University Medical Center of Princeton

- Introduction
- Architecture
- Proposed Solution
- Gravity Redesign
- Vibration Concern
- Lateral Redesign
- Design Comparisons
- Conclusion
- Questions/Comments

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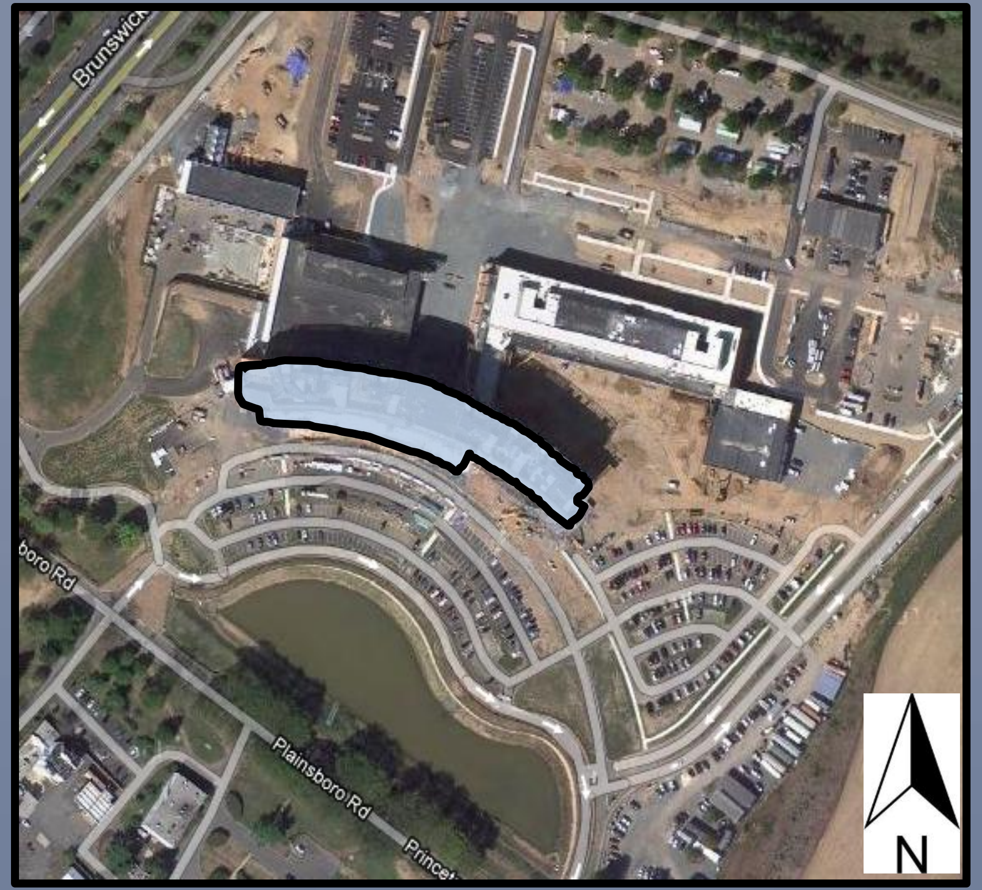
Structures Option
Senior Thesis 2012
Faculty Advisor: Professor Parfitt

Building Introduction

- University Medical Center of Princeton
- Located in Plainsboro, NJ
- 639,000 square feet
- Stands seven stories, 91' tall
- $300 million project
- Broke ground Spring 2008
- Plans to finish May 2012
- Sustainable features are implemented in the design, but the project is not LEED Certified

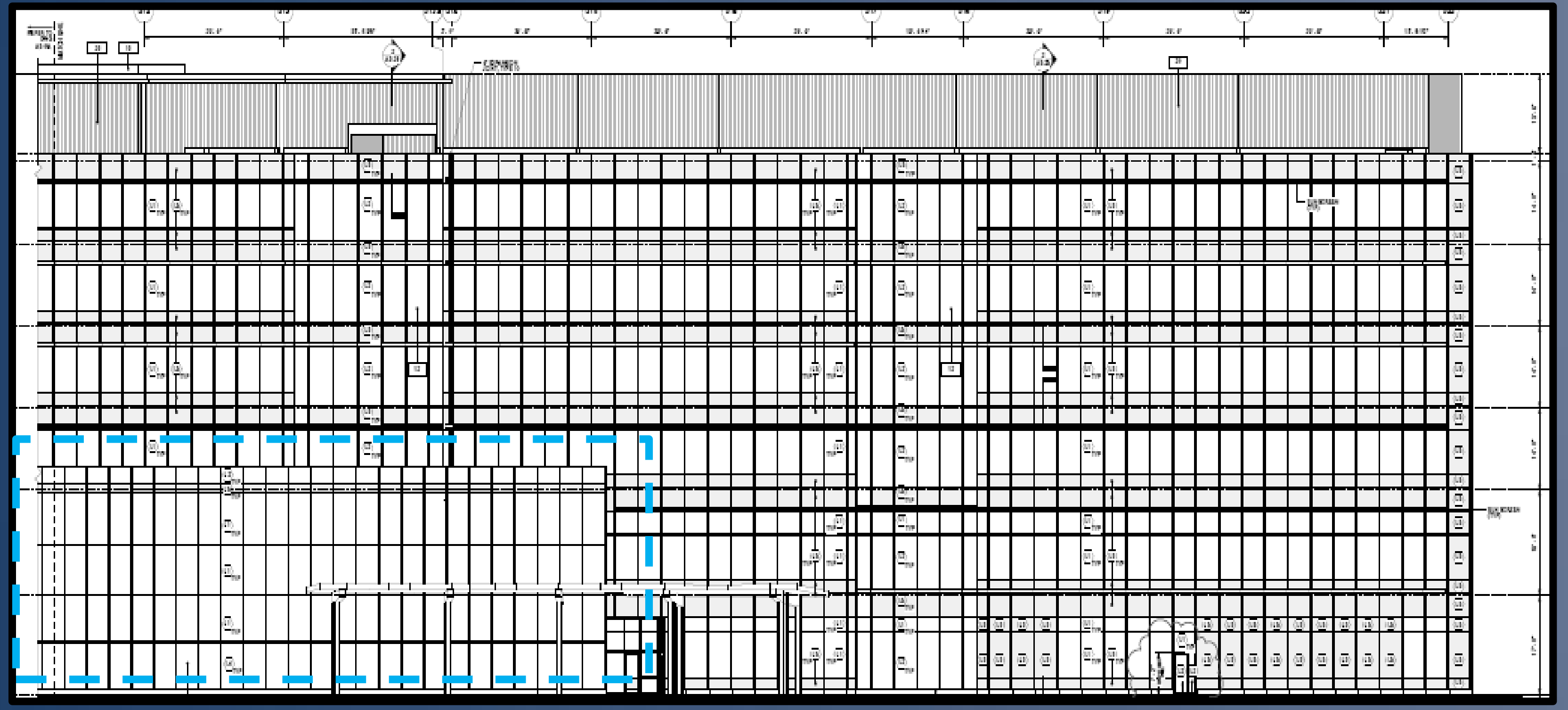
Site Map



Project Team

- **Owner:** Princeton University
- **General Contractor:** Turner Construction
- **Architect:** HOK
- **Structural Engineer:** O'Donnell & Naccarato
- **Civil Engineer:** French & Parrello Associates
- **MEP:** Syska Hennessy Group, Inc.

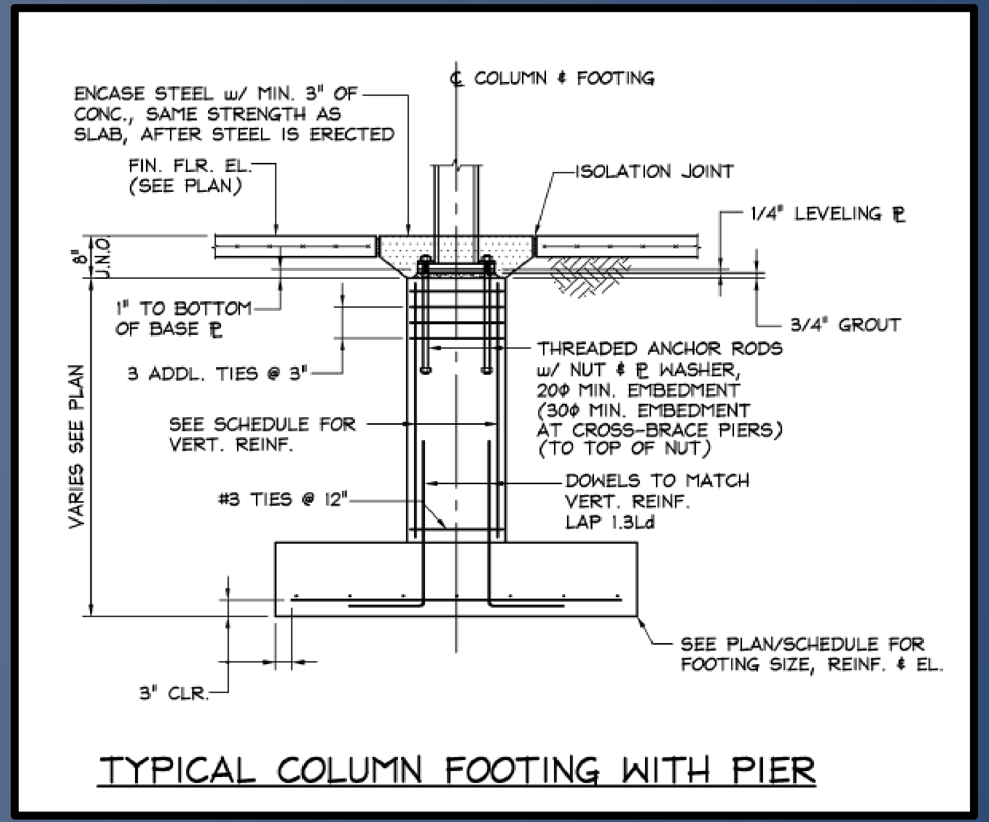
South Elevation



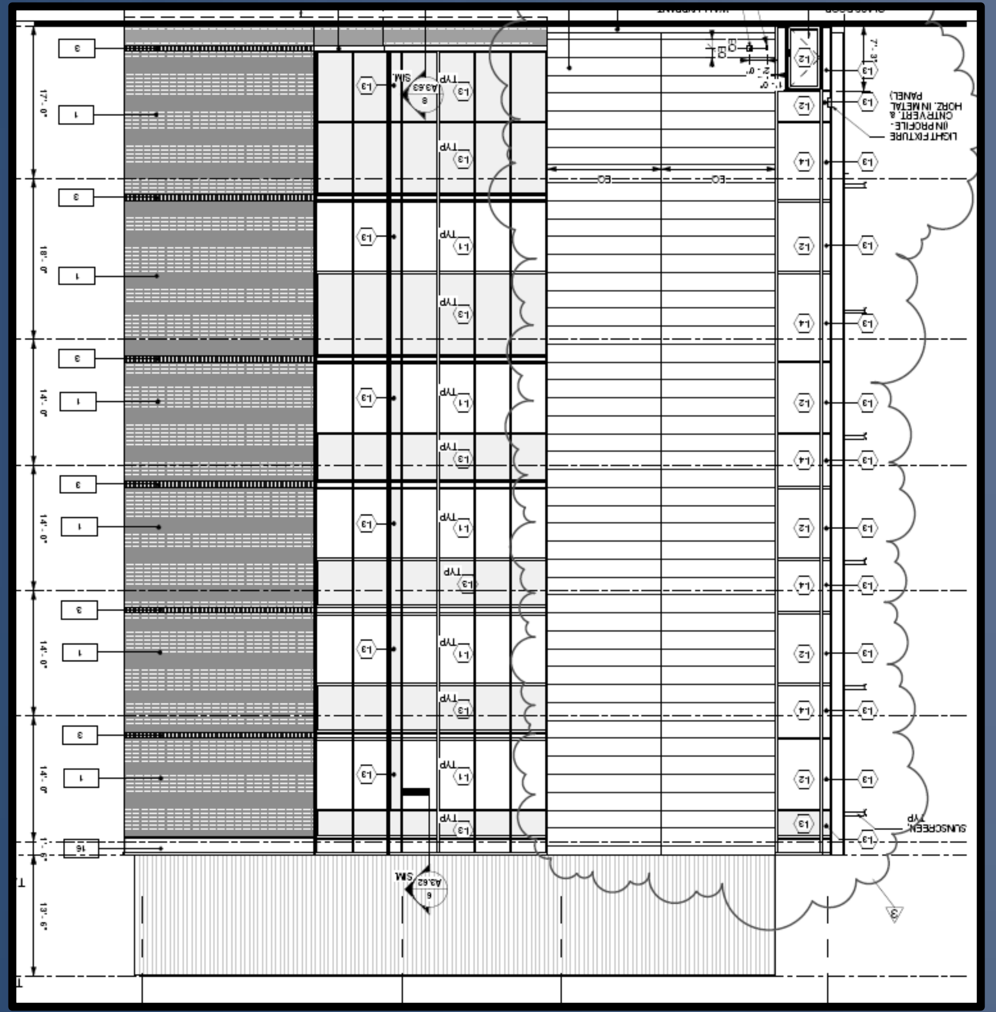
Architecture Features

- Façade
  - Glass Curtain Wall
  - Two Story Atrium
- Existing gravity system
  - Composite beam/decking
    - Slab thickness: 3”
    - Beam depth: 16”
    - Girder depth: 24”
  - Concrete spread footing foundation

Existing Gravity System



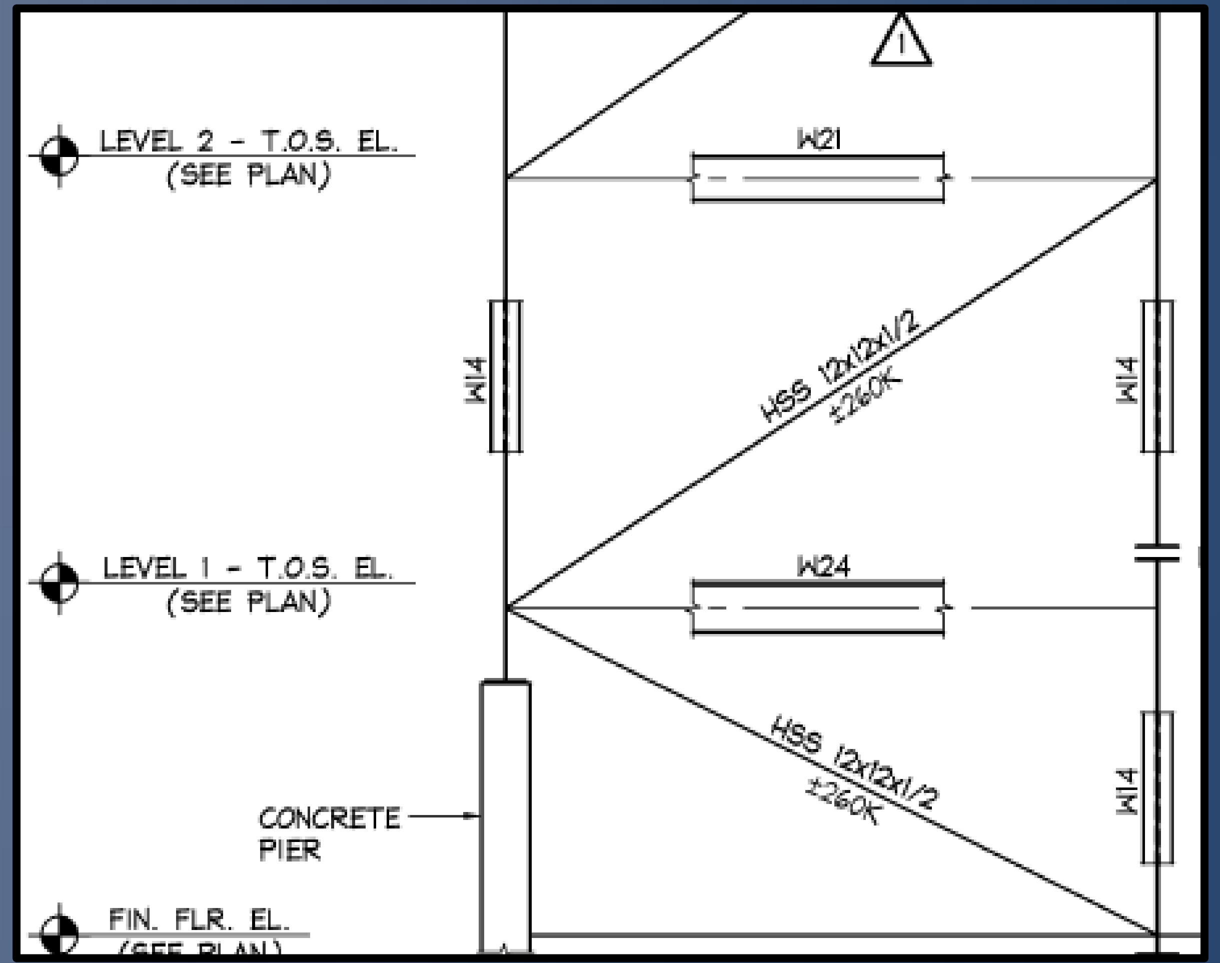
East Elevation



Architecture Features

- Façade
  - Brick Veneer
  - Glass
  - Aluminum Panel
- Existing lateral System
  - Braced frame
  - Moment frame

Typical Braced Frame



Goals

- Design adequate system
  - Strength
  - Serviceability
- Create a cost efficient structure that is simple, fast, and quick to erect
- Become LEED Certified

Structural Depth

- Gravity System
  - One-way slab with beams
- Lateral System
  - Concrete moment frames
  - Shear walls

Thesis Breadth

- Comparison Breadth
  - Cost analysis
  - Schedule analysis
- Sustainability Breadth
  - Green roof design
  - Interior sustainable techniques

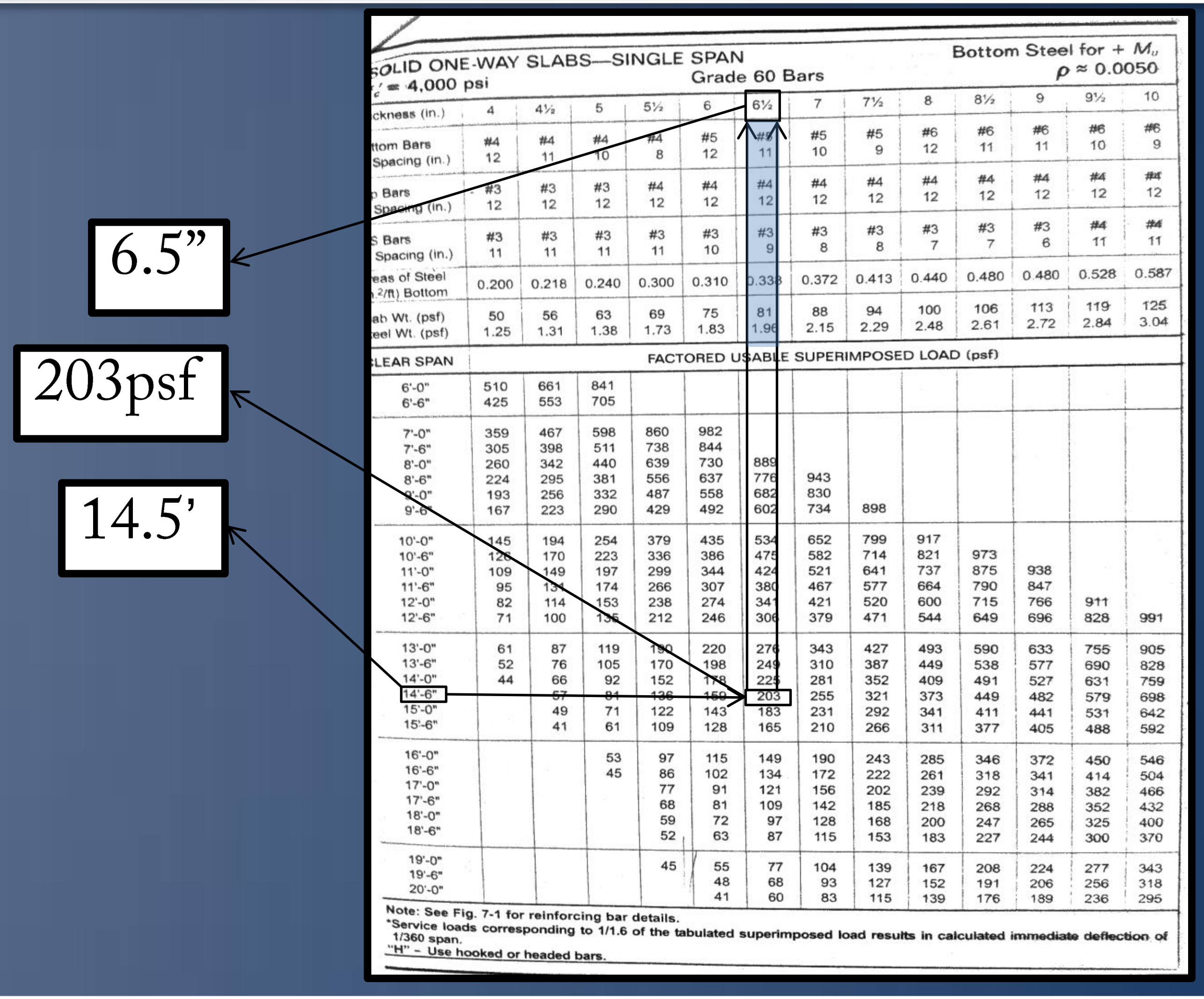
Gravity Redesign

Slab Design

Slab Design Table

- Controlling load combination: 1.2D+1.6L
- Live load: 80 psf
- Superimposed dead load: 35 psf
  - MEP system
  - ACT tiles
  - Certain hospital equipment
  - Other finishes
  - Collateral
- Factored superimposed load: 170 psf
- Table 9.5 from the ACI manual helped narrow down min depths of beams and girders for deflection concerns

- Span: 14.5'
- Allowable weight: 203psf
- Slab thickness: 6.5"
- Bottom reinforcement: #7 spaced at 11"
- Top reinforcement: #4 spaced at 12"
- T&S: #3 spaced at 9"
- Slab weight: 81 psf



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	#4	#4	#4	#4	#5	#5	#5	#5	#6	#6	#6	#6	#6
Spacing (in.)	12	11	10	8	12	11	10	9	12	11	11	10	9
p Bars	#3	#3	#3	#4	#4	#4	#4	#4	#4	#4	#4	#4	#4
Spacing (in.)	12	12	12	12	12	12	12	12	12	12	12	12	12
S Bars	#3	#3	#3	#3	#3	#3	#3	#3	#3	#3	#3	#4	#4
Spacing (in.)	11	11	11	11	10	9	8	8	7	7	6	11	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)	50	56	63	69	75	81	88	94	100	106	113	119	125
eel Wt. (psf)	1.25	1.31	1.38	1.73	1.83	1.96	2.15	2.29	2.48	2.61	2.72	2.84	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

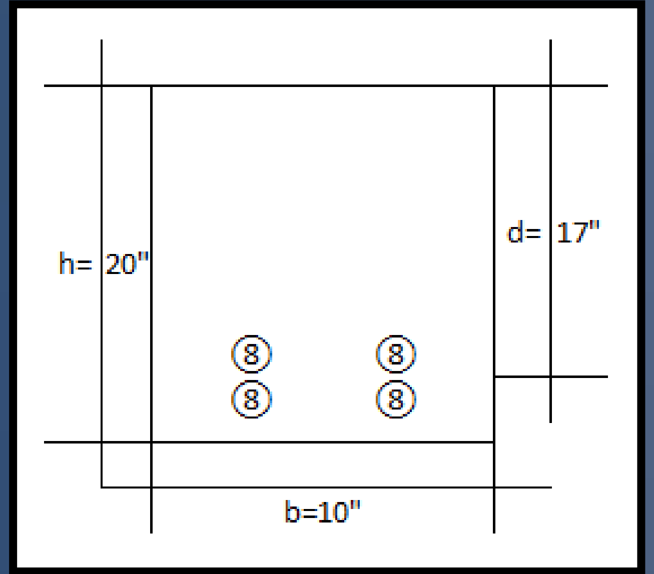
Note: See Fig. 7-1 for reinforcing bar details.
*Service loads corresponding to 1/1.6 of the tabulated superimposed load results in calculated immediate deflection of 1/360 span.
"H" – Use hooked or headed bars.

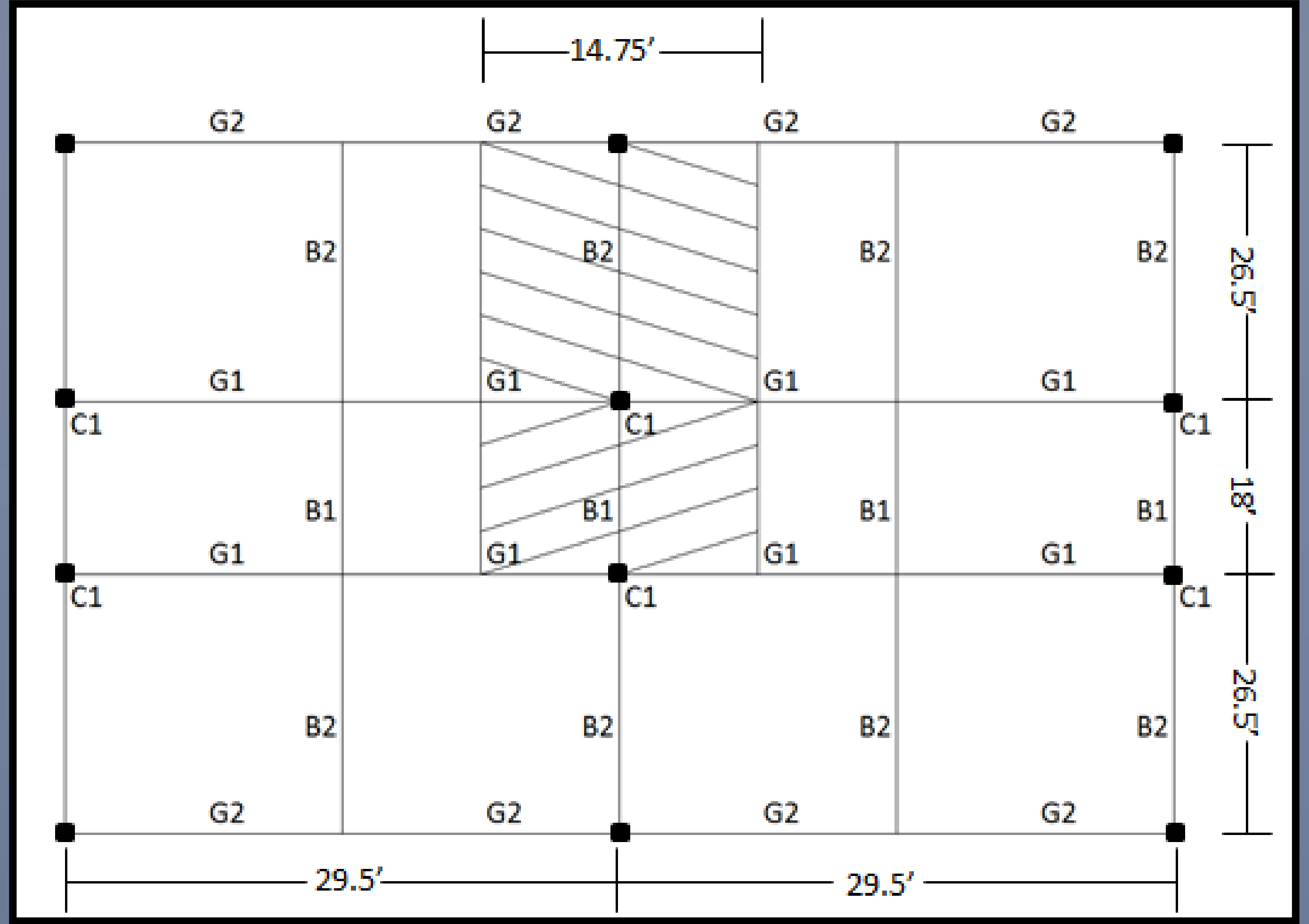
B1 Design

Beam Design

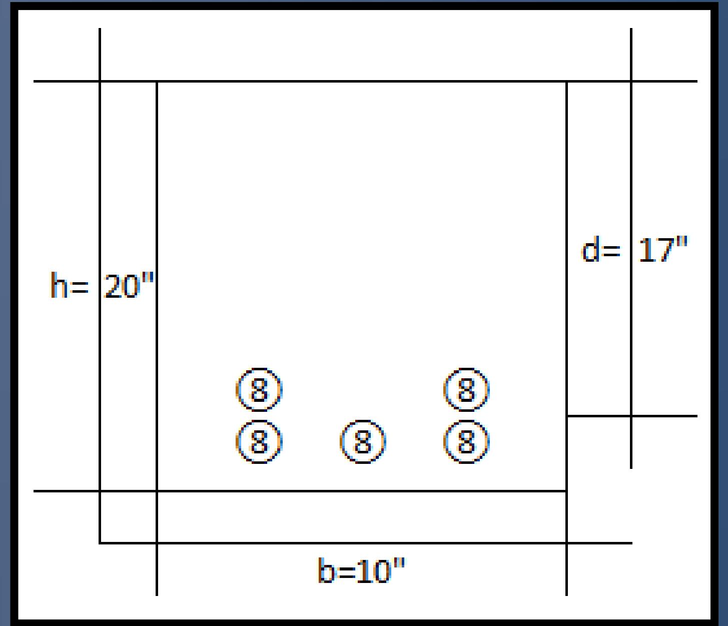
B2 Design

- Reinforcement: (4) #8 with #3 Stirrups
- Section Size: 10x20
- Mu=139k-ft < ΦMn=234k-ft
- Vu=34kips < ΦVn=126kips



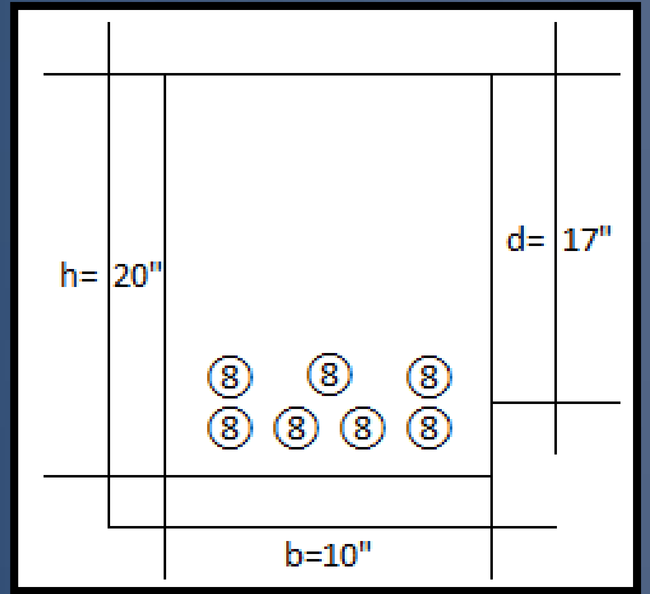


- Reinforcement: (5) #8 with #3 Stirrups
- Section Size: 10x20
- Mu=292k-ft < ΦMn=295k-ft
- Vu=47kips < ΦVn=127kips

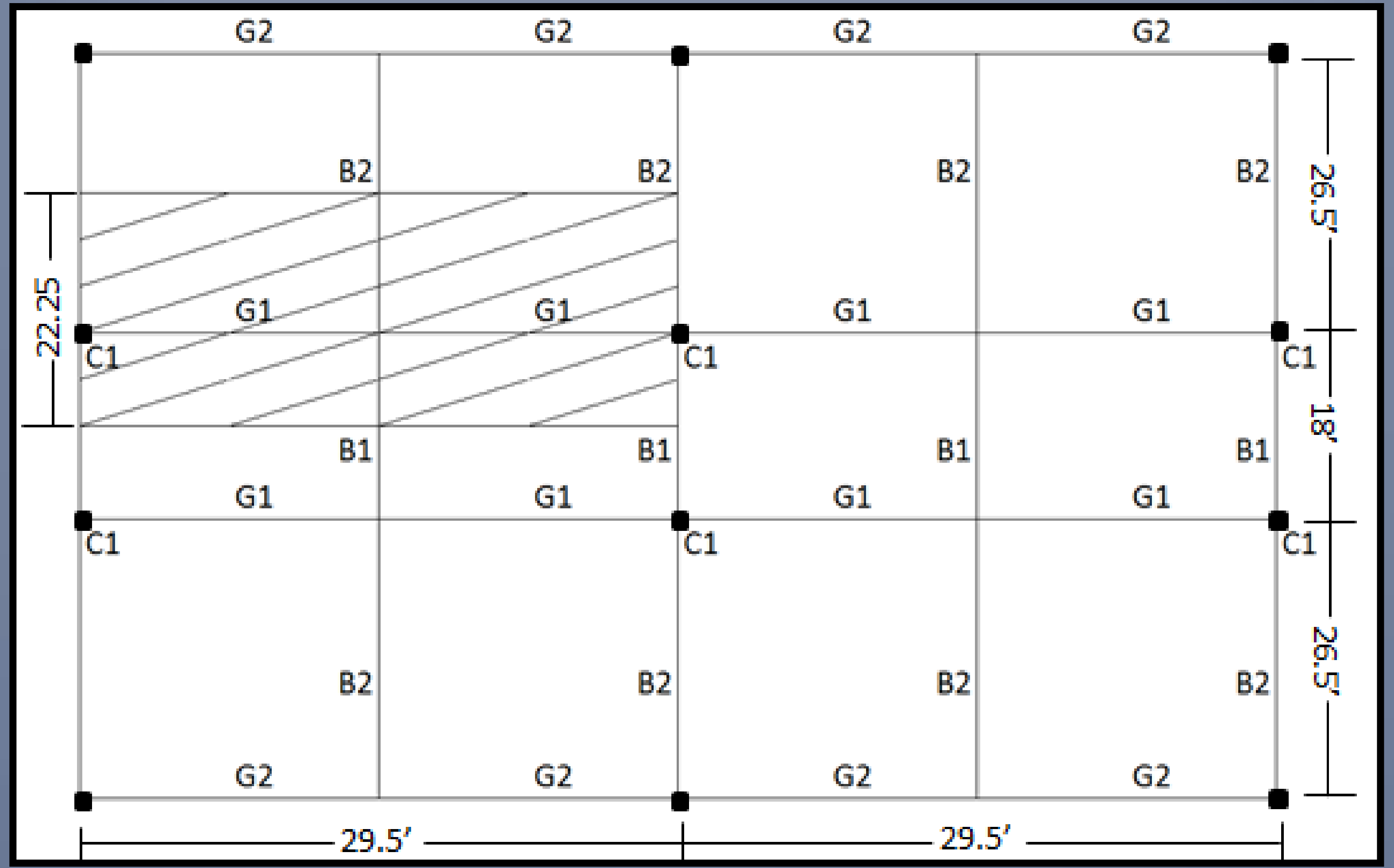


Typical G1 Design

- Reinforcement: (7) #8 with #3 Stirrups
- Section Size: 10x20
- Mu=398k-ft < ΦMn=409k-ft
- Vu=84kips < ΦVn=126kips

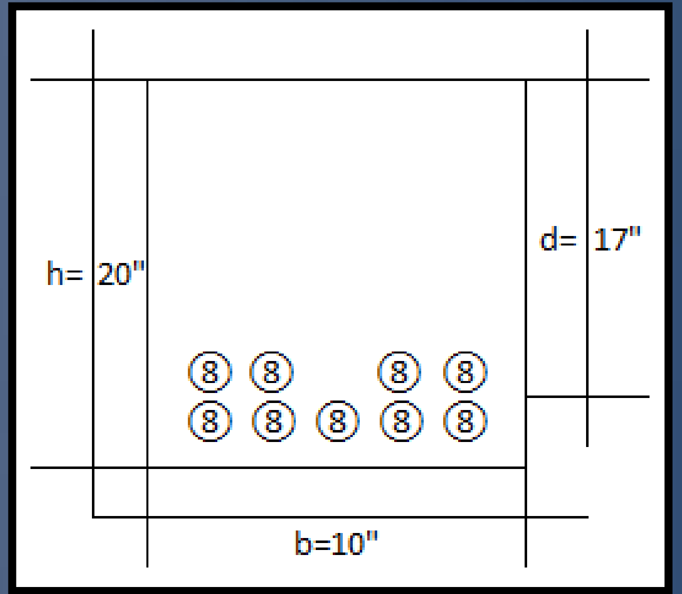


Girder Design



32' Max Span G1 Design

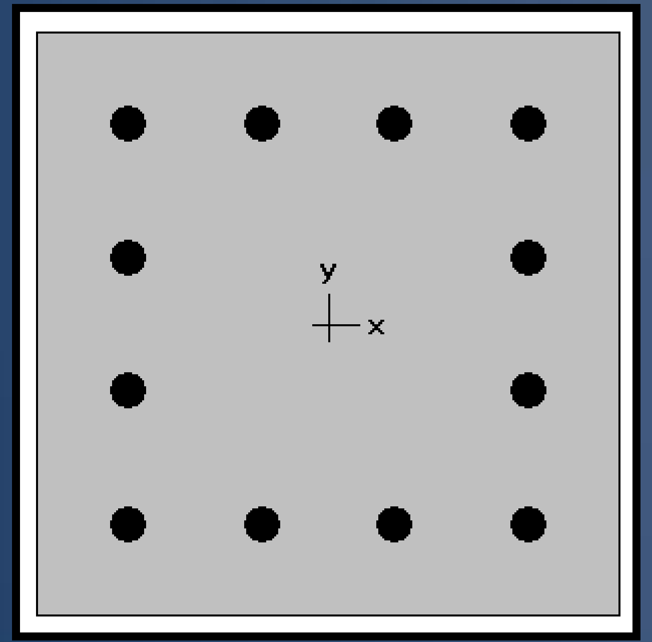
- Reinforcement: (9) #8 with #3 Stirrups
- Section Size: 10x20
- Mu=469k-ft < ΦMn=523k-ft
- Vu=90kips < ΦVn=152kips

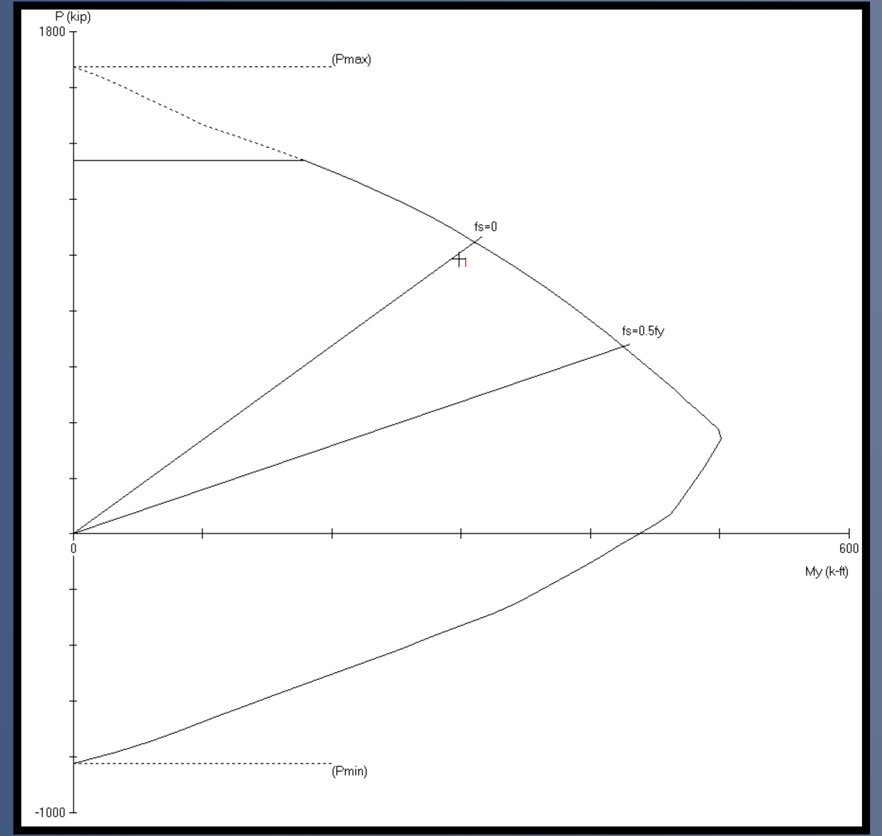


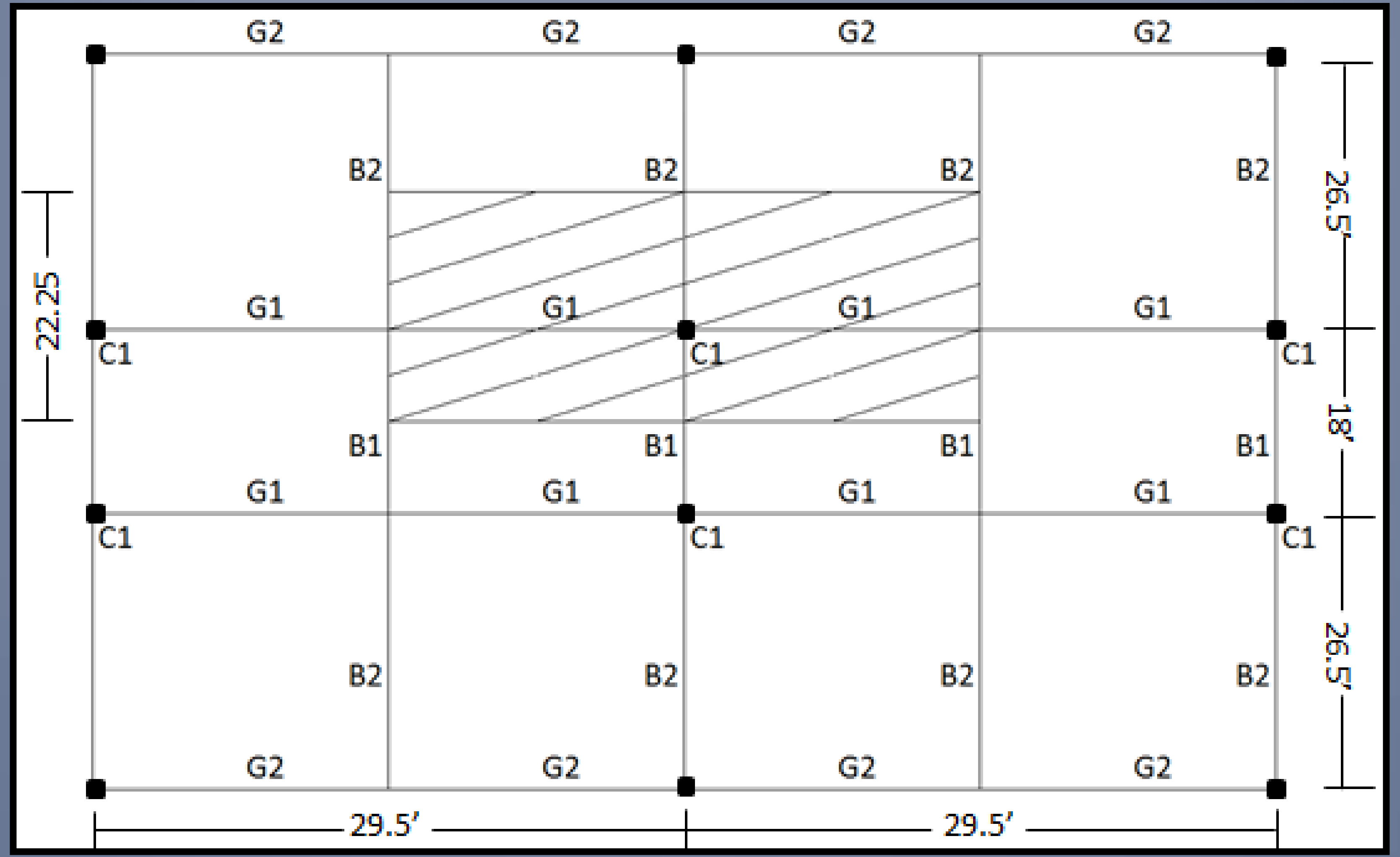
Column Design

Typical C1 Design

- Reinforcement: (12) #10 with #3 Stirrups
- Section Size: 20x20
- Pu = 986 kips
- Mu = 298 k-ft

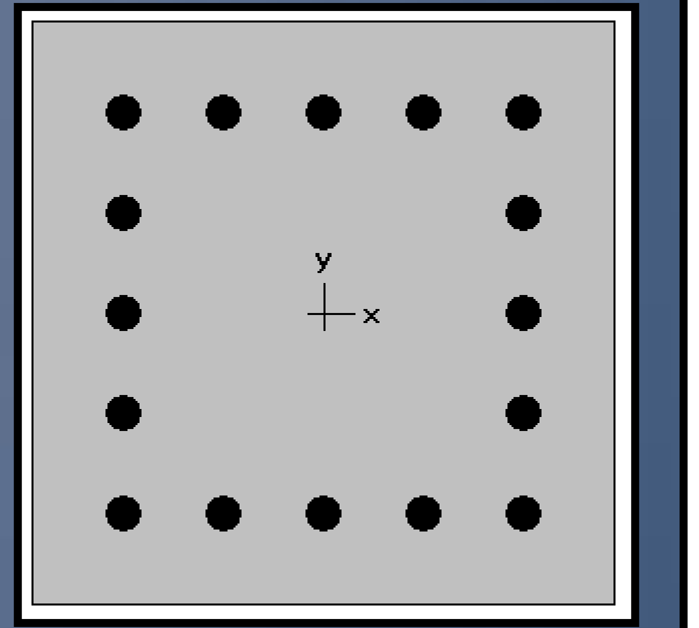


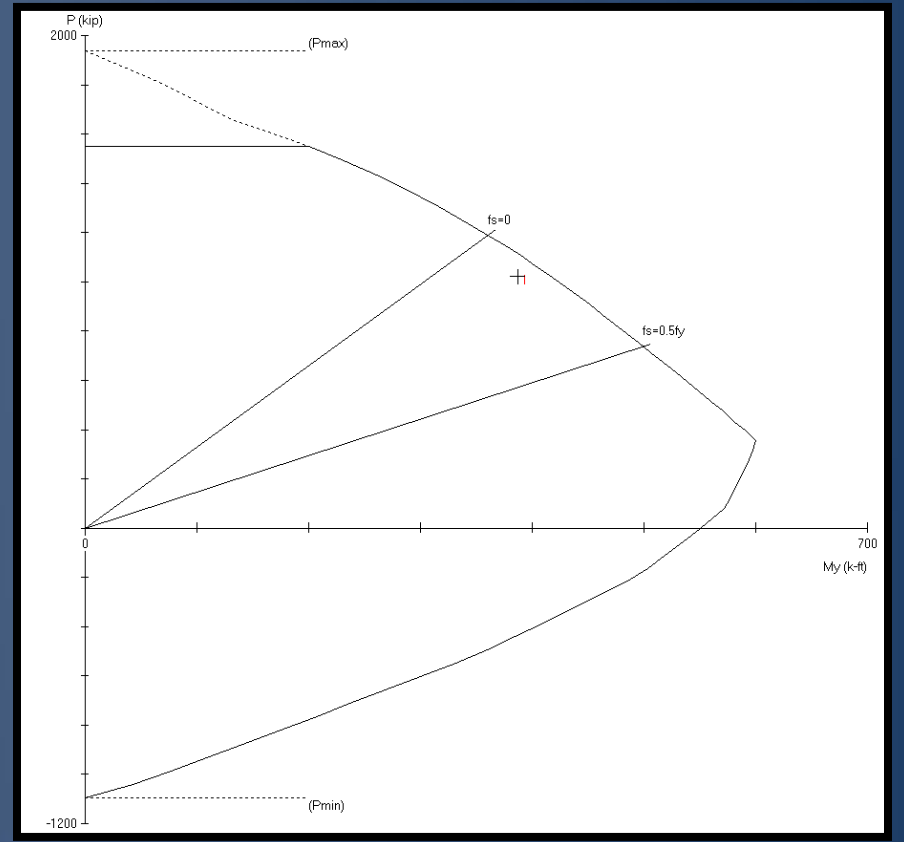




32' Max Span C1 Design

- Reinforcement: (16) #10 with #3 Stirrups
- Section Size: 20x20
- Pu = 1022 kips
- Mu = 387 k-ft





Vibration Concern

- Steel structures
  - Vibration concerns at 4000 – 2000 micro-in/s
- Concrete structures
  - Vibration concerns at 1000 micro-in/s
- Operating room & patient room concern
  - Vibration should be kept below 4000 micro-in/s

Vibration Allowance

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

Vibration Phenomenon

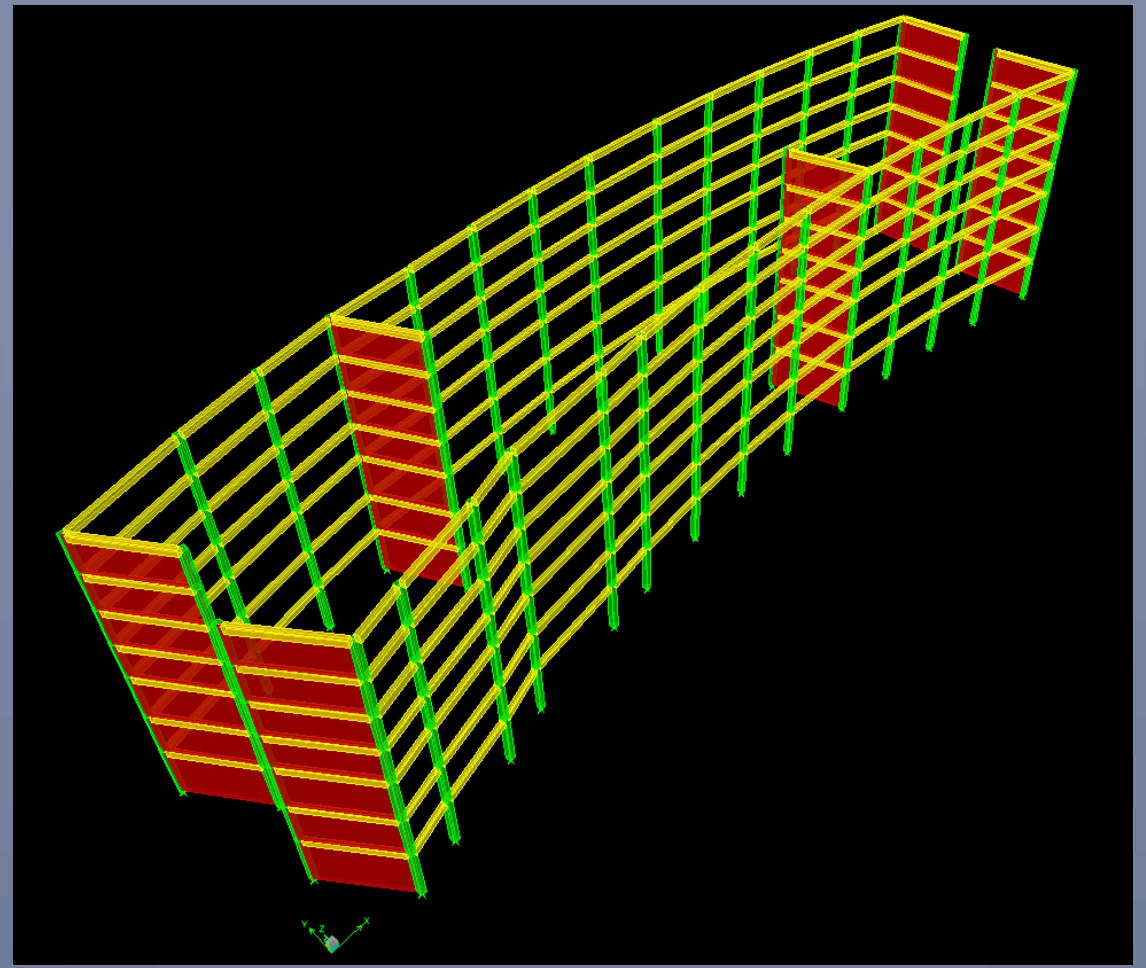
- Vibration occurs during
  - Footfall of a person or persons
    - 50 steps/min is considered to equal 4000 micro-in/s
  - Mechanical gyration from a machine
    - Isolated slabs
- Concrete structures are better for vibration because the slabs are so massive creating a better damping quality

ETAB Multipliers

Lateral Redesign

ETAB's Model

- Multipliers
  - 0.7Ig multiplier applied to columns
    - Chapter 10.10.4.1 in ACI 318-08
  - 0.35Ig multiplies applied to girders
    - Chapter 10.10.4.1 in ACI 318-08
  - 0.5 end offset applied



- Model
  - Column Constraints: Pin
  - Controlling Load Combo: 1.2D+W+L
  - Shear Walls (Red)
  - Moment Frames (Yellow/Green)
- Assumptions
  - Slab acts as rigid diaphragm
  - P-Δ effects considered within the model
  - Shear walls take no out-of-plane shear
  - Accidental torsional effects, e = 0.05

Wind Forces North/South

- Wind Force
  - ASCE 7-10 wind load cases applied
  - Load Case 3 controlled in both directions
  - Story drift checked for H/400

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	

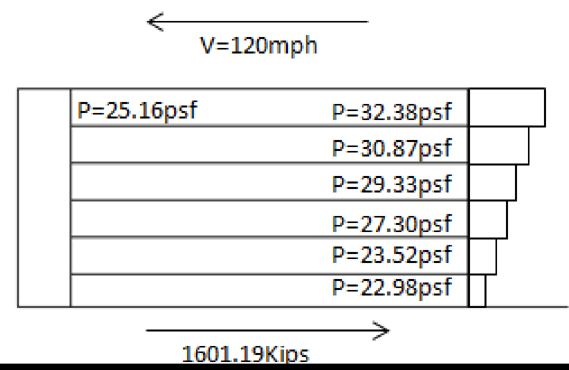
Critical Variables Found for Wind Analysis

B:	370.00	G:	0.85
g_q:	3.40	n_a:	0.75
gv:	3.40	I_z:	0.18
c:	0.20	Q:	0.80
z(bar):	54.60	N_1:	3.36
L_z:	552.98	R_n:	0.07
b(bar):	0.65	β:	0.02
$\bar{\alpha}$:	0.15	g_R:	4.12
Vz(bar):	123.61	$R_{h,n}$:	2.54
l:	500.00	$R_{L,n}$:	7.94
$\bar{\epsilon}$:	0.20	$R_{B,n}$:	10.33
h:	91	R_h:	0.32
L:	85	R_L:	0.12
V:	120.00	R_B:	0.09
$\hat{a}$:	9.5	R:	0.24
z_g:	900		

L_{EFF}
1350

Base Shear
1601.19 k

Over Turning Moment
82767 k-ft



Base shear: 1,601 kips

Overturning moment: 82,767 k-ft

Wind Drift North/South Direction (Y)

Story	Allowable Drift	Check Y-Dir.	X-Dir.	Y-Dir.	Total Drift
1	**0.51**	OK	0.0002	0.0002	**0.0003**
2	**0.54**	OK	0.0004	0.0002	**0.0005**
3	**0.42**	OK	0.0006	0.0003	**0.0007**
4	**0.42**	OK	0.0009	0.0003	**0.0009**
5	**0.42**	OK	0.0014	0.0002	**0.0014**
6	**0.42**	OK	0.0020	0.0002	**0.0021**
Roof	**0.42**	OK	0.0043	0.0002	**0.0043**

- Wind Force
  - ASCE 7-10 wind load cases applied
  - Load Case 3 controlled in both directions
  - Story drift checked for H/400

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	

Critical Variables Found for Wind Analysis

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
':	0.20
h:	91
L:	370
V:	120.00
º:	9.5
z_g:	900

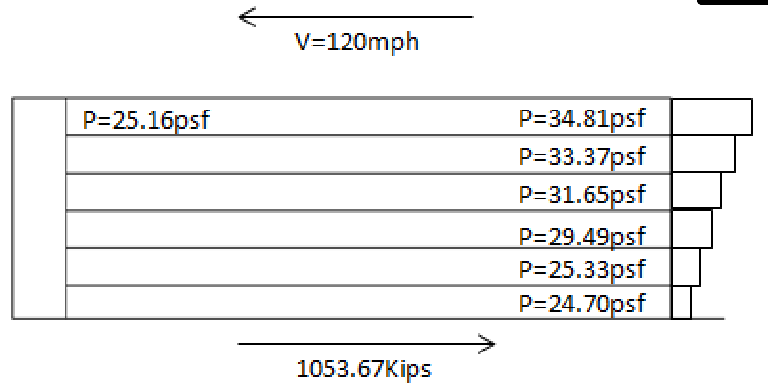
G:	0.93
n_1:	0.75
I_z:	0.18
Q:	0.87
N_1:	3.36
R_n:	0.07
β:	0.02
g_R:	4.12
$R_{h,n}$:	2.54
$R_{L,n}$:	34.59
$R_{B,n}$:	2.37
R_h:	0.32
R_L:	0.03
R_B:	0.33
R:	0.43

L_{EFF} 310

Base Shear	Over Turning Moment
1053.67 k	**52517.79 k-ft**

Base shear: 1,054 kips

Overturning moment: 52,518 k-ft



Wind Drift East/West Direction (X)

Story	Allowable Drift	Check X-Dir.	X-Dir.	Y-Dir.	Total Drift
1	**0.51**	OK	0.0004	0.0000	**0.0004**
2	**0.54**	OK	0.0009	0.0001	**0.0009**
3	**0.42**	OK	0.0017	0.0001	**0.0017**
4	**0.42**	OK	0.0025	0.0001	**0.0025**
5	**0.42**	OK	0.0041	0.0002	**0.0041**
6	**0.42**	OK	0.0058	0.0002	**0.0059**
Roof	**0.42**	OK	0.0277	0.0010	**0.0277**

Lateral Forces

Seismic Forces

Serviceability

- Seismic Force
  - Response modification factor, R=3
  - Seismic design category, “B”
  - Importance factor, I = 1.25
  - Deflection amplification factor,
    - Moment Frame, C_D = 2.5
    - Shear Wall, C_D = 3.0
  - Story drift checked for $0.015h_{sx}$
  - Story drifts taken from ETABs were adjusted by code by multiplying them by $h_{sx}(C_D/I)$

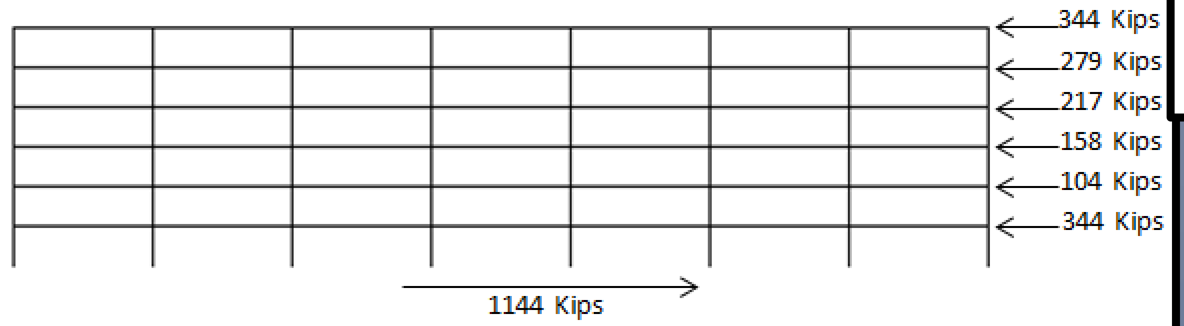
Seismic Loading

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



Base shear: 1,601 kips

Overturning moment: 82,767 k-ft

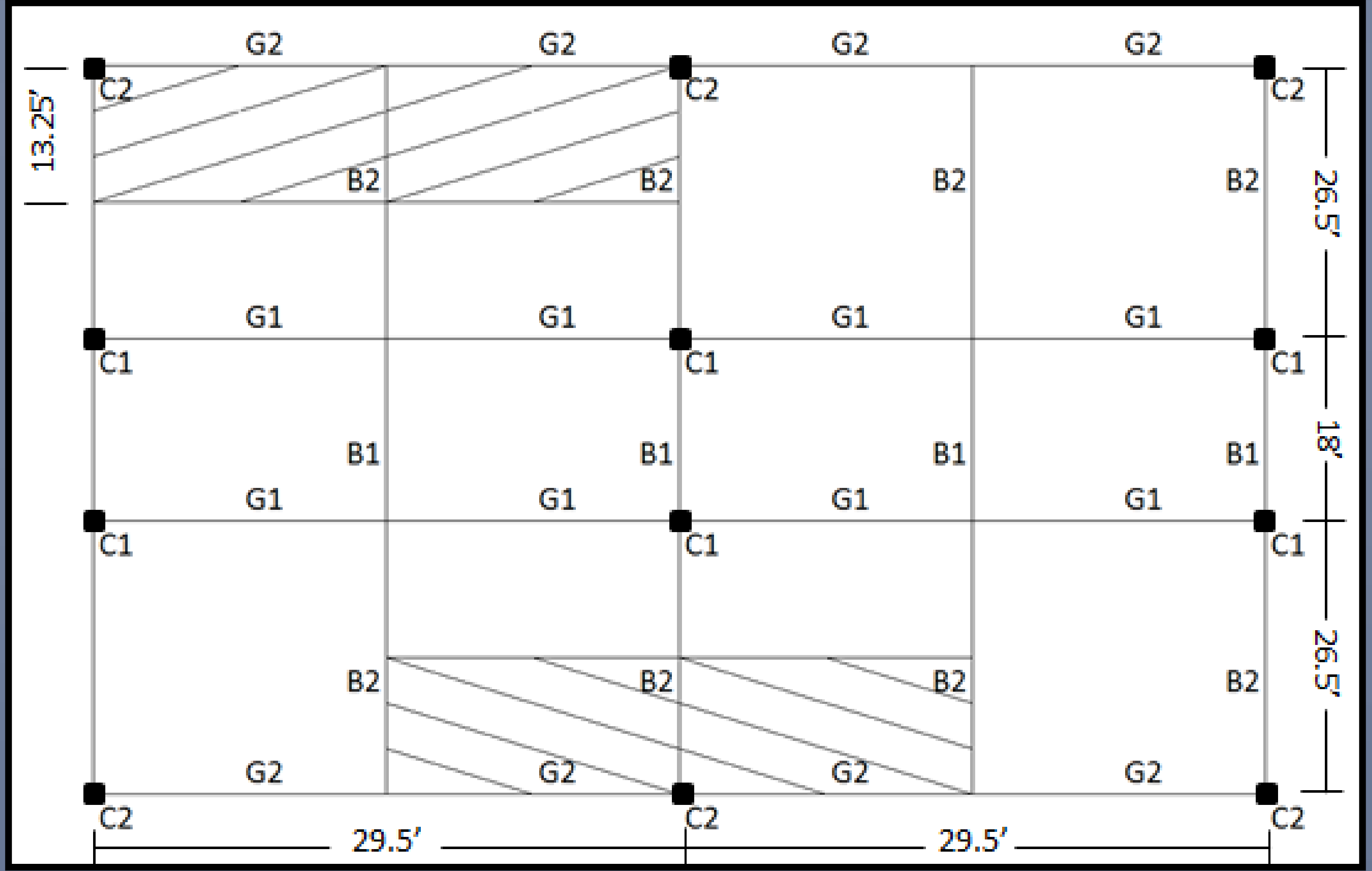
Seismic Drift East/West Direction (X- Direction), I=1.25 & C_D=2.5					
Story	Allowable Drift	Check X-Dir.	X-Dir.	Y-Dir.	Total Drift
Roof	2.52	OK	0.648	0.030	0.648
6	2.52	OK	1.043	0.041	1.044
5	2.52	OK	1.413	0.052	1.414
4	2.52	OK	1.785	0.064	1.787
3	2.52	OK	2.436	0.081	2.437
2	3.24	OK	3.197	0.110	3.199
1	3.06	OK	2.927	0.186	2.933

Seismic Drift North/South Direction (Y-Direction), I=1.25 & C_D=3.0					
Story	Allowable Drift	Check Y-Dir.	X-Dir.	Y-Dir.	Total Drift
Roof	2.52	OK	0.016	0.016	0.023
6	2.52	OK	0.026	0.017	0.031
5	2.52	OK	0.035	0.017	0.039
4	2.52	OK	0.044	0.016	0.047
3	2.52	OK	0.060	0.015	0.062
2	3.24	OK	0.098	0.015	0.099
1	3.06	OK	0.185	0.011	0.185

Moment Frame Girders, G2

Girder Design, G2	
1st Floor Section Size:	18x30
1st Floor Reinforcement:	(13) # 8 rebar
2nd Floor Section Size:	18x30
2nd Floor Reinforcement:	(9) # 8 rebar
3rd Floor Section Size:	18x30
3rd Floor Reinforcement:	(7) # 8 rebar
4th Floor Section Size:	18x30
4th Floor Reinforcement:	(5) # 8 rebar
5th Floor Section Size:	18x30
5th Floor Reinforcement:	(4) # 8 rebar
6th Floor Section Size:	18x30
6th Floor Reinforcement:	(3) # 8 rebar
7th Floor Section Size:	18x30
7th Floor Reinforcement:	(3) # 8 rebar
f'c:	4 ksi
fy:	60 ksi

Tributary Area Layout



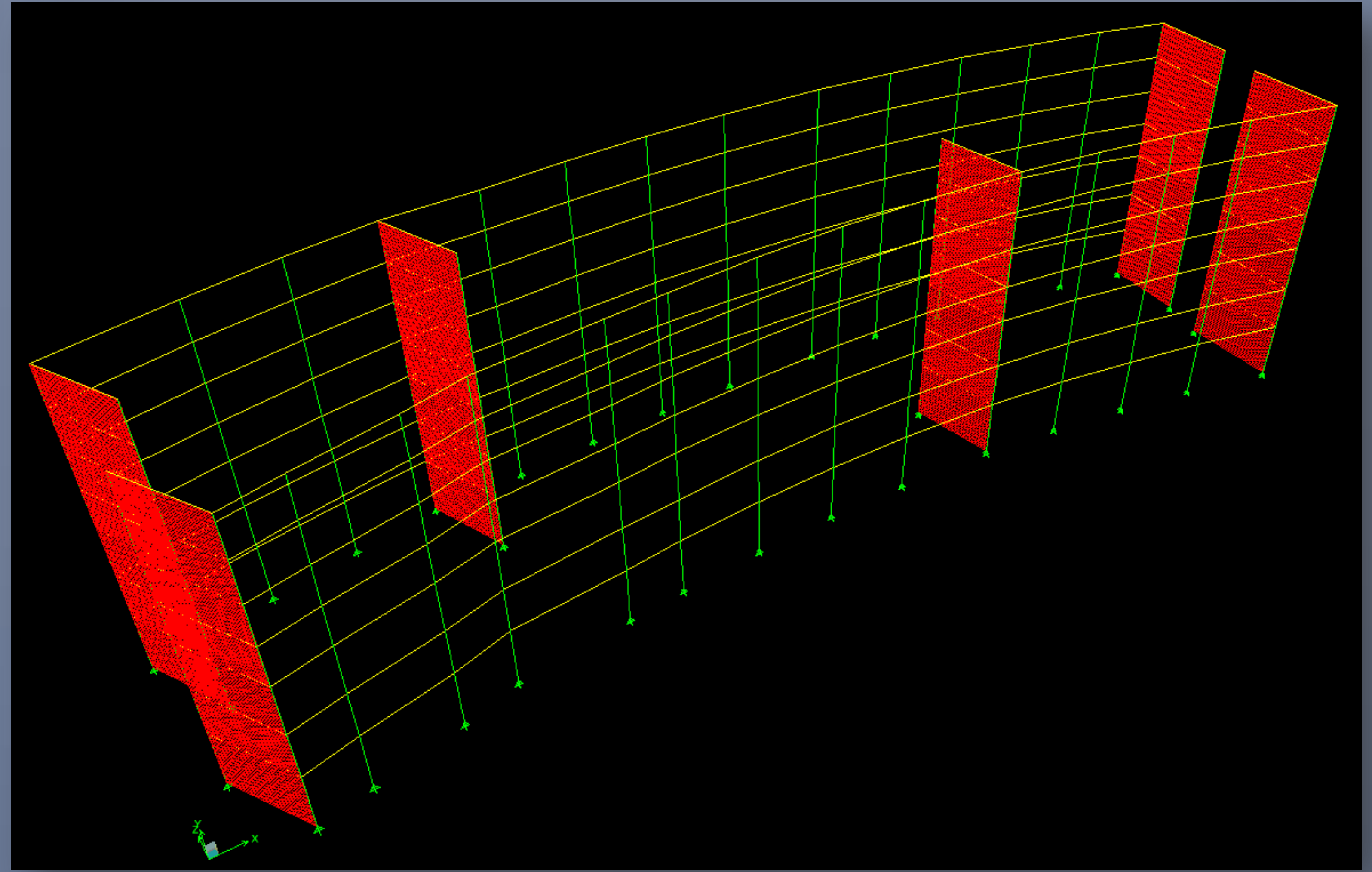
Moment Frame Columns, C2

Column Design, C2	
1st Floor Section Size:	26x26
1st Floor Reinforcement:	(16) # 10 rebar
2nd Floor Section Size:	24x24
2nd Floor Reinforcement:	(12) # 8 rebar
3rd Floor Section Size:	20x20
3rd Floor Reinforcement:	(12) # 8 rebar
4th Floor Section Size:	18x18
4th Floor Reinforcement:	(8) # 8 rebar
5th Floor Section Size:	16x16
5th Floor Reinforcement:	(8) # 8 rebar
6th Floor Section Size:	14x14
6th Floor Reinforcement:	(4) # 8 rebar
7th Floor Section Size:	12x12
7th Floor Reinforcement:	(4) # 8 rebar
f'c:	4 ksi
fy:	60 ksi
ρ:	% < 4.0% O.K.

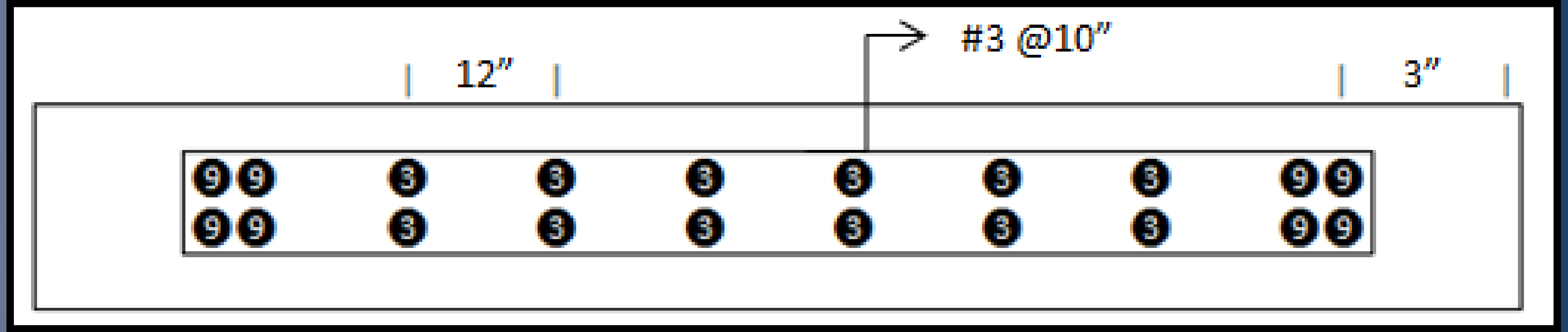
Shear Wall Design

Shear Wall Design	
Horizontal Reinforcement:	#3 rebar spaced at 10"
Vertical Reinforcement:	#3 rebar spaced at 12"
Flexural Reinforcement:	(4) #9
Thickness:	8 inches

Shear Wall Placement



Shear Wall Figure



Cost Analysis

- Costs determined from RS Means
- Accounts for material, labor, & equipment cost
- Location factor of 1.1 for Plainsboro, NJ
- $94,322 saved on bare cost
- $786,922 more expensive for proposed structure with overhead & profit

Original Cost

Existing Steel Structure

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

Total: $5,972,015

Total O&P: $7,030,234

Proposed Cost

Proposed Concrete Structure

	Size	Unit	Material	Labor	Equipment	Total	Total Incl. O&P	Amount	Total(No O&P)	Total (w/ O&P)
4000psi Concrete		C.Y.	$ 103.00	-	-	$ 103.00	$ 113.00	16,740.67	$ 1,724,288.73	$ 1,891,695.40
Concrete Finish	Bull Float	S.F.	-	$ 0.35	-	$ 0.35	$ 0.57	43,842.00	$ 15,344.70	$ 24,989.94
Concrete Slab	6.5" Slab	C.Y.	-	$ 10.95	$ 5.00	$ 15.95	$ 23.50	16,740.67	$ 267,013.64	$ 393,405.68
Slab Reinforcing		Ton	$ 850.00	$ 385.00	-	$ 1,235.00	$ 1,625.00	445.00	$ 549,575.00	$ 723,125.00
Slab Form	4 use	SFCA	$ 1.32	$ 2.48	-	$ 3.80	$ 5.60	306,894.00	$ 1,166,197.20	$ 1,718,606.40
Edge Form	4 use	L.F.	$ 0.12	$ 1.84	-	$ 1.96	$ 3.22	9,639.00	$ 18,892.44	$ 31,037.58
Concrete Beam	10x20	C.Y.	-	$ 19.45	$ 8.90	$ 28.35	$ 42.50	4,142.00	$ 117,425.70	$ 176,035.00
Beam Reinforcing		Ton	$ 800.00	$ 415.00	-	$ 1,215.00	$ 1,600.00	132.00	$ 160,380.00	$ 211,200.00
Beam Form	4 use	SFCA	$ 1.09	$ 3.08	-	$ 4.17	$ 6.38	16,567.00	$ 69,029.17	$ 105,614.63
Concrete Girder	10x20	C.Y.	-	$ 19.45	$ 8.90	$ 28.35	$ 42.50	1,639.00	$ 46,465.65	$ 69,657.50
G1 Reinforcing		Ton	$ 800.00	$ 415.00	-	$ 1,215.00	$ 1,600.00	77.00	$ 93,555.00	$ 123,200.00
G1 Form	4 use	SFCA	$ 1.09	$ 3.08	-	$ 4.17	$ 6.38	7,204.00	$ 30,016.67	$ 45,925.50
Concrete Girder	18x30	C.Y.	-	$ 19.45	$ 8.90	$ 28.35	$ 42.50	4,425.00	$ 125,448.75	$ 188,062.50
G2 Reinforcing		Ton	$ 800.00	$ 415.00	-	$ 1,215.00	$ 1,600.00	310.00	$ 376,650.00	$ 496,000.00
G2 Form	4 use	SFCA	$ 0.91	$ 4.41	-	$ 5.32	$ 8.40	12,968.00	$ 68,989.76	$ 108,931.20
Concrete Column	20x20	C.Y.	-	$ 19.05	$ 8.70	$ 27.75	$ 41.00	384.00	$ 10,656.00	$ 15,744.00
C1 Reinforcing		Ton	$ 1,175.00	$ 510.00	-	$ 1,685.00	$ 2,175.00	94.00	$ 158,390.00	$ 204,450.00
C1 Form	4 use	SFCA	$ 0.62	$ 3.22	-	$ 3.83	$ 6.08	13,347.00	$ 51,163.50	$ 81,194.25
Concrete Column	24x24-14x14	C.Y.	-	$ 15.88	$ 14.50	$ 30.38	$ 34.17	425.00	$ 12,909.38	$ 14,520.83
C2 Reinforcing		Ton	$ 850.00	$ 385.00	-	$ 1,235.00	$ 1,625.00	85.00	$ 104,975.00	$ 138,125.00
C2 Form	4 use	SFCA	$ 0.74	$ 3.86	-	$ 4.60	$ 7.30	16,016.00	$ 73,673.60	$ 116,916.80
Concrete Shear Wall	8" Thick	C.Y.	-	$ 14.25	$ 0.68	$ 14.93	$ 24.50	357.00	$ 5,330.01	$ 8,746.50
Wall Reinforcing		Ton	$ 760.00	$ 281.00	-	$ 1,041.00	$ 1,325.00	94.00	$ 97,854.00	$ 124,550.00
Wall Form	4 use	SFCA	$ 0.59	$ 3.52		$ 4.11	$ 6.55	14,469.00	$ 59,467.59	$ 94,771.95
									$ 5,944,060.63	**$ 7,817,156.22**

Total: $5,878,646

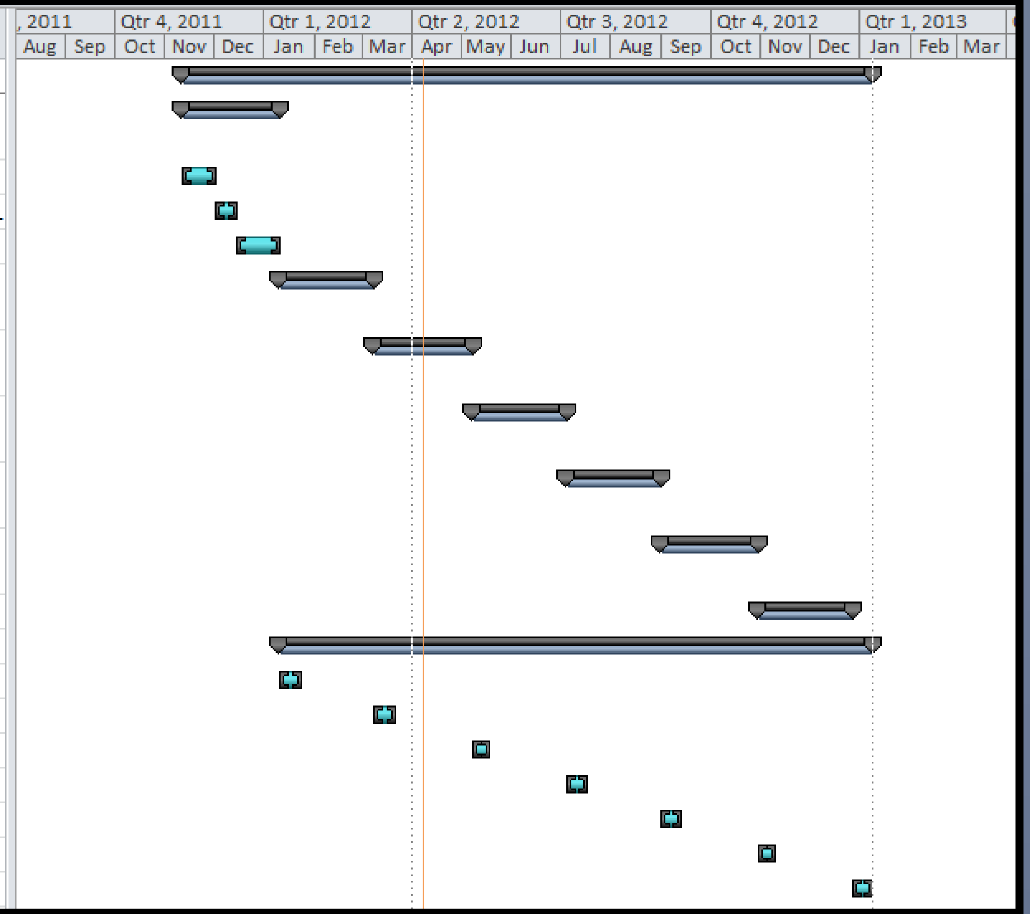
Total O&P: $7,817,156

Schedule Analysis

- Based on RS Means output
- Assumptions
  - Multiple crews per task
  - Ideal construction process
  - 8 day waiting period to construct on top of cured concrete
- The Proposed design takes 140 days longer to erect

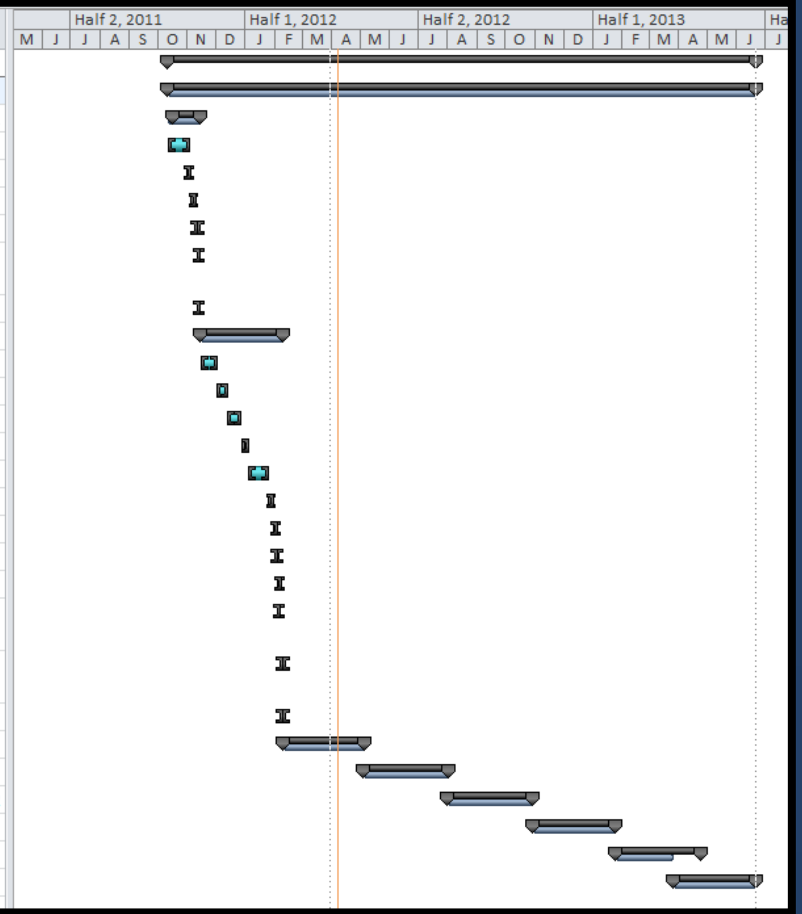
Original Schedule

	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**
10	**3rd Floor Steel Structure**	**43 days**	**Thu 3/8/12**	**Mon 5/7/12**
14	**4th Floor Steel Structure**	**43 days**	**Mon 5/7/12**	**Wed 7/4/12**
18	**5th Floor Steel Structure**	**43 days**	**Wed 7/4/12**	**Fri 8/31/12**
22	**6th Floor Steel Structure**	**43 days**	**Fri 8/31/12**	**Tue 10/30/12**
26	**Roof Steel Structure**	**43 days**	**Tue 10/30/12**	**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
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



Proposed Schedule

	Task Name	Duration	Start	Finish
1	**Concrete SuperStructure**	**444 days**	**Tue 10/11/11**	**Fri 6/21/13**
2	**Concrete SuperStructure**	**444 days**	**Tue 10/11/11**	**Fri 6/21/13**
3	**Ground Floor**	**21 days**	**Mon 10/17/11**	**Mon 11/14/11**
4	Frame Columns	17 days	Tue 10/11/11	Wed 11/2/11
5	Reinforce Columns	5 days	Mon 10/31/11	Fri 11/4/11
6	Frame Walls	5 days	Fri 11/4/11	Thu 11/10/11
7	Reinforce Walls	2 days	Thu 11/10/11	Fri 11/11/11
8	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 & Girde	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**
36	**Third Floor**	**64 days**	**Fri 5/4/12**	**Wed 8/1/12**
49	**Fourth Floor**	**64 days**	**Wed 8/1/12**	**Mon 10/29/12**
62	**Fifth Floor**	**64 days**	**Mon 10/29/12**	**Thu 1/24/13**
75	**Sixth Floor**	**64 days**	**Thu 1/24/13**	**Tue 4/23/13**
88	**Roof**	**64 days**	**Tue 3/26/13**	**Fri 6/21/13**



Conclusion

- More expensive design
- Longer to construct
- Meets all strength and serviceability issues
- Recommendations:
  - Do not use unless certain hospital equipment/procedures need very little vibrations within the room

Acknowledgements

- PSU AE faculty
  - Thanks for pushing me past my yield point, almost to my breaking point, but changing my knowledge and outlook on life and engineering forever
- AE friends
  - If it wasn't for you all, I would never of made it through the AE program
- Family
  - Thanks for all of you support, and standing by all of my decisions

Questions?

I will be glad to answer any questions you have at this time.

Special Thanks To…

- Professor Parfitt
- Professor Holland
- Dr. Linda Hanagan
- Jeff Cerquetti, M. CE, P.E., Vice President Facilities Structures & Coastal Engineering, JMT
- Andy Verrengia, P.E., LEED AP, Project Engineer, Atlantic Engineering Services
- Turner Construction
- The University Medical Center of Princeton University

	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, B2							Tributary Area:	391	ft^2			
	Span:	26.5	feet	Two Row	Yes		Influence Area:	782	ft^2	Live Load Reduction		
	Spaced:	14.75	feet	Reinforcement?			L_L=Max of	0.4				
								.25+15/√(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/β_1	
h:	20	inch	OK		d:	17		a:	0.88	Rectangular Section	c:	1.03
b_{eff}= min	b*16*h_f	1040			ε_t=ε_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					
ΦMn=0.9*$A_s$fy(d-a/2)					Vn=10*(f'c)^(1/2)*b*d							
ΦMn:	294.4	kip-ft	OK		Vn:	126.5	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, B1							Tributary Area:	266	ft^2			
	Span:	18	feet	Two Row	Yes		Influence Area:	531	ft^2	Live Load Reduction		
	Spaced:	14.75	feet	Reinforcement?			L_L=Max of	0.4				
								.25+15/√(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			ε_t=ε_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:	10	inch		
Mu=Wu*Ln^2/8					Vu=Wu*Ln/2							
Mu:	138.5	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:	126.5	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		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)

Span:	29.5	feet	Two Row Reinforcement?	Yes	Tributary Area:	656 ft^2	
Spaced Left:	26.5	feet			Influence Area:	2626 ft^2	Use Live Load Reduction
Spaced Right:	18	feet			L_L=Max of	0.4	
						.25+15/√(K_{LL}*A_T)= 0.54	L_L: 43.42

Spacing,S Max of d_b, 1", 3/4A_b		
d_b:	1.0	inch
3/4A_b:	0.6	inch
1":	1.0	inch
S:	1.0	inch

b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S
b_{min}: 9.71 inch

h_{min}>l/18.5 (ACI 318-08)
h_{min}: 19.14 inch

Total Factored Weight, Wu
Dead Load: Misc. dead + Slab: 116 psf
Wu=1.2D +1.6L: 209 psf

Beam Wieght: 141 plf
Pu=1.2D: 3755 pounds

Try a

b:	10	inch	OK
h:	20	inch	OK

d=h-d_b/2-Cc
d: 17

a=A_s*fy/(.85*f'c*b_{eff})
a: 1.10 Rectangular Section

c=a/β_1
c: 1.30

b_{eff}= min		
	b*16*h_f	1040
	Trib width	318
	.25L	88.5
b_{eff}:	88.5	inch

ε_t=ε_u(d-c)/c
ε_t: 0.04 Φ=0.9

Check Flexure, ΦMn>Mu
Mu=0.107*Ln^2+Pu*Ln/8, continuous + point load
Mu: 398 kip-ft

ΦMn=0.9*$A_s$fy(d-a/2)
ΦMn: 409 kip-ft OK

Cheak Shear, Vn>Vu
Vu=Wu*Ln/2+Pu/2
Vu: 84 Kip

Vn=10*(f'c)^(1/2)*b*d
Vn: 126 Kip OK

Column width: 20 inch

Use Member Size: **10 x 20**
Beam Wieght: 141 plf

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)

Span:	29.5	feet	Two Row Reinforcement?	Yes	Tributary Area:	391 ft^2	
Spaced Left:	26.5	feet			Influence Area:	1564 ft^2	Use Live Load Reduction
Spaced Right:	0	feet			L_L=Max of	0.4	
						.25+15/√(K_{LL}*A_T)= 0.63	L_L: 50.35

Spacing,S Max of d_b, 1", 3/4A_b		
d_b:	1.0	inch
3/4A_b:	0.6	inch
1":	1.0	inch
S:	1.0	inch

b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S
b_{min}: 15.71 inch

h_{min}>l/18.5 (ACI 318-08)
h_{min}: 19.14 inch

Total Factored Weight, Wu
Dead Load: Misc. dead + Slab: 116 psf
Wu=1.2D +1.0W+L: 190 psf

Beam Wieght: 141 plf
Pu=1.2D: 2236 plf

Try a

b:	18	inch	OK
h:	30	inch	OK

d=h-d_b/2-Cc
d: 27

a=A_s*fy/(.85*f'c*b_{eff})
a: 2.05 Rectangular Section

c=a/β_1
c: 2.41

b_{eff}= min		
	b*16*h_f	1872
	Trib width	318
	.25L	88.5
b_{eff}:	88.5	inch

ε_t=ε_u(d-c)/c
ε_t: 0.03 Φ=0.9

Check Flexure, ΦMn>Mu
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load
Mu: 1182 kip-ft Combo Controls

ΦMn=0.9*$A_s$fy(d-a/2)
ΦMn: 1200 kip-ft OK

Cheak Shear, Vn>Vu
Vu=Wu*Ln/2+Pu/2
Vu: 75 Kip

Vn=10*(f'c)^(1/2)*b*d
Vn: 341526 Kip OK

Column width: 26 inch
Etabs Moment: 973 kip-ft

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)

Span:	29.5	feet	Two Row Reinforcement?	Yes	Tributary Area:	391 ft^2	
Spaced Left:	26.5	feet			Influence Area:	1564 ft^2	Use Live Load Reduction
Spaced Right:	0	feet			L_L=Max of	0.4	
						.25+15/√(K_{LL}*A_T)= 0.63	L_L: 50.35

Spacing,S Max of d_b, 1", 3/4A_b		
d_b:	1.0	inch
3/4A_b:	0.6	inch
1":	1.0	inch
S:	1.0	inch

b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S
b_{min}: 11.71 inch

h_{min}>l/18.5 (ACI 318-08)
h_{min}: 19.14 inch

Total Factored Weight, Wu
Dead Load: Misc. dead + Slab: 116 psf
Wu=1.2D +1.0W+L: 190 psf

Beam Wieght: 141 plf
Pu=1.2D: 2236 plf

Try a

b:	18	inch	OK
h:	30	inch	OK

d=h-d_b/2-Cc
d: 27

a=A_s*fy/(.85*f'c*b_{eff})
a: 1.42 Rectangular Section

c=a/β_1
c: 1.67

b_{eff}= min		
	b*16*h_f	1872
	Trib width	318
	.25L	88.5
b_{eff}:	88.5	inch

ε_t=ε_u(d-c)/c
ε_t: 0.05 Φ=0.9

Check Flexure, ΦMn>Mu
Mu=0.107*Ln^2+Pu*Ln/8, Etabs+continuous+point load
Mu: 833 kip-ft Combo Controls

ΦMn=0.9*$A_s$fy(d-a/2)
ΦMn: 841 kip-ft OK

Cheak Shear, Vn>Vu
Vu=Wu*Ln/2+Pu/2
Vu: 75 Kip

Vn=10*(f'c)^(1/2)*b*d
Vn: 341526 Kip OK

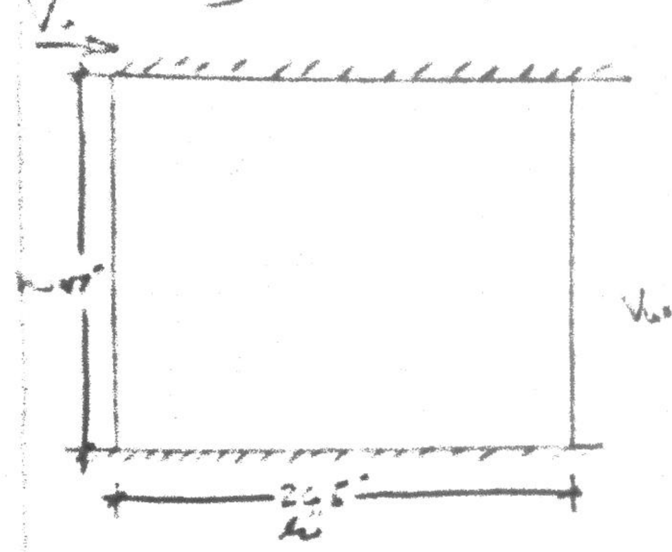
Column width: 26 inch
Etabs Moment: 624 kip-ft

Use Member Size: **18 x 30**
Beam Wieght: 441 plf

	Floor:	All	/	7										
	f'c:	4	ksi	Clear Cover:	2.5	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:	#	10	Bar Diameter, d_b:	1.270	inch					
	Live Load:	80	psf	# of bars, n:	12		Area of Steel, A_b:	1.27	in^2					
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	15.24	in^3					
Column Design, C1		(Gravity)					Tributary Area:	4789	ft^2					
	Span:	30.75	feet	Equal	Yes		Influence Area:	19157	ft^2	Use Live Load Reduction				
	Spaced Left:	26.5	feet	Spacing?			L_L=Max of	0.4						
	Spaced Right:	18	feet	Column Hieght	91	feet		.25+15/√(K_{LL}*A_T)=		0.36	L_L:	28.67	psf	
					7	stories								
Spacing	Max of d_b, 1", 3/4A_b			b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S										
d_b:	1.3	inch		b_{min}:	19.68	inch	Total Factored Weight, Wu							
3/4A_b:	1.0	inch					Dead Load: Misc. dead + Slab				Beam Wieght		Girder Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					116.25	psf	140.63	plf	141	plf
				h_{min}:	19.95	inch		Wu=1.2D +1.6L			Pu=1.2D		Pu=1.2D	
S:	1.3	inch						185	psf		4	Kips	5	Kips
											3755	Pounds		
											Total Pu:	1022	Kips	
Try a														
b:	20	inch	OK		d=h-Cc			a=c*β_1						
h:	20	inch	OK		d:	17.5		a:	8.80					
Wieght:	60	kips												
b_{eff}= min	b*16*h_f	2080			ε_y=fy/E_s			=0.003*d_2/(0.003+ε						
	Trib width	318			ε_y:	0.00207		c:	10.36					
	.25L	92.25												
b_{eff}:	92.25	inch												
Pure Axial, Po				Balanced Strain, Mb & Pb										
Po=0.85*f'c*Ac+As*f				Pb=0.85*f'c*b*a			ε_{s1}=.003(c-d)/c							
Po:	2223	Kips	OK	Pb:	599	Kips	ε_{s1}:	0.00228	OK					
				Mb=ΣMo	8	bars	4	on each side		2	in an inside row		4	in a row
				Mb:	4304	Kip-in	Spaced at:	3	inch					
					359	Kip-ft								
Pure Bending, Mo														
Solve for c from T=C										Mo=ΣM about o			moment arm in ten	
ε_{s1}:	441.96	*(c-2.5)/c	+	ε_{s1}:	0.0014	OK	f_{s1}:	42		Mo:	-1137	Kip-in	1911	Kip-in
ε_{s2}:	220.98	*(c-(2.5+3))/	+	ε_{s2}:	-0.0908	use 60	f_{s2}:	152			-95	Kip-ft	159	Kip-ft
ε_{s3}:	152.4		-	ε_{s3}:	-0.0061	use 60	f_{s3}:	152						
ε_{s4}:	304.8		-	ε_{s4}:	-0.0079	use 60	f_{s4}:	305						
P:	57.8	*c	+	P:	######									
c:	4.8													
Check Flexure, ΦMn>Mu														
Mu=0.107*Ln^2, continuous, Point load														
Mu:	387	kip-ft	N.G.											

	Floor:	1	/	7										
	f'c:	4	ksi	Clear Cover:	2.5	inch								
	fy:	60	ksi	Conc. Wieght:	150	pcf								
	Slab, t:	0	inch	Stirrup Size:	#	3	Stirrup Diameter:	0.357	inch					
	Misc. Dead Load:	35	psf	Bar Size:	#	10	Bar Diameter, d_b:	1.270	inch					
	Live Load:	80	psf	# of bars, n:	16		Area of Steel, A_b:	1.27	in^2					
	β_1:	0.85		ε_u:	0.003		Area of Steel, A_s:	20.32	in^3					
Column Design, C2		(Lateral)					Tributary Area:	2736	ft^2					
	Span:	29.5	feet	Equal	Yes		Influence Area:	10945	ft^2	Use Live Load Reduction				
	Spaced Left:	26.5	feet	Spacing?			L_L=Max of	0.4						
	Spaced Right:	0	feet	Column Hieght	91	feet		.25+15/√(K_{LL}*A_T)=		0.39	L_L:	31.47	psf	
	Column Hieght:	17	feet		7	stories above								
Spacing	Max of d_b, 1", 3/4A_b			b_{min}=2*Cc + n*d_b + 2d_{st} + (n-1)*S										
d_b:	1.3	inch		b_{min}:	17.14	inch	Total Factored Weight, Wu							
3/4A_b:	1.0	inch					Dead Load: Misc. dead + Slab				Beam Wieght		Girder Wieght	
1":	1.0	inch		h_{min}>l/18.5 (ACI 318-08)					35	psf	140.63	plf	141	plf
				h_{min}:	19.14	inch		Wu=1.2D +1.0W+L			Pu=1.2D		Pu=1.2D	
S:	1.3	inch						73	psf		4	Kips	5	Kips
											3755	Pounds		
											Total Pu:	339	Kips	
Try a														
b:	26	inch	OK		d=h-Cc			a=c*β_1						
h:	26	inch	OK		d:	23.5		a:	11.82					
Wieght:	0.704	kips												
b_{eff}= min	b*16*h_f	0			ε_y=fy/E_s			=0.003*d_2/(0.003+ε						
	Trib width	318			ε_y:	0.00207		c:	13.91					
	.25L	88.5												
b_{eff}:	0	inch												
Pure Axial, Po				Balanced Strain, Mb & Pb										
Po=0.85*f'c*Ac+As*f				Pb=0.85*f'c*b*a			ε_{s1}=.003(c-d)/c							
Po:	3449	Kips	OK	Pb:	1045	Kips	ε_{s1}:	0.00246	OK					
				Mb=ΣMo	16	bars	5	on each side		2	in an inside row		5	in a row
				Mb:	8818	Kip-in	Spaced at:	3	inch					
					735	Kip-ft								
Pure Bending, Mo														
Solve for c from T=C										Mo=ΣM about o				
ε_{s1}:	552.45	*(c-2.5)/c	+	ε_{s1}:	0.0014	OK	f_{s1}:	40		Mo:	-2114	Kip-in	2763	Kip-in
ε_{s2}:	220.98	*(c-(2.5+3))/	+	ε_{s2}:	-0.0942	use 60	f_{s2}:	152			-176	Kip-ft	230	Kip-ft
ε_{s3}:	152.4	mid-point		ε_{s3}:	-0.0103	use 60	f_{s3}:	152						
ε_{s4}:	152.4		-	ε_{s4}:	-0.0122	use 60	f_{s4}:	305						
ε_{s5}:	381		-	ε_{s5}:	-0.0122	use 60	f_{s5}:	381						
P:	75.14	*c	+	P:	######									
c:	4.63													
Check Flexure, ΦMn>Mu									Etabs Moment	973	kip-ft			
Mu=0.107*Ln^2, continuous, Point load, ETABs														
Mu:	1064	kip-ft	N.G.											

Alex Burg | Proposed Solution | Shear Wall design 1/4



V_u Emos = ... $V_u = 252^k$

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

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

Try Anderson 8":

$\phi V_n = 965^k$ ∴ ✓

Check Shear Strength provided by V_c

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

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

t = 8": $V_c = \underline{257}^k$

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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 Reinforcement

t = 8"

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

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

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

t = 8": $V_s = 79^k$

t = 12": $V_s = \frac{A_v \cdot f_y \cdot d}{s} \Rightarrow \frac{A_v}{s} = \frac{V_s}{f_y d} = -\frac{50}{60(21.2 \times 12)} = 0.00327$

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"

$\rho_t = \frac{2(.011)}{10(8)} = 0.00275 > 0.0025$ ∴ OK

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Max Spacing:

s_s | $l_w/5 = (26.5 \times 12)/5 = 63.6"$
$3t = 3(8) = 24"$
min $18"$ ← governs

$10" < 18"$ ✓

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

Vertical Shear Rienforcement

$\rho_l = \frac{A_v}{s \cdot h} \ge 0.0025 + 0.5(2.5 - h_w/l_w)(\rho_t - 0.0025)$
$= 0.0025 + 0.5(2.5 - 17/26.5)(0.00275 - 0.0025)$
$= 0.0027 > 0.0025$ ∴ ✓OK

Max spacing: | $l_w/3 = 26.5(12)/3 = 106"$
$3(t) = 24"$
min $18"$ ← governs

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

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

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

Design for Flexure:

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

★Assume Tension Controlled★

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

$C = T \Rightarrow 0.85 f'_c\, a\, b = A_s f_y$

$jd = .9d = (.9)(21.2)(12) = 229"$

$M_u \le \phi M_n = \phi A_s f_y\, jd$

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

$A_s = 4.16\ in^2$

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

$jd = d - a/2$

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

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

$A_s = 3.8\ in^2$

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

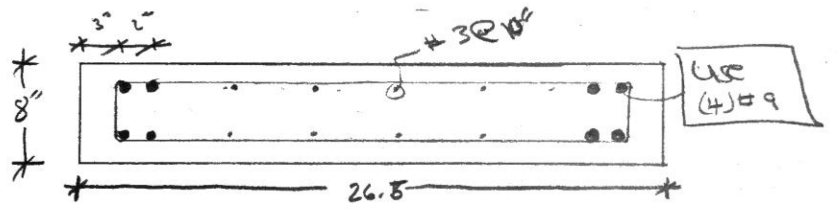
$C = T; \quad 0.85 f'_c ab = A_s f_y$

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

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

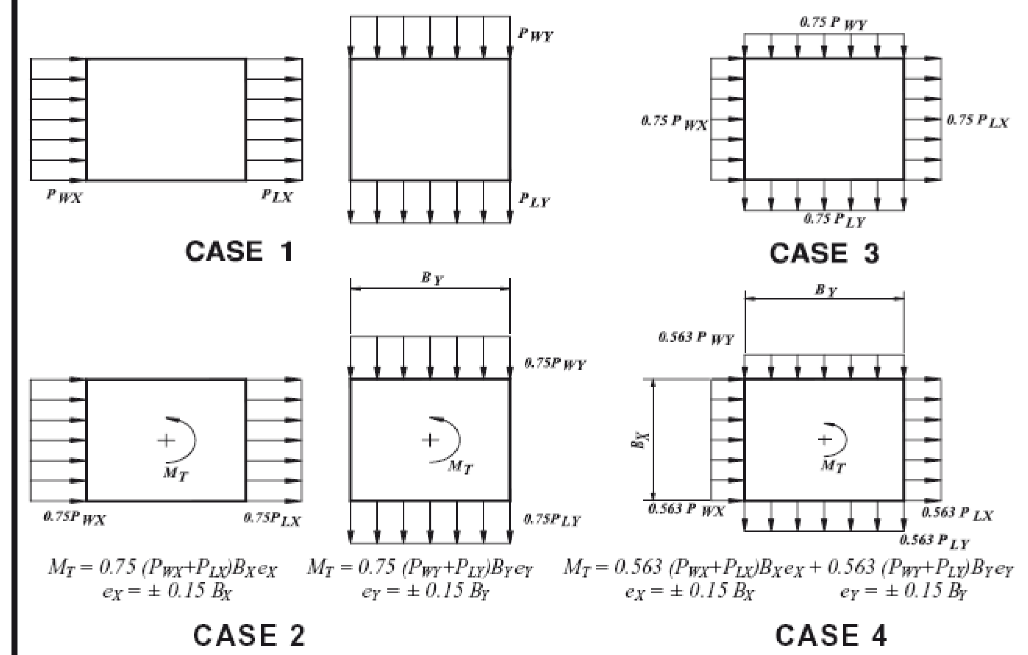
$\varepsilon_t = \varepsilon_u \left(\dfrac{d-c}{c}\right) = 0.003\left(\dfrac{(21.2 \times 12) - 10.4}{10.4}\right)$

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



AMPAD

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.