Executive Tower
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Structural

Technical Assignment 1
Structural Concepts/ Existing Conditions

Executive Summary

This document is an analysis of the current conditions and structural concepts used in the designing of the structural system for the Executive Tower. In this report, a thorough description of the Dead Loads calculated and Live Loads used from the District of Columbia Building Code Supplement 2003 and ASCE7-02 and their uses in determining reactions due wind and seismic forces.

The Initial design of the Executive Tower was to utilize the most available space by using a flat plate concrete system minimizing the floor to ceiling thicknesses. The typical slab is 8" cast in place concrete with additional 10'x8'x8" drop panels at column locations. Through the use of this system the designers were able to construct a 12 story Class A office building with 9 foot ceilings under the buildings restricted height of 160 feet.

Spot checks of the analysis have been performed and can be found in Appendix A, B, C, D of the following systems, respectively: Slab thickness and reinforcement, exterior beam including weight from curtain wall, exterior column including worse case scenario from wind and seismic forces, and a minimal shear wall analysis.

Overview

The Executive Tower quietly rests in it elegance at the intersection of New York and 14th St. The Executive Tower is a class A office structure in the downtown area of N.W. Washington DC located only two blocks from the White House. This 12 story building houses such firms as Bloomberg Financial, Merrill Lynch, and AIG satisfying their needs by supplying permanent office and open plan floors. The Executive Tower was completed in December of 2000. Since then, it has remained one of the highest rental rates per square foot of office space in all of Washington DC. The Executive Tower's prestige is most noted from atop its roof that

Figure 1.1 – Viewing terrace of Executive Tower with Washington Monument in Background

overlooks the City of Washington DC as seen in figure 1.1.

Architecture

The Executive Tower was designed by the international group Hellmuth, Obata + Kassabaum, Inc. (HOK). The Main architectural challenge was designing a building that would fit in a non rectangular lot. The designers skewed the south wall to be aligned with New York Ave and use the south section of the west wall to create the building's trademarked curved

façade. To keep this portion of the façade separate from the rest of the building, the designer extended the roof an extra floor, which created the viewing terrace, and keeping this section bear of the granite façade found on the first two floors. Capitols are used at the top of the building to generate a sense of closure to the building's slender feel. Again to keep the curved façade separate the capitol is discontinued across this section at the 10^{th} floor and 11^{th} floor ceiling and a large capitol is constructed to complete the structure at the roof of the view area along the curved façade.

Structural Systems

Floor system

The floor system of the Executive Towers is a two-way flat plate concrete slab, a typical systems used in and around the DC area to allow a maximum number of floors to be constructed in a region with specific height restrictions. The typical thickness for this slab is 8" reinforced with #4 at 12" O.C. The slab around the exterior of the building has an additional 3½" thickness acting as wide exterior beams. Drop Panels at interior and exterior column locations of 10'x8'x8" allow of for the thinner slabs across the longer span.

Column

The columns of the Executive Tower consist of all cast in place concrete, mostly rectangular spread out variably throughout the floor system as seen in figure 2.1. The flat plate concrete slab allows the column location to be irregular and having a typical bay is virtually non-existent in the Executive Tower. However, they typical column consists of 20"x20" with roughly 6 #10 bars of reinforcement.

Foundation

A MAT Foundation is utilized to maximize ground contact and distribution of the buildings loads. An additional 13'x13'x1' spread footings at column locations. The MAT is a 42" thick slab fully reinforced with #10@12" O.C. each way bottom steel and #7@12" O.C. each way top steel. Sheeting and shoring is placed on the north, south and west side of building and underpinning is required only on the east side.

Lateral Resistance

The lateral resisting system consists of six shear walls forming the enclosure of the elevator shafts in the center of the building. The shear walls are all 12" thick extending the height of the building and is reinforced with #6@8" horizontal steel through the height of the building.

Codes

District of Columbia Building Code Supplement 2003

Standards

IBC 2003
ASCE7-02

Loadings

Live Loads

• Office + partitions	80 + 20 = 100 psf
• Lobby	100 psf
• Mechanical	150 psf
• terrace (Viewing Area)	100 psf
• Roof	30 psf
• Corridor	100 psf
• Corridor above 1st floor	80 psf
• Parking	40 psf
• Stairs	100 psf

Dead Loads

Parking

8" reinforced Concrete slab	100 psf
Sprinklers	5 psf
MEP ducts	5 psf
	110 psf

Floors 1st – Roof

8" reinforced Concrete slab	100 psf
Sprinklers	5 psf
MEP ducts	5 psf
Finishes	10 psf
	120 psf

Curtain Wall

1st floor, tributary area = 11'-0"

6" Granite (trib = 50")	(98 pcf)*(6"x50") = 200 pLf
½" Glass	(160 pcf)*(.5"x132") = 73.3 pLf
	~275 pLf

2nd Floor, tributary area = 9'-0"

6" Granite Panels (trib = 50")	(98 pcf)*(6"x50") = 200 pLf
½" Glass	(160 pcf)*(.5"x108") = 60 pLf
	~260 pLf

3rd Floor – Penthouse, tributary area = 9'-0"

6" Precast Panel (trib = 50")	(150 pcf)*(6"x45") = 280 pLf
½" Glass	(160 pcf)*(.5"x108") = 60 pLf
	340 pLf

Snow Loads

Expose "B" p_g = 25 psf

C_e = 1.0 table 7-2

C_t = 1.0 table 7-3

I = 1.0 table 7-4

$P_f = .7C_eC_tLp_g$

$S_L = P_f$ = 17.5 psf

Seismic

Location – DC Category II Site Class D

S_s = 18.0%g Fig 9.4.1.1a seismic group I

S_1 = 6.2%g Fig 9.4.1.1b Importance factor = 1.0

S_{ms} = FaSs F_a = 1.6

S_{m1} = FvS1 F_v = 2.4

S_{ms} = (1.6)(.18) = .288

S_{m1} = (2.4)(.062) = .1488

S_{Ds} = 2/3 S_{ms} = .192 Seismic Design Category - D

S_{D1} = 2/3 S_{m1} = .099

$V = C_sW$

C_s (min) = .044IS_{Ds} = .0084 R = 5

= SDs/(R/I) = .0384 I = 1.0

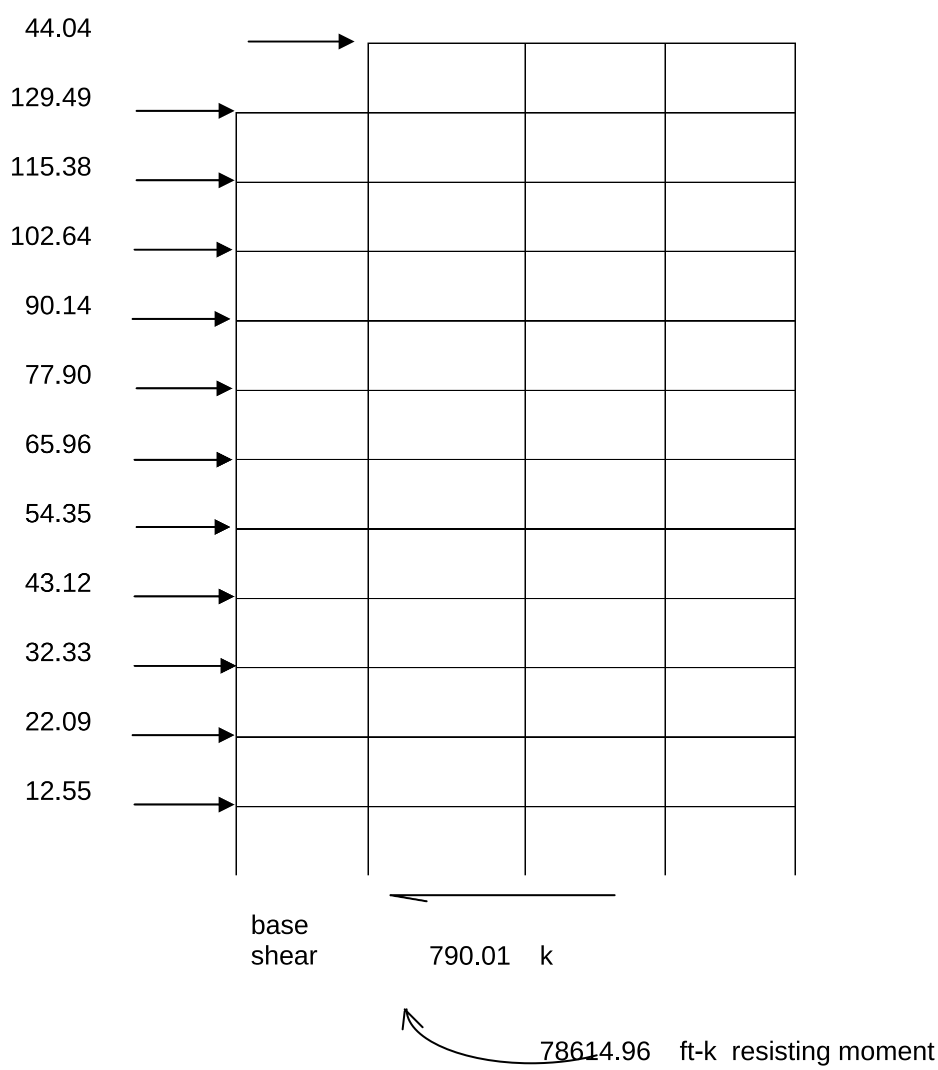
(max) = SD1/(T(R/I)) = .0227 ← controls $T = C_Th_{nx}$

T = .871

See spread sheet 4.1 for seismic loadings, base shear and over turning moment

SEISMIC k interpolation 1.1855 V= 790.0054

	ht (ft)	load (k)	$W*ht^k$	C_{vx}		story force (k) =VCs
roof	153.33	900	350997	0.0557		44.04
penthouse/roof	134.83	3082	1032035	0.1639		129.49
11	122.33	3082	919609	0.1461		115.38
10	110.83	3082	818039	0.1299		102.64
9	99.33	3082	718409	0.1141		90.14
8	87.83	3082	620899	0.0986		77.90
7	76.33	3082	525736	0.0835		65.96
6	64.83	3082	433205	0.0688		54.35
5	53.33	3082	343683	0.0546		43.12
4	41.83	3082	257695	0.0409		32.33
3	30.33	3082	176032	0.0280		22.09
2	18.83	3082	100039	0.0159		12.55
		sum	sum	sum check		base shear
base	0	34802	6296377	1	ok	790.01



Wind Loads

Exposure B case 2

K_{zt} = 1.0 (no hill)

K_d = .85

V = 90 mph

I = 1.0

$q_z = 0.00256K_zK_{zt}K_dV^2I$

Windward $p_w = q_zGC_p$ for windward C_p = .8

Leeward $p_L = q_hGC_p$ for leeward NS Cp = -.5

EW Cp = -.3

<u>NS G Calculation</u>

$T_a = C_th^x$ (9.5.5.3.2-1)

C_t	0.02
h	153.33
x	0.75
T_a	0.871465798
f	1.147491963

V	90	F 6-1
β	0.05	
$V_z = b(z/33)^{\alpha}V(88/60)$ =	68.7694659	
n_1 = f =	1.14749196	
$\eta_h = 4.6n_1h/V_z$ =	11.7689839	
$\eta_B = 4.6n_1B/V_z$ =	7.06154385	
$\eta_L = 15.4n_1L/V_z$ =	28.7801295	

$R_i = (1/\eta_i)-(1/(2\eta^2))(1-e^{-2\eta})$

R_h	0.08135923
R_B	0.13158511
R_L	0.03414255

flexible buildings (f < 1.0 Hz) (6.5.8): G_f use

Table 6-2

z_{min}	30	
c	0.30	
ℓ	320	
ε	0.33	
b	0.45	
α	0.142857143	
L	112	
B	92	
g_Q	3.4	
g_v	3.4	
z = 0.6h =	91.998	z>30?
$L_z = \ell(z/33)^{\varepsilon}$ =	450.372081	TRUE
$I_z = c(33/z)^{1/6}$ =	0.252877689	91.998

$Q = (1/(1+0.63((B+h)/L_z)^{0.63}))^{1/2}$ = 0.83633898

$N_1 = n_1L_z/V_z$ = 7.51493903

$R_n = 7.47N_1/(1+10.3N_1)^{5/3}$ = 0.03908561

R = $((1/\beta)R_nR_hR_B(0.53+0.47R_L))^{1/2}$ = 0.06759977

$g_R = (2\ln(3600n_1))^{1/2}+0.577/(2\ln(3600n_1))^{1/2}$ = 4.222147352

$G_f = 0.925(((1+1.7I_z(g_Q^2Q^2+g_R^2{}_R^2)^{1/2})/(1+1.7g_vI_z))$ = 0.837420031

EW G Calculation

flexible buildings (f < 1.0 Hz) (6.5.8): G_f

Table 6-2

z_{min}	30
c	0.3
ℓ	320
ε	0.3333
b	0.45
α	0.142857143

L	92
B	112
g_Q	3.4
g_v	3.4

$z = 0.6h =$ 91.998 z>30?

$L_z = \ell(z/33)^{\varepsilon} =$ 450.3566897 TRUE

use

$I_z = c(33/z)^{1/6} =$ 0.252877689 91.998

$Q = (1/(1+0.63((B+h)/L_z)^{0.63}))^{1/2}$ = 0.83004713

V	90	F 6-1
β	0.05	

$V_z = b(z/33)^{\alpha}V(88/60) =$ 68.7694659

$n_1 = f =$ 1.14749196

$\eta_h = 4.6n_1h/V_z =$ 11.7689839

$\eta_B = 4.6n_1B/V_z =$ 8.59666207

$\eta_L = 15.4n_1L/V_z =$ 23.6408207

$R_i = (1/\eta_i)-(1/(2\eta^2))(1-e^{-2\eta})$

R_h	0.08135923
R_B	0.10955856
R_L	0.04140508

$N_1 = n_1L_z/V_z$ = 7.51468221

$R_n = 7.47N_1/(1+10.3N_1)^{5/3}$ = 0.03908647

$R = ((1/\beta)R_nR_hR_B(0.53+0.47R_L))^{1/2}$ = 0.06187611

$g_R = (2\ln(3600n_1))^{1/2}+0.577/(2\ln(3600n_1))^{1/2} =$ 4.222147352

$G_f = 0.925(((1+1.7I_z(g_Q{}^2Q^2+g_R{}^2{}_R{}^2)^{1/2})/(1+1.7g_vI_z)) =$ 0.833605419

<u>Story Pressures</u>

k_z ht (ft)	k_z	q_z	N/S $P_w + P_L$ (psf)	N/S W_w (psf)	E/W $P_w + P_L$(psf)	E/W W_w (psf)
160	1.13	19.917	18.35	1687.9	21.58	2417.4
140	1.09	19.212	17.87	1644.4	21.11	2364.7
120	1.04	18.331	17.28	1590.1	20.53	2298.9
100	0.99	17.449	16.69	1535.8	19.94	2233.1
90	0.96	16.921	16.34	1503.2	19.59	2193.6
80	0.93	16.392	15.98	1470.6	19.23	2154.1
70	0.89	15.687	15.51	1427.2	18.76	2101.4
60	0.85	14.982	15.04	1383.7	18.29	2048.8
50	0.81	14.277	14.57	1340.2	17.82	1996.1
40	0.76	13.395	13.98	1285.9	17.23	1930.3
30	0.70	12.338	13.27	1220.8	16.53	1851.3
25	0.66	11.633	12.80	1177.3	16.06	1798.6
20	0.62	10.928	12.32	1133.8	15.59	1746.0
0-15	0.57	10.047	11.73	1079.5	15.00	1680.1

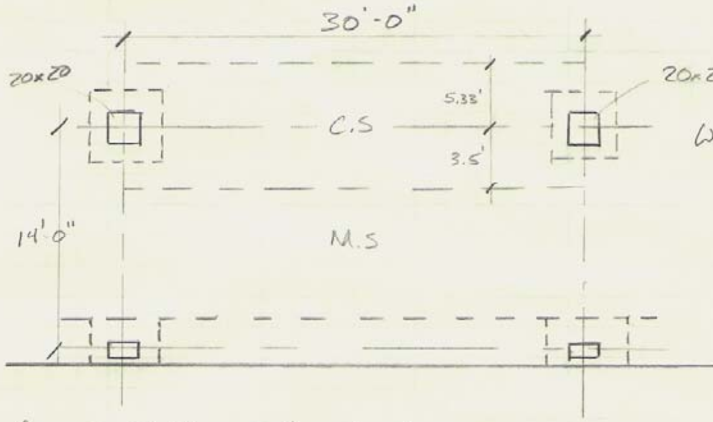
		q_z ht	Trib Area (ft)	N/S (pLf)	E/W (pLf)	story force N/S (lbs)	story force E/W (lbs)
roof	153.33		9.25	1687.9	2417.4	15613.1	22361.0
	144.08			1687.9	2417.4		
penthouse/		140	15.5	1644.4	2364.7	25665.7	36867.9
roof	134.83			1644.4	2364.7		
	128.58			1644.4	2364.7	19547.1	28151.4
11	122.33		12	1644.4	2364.7		
		120		1590.1	2298.9		
	116.58			1590.1	2298.9	18286.2	26437.4
10	110.83		11.5	1590.1	2298.9		
	105.08			1590.1	2298.9		
		100	11.5	1535.8	2233.1	17937.5	26014.9
9	99.33			1535.8	2233.1		
	93.58			1535.8	2233.1		
		90	11.5	1503.2	2193.6	17403.5	25367.8
8	87.83			1503.2	2193.6		
	82.08			1503.2	2193.6		
		80	11.5	1470.6	2154.1	16979.7	24854.3
7	76.33			1470.6	2154.1		
	70.58			1470.6	2154.1		
		70		1427.2	2101.4	16398.0	24148.3
6	64.83		11.5	1427.2	2101.4		
		60		1383.7	2048.8		
	59.08			1383.7	2048.8	15807.3	23433.7
5	53.33		11.5	1383.7	2048.8		
		50		1340.2	1996.1		
	47.58			1340.2	1996.1	15199.4	22697.2
4	41.83		11.5	1340.2	1996.1		
		40		1285.9	1930.3		
	36.08			1285.9	1930.3	14416.7	21748.1
3	30.33		11.5	1285.9	1930.3		
		30		1220.8	1851.3		
		25		1177.3	1798.6		
	24.58			1177.3	1798.6		
		20		1133.8	1746.0	17084.6	26342.5
2	18.83		15.16	1133.8	1746.0		
		15		1079.5	1680.1		
base	9.42		9.42	1079.5	1680.1	10168.9	15826.5

base shear (kips) 220.5 324.3

over turning moment (ft-k)
18179.49 26402.88

AMPAD 22-141 50 SHEETS 22-142 100 SHEETS 22-144 200 SHEETS

Spot check slab thickness + reinforcement



LL = 100 psf
DL = 120 psf

$\omega_u = 1.2(120\,psf) + 1.6(100\,psf)$

$\omega_u = 304\,psf$

$l_n = 30'(12") - 20" = 340"$
$l_1 = 30'(12") = 360"$
$l_2 = 14'(12") = 168"$

static Moment

$M_o = \frac{\omega_u l_2 l_n^2}{8} = \frac{(.304\,ksf)(14')(28.33')^2}{8} = 427.0^k$

Location	Strip	Moment	width	mom/width
int support 70% 298.9^k	Col strip (50%)	149.5^{k}	8.83'	16.9 $^{k/ft}$
	Mid strip (50%)	149.5^k	10.66'	14.0 $^{k/ft}$
ext support 30% 128.1^k	col strip (100%)	128.1^k	8.83'	14.5 $^{k/ft}$
	mid strip (0%)	0	—	

Spec's use #4 @ 12" w/ additional 6#5 for col strip
& 10 #5 for mid strip

col strip

$A_{s/ft} = .31 + .2$

$A_{s/ft} = .51\,in^2/ft$

$d = 8 - .75 - .31/2 = 7.095$

$a = \frac{(.51)(60\,ksi)}{.85(4\,ksi)(12")} = .75"$

$\phi M_n = .9(.51\,in^2/ft)(60\,ksi)(7.10" - \frac{.75}{2})$

$\phi M_n = 15.43^{k/ft} < 16.9$

off due to different loading and assumption during analysis mid strip

$A_{s/ft} = .2 + .31$

$= .51\ in^2/ft$

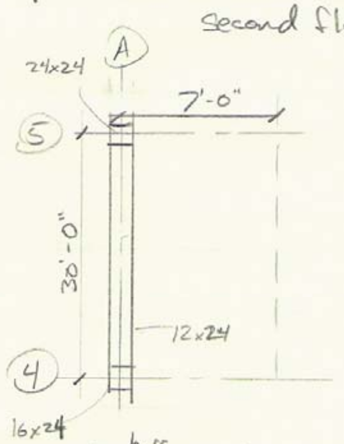
$d = 8 - .75 - .31/2 = 7.09$

$$a = \frac{(.51\ in^2/ft)(60ksi)}{.85(4)(12'')} = .75$$

$\phi M_n = .9(.51)(60)(7.10'' - .75/2)$

$\phi M_n = 15.43\ ^{ik/ft} > 14.0\ ^{ik/ft}$ ok

spot check for Beam
second floor



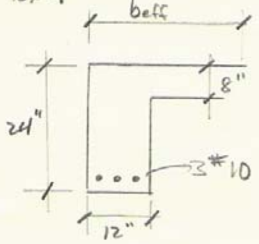
$DL = 120(7') + 150(16 \times 12")/144$

$DL = 1040\ plf + 260\ plf = 1300$

$LL = 100(7') = 700\ plf$

$w_u = 1.2(1.30) + 1.6(.7)$

$w_u = 2.68^k$



$b_{eff} = 1/12(30)(12") = 30"$ ← controls

$= 6(8") = 48"$

$A_s = 3.81 in^2$

$d = 24 - .5 - 1.0 - 1.27/2 = 21.86"$

$M_u = \frac{w_u l^2}{8} = 301.5$

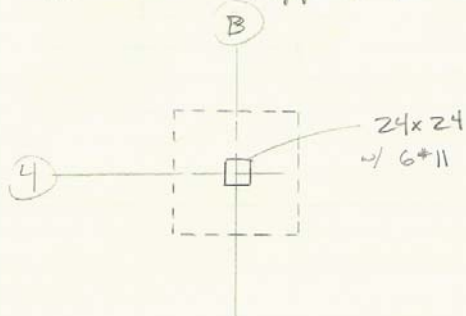
$a = \frac{(3.81 in^2)(60)}{.85(4 ksi)(30)} = 2.24 < 8"$ ok

$\phi M_n = .9(3.81)(60)(21.86 - \frac{2.24}{2})$

$\phi M_n = 355.6'^k > 301.5'^k$ ok

spot check column — 6th floor — supports 6 floors total

trib Area ≈ 500 ft²



DL = 145 psf
LL = 100 psf

Live load reduction $K_{LL} = 4$

$A_T = (500\ ft^2)(6\ floor)$
$A_T = 3000\ ft^2$

$$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) > .4L_o$$

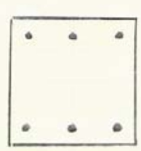
$$L = 100\left(.25 + \frac{15}{\sqrt{4(3500)}}\right)$$

$$L = 37.7\ psf > .4(L_o) = 40\ psf \leftarrow \text{controls}$$

$$W_u = 1.2(145\ psf) + 1.6(40\ psf) = 238\ psf$$

$$P_u = (238\ psf)(3500\ ft^2)$$

$$= 833000 \Rightarrow 833^k$$



$A_s = 9.36\ in^2$

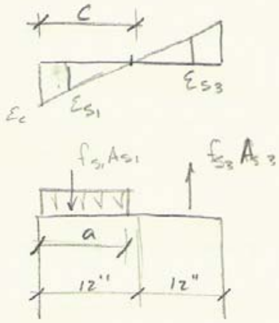
a) Axial strength, P_o
b) Balanced strain strength, P_b, M_b
c) Pure bending, M_o
d) $C = h$
e) $\varepsilon_t = .005$

a) $P_o = .85 f'c\,(A_g - A_s) + A_s f_y$

$= .85(4)(24 \times 24 - 9.36) + 9.36(60)$

$= 2488.2^k$

b) strain



$\varepsilon_y = \frac{60}{29000} = .00207$

$c = \frac{.003}{.003 + .00207}(21.5) = 12.72"$

$\varepsilon_{s1} = \frac{.003}{12.72}(12.72 - 2.5) = .00241$

$f_{s1} = 60\,ksi$

$\varepsilon_{s2} = \frac{.003}{12.72}(12.72 - 12) = .00017$

$f_{s2} = 4.9\,ksi$

$\varepsilon_{s3} = \frac{.003}{12.72}(12.72 - 21.5) = -.00207$

$f_{s3} = -60\,ksi$

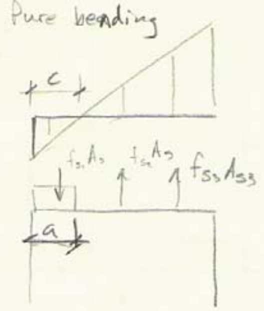
$P_b = .85(4)(24)(.85)(12.72) - 2(60) + 2(4.9) + 2(60)$

$= 892.1^k$

$M_b = 882.3\left(12 - \frac{.85(12.72)}{2}\right) + 2(60)(12 - 2.5) + 2(4.9)(12-12) + 2(60)(12 - 21.5)$

$= 674.8^{'k}$

c) Pure bending



$f_{s1} = \frac{.003}{c}(c - 2.5)(29000)$

$f_{s2} = \frac{.003}{c}(c - 12)(29000)$

$f_{s3} = -60\,ksi$

$\Sigma F = 0 = .85(4)(24)(.85)c + 2\frac{.003}{c}(c - 2.5)(29000) + 2\frac{.003}{c}(c - 12)(29000) + 2(-60)$ yeilded

~~$c = 4.61"$~~

new $c = 3.03"$

$f_{s1} = 39.8\,ksi$

$f_{s2} = -139.5$ no good $\Rightarrow -60\,ksi$

$\varepsilon_{s2} = -.008$

$\varepsilon_{s3} = -.018$

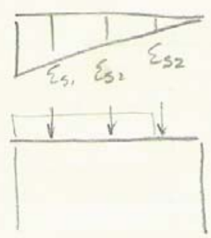
$< -.00172$ ok

$M_o = .85(4)(24)(.85)(3.03)\left(12 - \frac{.85(3.03)}{2}\right) + 2(39.8)(12 - 2.5) + 2(-60)(12-12) + 2(-60)(12 - 21.5)$

$= 345.6^{'k}$

d)

strain



$c = 24$

$\varepsilon_1 = \frac{.003}{24}(24-2.5) = .0027$

$f_{s1} = 60$ ksi

$\varepsilon_{s2} = \frac{.003}{24}(24-12) = .0015$

$f_{s2} = 43.5$ ksi

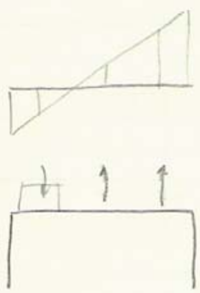
$\varepsilon_{s3} = \frac{.003}{24}(24-21.5) = .00031$

$f_{s3} = 9.1$ ksi

$P_n = .85(4)(24)(.85)(24) + 2(60)$
$+ 2(43.5) + 2(9.1)$
$= 1889.8^k$

$M_n = 1664.6(12 - \frac{.85(24)}{2}) + 2(60)(12-2.5) + ~~2(43.5)(12-12)~~$
$+ 2(9.1)(12-21.5)$
$= 330.3'^k$

e) $\varepsilon_t = \varepsilon_3 = .005$



$c = \frac{.003}{.003+.005}(21.5) = 8.06''$

$\varepsilon_{s1} = \frac{.003}{8.06}(8.06-2.5) = .0021$

$f_{s1} = 60$ ksi

$\varepsilon_{s2} = \frac{.003}{8.06}(8.06-12) = -.0015$

$f_{s2} = -42.5$ ksi

$\varepsilon_{s3} = \frac{.003}{8.06}(8.06-21.5) = -.005$

$f_{s3} = -60$ ksi

$P_n = .85(4)(24)(.85)(8.06) + ~~2(60)~~ + 2(-42.5) + ~~2(-60)~~$
$= 474.0^k$

$M_n = 559.0(12 - \frac{.85(8.06)}{2}) + 2(60)(12-2.5) + ~~2(-42.5)(12-12)~~$
$+ 2(-60)(12-21.5)$

$= 589.4'^k$

$\ell_1 = 14$
$\ell_2 = 27$

$\ell_x = 30'$

$W_u = 1.2(145) + 1.6(100)$

$W_u = 334$ psf

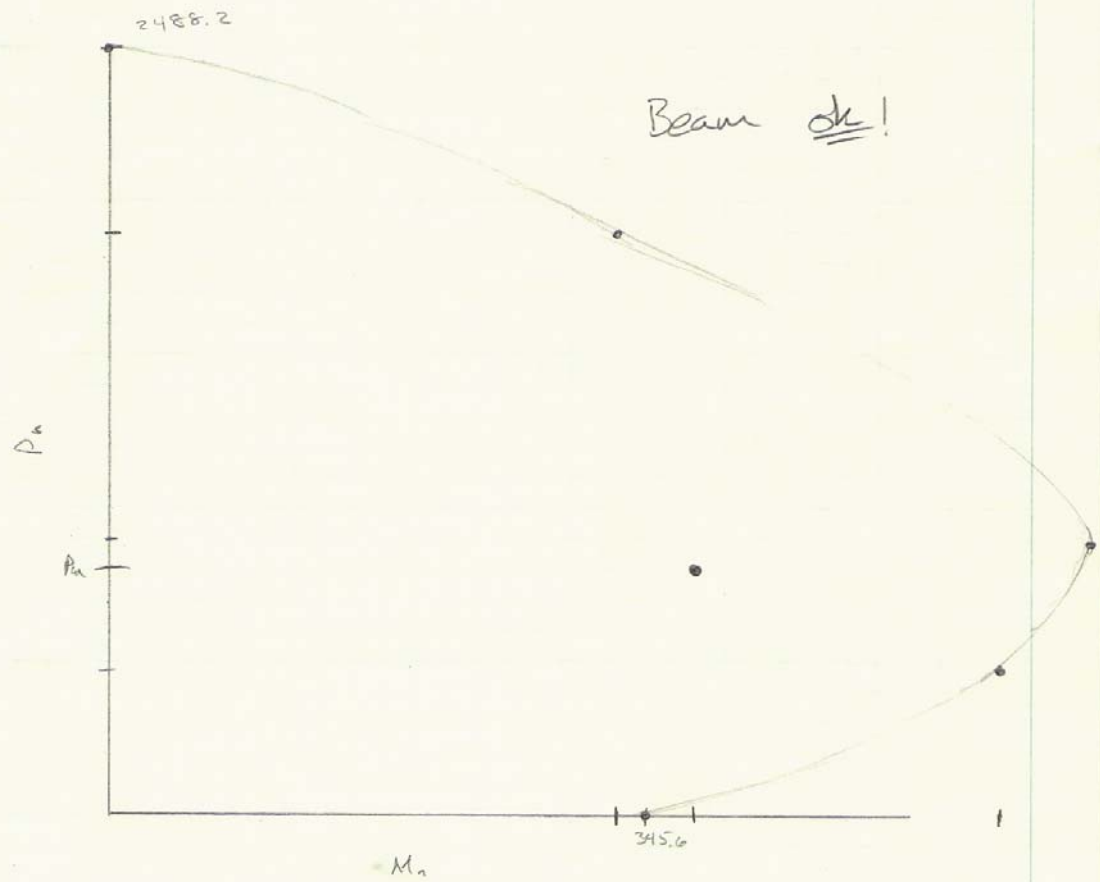
$w = (334)(30')$
$= 10020$ plf

$P_u = 833k$

$$M_u = \frac{w\ell_1^2}{8} + \frac{w\ell_2^2}{12}$$

$$= \frac{(10.02klf)(14^2)}{8} - \frac{(10.02)(27^2)}{12}$$

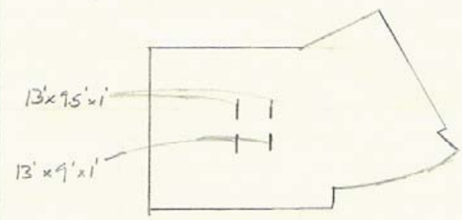
$$= -363.2^{k'}$$



Beam ok!

Shear Design using Seismic & wind loads

	base shear	
seismic	790.01^k	← controls
wind	324.00^k	



$V_n = 10\sqrt{f'c}\ h\ d$ $\quad h = 12"$ $\quad d = .8\, l_w$

$l_w = 9.5'$ $\quad d = 7.6'$

$V_n = 10\sqrt{4000}\ (12")(7.6\times12")$

$V_n = 692^k$

$\phi V_n = .8(692^k)$

$\phi V_n = 553.6 > 197.5^k$ ok $\quad V_u = \frac{790.01}{4} = 197.5^k$

AMPAD 22-141 50 SHEETS 22-142 100 SHEETS 22-144 200 SHEETS