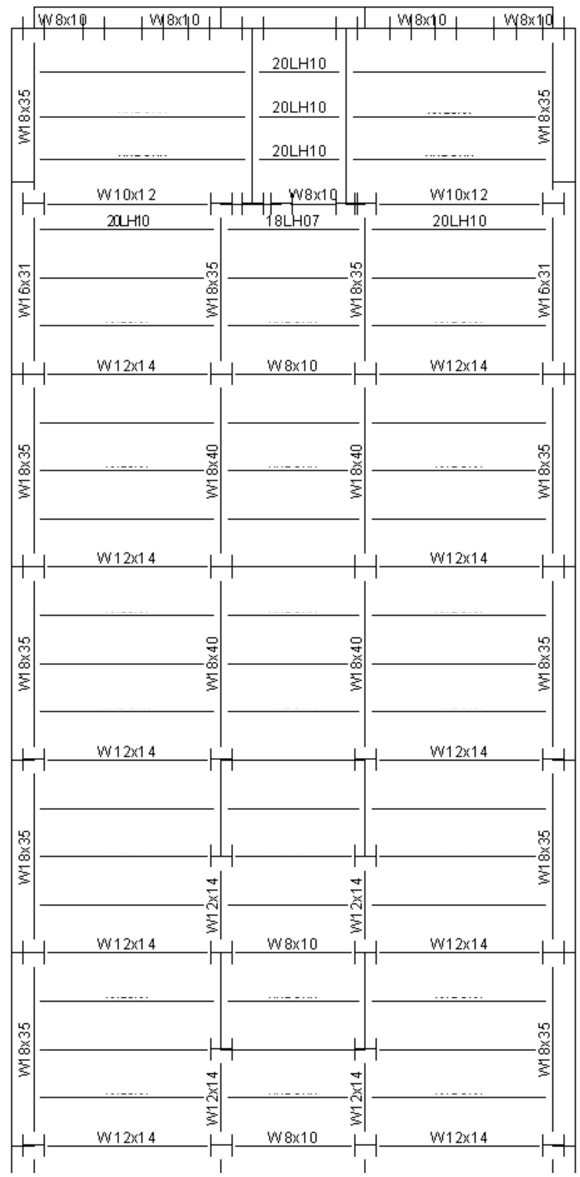
3/16" WELD EACH SIDE ⇒ INSPECTION*

Appendix C

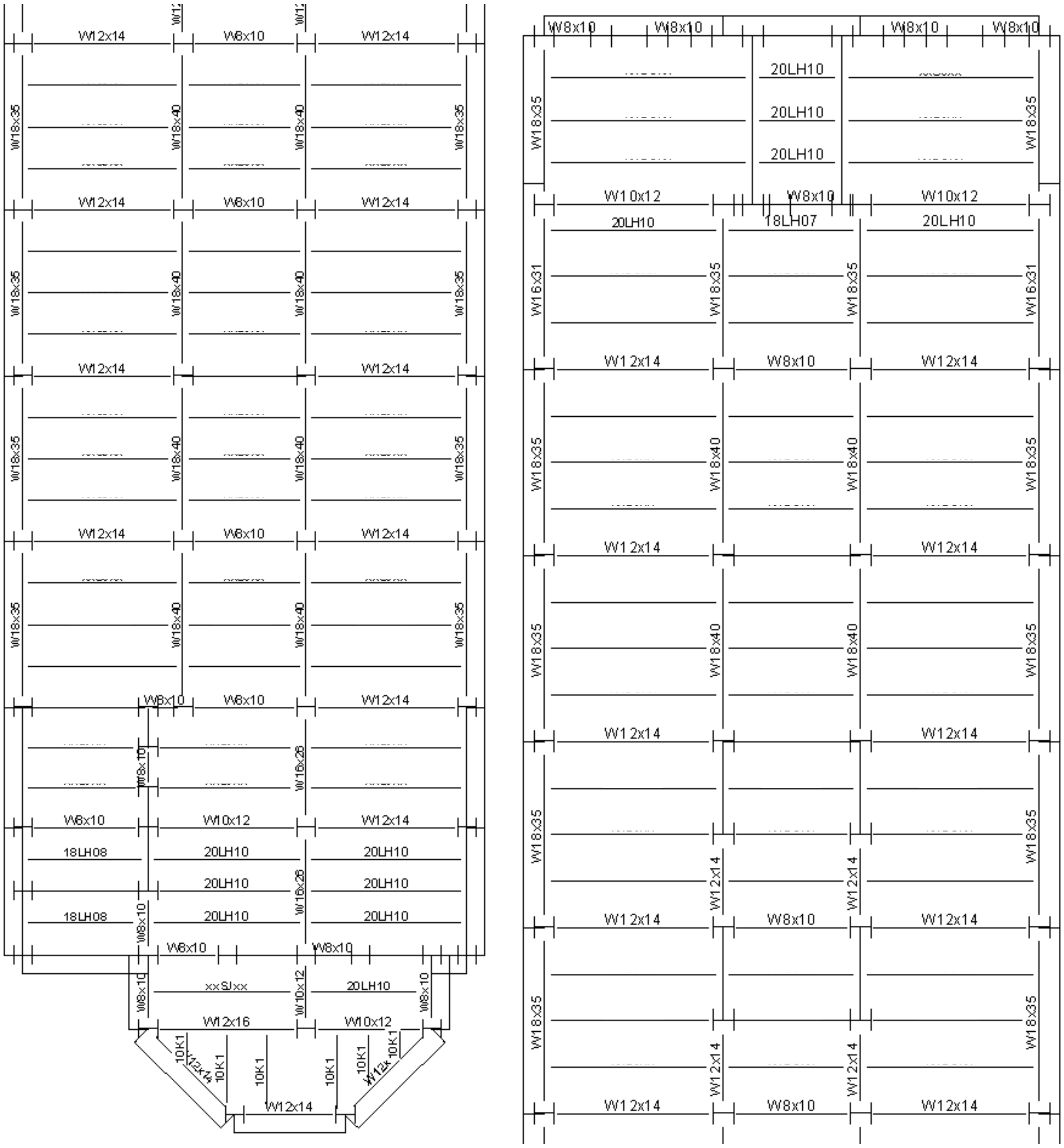
Floor Framing Results

Floor 1 Layout (Typical)

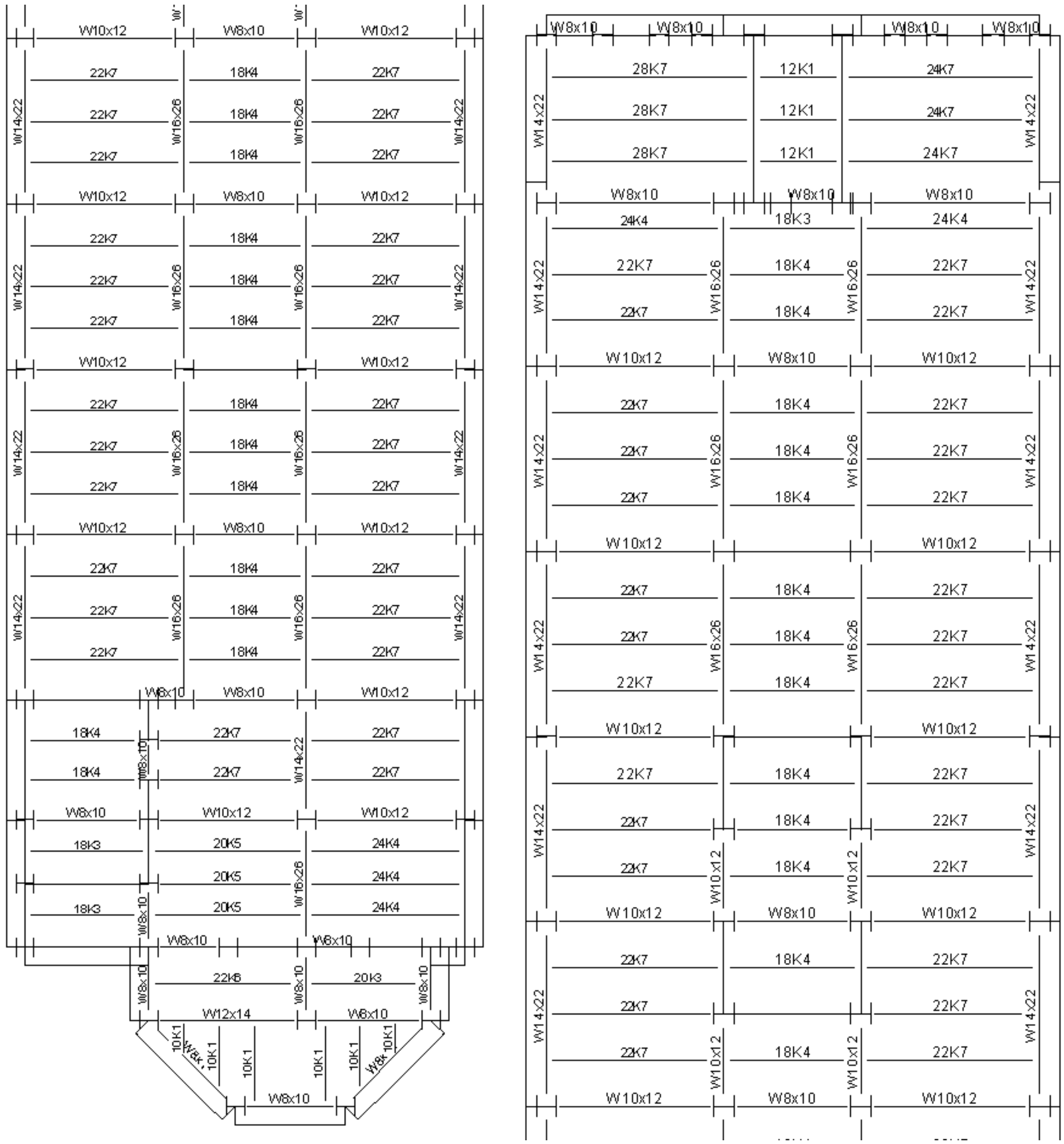




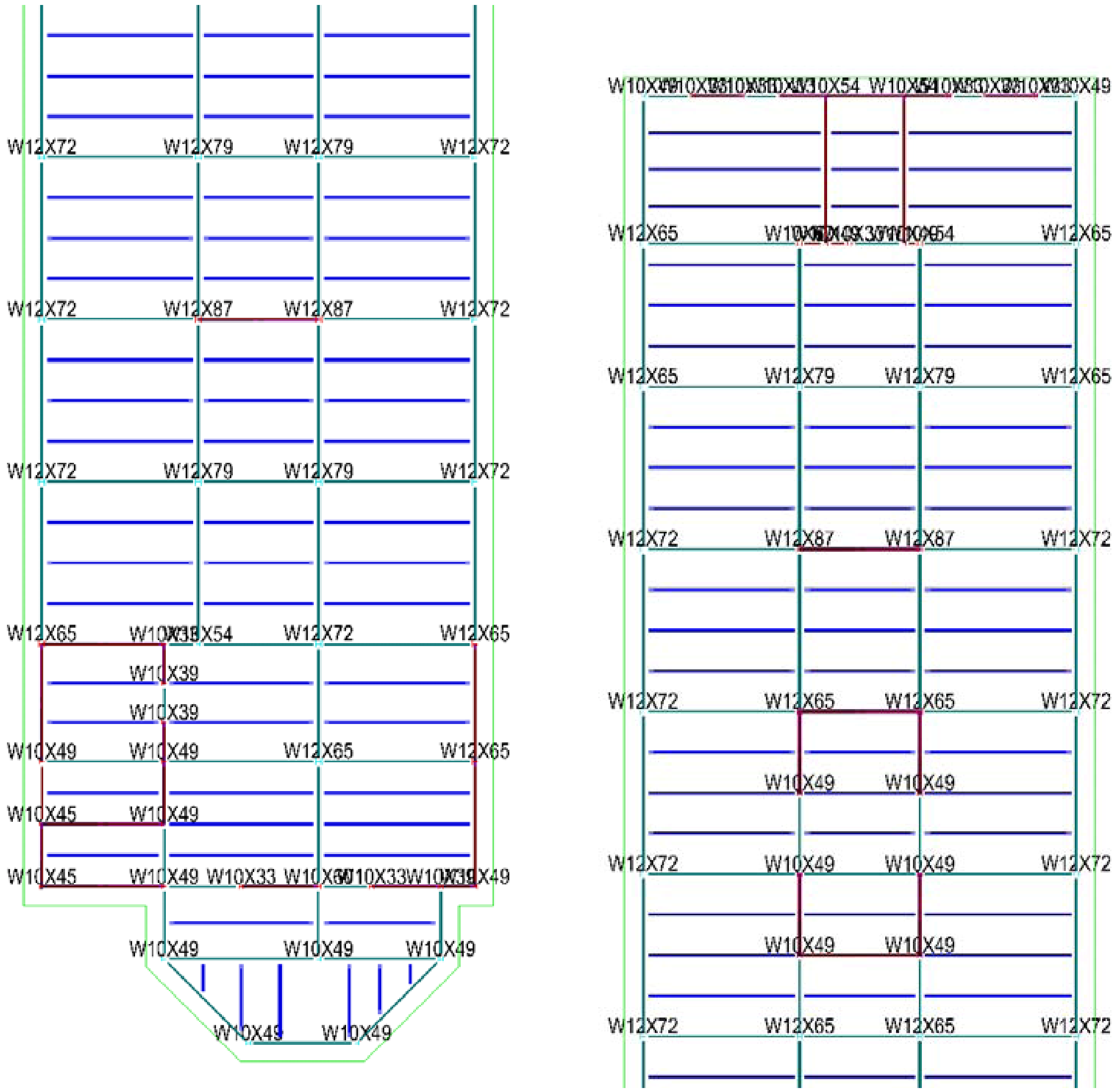
Level 5 Layout (Typical)



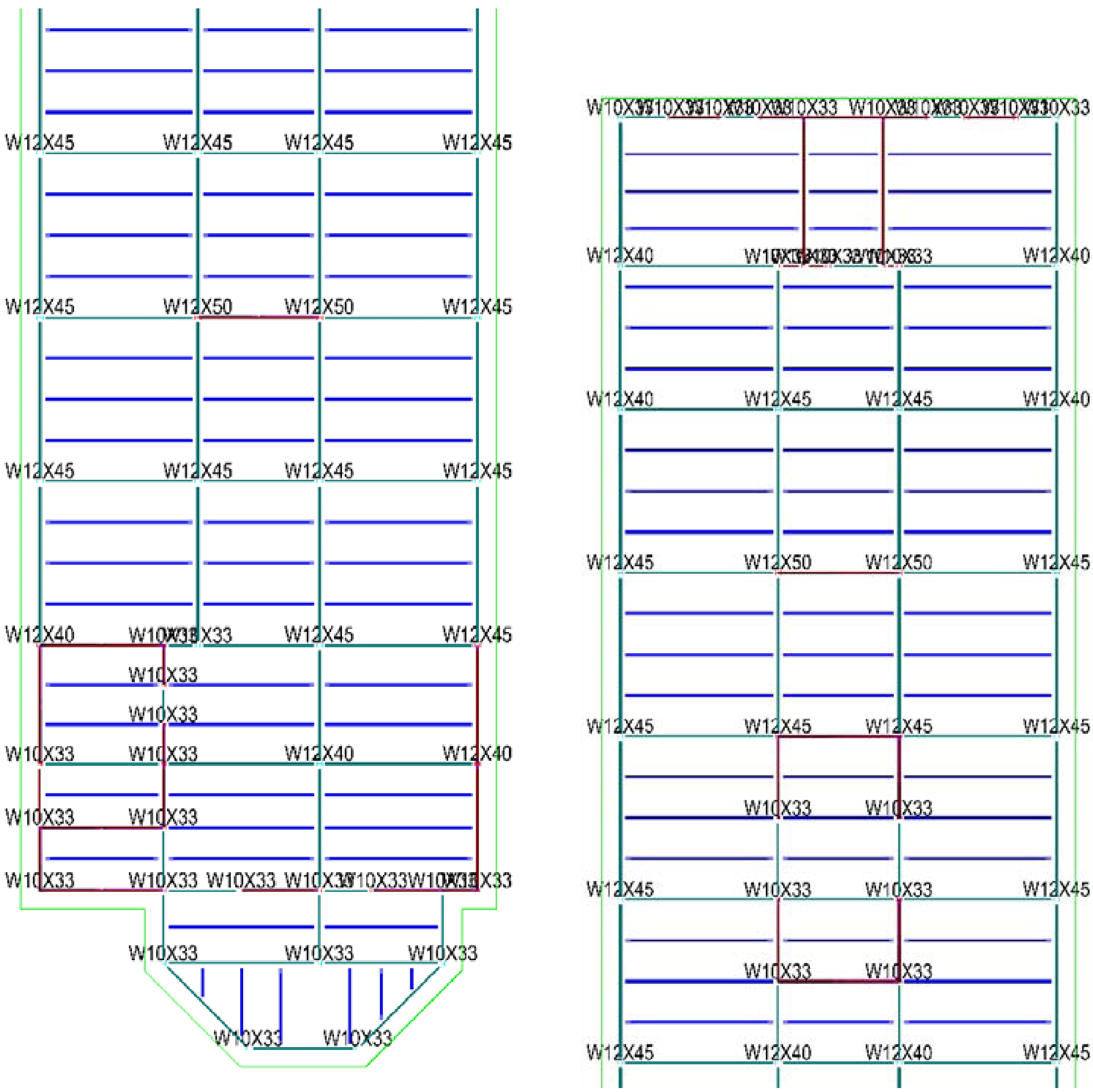
Roof Layout


W10x12
W8x10
22K7
18K4
W14x22
W16x26
W12x14
20K5
24K4
18K3
22K6
20K3
10K1
W8x10
28K7
12K1
24K7
W10x12

1st-2nd Story Column Plan



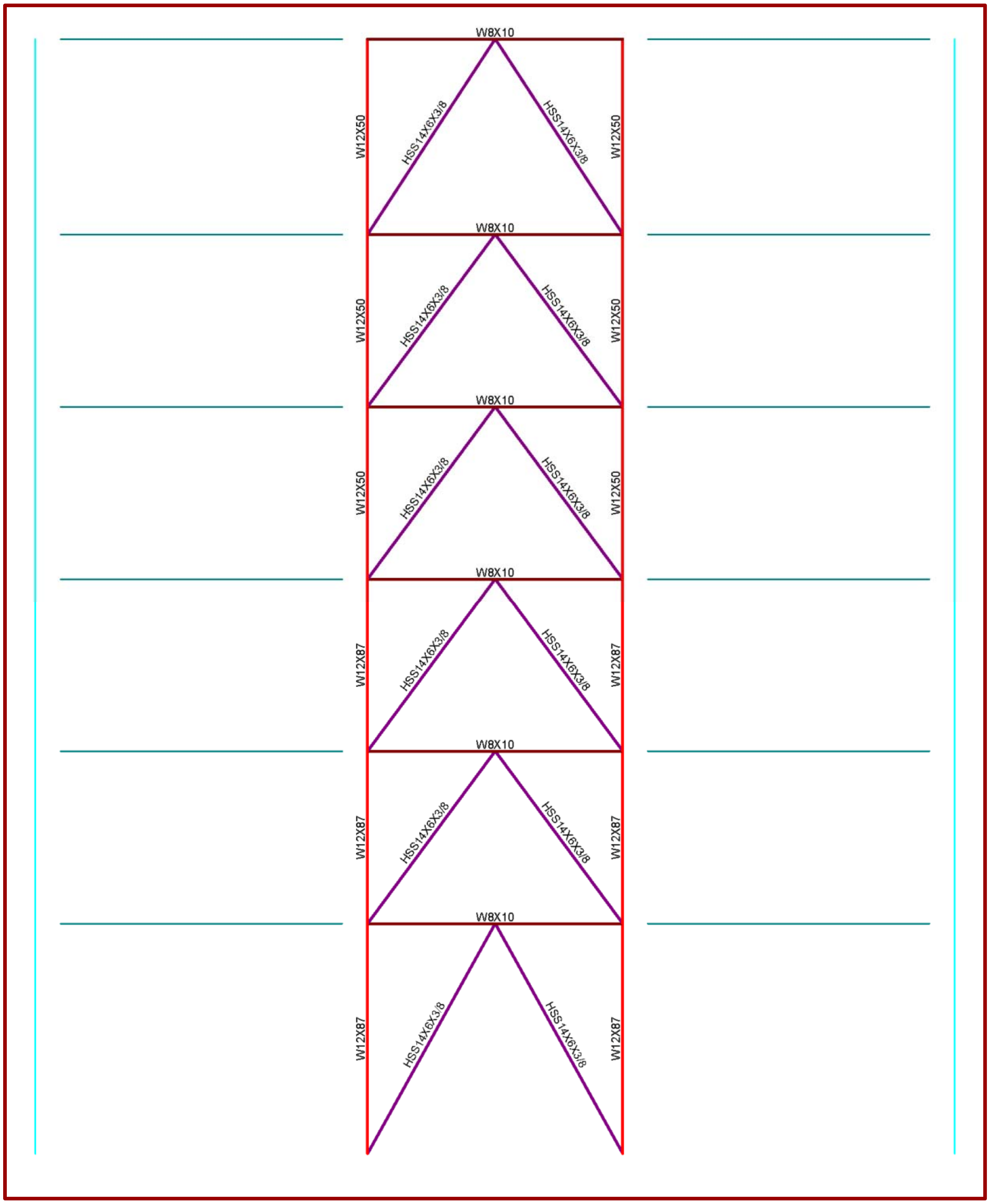
5th-6th Story Column Plan



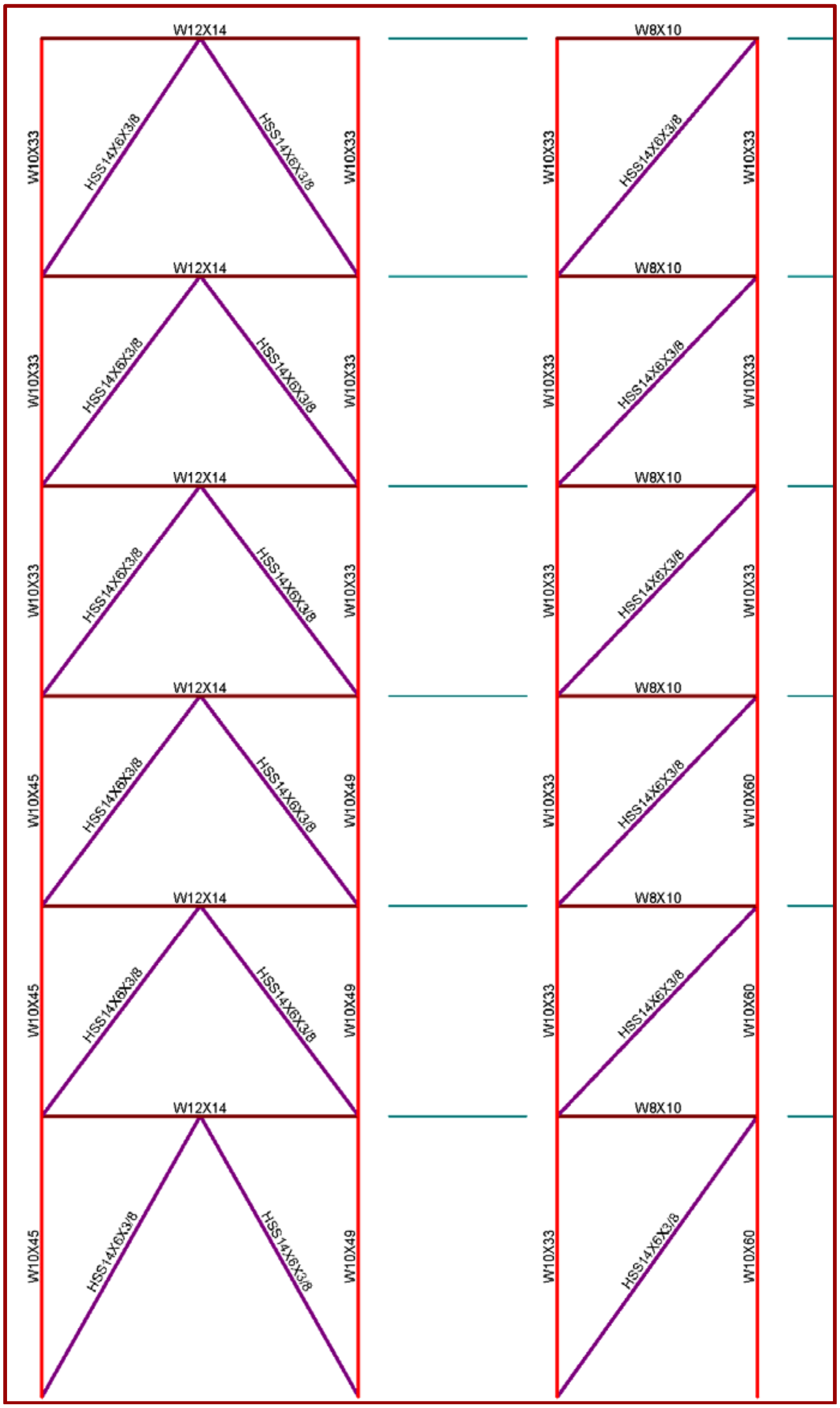
Appendix D

Braced Frame Elevations

Typical Braced Frame Configuration



Typical Braced Frame Configuration



Appendix E

Construction Management Breadth Study

CAST-IN-PLACE CONCRETE SYSTEM - EXISTING CONDITION													
COLUMNS	Amount	Unit	Crew	# Crews	Units/Day	Days	Labor Cost/Day	Labor	Mat'l Cost/Unit	Mat'l	Equip. Cost/Day	Equip.	TOTAL COST
Formwork	36504	SFCA	C-1	4	800	46	4624	210993	1.23	44899.92			255893
Concrete	2704	CY	C-20	1	150	18	2860	51556	109	294736	600	10816	357108
Reinf.	85	Ton	4 Rodm	4	11	8	5504	42531	935	79475			122006
SLABS	Amount	Unit	Crew	# Crews	Units/Day	Days	Labor Cost/Day	Labor	Mat'l Cost/Unit	Mat'l	Equip. Cost/Day	Equip.	TOTAL COST
Formwork	159000	SFCA	C-20	7	3150	50	12474	629640	1.23	195570			825210
Concrete	4907	CY	C-20	4	600	8	11440	93560	109	534863	600	4907	633330
Slab Finish	159000	SF	CeFl	8	4000	40	6608	262668					262668
Reinf.	600	Ton	4 Rodm	6	16.5	36	8256	300218	935	561000			861218
SHEARWALLS	Amount	Unit	Crew	# Crews	Units/Day	Days	Labor Cost/Day	Labor	Mat'l Cost/Unit	Mat'l	Equip. Cost/Day	Equip.	TOTAL COST
Formwork	24300	SFCA	C-2	2	900	27	3264	88128	1.23	29889			118017
Concrete	1440	CY	C-20	1	150	10	2860	27456	109	156960	600	5760	190176
Reinf.	100	Ton	4 Rodm	2	5.5	18	2752	50036	935	93500			143536
CRANE					Days:	261					300	78280	78280
											COST OF SYSTEM:		$3,847,443
											TIME TO CONSTRUCT:		261 Days

COMPOSITE STEEL SYSTEM - REDESIGN													
COLUMNS	Amount	Unit	Crew	# Crews	Units/Day	Days	Labor Cost/Day	Labor	Mat'l Cost/Unit	Mat'l	Equip. Cost/Day	Equip.	TOTAL COST
Steel	1701	CWt	E-6	1	250	7	5091	34639	53	90153			124792
Baseplates	36		E-6	1	60	1	5091	3055	45	1620			4675
Fireproofing	30000	SF	G-2	1	1500	20	900	18000	1	30000			48000
FLOORS	Amount	Unit	Crew	# Crews	Units/Day	Days	Labor Cost/Day	Labor	Mat'l Cost/Unit	Mat'l	Equip. Cost/Day	Equip.	TOTAL COST
Framing	3263	CWt	E-6	1	250	13	5091	66448	53	172939			239387
Steel Deck	159000	SF	E-4	3	10140	16	7968	124942	2	318000			442942
Shear Studs	8863	Studs	E-10	1	950	9	1060	9889	1	8863			18752
Fireproofing	159000	SF	G-2	2	3000	53	1800	95400	1	159000			254400
Concrete	4907	CY	C-20	2	300	16	5720	93560	109	534863			628423
WWF	1000	CSF	2 Rodmn	4	108	9	2752	25481	29	29000			54481
Slab Finish	159000	SF	CeFl	6	3000	53	4956	262668					262668
BRACES	Amount	Unit	Crew	# Crews	Units/Day	Days	Labor Cost/Day	Labor	Mat'l Cost/Unit	Mat'l	Equip. Cost/Day	Equip.	TOTAL COST
HSS Steel	2132	CWt	E-6	1	250	9	5091	43416	59	125788			169204
Fireproofing	3000	SF	G-2	1	1500	2	900	1800	1	3000			4800
CRANE					Days:	208					300	62283	62283
											COST OF SYSTEM:		$2,314,807
											TIME TO CONSTRUCT:		208 Days

Floor Framing Takeoffs

TOTAL STRUCTURE GRAVITY BEAM TAKEOFF

Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	121	1613.01	16247
W10X12	48	1097.67	13222
W12X14	126	2923.66	41386
W12X16	5	128.35	2057
W14X22	21	548.10	12104
W16X26	25	594.08	15525
W16X31	10	238.00	7394
W18X35	100	2643.00	92633
W18X40	60	1620.00	65048
	-----		-------------
	516		**265617**

Total Number of Studs = **8863**

TOTAL STRUCTURE JOIST SELECTION TAKEOFF

Standard Joists:

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
10K1	36	374.92	1875
12K1	3	39.00	195
18K3	3	60.66	400
18LH07	5	100.00	1700
18LH08	10	203.30	3863
20K3	1	20.33	136
18K4	27	540.66	3893
20K5	3	77.01	631
20LH10	60	1331.70	30629
22K6	1	25.67	236
22K7	56	1455.34	14117
24K4	5	130.00	1092
24K7	3	86.01	869
28K7	3	90.99	1074
	-----		-------------
	666		**60710**

Lateral Bracing System Takeoffs

TOTAL STRUCTURE FRAME TAKEOFF

Floor Area (ft**2): 151997.1

Columns:

Wide Flange:

Steel Grade: 50

Size	#	Length ft	Weight lbs	UnitWt psf
W10X33	129	1846.9	61022	
W10X39	9	135.0	5283	
W10X49	39	585.0	28665	
W10X45	6	90.0	4073	
W10X54	9	135.0	7258	
W10X60	6	90.0	5390	
W12X40	6	84.7	3370	
W12X45	9	127.0	5661	
W12X65	15	225.0	14623	
W12X50	12	169.3	8412	
W12X87	12	180.0	15680	
	252		159436	1.05

Beams:

Wide Flange:

Steel Grade: 50

Size	#	Length ft	Weight lbs	UnitWt psf
W8X10	126	1379.9	13898	
W12X14	36	568.0	8040	
W12X19	6	117.0	2218	
W14X22	6	117.0	2584	
W16X26	6	125.0	3266	
W18X35	12	294.0	10304	
	192		40309	0.27

Braces:

Tube:

Steel Grade: 36

Size	#	Length ft	Weight lbs	UnitWt psf
HSS14X6X3/8	276	4748.1	213265	
	276		213265	1.40

Column Takeoffs

Size	#	Length (ft)	Weight (lbs)
W10X33	8	338.6	11189
W12X40	7	296.3	11797
W12X45	21	888.9	39625
W10X49	7	315.0	15435
W10X54	1	45.0	2419
W12X65	7	315.0	20473
W12X72	15	675.0	48464
W12X79	6	270.0	21315
	72		170717

Baseplate Takeoffs

Column Line	Column Size	(ksi)	N (in)	B (in)	tp (in)
A - 7	W12X72	36	14.50	14.00	1.375
A - 8	W12X72	36	14.50	14.00	1.375
A - 9	W12X72	36	14.50	14.00	1.375
A - 10	W12X72	36	14.50	14.00	1.375
A - 12	W12X72	36	14.50	14.00	1.375
A - 13	W12X72	36	14.50	14.00	1.375
A - 14	W12X72	36	14.50	14.00	1.375
A - 15	W12X65	36	14.25	14.00	1.250
A - 16	W12X65	36	14.25	14.00	1.250
A - 18	W10X49	36	12.00	12.00	1.000
B - 2	W10X49	36	12.00	12.00	0.875
C - 6	W10X54	36	12.25	12.00	1.000
C - 7	W12X79	36	15.00	14.25	1.375
C - 9	W12X79	36	15.00	14.25	1.375
C - 10	W12X65	36	14.25	14.00	1.250
C - 15	W12X79	36	14.50	14.25	1.375
D - 1	W10X49	36	12.00	12.00	0.875
F - 2	W10X49	36	12.00	12.00	1.000
F - 5	W12X65	36	14.25	14.00	1.250
F - 6	W12X72	36	14.50	14.00	1.375
F - 7	W12X79	36	15.00	14.25	1.375
F - 9	W12X79	36	15.00	14.25	1.375
F - 10	W12X65	36	14.25	14.00	1.250
F - 15	W12X79	36	14.50	14.25	1.375
G - 1	W10X49	36	12.00	12.00	0.875
H - 2	W10X49	36	12.00	12.00	0.750
I - 7	W12X72	36	14.50	14.00	1.375
I - 8	W12X72	36	14.50	14.00	1.375
I - 9	W12X72	36	14.50	14.00	1.375
I - 10	W12X72	36	14.50	14.00	1.375
I - 12	W12X72	36	14.50	14.00	1.375
I - 13	W12X72	36	14.50	14.00	1.375
I - 14	W12X72	36	14.50	14.00	1.375
I - 15	W12X65	36	14.25	14.00	1.250
I - 16	W12X65	36	14.25	14.00	1.250
I - 18	W10X49	36	12.00	12.00	0.875