Appendix A

Gravity Load Documentation

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Live Loads

Area	Designed (psf)	ASCE 7-05 (psf)
Corridors (1st level)	100	100
Corridor (above 1st)	100	80
Lobbies	100	100
Marquees / Canopies	75	75
Mech. Rooms	150 (U)	125
Offices	80 + 20 (partition)	50 + 20 (partitions)
Parking garage	50 *	40 *
Retail - First Floor	100	100
Stairs / Exitways	100 (U)	100
Storage (Light)	125 (U)	125

Notes:

(U) = unreducible

* = 50 psf is truck / bus load where 40 psf is vehicular load

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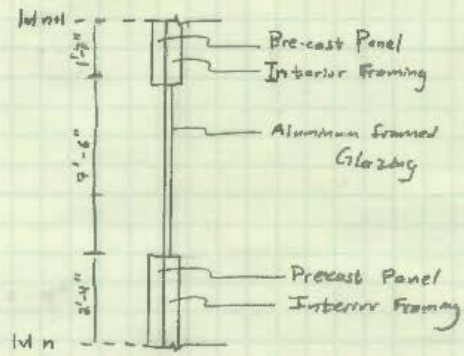
Façade Load

Typical Wall section:



Precast Panels : 5" thick [on average] with LWC

Glazing : 15 psf [assumed]

Interior Framing/wall : 5 psf [assumed]

Line Load = $4'\text{-}1"\,(5/12)(110+5) + 7'\text{-}6"\,(15)$

$= 347.3$ plf

Note: original design assumed 500 plf

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Exposure Category - B
Occupancy - II

Exposure factor	$C_e = 1.0$	[Table 7-2]
Thermal factor	$C_t = 1.0$	[Table 7-3]
Importance factor	$I_s = 1.0$	[Table 7-4]

Flat Roof Snow Loads, p_f

$$p_f = 0.7\, C_e C_t I_s p_g$$

p_g = ground snow load = 25 psf

$$p_f = 0.7(1.0)(1.0)(1.0)(25)$$

$$p_f = 17.5 \text{ psf}$$

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Snow Drift

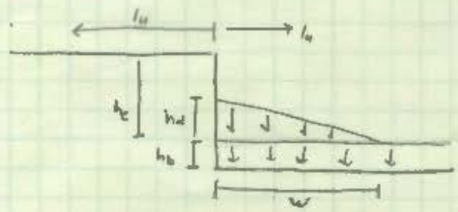
Snow density

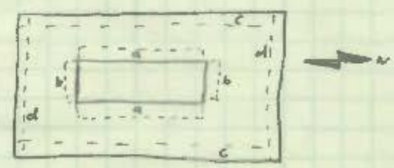
$\gamma = 0.13 p_g + 14 \leq 30 pcf$ [Eq 7-3]

Height of Snow drift, hd

Leeward: $h_d = (3/4)[0.43 (l_u)^{1/3} (p_g + 10)^{1/4} - 1.5]$ [Fig 7-9]

windward: $h_d = (3/4)[0.43 (l_u)^{1/3} (p_g + 10)^{1/4} - 1.5]$ [Sect 7.7.1]





Value of "w"

- if $h_d \leq h_c$ ∴ $w = 4h_d$
- if $h_d > h_c$ ∴ $w = 4h_d^2/h_c$

Area	hd	w
a	2.4'	9.6'
b	4.0'	16'
c	[2.7']	15.7'
d	[3.5']	26.1'

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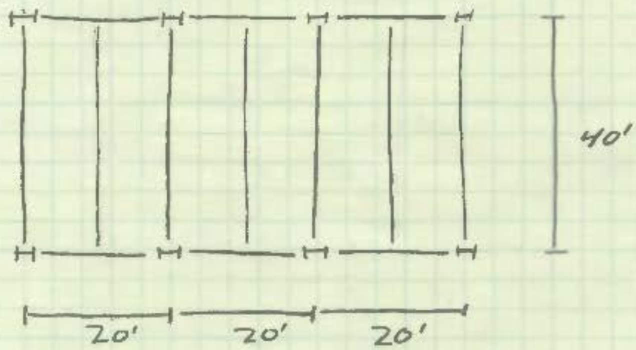
Floor Beam Layout



Loads

- LL = 80 psf + 20psf (partitions)
- SDL = 5 psf
- Facade load = 500 plf

Layout / Design

- Span direction minimizes floor depth
- Span direction minimizes number of connections
- Span provides 10'-0" unshored spans
- Layout / Design best suited for composite design (see tech III)
- Use composite decking -
  3 1/4 LW on 2VLI20 with 6x6 - W1.4xW1.4

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Deck Comparisons

Free construction Cost Data

Unprotected Deck - composite

- 3¼ LW on 2VLI20
  - Superimposed LL - 142 psf
  - Total thickness - 5¼"
  - Topping thickness - 3¼"
  - Max Unshored Span - 10'-11"
  - Material Costs - \$3.85 /sf
  - Labor Costs - \$0.93 /sf
  - Material weight - 42 psf
  - Material Volume - 0.36 ft^3/ft^2

- 4½" NW on 2VLI 19
  - Superimposed LL - 193 psf
  - Total thickness - 6½"
  - Topping thickness - 4½"
  - Max Unshored Span - 10'-0"
  - Material Costs - \$3.88/sf
  - Labor Costs - \$1.00/sf
  - Material Weight - 69 psf
  - Material Volume - 0.46 ft^3/ft^2

Sprayed Fiber Deck - composite

- 2½" LW on 2VLI20
  - Superimposed LL - 117 psf
  - Total thickness - 4.875"
  - Topping thickness - 2½"
  - Fire Proof thickness - 3/8" (≈ 22pcf)
  - Max Unshored Span - 11'-7"
  - Material Costs - \$4.27 /sf
  - Labor Costs - \$1.62 /sf
  - Material Weight - 33 psf
  - Material Volume - 0.28 ft^3/ft^2

- 2½" NW on 2VLI20
  - Superimposed LL - 112 psf
  - Total thickness - 4.875"
  - Topping thickness - 2½"
  - Fire Proof thickness - 3/8" (≈ 22 pcf)
  - Max Unshored Span - 10'-7"
  - Material Costs - \$4.12 /sf
  - Labor Costs - \$1.64/sf
  - Material weight - 45 psf
  - Material Volume - 0.31 ft^3/ft^2

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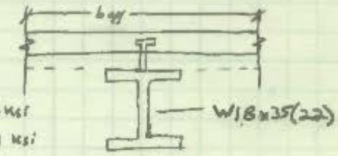
Typical Gravity Beam Check



$F_y = 50$ ksi
$f'_c = 3$ ksi

$w_u = [1.2(42+5)(10) + 1.2(35) + 1.6(20+62.4)(10)] = 1.924$ klf

LL reduction $= 0.25 + 15/\sqrt{800} = 78.0\%$

$b_{eff} = 2\times\left[\min\begin{cases} 40/2 \times 12 \\ 40\times 12/8 \end{cases}\right] \begin{matrix} = 240'' \\ = 120'' \end{matrix}$ ← controls

$M_u = \frac{w_u l^2}{8} = \frac{1.924(40)^2}{8} = 384.8$ kip-ft

Max concentrated force $= 0.85(3\text{ ksi})(120\text{ in})(3.25'') = 995$ kips

Stud strength = 17.1 kips per stud [65 ksi, 3.5", 3/4" diam]

$\Sigma Q_n = (17.1)(11) = 188.1$ kips [11 studs per side]

$a_{act} = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{188.1}{0.85(3)(120)} = 0.615''$ $\therefore Y_2 = 4.675''$

Consider table 3-19

capacity = 395 kip-ft > 384.8 kip-ft ∴ ok

- check unshored strength : $\phi M_p = 249$ kip-ft

$w_u = 1.4D + 1.4(42+5)(10) + 1.4(35) = 0.707$ klf

$w_u = 1.2D + 1.6L = 1.2(47)(10) + 1.2(35) + 1.6(20)(10) = 0.926$ klf

$M_u = \frac{w_u l^2}{8} = \frac{(0.926)(40)^2}{8} = 185.2$ kip-ft < 249 kip-ft ∴ ok

- check wet concrete deflection : $\Delta_{wc} = L/240 = 2''$

$W_{wc} = 42(10) + 35 = 455$ plf

$\Delta_{wc} = \frac{5wL^4}{384EI_x} = \frac{5(0.455)(40)^4(12)^3}{384(29000)(510)} = 1.77'' < 2''$ ∴ ok

- check LL deflection : $\Delta_{LL} = L/360 = 1\frac{1}{3}''$

$W_{LL} = 62.5\%(100\text{ psf})(10) = 0.625$ klf $I_{eff} = 1249$

$\Delta_{LL} = \frac{5wL^4}{384EI_{LB}} = \frac{5(0.625)(40)^4(12)^3}{384(29000)(1249)} = 1.31'' < 1.33''$ ∴ ok

Summary : W18x35(22) is adequate for the design

Gravity Beam Design

RAM
RAM Steel v14.05.03.00
DataBase: RMP T Model 12 02/13/14 14:57:41
Building Code: IBC Steel Code: AISC 360-10 LRFD

Floor Type: Typical **Beam Number = 78**

SPAN INFORMATION (ft): I-End (50.00,0.00) J-End (50.00,40.00)

Beam Size (User Selected) = W18X35 Fy = 50.0 ksi
Total Beam Length (ft) = 40.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Deck Label	Typical Flooring	Typical Flooring
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	perpendicular	perpendicular
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	= 120.00	Y bar(in)	=	18.13
Mnf (kip-ft)	= 569.01	Mn (kip-ft)	=	445.40
C (kips)	= 189.53	PNA (in)	=	15.17
Ieff (in4)	= 1249.74	Itr (in4)	=	1729.41
Stud length (in)	= 3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.2 Rg = 1.00 Rp = 0.60
# of studs: Full = 70 Partial = 40 Actual = 22
Number of Stud Rows = 1 Percent of Full Composite Action = 36.80

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.427	0.427	0.000	---	NonR	0.000	0.000
	40.000	0.427	0.427	0.000			0.000	0.000
2	0.000	0.050	0.000	0.800	22.0%	Red	0.200	0.200
	40.000	0.050	0.000	0.800			0.200	0.200
3	0.000	0.035	0.035	0.000	---	NonR	0.000	0.000
	40.000	0.035	0.035	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 38.67 kips 1.00Vn = 159.30 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	175.0	20.0	0.0	1.00	0.90	249.37
	Init DL	1.4DL	129.5	20.0	---	---		
	Max +	1.2DL+1.6LL	386.7	20.0	---	---	0.90	400.86
Controlling		1.2DL+1.6LL	386.7	20.0	---	---	0.90	400.86

REACTIONS (kips):

	Left	Right
Initial reaction	13.25	13.25
DL reaction	10.25	10.25
Max +LL reaction	16.49	16.49
Max +total reaction (factored)	38.67	38.67

DEFLECTIONS: (Camber = 1-1/4)

Initial load (in)	at	20.00 ft	=	-1.801	L/D =	267
Live load (in)	at	20.00 ft	=	-1.310	L/D =	366
Post Comp load (in)	at	20.00 ft	=	-1.389	L/D =	345
Net Total load (in)	at	20.00 ft	=	-1.940	L/D =	247

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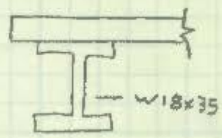
Typical Gravity Girder Check

[W18x35 selected for connection purposes]



(12" overhang)

$P_u = [1.2(42+5+3.5)+1.6(20+63.4)(10)+1.2(40)]\,20$

$P_u = 39.45$ kips

$LL_{red} = 0.25 + 15/\sqrt{800} = 78\%$

$P_{\not{c}} = 5$ kips @ 1.5' from each end (facade)

$1.2 \times 5 = 6$ kips

Note: controlling load combo:
1.2D + 1.6L

$V_u = 39.45/2 + 6 + (20')(1.6 \times 100 + 1.2[5+35+42]) = 29.69$ kip

$M_u = \left(\frac{39.45}{2}\right)(10') + (6)(1.5') + \frac{(1.6 \times 100 + 1.2[82])(20)^2}{8} = 219.17$ kip-ft

$\phi M_p = 249$ kips-ft

$\phi V_v = 159$ kips

strength: $\phi V_v > V_u$ & $\phi M_p > M_u$ $\therefore$ ok

$\Delta_{LL} = \frac{5(100)(20)^4(12)^3}{384(29000)(510)} + \frac{(82.4 \times 20)(20)^3(12)^3}{48(29000)(510)} = 0.35"$

$\Delta_{LL} = \ell/360 = 20 \times 12/360 = 2/3"$

servicibility: $\Delta_{LL} < \Delta_{LL\,allowed}$ $\therefore$ ok

Summary: W18x35 is adequate for design

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: RMP T Model walled 2 02/25/14 13:01:37
Building Code: IBC Steel Code: AISC 360-10 LRFD

Floor Type: Typical **Beam Number = 58**

SPAN INFORMATION (ft): I-End (100.00,120.00) J-End (120.00,120.00)

Beam Size (Optimum) = W18X35 Fy = 50.0 ksi
Total Beam Length (ft) = 20.00
Mp (kip-ft) = 277.08

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL
10.000	9.73	15.20	1.2	0.00	0.00	0.0	0.00	0.0	3.80
18.500	5.00								
1.500	5.00								

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.036	0.000	---	NonR	0.000
	20.000	0.036	0.000			0.000
2	0.000	0.004	0.067	1.2%	Red	0.017
	20.000	0.004	0.067			0.017
3	0.000	0.035	0.000	---	NonR	0.000
	20.000	0.035	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 29.12 kips 1.00Vn = 159.30 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu	@	Lb	Cb	Phi	Phi*Mn
			kip-ft	ft	ft			kip-ft
Center	Max +	1.2DL+1.6LL	229.1	10.0	10.0	1.60	0.90	249.37
Controlling		1.2DL+1.6LL	229.1	10.0	10.0	1.60	0.90	249.37

REACTIONS (kips):

	Left	**Right**
DL reaction	10.62	10.62
Max +LL reaction	10.24	10.24
Max +total reaction (factored)	29.12	29.12

DEFLECTIONS:

Dead load (in)	at	10.00 ft =	-0.251	L/D	=	955
Live load (in)	at	10.00 ft =	-0.387	L/D	=	621
Net Total load (in)	at	10.00 ft =	-0.638	L/D	=	376

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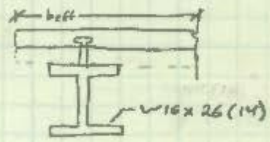
Typical Exterior Girder Check



$P_u = [1.2(42+5+3.5)10 + 1.6(20+62.4)(10) + 1.2(40)](30)$
$= 39.45$ kips

LL red = $0.25 + 15/\sqrt{800} = 78\%$

P_e = 5 kips @ 1.5' from each end (facade)
$1.2 \times 5 = 6$ kips

M_u = 208.6 kip-ft
(Risa 2D analysis)

$b_{eff} = \min \begin{cases} 40/2 \times 12 + 10 = 250" \\ 20 \times 12/8 + 10 = 40" \leftarrow \text{controls} \end{cases}$

[Assume 10" overhang]

Max Concrete force = 0.85(3ksi)(40")(3.25) = 331.5 kips

Stud strength = 17.1 kips per stud [65 ksi, 3.5 in, 3/4" diameter]

$\Sigma Q_n = (17.1)(7) = 119.7$ kips [7 studs per side]

$a_{act} = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{119.7}{0.85(3)(40)} = 1.17"$ ∴ let $Y_2 = 4.66"$

Consider Table 3-19: capacity = kip-ft > 208.2 kip-ft ∴ ok

– check unshored strength : $\phi M_p = 166$ kip-ft

1.4D → M_u = 84 kip-ft
1.6L + 1.2D → M_u = 98.6 kip-ft > 166 kip-ft ∴ ok

– check wet concrete deflection : $\Delta_{wc} = L/240 = 1"$

W_{wc} = 35 plf $P_{wc} = 35(20) + 42(10)(20) = 9.1$ kips

$\Delta_{wc} = \frac{PL^3}{48EI} + \frac{5wL^4}{384EI_x} = \frac{(9.1)(20)^3(12)^3}{48(29000)(301)} + \frac{5(0.035)(20)^4(12)^3}{384(29000)(301)} = 0.31" < 1"$ ∴ ok

– check LL deflection : $\Delta_{LL} = L/360 = 2/3"$

$P_{LL} = (82.4)(20)(10) = 16.5$ kips $I_{LB} = 584$ [by interpolation – table 3-20]

$\Delta_{LL} = \frac{PL^3}{48EI} = \frac{(16.5)(20)^3(12)^3}{48(29000)(584)} = 0.28" < 0.66"$ ∴ ok

Summary : W16x26 (14) is adequate for design

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: RMP T Model 32 02/20/14 16:46:27
Building Code: IBC Steel Code: AISC 360-10 LRFD

Floor Type: Typical **Beam Number = 58**

SPAN INFORMATION (ft): I-End (100.00,120.00) J-End (120.00,120.00)

Beam Size (User Selected) = W16X26 Fy = 50.0 ksi
Total Beam Length (ft) = 20.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Deck Label	Typical Flooring	Typical Flooring
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	parallel	parallel
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	=	40.00	Y bar(in)	=	14.34
Mnf (kip-ft)	=	351.14	Mn (kip-ft)	=	287.64
C (kips)	=	123.78	PNA (in)	=	12.80
Ieff (in4)	=	655.77	Itr (in4)	=	881.59
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.7 Rg = 1.00 Rp = 0.75
\# of studs: Full = 38 Partial = 20 Actual = 14
Number of Stud Rows = 1 Percent of Full Composite Action = 37.34

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL	CLL
10.000	9.73	8.78	15.20	1.2	0.00	0.00	0.0	0.00	0.0	3.80	3.80
18.500	5.00	0.00									
1.500	5.00	0.00									

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.036	0.036	0.000	---	NonR	0.000	0.000
	20.000	0.036	0.036	0.000			0.000	0.000
2	0.000	0.004	0.000	0.067	1.2%	Red	0.017	0.017
	20.000	0.004	0.000	0.067			0.017	0.017
3	0.000	0.026	0.026	0.000	---	NonR	0.000	0.000
	20.000	0.026	0.026	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 29.02 kips 0.90Vn = 105.97 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	88.2	10.0	10.0	1.64	0.90	165.75
	Init DL	1.4DL	65.8	10.0	---	---		
	Max +	1.2DL+1.6LL	228.5	10.0	---	---	0.90	258.88
Controlling		1.2DL+1.6LL	228.5	10.0	---	---	0.90	258.88

REACTIONS (kips):

	Left	Right
Initial reaction	7.08	7.08
DL reaction	10.53	10.53
Max +LL reaction	10.24	10.24
Max +total reaction (factored)	29.02	29.02

DEFLECTIONS:

Initial load (in)	at	10.00 ft	=	-0.315	L/D	=	761
Live load (in)	at	10.00 ft	=	-0.301	L/D	=	798
Post Comp load (in)	at	10.00 ft	=	-0.350	L/D	=	686
Net Total load (in)	at	10.00 ft	=	-0.665	L/D	=	361

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Column Spot Checks

Column at Grid C-3, Level 1 (Interior Column)

P_u = 928 kips

AISC Table 4-1 : 17' unbraced length

Use W12 x 120 → ϕP_n = 1160 kips > 928 kips ∴ ok

Summary: W12x120 is adequate for design

Column at Grid F-1, Level 3 (exterior Column)

P_u = 580 kips M_{uy} = 9.82 kip-ft M_{ux} = 2.82 kip-ft

check 12 x 65 [Table 6-1] 14' unbraced length

$p = 1.46 \times 10^{-3}$

$b_y = 5.53 \times 10^{-3}$ $b_x = 2.58 \times 10^{-3}$

$pP_u = 1.46 \times 10^{-3}(580) = 0.847 \geq 0.2$ ∴ Equation H1-1a applies

$pP_u + b_x M_{rx} + b_y M_{ry} \leq 1.0$

$0.847 + (2.82)(2.58 \times 10^{-3}) + (9.82)(5.53 \times 10^{-3})$

$= 0.908 \leq 1.0$ ∴ ok

Summary: W12 x 65 is adequate for design

Gravity Column Design

RAM Steel v14.05.03.00 Page 2/2
DataBase: RMP T Model 42 02/20/14 19:30:54
Building Code: IBC Steel Code: AISC 360-10 LRFD

Story level Garage, Column Line C-3

Fy (ksi)	=	50.00	Column Size	=	W12X120
Orientation (deg.)	=	0.0			

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		16.75	16.75
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	9.05	8.65
	Bottom	0.00	0.00

CONTROLLING COLUMN LOADS - Skip-Load Case 1:

			Dead	Live	Roof
Axial (kip)			380.02	294.94	0.00
Moments	Top	Mx (kip-ft)	0.88	0.31	0.00
		My (kip-ft)	1.62	0.56	0.00
	Bot	Mx (kip-ft)	0.00	0.00	0.00
		My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL + 0.5RF)

Pu (kip)	=	927.92	0.90*Pn (kip)	=	1174.14
Mux (kip-ft)	=	1.55	0.90*Mnx (kip-ft)	=	697.50
Muy (kip-ft)	=	2.84	0.90*Mny (kip-ft)	=	320.25
Rm	=	1.00			
Cbx	=	1.67			
Cmx	=	0.60	Cmy	=	0.60
Pex (kip)	=	7580.35	Pey (kip)	=	2444.13
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.790
Eq H1-1a: 0.790 + 0.002 + 0.008 = 0.800

Gravity Column Design

RAM Steel v14.05.03.00 Page 2/2
DataBase: RMP T Model 42 02/20/14 19:37:30
Building Code: IBC Steel Code: AISC 360-10 LRFD
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Story level Office 5th Floor, Column Line F-1

Fy (ksi)	= 50.00	Column Size	= W12X65
Orientation (deg.)	= 0.0		

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	13.67	13.67
K	1	1
Braced Against Joint Translation	Yes	Yes
Column Eccentricity (in) Top	8.55	8.50
Bottom	8.55	8.50

CONTROLLING COLUMN LOADS - Skip-Load Case 2:

	Dead	Live	Roof
Axial (kip)	274.15	156.73	0.00
Moments Top Mx (kip-ft)	0.00	0.00	0.00
My (kip-ft)	-3.48	-3.53	0.00
Bot Mx (kip-ft)	0.00	1.76	0.00
My (kip-ft)	-3.18	0.00	0.00

Single curvature about X-Axis
Reverse curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL + 0.5RF)

Pu (kip)	=	579.74	0.90*Pn (kip)	=	692.55
Mux (kip-ft)	=	2.82	0.90*Mnx (kip-ft)	=	356.22
Muy (kip-ft)	=	9.82	0.90*Mny (kip-ft)	=	160.81
Rm	=	1.00			
Cbx	=	1.67			
Cmx	=	0.60	Cmy	=	0.44
Pex (kip)	=	5669.25	Pey (kip)	=	1850.75
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.837
Eq H1-1a: 0.837 + 0.007 + 0.054 = 0.898

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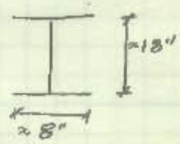
Redesign - Fire Proofing [2 Hr, Rating : 3/4" - 1"]

Average W18x55 for beams



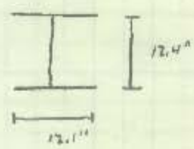
Surface Area = 4+18+4+8+4+18+4 = 60"

60"/12" = 5' → 5 sf per lineal ft. of beam

Total length of beams ≈ 31842 sf

31268 (5) = 159,210 sf fire proofing on beams

Average W12x79 for Column [3 Hr Rating: 2 3/16"]



Surface area = 2 (12.4) + 4 (12.1) = 73.2"

74/12" = 6.2' → 6.2 sf per lineal ft of column

Total length of column = 6075 ft

6075 (6.2) = 37666 sf fire proofing on beams

Total Area of Fire Proofing Required = 196,876 sf

[Note: Deck need not be sprayed]

Appendix B

Wind Loading Calculations

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Calculation For Wind Analysis

ASCE 7-05 →

Method 2 → Building Meets req 6.5.1

Basic Wind Speed

Rockville, MD $V = 90$ mph [Fig. 6-1]

Directionality Factor

$K_d = 0.85$ [Table 6-4]

Importance Factor

$I_w = 1.0$ [Table 6-1]

Exposure Category : B

Topographic Factor

$K_{zt} = 1.0$ [Sect. 6.5.7]

Determine Velocity Pressure Exposure Coefficient

K_z, K_h → See calc tables for values [Table 6-3]

Determine Velocity Pressures

$q_z, q_h = 0.00256 K_z K_{zt} K_d V^2 I$ [Eq. 6-15]

Determine Building Enclosure : Fully Enclosed [Sect. 6.5.9]

$G C_{pn} = +1.5$ windward
$G C_{pn} = -1.0$ leeward [Eq 6-20]

Combined Net Design Pressure (Internal Pressure)

$P_i = q_h G C_{pi}$ → see calc tables for values [Sect. 6.5.11.1]

Determine Pressure coefficients

$C_p = 0.8$ (windward), -0.5(leeward) [Fig 6-6]
$G C_{pi} = \pm 0.18$ [Fig 6-5]

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Determine Gust Effect Factor [Sect 6.5.8]

$$G = 0.925\left(\frac{(1+1.7I_{\bar{z}}\sqrt{g_Q^2Q^2+g_R^2R^2})}{(1+1.7g_vI_{\bar{z}})}\right) \quad [\text{Eq } 6\text{-}8]$$

$$I_{\bar{z}} = c\,(33/\bar{z})^{1/6} \quad [\text{Eq } 6\text{-}5]$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \quad [\text{Eq } 6\text{-}6]$$

$$L_{\bar{z}} = \ell\,(\bar{z}/10)^{\bar{\epsilon}} \quad [\text{Eq } 6\text{-}7]$$

$$\bar{V}_{\bar{z}} = \bar{b}\,(\bar{z}/33)^{\bar{\alpha}}\,V(88/60) \quad [\text{Eq } 6\text{-}14]$$

$$N_1 = (n_1 L_{\bar{z}})/(\bar{V}_{\bar{z}}) \quad [\text{Eq } 6\text{-}12]$$

$$R_n = 7.47N_1/(1+10.3N_1)^{5/3} \quad [\text{Eq } 6\text{-}11]$$

$$R_\ell = 1/\eta - 1/2\eta^2\,(1-e^{-2\eta}) \quad [\text{Eq } 6\text{-}13a]$$

$R_\ell = R_h$ for $\eta = 4.6n_1h/\bar{V}_{\bar{z}}$

$R_\ell = R_B$ for $\eta = 4.6n_1B/\bar{V}_{\bar{z}}$

$R_\ell = R_L$ for $\eta = 15.4n_1L/\bar{V}_{\bar{z}}$

$$R = \sqrt{(1/\beta)R_nR_hR_B(0.53+0.47R_L)} \quad [\text{Eq } 6\text{-}10]$$

$$n_1\,(\text{approx}) = 100/H \quad [\text{Eq } C6\text{-}17]$$

$$g_R = \sqrt{2\ln(3{,}600n_1)}+0.577/(\sqrt{2\ln(3{,}600n_1)}) \quad [\text{Eq } 6\text{-}9]$$

$$g_Q = g_v = 3.4 \quad [\text{Sect } 6.5.8.2]$$

Note building is considered flexible by sect 6.2

Determine Design Wind Pressures

Windward : $P_z = q_zG_fC_p - q_h(GC_{pi})$ [Eq 6-19]

Leeward : $P_h = q_hG_fC_p - q_h(GC_{pi})$ [Eq 6-19]

See calc tables for results

Wind: East-West Direction

Table 18: East-West Design Factors	
Exposure B	
Case 2	
L	120 ft
B	210 ft
L/B	0.571
Natural Period (approx.) (n_1)	0.833
Damping Coeff. (approx.) (β)	0.02
Basic Wind Speed (V)	90 mph
Wind Directionality Factor (K_d)	0.85
Importance Factor (I)	1.0
Exposure Category	B
Topographical Factor (K_{zt})	1.0
Gust Effect Factor (G)	0.825
C_p Windward	0.8
C_p Leeward	-0.5
G_{cpi} Windward	0.18
G_{cpi} Leeward	-0.18
G_{pn} Windward	1.5
G_{pn} Leeward	-1.0

Table 19: East-West Calculation of Design Pressures								
	Height	**K_z, K_h**	**q_z, q_h**	**External Pressure**	**Internal Pressure**	**Net Positive**	**Net Negative**	**Total Pressure**
	(ft)			(psf)	(psf)	(psf)	(psf)	(psf)
Penthouse	150.33	1.11	19.57	12.92	3.52	9.39	16.44	20.64
	139.75	1.09	19.17	12.65	3.52	9.13	16.17	20.38
Main Roof	129.17	1.06	18.74	12.37	3.52	8.84	15.89	20.09
	122.88	1.05	18.47	12.19	3.52	8.67	15.71	19.92
11th	116.58	1.03	18.20	12.01	3.52	8.49	15.53	19.74
	110.29	1.02	17.91	11.82	3.52	8.30	15.34	19.55
10th	104.00	1.00	17.61	11.62	3.52	8.10	15.15	19.35
	97.71	0.98	17.30	11.42	3.52	7.90	14.94	19.15
9th	91.42	0.96	16.98	11.20	3.52	7.68	14.73	18.93
	85.13	0.94	16.63	10.98	3.52	7.46	14.50	18.71
8th	78.83	0.92	16.27	10.74	3.52	7.22	14.26	18.47
	72.54	0.90	15.89	10.49	3.52	6.97	14.01	18.21
7th	66.25	0.88	15.49	10.22	3.52	6.70	13.74	17.95
	59.96	0.85	15.05	9.93	3.52	6.41	13.45	17.66
6th	53.67	0.83	14.58	9.62	3.52	6.10	13.15	17.35
	47.38	0.80	14.07	9.29	3.52	5.76	12.81	17.01
5th	41.08	0.77	13.51	8.92	3.52	5.39	12.44	16.64
	34.25	0.73	12.82	8.46	3.52	4.94	11.99	16.19
4th	27.42	0.68	12.03	7.94	3.52	4.42	11.46	15.67
	22.08	0.64	11.31	7.47	3.52	3.94	10.99	15.19
P6	16.75	0.59	10.45	6.90	3.52	3.38	10.42	14.63
	8.38	0.57	10.05	6.63	3.52	3.11	10.15	14.36
Plaza Level	0.00	0.57	10.05	6.63	3.52	3.11	10.15	14.36
Leeward	129	1.06	18.73	-7.73	3.52	-11.25	-4.20	-

Table 20: East-West Design Pressures							
	Height	**Windward Pressure**	**Leeward Pressure**	**Total Pressure**	**Total Force**	**Story Shear**	**Moment Windward**
	(ft)	(psf)	(psf)	(psf)	(kips)	(kips)	(k-ft)
Penthouse	150.33	12.92	-7.73	20.64	29.49	29.49	4433.68
	139.75	12.65	-7.73	20.38			
Main Roof	129.17	12.37	-7.73	20.09	61.58	91.07	7953.77
	122.88	12.19	-7.73	19.92			
11th	116.58	12.01	-7.73	19.74	51.91	142.98	6051.35
	110.29	11.82	-7.73	19.55			
10th	104.00	11.62	-7.73	19.35	50.86	193.84	5289.95
	97.71	11.42	-7.73	19.15			
9th	91.42	11.20	-7.73	18.93	49.73	243.57	4545.84
	85.13	10.98	-7.73	18.71			
8th	78.83	10.74	-7.73	18.47	48.47	292.03	3820.68
	72.54	10.49	-7.73	18.21			
7th	66.25	10.22	-7.73	17.95	47.04	339.08	3116.62
	59.96	9.93	-7.73	17.66			
6th	53.67	9.62	-7.73	17.35	45.40	384.48	2436.49
	47.38	9.29	-7.73	17.01			
5th	41.08	8.92	-7.73	16.64	45.22	429.70	1857.86
	34.25	8.46	-7.73	16.19			
4th	27.42	7.94	-7.73	15.67	39.50	469.20	1083.00
	22.08	7.47	-7.73	15.19			
P6	16.75	6.90	-7.73	14.63	41.63	510.83	697.33
	8.38	6.63	-7.73	14.36			
Plaza Level	0.00	6.63	-7.73	14.36	25.25	**536.08**	0.00
							41286.57

Base Shear	**536.08 Kips**
Overturning Moment	**41286.57 Kip-ft**

Wind: North-South Direction

Table 21: North-South Design Factors	
Exposure B	
Case 2	
L	210 ft
B	120 ft
L/B	1.75
Natural Period (approx.) (n_1)	0.833
Damping Coeff. (approx.) (β)	0.02
Basic Wind Speed (V)	90 mph
Wind Directionality Factor (K_d)	0.85
Importance Factor (I)	1.0
Exposure Category	B
Topographical Factor (K_{zt})	1.0
Gust Effect Factor (G)	0.845
C_p Windward	0.8
C_p Leeward	-0.5
G_{cpi} Windward	0.18
G_{cpi} Leeward	-0.18
G_{pn} Windward	1.5
G_{pn} Leeward	-1.0

Table 22: North-South Calculation of Design Pressures

	Height	K_z, K_h	q_z, q_h	External Pressure	Internal Pressure	Net Positive	Net Negative	Total Pressure
	(ft)			(psf)	(psf)	(psf)	(psf)	(psf)
Penthouse	150.33	1.11	19.57	13.23	3.52	9.70	16.75	18.77
	139.75	1.09	19.17	12.95	3.52	9.43	16.48	18.49
Main Roof	129.17	1.06	18.74	12.67	3.52	9.14	16.19	18.21
	122.88	1.05	18.47	12.49	3.52	8.96	16.01	18.03
11th	116.58	1.03	18.20	12.30	3.52	8.78	15.82	17.84
	110.29	1.02	17.91	12.11	3.52	8.58	15.63	17.65
10th	104.00	1.00	17.61	11.90	3.52	8.38	15.43	17.45
	97.71	0.98	17.30	11.69	3.52	8.17	15.22	17.24
9th	91.42	0.96	16.98	11.47	3.52	7.95	15.00	17.01
	85.13	0.94	16.63	11.24	3.52	7.72	14.77	16.78
8th	78.83	0.92	16.27	11.00	3.52	7.48	14.52	16.54
	72.54	0.90	15.89	10.74	3.52	7.22	14.26	16.28
7th	66.25	0.88	15.49	10.47	3.52	6.94	13.99	16.01
	59.96	0.85	15.05	10.17	3.52	6.65	13.69	15.71
6th	53.67	0.83	14.58	9.85	3.52	6.33	13.38	15.40
	47.38	0.80	14.07	9.51	3.52	5.99	13.03	15.05
5th	41.08	0.77	13.51	9.13	3.52	5.61	12.65	14.67
	34.25	0.73	12.82	8.67	3.52	5.14	12.19	14.21
4th	27.42	0.68	12.03	8.13	3.52	4.61	11.66	13.67
	22.08	0.64	11.31	7.65	3.52	4.12	11.17	13.19
P6	16.75	0.59	10.45	7.07	3.52	3.54	10.59	12.61
	8.38	0.57	10.05	6.79	3.52	3.27	10.31	12.33
Plaza Level	0.00	0.57	10.05	6.79	3.52	3.27	10.31	12.33
Leeward	129	1.06	18.74	-5.54	3.52	-9.06	-2.02	-

Table 23: North-South Design Pressures

	Height	Windward Pressure	Leeward Pressure	Total Pressure	Total Force	Story Shear	Moment Windward
	(ft)	(psf)	(psf)	(psf)	(kips)	(kips)	(kip-ft)
Penthouse	150.33	13.23	-5.54	18.77	10.33	10.33	1552.67
	139.75	12.95	-5.54	18.49			
Main Roof	129.17	12.67	-5.54	18.21	29.02	39.35	3748.32
	122.88	12.49	-5.54	18.03			
11th	116.58	12.30	-5.54	17.84	26.79	66.14	3123.62
	110.29	12.11	-5.54	17.65			
10th	104.00	11.90	-5.54	17.45	26.18	92.32	2723.13
	97.71	11.69	-5.54	17.24			
9th	91.42	11.47	-5.54	17.01	25.52	117.84	2332.75
	85.13	11.24	-5.54	16.78			
8th	78.83	11.00	-5.54	16.54	24.78	142.62	1953.47
	72.54	10.74	-5.54	16.28			
7th	66.25	10.47	-5.54	16.01	23.95	166.57	1586.53
	59.96	10.17	-5.54	15.71			
6th	53.67	9.85	-5.54	15.40	22.99	189.56	1233.60
	47.38	9.51	-5.54	15.05			
5th	41.08	9.13	-5.54	14.67	22.73	212.28	933.72
	34.25	8.67	-5.54	14.21			
4th	27.42	8.13	-5.54	13.67	19.65	231.94	538.81
	22.08	7.65	-5.54	13.19			
P6	16.75	7.07	-5.54	12.61	20.46	252.40	342.72
	8.38	6.79	-5.54	12.33			
Plaza Level	0.00	6.79	-5.54	12.33	12.39	**264.79**	0.00
							20069.35

Base Shear	**264.79 Kips**
Overturning Moment	**20069.35 Kip-ft**

Appendix C

Seismic Loading Calculations

Level Self Weight

Table 24: Penthouse Roof Weight	
Item	**Design Weight (kips)**
Beams	22.4
Columns	5.6
Slab	164
Roofing	156
SDL	39
Shear Wall	60
Façade	103.5
Total	550.5

Table 25: Main Roof Weight	
Item	**Design Weight (kips)**
Beams	102
Slab	1144.4
Columns	49.5
Roofing	728.1
Shear Wall	217.7
Equipment	52.8
SDL	221
Façade	167.6
Total	2683.1

Table 26: Office (11th) Weight	
Item	**Design Weight (kips)**
Beams	106.1
Slab	1144.4
Columns	41.2
Shear Wall	154.7
Partitions	194.6
Equipment	23.7
SDL	110.5
Façade	223.5
Total	1998.7

Table 27: Office (4th) Weight	
Item	**Design Weight (kips)**
Beams	106.1
Slab	1144.4
Columns	62.8
Shear Wall	161.4
Partitions	204.2
Equipment	23.7
SDL	115.3
Façade	223.5
Total	2041.3

Table 28: P6 Level Weight	
Item	**Design Weight (kips)**
Beams	100.7
Slab	1144.4
Columns	70.8
Shear Wall	126
Equipment	2.2
SDL	124.5
Façade	300.0
Total	1868.6

Seismic Calculations

Table 29: Seismic Calculations East - West							
Level	**Story Weight**	**Height**	**$w_x h_x^k$**	**C_{vx}**	**Forces (F_x)**	**Story Shear (V_x)**	**Moments (M_x)**
	(kips)	(ft)			(kips)	(kips)	(k-ft)
Pent Roof	551	150.33	200447.3	0.06	26.8	26.8	4024.104
Main Roof	2683	129.17	817230.5	0.23	109.1	135.9	14096.42
11th Floor	1999	116.58	539617.3	0.15	72.1	208.0	8401.099
10th Floor	1999	104.00	471768.8	0.13	63.0	271.0	6552.037
9th Floor	1999	91.42	405356.8	0.11	54.1	325.1	4948.535
8th Floor	2010	78.83	342519.6	0.10	45.7	370.8	3605.862
7th Floor	2010	66.25	279147.4	0.08	37.3	408.1	2469.638
6th Floor	2010	53.67	217875.6	0.06	29.1	437.2	1561.446
5th Floor	2035	41.08	161031.3	0.04	21.5	458.7	883.4656
4th Floor	2041	27.42	100389.8	0.03	13.4	472.1	367.5514
P6	1869	16.75	51467.7	0.01	6.9	479.0	115.1233
Plaza Level	-	0.00	-	-	-	-	-
Total	21205	-	3586852.03	1.00	479.0	-	47025.28

Table 30: Design Values	
Effective Seismic Weight	21205 kips
Base Shear	479.0 kips
Overturning Moment	47025.3 kips-ft

Seismic Calculations

Table 31: Seismic Calculations North - South							
Level	**Story Weight**	**Height**	$w_x h_x^k$	C_{vx}	**Forces (F_x)**	**Story Shear (V_x)**	**Moments (M_x)**
	(kips)	(ft)			(kips)	(kips)	(k-ft)
Pent Roof	551	150.33	200447.3	0.06	24.7	24.7	3714.558
Main Roof	2683	129.17	817230.5	0.23	100.7	125.4	13012.08
11th Floor	1999	116.58	539617.3	0.15	66.5	192.0	7754.861
10th Floor	1999	104.00	471768.8	0.13	58.2	250.1	6048.034
9th Floor	1999	91.42	405356.8	0.11	50.0	300.1	4567.878
8th Floor	2010	78.83	342519.6	0.10	42.2	342.3	3328.488
7th Floor	2010	66.25	279147.4	0.08	34.4	376.7	2279.666
6th Floor	2010	53.67	217875.6	0.06	26.9	403.6	1441.335
5th Floor	2035	41.08	161031.3	0.04	19.9	423.4	815.5067
4th Floor	2041	27.42	100389.8	0.03	12.4	435.8	339.2782
P6	1869	16.75	51467.7	0.01	6.3	442.1	106.2676
Plaza Level	-	0.00	-	-	-	-	-
Total	21205	-	3586852.03	1.00	442.1	-	43407.95

Table 32: Design Values	
Effective Seismic Weight	21205 kips
Base Shear	442.1 kips
Overturning Moment	43407.9 kips-ft

USGS Design Maps Summary Report

Print View Detailed Report

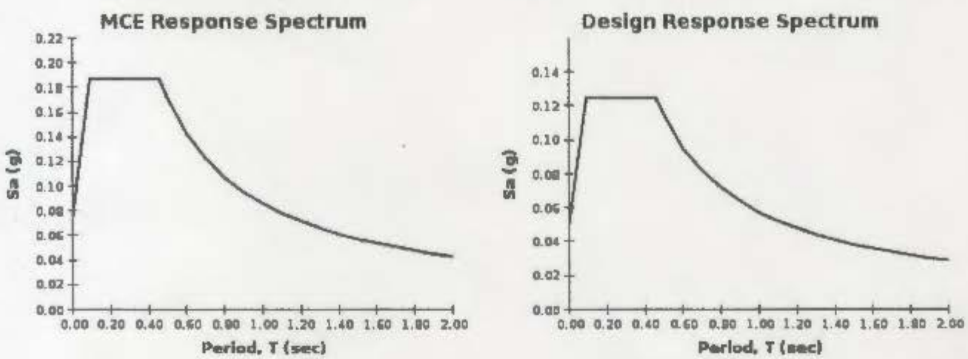
User–Specified Input

Building Code Reference Document	ASCE 7-05 Standard (which utilizes USGS hazard data available in 2002)
Site Coordinates	39.0836°N, 77.1483°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Occupancy Category	I/II/III

USGS–Provided Output

$S_S =$ 0.156 g $S_{MS} =$ 0.187 g $S_{DS} =$ 0.125 g

$S_1 =$ 0.051 g $S_{M1} =$ 0.086 g $S_{D1} =$ 0.057 g



Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

J.M.V. Thesis

Calculation for Seismic Analysis

Not detached 1 or 2 Family Dwelling
Not Agricultural Storage
Not Special Considerations
$\Big\}$ ∴ Not Exempt [Sect 11.1.2]

Seismic Ground Motion Values

$S_S = 0.156\,g$ [Fig 22-1]

$S_1 = 0.051\,g$ [Fig 22-4]

$S_1 > 0.04$ & $S_S > 0.15$ [Sect. 11.4.1]

Determine Soil Site Class → C

$S_{MS} = F_a S_S = (1.2)(0.156)$ [Eq. 11.4-1]

$S_{M1} = F_v S_1 = (1.7)(0.051)$ [Eq. 11.4-2]

$S_{DS} = \frac{2}{3} S_{MS} = 0.1248$ [Eq. 11.4-3]

$S_{D1} = \frac{2}{3} S_{D1} = 0.0578$ [Eq. 11.4-4]

Seismic Design Category

$S_{DS} < 0.167 \rightarrow A$ [TABLE 11.6-1]

$S_{D1} < 0.067 \rightarrow A$ [Table 11.6-2]

Determine Occupancy Category → II

∴ Importance Factor = 1.0 [Table 1-1]

- Section 11.6 requirements for simplified design

- I, II or III → Yes
- $S_1 < 0.75$ → Yes
- $h < 40'$ → No

∴ Simplified does not apply

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Permitted Analytical Procedures → SDC B [Table 12.6-1]

- Equivalent Lateral Force Analysis
- Model Response Spectrum Analysis
- Seismic Response History Procedures

Use Equivalent Lateral Force Analysis

Determine Response Modification Factor - Controling element

X - Ordinary Moment Frames ; $R = 3.25$ [Table 12.2-1]

Y - Ordinary Concentrically Braced Frames
$R = 3.0$

Determine Approx. Fundamental Period

$T_a = C_t h_n^x = 0.02(149)^{0.75}$ [Eq. 12.8-7]

$T_a = 0.8529$ sec

$T_L = 8_{sec} > T_a$ [Fig 22-15]

$C_s = S_{DS}/R/I = \frac{0.1248}{4.5/1.0} = 0.02773$ [Eq 12.8-2]

not to exceed $C_s = S_{D1}/T(R/I) = \frac{0.0578}{0.85(4.5/1)} = 0.0156$ [Eq 12.8-3]

must be greater than 0.01 , $S_1 = 0.051 < 0.6$ [Eq 12.8-5]

$\therefore C_s = 0.0156$

$K = 1.161$ (by interpolation) [Sect 12.8-3]

Determine Story Force

$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$, $V = C_s W$ [Eq 12.8-12] [Eq 12.8-1]

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

see figures and tables provided for building weights and force calculations

Appendix D

Lateral Analysis

AISC 360 Direct Analysis Validation Report

RAM Frame v14.05.03.00
DataBase: RMP T Model walled 02/23/14 10:05:03

DESIGN CODE
AISC 360-10 LRFD

SECOND-ORDER ANALYSIS
P-Delta analysis was performed with gravity loads.
Scale factor (DL) : 1.20 Scale factor (LL) : 0.50

B1 Factors:
B1 factors were calculated and applied to gravity load case moments.

B2 Factors:
B2 factors were not applied.

NOTIONAL LOADS
Fraction of gravity loads used for Notional Loads:
Global X-axis : 0.0020
Global Y-axis : 0.0020

Generated Load Combinations:
Number of Selected Load Combinations = 104
Notional Loads were included with all combinations

REDUCED STIFFNESS
Flexural Stiffness:
The flexural stiffnesses were reduced.

Number of members with required $\tau b < 1.0 = 7$
Smallest required $\tau b = 0.981$
Column #: 20 on Level: Garage
Load Combination: 1.200 D + 1.200 ND2 + 1.600 Lp + 1.600 NL2
τb used in Analysis : 0.982

Axial Stiffness:
The axial stiffnesses were reduced.

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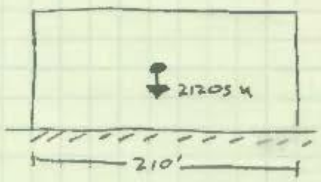
Overturning Moment

N-S Direction



$M_r = 21205(210)/2 =$

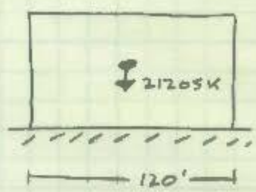
$= 2,226,525$ kip-ft

wind controls → $0.9D \overset{?}{>} 1.6W$

$2,003,872 > 66,059$

∴ ok

E-W Direction



$M_r = 21205(120)/2 =$

$= 1,272,300$ kip-ft

seismic controls → $0.9D \overset{?}{>} 1.0E_h$

$1,145,070 > 43,408$

∴ ok

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Moment Frame Column Check

W 14 x 176
L = 13.67'

Load 1 - $P_{u,max}$
P_{r_1} = 589.41 kips
M_{r_1} = -3.27 kip-ft
1.2D + 1.6L

Load 2 - $M_{u,max}$
P_{r_2} = 421.94 kip-ft
M_{r_2} = -90.36 kip-ft
1.2D + 0.5L + 1.6 W,

- Forces above result from 2nd order analysis in software
∴ notional loads & appropriate factors already included in values

P_c = 2023.3 k [Table 4-1]

M_c = 1200 kip-ft [Table 3-6] $L_b < L_p$ ∴ $M_c = \phi M_n$

Case 1:

$P_u/\phi P_n$ = 589.41/2023 = 0.29 > 0.2

[use equation H1-1a]

$P_r/\phi P_n + (8/9)\, M_{rx}/\phi M_{nx} \leq 1.0$

$(589.41/2023) + (3.27/1200)(8/9)$ = 0.29 ≤ 1.0 ∴ ok

Case 2:

$P_u/\phi P_n$ = 421.94/2023.3 = 0.21 > 0.2

[use equation H1-1a]

$P_r/\phi P_n + (8/9)\, M_{rx}/\phi M_{nx} \leq 1.0$

$421.94/2023 + (8/9)(90.36/1200)$ = 0.28 ≤ 1.0 ∴ ok

∴ Column is adequate for moment frame

- over capacity due to drift control needs

Member Force Envelope

RAM Frame v14.05.03.00
DataBase: RMP T Model walled 02/25/14 13:01:37
Building Code: IBC

STEEL COLUMN INFORMATION:

Column Number: 7 **Frame Number: 2**

Level Top: Office 5th Floor Column Line (120.00,0.00)
Bot: Office 4th Floor
Fy (ksi) = 50.00 Column Size = W14X176
Elastic Modulus (ksi) = 29000.00
Orientation (deg) = 0.00 Length (ft) = 13.67

INPUT PARAMETERS:

		Top	Bottom
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Joint Face Dist (in):			
	Major:	10.40	10.40
	Minor:	0.00	0.00
Rigid End Zone (in):			
	Major:	0.00	0.00 (Ignore)
	Minor:	0.00	0.00 (Ignore)

Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor (DL): 1.20 Scale Factor (LL): 0.50
Scale Factor (Roof): 1.00 Scale Factor (Snow): 1.00
Ground Level: Base

LOAD COMBINATIONS: User Specified
No. of Specified Combinations: 98

MEMBER FORCE MAXIMA AND MINIMA

	P	Mmajor	Mminor	Vmajor	Vminor	Tors
	kips	kip-ft	kip-ft	kips	kips	kip-ft
Max @ T:	589.41	66.63	1.29	7.21	0.09	0.04
LC:	2	26	4	51	7	68
Max @ B:	589.41	57.48	2.48	7.21	0.09	0.04
LC:	2	51	4	51	7	68
Maximum:	589.41	66.63	2.48	7.21	0.09	0.04
LC:	2	26	4	51	7	68
@ (ft):	0.87	0.87	13.67	0.87	0.00	0.87
Min @ T:	157.01	-30.26	-1.29	-13.01	-0.09	-0.05
LC:	51	62	40	15	67	8
Min @ B:	157.01	-90.36	-2.47	-13.01	-0.09	-0.05
LC:	51	15	64	15	67	8
Minimum:	157.01	-90.36	-2.47	-13.01	-0.09	-0.05
LC:	51	15	64	15	67	8
@ (ft):	0.87	12.80	13.67	0.87	0.00	0.87

JMV | Thesis

Moment Frame Column Check

W14x176
$L = 16.75'$

Load 1 - $P_{u,max}$
$P_{r_1} = 741.71$ kips
$M_{r_1} = 0.45$ kip-ft
$1.2D + 1.6L$

Load 2 - $M_{u,max}$
$P_{r_2} = 532.41$ kips
$M_{r_2} = 109.13$ kip-ft
$1.2D + 0.5L - 1.6W_{12}$

- Forces above result from 2nd order analysis in software
  ∴ notional loads & appropriate factors included in values

$P_c = 2023.3$ k [Table 4-1]

$M_c = 1200$ kip ft

$\phi M_p = 1200$ kip-ft [Table 3-6]

$\phi M_n = \phi M_p - \phi BF(L_b - L_p) = 1200 - 7.83(16.75 - 14.2) = 1180.03$ kip-ft

$M_c = \min \begin{cases} \phi M_p = 1200 \leftarrow \text{controls} \\ \phi M_n C_b = 1180.03(2.67) = 3150 \end{cases}$

Case 1:

$P_u/\phi P_n = 741.71/2023 = 0.36 > 0.2$ [∴ use equation H1-1a]

$P_r/\phi P_n + (8/9)\, M_{rx}/\phi M_{nx} \leq 1.0$

$741.71/2023 + (8/9)(0.45/1200) = 0.37 \leq 1.0$ ∴ ok

Case 2: $P_u/\phi P_n = 532.41/2023 = 0.26 > 0.2$ [∴ use equation H1-1a]

$P_r/\phi P_n + (8/9)\, M_{rx}/\phi M_{nx} \leq 1.0$

$532.41/2023 + (8/9)\, 109.13/1200 = 0.34 \leq 1.0$ ∴ ok

∴ column is adequate for moment frame

- over capacity due to drift control needs

Member Force Envelope

RAM Frame v14.05.03.00
DataBase: RMP T Model walled 02/25/14 13:01:37
Building Code: IBC
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STEEL COLUMN INFORMATION:

Column Number: 3 **Frame Number: 1**
Level Top: Garage Column Line (40.00,0.00)
Bot: Base
Fy (ksi) = 50.00 Column Size = W14X176
Elastic Modulus (ksi) = 29000.00
Orientation (deg) = 0.00 Length (ft) = 16.75

INPUT PARAMETERS:

		Top	Bottom
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Joint Face Dist (in):			
	Major:	10.40	0.00
	Minor:	0.00	0.00
Rigid End Zone (in):			
	Major:	0.00	0.00 (Ignore)
	Minor:	0.00	0.00 (Ignore)

Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor (DL): 1.20 Scale Factor (LL): 0.50
Scale Factor (Roof): 1.00 Scale Factor (Snow): 1.00
Ground Level: Base

LOAD COMBINATIONS: User Specified
No. of Specified Combinations: 98

MEMBER FORCE MAXIMA AND MINIMA

	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
Max @ T:	741.71	33.07	2.73	8.93	2.08	0.03
LC:	2	74	14	62	56	68
Max @ B:	741.71	108.70	32.20	8.93	2.08	0.03
LC:	2	62	8	62	56	68
Maximum:	741.71	108.70	32.20	8.93	2.08	0.03
LC:	2	62	8	62	56	68
@ (ft):	0.87	16.75	16.75	0.87	0.00	0.87
Min @ T:	298.80	-33.10	-2.65	-8.95	-2.08	-0.03
LC:	91	14	74	26	20	8
Min @ B:	298.80	-109.13	-32.13	-8.95	-2.08	-0.03
LC:	91	26	68	26	20	8
Minimum:	298.80	-109.13	-32.13	-8.95	-2.08	-0.03
LC:	91	26	68	26	20	8
@ (ft):	0.87	16.75	16.75	0.87	0.00	0.87

JMV Thesis

Moment Frame Beam Check

1) Girder - level : 5th Floor [G-1 to H-1]

W21x50 $L = 20'-0"$

$M_u = 167.25$ kip-ft $1.2D + 0.5L - 1.6W_1$

$V_u = 30.85$ kips $1.2D + 0.5L + 1.6W_1$

$\phi M_p = 413$ kip-ft

$\phi V_n = 237$ kips

$\phi M_p > M_u$ ∴ ok

$\phi V_n > V_u$ ∴ ok

∴ Member is adequate for moment frame loads

- over capacity due to drift control needs

2) Girder - level : 7th Floor [I-7 to J-7]

W21x50 $L = 20'-0"$

$M_u = 187.33$ kip-ft $1.2D + 0.5L - 1.6W_1$

$V_u = 32.73$ kips $1.2D + 0.5L - 1.6W_1$

$\phi M_p = 413$ kip-ft

$\phi V_n = 237$ kip-ft

$\phi M_p > M_u$ ∴ ok

$\phi V_n > V_u$ ∴ ok

∴ Member is adequate for moment frame loads

- over capacity due to drift control needs

Member Force Envelope

RAM Frame v14.05.03.00
DataBase: RMP T Model walled 02/25/14 13:01:37
Building Code: IBC
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STEEL BEAM INFORMATION:

Beam Number: 68	**Frame Number: 2**	
Level: Office 5th Floor	I-End (120.00,0.00)	J-End (140.00,0.00)
Fy (ksi) = 50.00	Beam Size = W21X50	
Length (ft) = 20.00		
Elastic Modulus (ksi) = 29000.00		

INPUT PARAMETERS:

		I-End	J-End
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Rigid End Zone (in):		0.00	0.00 (Ignore)

Member Force Output:		At Face of Joint			
P-Delta:	Yes	Scale Factor (DL):	1.20	Scale Factor (LL):	0.50
		Scale Factor (Roof):	1.00	Scale Factor (Snow):	1.00
Ground Level:	Base				

LOAD COMBINATIONS: User Specified
No. of Specified Combinations: 98

MEMBER FORCE MAXIMA AND MINIMA

	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
Max @ i:	0.00	93.51	0.00	30.08	0.00	0.00
LC:	26	51	57	15	55	8
Max @ j:	0.00	82.71	0.00	1.99	0.00	0.00
LC:	26	63	55	63	55	8
Maximum:	0.00	117.64	0.00	30.08	0.00	0.00
LC:	26	2	55	15	55	8
@ (ft):	0.00	10.00	19.37	0.64	0.64	0.00
Min @ i:	-0.00	-167.32	-0.00	-2.35	-0.00	-0.00
LC:	38	15	21	51	19	68
Min @ j:	-0.00	-167.17	-0.00	-30.85	-0.00	-0.00
LC:	38	3	19	3	19	68
Minimum:	-0.00	-167.25	-0.00	-30.85	-0.00	-0.00
LC:	38	15	19	3	19	68
@ (ft):	0.00	0.64	19.37	19.37	0.64	0.00

Member Force Envelope

RAM Frame v14.05.03.00
DataBase: RMP T Model walled 02/25/14 13:01:37
Building Code: IBC
Academic License. Not For Commercial Use.

STEEL BEAM INFORMATION:

Beam Number: 61 **Frame Number: 4**
Level: Office 7th Floor I-End (160.00,120.00) J-End (180.00,120.00)
Fy (ksi) = 50.00 Beam Size = W21X50
Length (ft) = 20.00
Elastic Modulus (ksi) = 29000.00

INPUT PARAMETERS:

		I-End	J-End
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Rigid End Zone (in):		0.00	0.00 (Ignore)

Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor (DL): 1.20 Scale Factor (LL): 0.50
Scale Factor (Roof): 1.00 Scale Factor (Snow): 1.00
Ground Level: Base

LOAD COMBINATIONS: User Specified
No. of Specified Combinations: 98

MEMBER FORCE MAXIMA AND MINIMA

	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
Max @ i:	0.00	90.07	0.00	32.73	0.00	0.00
LC:	13	51	8	15	8	8
Max @ j:	0.00	112.22	0.00	4.41	0.00	0.00
LC:	13	63	8	63	8	8
Maximum:	0.00	112.94	0.00	32.73	0.00	0.00
LC:	13	2	8	15	8	8
@ (ft):	0.00	10.00	19.38	0.62	0.62	0.00
Min @ i:	-0.00	-187.33	-0.00	-2.96	-0.00	-0.00
LC:	25	15	68	51	55	68
Min @ j:	-0.00	-168.50	-0.00	-29.82	-0.00	-0.00
LC:	25	3	68	3	55	68
Minimum:	-0.00	-187.26	-0.00	-29.82	-0.00	-0.00
LC:	25	15	68	3	55	68
@ (ft):	0.00	0.62	19.38	19.38	0.62	0.00

JMV | Thesis

30' shear wall — Base level

M_u = 8929.8 kip-ft — Length: 30'

V_u = 583.4 kips — thickness = 12"

P_u = 374.9 kip — #4 @ 12", (4) #11 at each end (vert & horiz)

shear:

$V_u \le \phi V_{n,max} = \phi 10\sqrt{f'_c}\, bd$

$d = (0.8)(360) = 288"$

$\phi V_n = (0.75)(10)\sqrt{5000}\,(12)(288) = 1832.8$ kips

$\phi V_n > V_u$ ∴ ok

shear strength of concrete:

$V_c = 2\sqrt{f'_c}\, hd = 2\sqrt{5000}\,(12)(288)/1000 = 488.75$ kips

$\frac{1}{2}\phi V_c = \frac{1}{2}(0.75)(488.75) = 183.28$ kips $< V_u$ ∴ V_s required

Check provided reinforcing

$A_s = 0.4$ in²/ft — $\frac{A_v}{s}(F_y d) = V_s = \left(\frac{0.4}{12}\right)(60)(288) = 576$ kips

$\phi V_n = \phi(V_c + V_s) = 183.28 + 0.75(576) = 615.3$ kips

$\phi V_n = 615.3$ kips > 583.4 kips $= V_u$ ∴ ok

$\rho_h = \frac{A_v}{sh} = \frac{2(0.2)}{12 \times 12} = 0.00278 > 0.0025$ ∴ ok

spacing is okay by inspection

∴ (2) #4 bars @ 12" o.c. is adequate for horizontal shear reinf.

JMV | Thesis

check vertical shear reinforcement

$$\rho_\ell = A_v/s_h \geq 0.0025 + 0.5\left(2.5 - \frac{129.83}{18}\right)(0.00278 - 0.0025) < 0.0025$$

$\therefore$ use $\rho_\ell \geq 0.0025$ as min

$$\frac{A_v}{s_h} = \frac{2(0.2)}{12 \times 12} = 0.00278 > 0.0025 \quad \underline{\therefore ok}$$

check flexural reinforcing

$$M_u \leq \phi M_n = \phi A_s F_y J_d$$

$$J_d = d - a/2 = 288 - 14.68/2 = 280.66"$$

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{(12.48)(60)}{0.85(5)(12)} = 14.68"$$

$$8929.8 \leq (0.9)(12.48)(60)(280.66)(1/12)$$

$$8929.8 \leq 15761.9 \text{ kip-ft}$$

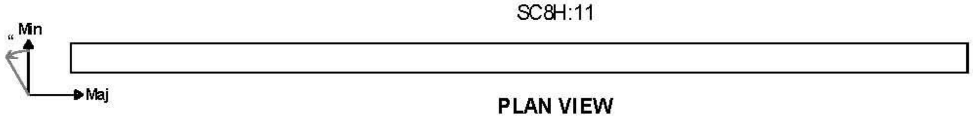
$\therefore$ (8) #11 bars is adequate for boundary reinforcing for flexure

Section Cut Design Summary

RAM Concrete Shearwall v14.05.03.00
Database: RMP T Model walled 02/21/14 17:40:13
Design Code: ACI 318-11
Academic License. Not For Commercial Use.

Section Cut ID: SC8H:11 (Horizontal)
Story: Garage
Ag = 4320 in2 Imaj = 46655996 in4 Imin = 51840 in4
Major Axis Orientation: 90.00 degrees (CCW from global X-axis)
Wall Design Group: 8
Design Status: PASS

SC8H:11



PLAN VIEW

Axial/Flexural Results:
Interaction: 0.791 OK
Pu = -374.88 kips phiPn = -474.19 kips
Mu = 8929.8 kip-ft at Beta = -0.0 deg CCW from Major axis
Controlling Load Combo: 0.900 D - 1.600 W7 (LC 69)
Code Ref: 10.3.7

Shear Results:
Segment SC8H:11:
Length = 30.00 ft Thick = 12.00 in f'c = 6000 psi fy = 60 ksi
Vert Bar Pat: 28 garage Horiz Bar Pat: 28 garage
Vu = 583.4 kip phiVn = 825.7 kip OK
Controlling Load Combo: 1.200 D - 1.600 W5 (LC 43)
Code Ref: 14.2.3 & 11.9.5

Reinforcement Checks:
Min Vert Reinf Ratio: Limit: 0.250% Actual: 0.535% (11.9.9.4) OK
Segment SC8H:11:
Max Vert Bar Spacing Limit: 18.00 in Actual: 12.00 in (11.9.9.5) OK
Min Vert Bar Spacing Limit: 1.00 in Actual: 11.50 in (7.6.1) OK
Min Number of Reinf Curtains: 2 Actual: 2 (14.3.4) OK

JMV | Thesis

<u>20' shear wall</u> - Base level

M_u = 3220.8 kip-ft

V_u = 410 kips

P_u = 2134.1 kips

Length = 20'

thickness = 12"

#4 @ 12" horiz & vert, (4) #11 at each end

f'_c = 5 ksi

f_y = 60 ksi

shear :

$V_u \le \phi V_{n,max} = \phi 10 \sqrt{f'_c}\, bd$

$d = (0.8)(240) = 192"$

$\phi V_n = 0.75(10)\sqrt{5000}(12)(192)/1000 = 1221.9$ kips

$\phi V_n > V_u$ ∴ <u>ok</u>

shear strength of concrete :

$V_c = 2\sqrt{f'_c}\, hd = 2\sqrt{5000}(12)(198)/1000 = 336.25$ kips

$\frac{1}{2}\phi V_c = \frac{1}{2}(0.75)(336.25) = 126.01$ kips $< V_u$ ∴ V_s required

check provided reinforcing

$A_s = 0.4\ in^2/ft$

$\frac{A_v}{s}(F_y d) = V_s = \left(\frac{0.4}{12}\right)(60000)(192) = 384$ kips

$\phi V_n = \phi(V_c + V_s) = 126.01 + 0.75(384) = 414$ kips

$\phi V_n = 414 > 410 = V_u$ ∴ <u>ok</u>

$\rho_t = \frac{A_v}{sh} = \frac{2(0.2)}{12 \times 12} = 0.00278 > 0.0025$ ∴ ok

spacing is okay by inspection

∴ (2) #4 bars @ 12" o.c. is adequate for horizontal shear rein.

JMV | Thesis

check vertical shear reinforcement

$$\rho_e = A_v/s_h \geq 0.0025 + 0.5\left(2.5 - \frac{129.6}{18}\right)(0.00278 - 0.0025) < 0.0025$$

$\therefore$ use $\rho_e \geq 0.0025$ as min

$$\frac{A_v}{s_h} = \frac{2(0.2)}{12 \times 12} = 0.00278 > 0.0025 \quad \therefore \underline{ok}$$

check flexural reinforcing

$$M_u \leq \phi M_n = \phi A_s F_y I_d$$

$$I_d = d - a/2 = 196 - 14.68/2 = 188.6''$$

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{(12.48)(60)}{0.85(5)(12)} = 14.68''$$

$$3220.8 \leq (0.9)(12.48)(60)(188.6)(1/12)$$

$$3220.8 \leq 10591$$

$\therefore$ (8) #11 bars is adequate for boundary reinforcing for flexure

Section Cut Design Summary

RAM Concrete Shearwall v14.05.03.00
Database: RMP T Model walled 02/22/14 13:38:24
Design Code: ACI 318-11
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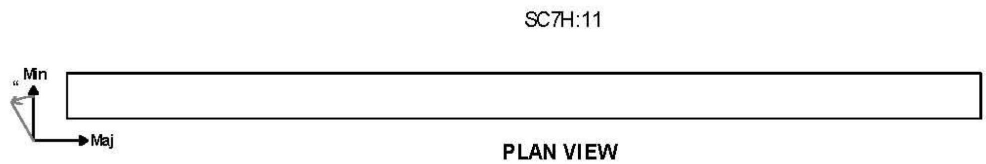
Section Cut ID: SC7H:11 (Horizontal)
Story: Garage
Ag = 2880 in2 Imaj = 13823999 in4 Imin = 34560 in4
Major Axis Orientation: 90.00 degrees (CCW from global X-axis)
Wall Design Group: 7
Design Status: PASS



Axial/Flexural Results:
Interaction: 0.736 OK
Pu = -434.12 kips phiPn = -590.15 kips
Mu = 3220.8 kip-ft at Beta = 0.0 deg CCW from Major axis
Controlling Load Combo: 0.900 D + 1.600 W8 (LC 58)
Code Ref: 10.3.7

Shear Results:
Segment SC7H:11:
Length = 20.00 ft Thick = 12.00 in f'c = 5000 psi fy = 60 ksi
Vert Bar Pat: 20 garage Horiz Bar Pat: 20 garage
Vu = 410.0 kip phiVn = 527.1 kip OK
Controlling Load Combo: 1.200 D + 0.500 Lp - 1.600 W6 (LC 20)
Code Ref: 14.2.3 & 11.9.5

Reinforcement Checks:
Min Vert Reinf Ratio: Limit: 0.250% Actual: 0.666% (11.9.9.4) OK
Segment SC7H:11:
Max Vert Bar Spacing Limit: 18.00 in Actual: 12.00 in (11.9.9.5) OK
Min Vert Bar Spacing Limit: 1.00 in Actual: 11.50 in (7.6.1) OK
Min Number of Reinf Curtains: 2 Actual: 2 (14.3.4) OK

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Return Wall Check - Base level

$M_u = 1190.6$ kip-ft Length = 10' $f'_c = 5$ ksi

$V_u = 123.8$ kips thickness = 12" $f_y = 60$ ksi

$P_u = 472.5$ kips #4 @ 12", (4) #11 at each end

vert & horiz

Shear:

$V_u \leq \phi V_{n,max} = \phi 10\sqrt{f'_c}\, hd$

$d = 0.8(120) = 96"$

$\phi V_n = 0.75(10)\sqrt{5000}(12)(96)/1000 = 610.9$ kips

$\phi V_n > V_u \therefore$ ok

shear strength of Concrete:

$V_c = 2\sqrt{f'_c}\, hd = 2\sqrt{5000}(12)(96)/1000 = 162.9$ kips

$\frac{1}{2}\phi V_c = \frac{1}{2}(0.75)(162.9) = 61.1$ kips $< V_u \therefore V_s$ required

Check provided reinforcing

$A_s = 0.4$ in²/ft $\frac{A_v}{s}(F_y d) = V_s = \left(\frac{0.4}{12}\right)(60000)(96) = 192$ kips

$\phi V_n = \phi(V_c + V_s) = 61.1 + 0.75(192) = 205.1$ kips

$\phi V_n = 205$ kips > 123.8 kips $= V_u \therefore$ ok

$\rho_t = \frac{A_v}{sh} = \frac{2(0.2)}{12\times12} = 0.00278 > 0.0025 \therefore$ ok

spacing is ok by inspection

$\therefore$ (2) #4 bars @ 12" o.c. is adequate for horizontal shear reinf.

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Check vertical shear reinforcing

$\rho_\ell = A_v/sh \geq 0.0025 + 0.5\left(2.5 - \frac{129.83}{18}\right)(0.00278 - 0.0025) < 0.0025$

$\therefore$ use $\rho_\ell \geq 0.0025$ as min

$\frac{A_v}{sh} = \frac{2(0.2)}{12(12)} = 0.00278 \geq 0.0025 \quad \therefore$ ok

$\therefore$ (2) #4 bars @ 12" o.c. is adequate for vertical shear reinf.

Check flexure

$M_u \leq \phi M_n = \phi A_s F_y j_d$

$1190.6(12) \leq (0.9)(8 \times 1.56)(60)(88.66)$

$j_d = d - a/2 = 96 - 14.68/2 = 88.66$

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{(12.48)(60)}{0.85(5)(12)} = 14.68$

1190.6 kip-ft $\leq$ 4979.1 kip-ft

$\therefore$ (8) #11 bars is adequate for boundary reinforcing for flexure

Section Cut Design Summary

RAM Concrete Shearwall v14.05.03.00
Database: RMP T Model walled 02/21/14 18:05:01
Design Code: ACI 318-11
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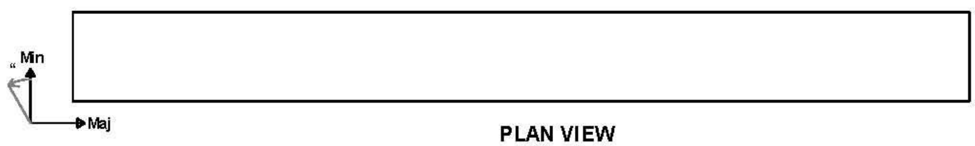
Section Cut ID: SC1H:11 (Horizontal)
Story: Garage
Ag = 1440 in2 Imaj = 1728000 in4 Imin = 17280 in4
Major Axis Orientation: 0.00 degrees (CCW from global X-axis)
Wall Design Group: 1
Design Status: PASS

SC1H:11



PLAN VIEW

Axial/Flexural Results:
Interaction: 0.871 OK
Pu = -472.49 kips phiPn = -542.22 kips
Mu = 1190.6 kip-ft at Beta = -0.0 deg CCW from Major axis
Controlling Load Combo: 0.900 D - 1.600 W8 (LC 70)
Code Ref: 10.3.7

Shear Results:
Segment SC1H:11:
Length = 10.00 ft Thick = 12.00 in f'c = 6000 psi fy = 60 ksi
Vert Bar Pat: 20 s return Horiz Bar Pat: 20 s return
Vu = 123.8 kip phiVn = 192.0 kip OK
Controlling Load Combo: 0.900 D - 1.600 W12 (LC 74)
Code Ref: 14.2.3 & 11.9.5

Reinforcement Checks:
Min Vert Reinf Ratio: Limit: 0.250% Actual: 1.058% (11.9.9.4) OK
Segment SC1H:11:
Max Vert Bar Spacing Limit: 18.00 in Actual: 12.00 in (11.9.9.5) OK
Min Vert Bar Spacing Limit: 1.00 in Actual: 11.50 in (7.6.1) OK
Min Number of Reinf Curtains: 2 Actual: 2 (14.3.4) OK

Appendix E

Connection Design

Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: RMP T Model 12 02/13/14 14:57:41
Building Code: IBC Steel Code: AISC 360-10 LRFD

Floor Type: Typical **Beam Number = 78**

SPAN INFORMATION (ft): I-End (50.00,0.00) J-End (50.00,40.00)

Beam Size (User Selected) = W18X35 Fy = 50.0 ksi
Total Beam Length (ft) = 40.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Deck Label	Typical Flooring	Typical Flooring
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	perpendicular	perpendicular
Decking type	VULCRAFT 2.0VL	VULCRAFT 2.0VL

beff (in)	=	120.00	Y bar(in)	=	18.13
Mnf (kip-ft)	=	569.01	Mn (kip-ft)	=	445.40
C (kips)	=	189.53	PNA (in)	=	15.17
Ieff (in4)	=	1249.74	Itr (in4)	=	1729.41
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.2 Rg = 1.00 Rp = 0.60
\# of studs: Full = 70 Partial = 40 Actual = 22
Number of Stud Rows = 1 Percent of Full Composite Action = 36.80

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.427	0.427	0.000	---	NonR	0.000	0.000
	40.000	0.427	0.427	0.000			0.000	0.000
2	0.000	0.050	0.000	0.800	22.0%	Red	0.200	0.200
	40.000	0.050	0.000	0.800			0.200	0.200
3	0.000	0.035	0.035	0.000	---	NonR	0.000	0.000
	40.000	0.035	0.035	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 38.67 kips 1.00Vn = 159.30 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	175.0	20.0	0.0	1.00	0.90	249.37
	Init DL	1.4DL	129.5	20.0	---	---		
	Max +	1.2DL+1.6LL	386.7	20.0	---	---	0.90	400.86
Controlling		1.2DL+1.6LL	386.7	20.0	---	---	0.90	400.86

REACTIONS (kips):

	Left	Right
Initial reaction	13.25	13.25
DL reaction	10.25	10.25
Max +LL reaction	16.49	16.49
Max +total reaction (factored)	38.67	38.67

DEFLECTIONS: (Camber = 1-1/4)

Initial load (in)	at	20.00 ft	=	-1.801	L/D	=	267
Live load (in)	at	20.00 ft	=	-1.310	L/D	=	366
Post Comp load (in)	at	20.00 ft	=	-1.389	L/D	=	345
Net Total load (in)	at	20.00 ft	=	-1.940	L/D	=	247

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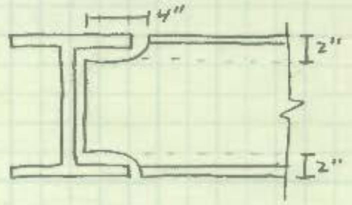
Typical Beam-to-Girder Connection



V_u = 38.67 kips
Beam = W18x35, A992
Girder = W18x35, A992
Bolts = 3/4" ϕ, A325-N
Plate = A36

- Determine eccentricity - Table 10-9a : let $a = 3''$ $\therefore e = a/2 = 1.5''$ $\therefore C = 2.23$

- Determine # of bolts - $C_{req} = 38.67/17.9 = 2.16$ bolts $\therefore$ use (3) bolts

- Determine plate thickness - [table 10-10a] : 3 bolts & 5/16" thick plate = 43.4 kips & 1/4" weld

- check web bearing strength :

$$\phi r_n = (0.75)(2.4)(65)(3/4)(0.300) = 26.32 \text{ kips} > 17.9 \text{ kips} \therefore \underline{ok}$$

- check flexural capacity of coped beam

$c = 4 \leq 2d = 2(17.7) = 35.4''$ $\quad d_{ct} = d_{cb} = 2''$ $\quad c = 4''$

$d_c = 2'' \leq 0.2d = 0.2(17.7) = 3.54''$ $\quad h_o = 13.7''$

$$F_{bc} = 56{,}490\left[t_w^2/ch_o\right]\left[3.5 - 7.5(d_c/d)\right] \leq F_y$$

$$56{,}490\left[0.3^2/(4 \times 13.7)\right]\left[3.5 - 7.5(2/17.7)\right] = 246 > F_y \therefore \text{use } F_y$$

$$\phi M_n = \phi F_y S_{net} = (0.9)(50)\left(\frac{0.3(13.7)^2}{6}\right) = 422.3 \text{ kip-ft}$$

$$\frac{\phi M_n}{e} = \phi V_n = 422.3/4.5 = 93.8 \text{ kips} > 38.67 \text{ kips} \therefore \underline{ok}$$

- check web block shear

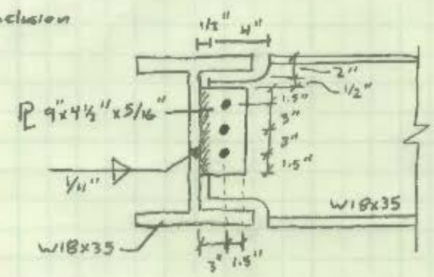
$L_{eh} = 3'' - 1/2'' = 2.5''$ BS T Rupture - table 9.3a = 101 kips/in

$L_{ev} = 1.5'' + 1/2'' = 2.0''$ BS V Yielding - table 9.3b = 180 kips/in

$t = 0.30$ BS V Rupture - table 9.3c = 170 kips/in

$$\phi R_n = 0.3(170 + 101) = 81.3 \text{ kips} > 40 \text{ kips} \therefore \underline{ok}$$

- conclusion



Gravity Beam Design

RAM Steel v14.05.03.00
DataBase: RMP T Model walled 2 02/25/14 13:01:37
Building Code: IBC Steel Code: AISC 360-10 LRFD

Floor Type: Typical **Beam Number = 58**

SPAN INFORMATION (ft): I-End (100.00,120.00) J-End (120.00,120.00)

Beam Size (Optimum) = W18X35 Fy = 50.0 ksi
Total Beam Length (ft) = 20.00
Mp (kip-ft) = 277.08

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL
10.000	9.73	15.20	1.2	0.00	0.00	0.0	0.00	0.0	3.80
18.500	5.00								
1.500	5.00								

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type	PartL
1	0.000	0.036	0.000	---	NonR	0.000
	20.000	0.036	0.000			0.000
2	0.000	0.004	0.067	1.2%	Red	0.017
	20.000	0.004	0.067			0.017
3	0.000	0.035	0.000	---	NonR	0.000
	20.000	0.035	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 29.12 kips 1.00Vn = 159.30 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	229.1	10.0	10.0	1.60	0.90	249.37
Controlling		1.2DL+1.6LL	229.1	10.0	10.0	1.60	0.90	249.37

REACTIONS (kips):

	Left	**Right**
DL reaction	10.62	10.62
Max +LL reaction	10.24	10.24
Max +total reaction (factored)	29.12	29.12

DEFLECTIONS:

Dead load (in)	at	10.00 ft	=	-0.251	L/D	=	955
Live load (in)	at	10.00 ft	=	-0.387	L/D	=	621
Net Total load (in)	at	10.00 ft	=	-0.638	L/D	=	376

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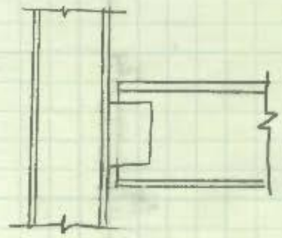
Typical Girder-to-Column Connection



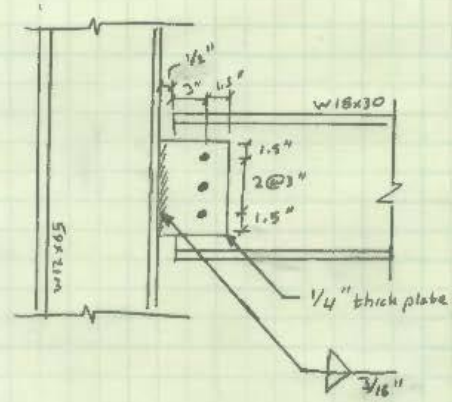
V_u = 29.88 Kips
Beam = W18x35, A997
Column = W12x65, A997 [5th Floor]
Bolts = 3/4" ϕ, A325-N
Plate = A36

- Determine eccentricity - Table 10-9a : let a = 3" ∴ e = a/2 = 1.5" → let c = 2.33
- Determine number of bolts : 29.88/17.9 = 1.67 < C → use 3 bolts
- Determine plate thickness [table 10-10a] : 3 bolts & 1/4" thick plate & 3/16" weld = 38.3 kips
- check web bearing strength

$\phi r_n = (0.75)(2.4)(65)(3/4)(0.30) = 26.32$ kips > 17.9 kips ∴ ok

- no cope ∴ no web block shear
  ∴ no need to check flexural capacity of cope

- Summary



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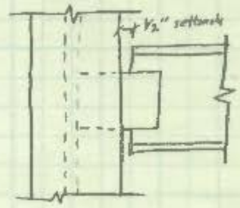
Typical Beam-to-Column Web Connection [Higher levels]



$V_u = 38.67$ kips
Beam = W18x35, A992
Column = W12x53 [E-1, 7th Floor]
Bolts = 3/4" ϕ, A325-N
Plate = A36

- Determine eccentricity : $e = \frac{1}{2}(10 - 0.345) + \frac{1}{2}" + 1.5" = 6.83"$

- Determine # of bolts & Plate thickness:

ϕr_n (bolt) $= 17.9$ kips
ϕr_n (web bear) $= \phi(2.4) F_u d_b t_w = (0.75)(2.4)(65)(3/4)(0.3) = 26.3$ kips
ϕr_n (plate bear) $= \phi(2.4) F_u d_b t_w = (0.75)(2.4)(58)(3/4)\, t_p = 78.3\, t_p$

Table 7-6 : Let $s = 3"$, $e_x = 6.83"$, $C \geq \frac{38.67}{17.9} = 2.16$

if $C = 2.33$ (by interpolation) $\therefore n = 5$ bolts

Plate thickness : $78.3\, t_p \geq 17.9 \therefore t_p \geq 0.23" \rightarrow$ try 5/16" thick

- Try (5) Bolts with 5/16" thick plate

- Determine strength of Bolts

ϕr_n, bolt $= 17.9$ kips
ϕr_n, web bearing $= 26.3$ kips
ϕr_n, plate bearing $= 24.5$ kips

$\phi R_n = C \phi r_n = 2.33(17.9)$
$= 41.76 \text{ kips} > 38.67 \text{ kips} \therefore \underline{ok}$

- check maximum plate thickness

$A_b = \frac{\pi}{4}(0.75)^2$ $C' = 17.1$ in $F_v = 54$ $F_y = 36$ $d = 2(1.5) + 4(3) = 15"$

$M_{max} = (F_v/0.9) A_b C' = (54/0.9)(\frac{\pi}{4}(0.75)^2)(17.1) = 453.3$ kip-in

$t_{max} = \frac{6 M_{max}}{F_y d^2} = \frac{6(453.3)}{(36)(15)^2} = 0.336 \text{ in} > 0.3125 \text{ in} \therefore \underline{ok}$

- Plate Shear Yielding

$\phi R_n = \phi(0.6) F_y A_g = (0.6)(1.0)(36)(15 \times 5/16) = 101.25 \text{ kips} > 38.67 \text{ kips} \therefore \underline{ok}$

- shear plate Rupture

$\phi R_n = \phi(0.6) F_u A_n = (0.75)(0.6)(58)[15 - 5(3/4 + 1/16 + 1/16)](5/16)$

$= 86.66 \text{ kips} > 38.67 \text{ kips} \therefore \underline{ok}$

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- Plate Block Shear

$L_{ev} = 1.5"$ — BS T-Rupt [Table 9.3a] = 46.2 kips

$L_{eh} = 1.5"$ — BS V-Yield [Table 9.3b] = 219 kips

$t = 5/16"$ — BS V-rupt [Table 9.3c] = 250 kips

$$\phi R_n = (5/16)(46.2 + 219) = 82.875 \text{ kips} > 38.67 \text{ kips} \therefore \underline{ok}$$

- check flexure / shear interaction

$V_r = V_u = 38.67$ kips

$V_c = \phi V_n = (1.0)(0.6)(36)(15 \times 5/16) = 101.25$ kips

$M_r = V_r \times e = V_u \times e = (38.67)(6.83) = 264.12$ kips-in

$M_c = \phi M_n = 0.9 F_y \frac{t_p d^2}{4} = (0.9)(36)(1/4)(5/16)(15)^2 = 569.53$ kip-in

$$\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2 \leq 1.0 \rightarrow \left(\frac{38.67}{101.25}\right)^2 + \left(\frac{264.12}{569.53}\right)^2 = 0.361 \leq 1.0 \therefore \underline{ok}$$

- check plate buckling

$$\lambda = \frac{h_o \sqrt{F_y}}{10 t_p \sqrt{475 + 280 (h_o/c)^2}} = \frac{15\sqrt{36}}{(10)(5/16)\sqrt{475 + 280(15/6.83)^2}} = 0.674 < 0.7$$

$\therefore F_{cr} = F_y$

$$\phi M_n = \phi F_{cr} S_{net} = 0.9(36)\frac{(5/16)(15)^2}{6} = 379.7 \text{ kip-in} > 264 \therefore \underline{ok}$$

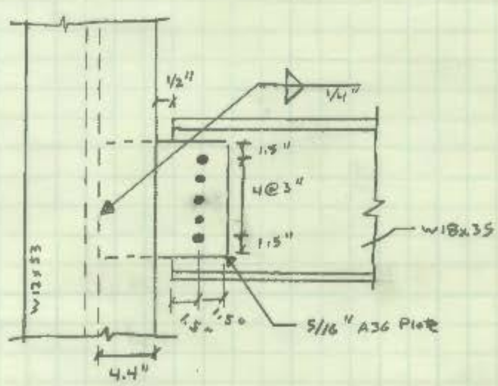
- check weld strength — min weld = $(5/8)(5/16) = 0.195 < 1/4" \therefore \underline{ok}$

$l = 15 - 2(1/4) = 14.5"$ $\quad e_x = 6.83"$

$e_x = al \therefore a = 6.83/14.5 = 0.47 \rightarrow$ table 8.4, let $k = 0$ $\therefore C = 2.408$ (interpolated)

$$\phi R_n = \phi C C_1 l D = (0.75)(1.0)(2.408)(14.5)(4) = 104.75 \text{ kips} > 38.67 \therefore \underline{ok}$$

- conclusion



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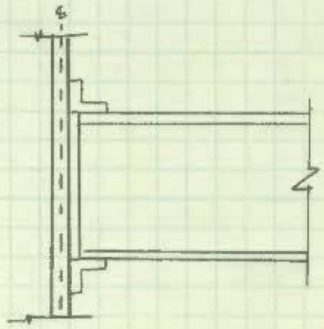
Typical Beam-to-Column Web Connection [lower levels]



V_u = 38.67 Kips
Beam = W18x35, A992
Column : W12x65, A992
Bolts : 3/4" ϕ, A325-N
Angles = A36

K_{des} = 0.827"
K_{det} = 1 1/8"

- beam shear yielding : $\phi R_n = \phi 0.6 F_y A_{gv} = (1.0)(0.6)(50)(17.7 \times 0.3) = 159.3$ kips $> V_u$ ∴ ok

- beam shear rupture : $\phi R_n = \phi 0.6 F_u A_{nv} = (1.0)(0.6)(65)(17.7 \times 0.3) = 207.1$ kips $> V_u$ ∴ ok

- Beam local web yielding

$$l_{b,min} = \frac{R_u}{1.0 F_y t_w} - 2.5 K_{des} = \frac{38.67}{1.0(50)(0.39)} - 2.5(0.827) = -0.08 < K_{det}$$

∴ use K_{det}

- Beam local web crippling , assume $l_b/d \le 0.2$

$$l_{b,min} = \frac{d}{3}\left(\frac{t_f}{t_w}\right)^{1.5}\left[\frac{R_u}{0.75(0.4)t_w^2}\sqrt{\frac{t_w}{E F_{yw} t_f}} - 1\right]$$

$$= \left(\frac{17.7}{3}\right)\left(\frac{0.425}{0.300}\right)^{1.5}\left[\frac{38.67}{0.75(0.4)(0.3)^2}\sqrt{\frac{0.3}{29000(50)(0.425)}} - 1\right] = -0.007$$

∴ use K_{det}

$K_{det}/d = l_b/d = 1.125/17.7 = 0.064 < 0.2$ ∴ ok

Try L4x4 x 1/2" x 6" long

- Seat angle Flexure

$$e = \frac{1.125}{2} + 0.75 - 4/8 - 3/8 = 0.4375$$

$$\phi R_n = \frac{0.9 F_y L t_a^2}{4e} = \frac{(0.9)(36)(6)(3/4)^2}{4(0.4375)} = 62.5 \text{ kips} > V_u \therefore ok$$

- web yielding

$$\phi R_n = 1.0 F_y t_w (2.5 k_{des} + l_b)$$

$$= 1.0(50)(0.3)(2.5 \times 0.827 + 1.125) = 47.89 \text{ kips} > V_u \therefore ok$$

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- local web crippling $\quad l/d = 1.125/17.7 < 0.2$

$$\phi R_n = \phi 0.4 t_w^2 \left[1 + \left(\frac{4 l_b}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_f}{t_w}}$$

$$= 0.75(0.4)(0.3)^2\left[1 + \left(\frac{4(1.125)}{17.7} - 0.2\right)\left(\frac{0.3}{0.425}\right)^{1.5}\right]\sqrt{\frac{(29000)(50)(0.425)}{0.3}} = 39.94 \text{ kips}$$

$> V_u \therefore \underline{ok}$

- Angle shear yielding

$$\phi R_n = \phi 0.6 F_y L_a t_a = (1.0)(0.6)(36)(6)(1/2) = 64.8 \text{ kips} > V_u \therefore \underline{ok}$$

- Weld Rupture

$\theta = 0°$ $\quad K = 0$

$e = 3/4'' + l_b/2 = 3/4 + 1.125/2 = 1.3125''$

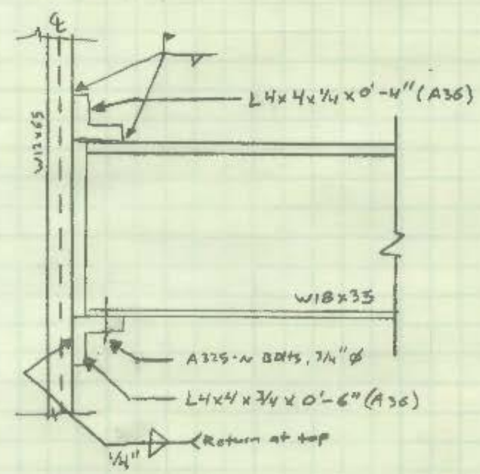
$a = e/L = 1.3125/8 = 0.164''$

$\therefore C = 3.52$ [Table 8-4]

$$D_{min} = \frac{R_u}{\phi C C_1 L} = \frac{38.67}{(0.75)(3.52)(1.0)(6)} = 2.46 \rightarrow 3/16''$$

Minimum weld = 1/4'' $\therefore$ use 1/4'' fillet weld

- Summary



Member Force Envelope

RAM Frame v14.05.03.00
DataBase: RMP T Model walled 02/25/14 13:01:37
Building Code: IBC
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STEEL BEAM INFORMATION:

Beam Number: 69	**Frame Number: 2**	
Level: Office 9th Floor	I-End (140.00,0.00)	J-End (160.00,0.00)
Fy (ksi) = 50.00	Beam Size = W21X50	
Length (ft) = 20.00		
Elastic Modulus (ksi) = 29000.00		

INPUT PARAMETERS:

		I-End	J-End
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Rigid End Zone (in):		0.00	0.00 (Ignore)

Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor (DL): 1.20 Scale Factor (LL): 0.50
Scale Factor (Roof): 1.00 Scale Factor (Snow): 1.00
Ground Level: Base

LOAD COMBINATIONS: User Specified
No. of Specified Combinations: 98

MEMBER FORCE MAXIMA AND MINIMA

	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
Max @ i:	0.00	93.13	0.00	30.42	0.00	0.00
LC:	36	51	4	15	24	19
Max @ j:	0.00	96.42	0.00	2.49	0.00	0.00
LC:	36	63	24	63	24	19
Maximum:	0.00	118.58	0.00	30.42	0.00	0.00
LC:	36	2	24	15	24	19
@ (ft):	0.00	10.00	19.40	0.61	0.61	0.00
Min @ i:	-0.00	-162.19	-0.00	-3.20	-0.00	-0.00
LC:	24	15	64	51	60	55
Min @ j:	-0.00	-185.86	-0.00	-31.88	-0.00	-0.00
LC:	24	3	60	3	60	55
Minimum:	-0.00	-185.86	-0.00	-31.88	-0.00	-0.00
LC:	24	3	60	3	60	55
@ (ft):	0.00	19.40	19.40	19.40	0.61	0.00

Member Force Envelope

RAM Frame v14.05.03.00
DataBase: RMP T Model walled 02/25/14 13:01:37
Building Code: IBC

STEEL BEAM INFORMATION:

Beam Number: 68 **Frame Number: 2**
Level: Office 9th Floor I-End (120.00,0.00) J-End (140.00,0.00)
Fy (ksi) = 50.00 Beam Size = W21X50
Length (ft) = 20.00
Elastic Modulus (ksi) = 29000.00

INPUT PARAMETERS:

		I-End	J-End
Fixity	Major Axis:	Fix	Fix
	Minor Axis:	Fix	Fix
	Torsion:	Fix	Fix
Rigid End Zone (in):		0.00	0.00 (Ignore)

Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor (DL): 1.20 Scale Factor (LL): 0.50
Scale Factor (Roof): 1.00 Scale Factor (Snow): 1.00
Ground Level: Base

LOAD COMBINATIONS: User Specified
No. of Specified Combinations: 98

MEMBER FORCE MAXIMA AND MINIMA

	P kips	Mmajor kip-ft	Mminor kip-ft	Vmajor kips	Vminor kips	Tors kip-ft
Max @ i:	0.00	101.49	0.00	30.78	0.00	0.00
LC:	36	51	60	15	55	19
Max @ j:	0.00	88.47	0.00	2.68	0.00	0.00
LC:	36	63	60	63	55	19
Maximum:	0.00	119.36	0.00	30.78	0.00	0.00
LC:	36	2	60	15	55	19
@ (ft):	0.00	10.00	19.40	0.61	0.61	0.00
Min @ i:	-0.00	-175.14	-0.00	-3.01	-0.00	-0.00
LC:	24	15	24	51	8	55
Min @ j:	-0.00	-172.05	-0.00	-31.52	-0.00	-0.00
LC:	24	3	24	3	8	55
Minimum:	-0.00	-175.07	-0.00	-31.52	-0.00	-0.00
LC:	24	15	24	3	8	55
@ (ft):	0.00	0.61	19.40	19.40	0.61	0.00

Joint Code Check

RAM Frame v14.05.03.00
DataBase: RMP T Model walled 02/24/14 19:11:43
Building Code: IBC Steel Code: AISC360-10 LRFD

Story Number: 7 **Joint Number:** 130

Final Design

No Web Plate Required
No Top Flange Stiffener Required
No Bot Flange Stiffener Required

Joint Data and Material Properties

Web Plate Nominal Yield (ksi) ________ 36.00
Stiffener Nominal Yield (ksi) ________ 36.00

	Size	Plan Angle	Elev Angle	Yield(ksi)
Col. At Jnt:	W14X120	0.00	---	50.00
Beam SideA :	W21X50	0.00	0.00	50.00
Beam SideB :	W21X50	180.00	0.00	50.00

Criteria

Force on column flange is from beam moment, axial and shear forces.
Use actual beam moments to determine panel zone shear at the joint.
Optimize design of each stiffener at a joint

Results

Panel Zone

Side	Moment (kip-ft)	Axial (kip)	Shear (kip)	Load Combination
A	80.00	-0.00	3.52	1.200 D + 1.200 ND1 + 0.500 Lp + 0.500 NL1 + 1.600 W1
B	-185.22	-0.00	-32.98	1.200 D + 1.200 ND1 + 0.500 Lp + 0.500 NL1 + 1.600 W1

Shear Force In Column Above Joint(kip) = 22.35
Controlling Shear Force (kip) = 134.94
Column Web Capacity w/o Web Plate (kip) = 281.57

Compression

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	59.9	57	242.9	57.1	69	242.9	NO
	Bot	103.8	21	242.9	109.7	9	242.9	NO
Web Crippling	Top	59.9	57	334.9	57.1	69	334.9	NO
	Bot	103.8	21	334.9	109.7	9	334.9	NO
Web Buckling	Bot	42.2	5	467.8	74.8	5	467.8	NO

Tension

	Flange	Side A Force (kip)	Side A LCo	Side A Cap. (kip)	Side B Force (kip)	Side B LCo	Side B Cap. (kip)	Stiffen
Local Web Yld	Top	103.8	21	242.9	109.7	9	242.9	NO
	Bot	59.9	57	242.9	57.1	69	242.9	NO
Flange Bend.	Top	103.8	21	248.5	109.7	9	248.5	NO
	Bot	59.9	57	248.5	57.1	69	248.5	NO

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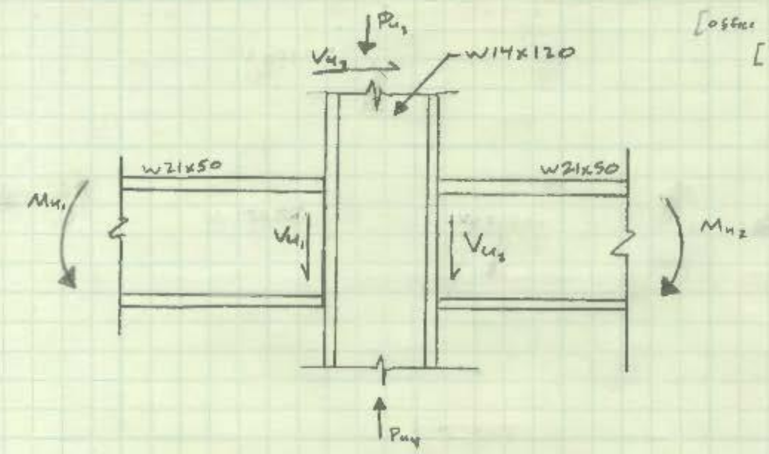
Typical Moment Connection

[office level 9]
[column line H-1]



Determine bolts

$M_{u_1} = 175.05$ kip-ft
$M_{u_2} = 185.86$ kip-ft ← controls

$(2F)\left[(20.8 - 0.535)/(12 \times 2)\right] = 185.86$ kip-ft $\therefore F = 110.1$ kip

$110.1/17.9 = 6.15 \rightarrow$ use 8 bolts [3/4" ø, A325-N]

Flexural Strength of Beam:

$A_{fg} = b_f t_f = (6.53)(0.535) = 3.49 \text{ in}^2$

$A_{fn} = b_f t_f - 2(d_b + 1/8) t_f = [(6.53) - 2(3/4 + 1/8)](0.535) = 2.56 \text{ in}^2$

$F_u A_{fn} = (65)(2.56) = 166.2$ $Y_t = 1.0$ since $F_y/F_u \le 0.8$

$F_y A_{fg} Y_t = (50)(3.49)(1.0) = 174.5 > 166.2 \Rightarrow$ use eq F13-1

$$\phi M_n = \frac{\phi F_u A_{fn}}{A_{fg}} S_x = \frac{(0.9)(65)(2.56)}{3.49}(94.5)(1/12) = 378 \text{ kip ft}$$

$\phi M_n > M_u \therefore \underline{ok}$

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Flange Plate Tensile Yeilding let $t_p = 0.50''$

$$\phi R_n = \phi F_y A_g = (0.9)(36)(0.50)(w_p) \geq P_{uf}$$

$$P_{uf} = \frac{M_u}{d + t_p} = \frac{185.86(12)}{(20.8 + 0.50)} = 104.7 \text{ kips}$$

$$\therefore w_p = \frac{104.7}{(0.9)(36)(0.75)} = 6.46'' \rightarrow 7''$$

Flange Plate Tensile Rupture

$$\phi R_n = \phi F_u A_e = (0.75)(58)[7 - 2(3/4 + 1/8)](1/2) = 114.2 \text{ kips} > 104.7 \therefore \underline{ok}$$

Flange Plate Block Shear

$$\phi R_n = \phi U_{bs} F_u A_{nt} + \min(\phi 0.6 F_y A_{gv}, \phi 0.6 F_u A_{nv})$$

$$F_y A_{gv} = (36)(10.5)(1/2) = 189$$
$$F_u A_{nv} = (58)(10.5 - 4.5(3/4 + 1/8))(1/2) = 190.3$$

Case 1: $\phi R_n = [0.75(1.0)(58)(1.75 - 1/2(7/8))(1/2) + 0.75(0.6)(189)] \times 2 = 227.2$ kips

Case 2: $\phi R_n = [0.75(1.0)(58)(3.5 - (7/8))(1/2)] + 0.75(0.6)(189) \times 2 = 227.2$ kips

Case 3: $\phi R_n = [0.75(1.0)(58)(1.75 + 3.5 - 1.5(7/8))(1/2)] + 0.75(0.6)(189) = 170.7$ kips

$\phi R_n > R_u$ $(170.7 > 104.7)$ $\therefore \underline{ok}$

Beam Flange Block Shear

$$F_y A_{gv} = (50)(10.5)(0.535) = 280.9$$
$$F_u A_{nv} = (65)(10.5 - 3.5(3/4 + 1/8))(0.535) = 258.6$$

$$\phi R_n = [0.75(1.0)(65)(1.52 - 1/2(3/4 + 1/8))(0.535) + 0.75(0.6)258.6] \times 2 = 289.2 \text{ kips}$$

$\phi R_n > R_u$ $\therefore \underline{ok}$ $(289.2 > 110.1$ kips$)$

Compression Flange Plate & connection – Flexural Buckling

$K = 0.65$ [Table C-A-7.1] $\frac{KL}{r} = \frac{(0.65)(2)}{(1/2)/\sqrt{12}} = 9.01 \leq 25 \therefore F_{cr} = F_y$

$L = 2.0''$

$$\phi P_n = \phi F_y A_g = (0.9)(36)(7)(1/2) = 113.4 \text{ kips} > 110.1 \text{ kips} \therefore \underline{ok}$$

Compression Flange Plate & connection – Local Buckling

$$s/t_p \leq 253/\sqrt{F_y} \rightarrow 3.5/(1/2) \leq 253/\sqrt{36} \rightarrow 7 \leq 42.2 \therefore \underline{ok}$$

$$\frac{w_p - s}{2t_p} \leq 95/\sqrt{F_y} \rightarrow 1.75/0.5 \leq 95/\sqrt{36} \rightarrow 3.5 \leq 15.8 \therefore \underline{ok}$$

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Bolt check

$\phi r_n = 17.9$ Kip/bolt [Table 7-1]

$\phi r_n = 44.0\,(1/2) = 22.0$ kip/bolt [Table 7-5]

$\phi r_n = 49.4\,(0.535) = 26.43$ Kip/bolt [Table 7-5]

$\phi r_n = 2(17.9) + 6(17.9) = 143.2$ kips > 110.1 kips $\therefore$ ok

Flange local bending of column

$\phi R_n = (0.9)(6.25)\,t_{fc}^2 F_{yc} = (0.9)(6.25)(0.94)^2(50) = 248.5$ kips > 110 kips $\therefore$ ok

Web local Yeilding of Column

$\phi R_n = \phi F_{yc}(5k_{des} + \ell_b)\,t_{wc} = (1.0)(50)(5 \times 1.54 + 1/2)(0.59) = 241.9\,K > 110\,K \therefore$ ok

Web crippling of column

$$\phi R_n = \phi\, 0.80\, t_w^2 \left[1 + 3\left(\frac{\ell_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_f}{t_w}}$$

$$= 0.75(0.8)(0.59)^2\left[1 + 3\left(\frac{0.5}{14.5}\right)\left(\frac{0.59}{0.94}\right)^{1.5}\right]\sqrt{\frac{29000(50)(0.94)}{(0.59)}} = 333\,K > 110 \therefore ok$$

web bucking of column

$$\phi R_n = \phi\, 24 t_{wc}^3 \sqrt{E F_{yc}}\,/h = (0.9)24(0.59)^3\sqrt{29000(50)}\,/11.387 = 469\,K > 110\,K \therefore ok$$

Panel zone shear yeilding

controling load combo: $1.2D + 0.5L + 1.6W$

$M_1 = -185.86$

$M_2 = 80.00$

$$\Sigma F_u = \frac{M_{u1}}{d_{m1}} + \frac{M_{u2}}{d_{m2}} - V_u \le \phi R_v$$

$$\Sigma F_u = (185.86 + 80)(12)/(0.95 \times 20.8) - 22.35 = 139.1 \text{ Kips}$$

$P_c = P_y = F_y A_g = (50)(35.3) = 1765$ kip

$P_r = 165.76$ kips $\qquad P_r < 0.4 P_c$

$\phi R_v = \phi\, 0.6 F_y d_c t_w = (0.9)(0.6)(50)(14.5)(0.59) = 230.98$ Kips

$139.1 < 230.98 \rightarrow \Sigma F_u \le \phi R_v \therefore$ ok

[Does not need doubler plate]

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Minimum weld for column-to-plate connection

$$D_{min} = \frac{P_{uf}}{2(1.5)1.392\,l} = \frac{110.1}{2(1.5)1.392(7)} = 3.79 \text{ sixteenths} \rightarrow 1/4'' > 3/16 \text{ min} \quad \therefore \underline{ok}$$

Connecting element strength at weld

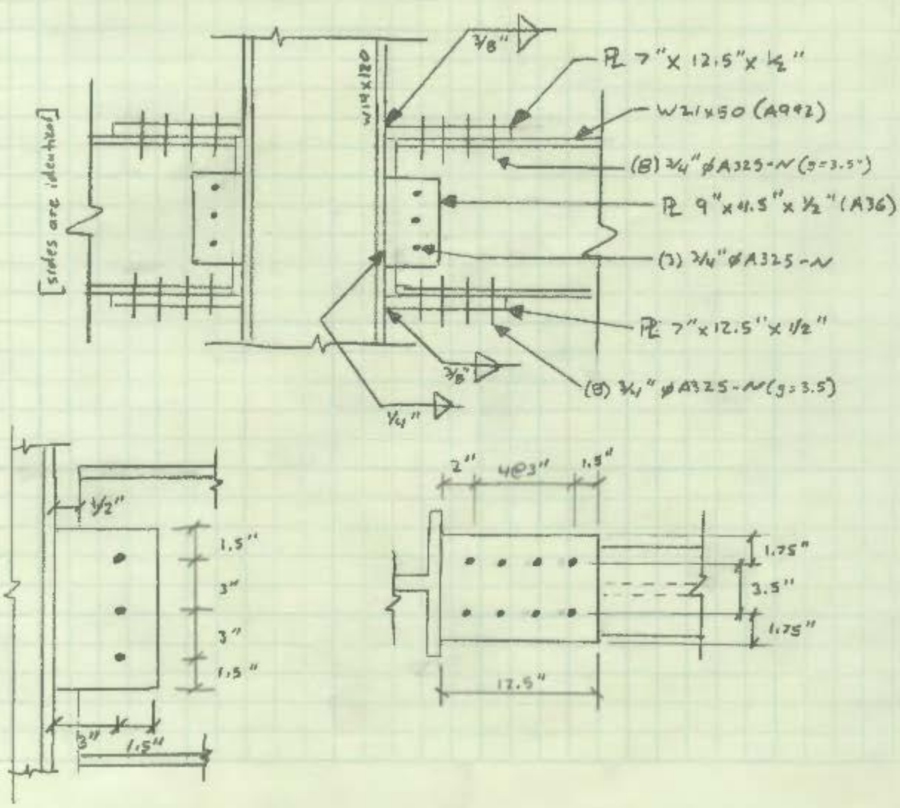
$$t_{min} = \frac{3.09D}{F_u} = \frac{3.09(3.79)}{65} = 0.18 \text{ in.} < 0.49 \quad \therefore \underline{ok}$$

Single plate web connection

- Determine eccentricity - Table 10-9a → let a = 3" ∴ e = 3/2 = 1.5 → let c = 2.23
- Determine number of bolts - 31.88/17.9 → 1.78 < c → use 3 bolts
- Determine plate thickness - table 10-10a : 3 bolts & 1/4" thick plate = 38.3 kips & 3/16" weld
- check web bearing strength

$$\phi r_n = 0.75(2.4)(65)(3/4)(0.38) = 33.34 > 17.9 \quad \therefore \underline{ok}$$

Summary



Gravity Column Design

RAM Steel v14.05.03.00 Page 2/2
DataBase: RMP T Model walled 02/24/14 19:17:33
Building Code: IBC Steel Code: AISC 360-10 LRFD

Story level Garage, Column Line E-1

Fy (ksi)	=	50.00	Column Size	=	W12X87
Orientation (deg.)	=	0.0			

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		16.75	16.75
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	8.75	8.55
	Bottom	0.00	0.00

CONTROLLING COLUMN LOADS - Skip-Load Case 1:

			Dead	Live	Roof
Axial (kip)			343.51	186.74	0.00
Moments	Top	Mx (kip-ft)	0.00	0.00	0.00
		My (kip-ft)	-3.12	-1.11	0.00
	Bot	Mx (kip-ft)	0.00	0.00	0.00
		My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL + 0.5RF)

Pu (kip)	=	710.99	0.90*Pn (kip)	=	841.74
Mux (kip-ft)	=	0.00	0.90*Mnx (kip-ft)	=	461.00
Muy (kip-ft)	=	5.67	0.90*Mny (kip-ft)	=	226.50
Rm	=	1.00			
Cbx	=	1.00			
Cmx	=	1.00	Cmy	=	0.60
Pex (kip)	=	5242.49	Pey (kip)	=	1707.35
B1x	=	1.16	B1y	=	1.03

INTERACTION EQUATION

Pu/0.90*Pn = 0.845
Eq H1-1a: 0.845 + 0.000 + 0.022 = 0.867

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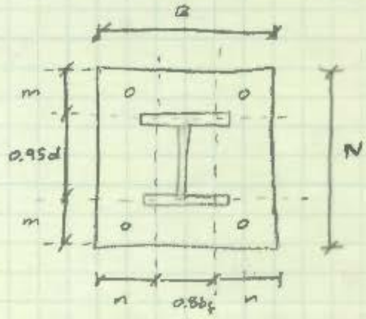
Typical Gravity Base Plate [Exterior]



$P_u = 711.0$ kips

Column : W12x87, A992 [Column E-1]

Plate : A36

Concrete Column : 24"x24", $f'_c = 4$ ksi (Assumed)

$$A_{s(req)} = \frac{P_u}{\phi_c 0.85 f'_c} = \frac{711.0}{(0.65)(0.85)(4)} = 322 \ in^2$$

Try 20"x20" base plate $= 400 in^2 > 322 \therefore ok$

$N \geq d + 2(3") \rightarrow 20" \geq 12.5" + 2(3") = 18.5 \quad \therefore ok$ (for bolt config.)

$B \geq b_f + 2(3") \rightarrow 20 \geq 12.1 + 2(3") = 18.1 \quad \therefore ok$

$$m = \frac{N - 0.95d}{2} = \frac{20 - 0.95(12.5)}{2} = 4.06"$$

$$n = \frac{B - 0.8b_f}{2} = \frac{20 - 0.8(12.1)}{2} = 5.16"$$

$$n' = \tfrac{1}{4}\sqrt{d b_f} = \tfrac{1}{4}\sqrt{(12.5)(12.1)} = 3.07$$

$$X = \left[\frac{4 d b_f}{(d+b_f)^2}\right]\frac{P_u}{\phi_c P_p} = \left[\frac{4(12.5)(12.1)}{(12.5+12.1)^2}\right]\frac{711}{[illegible]} = 0.67$$

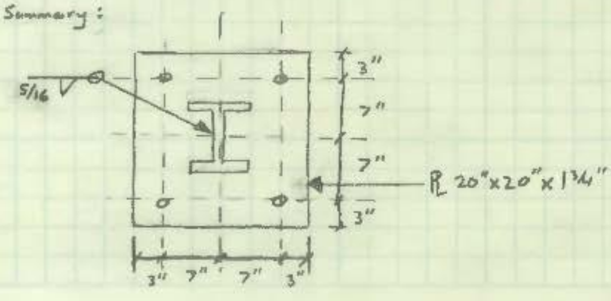
$$\lambda = \frac{2\sqrt{X}}{1+\sqrt{1-X}} = 1.04 > 1 \quad \therefore \text{use } \lambda = 1 \text{ as conservative}$$

$\lambda n' = (1)(3.07) = 3.07 \qquad \ell_{max} = \max(m, n, \lambda n') = 5.16"$

$$f_{pu} = \frac{P_u}{BN} = \frac{711}{(20 \times 20)} = 1.78$$

$$t_{min} = \ell\sqrt{\frac{2 f_{pu}}{0.9 F_y}} = 5.16\sqrt{\frac{2(1.78)}{0.9(36)}} = 1.71" \rightarrow 1\tfrac{3}{4}"$$

Summary:



Appendix F

Cost/Scheduling Analysis

Steel Estimate

Steel Deck - 05 31 13.50 (5300)						
Level	SF	Material	Labor	Equipment	Total	Tot Incl O&P
		2.05	0.46	0.04	2.55	3.14
P6	24893	51030.65	11450.78	995.72	63477.15	78164.02
4th	23058	47268.90	10606.68	922.32	58797.90	72402.12
5th	23058	47268.90	10606.68	922.32	58797.90	72402.12
6th	23058	47268.90	10606.68	922.32	58797.90	72402.12
7th	23058	47268.90	10606.68	922.32	58797.90	72402.12
8th	23058	47268.90	10606.68	922.32	58797.90	72402.12
9th	23058	47268.90	10606.68	922.32	58797.90	72402.12
10th	23058	47268.90	10606.68	922.32	58797.90	72402.12
11th	22102	45309.10	10166.92	884.08	56360.10	69400.28
Roof	22102	45309.10	10166.92	884.08	56360.10	69400.28
Pent	4000	8200.00	1840.00	160.00	10200.00	12560.00
Total	**234503**	**$480,731.15**	**$107,871.38**	**$9,380.12**	**$597,982.65**	**$736,339.42**

Welded Wire Fabric - 3 22 11.10 (0100)						
Level	C.S.F.	Material	Labor	Equipment	Total	Tot Incl O&P
		14.5	23	0	37.5	54
P6	248.93	3609.49	5725.39	0.00	9334.88	13442.22
4th	230.58	3343.41	5303.34	0.00	8646.75	12451.32
5th	230.58	3343.41	5303.34	0.00	8646.75	12451.32
6th	230.58	3343.41	5303.34	0.00	8646.75	12451.32
7th	230.58	3343.41	5303.34	0.00	8646.75	12451.32
8th	230.58	3343.41	5303.34	0.00	8646.75	12451.32
9th	230.58	3343.41	5303.34	0.00	8646.75	12451.32
10th	230.58	3343.41	5303.34	0.00	8646.75	12451.32
11th	221.02	3204.79	5083.46	0.00	8288.25	11935.08
Roof	221.02	3204.79	5083.46	0.00	8288.25	11935.08
Pent	40	580.00	920.00	0.00	1500.00	2160.00
Total	**2345.03**	**$34,002.94**	**$53,935.69**	**$0.00**	**$87,938.63**	**$126,631.62**

Placing Concrete - 03 31 13.70 (1400)						
Level	C.Y.	Material	Labor	Equipment	Total	Tot Incl O&P
		0	18	5.55	23.55	35
P6	326.37	0.00	5874.75	1811.38	7686.13	11423.12
4th	302.32	0.00	5441.69	1677.85	7119.54	10581.06
5th	302.32	0.00	5441.69	1677.85	7119.54	10581.06
6th	302.32	0.00	5441.69	1677.85	7119.54	10581.06
7th	302.32	0.00	5441.69	1677.85	7119.54	10581.06
8th	302.32	0.00	5441.69	1677.85	7119.54	10581.06
9th	302.32	0.00	5441.69	1677.85	7119.54	10581.06
10th	302.32	0.00	5441.69	1677.85	7119.54	10581.06
11th	289.78	0.00	5216.07	1608.29	6824.36	10142.36
Roof	289.78	0.00	5216.07	1608.29	6824.36	10142.36
Pent	52.44	0.00	944.00	291.07	1235.07	1835.56
Total	**3074.595**	**$0.00**	**$55,342.71**	**$17,064.00**	**$72,406.71**	**$107,610.82**

Finishing Concrete - 03 35 13.30 (0250)						
Level	SF	Material	Labor	Equipment	Total	Tot Incl O&P
		0	0.58	0.03	0.61	0.96
P6	24893	0.00	14437.94	746.79	15184.73	23897.28
4th	23058	0.00	13373.64	691.74	14065.38	22135.68
5th	23058	0.00	13373.64	691.74	14065.38	22135.68
6th	23058	0.00	13373.64	691.74	14065.38	22135.68
7th	23058	0.00	13373.64	691.74	14065.38	22135.68
8th	23058	0.00	13373.64	691.74	14065.38	22135.68
9th	23058	0.00	13373.64	691.74	14065.38	22135.68
10th	23058	0.00	13373.64	691.74	14065.38	22135.68
11th	22102	0.00	12819.16	663.06	13482.22	21217.92
Roof	22102	0.00	12819.16	663.06	13482.22	21217.92
Pent	4000	0.00	2320.00	120.00	2440.00	3840.00
Total	**234503**	**$0.00**	**$136,011.74**	**$7,035.09**	**$143,046.83**	**$225,122.88**

Concrete Topping - 03 30 53.40 (3300)						
Level	S.F.	Material	Labor	Equipment	Total	Tot Incl O&P
		1.24	0.88	0.27	2.39	3.09
P6	24893	30867.32	21905.84	6721.11	59494.27	76919.37
4th	23058	28591.92	20291.04	6225.66	55108.62	71249.22
5th	23058	28591.92	20291.04	6225.66	55108.62	71249.22
6th	23058	28591.92	20291.04	6225.66	55108.62	71249.22
7th	23058	28591.92	20291.04	6225.66	55108.62	71249.22
8th	23058	28591.92	20291.04	6225.66	55108.62	71249.22
9th	23058	28591.92	20291.04	6225.66	55108.62	71249.22
10th	23058	28591.92	20291.04	6225.66	55108.62	71249.22
11th	22102	27406.48	19449.76	5967.54	52823.78	68295.18
Roof	22102	27406.48	19449.76	5967.54	52823.78	68295.18
Pent	4000	4960.00	3520.00	1080.00	9560.00	12360.00
Total	**234503**	**$290,783.72**	**$206,362.64**	**$63,315.81**	**$560,462.17**	**$724,614.27**

Shear Studs - 05 05 23.85 (0030)						
Level	#	Material	Labor	Equipment	Total	Tot Incl O&P
		0.56	0.88	0.5	1.94	2.77
P6	1505	842.80	1324.40	752.50	2919.70	4168.85
4th	1565	876.40	1377.20	782.50	3036.10	4335.05
5th	1493	836.08	1313.84	746.50	2896.42	4135.61
6th	1481	829.36	1303.28	740.50	2873.14	4102.37
7th	1481	829.36	1303.28	740.50	2873.14	4102.37
8th	1481	829.36	1303.28	740.50	2873.14	4102.37
9th	1481	829.36	1303.28	740.50	2873.14	4102.37
10th	1481	829.36	1303.28	740.50	2873.14	4102.37
11th	1481	829.36	1303.28	740.50	2873.14	4102.37
Roof	1625	910.00	1430.00	812.50	3152.50	4501.25
Pent	0	0.00	0.00	0.00	0.00	0.00
Total	**15074**	**$8,441.44**	**$13,265.12**	**$7,537.00**	**$29,243.56**	**$41,754.98**

Structural Steel - Beams and Girders (Typical Floor) - 05 12 23.77 (0900)									
Member	#	Length	L.F.	Tons	Material	Labor	Equipment	Total	Tot Incl O&P
					2750	455	131	3336	4000
W 8 x 10	1	9.28	9.28	0.05	127.60	21.11	6.08	154.79	185.60
W 10 x 12	1	13.58	13.58	0.08	224.07	37.07	10.67	271.82	325.92
W 10 x 12	1	14	14	0.08	231.00	38.22	11.00	280.22	336.00
W 12 x 14	1	21.2	21.2	0.15	408.10	67.52	19.44	495.06	593.60
W 12 x 19	4	12	48	0.46	1254.00	207.48	59.74	1521.22	1824.00
W 12 x 19	1	22.06	22.06	0.21	576.32	95.35	27.45	699.13	838.28
W 12 x 19	1	22.56	22.56	0.21	589.38	97.52	28.08	714.97	857.28
W 14 x 22	1	18.28	18.28	0.20	552.97	91.49	26.34	670.80	804.32
W 14 x 22	1	18.56	18.56	0.20	561.44	92.89	26.74	681.08	816.64
W 14 x 22	1	20	20	0.22	605.00	100.10	28.82	733.92	880.00
W 14 x 22	1	21.27	21.27	0.23	643.42	106.46	30.65	780.52	935.88
W 14 x 22	1	22	22	0.24	665.50	110.11	31.70	807.31	968.00
W 14 x 22	1	22.28	22.28	0.25	673.97	111.51	32.11	817.59	980.32
W 14 x 26	1	15.5	15.5	0.20	554.13	91.68	26.40	672.20	806.00
W 14 x 34	1	20	20	0.34	935.00	154.70	44.54	1134.24	1360.00
W 16 x 26	1	21	21	0.27	750.75	124.22	35.76	910.73	1092.00
W 16 x 26	5	21.9	109.5	1.42	3914.63	647.69	186.48	4748.80	5694.00
W 16 x 26	1	24	24	0.31	858.00	141.96	40.87	1040.83	1248.00
W 16 x 31	1	22.84	22.84	0.35	973.56	161.08	46.38	1181.01	1416.08
W 18 x 35	1	18.35	18.35	0.32	883.09	146.11	42.07	1071.27	1284.50
W 18 x 35	17	20	340	5.95	16362.44	2707.25	779.45	19849.14	23799.94
W 18 x 35	1	20.24	20.24	0.35	974.05	161.16	46.40	1181.61	1416.80
W 18 x 35	1	29.2	29.2	0.51	1405.24	232.51	66.94	1704.69	2043.99
W 18 x 35	1	30	30	0.53	1443.74	238.88	68.78	1751.39	2099.99
W 18 x 35	1	32.15	32.15	0.56	1547.21	255.99	73.70	1876.91	2250.49
W 18 x 35	46	40	1840	32.20	88549.68	14651.00	4218.20	107418.88	128799.68
W 18 x 40	4	20	80	1.60	4399.98	728.00	209.60	5337.58	6399.98
W 18 x 40	2	22	44	0.88	2419.99	400.40	115.28	2935.67	3519.99
W 18 x 40	4	40	160	3.20	8799.97	1456.00	419.20	10675.17	12799.97
W 18 x 46	1	22	22	0.51	1391.49	230.23	66.29	1688.01	2023.99
W 21 x 44	2	20	40	0.88	2419.99	400.40	115.28	2935.67	3519.99
W 21 x 44	1	30.25	30.25	0.67	1830.12	302.80	87.18	2220.10	2661.99
W 21 x 50	8	20	160	4.00	10999.96	1820.00	524.00	13343.96	15999.96
W 24 x 55	1	30	30	0.83	2268.74	375.38	108.08	2752.19	3299.99
C 8 x 12	4	10	40	0.23	632.50	104.65	30.13	767.28	920.00
HSS6x6x5/16 23	4	16.71	66.84	0.78	2145.06	354.91	102.18	2602.16	3120.09
Total				**59.48**	**$163,572.09**	**$27,063.83**	**$7,792.00**	**$198,427.93**	**$237,923.28**
Note - Values vary from the base estimate as follows:									
Cost for Members 0 to 30 plf					0.00	0.00	0.00	0.00	0.00
Cost for Members 31 to 65 plf					-0.01	0.00	0.00	-0.01	-0.01
Cost for Members 66 to 100 plf					-5.65	0.00	0.00	-5.65	-6.25
Cost for Members 101 to 387 plf					55.50	0.00	0.00	55.50	61.00

Structural Steel - Columns (Typical Floor)													
Member					#	L.F.	Tons	Material	Labor	Equipment	Total	Tot Incl O&P
								2750	455	131	3336	4000
W	10	x	33		14	352.2	5.81	15981.02	2644.14	761.28	19386.44	23245.14
W	10	x	39		6	153.1	2.99	8209.96	1358.38	391.09	9959.43	11941.77
W	10	x	45		2	51.4	1.16	3180.36	526.21	151.50	3858.07	4625.99
W	10	x	49		5	131.4	3.22	8853.04	1464.78	421.73	10739.55	12877.17
W	10	x	54		3	81.1	2.19	6021.65	996.31	286.85	7304.82	8758.78
W	10	x	60		1	26.3	0.79	2169.74	359.00	103.36	2632.10	3155.99
W	10	x	68		1	26.3	0.89	2454.00	406.86	117.14	2978.00	3571.21
W	10	x	77		2	54.8	2.11	5790.03	959.96	276.38	7026.37	8426.01
W	10	x	88		1	27.4	1.21	3308.59	548.55	157.93	4015.07	4814.87
W	12	x	40		37	628.5	12.57	34567.37	5719.35	1646.67	41933.39	50279.87
W	12	x	45		9	188.7	4.25	11675.77	1931.82	556.19	14163.78	16982.96
W	12	x	50		2	50.3	1.26	3458.11	572.16	164.73	4195.01	5029.99
W	12	x	53		18	339.6	9.00	24748.26	4094.73	1178.92	30021.91	35997.51
W	12	x	58		14	188.7	5.47	15048.77	2489.90	716.87	18255.54	21889.15
W	12	x	65		12	307.4	9.99	27417.43	4545.68	1308.76	33271.86	39899.56
W	12	x	72		7	107.4	3.87	10610.75	1759.21	506.50	12876.47	15441.44
W	12	x	79		17	293.5	11.59	31815.94	5274.93	1518.72	38609.58	46300.54
W	12	x	87		11	250.2	10.88	29868.68	4952.08	1425.76	36246.53	43466.78
W	12	x	96		3	78.8	3.78	10380.23	1720.99	495.49	12596.72	15105.96
W	12	x	106		6	121	6.41	17991.67	2917.92	840.10	21749.69	26043.19
W	12	x	120		5	135.9	8.15	22876.05	3710.07	1068.17	27654.29	33113.39
W	12	x	136		3	82.3	5.60	15700.70	2546.36	733.13	18980.19	22726.98
W	12	x	152		1	27.4	2.08	5842.17	947.49	272.79	7062.46	8456.63
W	14	x	43		11	190.4	4.09	11257.36	1862.59	536.26	13656.21	16374.36
W	14	x	48		2	50.3	1.21	3319.79	549.28	158.14	4027.21	4828.79
W	14	x	53		1	25.2	0.67	1836.44	303.85	87.48	2227.77	2671.19
W	14	x	61		6	113.2	3.45	9494.62	1570.93	452.29	11517.84	13810.37
W	14	x	68		6	113.2	3.85	10562.45	1751.20	504.19	12817.85	15371.15
W	14	x	90		6	151	6.80	18647.86	3091.73	890.15	22629.73	27137.53
W	14	x	99		7	101.8	5.04	13829.05	2292.79	660.12	16781.97	20124.91
W	14	x	109		5	129.1	7.04	19739.36	3201.36	921.71	23862.42	28572.99
W	14	x	120		51	648.3	38.90	109128.34	17698.59	5095.64	131922.57	157964.78
W	14	x	132		7	147.4	9.73	27293.03	4426.42	1274.42	32993.87	39507.03
W	14	x	145		38	505.4	36.64	102797.73	16671.88	4800.04	124269.65	148801.13
W	14	x	176		38	547.9	48.22	135267.74	21937.92	6316.19	163521.85	195801.93
Total							**280.89**	**$781,144.07**	**$127,805.41**	**$36,796.72**	**$945,746.19**	**$1,133,117**
Note - Values vary from the base estimate as follows:												
		Cost for Members 0 to 30 plf						0.00	0.00	0.00	0.00	0.00
		Cost for Members 31 to 65 plf						-0.01	0.00	0.00	-0.01	-0.01
		Cost for Members 66 to 100 plf						-5.65	0.00	0.00	-5.65	-6.25
		Cost for Members 101 to 387 plf						55.50	0.00	0.00	55.50	61.00

Fire Proofing - Beams - 07 81 16.10 (400)						
Level	SF	Material	Labor	Equipment	Total	Tot Incl O&P
		0.53	0.62	0.09	1.24	1.68
P6	15255	8085.15	9458.10	1372.95	18916.20	25628.40
4th	15470	8199.10	9591.40	1392.30	19182.80	25989.60
5th	15745	8344.85	9761.90	1417.05	19523.80	26451.60
6th	15470	8199.10	9591.40	1392.30	19182.80	25989.60
7th	15470	8199.10	9591.40	1392.30	19182.80	25989.60
8th	15470	8199.10	9591.40	1392.30	19182.80	25989.60
9th	15470	8199.10	9591.40	1392.30	19182.80	25989.60
10th	15470	8199.10	9591.40	1392.30	19182.80	25989.60
11th	15470	8199.10	9591.40	1392.30	19182.80	25989.60
Roof	16120	8543.60	9994.40	1450.80	19988.80	27081.60
Pent	3800	2014.00	2356.00	342.00	4712.00	6384.00
Total	**159210**	**$84,381.30**	**$98,710.20**	**$14,328.90**	**$197,420.40**	**$267,472.80**

Fire Proofing - Columns - 07 81 16.10 (800)								
Level	Height	#	SF	Material	Labor	Equipment	Total	Tot Incl O&P
				1.2	1.32	0.19	2.71	3.67
P6	15.92	48	4737	5684.16	6252.58	899.99	12836.73	17384.06
4th	9.83	48	2926	3511.68	3862.85	556.02	7930.54	10739.89
5th	12.83	48	3819	4583.04	5041.34	725.65	10350.03	14016.46
6th	11.75	48	3497	4196.16	4615.78	664.39	9476.33	12833.26
7th	11.75	48	3497	4196.16	4615.78	664.39	9476.33	12833.26
8th	11.75	48	3497	4196.16	4615.78	664.39	9476.33	12833.26
9th	11.75	48	3497	4196.16	4615.78	664.39	9476.33	12833.26
10th	11.75	48	3497	4196.16	4615.78	664.39	9476.33	12833.26
11th	11.75	48	3497	4196.16	4615.78	664.39	9476.33	12833.26
Roof	11.75	48	3497	4196.16	4615.78	664.39	9476.33	12833.26
Pent	21.17	13	1706	2047.24	2251.96	324.15	4623.35	6261.14
Total			**37666.0**	**$45,199.24**	**$49,719.16**	**$7,156.55**	**$102,074.95**	**$138,234.34**

JMV | Thesis

Original Design

Typical Slab reinforcing [Area = 23058 ft²] & Beams

Bot: #4's @ 12" o.c. each way → (1) #4 = 0.668 lb/ft
Beams: Typically include (9) #9 bars & #4 stirrups @ 12" oc → (1) #9 = 3.4 lb/ft
Top: #5's @ 14" o.c. over beams (pt) → (1) #5 = 1.043 lb/ft
Assume bars are 12' long
Typical two way bay (20)
16 (#4) - 12' long
14 (#5) - 12' long
30 (#6) - 12' long → (1) #6 = 1.502 lb/ft

$$\text{Total} = (23058\ \text{ft}^2)([2 \times 0.668]\ \text{lb/ft}^2) + (12\ \text{ft})(1.043\ \text{lb/ft})(604\ \text{ft})\left(\tfrac{12''}{14''}\right)$$
$$+ 20(12\ \text{ft})\left[(16)(0.668) + 14(1.043) + (30)(1.502)\right]$$
$$+ (604 + 80)\left[9 \times 3.4 + (13')(0.668)\right] + 10\%\ \text{misc}$$

Total ≈ 59.59 lbs (slab) + 28.70 lbs (beams)

Typical Column Reinforcing [based on 24"x24"]

verticals : ≈ 6 #11 continuous (1) #11 → 5.313 lb/ft
ties : #4 @ 20" o.c. (1) #4 → 0.668 lb/ft

Reinforcing per foot ≈ $6(5.313) + 8(0.668)\left(\tfrac{20''}{12''}\right) =$ 40.78 lb/ft ht

Typical Shear wall Reinforcing

#4 bars at 12" o.c. each way (1) #4 → 0.668 lb/ft
(4) #11 bars at corners & edges (1) #11 → 5.313 lb/ft

- walls are 86 ft in length on each level
- walls require 8 boundary elements at each level

Total per foot of height = $(8)(4)(5.313) + (86')(2 \times 0.668\ \text{lb/ft})(2)$

Total = 400 lbs / ft of height

Typical Post Tensioning Reinforcing

≈ 540 k/Beam 27.0k/Tendon → 20 tendons per beam

(604 lineal ft beam)(20) = 12 080 ft of tendon

½" ϕ tendons → 0.52 lb/ft

Total = (12080)(0.52) = 6282 lbs

Concrete Estimate

Formwork - Columns - 03 11 13.25 (6550)									
Level	Height	Size	#	S.F.S.A	Material	Labor	Equiptment	Total	Tot Incl O&P
					1.58	6.45	0.00	8.03	12.35
Plaza	15.92	24"x24"	54	6876	10864.08	44350.20	0.00	55214.28	84918.60
		30" ϕ	12	1500	2370.17	9675.71	0.00	12045.89	18526.36
		12"x24"	8	764	1207.12	4927.80	0.00	6134.92	9435.40
P6	9.83	24"x24"	40	3147	4971.73	20296.00	0.00	25267.73	38861.33
		30" ϕ	9	695	1098.22	4483.25	0.00	5581.47	8584.21
		12"x24"	8	472	745.76	3044.40	0.00	3790.16	5829.20
4th	12.83	24"x24"	40	4107	6488.53	26488.00	0.00	32976.53	50717.33
		30" ϕ	9	907	1433.27	5851.02	0.00	7284.29	11203.12
		12"x24"	8	616	973.28	3973.20	0.00	4946.48	7607.60
5th	11.75	24"x24"	40	3760	5940.80	24252.00	0.00	30192.80	46436.00
		30" ϕ	9	831	1312.28	5357.10	0.00	6669.39	10257.40
		12"x24"	8	564	891.12	3637.80	0.00	4528.92	6965.40
6th	11.75	24"x24"	40	3760	5940.80	24252.00	0.00	30192.80	46436.00
		30" ϕ	9	831	1312.28	5357.10	0.00	6669.39	10257.40
		12"x24"	8	564	891.12	3637.80	0.00	4528.92	6965.40
7th	11.75	24"x24"	40	3760	5940.80	24252.00	0.00	30192.80	46436.00
		30" ϕ	9	831	1312.28	5357.10	0.00	6669.39	10257.40
		12"x24"	8	564	891.12	3637.80	0.00	4528.92	6965.40
8th	11.75	24"x24"	40	3760	5940.80	24252.00	0.00	30192.80	46436.00
		30" ϕ	9	831	1312.28	5357.10	0.00	6669.39	10257.40
		12"x24"	8	564	891.12	3637.80	0.00	4528.92	6965.40
9th	11.75	24"x24"	40	3760	5940.80	24252.00	0.00	30192.80	46436.00
		30" ϕ	9	831	1312.28	5357.10	0.00	6669.39	10257.40
		12"x24"	8	564	891.12	3637.80	0.00	4528.92	6965.40
10th	11.75	24"x24"	40	3760	5940.80	24252.00	0.00	30192.80	46436.00
		30" ϕ	9	831	1312.28	5357.10	0.00	6669.39	10257.40
		12"x24"	8	564	891.12	3637.80	0.00	4528.92	6965.40
11th	11.75	24"x24"	40	3760	5940.80	24252.00	0.00	30192.80	46436.00
		30" ϕ	5	461	729.05	2976.17	0.00	3705.21	5698.55
		18"x18"	4	282	445.56	1818.90	0.00	2264.46	3482.70
		12"x24"	8	564	891.12	3637.80	0.00	4528.92	6965.40
Roof	21.17	24"x24"	10	1693	2675.47	10922.00	0.00	13597.47	20912.67
		18"x18"	5	635	1003.30	4095.75	0.00	5099.05	7842.25
		12"x24"	8	1016	1605.28	6553.20	0.00	8158.48	12547.60
Total				**58423**	**$92,307.96**	**$376,826.82**	**$0.00**	**$469,134.78**	**$721,521.11**

Formwork - Exterior Beams - 03 11 13.20 (1500)								
Level	Size	L.F.	S.F.S.A	Material	Labor	Equiptment	Total	Tot Incl O&P
				2.47	8.10	0.00	10.57	16.00
P6	24"x20"	86	401	991.29	3250.80	0.00	4242.09	6421.33
	36"x24"	145	918	2268.28	7438.50	0.00	9706.78	14693.33
4th	24"x20"	86	401	991.29	3250.80	0.00	4242.09	6421.33
	36"x24"	145	918	2268.28	7438.50	0.00	9706.78	14693.33
5th	24"x20"	86	401	991.29	3250.80	0.00	4242.09	6421.33
	36"x24"	145	918	2268.28	7438.50	0.00	9706.78	14693.33
6th	24"x20"	86	401	991.29	3250.80	0.00	4242.09	6421.33
	36"x24"	145	918	2268.28	7438.50	0.00	9706.78	14693.33
7th	24"x20"	86	401	991.29	3250.80	0.00	4242.09	6421.33
	36"x24"	145	918	2268.28	7438.50	0.00	9706.78	14693.33
8th	24"x20"	86	401	991.29	3250.80	0.00	4242.09	6421.33
	36"x24"	145	918	2268.28	7438.50	0.00	9706.78	14693.33
9th	24"x20"	86	401	991.29	3250.80	0.00	4242.09	6421.33
	36"x24"	145	918	2268.28	7438.50	0.00	9706.78	14693.33
10th	24"x20"	86	401	991.29	3250.80	0.00	4242.09	6421.33
	36"x24"	145	918	2268.28	7438.50	0.00	9706.78	14693.33
11th	24"x20"	86	401	991.29	3250.80	0.00	4242.09	6421.33
	36"x24"	145	918	2268.28	7438.50	0.00	9706.78	14693.33
Roof	24"x20"	86	401	991.29	3250.80	0.00	4242.09	6421.33
	36"x24"	166	1051	2596.79	8515.80	0.00	11112.59	16821.33
Pent	18"x24"	160	773	1910.13	6264.00	0.00	8174.13	12373.33
Total			**14103**	**$34,834.41**	**$114,234.30**	**$0.00**	**$149,068.71**	**$225,648.00**
Note - Table assumes 8" thick slab								

Formwork - Slab - 03 11 13.35 (1100)							
Level	Total S.F	S.F.S.A	Material	Labor	Equiptment	Total	Tot Incl O&P
			1.46	3.93	0.00	5.39	8.05
P6	24893	21781	31800.81	85600.80	0.00	117401.61	175340.07
4th	23058	20176	29456.60	79290.70	0.00	108747.29	162414.79
5th	23058	20176	29456.60	79290.70	0.00	108747.29	162414.79
6th	23058	20176	29456.60	79290.70	0.00	108747.29	162414.79
7th	23058	20176	29456.60	79290.70	0.00	108747.29	162414.79
8th	23058	20176	29456.60	79290.70	0.00	108747.29	162414.79
9th	23058	20176	29456.60	79290.70	0.00	108747.29	162414.79
10th	23058	20176	29456.60	79290.70	0.00	108747.29	162414.79
11th	22102	19339	28235.31	76003.25	0.00	104238.56	155680.96
Roof	22102	19339	28235.31	76003.25	0.00	104238.56	155680.96
Pent	4000	3500	5110.00	13755.00	0.00	18865.00	28175.00
Total		**205190**	**$299,577.58**	**$806,397.19**	**$0.00**	**$1,105,975**	**$1,651,781**
Note - Slab formork was reduced in order to account for area of beams, openings, etc.							

Formwork - Interior Beams - 03 11 13.20 (2550)								
Level	Size	L.F.	S.F.S.A	Material	Labor	Equiptment	Total	Tot Incl O&P
				1.42	5.85	0.00	7.27	11.20
P6	24"x36"	46	307	435.47	1794.00	0.00	2229.47	3434.67
	12"x30"	60	280	397.60	1638.00	0.00	2035.60	3136.00
	48"x20"	72	384	545.28	2246.40	0.00	2791.68	4300.80
4th	48"x20"	604	3221	4574.29	18844.80	0.00	23419.09	36078.93
	12"x30"	60	280	397.60	1638.00	0.00	2035.60	3136.00
5th	48"x20"	604	3221	4574.29	18844.80	0.00	23419.09	36078.93
	12"x30"	60	280	397.60	1638.00	0.00	2035.60	3136.00
6th	48"x20"	604	3221	4574.29	18844.80	0.00	23419.09	36078.93
	12"x30"	60	280	397.60	1638.00	0.00	2035.60	3136.00
7th	48"x20"	604	3221	4574.29	18844.80	0.00	23419.09	36078.93
	12"x30"	60	280	397.60	1638.00	0.00	2035.60	3136.00
8th	48"x20"	604	3221	4574.29	18844.80	0.00	23419.09	36078.93
	12"x30"	60	280	397.60	1638.00	0.00	2035.60	3136.00
9th	48"x20"	604	3221	4574.29	18844.80	0.00	23419.09	36078.93
	12"x30"	60	280	397.60	1638.00	0.00	2035.60	3136.00
10th	48"x20"	604	3221	4574.29	18844.80	0.00	23419.09	36078.93
	12"x30"	60	280	397.60	1638.00	0.00	2035.60	3136.00
11th	48"x20"	604	3221	4574.29	18844.80	0.00	23419.09	36078.93
	12"x30"	60	280	397.60	1638.00	0.00	2035.60	3136.00
Roof	48"x20"	604	3221	4574.29	18844.80	0.00	23419.09	36078.93
	12"x30"	60	280	397.60	1638.00	0.00	2035.60	3136.00
Pent	12"x24"	165	605	859.10	3539.25	0.00	4398.35	6776.00
Total			**33088**	**$46,984.49**	**$193,562.85**	**$0.00**	**$240,547.34**	**$370,581.87**
Note - Table assumes 8" thick slab								

Formwork - Shear Walls - 03 11 13.								
Level	Size	Wall Length	S.F.S.A	Material	Labor	Equiptment	Total	Tot Incl O&P
				1.24	6.20	0.00	7.44	11.55
Plaza	15.92	86	2623	3252.52	16262.60	0.00	19515.12	30295.65
P6	9.83	86	1577	1955.07	9775.33	0.00	11730.40	18210.50
4th	12.83	86	2093	2594.91	12974.53	0.00	15569.44	24170.30
5th	11.75	86	1906	2363.85	11819.27	0.00	14183.12	22018.15
6th	11.75	86	1906	2363.85	11819.27	0.00	14183.12	22018.15
7th	11.75	86	1906	2363.85	11819.27	0.00	14183.12	22018.15
8th	11.75	86	1906	2363.85	11819.27	0.00	14183.12	22018.15
9th	11.75	86	1906	2363.85	11819.27	0.00	14183.12	22018.15
10th	11.75	86	1906	2363.85	11819.27	0.00	14183.12	22018.15
11th	11.75	86	1906	2363.85	11819.27	0.00	14183.12	22018.15
Roof	21.17	86	3526	4372.24	21861.20	0.00	26233.44	40725.30
Total			**23163**	**$28,721.71**	**$143,608.53**	**$0.00**	**$172,330.24**	**$267,528.80**

Level	Height	f'c	Size	#	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
Stuctural Concrete - Columns - 03 31 13.25 (0400)										
						110	0	0	110	121
Plaza	15.92	5000	24"x24"	54	127.33	14006.67	0.00	0.00	14006.67	15407.33
			30" ϕ	12	34.72	3819.73	0.00	0.00	3819.73	4201.70
			12"x24"	8	9.43	1037.53	0.00	0.00	1037.53	1141.28
P6	9.83	5000	24"x24"	40	58.27	6409.88	0.00	0.00	6409.88	7050.86
			30" ϕ	9	16.09	1769.87	0.00	0.00	1769.87	1946.86
			12"x24"	8	5.83	640.99	0.00	0.00	640.99	705.09
4th	12.83	5000	24"x24"	40	76.05	8365.43	0.00	0.00	8365.43	9201.98
			30" ϕ	9	21.00	2309.83	0.00	0.00	2309.83	2540.82
			12"x24"	8	7.60	836.54	0.00	0.00	836.54	920.20
Stuctural Concrete - Columns - 03 31 13.25 (0300)										
Level	Height	f'c	Size	#	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
						104	0	0	104	114
5th	11.75	4000	24"x24"	40	69.63	7241.48	0.00	0.00	7241.48	7937.78
			30" ϕ	9	19.23	1999.49	0.00	0.00	1999.49	2191.75
			12"x24"	8	6.96	724.15	0.00	0.00	724.15	793.78
6th	11.75	4000	24"x24"	40	69.63	7241.48	0.00	0.00	7241.48	7937.78
			30" ϕ	9	19.23	1999.49	0.00	0.00	1999.49	2191.75
			12"x24"	8	6.96	724.15	0.00	0.00	724.15	793.78
7th	11.75	4000	24"x24"	40	69.63	7241.48	0.00	0.00	7241.48	7937.78
			30" ϕ	9	19.23	1999.49	0.00	0.00	1999.49	2191.75
			12"x24"	8	6.96	724.15	0.00	0.00	724.15	793.78
8th	11.75	4000	24"x24"	40	69.63	7241.48	0.00	0.00	7241.48	7937.78
			30" ϕ	9	19.23	1999.49	0.00	0.00	1999.49	2191.75
			12"x24"	8	6.96	724.15	0.00	0.00	724.15	793.78
Stuctural Concrete - Columns - 03 31 13.25 (0150)										
Level	Height	f'c	Size	#	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
						99	0	0	99	109
9th	11.75	3000	24"x24"	40	69.63	6893.33	0.00	0.00	6893.33	7589.63
			30" ϕ	9	19.23	1903.36	0.00	0.00	1903.36	2095.62
			12"x24"	8	6.96	689.33	0.00	0.00	689.33	758.96
10th	11.75	3000	24"x24"	40	69.63	6893.33	0.00	0.00	6893.33	7589.63
			30" ϕ	9	19.23	1903.36	0.00	0.00	1903.36	2095.62
			12"x24"	8	6.96	689.33	0.00	0.00	689.33	758.96
11th	11.75	3000	24"x24"	40	69.63	6893.33	0.00	0.00	6893.33	7589.63
			30" ϕ	5	10.68	1057.42	0.00	0.00	1057.42	1164.23
			18"x18"	4	3.92	387.75	0.00	0.00	387.75	426.92
			12"x24"	8	6.96	689.33	0.00	0.00	689.33	758.96
Roof	21.17	3000	24"x24"	10	31.36	3104.44	0.00	0.00	3104.44	3418.02
			18"x18"	5	19.24	1904.86	0.00	0.00	1904.86	2097.27
			12"x24"	8	12.54	1241.78	0.00	0.00	1241.78	1367.21
Total					**1085.58**	**$113,307.95**	**$0.00**	**$0.00**	**$113,307.95**	**$124,520.03**

Stuctural Concrete - Slabs and Drop Panels - 03 31 13.25 (0350)											
Level	S.F.	Slab Depth	Drop Depth	Drop Size	# of Drops	C.Y.	Material 107	Labor 0	Equiptment 0	Total 107	Tot Incl O&P 117
P6	24893	8	3.25	8'x8'	26	631.33	67552.67	0.00	0.00	67552.67	73866.00
			8	8'x8'	7	11.06	1183.60	0.00	0.00	1183.60	1294.22
4th	23058	8	3.25	8'x8'	3	571.26	61124.74	0.00	0.00	61124.74	66837.33
5th	23058	8	3.25	8'x8'	3	571.26	61124.74	0.00	0.00	61124.74	66837.33
6th	23058	8	3.25	8'x8'	3	571.26	61124.74	0.00	0.00	61124.74	66837.33
7th	23058	8	3.25	8'x8'	3	571.26	61124.74	0.00	0.00	61124.74	66837.33
8th	23058	8	3.25	8'x8'	3	571.26	61124.74	0.00	0.00	61124.74	66837.33
9th	23058	8	3.25	8'x8'	3	571.26	61124.74	0.00	0.00	61124.74	66837.33
10th	23058	8	3.25	8'x8'	3	571.26	61124.74	0.00	0.00	61124.74	66837.33
11th	22102	8	3.25	8'x8'	3	547.65	58599.01	0.00	0.00	58599.01	64075.56
Roof	22102	8	3.25	8'x8'	9	551.51	59011.16	0.00	0.00	59011.16	64526.22
			5.5	8'x8'	8	8.69	929.98	0.00	0.00	929.98	1016.89
Total						**5749.062**	**$615,149.60**	**$0.00**	**$0.00**	**$615,149.60**	**$672,640.22**

Stuctural Concrete - Beams - 03 31 13.25 (0350)										
Level	Size Int Beams	L.F. Int Beams	Size Ext Beams	L.F. Ext Beams	C.Y.	Material 107	Labor 0	Equiptment 0	Total 107	Tot Incl O&P 117
P6	24"x36"	46	24"x20"	86	50.54	5408.12	0.00	0.00	5408.12	5913.56
	12"x30"	60	36"x24"	145						
	48"x20"	72	-	-						
4th	48"x20"	604	24"x20"	86	121.41	12990.59	0.00	0.00	12990.59	14204.67
	12"x30"	60	36"x24"	145						
5th	48"x20"	604	24"x20"	86	121.41	12990.59	0.00	0.00	12990.59	14204.67
	12"x30"	60	36"x24"	145						
6th	48"x20"	604	24"x20"	86	121.41	12990.59	0.00	0.00	12990.59	14204.67
	12"x30"	60	36"x24"	145						
7th	48"x20"	604	24"x20"	86	121.41	12990.59	0.00	0.00	12990.59	14204.67
	12"x30"	60	36"x24"	145						
8th	48"x20"	604	24"x20"	86	121.41	12990.59	0.00	0.00	12990.59	14204.67
	12"x30"	60	36"x24"	145						
9th	48"x20"	604	24"x20"	86	121.41	12990.59	0.00	0.00	12990.59	14204.67
	12"x30"	60	36"x24"	145						
10th	48"x20"	604	24"x20"	86	121.41	12990.59	0.00	0.00	12990.59	14204.67
	12"x30"	60	36"x24"	145						
11th	48"x20"	604	24"x20"	86	121.41	12990.59	0.00	0.00	12990.59	14204.67
	12"x30"	60	36"x24"	145						
Roof	48"x20"	604	24"x20"	86	124.52	13323.48	0.00	0.00	13323.48	14568.67
	12"x30"	60	36"x24"	166						
Pent	12"x24"	165	18"x24"	160	20.00	2140.00	0.00	0.00	2140.00	2340.00
Total					**1166.32**	**$124,796.35**	**$0.00**	**$0.00**	**$124,796.35**	**$136,459.56**
Note - Above values based on all floors having an 8" slab which will be counted for in the slab and drop panel table										

Stuctural Concrete - Shear Walls - 03 31 13.25 (0400)									
Level	Height	Wall Length	Wall Width	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
					110	0	0	110	121
Plaza	15.92	86	12"	50.70	5576.73	0.00	0.00	5576.73	6134.40
P6	9.83	86	12"	31.32	3445.31	0.00	0.00	3445.31	3789.84
4th	12.83	86	12"	40.88	4496.42	0.00	0.00	4496.42	4946.06
Stuctural Concrete - Shear Walls - 03 31 13.25 (0300)									
Level	Height	Wall Length	Wall Width	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
					104	0	0	104	114
5th	11.75	86	12"	37.43	3892.30	0.00	0.00	3892.30	4266.56
6th	11.75	86	12"	37.43	3892.30	0.00	0.00	3892.30	4266.56
7th	11.75	86	12"	37.43	3892.30	0.00	0.00	3892.30	4266.56
8th	11.75	86	12"	37.43	3892.30	0.00	0.00	3892.30	4266.56
Stuctural Concrete - Shear Walls - 03 31 13.25 (0150)									
Level	Height	Wall Length	Wall Width	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
					99	0	0	99	109
9th	11.75	86	12"	37.43	3705.17	0.00	0.00	3705.17	4079.43
10th	11.75	86	12"	37.43	3705.17	0.00	0.00	3705.17	4079.43
11th	11.75	86	12"	37.43	3705.17	0.00	0.00	3705.17	4079.43
Roof	21.17	86	12"	67.42	6674.56	0.00	0.00	6674.56	7348.75
Total				**452.30**	**$46,877.70**	**$0.00**	**$0.00**	**$46,877.70**	**$51,523.56**

Placing Concrete - Shear Walls - 3 31 13.70 (5100)									
Level	Height	Wall Length	Wall Width	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
					0	23	7.05	30.05	45
Plaza	15.92	86	12"	50.70	0.00	1166.04	357.42	1523.46	2281.39
P6	9.83	86	12"	31.32	0.00	720.38	220.81	941.20	1409.44
4th	12.83	86	12"	40.88	0.00	940.16	288.18	1228.34	1839.44
5th	11.75	86	12"	37.43	0.00	860.80	263.85	1124.65	1684.17
6th	11.75	86	12"	37.43	0.00	860.80	263.85	1124.65	1684.17
7th	11.75	86	12"	37.43	0.00	860.80	263.85	1124.65	1684.17
Placing Concrete - Shear Walls - 3 31 13.70 (5200)									
Level	Height	Wall Length	Wall Width	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
					0	32	13.6	45.6	66.5
8th	11.75	86	12"	37.43	0.00	1197.63	508.99	1706.62	2488.82
9th	11.75	86	12"	37.43	0.00	1197.63	508.99	1706.62	2488.82
10th	11.75	86	12"	37.43	0.00	1197.63	508.99	1706.62	2488.82
11th	11.75	86	12"	37.43	0.00	1197.63	508.99	1706.62	2488.82
Roof	21.17	86	12"	67.42	0.00	2157.43	916.91	3074.34	4483.41
Total				**452.30**	**$0.00**	**$12,356.93**	**$4,610.85**	**$16,967.77**	**$25,021.49**

Placing Concrete - Columns - 03 31 13.70 (0800)									
Level	Height	Size	#	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
					0	27.5	8.45	35.95	54
Plaza	15.92	24"x24"	54	127.33	0.00	3501.67	1075.97	4577.63	6876.00
		30" ϕ	12	34.72	0.00	954.93	293.42	1248.36	1875.14
		12"x24"	8	9.43	0.00	259.38	79.70	339.08	509.33
P6	9.83	24"x24"	40	58.27	0.00	1602.47	492.40	2094.86	3146.67
		30" ϕ	9	16.09	0.00	442.47	135.96	578.43	868.85
		12"x24"	8	5.83	0.00	160.25	49.24	209.49	314.67
4th	12.83	24"x24"	40	76.05	0.00	2091.36	642.62	2733.98	4106.67
		30" ϕ	9	21.00	0.00	577.46	177.44	754.90	1133.92
		12"x24"	8	7.60	0.00	209.14	64.26	273.40	410.67
5th	11.75	24"x24"	40	69.63	0.00	1914.81	588.37	2503.19	3760.00
		30" ϕ	9	19.23	0.00	528.71	162.46	691.17	1038.20
		12"x24"	8	6.96	0.00	191.48	58.84	250.32	376.00
6th	11.75	24"x24"	40	69.63	0.00	1914.81	588.37	2503.19	3760.00
		30" ϕ	9	19.23	0.00	528.71	162.46	691.17	1038.20
		12"x24"	8	6.96	0.00	191.48	58.84	250.32	376.00
7th	11.75	24"x24"	40	69.63	0.00	1914.81	588.37	2503.19	3760.00
		30" ϕ	9	19.23	0.00	528.71	162.46	691.17	1038.20
		12"x24"	8	6.96	0.00	191.48	58.84	250.32	376.00
Placing Concrete - Columns - 03 31 13.70 (0850)									
Level	Height	Size	#	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
					0	41	17.5	58.5	86
8th	11.75	24"x24"	40	69.63	0.00	2854.81	1218.52	4073.33	5988.15
		30" ϕ	9	19.23	0.00	788.26	336.45	1124.71	1653.43
		12"x24"	8	6.96	0.00	285.48	121.85	407.33	598.81
9th	11.75	24"x24"	40	69.63	0.00	2854.81	1218.52	4073.33	5988.15
		30" ϕ	9	19.23	0.00	788.26	336.45	1124.71	1653.43
		12"x24"	8	6.96	0.00	285.48	121.85	407.33	598.81
10th	11.75	24"x24"	40	69.63	0.00	2854.81	1218.52	4073.33	5988.15
		30" ϕ	9	19.23	0.00	788.26	336.45	1124.71	1653.43
		12"x24"	8	6.96	0.00	285.48	121.85	407.33	598.81
11th	11.75	24"x24"	40	69.63	0.00	2854.81	1218.52	4073.33	5988.15
		30" ϕ	5	10.68	0.00	437.92	186.92	624.84	918.57
		18"x18"	4	3.92	0.00	160.58	68.54	229.13	336.83
		12"x24"	8	6.96	0.00	285.48	121.85	407.33	598.81
Roof	21.17	24"x24"	10	31.36	0.00	1285.68	548.77	1834.44	2696.79
		18"x18"	5	19.24	0.00	788.88	336.72	1125.60	1654.73
		12"x24"	8	12.54	0.00	514.27	219.51	733.78	1078.72
Total				**1085.58**	**$0.00**	**$35,817.45**	**$13,171.29**	**$48,988.74**	**$72,758.27**

Placing Concrete - Slabs and Drop Panels - 03 31 13.70 (1500)											
Level	S.F.	Slab Depth	Drop Depth	Drop Size	# of Drops	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
							0	15.75	4.85	20.6	31
P6	24893	8	3.25	8'x8'	26	642.40	0.00	10117.72	3115.62	13233.34	19914.25
			8	8'x8'	7						
4th	23058	8	3.25	8'x8'	3	571.26	0.00	8997.33	2770.61	11767.94	17709.04
5th	23058	8	3.25	8'x8'	3	571.26	0.00	8997.33	2770.61	11767.94	17709.04
6th	23058	8	3.25	8'x8'	3	571.26	0.00	8997.33	2770.61	11767.94	17709.04
7th	23058	8	3.25	8'x8'	3	571.26	0.00	8997.33	2770.61	11767.94	17709.04
Placing Concrete - Slabs and Drop Panels - 03 31 13.70 (1550)											
Level	S.F.	Slab Depth	Drop Depth	Drop Size	# of Drops	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
							0	26	11.15	37.15	54.5
8th	23058	8	3.25	8'x8'	3	571.26	0.00	14852.74	6369.54	21222.28	31133.63
9th	23058	8	3.25	8'x8'	3	571.26	0.00	14852.74	6369.54	21222.28	31133.63
10th	23058	8	3.25	8'x8'	3	571.26	0.00	14852.74	6369.54	21222.28	31133.63
11th	22102	8	3.25	8'x8'	3	547.65	0.00	14239.01	6106.35	20345.36	29847.16
Roof	22102	8	3.25	8'x8'	9	560.20	0.00	14565.14	6246.20	20811.34	30530.77
			5.5	8'x8'	8						
Total						**5749.062**	**$0.00**	**$119,469.43**	**$45,659.22**	**$165,128.64**	**$244,529.21**

Placing Concrete - Beams - 03 31 13.70 (0050)										
Level	Size Int Beams	L.F. Int Beams	Size Ext Beams	L.F. Ext Beams	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
						0	42	12.95	54.95	82.5
P6	24"x36"	46	24"x20"	86	50.54	0.00	2122.81	654.53	2777.35	4169.81
	12"x30"	60	36"x24"	145						
	48"x20"	72	-	-						
4th	48"x20"	604	24"x20"	86	121.41	0.00	5099.11	1572.23	6671.34	10016.11
	12"x30"	60	36"x24"	145						
5th	48"x20"	604	24"x20"	86	121.41	0.00	5099.11	1572.23	6671.34	10016.11
	12"x30"	60	36"x24"	145						
6th	48"x20"	604	24"x20"	86	121.41	0.00	5099.11	1572.23	6671.34	10016.11
	12"x30"	60	36"x24"	145						
7th	48"x20"	604	24"x20"	86	121.41	0.00	5099.11	1572.23	6671.34	10016.11
	12"x30"	60	36"x24"	145						
Placing Concrete - Beams - 03 31 13.70 (0100)										
Level	Size Int Beams	L.F. Int Beams	Size Ext Beams	L.F. Ext Beams	C.Y.	Material	Labor	Equiptment	Total	Tot Incl O&P
						0	63.5	27.5	91	133
8th	48"x20"	604	24"x20"	86	121.41	0.00	7709.37	3338.70	11048.07	16147.19
	12"x30"	60	36"x24"	145						
9th	48"x20"	604	24"x20"	86	121.41	0.00	7709.37	3338.70	11048.07	16147.19
	12"x30"	60	36"x24"	145						
10th	48"x20"	604	24"x20"	86	121.41	0.00	7709.37	3338.70	11048.07	16147.19
	12"x30"	60	36"x24"	145						
11th	48"x20"	604	24"x20"	86	121.41	0.00	7709.37	3338.70	11048.07	16147.19
	12"x30"	60	36"x24"	145						
Roof	48"x20"	604	24"x20"	86	124.52	0.00	7906.93	3424.26	11331.19	16560.96
	12"x30"	60	36"x24"	166						
Pent	12"x24"	165	18"x24"	160	20.00	0.00	1270.00	550.00	1820.00	2660.00
Total					**1166.32**	**$0.00**	**$62,533.67**	**$24,272.51**	**$86,806.18**	**$128,043.96**

Note - Above values based on all floors having an 8" slab which will be counted for in the slab and drop panel table

Concrete Finishing - Slabs - 03 35 13.30 (0250)						
Level	S.F.	Material	Labor	Equiptment	Total	Tot Incl O&P
		0	0.58	0.03	0.61	0.96
P6	24893	0	14437.94	746.79	15184.73	23897.28
4th	23058	0	13373.64	691.74	14065.38	22135.68
5th	23058	0	13373.64	691.74	14065.38	22135.68
6th	23058	0	13373.64	691.74	14065.38	22135.68
7th	23058	0	13373.64	691.74	14065.38	22135.68
8th	23058	0	13373.64	691.74	14065.38	22135.68
9th	23058	0	13373.64	691.74	14065.38	22135.68
10th	23058	0	13373.64	691.74	14065.38	22135.68
11th	22102	0	12819.16	663.06	13482.22	21217.92
Roof	22102	0	12819.16	663.06	13482.22	21217.92
Total		**$0.00**	**$133,691.74**	**$6,915.09**	**$140,606.83**	**$221,282.88**

Finishing Concrete - Walls and Columns - 03 35 29.60 (0020)									
Level	Height	SFCA (Walls)	SFCA (Cols)	SF Total per Floor	Material	Labor	Equiptment	Total	Tot Incl O&P
					0.04	0.65	0	0.69	1.07
Plaza	15.92	2610	6982	9593	383.71	6235.29	0.00	6619.00	10264.24
P6	9.83	1613	4314	5926	237.06	3852.17	0.00	4089.22	6341.26
4th	12.83	2105	5630	7734	309.38	5027.40	0.00	5336.78	8275.88
5th	11.75	1927	5155	7082	283.26	4603.01	0.00	4886.28	7577.27
6th	11.75	1927	5155	7082	283.26	4603.01	0.00	4886.28	7577.27
7th	11.75	1927	5155	7082	283.26	4603.01	0.00	4886.28	7577.27
8th	11.75	1927	5155	7082	283.26	4603.01	0.00	4886.28	7577.27
9th	11.75	1927	5155	7082	283.26	4603.01	0.00	4886.28	7577.27
10th	11.75	1927	5155	7082	283.26	4603.01	0.00	4886.28	7577.27
11th	11.75	1927	5067	6994	279.78	4546.37	0.00	4826.15	7484.03
Roof	21.17	3471	3344	6816	272.63	4430.18	0.00	4702.81	7292.76
Total				**79553.07**	**$3,182.12**	**$51,709.49**	**$0.00**	**$54,891.62**	**$85,121.78**

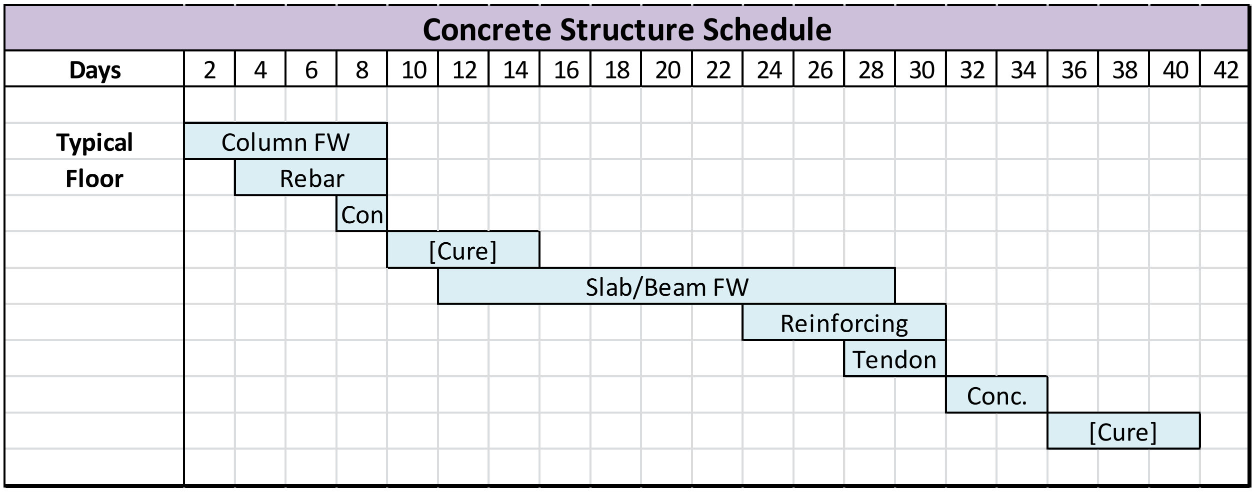
Reinforcing Bars - Slabs - 03 21 11.60 (0402)							
Level		Tons	Material	Labor	Equiptment	Total	Tot Incl O&P
			1000	560	0	1560	2020
P6	23058	32.89	32890.00	18418.40	0.00	51308.40	66437.80
4th	23058	29.79	29790.00	16682.40	0.00	46472.40	60175.80
5th	23058	29.79	29790.00	16682.40	0.00	46472.40	60175.80
6th	23058	29.79	29790.00	16682.40	0.00	46472.40	60175.80
7th	23058	29.79	29790.00	16682.40	0.00	46472.40	60175.80
8th	23058	29.79	29790.00	16682.40	0.00	46472.40	60175.80
9th	23058	29.79	29790.00	16682.40	0.00	46472.40	60175.80
10th	22102	28.55	28554.89	15990.74	0.00	44545.62	57680.87
11th	22102	28.55	28554.89	15990.74	0.00	44545.62	57680.87
Roof	4000	5.17	5167.84	2893.99	0.00	8061.83	10439.03
Total		**273.91**	**$273,907.61**	**$153,388.26**	**$0.00**	**$427,295.87**	**$553,293.37**

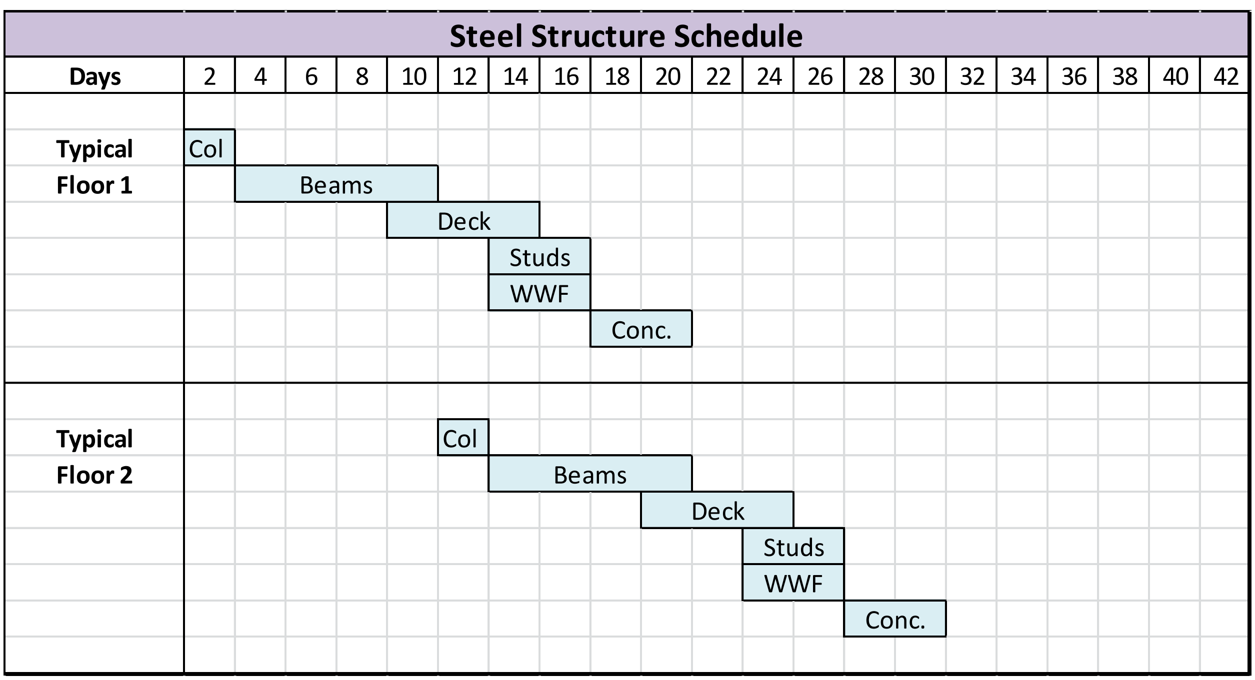
Reinforcing Bars - Columns - 03 21 11.60 (0202)								
Level	Height	#	Tons	Material	Labor	Equiptment	Total	Tot Incl O&P
				1000	1080	0	2080	2880
Plaza	15.92	74	24.02	24016.02	25937.30	0.00	49953.33	69166.14
P6	9.83	57	11.43	11428.60	12342.88	0.00	23771.48	32914.35
4th	12.83	57	14.92	14915.29	16108.51	0.00	31023.79	42956.02
5th	11.75	57	13.66	13656.20	14748.70	0.00	28404.90	39329.86
6th	11.75	57	13.66	13656.20	14748.70	0.00	28404.90	39329.86
7th	11.75	57	13.66	13656.20	14748.70	0.00	28404.90	39329.86
8th	11.75	57	13.66	13656.20	14748.70	0.00	28404.90	39329.86
9th	11.75	57	13.66	13656.20	14748.70	0.00	28404.90	39329.86
10th	11.75	57	13.66	13656.20	14748.70	0.00	28404.90	39329.86
11th	11.75	57	13.66	13656.20	14748.70	0.00	28404.90	39329.86
Roof	21.17	23	9.93	9926.53	10720.65	0.00	20647.19	28588.41
Total			**155.88**	**$155,879.85**	**$168,350.24**	**$0.00**	**$324,230.09**	**$448,933.97**
Note - Values above are based on the typical column requiring								

Reinforcing Bars - Walls - 03 21 11.60 (0702)								
Level	Height	Length	Tons	Material	Labor	Equiptment	Total	Tot Incl O&P
				1000	540	0	1540	1980
Plaza	15.92	86	3.18	3183.33	1719.00	0.00	4902.33	6303.00
P6	9.83	86	1.97	1966.67	1062.00	0.00	3028.67	3894.00
4th	12.83	86	2.57	2566.67	1386.00	0.00	3952.67	5082.00
5th	11.75	86	2.35	2350.00	1269.00	0.00	3619.00	4653.00
6th	11.75	86	2.35	2350.00	1269.00	0.00	3619.00	4653.00
7th	11.75	86	2.35	2350.00	1269.00	0.00	3619.00	4653.00
8th	11.75	86	2.35	2350.00	1269.00	0.00	3619.00	4653.00
9th	11.75	86	2.35	2350.00	1269.00	0.00	3619.00	4653.00
10th	11.75	86	2.35	2350.00	1269.00	0.00	3619.00	4653.00
11th	11.75	86	2.35	2350.00	1269.00	0.00	3619.00	4653.00
Roof	21.17	86	4.23	4233.33	2286.00	0.00	6519.33	8382.00
Total			**28.40**	**$28,400.00**	**$15,336.00**	**$0.00**	**$43,736.00**	**$56,232.00**
Note - Values above are based on #4's at 12" o.c. each way but include (4) #11 bars at each wall corner for bounday elements								

Post Tensioned Reinforcing Cables - Beams - 03 23 05.50 (2220)						
Level	Tons	Material	Labor	Equiptment	Total	Tot Incl O&P
		1220	2720	60	4000	5880
P6	0.37	456.28	1017.28	22.44	1496	2199.12
4th	3.14	3832.02	8543.52	188.46	12564	18469.08
5th	3.14	3832.02	8543.52	188.46	12564	18469.08
6th	3.14	3832.02	8543.52	188.46	12564	18469.08
7th	3.14	3832.02	8543.52	188.46	12564	18469.08
8th	3.14	3832.02	8543.52	188.46	12564	18469.08
9th	3.14	3832.02	8543.52	188.46	12564	18469.08
10th	3.14	3832.02	8543.52	188.46	12564	18469.08
11th	3.14	3832.02	8543.52	188.46	12564	18469.08
Roof	3.14	3832.02	8543.52	188.46	12564	18469.08
Total	**28.64**	**$34,944.46**	**$77,908.96**	**$1,718.58**	**$114,572.00**	**$168,420.84**

Schedule Comparison





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Redesigned Structure Schedule

Columns:

$\frac{280 \text{ tons}}{14.2 \text{ tons}} = 19.7$ days → 1.8 days per floor

Beams: $\frac{59.5 \text{ tons}}{14.2 \text{ tons}} = 4.2$ days / floor

Studs: $\frac{1481}{950} = 1.56$ days / floor

Slabs:

Decking: $\frac{23058 \text{ SF}}{3600 \text{ SF}} = 6.41$ → 2.2 days / floor (3 crews)

WWF: $\frac{230.58 \text{ CSF}}{35 \text{ CSF}} = 6.59$ → 2.2 days / floor (3 crews)

Concrete: $\frac{302.3 \text{ CY}}{140 \text{ CY}} = 2.16$ → 1.08 days / floor (2 crews)

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Original Structure Schedule

Columns:

Forms: $\frac{5155 \text{ SFCA}}{216 \text{ SFCA}} = 23.87 \rightarrow 7.96$ days / floor (3 crews)

Rebar: $\frac{27320 \text{ lbs}}{3000 \text{ lbs}} = 9.11 \rightarrow 4.6$ day / floor (2 crews)

Concrete: $\frac{95.8 \text{ C.Y.}}{92 \text{ C.Y.}} = 1.04$ days / floor (1 crew)

Beams / Slabs:

Forms: $\frac{4870 \text{ SFCA}}{365 \text{ SFCA}} + \frac{29456 \text{ SFCA}}{543 \text{ SFCA}} = 67.5 \rightarrow 22.5$ days/floor (3 crews)

Reinforcing: $\frac{59580 \text{ lbs}}{3200 \text{ lbs}} = 18.62 \rightarrow 6.2$ days / floor (3 crews)

Tendons: $\frac{6300 \text{ lbs}}{1200 \text{ lbs}} = 5.25 \rightarrow 2.6$ days / floor (2 crews)

Concrete: $\frac{730 \text{ C.Y.}}{140 \text{ C.Y.}} = 5.21 \rightarrow 2.6$ days / floor (2 crews)

JMV Thesis

Super Structure Schedule Comparison

Steel Project Duration

$10(10) + 10 = 110$ days

Concrete Project Duration

$10(34) + 6 = 346$ days

Days Saved

$346 - 110 = 236$ work days

$236 \times 7/5 = 330.4$ days

≈ 10.9 months

Assume 11 months saved on schedule