Appendix A

Structural Depth Calculations

Shear Wall Calculations

Estimates on how much load a certain shear wall absorbs can be made from the principle of relative stiffness, which involves direct shear, torsion and bending. After calculating the center of rigidity and the torsional constant for this building, it became clear that the overall effect of eccentric loading on the center of stiffness had a negligible impact on the outcome of the shear calculation. In fact, calculating each shear wall using the direct shear method yielded a shear value to within 99.2% of the actual shear.

Shear wall calculation (revised)

SW 3 $\quad l = 21'$

$V = 178k$

ACI 21.7.2.2

$2(12)(21)(12)\sqrt{6000}\,/1000 = 468.5 < V_u \quad \therefore$ 2 curtains

$\rho_l, \rho_t = \dfrac{A_{sl}}{A_{cv}} \geq 0.0025$

$A_{cv} = (144\ in^2/ft)(0.0025) = 0.36\ in^2/ft$ req'd

Assume #5

$A_{sl} = 0.62\ in^2 / \$$, $\$ =$ spacing

$\dfrac{0.36}{12} = \dfrac{0.62}{\$} \Rightarrow \$ = 20.67"$ MAX

Try #5 @ 18" o.c. Both Directions

$\dfrac{h_w}{l_w} = \dfrac{130}{21} = 6.19 > 2 \quad \therefore \alpha = 2.0$

$V_u = (12)(12)(21)(2\sqrt{6000} + 0.0043(60{,}000))\,/1000 = 1249k$

$\phi V_n = 0.6(1249) = 749.4k > V_u \therefore$ ok

Distribution of Lateral Loads

Force Distribution Calculations

Element	Height	Depth	h/d	(h/d)^3	3(h/d)	ΔF	R
SW1	10	11	0.909090909	0.751314801	2.727272727	3.478587528	0.287473002
SW2	10	11	0.909090909	0.751314801	2.727272727	3.478587528	0.287473002
SW3	10	21	0.476190476	0.1079797	1.428571429	1.536551128	0.650808152
SW4	10	9	1.111111111	1.371742112	3.333333333	4.705075446	0.212536443
SW5	10	11	0.909090909	0.751314801	2.727272727	3.478587528	0.287473002
SW6	10	11	0.909090909	0.751314801	2.727272727	3.478587528	0.287473002
SW7	10	9	1.111111111	1.371742112	3.333333333	4.705075446	0.212536443
SW8	10	21.33	0.468823254	0.103045121	1.406469761	1.509514882	0.662464486
SW9	10	15.33	0.652315721	0.277570646	1.956947162	2.234517808	0.447523844
SW10	10	20	0.5	0.125	1.5	1.625	0.615384615
SW11	10	21.33	0.468823254	0.103045121	1.406469761	1.509514882	0.662464486
SW12	10	21.33	0.468823254	0.103045121	1.406469761	1.509514882	0.662464486

Center of Mass

Center of Mass Calculations

					Distance from Reference			
Element	Area	Height	Unit Weight	W	x	y	Wx	Wy
Floor	9790	0.666	0.15	978.021	64.66	18.33	63238.83786	17927.12
SW1	11	10.25	0.15	16.9125	5.5	0	93.01875	0
SW2	11	10.25	0.15	16.9125	145.16	0	2455.0185	0
SW3	21	10.25	0.15	32.2875	111.16	20.33	3589.0785	656.4049
SW4	9	10.25	0.15	13.8375	76	31.16	1051.65	431.1765
SW5	11	10.25	0.15	16.9125	29.17	48.66	493.337625	822.9623
SW6	11	10.25	0.15	16.9125	168.83	48.66	2855.337375	822.9623
SW7	9	10.25	0.15	13.8375	76	51.16	1051.65	707.9265
SW8	21.33	10.25	0.15	32.794875	0	45.66	0	1497.414
SW9	15.33	10.25	0.15	23.569875	48.665	11	1147.027967	259.2686
SW10	20	10.25	0.15	30.75	48.83	47.84	1501.5225	1471.08
SW11	21.33	10.25	0.15	32.794875	45.66	139.66	1497.413993	4580.132
SW12	21.33	10.25	0.15	32.794875	45.66	150.66	1497.413993	4940.876

Xmass	Ymass
63.95047	27.11300782

Center of Rigidity

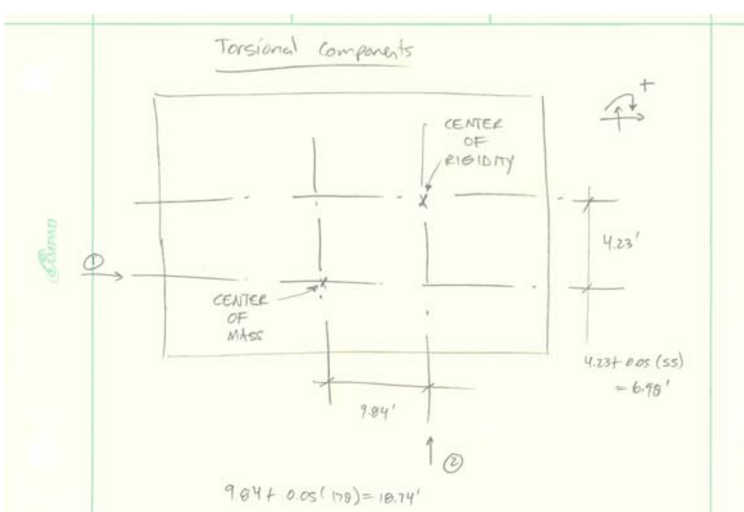
Center of Rigidity Calculations

	Distance from Reference					
Element	x	y	Rx	Ry	RxY	RyX
SW1		0	0.287473002		0	
SW2		0	0.287473002		0	
SW3		20.33	0.650808152		13.23092973	
SW4		31.16	0.212536443		6.622635569	
SW5		48.66	0.287473002		13.98843629	
SW6		48.66	0.287473002		13.98843629	
SW7		51.16	0.212536443		10.87336443	
SW8	0			0.662464486		0
SW9	11			0.447523844		4.922762289
SW10	47.83			0.615384615		29.43384615
SW11	139.66			0.662464486		92.51979008
SW12	150.66			0.662464486		99.80689942
			2.225773047	3.050301917	58.7038023	226.6832979

Xrigidity	Yrigidity
74.315036	26.37456788

Torsion Issues

Because the centers of mass and rigidity do not coincide, any wind or seismic loading will create inherent torsion on the building. The distance between the two centers is 9.84' East-West and 4.23' North-South. By taking a consistent sign convention, the results from the hand analysis closely match those found from the RAM output.



Lateral Load Distributions, Forces Parallel to Short Dimension

Controlling Shear (k): 541

Element	Ksn	Cn	Ksn	Cn	KsnCn^2	Direct Shear	Torsional Shear	Hn
SW1	0.277428033	28.83			230.5895527	0	2.313813323	-2.31381
SW2	0.277428033	28.83			230.5895527	0	2.313813323	-2.31381
SW3	0.63268393	8.51			45.8190335	0	1.557577614	-1.55758
SW4	0.204336725	2.32			1.099821987	0	0.137141137	0.137141
SW5	0.277428033	19.798			108.7409147	0	1.588930842	1.588931
SW6	0.277428033	19.798			108.7409147	0	1.588930842	1.588931
SW7	0.204336725	22.29			101.523496	0	1.31761894	1.317619
SW8			0.644080967	75.24	3646.179454	117.5532719	14.01917915	131.5725
SW9			0.433880125	64.24	1790.526782	79.1888457	8.063222091	87.25207
SW10			0.598046304	27.41	449.3170325	109.1513387	4.742170661	113.8935
SW11			0.644080967	64.44	2674.554976	117.5532719	12.00685678	105.5464
SW12			0.644080967	75.44	3665.589478	117.5532719	14.05644438	103.4968

Lateral Load Distributions, Forces Parallel to Long Direction

Controlling Shear (k): 541

Element	Ksn	Cn	Ksn	Cn	KsnCn^2	Direct Shear	Torsional Shear	Hn
SW1	0.277428033	28.83			230.5895527	69.77392639	6.212157833	75.98608
SW2	0.277428033	28.83			230.5895527	69.77392639	6.212157833	75.98608
SW3	0.63268393	8.51			45.8190335	159.1217786	4.181805801	163.3036
SW4	0.204336725	2.32			1.099821987	51.39125793	0.368198411	51.02306
SW5	0.277428033	19.798			108.7409147	69.77392639	4.265983377	65.50794
SW6	0.277428033	19.798			108.7409147	69.77392639	4.265983377	65.50794
SW7	0.204336725	22.29			101.523496	51.39125793	3.537561453	47.8537
SW8			0.644080967	75.24	3646.179454	0	37.63888499	-37.6389
SW9			0.433880125	64.24	1790.526782	0	21.64824957	-21.6482
SW10			0.598046304	27.41	449.3170325	0	12.73184501	-12.7318
SW11			0.644080967	64.44	2674.554976	0	32.23617423	32.23617
SW12			0.644080967	75.44	3665.589478	0	37.73893519	37.73894

Story Drift (┴ to long direction)

Level	Wind	Seismic
1	0.023	0.048
2	0.07	0.16
3	0.122	0.31
4	0.19	0.49
5	0.26	0.7
6	0.34	0.93
7	0.43	1.18
8	0.53	1.47
9	0.61	1.74
10	0.71	2.02
11	0.8	2.3
Roof	0.88	2.58
Penthouse	1.2	3.88

	Building Height to Low Roof
	130'-0"
	Equivalent Drift, Seismic
L/	402.0618557
	Equivalent Drift, Wind
L/	1300

Wind Calculations

Wind load calculations were performed according to ASCE 7-05 using method 2 – analytical procedure. K_{zt} was assumed to be equal to 1.0 and the building was considered enclosed when analyzing the main wind force resisting system (mwfrs) according to case 1. Through seismic calculations, the building was determined to be rigid. Linear interpolation was used where permitted.

Velocity Pressures by Floor

Level	Height	Kz	qz	
1	0	0.57	12.4032	
2	12	0.57	12.4032	
3	22.25	0.64	13.9264	
4	32.5	0.715	15.5584	
5	42.75	0.7725	16.8096	
6	53	0.8234	17.91718	
7	63.25	0.8634	18.78758	
8	74.25	0.908	19.75808	
9	84.5	0.943	20.51968	
10	94.75	0.975	21.216	
11	105	1.0025	21.8144	
Low Roof	115.25	1.0275	22.3584	
High Roof	130	1.065	23.1744	qh
Parapet	132	1.07	23.2832	

Design Pressure		⊥55'			⊥178'		
Level	Height	p w-w	p l-w	p roof	p w-w	p l-w	p roof
1	0	8.371749	-4.51661	-18.96587	8.147242	-9.51405	-19.7892
2	12	8.371749	-4.51661	-15.64195	8.147242	-9.51405	
3	22.25	9.399858	-4.51661		9.147781	-9.51405	
4	32.5	10.5014	-4.51661		10.21979	-9.51405	
5	42.75	11.34592	-4.51661		11.04166	-9.51405	
6	53	12.09351	-4.51661		11.76919	-9.51405	
7	63.25	12.681	-4.51661		12.34093	-9.51405	
8	74.25	13.33605	-4.51661		12.97841	-9.51405	
9	84.5	13.8501	-4.51661		13.47868	-9.51405	
10	94.75	14.3201	-4.51661		13.93607	-9.51405	
11	105	14.724	-4.51661		14.32914	-9.51405	
Low Roof	115.25	15.09118	-4.51661		14.68648	-9.51405	
High Roof	130	15.64195	-4.51661		15.22248	-9.51405	
Parapet	132	34.9248	-23.2832		34.9248	-23.2832	

Story Shear	⊥55'	⊥178'
Level		
1	8.506319	37.72452
2	7.265814	32.22303
3	7.845411	34.04851
4	8.466408	36.00438
5	8.942505	37.50389
6	9.363954	38.83127
7	10.40455	42.79204
8	10.06444	41.0375
9	10.35424	41.95024
10	10.6192	42.78475
11	10.84689	43.5019
Low Roof	15.90682	63.53848
Total	118.5866	491.9405

Overturning	⊥55'	⊥178'
Level		
1	51.03792	226.3471
2	124.4271	551.8193
3	214.7681	932.0779
4	318.5486	1354.665
5	428.1224	1795.499
6	544.2799	2257.068
7	715.3131	2941.953
8	798.8648	3257.351
9	927.9985	3759.79
10	1060.592	4273.127
11	1194.514	4790.647
Low Roof	1950.574	7791.406
Total	8329.041	33931.75

Seismic Calculations

As the vertical distribution of forces shows, seismic analysis was the controlling factor in both directions. That is, the seismic base shear, which is the same in both directions, was larger than either direction of wind base shear. This result is not surprising, as the seismic response is based on the building weight. Concrete buildings tend to carry more mass per story, and consequently are often controlled by seismic design criteria.

The overturning moment also turned out to be larger for seismic than wind. This can be attributed to larger forces being present at higher elevations for the seismic design. The vertical distribution of forces equation attempts to take a whiplash effect into account. As the base of the building moves one way, the top wants to catch up to it. As it does this, the base of the building switches directions and moves back, thus pulling the top of the building back to its original position with much greater force.

Once the seismic and wind forces are determined, the analysis of the lateral elements of the building can begin. Because the seismic load controls, the shear walls will be analyzed according to their relative stiffness within the group using seismic load.

Seismic Inputs	
Variable	Value
S_s	0.152
S_1	0.5
F_a	1.6
F_v	2.4
I	1
SM_s	0.2432
SM_1	1.2
SD_s	0.16213333
SD_1	0.8
R	5
C_s	0.03243
C_t	0.02
h_n	130
x	0.75
T_a	0.7699943
T_o	0.98684211
T_s	4.93421053
V (k)	530.594332

Total Weight by Floor		
Floor	Total Weight	Elevation
1	0	0
2	1276136	12
3	1566041	22.25
4	1566041	32.5
5	1566041	42.75
6	1566041	53
7	1566041	63.25
8	1566041	74.25
9	1566041	84.5
10	1566041	94.75
11	1566041	105
Low Roof	924500	115.25
High Roof	66214	130

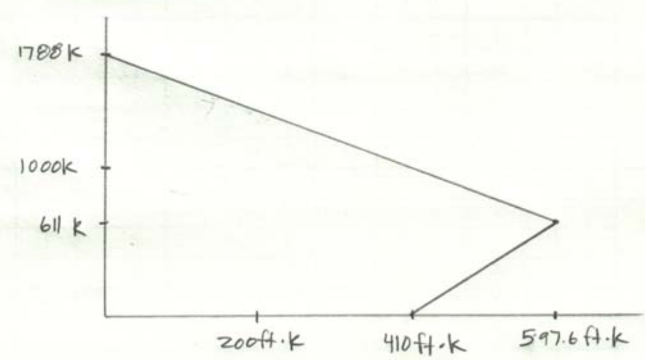
Weight Seen by Floor		
Floor	Weight	Story Shear
High Roof	66214	2.14732002
Low Roof	990714	32.128855
11	2556755	82.9155647
10	4122796	133.702274
9	5688837	184.488984
8	7254878	235.275694
7	8820919	286.062403
6	10386960	336.849113
5	11953001	387.635822
4	13519042	438.422532
3	15085083	489.209242
2	16361219	530.594332
1	16361219	530.594332

Vertical Distribution of Forces		
Floor	C_{vx}	F_x (k)
High Roof	0.00906249	4.80850739
Low Roof	0.11036758	58.5604134
11	0.16819967	89.2457906
10	0.14968997	79.4246482
9	0.13144911	69.7461519
8	0.1135052	60.225216
7	0.09461914	50.2043777
6	0.07741558	41.0762699
5	0.06065796	32.1847679
4	0.04443886	23.5790064
3	0.02890646	15.3376016
2	0.01168799	6.20158118
1	0	0
	1	530.594332

Base Moment	
First Floor	42546.1255

Column Calculations

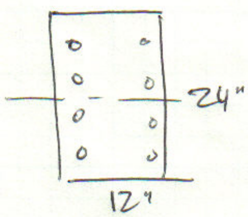
Column D-3 is a 12"x24" column with eight number 9 vertical reinforcing bars, 4 in each face. Assuming a cover of 1-1/2" all around, I found the pure axial capacity of the column to be 1788k. Similarly, the pure bending capacity of the column, about an axis perpendicular to the 24" side, was found to be 410 ft-k. The balanced strain condition is the last point needed to make a preliminary column interaction diagram. After calculating the balanced condition, which yielded 611k of compression and 597.6 ft-k of bending capacity, the diagram looked like this:



COLUMN INTERACTION DIAGRAM
FOR COLUMN D-3

If the actual point lies somewhere inside this conservative area, the column is deemed adequate.

Column Interaction



$f'_c = 6$ ksi
$f_y = 60$ ksi
8-#9 bars

Pure axial $P_0 = (0.85)(6)(12 \times 24 - 8.0) + (8)(60)$
$= 1788$ k

Balanced Condition

$\varepsilon_y = 60/29{,}000 = 0.00207$

$c = \frac{.003}{.003 + .00207}(22.5) = 11.84"$

$\varepsilon_{s1} = \frac{.003}{11.84}(11.84 - 1.5) = 0.00262 \quad f_{s1} = 60$ ksi

$\varepsilon_{s2} = \frac{.003}{11.84}(11.84 - 8.5) = 24.5$ ksi

$\varepsilon_{s3} = \frac{.003}{11.84}(11.84 - 15.5) = -26.9$ ksi

$\varepsilon_{s4} = -60$ ksi

$M_b = (.85)(6)(12)(.85)(11.84)\left(12 - \frac{(.85)(11.84)}{2}\right)$
$+ 2(60)(12 - 1.5) + 2(24.3)(12 - 8.5)$
$+ 2(-26.9)(12 - 15.5) + 2(-60)(12 - 22.5) = 597.6$ ft·k

$P_b = (.85)(6)(12)(.85)(11.84) + 2(60) + 2(24.5) + 2(-26.9)$
$+ 2(-60) = 611$ k

Pure bending   assume 2 don't yield, 2 do

$f_{s_1} = \frac{.003}{c}(c-1.5)(29k) =$

$f_{s_2} = \frac{.003}{c}(c-8.5)(29k)$

$f_{s_3} = f_{s_4} = -60$

$\Sigma F = 0 \quad (.85)(6)(12)(.85)c + 2f_{s_1} + 2f_{s_2} + 2f_{s_3} + 2f_{s_4}$

$0 = 52c + \frac{174}{c}(c-1.5) + \frac{174}{c}(c-1.5) - 4(60)$

$0 = 52c^2 + \frac{348}{c}(c-1.5) - 240c$

$-52c^2 + 240c = .348c - 522$

$-52c^2 - 108c + 522 = 0$

$$\frac{108 \pm \sqrt{108^2 - 4(-52)(522)}}{2(-52)}$$

$52c - 240 + \frac{.003}{c}((c-1.5)(29000)(4) = 0$

$52c^2 - 240c + \frac{348}{c}(c-1.5) = 0$

$52c^2 - 240c + 348c - 522 = 0$

$52c^2 + 108c - 522 = 0$

$$\frac{-108 \pm \sqrt{108^2 - 4(52)(-522)}}{2(52)} = \frac{-108 \pm 347}{104}$$

$c = 2.3''$

$f_{s_1} = \frac{.003}{2.3}(2.3-1.5)(29000) = 30.3k$

$f_{s_2} = \frac{.003}{2.3}(2.3-8.5) \quad > -60 \text{ ksi} \therefore \text{yielded}$

$.85(6)(12)(.85)c + \frac{.003}{c}(c-1.5)(29000)(2) \quad 360$

$52c - 360 + \frac{.003}{c}(c-1.5)(29000)(2)$

$52c - 360 + \frac{174}{c}(c-1.5) = 0$

$52c^2 - 360c + 174c - 261 = 0$

$$\frac{186 \pm \sqrt{186^2 - 4(52)(-261)}}{104}$$

$c = 4.655''$

$f_{s_1} = 58.9k$

$f_{s_2} = \frac{.003}{4.655}(4.655-8.5) = -60$

$f_{s_3} = f_{s_4} = -60$

$M_o = .85(6)(12)(.85)(4.655)\left(12 - \frac{.85(4.655)}{2}\right) \quad 2427$

$+2(58.9)(12-1.5) + 2(-60)(12-8.5)$

$+2(-60)(12-15.5)$

$+2(-60)(12-22.5) = 4924$

$= 410 \text{ ft} \cdot k$

Post Tensioning Analysis

SPAN LENGTH = 29'-0 $\frac{29}{45} = 0.64' \Rightarrow$ 8" PT SLAB
TRIB WIDTH = 17.5'

LOADING

(3.5')(100) = 350
(14')(40) = 560
(29)(20) = 580
1490 lb/ft²

$\frac{1490}{17.5} = 85.14$ lb/ft²

LOADS

100 DEAD
85.14 LIVE
8 MEP
194 TOTAL

(194)(17.5) = 3395 plf

BALANCE 90% DEAD ⇒ 90 psf

USE ½"ϕ 270k TENDONS, ASSUMING 30k LOSSES

$f_e = (0.153)(0.7)(270-30) = 25.7$ k PER TENDON

$F_e = (0.09)(8)(12) = 8.64$ k/ft

FOR A 29'-0 SPAN, $\frac{(29)(8.64)}{25.7} = 9.75$, SAY 10 TENDONS

$\rightarrow F_e = \frac{10(25.7)}{17.5} = 14.7$ k

$\sigma = \frac{F}{A} = \frac{14.7}{(12)(8)} = 0.153$ ksi

NET LOAD

194 - 90 = 104 psf

$$\frac{P}{A} = \frac{18(25.7)(1000)}{8(284)} = 204 \text{ psi}$$

$$\frac{P}{A} = \frac{9(25.7)(1000)}{8(11)(12)} = 219 \text{ psi}$$

$$\frac{P}{A} = \frac{(24\ ^{18})(25.78)(1000)}{8(12)(23.34)} = 206 \text{ psi}$$

$$A = (122)(8) = 976 \text{ in}^2$$

$$S = \frac{I}{c} = \frac{I}{4} = \frac{\frac{bh^3}{12}}{4} = \frac{\frac{(122)(8)^3}{12}}{4} = 1301 \text{ in}^3$$

$$W_b = -1.366 \text{ k/ft}$$

@ JACKING

$f'ci = 3000$ psi

$0.6f'ci = 1800$ psi

$3\sqrt{f'ci} = 164$ psi

@ SERVICE

$f'c = 5000$ psi

$0.45f'c = 2250$ psi

$6\sqrt{f'c} = 424$ psi

$w_b = 0.95$ k/ft

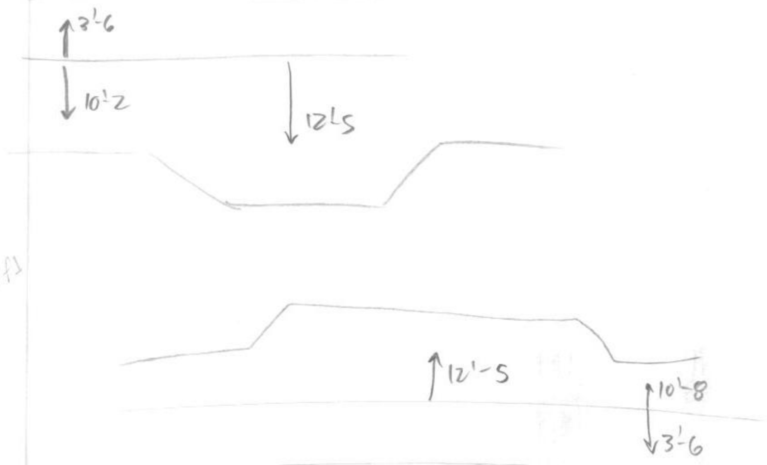
$\frac{P}{A} = \frac{15(25.7)}{13'\text{-}8\ (8)} = 293.8$ psi $\frac{15(25.7)}{15'\text{-}11(8)} = 252$ psi

(12)

$\frac{P}{A} = \frac{12(25.7)(1000)}{1312} = 235$ psi $\frac{12(25.7)(1000)}{1528} = 202$ psi

⟷

$\frac{9(25.7)(1000)}{976} = 237$ psi $\frac{9(25.7)(1000)}{12'\text{-}5\ (8)} = 194$ psi



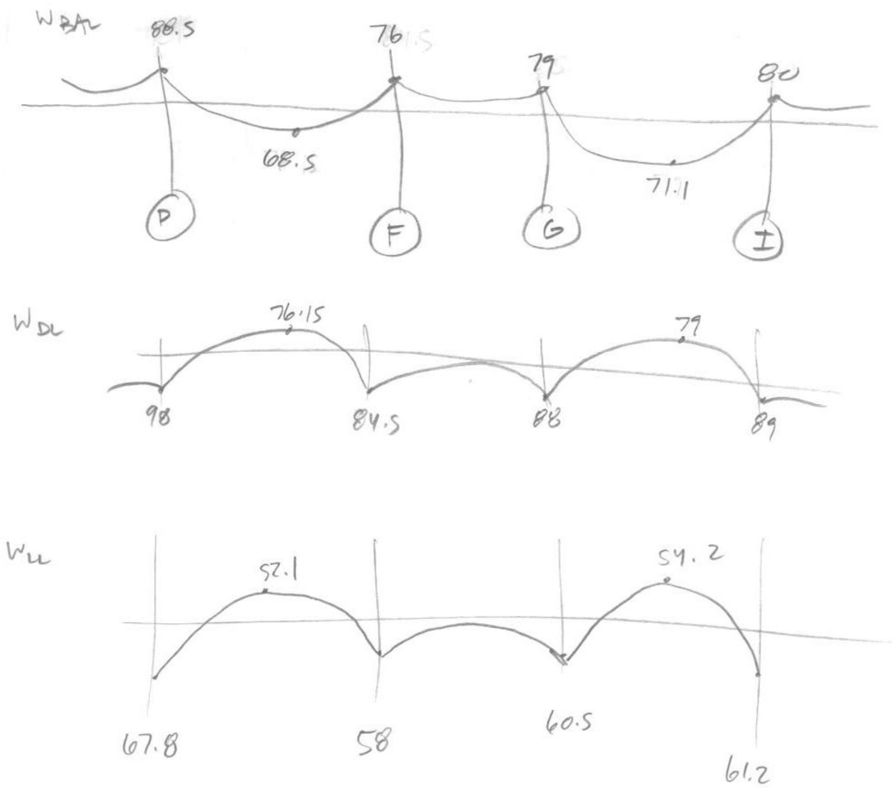
2366
2,366 k/ft

23'-8

24 TENDONS

$\frac{P}{A} = \frac{24(25.7)}{8(284)} = 271$ psi

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
1	w DL-->	1.366		1.366		1.479		1.592		1.592		1.592		1.479		1.366		1.366	
2		AB	BA	BC	CB	CD	DC	DF	FD	FG	GF	GI	IG	IJ	JI	JL	LJ	LM	ML
3	K	0.127	0.127	0.273	0.273	0.222	0.222	0.103	0.103	0.15	0.15	0.103	0.103	0.222	0.222	0.127	0.127	0.273	0.273
4	DF	1	0.3175	0.6825	0.551515	0.448485	0.683077	0.316923	0.407115	0.592885	0.592885	0.407115	0.316923	0.683077	0.636103	0.363897	0.3175	0.6825	1
5	COF	1	0	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0	1
6	FEM	-63.7557	63.75572	-13.7738	13.77383	-22.4623	22.46231	-111.573	111.5727	-53.0667	53.06667	-111.573	111.5727	-22.4623	22.46231	-63.7557	63.75572	-13.7738	13.77383
7																			
8	DIST	63.75572																	-13.7738
9	CO	0	31.87786															-6.88692	0
10	SUM	0	95.63358	-13.7738	13.77383	-22.4623	22.46231	-111.573	111.5727	-53.0667	53.06667	-111.573	111.5727	-22.4623	22.46231	-63.7557	63.75572	-20.6608	0
11																			
12	DIST		-25.9905	-55.8693	4.791828	3.896651	60.86923	28.24113	-23.8186	-34.6874	34.68735	23.81865	-28.2411	-60.8692	26.26687	15.02654	-13.6827	-29.4123	
13	CO			2.395914	-27.9346	30.43461	1.948326	-11.9093	14.12056	17.34368	-17.3437	-14.1206	11.90932	13.13343	-30.4346	-6.84133	7.513271		
14	SUM		69.64311	-67.2472	-9.36898	11.86895	85.27986	-95.2409	101.8746	-70.4103	70.41034	-101.875	95.24086	-70.1981	18.29457	-55.5705	57.58634	-50.0731	
15																			
16	DIST		-0.7607	-1.63521	-1.37877	-1.1212	6.804128	3.15687	-12.8096	-18.6547	18.65469	12.80955	-7.93663	-17.1061	23.71134	13.5646	-2.38546	-5.12781	
17	CO			-0.68939	-0.81761	3.402064	-0.5606	-6.40478	1.578435	9.327344	-9.32734	-3.96831	6.404776	11.85567	-8.55307	-1.19273	6.782299		
18	SUM		68.88241	-69.5718	-11.5654	14.14982	91.52339	-98.4888	90.64347	-79.7377	79.73769	-93.0333	93.70901	-75.4486	33.45285	-43.1986	61.98317	-55.2009	
19																			
20	DIST		0.21888	0.470506	-1.42537	-1.15909	4.757888	2.207488	-4.4399	-6.46588	7.882801	5.412857	-5.78716	-12.4733	6.199332	3.546465	-2.15338	-4.62892	
21	CO			-0.71268	0.235253	2.378944	-0.57955	-2.21995	1.103744	3.941401	-3.23294	-2.89358	2.706428	3.099666	-6.23665	-1.07669	1.773232		
22	SUM		69.10129	-69.814	-12.7555	15.36967	95.70174	-98.5012	87.30731	-82.2622	84.38755	-90.5141	90.62828	-84.8222	33.41553	-40.7289	61.60303	-59.8298	
23																			
24	DIST		0.226277	0.486407	-1.44177	-1.17243	1.912271	0.887225	-2.05395	-2.99119	3.632322	2.494195	-1.84009	-3.96601	4.652035	2.6613	-0.563	-1.21023	
25	CO			-0.72088	0.243203	0.956136	-0.58621	-1.02698	0.443612	1.816161	-1.4956	-0.92004	1.247097	2.326018	-1.983	-0.2815	1.33065		
26	SUM		69.32757	-70.0485	-13.954	15.15338	97.02779	-98.641	85.69697	-83.4372	86.52428	-88.9399	90.03529	-86.4622	36.08456	-38.3491	62.37068	-61.04	
27																			
28	DIST		0.228881	0.492004	-0.66145	-0.53789	1.101933	0.511257	-0.91999	-1.33979	1.432197	0.983442	-1.1324	-2.44071	1.440459	0.824046	-0.42248	-0.90817	
29	CO			-0.33073	0.246002	0.550966	-0.26894	-0.45999	0.255629	0.716098	-0.66989	-0.5662	0.491721	0.720229	-1.22036	-0.21124	0.412023		
30	SUM		69.55645	-69.8872	-14.3695	15.16646	97.86078	-98.5897	85.03261	-84.0609	87.28658	-88.5227	89.39461	-88.1827	36.30467	-37.7363	62.36022	-61.9482	
31																			
32	DIST		0.105006	0.225721	-0.43954	-0.35743	0.497919	0.231017	-0.3956	-0.57612	0.732862	0.503232	-0.3841	-0.82786	0.910643	0.520954	-0.13082	-0.28121	
33	CO			-0.21977	0.112861	0.24896	-0.17871	-0.1978	0.115508	0.366431	-0.28806	-0.19205	0.251616	0.455322	-0.41393	-0.06541	0.260477		
34	SUM		69.66145	-69.8812	-14.6962	15.05799	98.17999	-98.5565	84.75251	-84.2706	87.73138	-88.2115	89.26213	-88.5552	36.80138	-37.2807	62.48988	-62.2294	
35																			
36	DIST		0.069777	0.149993	-0.19955	-0.16227	0.25719	0.119327	-0.1962	-0.28573	0.28465	0.195459	-0.22404	-0.48289	0.304907	0.174429	-0.0827	-0.17778	
37	CO			-0.09977	0.074997	0.128595	-0.08114	-0.0981	0.059663	0.142325	-0.14287	-0.11202	0.09773	0.152454	-0.24145	-0.04135	0.087214		
38	SUM		69.73123	-69.831	-14.8207	15.02431	98.35604	-98.5353	84.61597	-84.414	87.87316	-88.1281	89.13582	-88.8856	36.86484	-37.1476	62.49439	-62.4072	
39																			
40	DIST		0.031678	0.068096	-0.11228	-0.09131	0.122433	0.056805	-0.08223	-0.11976	0.15112	0.103769	-0.07929	-0.17089	0.179888	0.102909	-0.02769	-0.05952	
41	CO			-0.05614	0.034048	0.061217	-0.04565	-0.04112	0.028402	0.07556	-0.05988	-0.03964	0.051885	0.089944	-0.08545	-0.01385	0.051454		
42	SUM		69.76291	-69.8191	-14.899	14.99422	98.43282	-98.5196	84.56214	-84.4582	87.9644	-88.0639	89.10841	-88.9666	36.95928	-37.0586	62.51815	-62.4667	
43		0.0	69.76	-69.82	-14.90	14.99	98.43	-98.52	84.56	-84.46	87.96	-88.06	89.11	-88.97	36.96	-37.06	62.52	-62.47	0.0



AFTER JACKING

INTERIOR SPAN

MIDSPAN STRESS

$$f_t = \frac{(79 - 71.1)(12)(1000)}{1301} - 219 = -146 \text{ psi}$$

$$f_b = \frac{(-79 + 71.1)(12)(1000)}{1301} - 219 = -292 \text{ psi}$$

SUPPORT STRESS

$$\frac{(88.5 - 98)(12)(1000)}{1301} - 219 = -307 \text{ psi}$$

$$\frac{(-88.5 + 84.5)(12)(1000)}{1301} - 219 = -256 \text{ psi}$$

ULTIMATE STRENGTH

$M = P \cdot e$

$= (231k)(8-4)/12 = 77 \text{ ft}\cdot\text{k}$

$M_{sec} = M_{BM} - M_1 = 88.5 - 77 = 11.5 \text{ ft}\cdot\text{k}$

@ MIDSPAN

$M = 1.2(79) + 1.6(54.2) + \frac{11.5}{2} = 187.3 \text{ ft}\cdot\text{k}$

@ SUPPORT

$M = 1.2(-98) + 1.6(-68) + \frac{11.5}{2} = -220.6 \text{ ft}\cdot\text{k}$

Punching Shear Calculations

$S_S = 0.1769$

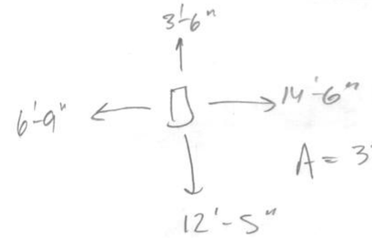
$S_1 = 0.0629$

$S_{DS} = 0.1887$

$S_{D1} = 0.1006$

SDC B

$C_S = 0.0377$?



$A = 338.23 ft^2$

PUNCHING SHEAR CHECK

$V_c = 4\sqrt{5000}\,(6.5)(98)$ $= 180k$

$V_c = \left(\frac{30(6.5)}{98} + 2\right)\sqrt{5000}\,(98)(6.5)$ $= 179.7k$

$0.75(179.7) = 134.78k$

$V_u = 1.2D + 1.6L$

$1.2(100)(339) + 1.6(100)(75) + (1.6)(60)(264)$

$= 78k < 134.78k$ ∴ ok FOR PUNCHING SHEAR

EXTERIOR ~~SLAB~~ COLUMN

$A = 264 ft^2$

$V_c = 4\sqrt{5000}\,(79.5)(6.5) = 146k$

$\phi V_c = 109.6k$

$V_u = 1.2(100)(264) + (1.6)(60)(264) = 57k$

$$V_c = \left(\frac{(30)(6.5)}{49} + 2 \right) \sqrt{5000}\,(6.5)(49) = 163k\,(0.75) = 122k$$

$$\phi V_c = 0.75(2)\sqrt{5000}\,(49)(6.5) = 31.2k$$

$$\frac{57k - 31.2k}{0.75k} = 34.4k < 122k$$

$$A_v = \frac{34{,}400}{60{,}000(0.707)} = 0.81\,in^2$$

$$\frac{0.81}{4} = 0.2\,in^2 \Rightarrow (4) - \#5$$