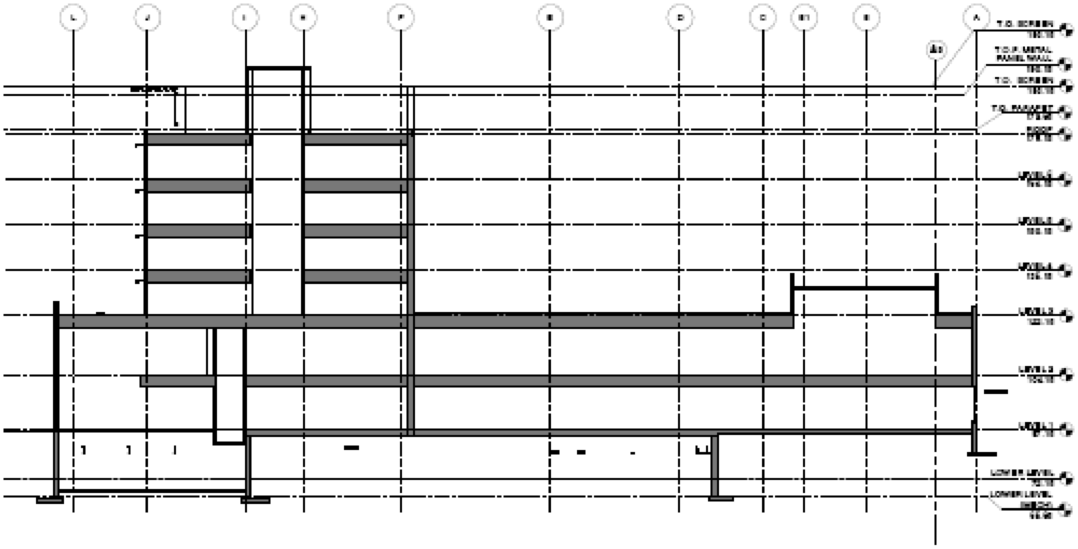
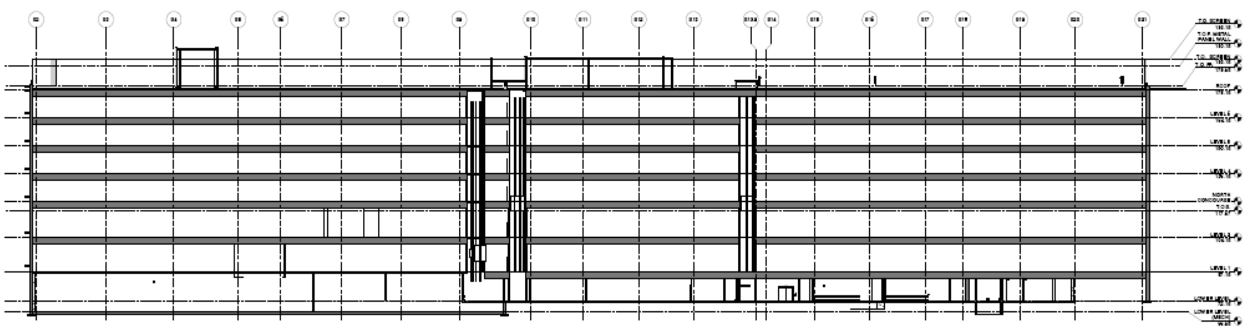
Appendix 1: Architectural Sections & Plans



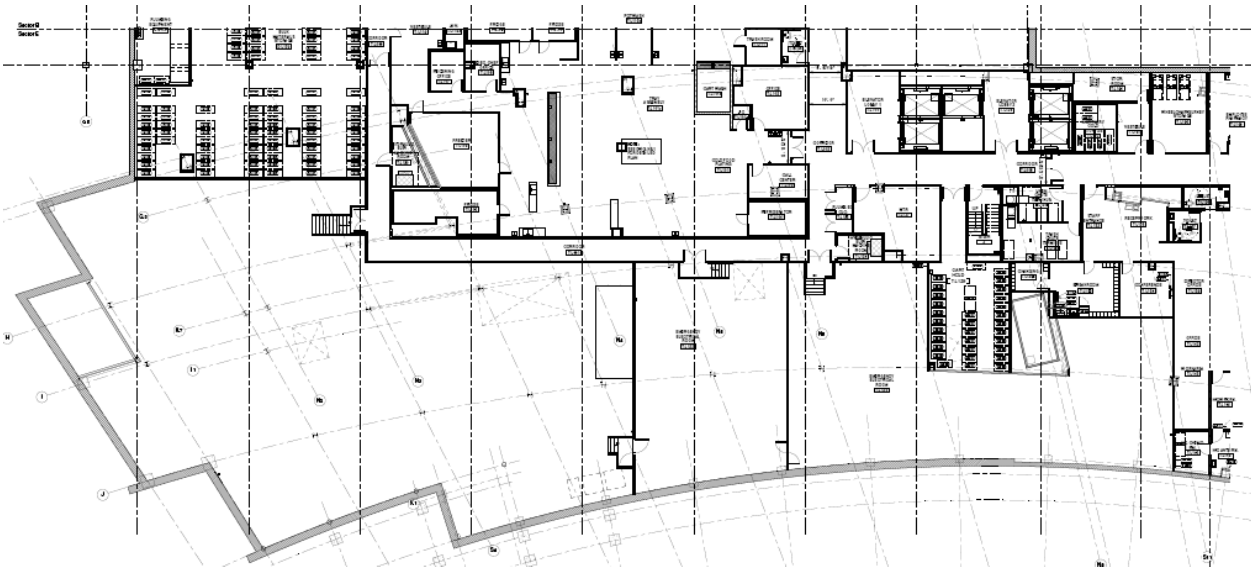
EAST/WEST SECTION

COURTESY OF TURNER CONSTRUCTION



NORTH/SOUTH SECTION

COURTESY OF TURNER CONSTRUCTION



TYPICAL WEST END FLOOR PLAN

COURTESY OF TURNER CONSTRUCTION

TYPICAL WEST END FLOOR PLAN

COURTESY OF TURNER CONSTRUCTION

Appendix 2: Slab Design

SOLID ONE-WAY SLABS—SINGLE SPAN

f'_c = 4,000 psi

Grade 60 Bars

Bottom Steel for + M_u

$\rho \approx 0.0050$

ckness (in.)	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
ttom Bars Spacing (in.)	#4 12	#4 11	#4 10	#4 8	#5 12	#5 11	#5 10	#5 9	#6 12	#6 11	#6 11	#6 10	#6 9
p Bars Spacing (in.)	#3 12	#3 12	#3 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12
S Bars Spacing (in.)	#3 11	#3 11	#3 11	#3 11	#3 10	#3 9	#3 8	#3 8	#3 7	#3 7	#3 6	#4 11	#4 11
eas of Steel 2/ft) Bottom	0.200	0.218	0.240	0.300	0.310	0.338	0.372	0.413	0.440	0.480	0.480	0.528	0.587
ab Wt. (psf) eel Wt. (psf)	50 1.25	56 1.31	63 1.38	69 1.73	75 1.83	81 1.96	88 2.15	94 2.29	100 2.48	106 2.61	113 2.72	119 2.84	125 3.04
:LEAR SPAN	FACTORED USABLE SUPERIMPOSED LOAD (psf)												
6'-0"	510	661	841										
6'-6"	425	553	705										
7'-0"	359	467	598	860	982								
7'-6"	305	398	511	738	844								
8'-0"	260	342	440	639	730	889							
8'-6"	224	295	381	556	637	776	943						
9'-0"	193	256	332	487	558	682	830						
9'-6"	167	223	290	429	492	602	734	898					
10'-0"	145	194	254	379	435	534	652	799	917				
10'-6"	126	170	223	336	386	475	582	714	821	973			
11'-0"	109	149	197	299	344	424	521	641	737	875	938		
11'-6"	95	131	174	266	307	380	467	577	664	790	847		
12'-0"	82	114	153	238	274	341	421	520	600	715	766	911	
12'-6"	71	100	135	212	246	306	379	471	544	649	696	828	991
13'-0"	61	87	119	190	220	276	343	427	493	590	633	755	905
13'-6"	52	76	105	170	198	249	310	387	449	538	577	690	828
14'-0"	44	66	92	152	178	225	281	352	409	491	527	631	759
14'-6"		57	81	136	159	203	255	321	373	449	482	579	698
15'-0"		49	71	122	143	183	231	292	341	411	441	531	642
15'-6"		41	61	109	128	165	210	266	311	377	405	488	592
16'-0"			53	97	115	149	190	243	285	346	372	450	546
16'-6"			45	86	102	134	172	222	261	318	341	414	504
17'-0"				77	91	121	156	202	239	292	314	382	466
17'-6"				68	81	109	142	185	218	268	288	352	432
18'-0"				59	72	97	128	168	200	247	265	325	400
18'-6"				52	63	87	115	153	183	227	244	300	370
19'-0"				45	55	77	104	139	167	208	224	277	343
19'-6"					48	68	93	127	152	191	206	256	318
20'-0"					41	60	83	115	139	176	189	236	295

Note: See Fig. 7-1 for reinforcing bar details.
*Service loads corresponding to 1/1.6 of the tabulated superimposed load results in calculated immediate deflection of 1/360 span.
"H" – Use hooked or headed bars.

Concrete Reinforcing Steel Institute					
Clear Span:	14.5	feet	1.2D+1.6L	170	psf
Thickness:	6.5	inch	Factored Load:		
f'c:	4	ksi	ῤ:	0.005	
Bar Grade:	60				
Concrete Slab Design					
Bottom Bars:	#7	spaced at	11	inch	
Top Bars:	#4	spaced at	12	inch	
T-S Bars	#3	spaced at	9	inch	
Ares of Steel:	0.655	in^2			
slab Wt.:	81	psf			

Total Weight for All Slab Reinforcement			
Number of Rebar Spaced Across the Slab	Length	Weight	
77	617.5	342.2	ton
71	617.5	102.5	ton
	Total:	**444.7**	ton

Appendix 3: Gravity Beam Design

	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	5		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	3.95	in^2			
Beam Design, B1							Tributary Area:	391	ft^2			
	Span:	26.5	feet	Two Row Reinforcement?	Yes		Influence Area:	782	ft^2	Live Load Reduction		
	Spaced:	14.75	feet				L_L=Max of	0.4				
								$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.79	L_L:	62.92
Spacing,S:	Max of d_b, 1", $3/4A_b$			$b_{min}=2*Cc+n*d_b+2d_{st}+(n-1)*S$								
d_b:	1.0	inch		b_{min}:	7.71	inch				Total Factored Weight, Wu		
$3/4A_b$:	0.6	inch								Dead Load: Misc. dead + Slab		
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)		Table 9.5 min h>l/18.5					116.25	psf
				h_{min}:	17.19	inch	min h:	17.2		Wu=1.2D +1.6L		
S:	1.0	inch								240.17004	psf	
Try a:												
b:	10	inch	OK		$d=h-d_b/2-Cc$			$a=A_s*fy/(.85*f'c*b_{eff})$			$c=a/\beta_1$	
h:	20	inch	OK		d:	17		a:	0.88	Rectangular Section	c:	1.03
b_{eff}= min	$b*16*h_f$	1040			$\varepsilon_t=\varepsilon_u(d-c)/c$							
	Trib width	177			ε_t:	0.05	Φ=0.9					
	.25L	79.5										
b_{eff}:	79.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Girder width, b:		10	inch		
Mu=Wu*Ln^2/8					Vu=Wu*Ln/2							
Mu:	291.7	kip-ft			Vu:	46.9	Kip					
$\Phi Mn=0.9*A_s fy(d-a/2)$					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	294.4	kip-ft	OK		Vn:	126491	Kip	OK				
Use Member Size:		**10**	x	**20**								
	Beam Wieght:	141	plf									

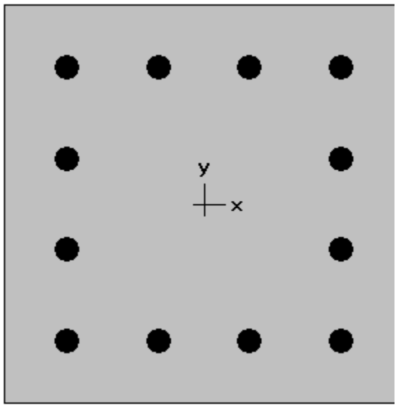
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	4		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	3.16	in^2			
Beam Design, B2							Tributary Area:	266	ft^2			
	Span:	18	feet	Two Row Reinforcement?	Yes		Influence Area:	531	ft^2	Live Load Reduction		
	Spaced:	14.75	feet				L_L=Max of	0.4				
								$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.90	L_L:	72.08
Spacing,S:	Max of d_b, 1", $3/4A_b$			$b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S$								
d_b:	1.0	inch		b_{min}:	6.71	inch				Total Factored Weight, Wu		
$3/4A_b$:	0.6	inch								Dead Load: Misc. dead + Slab		
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)		Table 9.5 min h>l/18.5					116.25	psf
					h_{min}:	11.68	inch	min h:	11.7	Wu=1.2D +1.6L		
S:	1.0	inch								254.8209	psf	
Try a:												
b:	10	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	20	inch	OK		d:	17		a:	1.03	Rectangular Section	c:	1.21
b_{eff}= min	b*16*h_f	1040			$\varepsilon_t=\varepsilon_u(d-c)/c$							
	Trib width	177			ε_t:	0.04	Φ=0.9					
	.25L	54										
b_{eff}:	54	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu			Girder width, b:	0	inch		
Mu=Wu*Ln^2/8					Vu=Wu*Ln/2							
Mu:	152.2	kip-ft			Vu:	33.8	Kip					
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	234.4	kip-ft	OK		Vn:	126491	Kip	OK				
Use Member Size:		10	x	20								
Beam Wieght:		141	plf									

Appendix 4: Gravity Girder Design

	Floor:	All	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	7		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	5.53	in^3			
	Typical											
	Girder Design, G1	(gravity)					Tributary Area:	656	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	Yes		Influence Area:	2626	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_r=Max of	0.4				
	Spaced Right:	18	feet					.25+15/√(K_{LL}*A_T)=		0.54	L_r:	43.42
Spacing,S	Max of d_b, 1", 3/4A_b			b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S								
d_b:	1.0	inch		b_{min}:	9.71	inch	Total Factored Weight, Wu					
3/4A_b:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch		Wu=1.2D +1.6L			Pu=1.2D	
S:	1.0	inch						209	psf		3755	pounds
Try a												
b:	10	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	20	inch	OK		d:	17		a:	1.10	Rectangular Section	c:	1.30
b_{eff}= min	b*16*h_f	1040			ε_t=ε_u(d-c)/c							
	Trib width	318			ε_t:	0.04	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu			Column width:	20	inch		
Mu=0.107*Ln^2+Pu*Ln/8, continuous + point load					Vu=Wu*Ln/2+Pu/2							
Mu:	398	kip-ft			Vu:	84	Kip					
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	409	kip-ft	OK		Vn:	126491	Kip	OK				
Use Member Size:		**10**	**x**	**20**								
	Beam Wieght:	141	plf									

	Floor:	All	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	meter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	9	Area of Steel, A_b:		0.79	in^2			
	β_1:	0.85		ε_u:	0.003	Area of Steel, A_s:		7.11	in^3			
Long Span												
Girder Design, G1	(gravity)						Tributary Area:	712	ft^2			
	Span:	32	feet	Two Row Reinforcement?	Yes		Influence Area:	2848	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_r=Max of	0.4				
	Spaced Right:	18	feet					$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.53	L_r:	42.49
Spacing,S	Max of d_b, 1", $3/4A_b$			$b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S$								
d_b:	1.0	inch		b_{min}:	11.71	inch	Total Factored Weight, Wu					
$3/4A_b$:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/21 (ACI 318-08)					116	psf	141	plf
				h_{min}:	18.29	inch	Wu=1.2D +1.6L				Pu=1.2D	
S:	1.0	inch							207	psf	3755	pounds
Try a												
b:	12	inch	OK		$d=h-d_b/2-Cc$			$a=A_s*fy/(.85*f'c*b_{eff})$			$c=a/\beta_1$	
h:	20	inch	OK		d:	17			a:	1.31	Rectangular	c:	1.54
b_{eff}= min	$b*16*h_f$	1248			$\varepsilon_t=\varepsilon_u(d-c)/c$							
	Trib widtl	318			ε_t:	0.03	Φ=0.9					
	.25L	96										
b_{eff}:	96	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		20	inch		
Mu=0.107*Ln^2+Pu*Ln/8, continuous + point load					Vu=Wu*Ln/2+Pu/2							
Mu:	469	kip-ft			Vu:	90	Kip					
$\Phi Mn=0.9*A_sfy(d-a/2)$					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	523	kip-ft	OK		Vn:	151789	Kip	OK				
Use Member Size:		**12**	**x**	**20**								
Beam Wieght:		169	plf									

Appendix 5: Gravity Column Design

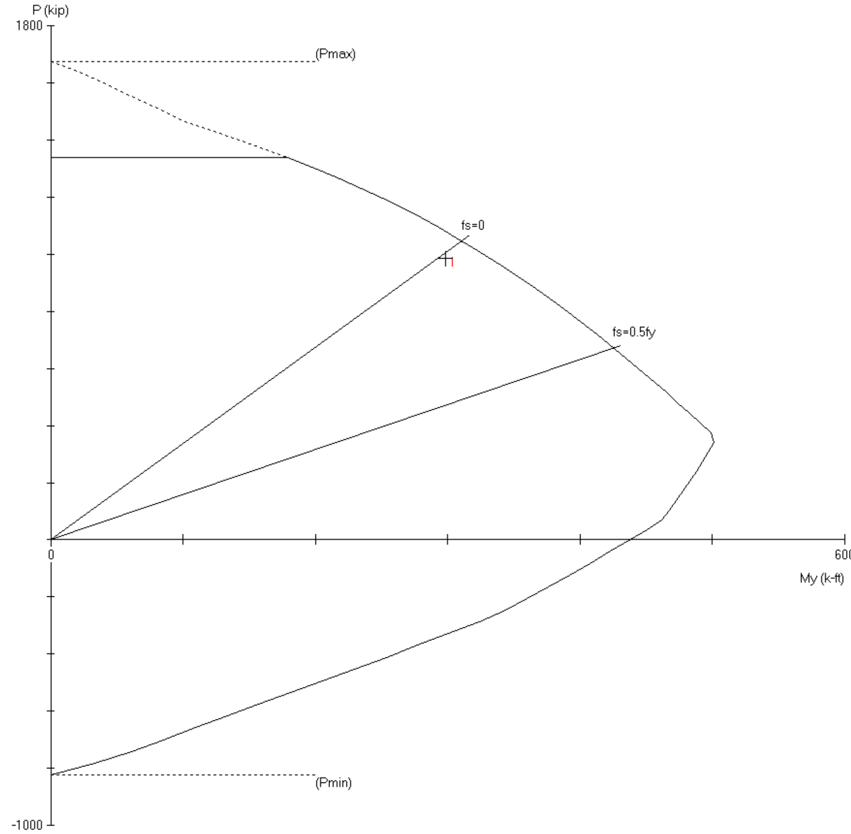


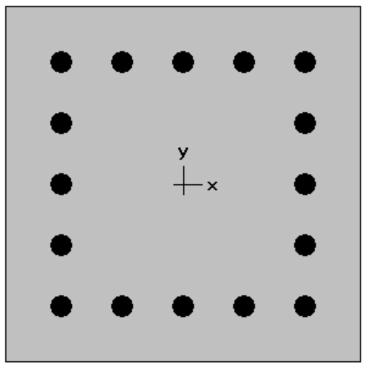
20 x 20 in
3.81% reinf.

MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 400 in^2
Ix = 13333.3 in^4
Iy = 13333.3 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
12 #10 bars @ 3.810%
As = 15.24 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 3.31 in



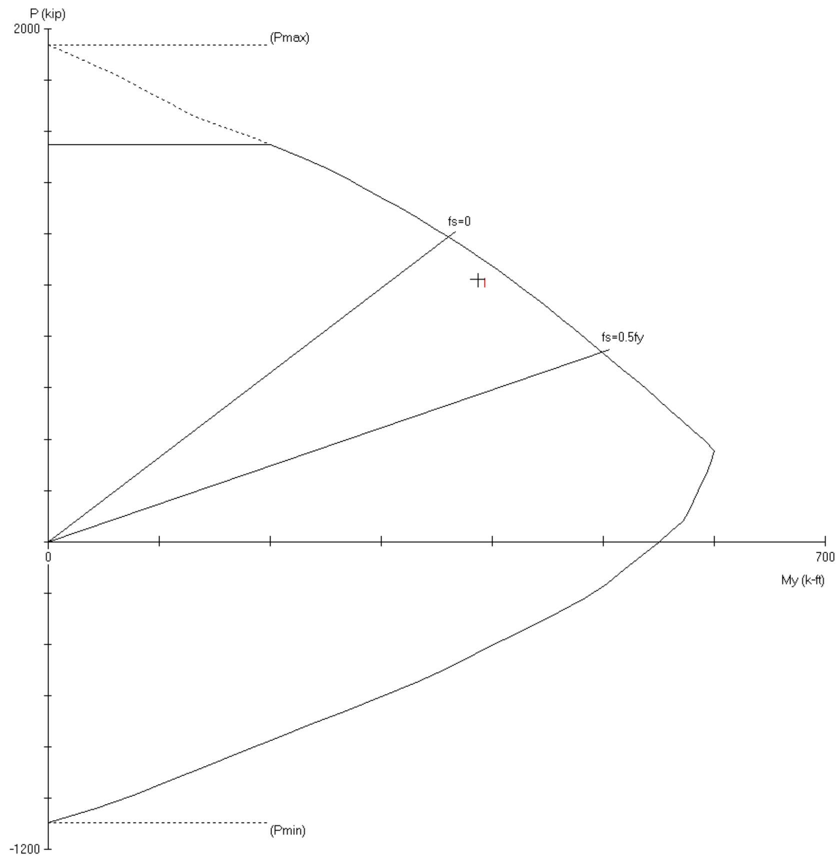


20 x 20 in
5.08% reinf.

MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 400 in^2
Ix = 13333.3 in^4
Iy = 13333.3 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
16 #10 bars @ 5.080%
As = 20.32 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 2.16 in



Appendix 6: Vibration Table

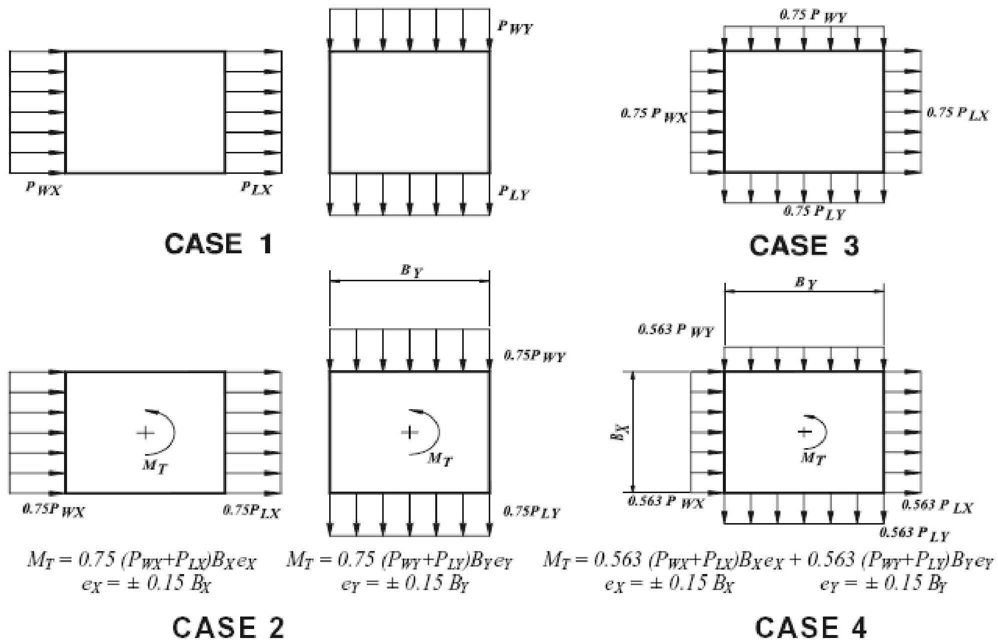
Table 1.2-5

Maximum Limits on Footfall Vibration in Health Care Facilities

Space Type	Footfall Vibration Peak Velocity (micro-in/s)
Patient rooms and other patient areas	4000
Operating and other treatment rooms	4000
Administrative areas	8000
Public circulation areas	8000

Appendix 7: Wind Calculations

Main Wind Force Resisting System – Part 1		All Heights
Figure 27.4-8	Design Wind Load Cases	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 27.4.1 and 27.4.2 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   - e (e_X, e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T: Torsional moment per unit height acting about a vertical axis of the building.

Alex Burg | Final Thesis Design | Wind Loads | 1/1

Short Direction

Exposure C
V = 120 mph (fig 26.5-1B)
I = 1.0 (Table 1.5-2)
kd = 0.85 (Table 26.6-1)
GCpi = ± 0.18 (Enclosed Building)
$k_{zt} = 1.0$ (26.8.2)
L_i = 370'

$L_{eff} = \frac{\Sigma h_i L_i}{\Sigma h_i} = 310$

· 91' < 300' } approximate
· 91' < 4(310) } frequency applies, n

$n_a = \frac{43.5}{h^{0.9}}$ (concrete moment resisting frame)

$n_a = \frac{43.5}{91^{0.9}} = \boxed{0.75} < 1.0$ Flexible

Gust Factor

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

$= 0.925\left(\frac{1+1.7(0.18)\sqrt{3.4^2(.87)^2+(4.12)^2(0.43)^2}}{1+1.7(3.4)(0.18)}\right)$

G = 0.93

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .2\left(\frac{33}{54.6}\right)^{1/6} = 0.18$

$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = .87$

$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 552$

$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}$

$g_R = 4.12$

$V_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V = 123.61$

$N_1 = \frac{n_a L_{\bar{z}}}{V_{\bar{z}}} = 3.36$

$R_n = 0.07$ $\beta = 0.02$

$R = 0.43$

Velocity Exposure Coefficient

$K_z = 2.01(z/z_g)^{2/\alpha}$

$= 2.01(17/900)^{2/9.5}$

$K_z = 0.87$ ← first floor only

Velocity Pressure, q_z

$q_z = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d V^2$

$= 0.00256(0.87)(.85)(1)(120)^2$

$= 27.31$

Wind Pressure, P

Refer to Wind Analysis Spread Sheet

Windward Pressure North/South										
floor	z	li	kz	q	windward, p	Windward Pressure	Windward Force	Leward, p	Leward Force	Floor Hight
1	0	85	0.85	26.63	18.18 (+/-) 4.79	22.98	72.26	23.61	74.26	0
2	17	85	0.87	27.26	18.61 (+/-) 4.91	23.52	152.28	23.61	152.88	17
3	35	85	1.01	31.65	21.61 (+/-) 5.70	27.30	161.63	23.61	139.78	18
4	49	85	1.085	34.00	23.21 (+/-) 6.12	29.33	151.93	23.61	122.30	14
5	63	85	1.142	35.78	24.43 (+/-) 6.44	30.87	159.91	23.61	122.30	14
6	77	85	1.198	37.54	25.63 (+/-) 6.76	32.38	167.75	23.61	122.30	14
roof	91	85	1.242	38.92	26.57 (+/-) 7.01	33.57	86.96	23.61	61.15	14
						Σ	880.47		720.72	

L_{EFF}	Base Shear	Over Turning Moment
1350	**1601.19 k**	**82767 k-ft**

B:	370.00
g_q:	3.40
gv:	3.40
c:	0.20
z(bar):	54.60
L_z:	552.98
b(bar):	0.65
ά:	0.15
Vz(bar):	123.61
l:	500.00
ε:	0.20
h:	91
L:	85
V:	120.00
α:	9.5
z_g:	900

G:	0.85
n_a:	0.75
I_z:	0.18
Q:	0.80
N_1:	3.36
R_n:	0.07
β:	0.02
g_R:	4.12
$R_{h,n}$:	2.54
$R_{L,n}$:	7.94
$R_{B,n}$:	10.33
R_h:	0.32
R_L:	0.12
R_B:	0.09
R:	0.24

Case 1, P	Case 2, Mt	Case 3, P	Case 4, Mt	Case 4, P
147	1939	110	1893	82
305	1962	229	1916	172
301	2119	226	2073	170
274	2204	206	2155	154
282	2268	212	2220	159
290	2331	218	2276	163
148	2380	111	2323	83

Windward Pressure East/West												
floor	z	li	kz	q	windward, p		Windward Pressure	Windward Force	Leward, p	Leward Force	Floor Hight
1	0	370	0.85	26.63	19.91 (+/-)	4.79	24.70	17.85	25.16	18.18	0
2	17	370	0.872	27.31	20.41 (+/-)	4.92	25.33	164.01	25.16	37.43	17
3	35	370	1.015	31.79	23.77 (+/-)	5.72	29.49	174.58	25.16	34.22	18
4	49	370	1.089	34.13	25.51 (+/-)	6.14	31.65	163.97	25.16	29.94	14
5	63	370	1.148	35.98	26.90 (+/-)	6.48	33.37	172.88	25.16	29.94	14
6	77	370	1.198	37.53	28.06 (+/-)	6.76	34.81	180.34	25.16	29.94	14
roof	91	370	1.241	38.88	29.06 (+/-)	7.00	36.06	21.46	25.16	14.97	14
							Σ	877.22		176.45	

L_{EFF}	Base Shear	Over Turning Moment
310	**1053.67 k**	**52517.79 k-ft**

B:	85.00
g_q:	3.40
gv:	3.40
c:	0.20
z(bar):	54.60
L_z:	552.98
b(bar):	0.65
ά:	0.15
Vz(bar):	123.61
l:	500.00
$^{\epsilon}$:	0.20
h:	91
L:	370
V:	120.00
$^{\alpha}$:	9.5
z_g:	900

G:	0.93
n_a:	0.75
I_z:	0.18
Q:	0.87
N_1:	3.36
R_n:	0.07
β:	0.02
g_R:	4.12
$R_{h,n}$:	2.54
$R_{L,n}$:	34.59
$R_{B,n}$:	2.37
R_h:	0.32
R_L:	0.03
R_B:	0.33
R:	0.43

Case 1, P	Case 2, Mt	Case 3, P	Case 4, Mt	Case 4, P
36	477	27	1893	20
201	483	151	1916	113
209	523	157	2073	118
194	543	145	2155	109
203	560	152	2220	114
210	574	158	2276	118
36	585	27	2323	21

Appendix 8: Seismic Calculations

Alex Burg | Final Thesis Design | Seismic Loads

$S_s = 0.31$ (usgs)
$S_1 = 0.064$ (usgs)

$F_a = 1.45$ (11.4-1)
$F_v = 2.4$ (11.4-2)

$S_{MS} = F_a S_s = 1.45(0.31) = \underline{0.45}$

$S_{M1} = F_v S_1 = 2.4(0.064) = \underline{0.15}$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.45) = \underline{0.3}$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.15) = \underline{0.1}$

Design Category = B => C-1

$I = 1.25$ Table 1.5-2

$R = 3$ Commerce (Non-Special)

$C_t = 0.02$ $x = 0.75$ Table 12.8-2

$h_n = 45'$

$T_a = C_t h_n^x = 0.02(45)^{.75} = 0.59$

$T_L = 6$ sec (Fig 22-2) $C_u = 1.7$ (table 12.8-1)

$T = C_u T_a = 1.7(0.59) = 1.003$

$T_L > T$ use $C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.1}{1.003\left(\frac{3}{1.25}\right)} = 0.042$

$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.3}{\left(\frac{3}{1.25}\right)} = 0.125$

$C_s = 0.042 > 0.01$ ∴ ✓
$C_s = 0.042 < 0.125$ ∴ ✓
so
$C_s = 0.039$

Base Shear

$V = C_s W$
$V = \frac{0.039(32.551E6)}{1,000}$
$\boxed{V = 1,269.5 \text{ Kip}}$

Assuming floors 2-7 are equal weight

$w = (6''/12'')150pcf + 20psf + 15psf = 172.5psf$

Area = 330' x 35' = 3,450 ft²

$W_{total} = 3,450(172.5)(6\ floors) = 32,551E6$

Refer To Seismic Analysis spreadsheet

Seismic Loading

Mass/Area: 6.56E-05 kip
Floor Wieght: 4,889.05 kips

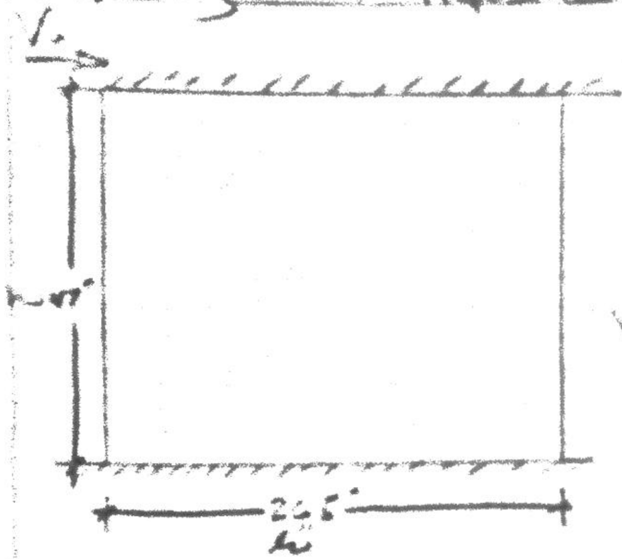
$T=$ 1.003 s
$k=$ 1.250
$C_S=$ 0.039
$V_b=$ 1,144.04 kips

Floor	h_i (ft.)	h (ft)	w (kips)	$w*h^k$	C_{VX}	f_i (kips)
Roof	14	91	29,334	8,244,749.59	0.300	344
6	14	77	29,334	6,690,970.82	0.244	279
5	14	63	29,334	5,206,566.46	0.190	217
4	14	49	29,334	3,802,951.87	0.139	158
3	18	35	29,334	2,497,242.89	0.091	104
2	17	17	29,334	1,012,597.57	0.037	42
		Σ	176,005.80	27,455,079.19		1,144.04

Overturning Moment: 78,524.09

Appendix 9: Shear Wall Design

Alex Burg — Proposed Shear Wall Design ~



$f'_c = 4$ ksi
$E_c = 57\sqrt{f'_c} = 3,605$ ksi
$d = 0.8\,l_w = 21.2'$

V_u = Envelope
$= 252^k$
$V_u = 252^k$

<u>Check Max. Permitted Shear Strength</u> $V_u < \phi V_n$

<u>Try Anchorage</u>

$\phi V_n = \phi 10\sqrt{f'_c}\,h\,d$
$= 0.75(10)\sqrt{4000}(12)(21.2 \times 12')/1000 = \underline{1448}^k > 252^k$ ✓

<u>Try Anchorage = 8"</u>

$\phi V_n = \underline{965}^k$ ∴ ✓

<u>Check Shear Strength provided by V_c</u>

Critical Section at: $l_w/2 = 26.5/2 = 14.75'$
$h_w/2 = 17/2 = \underline{8.5'}$ ← governs

<u>$t = 12"$</u> $V_c = 2\sqrt{f'_c}\,h\,d$
$= 2\sqrt{4000}(12)(21.2 \times 12")/1000 = \underline{386}^k$

<u>$t = 8"$</u> $V_c = \underline{257}^k$

Alex Burg | Proposed Solution | Shear Wall Design 3/4

Required Steel:

If $V_u > \frac{1}{2}\phi V_c$ According to ACI 11.9.9 use Horizontal Reinforcement

t=12"

$\frac{1}{2}(.75)(386) = \underline{145^k}$ ∴ Needs Horizontal Reinforcemen

t=8"

$\frac{1}{2}(.75)(257) = \underline{96}^k$ ∴ Needs Horizontal Reinforcement

$\underline{V_u \le \phi V_n = \phi(V_c + V_s)}$

t=12" $252 = (386 + V_s)0.75 \Rightarrow V_s = -50^k$ ∴

t=8" $V_s = 79^k$

t=12" $V_s = \frac{A_v \cdot f_y \cdot d}{s} \Rightarrow \frac{A_v}{s} = \frac{V_s}{f_y d} = \frac{-50}{60(2)(.2 \times 12)} = 0.00327$

t=8" $V_s = 0.005$

t=8"

try (2)#3 $s = \frac{2(0.11)}{0.005} = 44"$

try (2)#3 @ 36"

$\rho_t = \frac{A_v}{s(t)} = \frac{2(0.11)}{36(8)} = 0.000764 < 0.0025$ ∴ NG

try (2)#3 @ 12"

$\rho_t = \frac{2(0.11)}{12(8)} = 0.0022 < 0.0025$ ∴ NG

try (2) #(3) @ 10"

$\rho_t = \frac{2(0.11)}{10(8)} = 0.00275 < 0.0025$ ∴ OK

Alex Burg | Proposed Solution | Shear Wall Design 3

Max Spacing:

s_2 | $l_w/5 = (26.5)(12)/5 = 63.6''$
$3t = 3(8) = 24''$
min $18''$ ← governs.

$10'' < 18''$ ✓

$\boxed{\text{Use (2) \#3 @ 10'' for Horizontal Shear}}$

Vertical Shear Rienforcement

$$\rho_l = \frac{A_v}{s \cdot h} \geq 0.0025 + 0.5(2.5 - h_w/l_w)(\rho_h - 0.0025)$$

$$= 0.0025 + 0.5(2.5 - 17/26.5)(0.00275 - 0.0025)$$

$$= 0.0027 > 0.0025 \quad \therefore \text{ OK}$$

Max spacing: | $l_w/3 = 26.5(12)/3 = 106''$
$3(t) = 24''$
min $18''$ ← governs

$$\rho_l = \frac{A_v}{s h} = 0.00273 \Rightarrow s = \frac{A_v}{0.00273(t)}$$

$$\text{try (2) \#3} = s = \frac{2(.11)}{0.00273(8)} = 10.2''$$

$\boxed{\text{Use (2) \#3 @ 12'' for Vertical Shear}}$

Design for Flexure:

$M_u = V_u(h_w) = 252^k(17') = 4{,}284^{k-'}$

* Assume Tension Controlled *

$M_n = A_s f_y (d - a/2) = A_s f_y jd$

$C = T \Rightarrow 0.85 f'_c a b = A_s f_y$ $\qquad jd = .9d = (.9)(21.2)(12) = 229''$

$M_u \leq \phi M_n = \phi A_s f_y jd$

$4{,}284(12000) \leq .9(A_s)(60000)(229)$

$A_s \geq 4.16 \text{ in}^2$

$.85(4000)(a)(8) = (4.16)(60000) \Rightarrow a = 9.18''$

Alex Burg | Proposed Solution | Shear Wall Design 4/4

$jd = d - a/2$

$= (21.2 \times 12) - 9.18/2 = 249''$

$4{,}284(12000) = 0.9\, A_s\, (60{,}000)\, 249$

$A_s = 3.8\ in^2$

Try (4) #9 $\quad A_s = 4\ in^2$

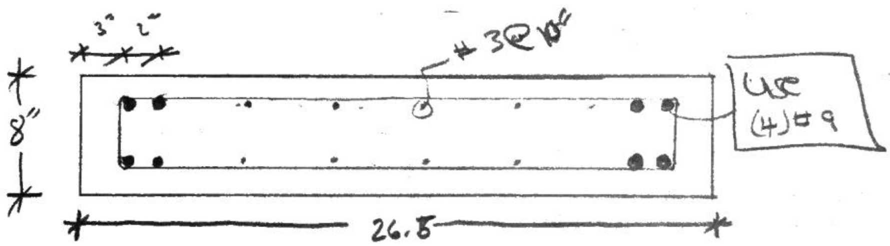
$C = T; \quad 0.85 f'_c a b = A_s f_y$

$a = \dfrac{4(60)}{.85(4)(8)} = 8.8$

$c = a/\beta_1 = \dfrac{8.8}{.85} = 10.4''$

$\varepsilon_t = \varepsilon_u \left(\dfrac{d-c}{c}\right) = 0.003 \left(\dfrac{(21.2 \times 12) - 10.4}{10.4}\right)$

$= 0.07 > 0.005 \therefore$ OK ✓ Tension Controlled



Appendix 10: Moment Frame Design

	Floor:	1	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	13		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	10.27	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	Yes		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_r=Max of	0.4				
	Spaced Right:	0	feet					$.25+15/\sqrt{(K_{LL}*A_T)}=$		0.63	L_L:	50.35
Spacing,S	Max of d_b, 1", 3/4A_b			b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S								
d_b:	1.0	inch			b_{min}:	15.71	inch	Total Factored Weight, Wu				
3/4A_b:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch	Wu=1.2D +1.0W+L				Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	30	inch	OK		d:	27		a:	2.05	Rectangular Section	c:	2.41
b_{eff}= min	b*16*h_f	1872			ε_t=ε_u(d-c)/c							
	Trib width	318			ε_t:	0.03	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	1182	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	973	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	1200	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		**18**	**x**	**30**								
	Beam Wieght:	441	plf									

	Floor:	2	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	9		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	7.11	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	Yes		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_r=Max of	0.4				
	Spaced Right:	0	feet					.25+15/√(K_{LL}*A_T)=		0.63	L_r:	50.35
Spacing,S	Max of d_b, 1", 3/4A_b			b_{min} =2*Cc + n*d_b + 2d_{st} + (n-1)*S								
d_b:	1.0	inch		b_{min}:	11.71	inch	Total Factored Weight, Wu					
3/4A_b:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch		Wu=1.2D +1.0W+L			Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	30	inch	OK		d:	27		a:	1.42	Rectangular Section	c:	1.67
b_{eff}= min	b*16*h_f	1872			ε_t=ε_u(d-c)/c							
	Trib width	318			ε_t:	0.05	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	833	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	624	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	841	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		**18**	**x**	**30**								
	Beam Wieght:	441	plf									

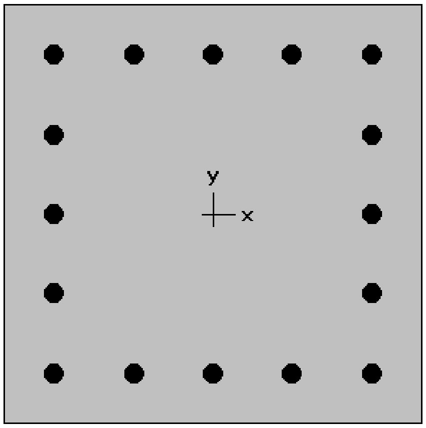
	Floor:	3	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
Misc. Dead Load:		35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	7		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	5.53	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	Yes		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_r=Max of	0.4				
	Spaced Right:	0	feet					$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.63	L_r:	50.35
Spacing,S	Max of d_b, 1", $3/4A_b$			$b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S$								
d_b:	1.0	inch		b_{min}:	9.71	inch		Total Factored Weight, Wu				
$3/4A_b$:	0.6	inch						Dead Load: Misc. dead + Slab			Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch		Wu=1.2D +1.0W+L			Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		$d=h-d_b/2-Cc$			$a=A_s*fy/(.85*f'c*b_{eff})$			$c=a/\beta_1$	
h:	30	inch	OK		d:	27		a:	1.10	Rectangular Section	c:	1.30
b_{eff}= min	$b*16*h_f$	1872			$\varepsilon_t=\varepsilon_u(d-c)/c$							
	Trib width	318			ε_t:	0.06	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	616	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	407	kip-ft
$\Phi Mn=0.9*A_s fy(d-a/2)$					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	658	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		**18**	**x**	**30**								
	Beam Wieght:	441	plf									

	Floor:	4	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	5		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	3.95	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	No		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_r=Max of	0.4				
	Spaced Right:	0	feet					$.25+15/\sqrt{(K_{LL}*A_T)}=$		0.63	L_L:	50.35
Spacing,S	Max of d_b, 1", 3/4A_b			$b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S$								
d_b:	1.0	inch		b_{min}:	12.71	inch	Total Factored Weight, Wu					
3/4A_b:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch	Wu=1.2D +1.0W+L				Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	30	inch	OK		d:	28		a:	0.79	Rectangular Section	c:	0.93
b_{eff}= min	b*16*h_f	1872			$\varepsilon_t=\varepsilon_u$(d-c)/c							
	Trib width	318			ε_t:	0.09	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu			Column width:	26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	465	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	256	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	491	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		18	x	30								
	Beam Wieght:	441	plf									

	Floor:	5	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	4		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	3.16	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	No		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_r=Max of	0.4				
	Spaced Right:	0	feet					.25+15/√(K_{LL}*A_T)=		0.63	L_r:	50.35
Spacing,S	Max of d_b, 1", 3/4A_b			b_{min} =2*Cc + n*d_b + 2d_{st} + (n-1)*S								
d_b:	1.0	inch		b_{min}:	10.71	inch	Total Factored Weight, Wu					
3/4A_b:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch	Wu=1.2D +1.0W+L				Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	30	inch	OK		d:	28		a:	0.63	Rectangular Section	c:	0.74
b_{eff}= min	b*16*h_f	1872			ε_t=ε_u(d-c)/c							
	Trib width	318			ε_t:	0.11	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	357	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	148	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	394	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		**18**	**x**	**30**								
Beam Wieght:		441	plf									

	Floor:	6	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
	Misc. Dead Load:	35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	3		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	2.37	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	No		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_r=Max of	0.4				
	Spaced Right:	0	feet					$.25+15/\sqrt{(K_{LL}*A_T)}$=		0.63	L_r:	50.35
Spacing,S	Max of d_b, 1", $3/4A_b$			$b_{min} =2*Cc + n*d_b + 2d_{st} + (n-1)*S$								
d_b:	1.0	inch			b_{min}:	8.71	inch	Total Factored Weight, Wu				
$3/4A_b$:	0.6	inch						Dead Load: Misc. dead + Slab			Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
					h_{min}:	19.14	inch	Wu=1.2D +1.0W+L			Pu=1.2D	
S:	1.0	inch							190	psf	2236	plf
Try a												
b:	18	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	30	inch	OK		d:	28		a:	0.47	Rectangular Section	c:	0.56
b_{eff}= min	b*16*h_f	1872			$\varepsilon_t=\varepsilon_u$(d-c)/c							
	Trib width	318			ε_t:	0.15	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu			Column width:	26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	279	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	70	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	296	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		18	x	30								
	Beam Wieght:	441	plf									

	Floor:	7	/	7								
	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch			
Misc. Dead Load:		35	psf	Bar Size:	#	8	Bar Diameter, d_b:	1.000	inch			
	Live Load:	80	psf	# of bars, n:	3		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	2.37	in^3			
Girder Design, G2		(Lateral)					Tributary Area:	391	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	No		Influence Area:	1564	ft^2	Use Live Load Reduction		
	Spaced Left:	26.5	feet				L_r=Max of	0.4				
	Spaced Right:	0	feet					.25+15/√(K_{LL}*A_T)=		0.63	L_r:	50.35
Spacing,S	Max of d_b, 1", 3/4A_b			b_{min} =2*Cc + n*d_b + 2d_{st} + (n-1)*S								
d_b:	1.0	inch		b_{min}:	8.71	inch	Total Factored Weight, Wu					
3/4A_b:	0.6	inch					Dead Load: Misc. dead + Slab				Beam Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116	psf	141	plf
				h_{min}:	19.14	inch	Wu=1.2D +1.0W+L				Pu=1.2D	
S:	1.0	inch						190	psf		2236	plf
Try a												
b:	18	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	30	inch	OK		d:	28		a:	0.47	Rectangula r Section	c:	0.56
b_{eff}= min	b*16*h_f	1872			ε_t=ε_u(d-c)/c							
	Trib width	318			ε_t:	0.15	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Column width:		26	inch		
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load					Vu=Wu*Ln/2+Pu/2							
Mu:	253	kip-ft		Combo Controls	Vu:	75	Kip			Etabs Moment	44	kip-ft
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	296	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		18	x	30								
	Beam Wieght:	441	plf									
					Total Reinforcment Wieght in Lateral Girders:				309.7	Ton		

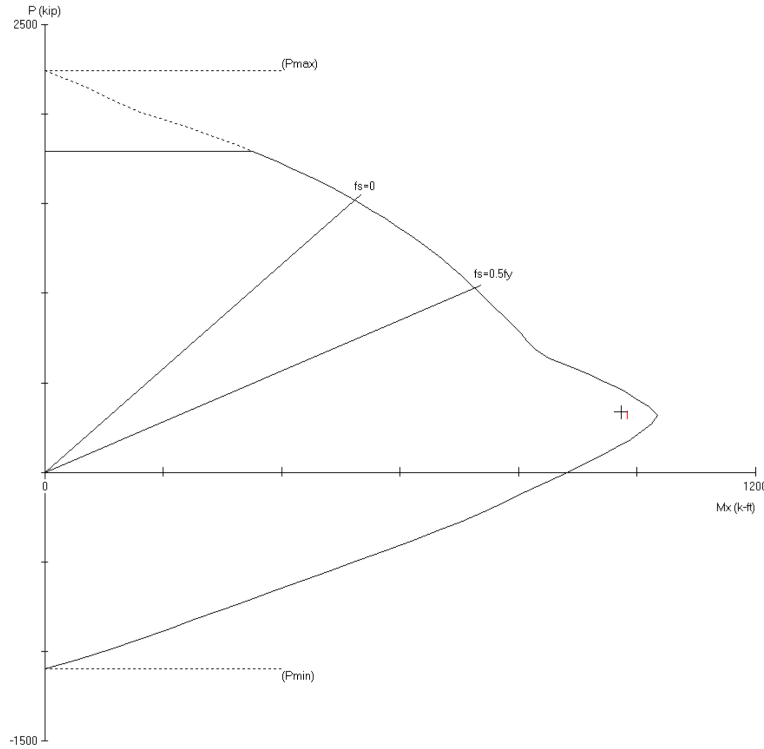


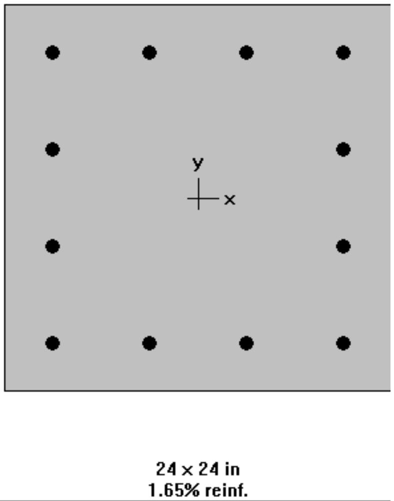
26 x 26 in
3.01% reinf.

MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 676 in^2
Ix = 38081.3 in^4
Iy = 38081.3 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
16 #10 bars @ 3.006%
As = 20.32 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 3.66 in

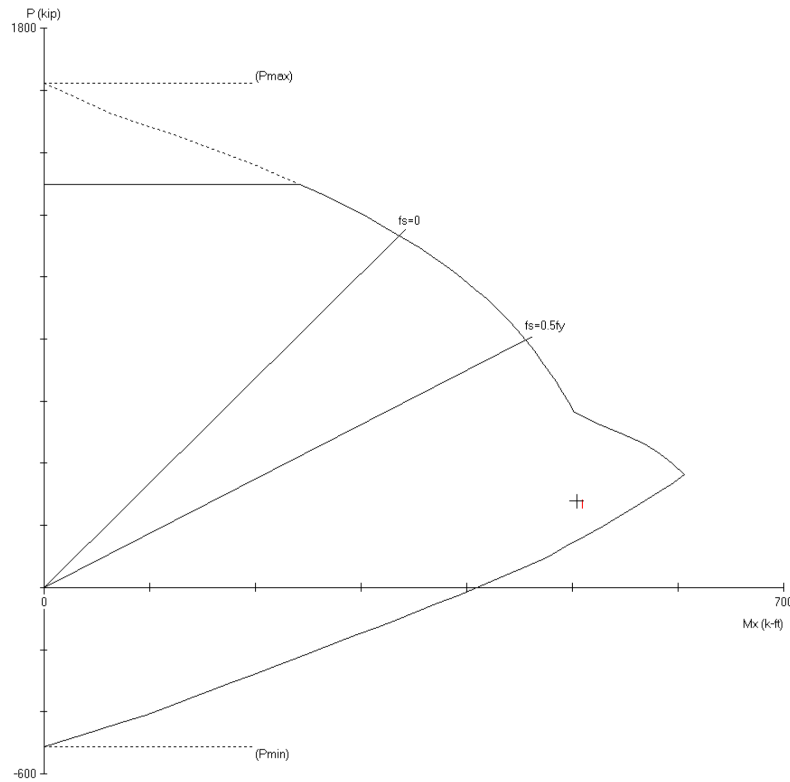


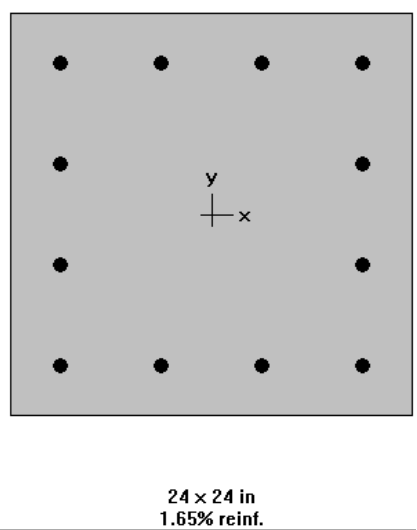


MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 576 in^2
Ix = 27648 in^4
Iy = 27648 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
12 #8 bars @ 1.646%
As = 9.48 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 5.00 in

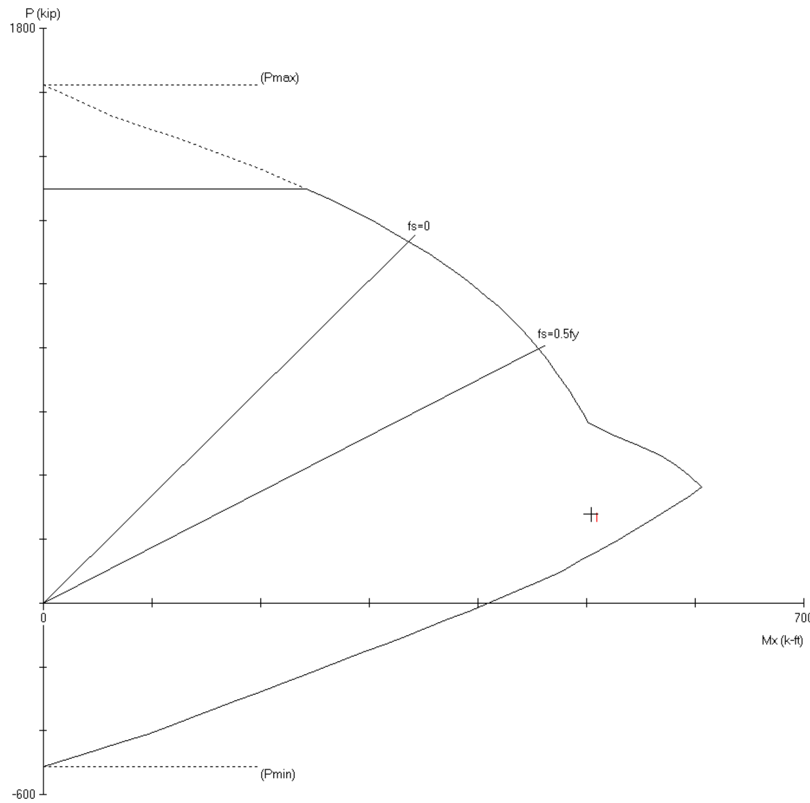


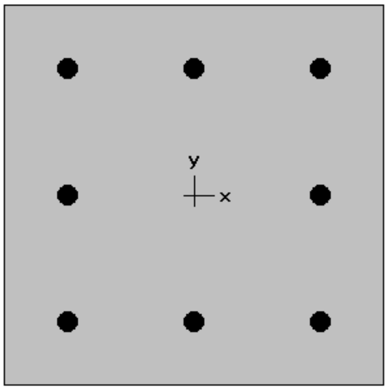


MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 576 in^2
Ix = 27648 in^4
Iy = 27648 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
12 #8 bars @ 1.646%
As = 9.48 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 5.00 in



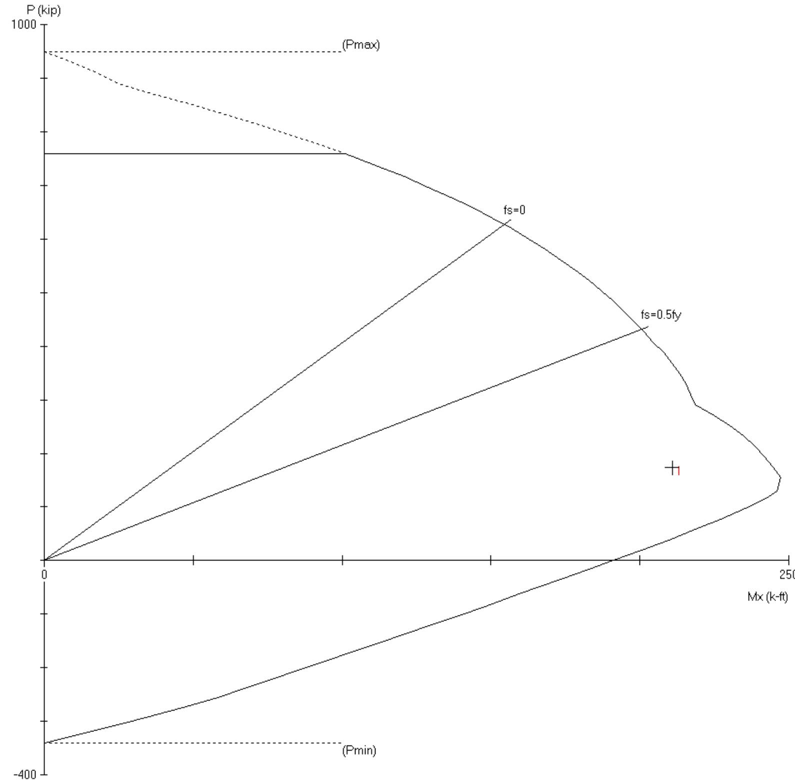


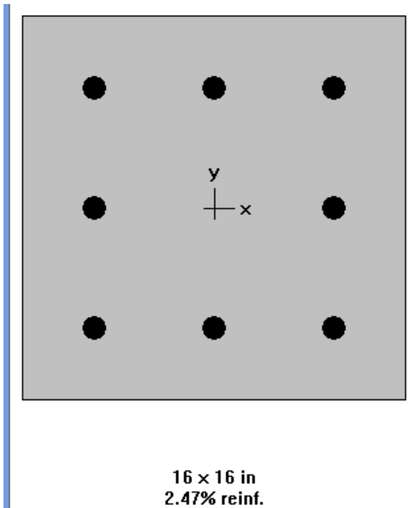
18 x 18 in
1.95% reinf.

MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 324 in^2
Ix = 8748 in^4
Iy = 8748 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
8 #8 bars @ 1.951%
As = 6.32 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 5.00 in

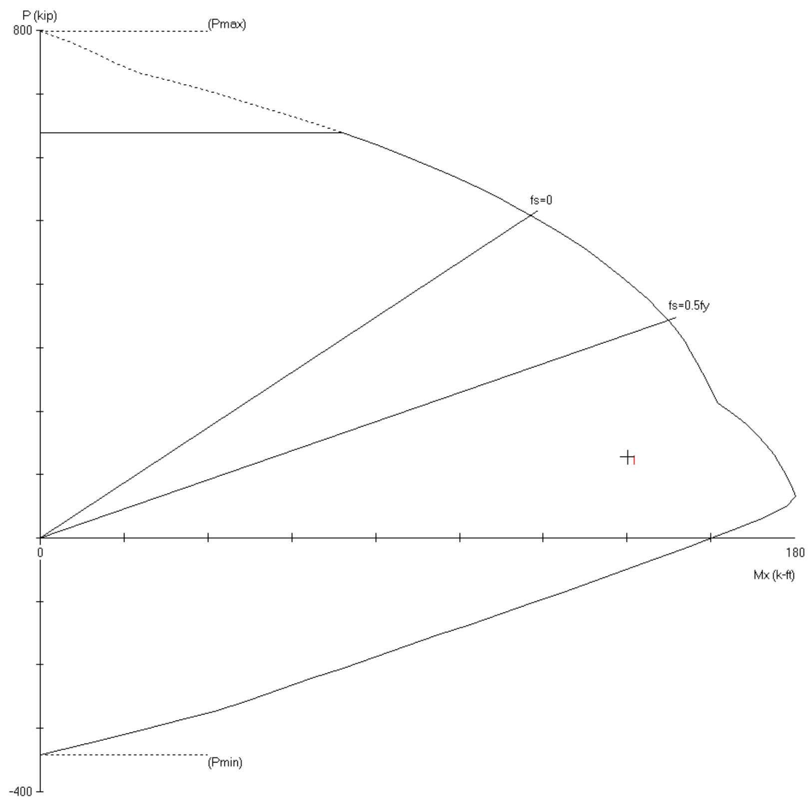


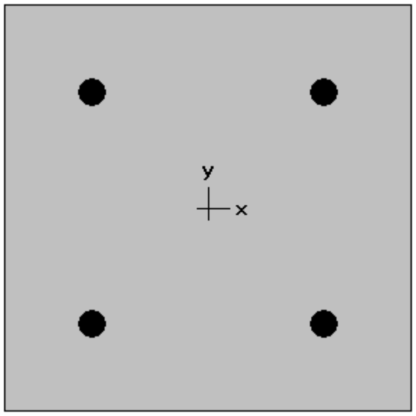


MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 256 in^2
Ix = 5461.33 in^4
Iy = 5461.33 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
===============
8 #8 bars @ 2.469%
As = 6.32 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 4.00 in



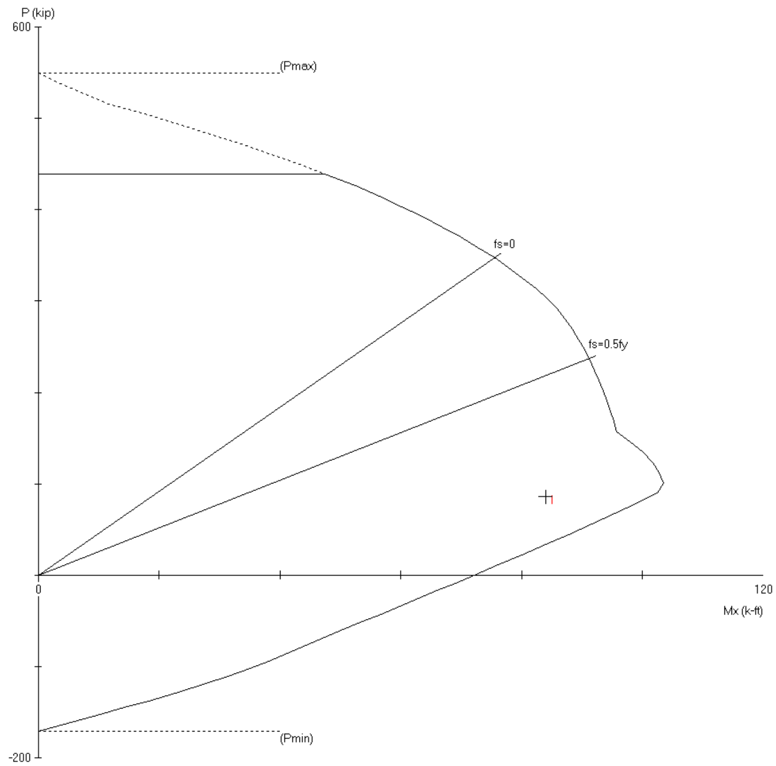


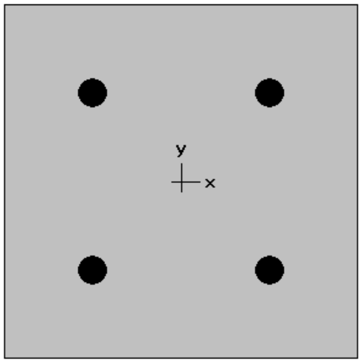
14 x 14 in
1.61% reinf.

MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 196 in^2
Ix = 3201.33 in^4
Iy = 3201.33 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
==============
4 #8 bars @ 1.612%
As = 3.16 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 7.00 in



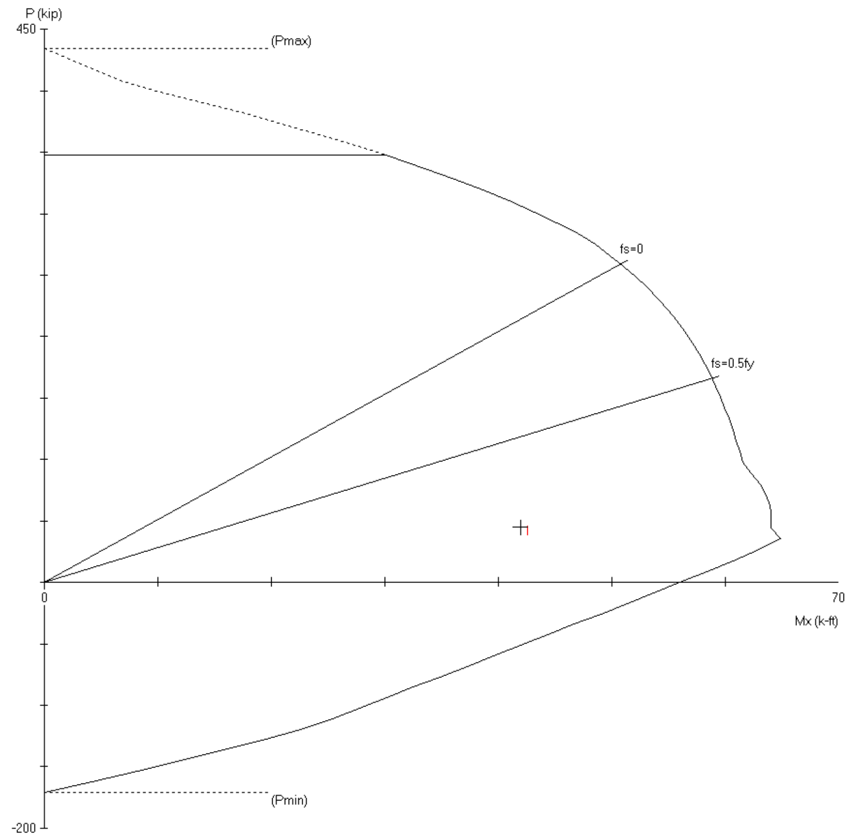


12 x 12 in
2.19% reinf.

MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 144 in^2
Ix = 1728 in^4
Iy = 1728 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
===============
4 #8 bars @ 2.194%
As = 3.16 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 5.00 in



Appendix 11: Cost Analysis

Existing Steel Structure

	Project Number/Item		Size	Unit	Material	Labor	Equipment	Total	Total Incl. O&P	Location	Amount	Total(No O&P)	Total (w/ O&P)
Steel Decking	05 31 13.50	5900	18 Gauge	S.F.	$ 1.80	$ 0.40	$ 0.05	$ 2.25	$ 2.80	All Stories	306,894.00	$ 690,511.50	$ 859,303.20
Deck FireProofing	07 81 16.10	100	1" thick	S.F.	$ 0.53	$ 0.22	$ 0.04	$ 0.79	$ 0.99	All Stories	306,894.00	$ 242,446.26	$ 303,825.06
3" Slab Pumped	03 31 15.70	1400	pumped	C.Y.	-	$ 12.50	$ 5.70	$ 18.20	$ 27.50	All Stories	2,838.77	$ 51,665.60	$ 78,066.16
4000psi Concrete	03 31 05.35	300	3" Slab	C.Y.	$ 103.00	-	-	$ 103.00	$ 113.00	All Stories	2,838.77	$ 292,393.26	$ 320,780.95
Concrete Finish	03 35 29.30	125	Bull Float	S.F.	-	$ 0.35	-	$ 0.35	$ 0.57	All Stories	306,894.00	$ 107,412.90	$ 174,929.58
Curb Edging	05 12 23.12	1501	12" Channel	L.F.	$ 28.00	$ 7.40	-	$ 35.40	$ 43.50	All Stories	1,377.00	$ 48,745.80	$ 59,899.50
Steel Beam	05 12 23.75	1300	W12x19	L.F.	$ 22.89	$ 1.93	$ 1.83	$ 26.65	$ 30.64	All Stories	7,560.00	$ 201,446.51	$ 231,663.49
Beam FireProofing	07 81 16.10	400	1" thick	S.F.	$ 0.53	$ 0.43	$ 0.09	$ 1.05	$ 1.39	All Stories	30,240.00	$ 31,752.00	$ 42,033.60
Steel Beam	05 12 23.75	1300	W16x26	L.F.	$ 31.50	$ 1.70	$ 1.61	$ 34.81	$ 39.50	All Stories	22,292.50	$ 776,001.93	$ 880,553.75
Beam FireProofing	07 81 16.10	400	1" thick	S.F.	$ 0.53	$ 0.43	$ 0.09	$ 1.05	$ 1.39	All Stories	89,170.00	$ 93,628.50	$ 123,946.30
Steel Girder	05 12 23.75	4900	W24x55	L.F.	$ 66.50	$ 2.29	$ 1.58	$ 70.37	$ 79.00	All Stories	14,840.00	$ 1,044,290.80	$ 1,172,360.00
Girder FireProofing	07 81 16.10	400	1" thick	S.F.	$ 0.53	$ 0.43	$ 0.09	$ 1.05	$ 1.39	All Stories	118,720.00	$ 124,656.00	$ 165,020.80
Steel Column	05 12 23.17	7350	W14x99	L.F.	$ 89.50	$ 1.72	$ 1.63	$ 92.85	$ 104.00	Top 2 Stories	3,744.00	$ 347,630.40	$ 389,376.00
Column FireProofing	07 81 16.10	800	1" thick	S.F.	$ 1.13	$ 0.93	$ 0.19	$ 2.25	$ 2.98	All Stories	17,472.00	$ 39,312.00	$ 52,066.56
Steel Column	05 12 23.17	7400	W14x120	L.F.	$ 145.00	$ 1.77	$ 1.67	$ 148.44	$ 165.00	Mid 2 Stories	3,744.00	$ 555,759.36	$ 617,760.00
Column FireProofing	07 81 16.10	800	1" thick	S.F.	$ 1.13	$ 0.93	$ 0.19	$ 2.25	$ 2.98	All Stories	17,472.00	$ 39,312.00	$ 52,066.56
Steel Column	05 12 23.17	7450	W14x176	L.F.	$ 213.00	$ 1.86	$ 1.76	$ 216.62	$ 239.00	Bot 2 Stories	3,430.00	$ 743,006.60	$ 819,770.00
Column FireProofing	07 81 16.10	800	1" thick	S.F.	$ 1.13	$ 0.93	$ 0.19	$ 2.25	$ 2.98	All Stories	16,006.67	$ 36,015.00	$ 47,699.87
												$ 5,972,968.56	**$ 7,030,233.51**

Location Factor:	1.1
Floor Area	
43842	ft^2
Building Area	
306894	ft^2
Concrete Volume	
2,838.77	C.Y.

Proposed Concrete Structure

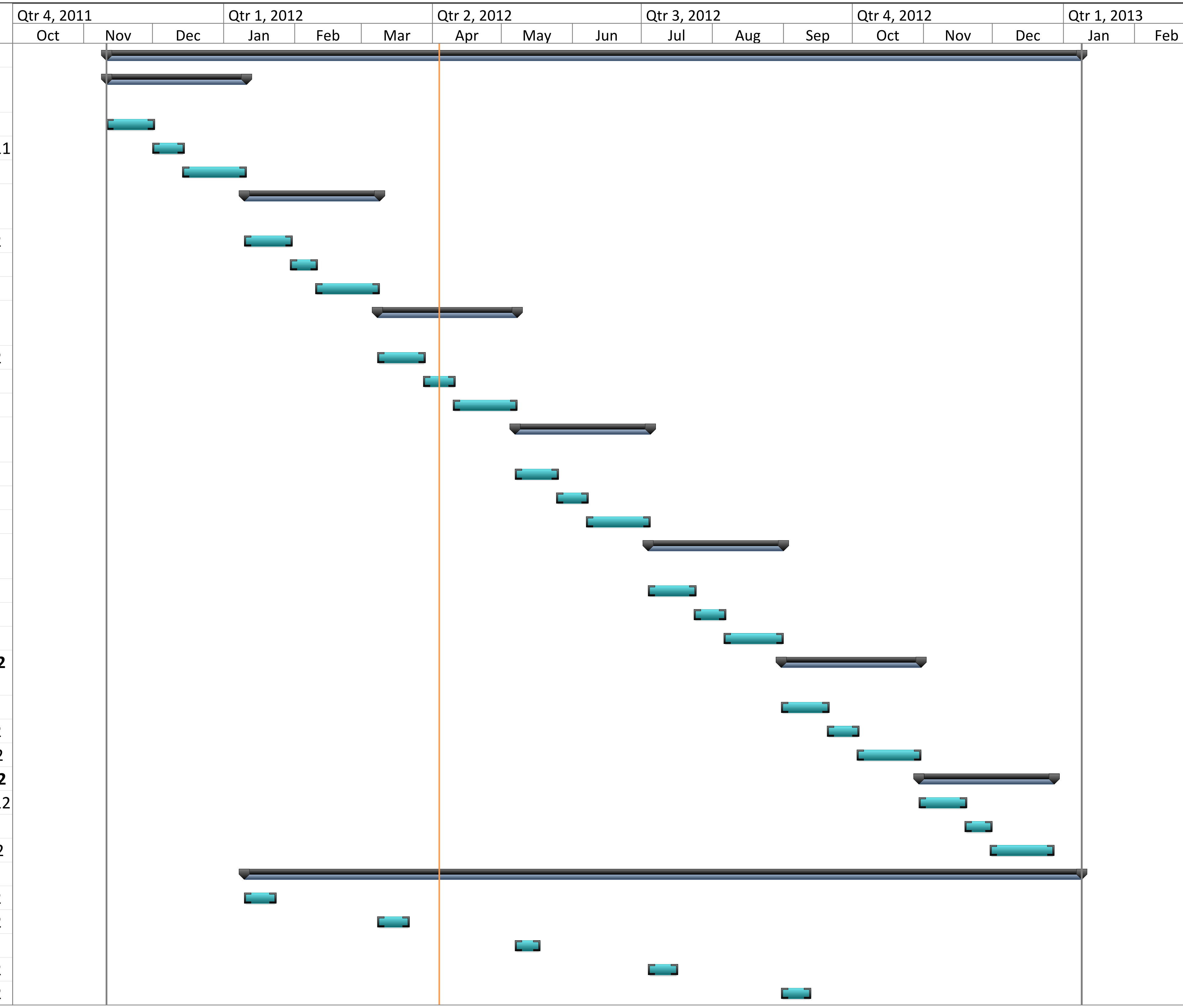
	Project Number/Item		Size	Unit	Material	Labor	Equipment	Total	Total Incl. O&P	Location	Amount	Total(No O&P)	Total (w/ O&P)
4000psi Concrete	03 31 05.35	300		C.Y.	$ 103.00	-	-	$ 103.00	$ 113.00	All Stories	16,740.67	$ 1,724,288.73	$ 1,891,695.40
Concrete Finish	03 35 29.30	125	Bull Float	S.F.	-	$ 0.35	-	$ 0.35	$ 0.57	All Stories	43,842.00	$ 15,344.70	$ 24,989.94
Concrete Slab	03 31 05.70	1400	6.5" Slab	C.Y.	-	$ 10.95	$ 5.00	$ 15.95	$ 23.50	All Stories	16,740.67	$ 267,013.64	$ 393,405.68
Slab Reinforcing	03 21 10.60	400		Ton	$ 850.00	$ 385.00	-	$ 1,235.00	$ 1,625.00	All Stories	445.00	$ 549,575.00	$ 723,125.00
Slab Form	03 11 13.35	1150	4 use	SFCA	$ 1.32	$ 2.48	-	$ 3.80	$ 5.60	All Stories	306,894.00	$ 1,166,197.20	$ 1,718,606.40
Edge Form	03 11 13.35	7000	4 use	L.F.	$ 0.12	$ 1.84	-	$ 1.96	$ 3.22	All Stories	9,639.00	$ 18,892.44	$ 31,037.58
Concrete Beam	03 31 05.70	200	10x20	C.Y.	-	$ 19.45	$ 8.90	$ 28.35	$ 42.50	All Stories	4,142.00	$ 117,425.70	$ 176,035.00
Beam Reinforcing	03 21 10.60	150		Ton	$ 800.00	$ 415.00	-	$ 1,215.00	$ 1,600.00	All Stories	132.00	$ 160,380.00	$ 211,200.00
Beam Form	03 11 13.20	1150	4 use	SFCA	$ 1.09	$ 3.08	-	$ 4.17	$ 6.38	All Stories	16,567.00	$ 69,029.17	$ 105,614.63
Concrete Girder	03 31 05.70	200	10x20	C.Y.	-	$ 19.45	$ 8.90	$ 28.35	$ 42.50	All Stories	1,639.00	$ 46,465.65	$ 69,657.50
G1 Reinforcing	03 21 10.60	150		Ton	$ 800.00	$ 415.00	-	$ 1,215.00	$ 1,600.00	All Stories	77.00	$ 93,555.00	$ 123,200.00
G1 Form	03 11 13.20	1150	4 use	SFCA	$ 1.09	$ 3.08	-	$ 4.17	$ 6.38	All Stories	7,204.00	$ 30,016.67	$ 45,925.50
Concrete Girder	03 31 05.70	200	18x30	C.Y.	-	$ 19.45	$ 8.90	$ 28.35	$ 42.50	All Stories	4,425.00	$ 125,448.75	$ 188,062.50
G2 Reinforcing	03 21 10.60	150		Ton	$ 800.00	$ 415.00	-	$ 1,215.00	$ 1,600.00	All Stories	310.00	$ 376,650.00	$ 496,000.00
G2 Form	03 11 13.20	1150	4 use	SFCA	$ 0.91	$ 4.41	-	$ 5.32	$ 8.40	All Stories	12,968.00	$ 68,989.76	$ 108,931.20
Concrete Column	03 31 05.70	800	20x20	C.Y.	-	$ 19.05	$ 8.70	$ 27.75	$ 41.00	Top 2 Stories	384.00	$ 10,656.00	$ 15,744.00
C1 Reinforcing	03 21 10.60	250		Ton	$ 1,175.00	$ 510.00	-	$ 1,685.00	$ 2,175.00	All Stories	94.00	$ 158,390.00	$ 204,450.00
C1 Form	03 11 13.25	6650	4 use	SFCA	$ 0.62	$ 3.22	-	$ 3.83	$ 6.08	All Stories	13,347.00	$ 51,163.50	$ 81,194.25
Concrete Column	03 31 05.70	800	24x24-14x14	C.Y.	-	$ 15.88	$ 14.50	$ 30.38	$ 34.17	Top 2 Stories	425.00	$ 12,909.38	$ 14,520.83
C2 Reinforcing	03 21 10.60	400		Ton	$ 850.00	$ 385.00	-	$ 1,235.00	$ 1,625.00	All Stories	85.00	$ 104,975.00	$ 138,125.00
C2 Form	03 11 13.25	6650	4 use	SFCA	$ 0.74	$ 3.86	-	$ 4.60	$ 7.30	All Stories	16,016.00	$ 73,673.60	$ 116,916.80
Concrete Shear Wall	03 31 05.70	800	8" Thick	C.Y.	-	$ 14.25	$ 0.68	$ 14.93	$ 24.50	All Stories	357.00	$ 5,330.01	$ 8,746.50
Wall Reinforcing	03 21 10.60	400		Ton	$ 760.00	$ 281.00	-	$ 1,041.00	$ 1,325.00	All Stories	94.00	$ 97,854.00	$ 124,550.00
Wall Form	03 11 13.85	2550	4 use	SFCA	$ 0.59	$ 3.52		$ 4.11	$ 6.55	All Stories	14,469.00	$ 59,467.59	$ 94,771.95
												$ 5,878,646.28	**$ 7,817,156.22**

Location Factor:	1.1
Floor Area	
43842	ft^2
Building Area	
306894	ft^2
Concrete Volume	
16,740.67	C.Y.

Difference in Cost	$ (94,322.28)	$ 786,922.71

Appendix 12: Schedule Analysis

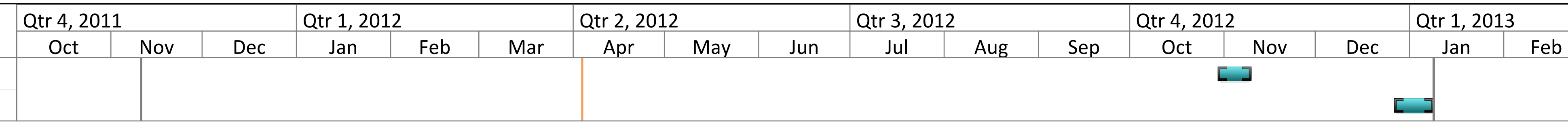
ID		Task Mode	Task Name	Duration	Start	Finish
1			**Steel Superstructure**	**303 days**	**Fri 11/11/11**	**Tue 1/8/13**
2			**1st Floor Steel Structure**	**43 days**	**Fri 11/11/11**	**Tue 1/10/12**
3			Set Steel	15 days	Fri 11/11/11	Thu 12/1/11
4			Detail Steel	10 days	Thu 12/1/11	Wed 12/14/11
5			Install Decking	20 days	Wed 12/14/11	Tue 1/10/12
6			**2nd Floor Steel Structure**	**43 days**	**Tue 1/10/12**	**Thu 3/8/12**
7			Set Steel	15 days	Tue 1/10/12	Mon 1/30/12
8			Detail Steel	10 days	Mon 1/30/12	Fri 2/10/12
9			Install Decking	20 days	Fri 2/10/12	Thu 3/8/12
10			**3rd Floor Steel Structure**	**43 days**	**Thu 3/8/12**	**Mon 5/7/12**
11			Set Steel	15 days	Thu 3/8/12	Wed 3/28/12
12			Detail Steel	10 days	Wed 3/28/12	Tue 4/10/12
13			Install Decking	20 days	Tue 4/10/12	Mon 5/7/12
14			**4th Floor Steel Structure**	**43 days**	**Mon 5/7/12**	**Wed 7/4/12**
15			Set Steel	15 days	Mon 5/7/12	Fri 5/25/12
16			Detail Steel	10 days	Fri 5/25/12	Thu 6/7/12
17			Install Decking	20 days	Thu 6/7/12	Wed 7/4/12
18			**5th Floor Steel Structure**	**43 days**	**Wed 7/4/12**	**Fri 8/31/12**
19			Set Steel	15 days	Wed 7/4/12	Tue 7/24/12
20			Detail Steel	10 days	Tue 7/24/12	Mon 8/6/12
21			Install Decking	20 days	Mon 8/6/12	Fri 8/31/12
22			**6th Floor Steel Structure**	**43 days**	**Fri 8/31/12**	**Tue 10/30/12**
23			Set Steel	15 days	Fri 8/31/12	Thu 9/20/12
24			Detail Steel	10 days	Thu 9/20/12	Wed 10/3/12
25			Install Decking	20 days	Wed 10/3/12	Tue 10/30/12
26			**Roof Steel Structure**	**43 days**	**Tue 10/30/12**	**Thu 12/27/12**
27			Set Steel	15 days	Tue 10/30/12	Mon 11/19/12
28			Detail Steel	10 days	Mon 11/19/12	Fri 11/30/12
29			Install Decking	20 days	Fri 11/30/12	Thu 12/27/12
30			**Concrete Pour**	**261 days**	**Tue 1/10/12**	**Tue 1/8/13**
31			Pour 1st Floor	10 days	Tue 1/10/12	Mon 1/23/12
32			Pour 2nd Floor	10 days	Thu 3/8/12	Wed 3/21/12
33			Pour 3rd Floor	9 days	Mon 5/7/12	Thu 5/17/12
34			Pour 4th Floor	9 days	Wed 7/4/12	Mon 7/16/12
35			Pour 5th Floor	9 days	Fri 8/31/12	Wed 9/12/12



Project: Existing Schedule
Date: Wed 4/4/12

Task	Project Summary	Inactive Milestone	Manual Summary Rollup
Split	External Tasks	Inactive Summary	Manual Summary
Milestone	External Milestone	Manual Task	Start-only
Summary	Inactive Task	Duration-only	Finish-only
Deadline	Progress		

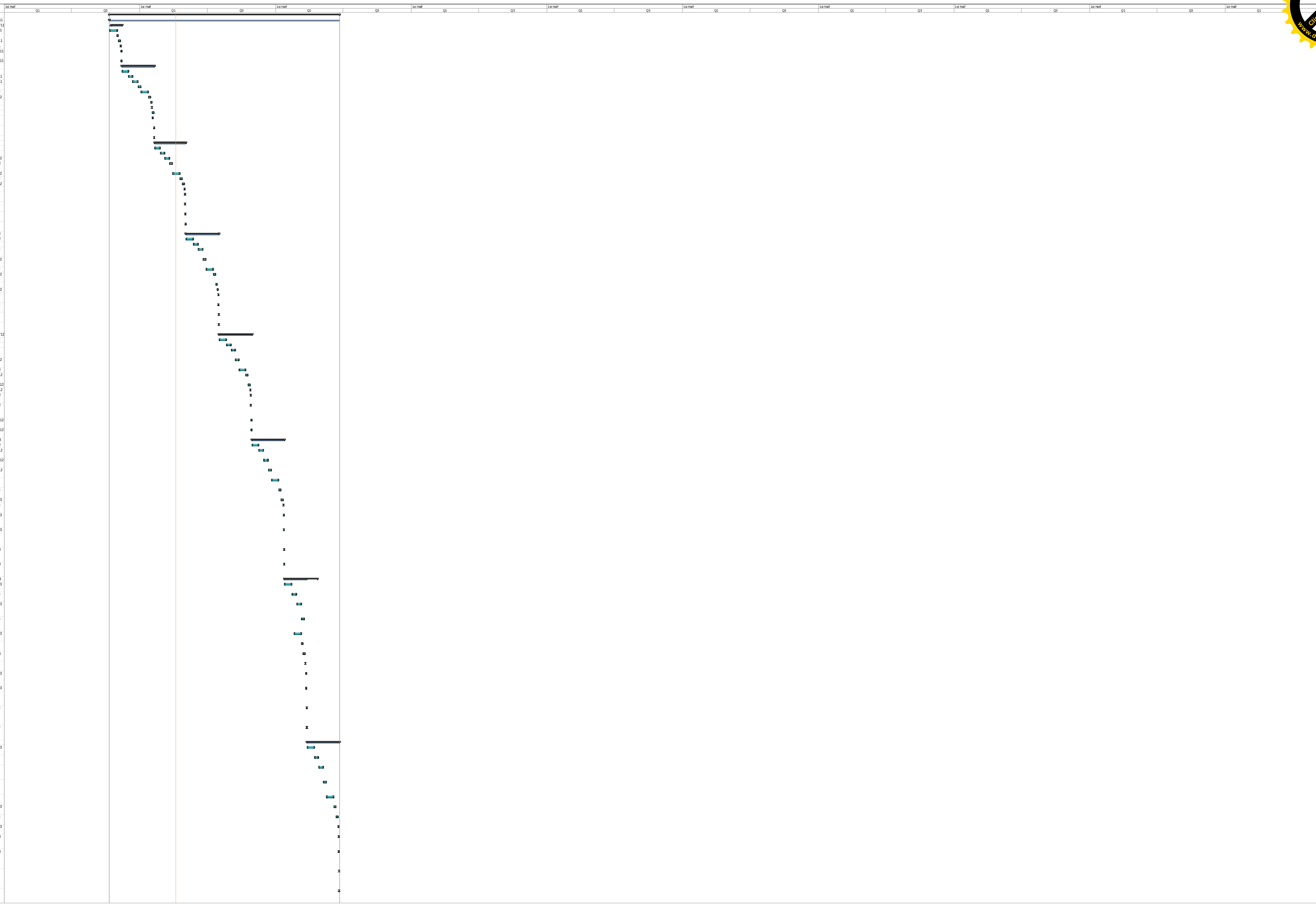
ID		Task Mode	Task Name	Duration	Start	Finish
36			Pour 6th Floor	9 days	Tue 10/30/12	Fri 11/9/12
37			Pour Roof	9 days	Thu 12/27/12	Tue 1/8/13



Project: Existing Schedule
Date: Wed 4/4/12

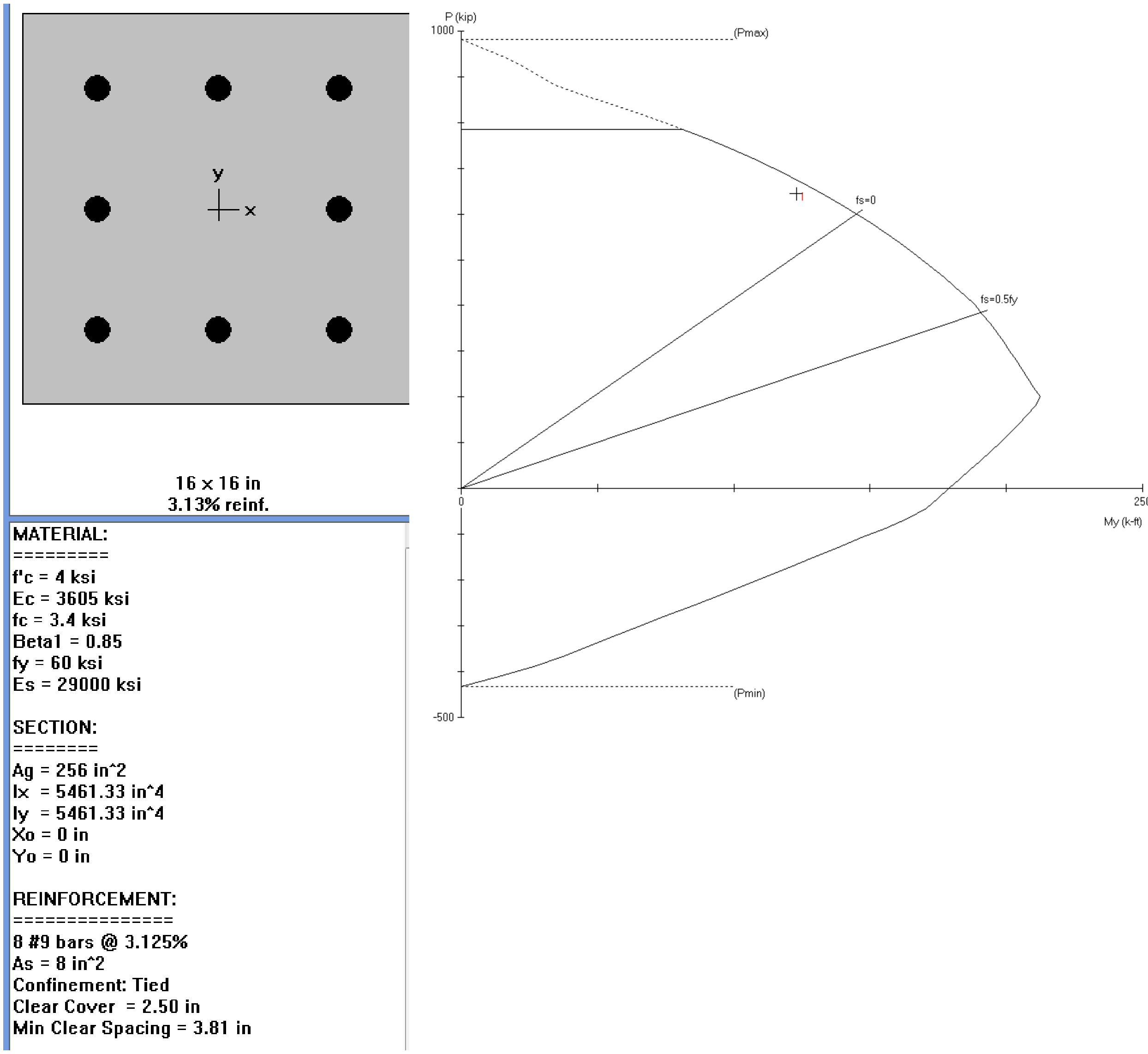
Task	Project Summary	Inactive Milestone	Manual Summary Rollup	Deadline
Split	External Tasks	Inactive Summary	Manual Summary	Progress
Milestone	External Milestone	Manual Task	Start-only	
Summary	Inactive Task	Duration-only	Finish-only	

	Task Mode	Task N[illegible]	Duration	Start	Finish
1		Con[illegible]erStructure	444 days	Tue 10/11/11	Fri 6/21/13
2		[illegible]uperStructure	1 day	Tue 10/11/11	Tue 10/11/11
3		[illegible]nd Floor	21 days	Mon 10/17/11	Mon 11/14/11
4		[illegible]me Columns	17 days	Tue 10/11/11	Wed 11/2/11
[illegible]		Reinforce Columns	5 days	Mon 10/31/11	Fri 11/4/11
[illegible]		Frame Walls	5 days	Fri 11/4/11	Thu 11/10/11
[illegible]		Reinforce Walls	2 days	Thu 11/10/11	Fri 11/11/11
[illegible]		Place Concrete in Columns	2 days	Fri 11/11/11	Mon 11/14/11
9		Place Concrete in Walls	2 days	Fri 11/11/11	Mon 11/14/11
10		First Floor	64 days	Mon 11/14/11	Thu 2/9/12
11		Frame Slab	15 days	Mon 11/14/11	Fri 12/2/11
12		Rienforce Slab	9 days	Thu 12/1/11	Tue 12/13/11
13		Frame Beam & Girder	12 days	Mon 12/12/11	Tue 12/27/11
14		Rienforce Beam & Girder	7 days	Tue 12/27/11	Wed 1/4/12
15		Frame Columns	15 days	Wed 1/4/12	Tue 1/24/12
16		Reinforce Columns	5 days	Tue 1/24/12	Mon 1/30/12
17		Frame Walls	5 days	Mon 1/30/12	Fri 2/3/12
18		Reinforce Walls	2 days	Thu 2/2/12	Fri 2/3/12
19		Place Concrete in Slab	4 days	Fri 2/3/12	Wed 2/8/12
20		Place Concrete in Beams & Girder	2 days	Fri 2/3/12	Mon 2/6/12
21		Place Concrete in Columns	2 days	Wed 2/8/12	Thu 2/9/12
22		Place Concrete in Walls	2 days	Wed 2/8/12	Thu 2/9/12
23		Second Floor	61 days	Fri 2/10/12	Fri 5/4/12
24		Frame Slab	12 days	Fri 2/10/12	Sat 2/25/12
25		Rienforce Slab	10 days	Sat 2/25/12	Thu 3/8/12
26		Frame Beam & Girder	10 days	Thu 3/8/12	Wed 3/21/12
27		Rienforce Beam & Girder	7 days	Wed 3/21/12	Thu 3/29/12
28		Frame Columns	15 days	Thu 3/29/12	Wed 4/18/12
29		Reinforce Columns	5 days	Wed 4/18/12	Tue 4/24/12
30		Frame Walls	5 days	Tue 4/24/12	Mon 4/30/12
31		Reinforce Walls	2 days	Mon 4/30/12	Tue 5/1/12
32		Place Concrete in Slab	2 days	Tue 5/1/12	Wed 5/2/12
33		Place Concrete in Beams & Girder	2 days	Tue 5/1/12	Wed 5/2/12
34		Place Concrete in Columns	2 days	Wed 5/2/12	Thu 5/3/12
35		Place Concrete in Walls	2 days	Thu 5/3/12	Fri 5/4/12
36		Third Floor	64 days	Fri 5/4/12	Wed 8/1/12
37		Frame Slab	15 days	Fri 5/4/12	Thu 5/24/12
38		Rienforce Slab	10 days	Thu 5/24/12	Wed 6/6/12
39		Frame Beam & Girder	10 days	Wed 6/6/12	Tue 6/19/12
40		Rienforce Beam & Girder	7 days	Tue 6/19/12	Wed 6/27/12
41		Frame Columns	15 days	Wed 6/27/12	Tue 7/17/12
42		Reinforce Columns	5 days	Tue 7/17/12	Mon 7/23/12
43		Frame Walls	5 days	Mon 7/23/12	Fri 7/27/12
44		Reinforce Walls	2 days	Fri 7/27/12	Mon 7/30/12
45		Place Concrete in Slab	2 days	Mon 7/30/12	Tue 7/31/12
46		Place Concrete in Beams & Girder	2 days	Mon 7/30/12	Tue 7/31/12
47		Place Concrete in Columns	2 days	Tue 7/31/12	Wed 8/1/12
48		Place Concrete in Walls	2 days	Tue 7/31/12	Wed 8/1/12
49		Fourth Floor	64 days	Wed 8/1/12	Mon 10/29/12
50		Frame Slab	15 days	Wed 8/1/12	Tue 8/21/12
51		Rienforce Slab	10 days	Tue 8/21/12	Mon 9/3/12
52		Frame Beam & Girder	10 days	Mon 9/3/12	Fri 9/14/12
53		Rienforce Beam & Girder	7 days	Fri 9/14/12	Mon 9/24/12
54		Frame Columns	15 days	Mon 9/24/12	Fri 10/12/12
55		Reinforce Columns	5 days	Fri 10/12/12	Thu 10/18/12
56		Frame Walls	5 days	Thu 10/18/12	Wed 10/24/12
57		Reinforce Walls	2 days	Wed 10/24/12	Thu 10/25/12
58		Place Concrete in Slab	2 days	Thu 10/25/12	Fri 10/26/12
59		Place Concrete in Beams & Girder	2 days	Thu 10/25/12	Fri 10/26/12
60		Place Concrete in Columns	2 days	Fri 10/26/12	Mon 10/29/12
61		Place Concrete in Walls	2 days	Fri 10/26/12	Mon 10/29/12
62		Fifth Floor	64 days	Mon 10/29/12	Thu 1/24/13
63		Frame Slab	15 days	Mon 10/29/12	Fri 11/16/12
64		Rienforce Slab	10 days	Fri 11/16/12	Thu 11/29/12
65		Frame Beam & Girder	10 days	Thu 11/29/12	Wed 12/12/12
66		Rienforce Beam & Girder	7 days	Wed 12/12/12	Thu 12/20/12
67		Frame Columns	15 days	Thu 12/20/12	Wed 1/9/13
68		Reinforce Columns	5 days	Wed 1/9/13	Tue 1/15/13
69		Frame Walls	5 days	Tue 1/15/13	Mon 1/21/13
70		Reinforce Walls	2 days	Mon 1/21/13	Tue 1/22/13
71		Place Concrete in Slab	2 days	Tue 1/22/13	Wed 1/23/13
72		Place Concrete in Beams & Girder	2 days	Tue 1/22/13	Wed 1/23/13
73		Place Concrete in Columns	2 days	Wed 1/23/13	Thu 1/24/13
74		Place Concrete in Walls	2 days	Wed 1/23/13	Thu 1/24/13
75		Sixth Floor	64 days	Thu 1/24/13	Tue 4/23/13
76		Frame Slab	15 days	Thu 1/24/13	Wed 2/13/13
77		Rienforce Slab	10 days	Wed 2/13/13	Tue 2/26/13
78		Frame Beam & Girder	10 days	Tue 2/26/13	Mon 3/11/13
79		Rienforce Beam & Girder	7 days	Mon 3/11/13	Tue 3/19/13
80		Frame Columns	15 days	Tue 2/19/13	Mon 3/11/13
81		Reinforce Columns	5 days	Mon 3/11/13	Fri 3/15/13
82		Frame Walls	5 days	Fri 3/15/13	Thu 3/21/13
83		Reinforce Walls	2 days	Thu 3/21/13	Fri 3/22/13
84		Place Concrete in Slab	2 days	Fri 3/22/13	Mon 3/25/13
85		Place Concrete in Beams & Girder	2 days	Fri 3/22/13	Mon 3/25/13
86		Place Concrete in Columns	2 days	Mon 3/25/13	Tue 3/26/13
87		Place Concrete in Walls	2 days	Mon 3/25/13	Tue 3/26/13
88		Roof	64 days	Tue 3/26/13	Fri 6/21/13
89		Frame Slab	15 days	Tue 3/26/13	Mon 4/15/13
90		Rienforce Slab	10 days	Mon 4/15/13	Fri 4/26/13
91		Frame Beam & Girder	10 days	Fri 4/26/13	Thu 5/9/13
92		Rienforce Beam & Girder	7 days	Thu 5/9/13	Fri 5/17/13
93		Frame Columns	15 days	Fri 5/17/13	Thu 6/6/13
94		Reinforce Columns	5 days	Thu 6/6/13	Wed 6/12/13
95		Frame Walls	5 days	Wed 6/12/13	Tue 6/18/13
96		Reinforce Walls	2 days	Tue 6/18/13	Wed 6/19/13
97		Place Concrete in Slab	2 days	Wed 6/19/13	Thu 6/20/13
98		Place Concrete in Beams & Girder	2 days	Wed 6/19/13	Thu 6/20/13
99		Place Concrete in Columns	2 days	Thu 6/20/13	Fri 6/21/13
100		Place Concrete in Walls	2 days	Thu 6/20/13	Fri 6/21/13



Appendix 13: Green House Structure

	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Diameter:	0.357	inch			
	Misc. Dead Load:	45	psf	Bar Size:	#	8	ameter, d_b:	1.000	inch			
	Live Load:	100	psf	# of bars, n:	4		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	3.16	in^2			
Green Roof Beam Design												
							Tributary Area:	295	ft^2			
	Span:	20	feet	Two Row Reinforcement?	Yes		Influence Area:	590	ft^2	Live Load Reduction		
	Spaced:	14.75	feet				L_L=Max of	0.4				
								.25+15/√(K_{LL}*A_T)=		0.87	L_L:	86.75
Spacing,S:	Max of d_b, 1", 3/4A_b			b_{min} =2*Cc + n*d_b + 2d_{st} + (n-1)*S								
d_b:	1.0	inch		b_{min}:	6.71	inch				Total Factored Weight, Wu		
3/4A_b:	0.6	inch								Dead Load: Misc. dead + Slab		
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)		able 9.5 min h>l/18.5					126.25	psf
				h_{min}:	12.97	inch	min h:	13.0		Wu=1.2D +1.6L		
S:	1.0	inch								290.306	psf	
Try a:												
b:	10	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	20	inch	OK		d:	17		a:	0.93	Rectangular	c:	1.09
b_{eff}= min	b*16*h_f	1040			ε_t=ε_u(d-c)/c							
	Trib widt	177			ε_t:	0.04	Φ=0.9					
	.25L	60										
b_{eff}:	60	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Girder width, b:		12	inch		
Mu=Wu*Ln^2/8					Vu=Wu*Ln/2							
Mu:	193.2	kip-ft			Vu:	42.8	Kip					
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	235.1	kip-ft	OK		Vn:	126491	Kip	OK				
Use Member Size:		**10**	x	**20**								
	Beam Wieght:	141	plf									


y
x
16 x 16 in
3.13% reinf.
MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi
SECTION:
========
Ag = 256 in^2
Ix = 5461.33 in^4
Iy = 5461.33 in^4
Xo = 0 in
Yo = 0 in
REINFORCEMENT:
===============
8 #9 bars @ 3.125%
As = 8 in^2
Confinement: Tied
Clear Cover = 2.50 in
Min Clear Spacing = 3.81 in
P (kip)
1000
(Pmax)
fs=0
fs=0.5fy
0
250
My (k-ft)
(Pmin)
-500

	f'c:	4	ksi	Clear Cover:	1.5	inch						
	fy:	60	ksi	Conc. Wieght:	150	pcf						
	Slab, t:	6.5	inch	Stirrup Size:	#	3	Diameter:	0.357	inch			
	Misc. Dead Load:	45	psf	Bar Size:	#	8	ameter, d_b:	1.000	inch			
	Live Load:	100	psf	# of bars, n:	7		Area of Steel, A_b:	0.79	in^2			
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	5.53	in^2			
Green Roof Girder Design							Tributary Area:	590	ft^2			
	Span:	29.5	feet	Two Row Reinforcement?	Yes		Influence Area:	1180	ft^2	Live Load Reduction		
	Spaced:	20	feet				L_L=Max of	0.4				
								.25+15/√(K_{LL}*A_T)=		0.69	L_L:	68.67
Spacing,S:	Max of d_b, 1", 3/4A_b			b_{min} =2*Cc + n*d_b + 2d_{st} + (n-1)*S								
d_b:	1.0	inch		b_{min}:	9.71	inch				Total Factored Weight, Wu		
3/4A_b:	0.6	inch								Dead Load: Misc. dead + Slab		
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)		able 9.5 min h>l/18.5					126.25	psf
				h_{min}:	19.14	inch	min h:	19.1		Wu=1.2D +1.6L		
S:	1.0	inch								261.367	psf	
Try a:												
b:	18	inch	OK		d=h-d_b/2-Cc			a=A_s*fy/(.85*f'c*b_{eff})			c=a/β_1	
h:	30	inch	OK		d:	27		a:	1.10	Rectangular	c:	1.30
b_{eff}= min	b*16*h_f	1872			ε_t=ε_u(d-c)/c							
	Trib width	240			ε_t:	0.06	Φ=0.9					
	.25L	88.5										
b_{eff}:	88.5	inch										
Check Flexure, ΦMn>Mu					Cheak Shear, Vn>Vu		Girder width, b:	0	inch			
Mu=Wu*Ln^2/8					Vu=Wu*Ln/2							
Mu:	568.6	kip-ft			Vu:	77.1	Kip					
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	658.2	kip-ft	OK		Vn:	341526	Kip	OK				
Use Member Size:		**18**	**x**	**30**								
Beam Wieght:		441	plf									

Appendix 14: LEED References

Table I. Functional Unit—30% Green Roof Replacement on Typical Urban Building Stock

Building type	Number of households	Conditioned space per household (sq ft)	Average number of floors	Annual energy use (Mill BTU/HH resid; kBTU/sf comm)	Total roof area (1,000 sq ft)	Total replaced roofing (1,000 sq ft)
Single-family detached	3,000	2,500	1.5	59	5,000	1,500
Single-family attached	500	1,800	2	59	450	140
Multifamily, 2–4 units	500	800	3	51	130	40
Multifamily, >5 units	1,400	700	5	18	200	60
Commercial	—	3,400	5	57	680	200

Units	$/MT	$/kWh	$/kgal
Market value	$21.47	$0.0982	$2.27
Reference	Capoor and Ambrosi (2008)[6]	Energy Information Administration (2009)[18]	Fisher et al. (2008)[19]

Table IV. Costs, Energy Used, and GHGs Released from Producing and Replacing 30% of Existing Roofs with Green Roofs in a Typical Urban Neighborhood over 30 Years

		Private costs ($1,000)			Energy used (MWh)		GHGs released (MT CO2 eq)	
Building type	Roofing replaced (1,000 sq ft)	Materials	Construction	Total	Materials	Construction	Materials	Construction
Single family	1,600	($5,100)	($7,600)	($13,000)	(59)	(0.41)	(19,000)	(3,000)
Multifamily	100	($690)	($690)	($1,400)	(5.9)	(0.042)	(1,800)	(270)
Commercial	200	($1,400)	($2,200)	($3,600)	(15)	(0.14)	(4,600)	(840)
All	1,900	($7,200)	($10,000)	($17,000)	(79)	(0.59)	(25,000)	(4,100)

Table V. Reduced Electricity Use from Green Roof Installation over a 30-Year Planning Horizon

	Electricity use reductions (MWh)				Private benefits	Public benefits
	Private	Social				
Building type	Direct energy savings	UHI energy savings	CSO energy savings	Total	Market value of energy savings ($1,000)	Market value of energy savings ($1,000)
Single family	4,700	67,000	530	67,000	$210	$7,200
Multifamily	790	11,000	32	11,000	$34	$1,200
Commercial	3,500	28,000	67	28,000	$150	$3,100
All categories	9,100	110,000	640	110,000	$390	$12,000

Table VI. Greenhouse Gas Reductions from Green Roof Installation over a 30-Year Planning Horizon

	GHG reductions (MT CO2 eq)					Public benefits
Building type	Direct energy mitigation	UHI energy mitigation	CSO energy mitigation	Sequestered	Total GHG mitigation	Market value of CO2 mitigated ($1,000)
Single family	3,300	47,000	370	390	51,000	$630
Multifamily	530	7,700	20	24	8,300	$130
Commercial	2,300	19,000	46	49	21,000	$340
All categories	6,100	74,000	436	470	81,000	$1,100