APPENDIX B

HAND AND EXCEL DESIGN CALCULATIONS

APPENDIX B

TABLE 4-2

TABLE 4-2
LIVE LOAD ELEMENT FACTOR, K_{LL}

Element	K_{LL} (Note 1)
Interior Columns	4
Exterior Columns without cantilever slabs	4
Edge Columns with cantilever slabs	3
Corner Columns with cantilever slabs	2
Edge Beams without cantilever slabs	2
Interior Beams	2
All Other Members Not Identified Above including: Edge Beams with cantilever slabs; Cantilever Beams; One-way Slabs; Two-way Slabs; Members without provisions for continuous shear transfer normal to their span	1

Note 1. In lieu of the values above, K_{LL} is permitted to be calculated.

COLUMN DESIGN

Assumptions:

1) S.D.L. = 15 lbs
2) Roof Snow Load = 23 psf
 - added in after Roof LL accessed

**** Air Handling Units add approximately 50 psf to DL

Roof Live Load Reduction	
R_1 =	0.6
R_2 =	1

Perimeter Wall DL			
Span	Weight	Story Ht.	Total DL
ft	psf	ft	kips
30	75	14	31.5

Gravity Column Design A

Typical Interior Gravity Loads

Column	Floor Above	Span	Area	A_T	K_{LL}	A_I	L	DL	LL	LLR	Tot. LL	Tot. DL	Factored	Unfactored	
		ft x ft	sq. ft	sq. ft		sq. ft		psf	psf	psf	kips	kips	kips	kips	
9	Roof	30 x 30	900	900	4	3600	0.600	125	40	47	42.3	112.5	202.68	154.80	------->> Air Handling Units
8	9	30 x 30	900	1800	4	7200	0.427	85	100	42.68	38.41	76.5	355.94	269.71	
7	8	30 x 30	900	2700	4	10800	0.400	85	100	40	36	76.5	505.34	382.21	
6	7	30 x 30	900	3600	4	14400	0.400	85	100	40	36	76.5	654.74	494.71	
5	6	30 x 30	900	4500	4	18000	0.400	135	100	40	36	121.5	858.14	652.21	------->> "Mechanical Floor"
4	5	30 x 30	900	5400	4	21600	0.400	85	100	40	36	76.5	1007.54	764.71	
3	4	30 x 30	900	6300	4	25200	0.400	85	100	40	36	76.5	1156.94	877.21	
2	3	30 x 30	900	7200	4	28800	0.400	85	100	40	36	76.5	1306.34	989.71	
1	2	30 x 30	900	8100	4	32400	0.400	85	100	40	36	76.5	1455.74	1102.21	------->> Last Steel Column
Conc. Pier	1	30 x 30	900	9000	4	36000	0.400	140	100	40	36	126	1664.54	1264.21	------->> Concrete Pier

Gravity Column Design B

Atypical Interior Gravity Loads (South Side)

Column	Floor Above	Span*	Area	A_T	K_{LL}	A_I	L	DL	LL	LLR	Tot. LL	Tot. DL	Factored	Unfactored	
		ft x ft	sq. ft	sq. ft		sq. ft		psf	psf	psf	kips	kips	kips	kips	
9	Roof	30 x 45	1125	1125	4	4500	0.600	125	40	47	52.875	140.625	253.35	193.50	------->> Air Handling Units
8	9	30 x 45	1125	2250	4	9000	0.408	85	100	40.81	45.91	95.625	441.56	335.04	
7	8	30 x 45	1125	3375	4	13500	0.400	85	100	40	45	95.625	628.31	475.66	
6	7	30 x 45	1125	4500	4	18000	0.400	85	100	40	45	95.625	815.06	616.29	
5	6	30 x 45	1125	5625	4	22500	0.400	135	100	40	45	151.875	1069.31	813.16	------->> "Mechanical Floor"
4	5	30 x 45	1125	6750	4	27000	0.400	85	100	40	45	95.625	1256.06	953.79	
3	4	30 x 45	1125	7875	4	31500	0.400	85	100	40	45	95.625	1442.81	1094.41	
2	3	30 x 45	1125	9000	4	36000	0.400	85	100	40	45	95.625	1629.56	1235.04	
1	2	30 x 30	900	9900	4	39600	0.400	85	100	40	36	76.5	1778.96	1347.54	------->> Last Steel Column
Conc. Pier	1	30 x 30	900	10800	4	43200	0.400	140	100	40	36	126	1987.76	1509.54	------->> Concrete Pier

*** Atypical Span constitutes 30' x 30' bay adjacent to 30' x 45' bay**

Gravity Column Design C

Typical Exterior Gravity Loads (South Side)

Column	Floor Above	Span	Area	A_T	K_{LL}	A_I	L	DL	LL	LLR	Tot. LL	Tot. DL	Factored	Unfactored	
		ft x ft	sq. ft	sq. ft		sq. ft		psf	psf	psf	kips	kips	kips	kips	
9	Roof	30 x 45	675	675	4	2700	0.600	125	40	47.00	31.73	84.38	152.01	116.10	------->> Air Handling Units
8	9	30 x 45	675	1350	4	5400	0.454	85	100	45.41	30.65	88.88	307.71	235.63	
7	8	30 x 45	675	2025	4	8100	0.417	85	100	41.67	28.13	88.88	459.36	352.63	
6	7	30 x 45	675	2700	4	10800	0.400	85	100	40.00	27.00	88.88	609.21	468.50	
5	6	30 x 45	675	3375	4	13500	0.400	135	100	40.00	27.00	122.63	799.56	618.13	------->> "Mechanical Floor"
4	5	30 x 45	675	4050	4	16200	0.400	85	100	40.00	27.00	88.88	949.41	734.00	
3	4	30 x 45	675	4725	4	18900	0.400	85	100	40.00	27.00	88.88	1099.26	849.88	
2	3	30 x 45	675	5400	4	21600	0.400	85	100	40.00	27.00	88.88	1249.11	965.75	
1	2	16 x 30	240	5640	4	22560	0.400	85	100	40.00	9.60	51.90	1326.75	1027.25	------->> Last Steel Column
Conc. Pier	1	16 x 30	240	5880	4	23520	0.400	140	100	40	9.6	65.1	1420.23	1101.95	------->> Concrete Pier

Gravity Column Design D

Typical Exterior Gravity Loads (North Side)

Column	Floor Above	Span	Area	A_T	K_{LL}	A_I	L	DL	LL	LLR	Tot. LL	Tot. DL	Factored	Unfactored	
		ft x ft	sq. ft	sq. ft		sq. ft		psf	psf	psf	kips	kips	kips	kips	
9	Roof	30 x 16	240	240	4	960	0.600	125	40	47.00	11.28	30.00	54.05	41.28	------->> Air Handling Units
8	9	30 x 16	240	480	4	1920	0.592	85	100	59.23	14.22	51.90	139.07	107.40	
7	8	30 x16	240	720	4	2880	0.530	85	100	52.95	12.71	51.90	221.69	172.00	
6	7	30 x16	240	960	4	3840	0.492	85	100	49.21	11.81	51.90	302.86	235.71	
5	6	30 x16	240	1200	4	4800	0.467	135	100	46.65	11.20	63.90	397.46	310.81	------->> "Mechanical Floor"
4	5	30 x16	240	1440	4	5760	0.448	85	100	44.76	10.74	51.90	476.92	373.45	
3	4	30 x16	240	1680	4	6720	0.433	85	100	43.30	10.39	51.90	555.83	435.74	
2	3	30 x16	240	1920	4	7680	0.421	85	100	42.12	10.11	51.90	634.28	497.75	
1	2	30 x16	240	2160	4	8640	0.411	85	100	41.14	9.87	51.90	712.36	559.53	------->> Last Steel Column
Conc. Pier	1	30 x16	240	2400	4	9600	0.403	140	100	40.30931	9.67	65.1	805.96	634.30	------->> Concrete Pier

APPENDIX B

Gravity Column Design E															
Atypical Interior Gravity Loads (North Side)															
Column	Floor Above	Span	Area	A_T	K_{LL}	A_I	L	DL	LL	LLR	Tot. LL	Tot. DL	Factored	Unfactored	
		ft x ft	sq. ft	sq. ft		sq. ft		psf	psf	psf	kips	kips	kips	kips	
9	Roof	30 x 23	690	690	4	2760	0.600	125	40	47.00	32.43	86.25	155.39	118.68	------->> Air Handling Units
8	9	30 x 23	690	1380	4	5520	0.452	85	100	45.19	31.18	58.65	275.66	208.51	
7	8	30 x 23	690	2070	4	8280	0.415	85	100	41.48	28.62	58.65	391.84	295.78	
6	7	30 x 23	690	2760	4	11040	0.400	85	100	40.00	27.60	58.65	506.38	382.03	
5	6	30 x 23	690	3450	4	13800	0.400	135	100	40.00	27.60	93.15	662.32	502.78	------->> "Mechanical Floor"
4	5	30 x 23	690	4140	4	16560	0.400	85	100	40.00	27.60	58.65	776.86	589.03	
3	4	30 x 23	690	4830	4	19320	0.400	85	100	40.00	27.60	58.65	891.40	675.28	
2	3	30 x 23	690	5520	4	22080	0.400	85	100	40.00	27.60	58.65	1005.94	761.53	
1	2	30 x 23	690	6210	4	24840	0.400	85	100	40.00	27.60	58.65	1120.48	847.78	------->> Last Steel Column
Conc. Pier	1	30 x 23	690	6900	4	27600	0.400	140	100	40.00	27.60	96.60	1280.56	971.98	------->> Concrete Pier

Gravity Column Redesign Comparison				
Hand Calcuations vs. RAM Design				
	Column Location	Hand Calculations		RAM
		Force (kips)	Size	Size Given
Design A	Typical Interior Column	1455.74	W12x152	W12x152
Design B	Atypical Interior Column (South Side)	1778.96	W12x190	W12x190
Design C	Typical Exterior Column (South Side)	1326.75	W12x136	W12x136
Design D	Typical Exterior Column (North Side)	712.36	W12x72	W12x79
Design E	Atypical Interior Column (North Side)	1120.48	W12x120	W12x120

Concrete Pier Reinforcement Design							
Hand Calculations							
	Column Location	Applied Loads		RAM	Steel	Checks	
		Axial	Moment	Rq'd Amt.	Configuation	Pnmax	Adequate?
Design A	Typical Interior Column	1664.54	468	13.82	(12) #10 bars	1714	YES
Design B	Atypical Interior Column (South Side)	1987.76	468	20.32	(16) # 11 bars	1996	YES
Design C	Typical Exterior Column (South Side)	1420.23	468	11.52	(12) #10 bars	1714	YES
Design D	Typical Exterior Column (North Side)	805.96	468	11.52	(12) #10 bars	1714	YES
Design E	Atypical Interior Column (North Side)	1280.56	468	11.52	(12) #10 bars	1714	YES

Allowable Bearing Stress for Fo **20 ksf**

Resizing of Pier Footings					
from Hand Calcuations					
	Column Location	Footing Loads		New Footing	Old Footing
		Force (kips)	Sq. Ft.	Redesigned Size	Previous Size
Design A	Typical Interior Column	1664.54	83.22679	9' x 10'	6' x 6'
Design B	Atypical Interior Column (South Side)	1987.76	99.38802	10' x 10'	6' x 6'
Design C	Typical Exterior Column (South Side)	1420.23	71.01127	8' x 10'	6' x 6'
Design D	Typical Exterior Column (North Side)	805.96	40.29798	7' x 7'	6' x 6'
Design E	Atypical Interior Column (North Side)	1280.56	64.0278	8' x 8'	6' x 6'

RAM Gravity Column Redesign Results (2nd Floor Column)

Row	Column	Given	Designed	Notes
H	RA	W10x33	W10x33	Entrance Canopy
H	15.3	W12x79	W12x136	Lateral System
H	16	W12x79	W12x136	Lateral System
H	17	W12x65	W12x79	
H	18	W12x96	W12x210	Lateral System
H	18.8	W12x45	W12x190	Lateral System
H	19.3	W12x45	W12x65	

Row	Column	Given	Designed	Notes
I	RB	W12x53	W12x65	
I	16	W12x72	W12x96	
I	17	W12x72	W12x120	
I	18	W12x120	W12x210	Lateral System
I	19	W12x65	W12x190	Lateral System
I	RC	W12x72	W12x106	

Row	Column	Given	Designed	Notes
J	RB	W12x53	W12x72	
J	16	W12x87	W12x120	
J	17	W12x96	W12x152	
J	18	W12x96	W12x152	
J	19	W12x87	W12x152	
J	20	W12x72	W12x120	
J	RC	W12x72	W12x106	

Row	Column	Given	Designed	Notes
K	RB	W12x53	W12x72	
K	16	W12x87	W12x120	
K	17	W12x106	W12x152	
K	18	W12x96	W12x190	
K	19	W12x96	W12x190	
K	20	W12x106	W12x210	
K	RC	W12x106	W12x170	Lateral System

Row	Column	Given	Designed	Notes
L	RB	W12x53	W12x65	
L	16	W12x96	W12x190	Lateral System
L	16.5	W12x79	W12x210	Lateral System
L	17	W12x40	W12x65	
L	18	W12x40	W12x40	One Floor
L	19	W12x40	W12x40	One Floor
L	20	W12x40	W12x40	One Floor
L	21	W12x40	W12x40	One Floor

Row	Column	Given	Designed	Notes
M	15.8	W12x45	W12x53	
M	16.5	W12x79	W12x210	Lateral System
M	17	W12x65	W12x65	
M	18	W12x87	W12x152	
M	19	W12x87	W12x152	
M	20	W12x72	W12x136	
M	21	W12x65	W12x136	
M	21.3	W12x45	W12x136	

Row	Column	Given	Designed	Notes
L3	RC	W12x45	W12x106	Lateral System
L7	RA	W10x33	W10x33	
L7	RB	W12x45	W12x45	

Row	Column	Given	Designed	Notes
RA	R7	W10x33	W10x33	Entrance Canopy
RA	R9	W10x33	W10x33	Entrance Canopy
RA	R11	W10x33	W10x33	Entrance Canopy
RA	R13	W10x33	W10x33	Entrance Canopy

Shear Wall Loads

North-South Direction

Lateral Frame # 1

W_1	16.82	kips
W_2	213.62	kips
W_3	14.96	kips
W_4	14.47	kips
W_5	186.3	kips
W_6	187.53	kips
W_7	172.83	kips
W_8	147.6	kips
W_9	151.88	kips
W_{10}	150.57	kips

Lateral Frame # 3

W_1	1.99	kips
W_2	204.49	kips
W_3	6.5	kips
W_4	-3.01	kips
W_5	159.24	kips
W_6	184.61	kips
W_7	148.86	kips
W_8	-145.87	kips
W_9	143.22	kips
W_{10}	117.28	kips

Lateral Frame # 5

W_1	-15.07	kips
W_2	120.84	kips
W_3	-15.21	kips
W_4	-11.17	kips
W_5	102.35	kips
W_6	91.62	kips
W_7	71.82	kips
W_8	-94.44	kips
W_9	57.35	kips
W_{10}	68.34	kips

Lateral Frame # 7

W_1	-3.87	kips
W_2	108.87	kips
W_3	-6.37	kips
W_4	-0.41	kips
W_5	90.05	kips
W_6	74.23	kips
W_7	-67.5	kips
W_8	-73.31	kips
W_9	50.96	kips
W_{10}	67.16	kips

East-West Direction

Lateral Frame # 2

W_1	81.32	kips
W_2	-12.47	kips
W_3	75.06	kips
W_4	67.25	kips
W_5	-21.39	kips
W_6	-0.43	kips
W_7	51.64	kips
W_8	70.34	kips
W_9	55.86	kips
W_{10}	34.51	kips
O_1	98.2	kips

Lateral Frame # 4

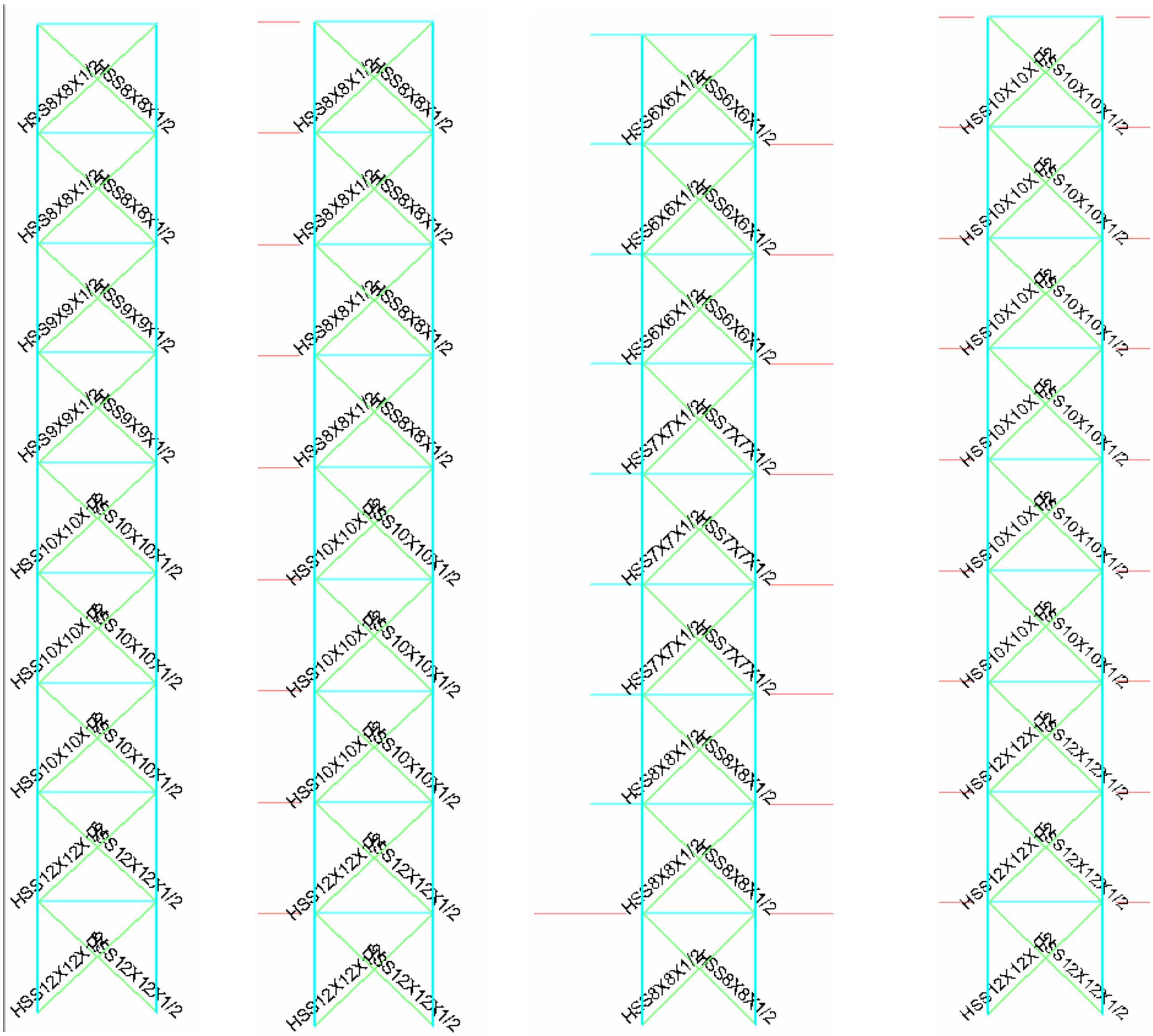
W_1	113.99	kips
W_2	9.63	kips
W_3	95.85	kips
W_4	103.63	kips
W_5	18.8	kips
W_6	-1.95	kips
W_7	92.71	kips
W_8	78.27	kips
W_9	70.52	kips
W_{10}	91.72	kips
O_1	136.4	kips

Lateral Frame # 6

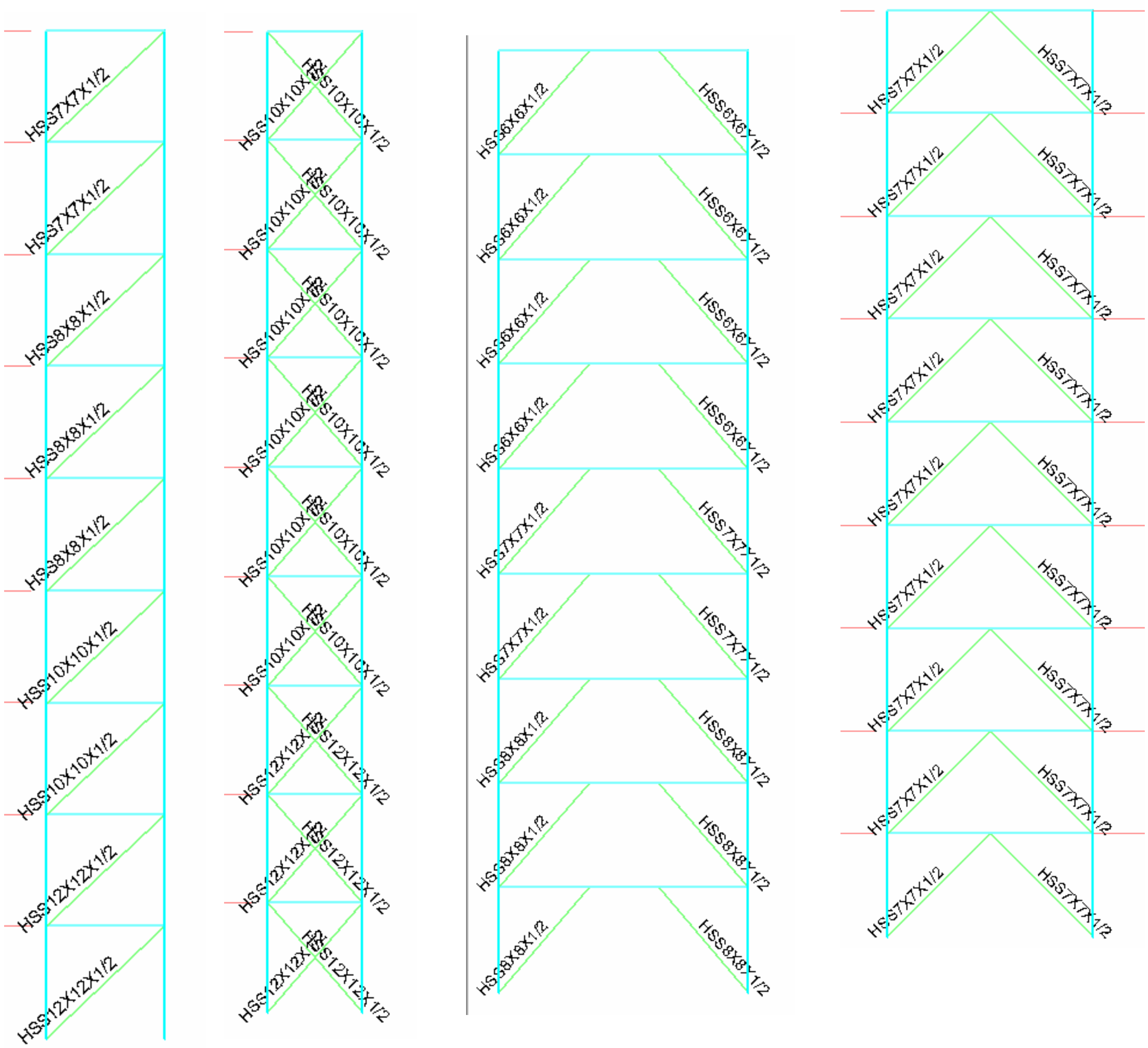
W_1	87.62	kips
W_2	27.24	kips
W_3	72.76	kips
W_4	80.58	kips
W_5	34.32	kips
W_6	13.36	kips
W_7	86.15	kips
W_8	45.28	kips
W_9	64.71	kips
W_{10}	86.06	kips
O_1	102.61	kips

Lateral Frame # 8

W_1	75.34	kips
W_2	-1.61	kips
W_3	69.1	kips
W_4	62.74	kips
W_5	-9.89	kips
W_6	7.06	kips
W_7	55.29	kips
W_8	57.71	kips
W_9	57.04	kips
W_{10}	39.72	kips
O_1	98.88	kips



Braced Frame Sections



Braced Frame Sections (Continued)

Shear Wall Check – Base Shear

E-W → FRAME #4 1

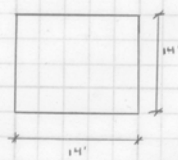
SHEAR WALL DESIGN

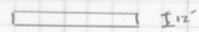
-Check Base Shear → Look @ Frame 4 → Worst in E-W Direction

→ $V_u = 136.40^k$

ELEVATION

PLAN VIEW





$t = 12"$
$b = (12)(14) = 168"$
$h = (12)(14) = 168"$

$$V_n = \frac{136.40^k}{.75} = 181.86^k$$

– BECAUSE SHEAR WALL SPANS BETWEEN TWO AXIAL COLUMNS, ASSUME SHEAR WALL ONLY TAKES SHEAR AND FLEXURE

$$V_c = 2\sqrt{f'c} \cdot b_w \cdot d = 2\sqrt{5000} \cdot (168)(12") = 285 \text{ K}$$

$$\frac{\phi V_c}{2} = \frac{(.65)(285)}{2} = 92^k \therefore \text{REINFORCEMENT REQUIRED}$$

→ TRY EXISTING REINFORCEMENT

→ Try #5 bars @ 12" o.c. for EACH FACE

$$A_v = .75\sqrt{5000} \cdot \frac{(12)(12)}{50} = .152$$

MUST BE GREATER THAN $(50 \cdot 12 \cdot 12)/_{60,000} = .12$

→ USE #5 bars @ 12" ON EACH FACE

d bending in STRONG AXIS ∴ d = 168"

$$V_s = \frac{(2 \cdot .31)(60,000)(168)}{12} = 520.8^K$$

$$\phi V_n = (.75)(520.8 + 285) = 604^k > 181.86^K$$

∴ SHEAR WALL DESIGN ADEQUATE FOR BASE SHEAR

Shear Wall Check – Overturning Moment

E-W → FRAME #4

- CHECK FRAME 4 FOR OVERTURNING MOMENT

→ from seismic loading = 12,245 ft kips on Frame 4

$t = 12''$
$b = 168''$
$h = 168''$

FIND LOADS ACTING ON COLUMNS

$A_T = 400\ ft^2$

$DL = 80$ psf
$LL = 100$ psf
$S_{DL} = 20$ psf

INTERIOR COLUMN $K_{LL} = 4$

$A_T = (10)(400) = 4000$

$(4)(4000) = 16000$

$LLR = .25\left(\frac{15}{\sqrt{16000}}\right) = .4$

$LL = (4000)(100)(.4)\ \ 160^K$

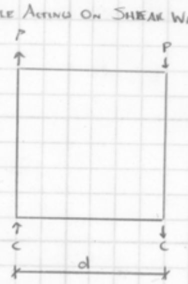
$DL = (140.75)(400) + 9(83.25)(400)$

$= 356^K$

$1.2(356K) + 1.6(160K) = 683.2^K$ →

NEED TO ALSO CONSIDER WEIGHT OF FOUNDATION

→ COUPLE ACTING ON SHEAR WALL



$Couple = M_u/d = \frac{12,245}{14} = 874.64^K$

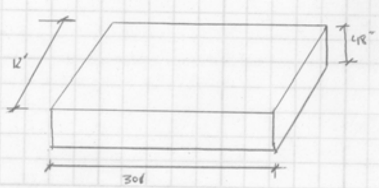
→ FOUNDATION UNDER REPRESENTATIVE SHEAR WALL is $30' \times 8' \times 4'$

$w = (4')(150)(8')(30') = 144^K$

$144^K + 683.2^K = 827.2^K < 874.64^K$

→ lager footing required

→ Try a $30' \times 12' \times 4'$ FOUNDATION



$w = (4')(150\ pcf)(12')(30') = 216^K$

$216^K + 683.2^K = 899.2^K$

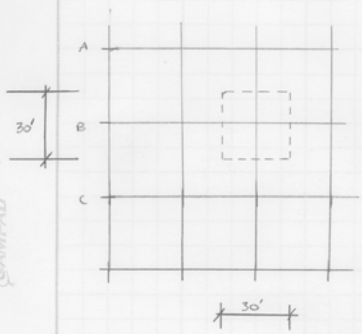
$899.2^K > 874.64^K$

$\therefore$ OK

Concrete Pier Design Moment

Concrete Pier Moments – INTERIOR Columns

→ 1st Floor Bays → 30' x 30'



$A_T = 30' \times 30' = 900\ ft^2$

$A_I = 4(900) = 3600\ ft^2$

k_{LL}

$LL_{reduction} = Lo\left(.25 + \frac{15}{\sqrt{3600}}\right) = .5$

Loads

DEAD LOAD

– Floor – 6" slab and 20 psf S.D.L

– GIRDERS – 24" x 30" → span from col. to col.

DEAD LOAD

$- (6/12")(150\ pcf) + (18/12)(24/12)(150\ pcf)(1/10)$

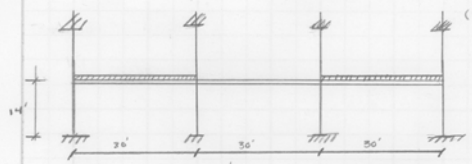
$+ (24/12)(30/12)(150\ pcf)(1/30) + 20\ psf.$

$= 165\ psf \cdot (30') = 4.95\ k/ft$

LIVE LOAD

100 psf

$(100\ psf)(.5)(30) = 1.5\ k/ft$



→ All bays 30' x 30'

– WORST CASE SENARIO: ALL LL ON ONE SIDE, NONE ON THE OTHER
– CAUSES MOMENT

– FIXED ENDS MOMENT – CONCRETE POURED MONOLITHICALLY

$- \frac{wl^2}{12}$

→ FULLY LOADED BAY → $1.2(4.95) + 1.6(1.5) = 8.34\ k/ft$

$\frac{(8.34)(30)^2}{12} = 625.5$ ft-kips

→ Dead LOAD ONLY → $1.4(1.5) = 2.1\ k/ft$

$\frac{(2.1)(30)^2}{12} = 157.5$

$\Delta FEM = 625.5 - 157.5$

$\boxed{\Delta FEM = 468\ ft\text{-}kips}$

– Assume 100% of moment goes into column due to no moment above

Concrete Pier Reinforcement Design

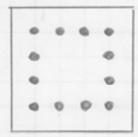
1

→ Typical Interior Column
w/ 30'x30' bay on all Floors

$P_u = 1665$ k
$M_u = 468$ ft-kips

→ 24"x24" concrete pier
- Square ties
→ $f'_c = 5$ ksi
– 1 ½" concrete cover

Try 12 bar configuration



→ USE Design Aid interaction diagram to estimate steel reinforcement

$\gamma = 24 - 2(1.5+.5+.5) = 19"$ $\gamma = 19/24 = .79 \rightarrow$ use .8 $A_g = 24 \times 24 = 576\ in^2$

$$R_n = \frac{M_u}{(\phi \cdot f_c \cdot A_g \cdot h)} = \frac{(468)(12)}{(.65 \cdot 5 \cdot 576 \cdot 24)} = 0.125$$

→ Look @ CRSI HANDBOOK @ 0% F_y
(12) #10's most efficient $\phi M_n = 6204 > 5616$ ∴ OK
$\phi P_n = 1547 > 1540$ ∴ OK

$$K_n = \frac{P_u}{\phi f_c \cdot A_g} = \frac{1665}{(.65 \cdot 5 \cdot 576)} = 0.88$$

→ Go into interaction diagram → $\rho = 0.024$

$A_{ST} = (576)(.024) = 13.82"$
→ Use (12) #10 bars
$A_{ST} = 15.24\ in^2$

Check

$$\phi P_{n_{max}} = 0.80 \cdot \phi [.85 \cdot f_c \cdot (A_g - A_{ST}) + f_y \cdot A_{ST}]$$

$$= 0.80 \cdot .65 [.85 \cdot 5 \cdot (576 - 15.24) + 60 \cdot 15.24] = 1714^k > 1665^k \therefore OK$$

→ Typical Interior Column
w/ 30'x45' bays on Floors 3 through Roof

$P_u = 1980$ K
$M_u = 468$ ft-kips

$$R_n = \frac{M_u}{(\phi \cdot f'_c \cdot A_g \cdot h)} = \frac{(468 \cdot 12)}{(.65 \cdot 5 \cdot 576 \cdot 24)} = 0.125$$

→ Go into interaction diagram → $\rho = 0.034$

$$K_n = \frac{P_u}{(\phi \cdot f_c \cdot A_g)} = \frac{1980}{(.65 \cdot 5 \cdot 576)} = 1.05$$

$A_{ST} = (576)(0.034) = 19.58$
→ Use (16) #10 bars
$A_{ST} = 20.32"$

Check

$$\phi P_{n_{max}} = 0.80 \cdot \phi [.85 \cdot f_c \cdot (A_g - A_{ST}) + f_y \cdot A_{ST}]$$

$$= 0.80 \cdot .65 [.85 \cdot 5 \cdot (576 - 20.32) + 60(20.32)] = 1862^k > 1830^k \therefore ok$$

→ Look @ CRSI Handbook @ 0% f_y
(16) #11's more efficent $\phi P_n = 2150^k$

→ Use (16) #11 bars $A_{ST} = 24.96"$

$$= .8 \cdot .65 [.85 \cdot 5 \cdot (576 - 24.96) + 60(24.96)] = \underline{1996^k > 1980^k} \therefore OK$$

→ EXTERIOR COLUMN

→ SOUTH SIDE → (WORSE SENARIO THAN NORTH SIDE)

$P_u = 1426$

$M_u = 625.5$

$R_n = \dfrac{(625.5 \cdot 12)}{(.65 \cdot 5 \cdot 576 \cdot 24)} = 0.16$

$K_n = \dfrac{1420}{(.65 \cdot 5 \cdot 576)} = .75$

$\rho = 0.02$

→ $A_{ST} = (.02)(576) = 11.52$

→ Try (12) #10 bars

$A_{ST} = 15.24"$

Check

$\varnothing P_{n_{max}} = 0.80 \cdot .65\left[.85 \cdot 5 \cdot (576 - 15.24) + 60(15.24)\right] = 1714^k > 1426^k$

→ Look @ CRSI Handbook

$\boxed{\begin{aligned} \varnothing M_n &= 6204 > 5616^k \\ \varnothing P_n &= 1547 > 1426^k \end{aligned}}$

→ Use (12) #10 bars

APPENDIX B

SQUARE TIED COLUMNS 22" × 22"

Short columns; no sidesway
Bars symmetrical in 4 faces

f'_c = 5,000 psi ϕM in inch-kips
f_y = 60,000 psi ϕP in kips

BARS	RHO	Max Cap φM	Max Cap φP	0% fy φM	0% fy φP	25% fy φM	25% fy φP	50% fy φM	50% fy φP	100% fy φM	100% fy φP	.1f'c Ag φM	.1f'c Ag φP	Zero Axial Load φM
4-#10	1.05	2931	1310	4111	1120	4855	943	5243	805	5604	596	3978	242	2575
4-#11	1.29	2998	1347	4326	1132	5088	951	5506	807	5943	589	4330	242	3087
4-#14	1.86	3172	1433	4780	1178	5637	984	6157	827	6811	580	5214	242	4298
4-#18	3.31	3566	1651	5868	1300	6963	1071	7738	879	8922	561	7363	242	7208
8-# 8	1.31	2885	1349	4035	1158	4785	972	5172	826	5520	606	4340	242	3180
8-# 9	1.65	2969	1402	4255	1189	5051	995	5488	839	5942	603	4866	242	3947
8-#10	2.10	3075	1469	4532	1229	5389	1024	5890	857	6478	600	5478	242	4911
8-#11	2.58	3166	1542	4831	1265	5729	1049	6282	871	6991	591	6044	242	5869
8-#14	3.72	3412	1714	5496	1370	6545	1124	7258	916	8297	582	7456	242	8157
8-#18	6.61	4011	2151	7097	1636	8522	1316	9628	1033	11474	556	10757	242	12884
12-#10	3.15	3333	1628	5080	1338	6050	1112	6687	917	7545	596	6797	242	7035
12-#11	3.87	3471	1736	5486	1399	6521	1156	7236	943	8269	584	7667	242	8224
12-#14	5.58	3866	1995	6423	1561	7665	1278	8607	1019	10104	570	9691	242	11123
16-#10	4.20	3574	1786	5660	1448	6760	1193	7537	970	8676	599	8115	242	8943
16-#11	5.16	3773	1931	6180	1532	7369	1255	8253	1006	9622	586	9191	242	10556
20-#10	5.25	3819	1945	6208	1564	7489	1274	8396	1030	9834	602	9420	242	10798

SQUARE TIED COLUMNS 24" × 24"

BARS	RHO	Max Cap φM	Max Cap φP	0% fy φM	0% fy φP	25% fy φM	25% fy φP	50% fy φM	50% fy φP	100% fy φM	100% fy φP	.1f'c Ag φM	.1f'c Ag φP	Zero Axial Load φM
4-#11	1.08	3825	1566	5367	1338	6348	1126	6866	961	7359	711	5247	288	3446
4-#14	1.56	4028	1652	5883	1384	6969	1159	7601	980	8338	702	6244	288	4812
4-#18	2.78	4494	1870	7123	1504	8476	1245	9395	1031	10731	682	8681	288	8112
8-# 8	1.10	3692	1568	5028	1366	5999	1150	6483	981	6878	730	5248	288	3540
8-# 9	1.39	3790	1621	5277	1397	6299	1172	6838	995	7350	727	5853	288	4403
8-#10	1.76	3912	1688	5589	1437	6678	1202	7289	1012	7951	724	6596	288	5489
8-#11	2.17	4019	1760	5933	1473	7067	1226	7736	1026	8534	715	7255	288	6580
8-#14	3.13	4303	1933	6687	1577	7989	1302	8837	1072	10006	706	8883	288	9180
8-#18	5.56	5000	2370	8508	1844	10234	1494	11525	1189	13606	684	12700	288	15057
12-#10	2.65	4206	1847	6204	1547	7420	1291	8182	1074	9148	721	8035	288	7994
12-#11	3.25	4368	1955	6670	1607	7957	1335	8808	1100	9970	710	9032	288	9407
12-#14	4.69	4797	2214	7732	1770	9251	1458	10357	1177	12042	698	11498	288	12717
16-#10	3.53	4479	2005	6854	1658	8216	1373	9136	1128	10416	726	9554	288	10118
16-#11	4.33	4691	2150	7450	1742	8912	1435	9952	1165	11492	715	10800	288	11978
16-#14	6.25	5270	2495	8837	1963	10604	1600	11978	1269	14200	702	13851	288	16273
20-#10	4.41	4734	2164	7461	1773	9034	1455	10100	1189	11700	724	11036	288	12313
20-#11	5.42	5021	2345	8186	1884	9892	1535	11107	1239	13049	719	12534	288	14458

(1) "0% f_y" indicates zero tension in bars on the tension side, "50% f_y" indicates 50% f_y stress in bars on the tension side, and "100% f_y" indicates 100% f_y stress (i.e., balance point) in bars on the tension side.

CONCRETE REINFORCING STEEL INSTITUTE

SQUARE TIED COLUMNS 26" × 26"

Short columns; no sidesway
Bars symmetrical in 4 faces

f'_c = 5,000 psi ϕM in inch-kips
f_y = 60,000 psi ϕP in kips

BARS	RHO	Max Cap φM	Max Cap φP	0% fy φM	0% fy φP	25% fy φM	25% fy φP	50% fy φM	50% fy φP	100% fy φM	100% fy φP	.1f'c Ag φM	.1f'c Ag φP	Zero Axial Load φM
4-#14	1.33	5023	1890	7140	1608	8496	1350	9256	1146	10078	835	7397	338	5325
4-#18	2.37	5561	2108	8533	1727	10184	1435	11262	1197	12752	814	10120	338	9015
8-# 9	1.18	4748	1859	6449	1624	7739	1366	8398	1164	8972	862	6949	338	4857
8-#10	1.50	4887	1926	6798	1664	8161	1395	8898	1182	9638	859	7802	338	6065
8-#11	1.85	5011	1998	7188	1699	8598	1419	9400	1194	10290	850	8596	338	7289
8-#14	2.66	5335	2171	8031	1803	9627	1495	10627	1241	11929	842	10445	338	10200
8-#18	4.73	6135	2608	10074	2070	12140	1688	13632	1360	15952	821	14792	338	17166
12-#10	2.25	5220	2085	7479	1775	8983	1486	9888	1244	10963	858	9397	338	8861
12-#11	2.77	5405	2193	8007	1834	9587	1530	10590	1270	11884	847	10521	338	10614
12-#14	3.99	5892	2452	9194	1998	11031	1653	12317	1348	14193	836	13292	338	14362
12-#18	7.10	7121	3107	12076	2413	14558	1965	16545	1547	19852	805	19485	338	23230
16-#10	3.01	5524	2243	8200	1886	9865	1569	10945	1299	12369	864	11103	338	11317
16-#11	3.69	5766	2388	8874	1970	10648	1631	11861	1336	13575	853	12522	338	13415
16-#14	5.33	6393	2733	10424	2192	12538	1797	14122	1443	16594	843	15955	338	18476
20-#10	3.76	5811	2402	8866	2001	10772	1652	12014	1361	13792	863	12789	338	13850
20-#11	4.62	6108	2583	9682	2111	11738	1731	13145	1412	15286	850	14483	338	16306

SQUARE TIED COLUMNS 28" × 28"

BARS	RHO	Max Cap φM	Max Cap φP	0% fy φM	0% fy φP	25% fy φM	25% fy φP	50% fy φM	50% fy φP	100% fy φM	100% fy φP	.1f'c Ag φM	.1f'c Ag φP	Zero Axial Load φM
4-#14	1.15	6169	2147	8561	1852	10232	1558	11137	1328	12048	979	8681	392	5837
4-#18	2.04	6781	2365	10109	1970	12103	1642	13358	1377	15004	957	11693	392	9916
8-# 9	1.02	5856	2116	7785	1870	9388	1576	10185	1347	10824	1008	8174	392	5311
8-#10	1.30	6012	2183	8170	1910	9852	1605	10735	1365	11555	1005	9113	392	6640
8-#11	1.59	6154	2255	8607	1944	10339	1629	11291	1377	12276	996	10047	392	7996
8-#14	2.30	6519	2428	9540	2048	11475	1704	12644	1424	14081	988	12152	392	11218
8-#18	4.08	7385	2865	11806	2315	14257	1898	15968	1543	18528	969	17041	392	18973
12-#10	1.94	6383	2342	8918	2021	10754	1696	11821	1428	13009	1005	10883	392	9725
12-#11	2.39	6594	2450	9507	2080	11426	1740	12600	1454	14028	994	12144	392	11679
12-#14	3.44	7140	2709	10821	2244	13021	1864	14505	1533	16574	985	15194	392	16057
12-#18	6.12	8522	3364	14018	2660	16927	2178	19184	1734	22834	957	22257	392	25955
16-#10	2.59	6721	2500	9710	2133	11724	1780	12982	1484	14553	1012	12790	392	12539
16-#11	3.18	6994	2645	10461	2217	12593	1842	13998	1521	15888	1002	14374	392	14878
16-#14	4.59	7694	2990	12176	2440	14681	2010	16493	1629	19219	994	18185	392	20499
20-#10	3.24	7040	2659	10434	2247	12719	1864	14155	1547	16116	1012	14663	392	15291
20-#11	3.98	7374	2840	11341	2358	13793	1944	15411	1598	17770	1000	16582	392	18211
20-#14	5.74	8235	3271	13436	2644	16385	2156	18502	1739	21894	990	21153	392	24793
24-#11	4.78	7801	3035	12274	2489	14958	2053	16848	1665	19681	1008	18748	392	21276