STRUCTURAL CALCULATIONS

For

BUILDING 3030 REMODEL (PHASES 2 THROUGH 4)
UTAH NATIONAL GAURD
CAMP W.G. WILLIAMS

Prepared for

CRSA ARCHITECTS
649 E South Temple
Salt Lake City, UT 84102

By

PROFESSIONAL STRUCTURAL ENGINEER
STEPHEN D. COHEN
#176558
STATE OF UTAH
8/30/2010

CALDER RICHARDS
CONSULTING ENGINEERS
634 S. 400 W, Suite 100
Salt Lake City, UT 84101
801-466-1699

PHASE 2-4

CALDER RICHARDS CONSULTING ENGINEERS

JOB TITLE Camp Williams Bldg [illegible] BY [illegible] DATE [illegible]

SUBJECT LOAD SHEET CHECKED SHEET 1 OF 31

ROOF LOADS

DEAD LOAD: LIVE LOAD:

N.A. I_S =

FLOOR LOADS

DEAD LOAD:

4" B-deck (20ga) = 38.9 psf
framing = 6.0 psf
misc = 15.1 psf
Σ 60.0 psf

LIVE LOAD:

LL = 50 psf + 20 (partition)
= 70 psf

SEISMIC

S_{DS} = 0.788 S_{D1} = 0.473 I_E = 1.0 R = 5.0 ← Special Reinf Masonry Shearwall

Seismic Design Category = D Site Class = D

WIND

V_{3S} = V_{fm} = I_W =

Exposure Category = N/A (Inside existing bldg)

SOILS

Allowable Soil Bearing Pressure = 1500 psf (Assumed, & to be field verified)

40.436356 -111.925302

3/3'

LIGHT STEEL FRAMING MEMBERS — COMBINED AXIAL & LATERAL LOADS

NO LATERAL LOAD FOR LOAD COMBINATION 1.0(D+L)

Wall Height (ft.)	Spacing (in.) o.c.	600S250-(mils) 33 ksi 43	600S250-(mils) 50 ksi 54	600S250-(mils) 50 ksi 68	600S250-(mils) 50 ksi 97	800S137-(mils) 33 ksi 33	800S137-(mils) 33 ksi 43	800S137-(mils) 50 ksi 54	800S137-(mils) 50 ksi 68	800S137-(mils) 50 ksi 97	800S162-(mils) 33 ksi 33	800S162-(mils) 33 ksi 43	800S162-(mils) 50 ksi 54	800S162-(mils) 50 ksi 68	800S162-(mils) 50 ksi 97	800S200-(mils) 33 ksi 43	800S200-(mils) 50 ksi 54	800S200-(mils) 50 ksi 68	800S200-(mils) 50 ksi 97
8	12	4.89	8.40	11.85	18.17	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.07	4.58	7.85	10.38
	16	4.89	8.40	11.85	18.17	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.07	4.58	7.85	10.38
	24	4.89	8.40	11.85	18.17	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.07	4.58	7.85	10.38
9	12	4.85	8.29	11.66	17.87	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.07	4.58	7.85	10.38
	16	4.85	8.29	11.66	17.87	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.07	4.58	7.85	10.38
	24	4.85	8.29	11.66	17.87	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.07	4.58	7.85	10.38
10	12	4.80	8.16	11.43	17.52	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.07	4.58	7.85	10.38
	16	4.80	8.16	11.43	17.52	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.07	4.58	7.85	10.38
	24	4.80	8.16	11.43	17.52	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.07	4.58	7.85	10.38
12	12	4.68	7.84	10.85	16.61	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.06	4.58	7.85	10.38
	16	4.68	7.84	10.85	16.61	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.06	4.58	7.85	10.38
	24	4.68	7.84	10.85	16.61	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.06	4.58	7.85	10.38
14	12	4.51	7.43	10.12	15.48	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.03	4.50	7.68	10.21
	16	4.51	7.43	10.12	15.48	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.03	4.50	7.68	10.21
	24	4.51	7.43	10.12	15.48	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	3.03	4.50	7.68	10.21
16	12	4.31	6.98	9.28	14.18	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	2.97	4.40	7.40	9.85
	16	4.31	6.98	9.28	14.18	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	2.97	4.40	7.40	9.85
	24	4.31	6.98	9.28	14.18	1.79	2.51	3.63	4.81	7.28	2.49	3.45	5.52	7.33	11.34	2.97	4.40	7.40	9.85

NO LATERAL LOAD — LATERAL LOAD = 5 PSF (X 0.75 FOR STRENGTH & BY 0.70 FOR DEFLECTION DETERMINATIONS)

Wall Height (ft.)	Spacing (in.) o.c.	800S250-(mils) 33 ksi 43	800S250-(mils) 50 ksi 54	800S250-(mils) 50 ksi 68	800S250-(mils) 50 ksi 97	350S162-(mils) 33 ksi 33	350S162-(mils) 33 ksi 43	350S162-(mils) 50 ksi 54	350S162-(mils) 50 ksi 68	362S137-(mils) 33 ksi 33	362S137-(mils) 33 ksi 43	362S137-(mils) 50 ksi 54	362S137-(mils) 50 ksi 68	362S162-(mils) 33 ksi 33	362S162-(mils) 33 ksi 43	362S162-(mils) 50 ksi 54	362S162-(mils) 50 ksi 68
8	12	5.12	8.97	12.76	19.97	1.86	2.60	3.98	5.05	1.54	2.17	3.28	4.29	1.92	2.69	4.13	5.36
	16	5.12	8.97	12.76	19.97	1.81	2.55	3.93	5.00	1.50	2.13	3.24	4.25	1.87	2.64	4.09	5.31
	24	5.12	8.97	12.76	19.97	1.70	2.45	3.84	4.91	1.42	2.03	3.16	4.16	1.77	2.54	4.00	5.22
9	12	5.10	8.93	12.71	19.86	1.74	2.45	3.67	4.66	1.46	2.07	3.09	4.01	1.81	2.55	3.87	4.98
	16	5.10	8.93	12.71	19.86	1.67	2.39	3.61	4.60	1.40	2.01	3.04	3.96	1.74	2.48	3.81	4.92
	24	5.10	8.93	12.71	19.86	1.54	2.26	3.50	4.48	1.30	1.89	2.95	3.86	1.61	2.36	3.70	4.81
10	12	5.08	8.88	12.64	19.71	1.60	2.28	3.32	4.22	1.36	1.94	2.86	3.69	1.67	2.38	3.57	4.55
	16	5.08	8.88	12.64	19.71	1.52	2.20	3.26	4.15	1.30	1.87	2.80	3.63	1.60	2.30	3.50	4.48
	24	5.08	8.88	12.64	19.71	1.37	2.04	3.13	4.01	1.17	1.73	2.68	3.51	1.44	2.15	3.37	4.34
12	12	5.03	8.73	12.44	19.30	1.30	1.90	2.66	3.37	1.14	1.64	2.36	3.03	1.38	2.01	2.89	3.66
	16	5.03	8.73	12.44	19.30	1.20	1.79	2.57	3.28	1.05	1.55	2.28	2.94	1.28	1.90	2.80	3.57
	24	5.03	8.73	12.44	19.30	1.01[7]	1.59	2.41	3.12	0.89[7]	1.37	2.13	2.79	1.09	1.71	2.64	3.40
14	12	4.95	8.51	12.07	18.70	1.00	1.50	2.08	2.65	0.90	1.32	1.88	2.41	1.08	1.62	2.28	2.89
	16	4.95	8.51	12.07	18.70	0.89[7]	1.38	1.99	2.55	0.80[6]	1.21	1.79	2.32	0.97[7]	1.49	2.18	2.79
	24	4.95	8.51	12.07	18.70	0.69[6]	1.17[6]	1.82[7]	2.38	0.62[6]	1.01[6]	1.63[7]	2.14	0.76[6]	1.27[6]	2.00[7]	2.61
16	12	4.83	8.22	11.54	17.91	0.74[6]	1.13[7]	1.63	2.08	0.67[6]	1.01[7]	1.48	1.90	0.81[6]	1.24	1.78	2.28
	16	4.83	8.22	11.54	17.91	0.63[6]	1.01[6]	1.53[7]	1.98	0.57[6]	0.90[6]	1.38	1.80	0.69[6]	1.12[6]	1.68[7]	2.18
	24	4.83	8.22	11.54	17.91	0.43[3]	0.80[3]	1.36[6]	1.80[6]	0.40[3]	0.69[3]	1.22[6]	1.62[6]	0.49[3]	0.90[6]	1.50[6]	1.98[6]

1 = Deflection Exceeds L/120
2 = Deflection Exceeds L/240
3 = Deflection Exceeds L/360
6 = Deflection Exceeds L/600
7 = Deflection Exceeds L/720
If not noted, deflection is less than L/720

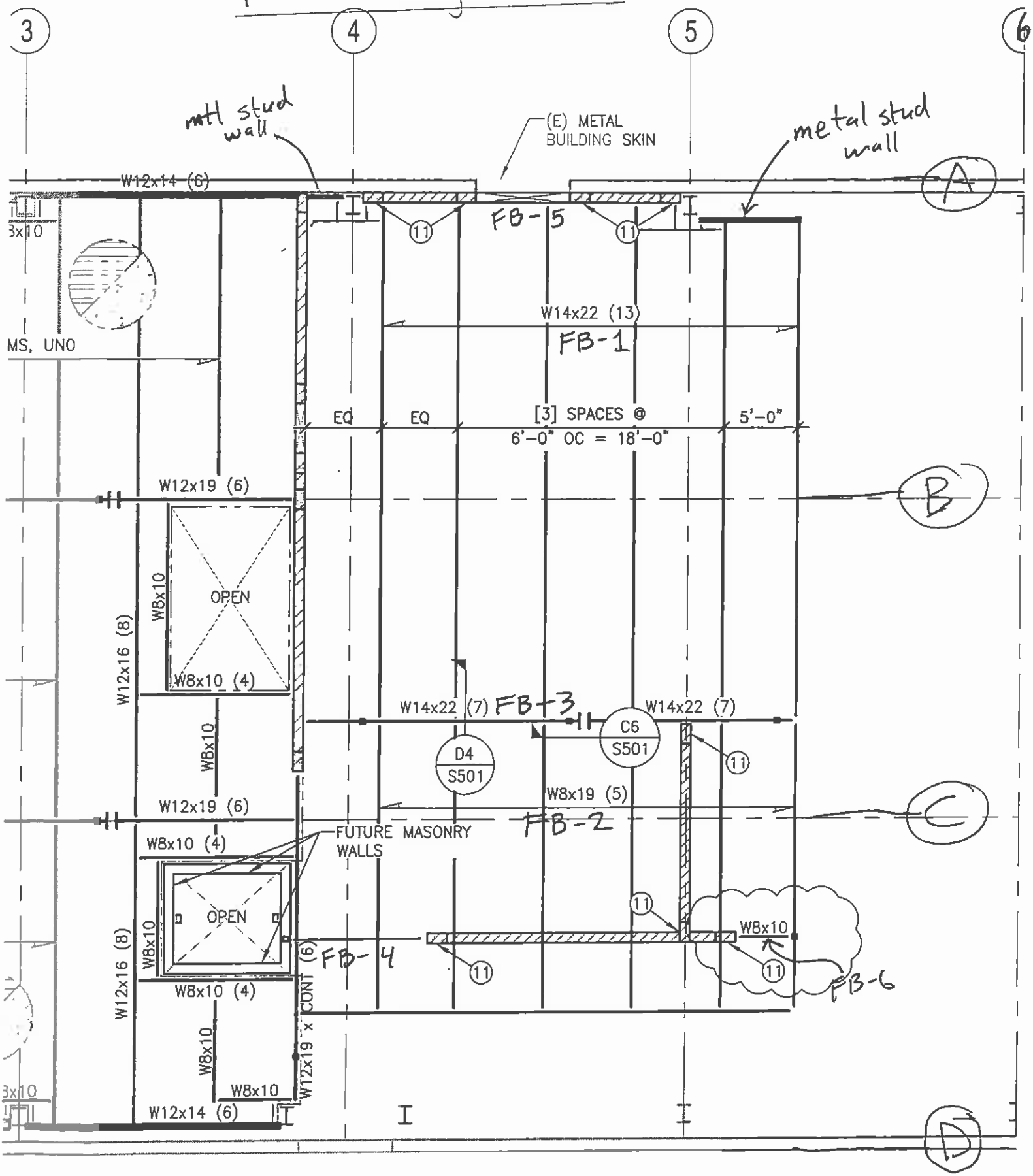
$P = 17'(60+70) \times \frac{16''}{12}$
$= 2.95k$
$< 3.50k$ ✓

Important Notes:

- Allowable loads are based on specified weak axis and torsional bracing at 48" o.c. maximum for axial load calculation and continuous support of each flange for flexural calculation.
- Sections are punched with a standard punchout 1.5" wide, located on the centerline of the web 24" o.c.
- Allowable loads are based on a punched section for axial load calculation, and an unpunched section for flexural calculation as per *ICC-ES AC46-Appendix B*.
- Weak axis and torsional bracing should have sufficient stiffness and strength to resist axial load.
- Lateral load is multiplied by 0.75 for strength determination and by 0.70 for deflection determination. Axial load is not multiplied by any factor (Load factor = 1.0)
- Strength increase due to **cold forming is incorporated** in calculating allowable loads as per ***AISI** Specification* **Sec. A7.2.**
- Check lateral end reactions **for web crippling if** applicable.
- Loads in tables are in kips/stud.

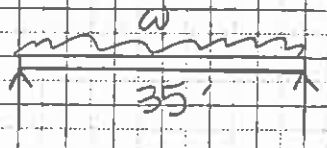
∴ May use 3⅝" x 1⅝" x 16ga metal studs @ 16" oc.

Floor Framing Keyplan



See Keyplan (previous page) for beam call outs.

FB-1 (Typical Long-Span Joists)



$\omega_{DL} = 6'(60psf) = 360plf$

$\omega_{P.C} = 6'(42psf) = 252plf$

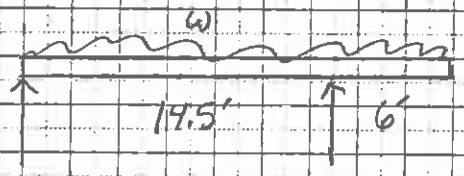
$\omega_{LL} = 6'(50+20)psf = 420plf$

Pre-composite

W14x22 (13) w/ ½" camber

See following pages.

FB-2 (Typical Short-Span Joists)



$\omega_{DL} = 360plf$

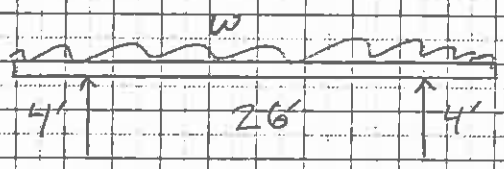
$\omega_{P-C} = 252plf$

$\omega_{LL} = 420plf$

W8x10 (5)

use for FB-6 & misc beams

FB-3



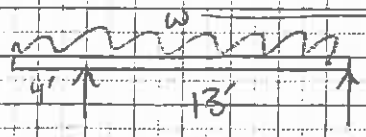
$\omega_{DL} = 25'(60) = 1500plf$

$\omega_{P-C} = 25'(42) = 1050plf$

$\omega_{LL} = 25'(70) = 1750plf$

W21x55 (15)

FB-3 (option 2)



~~W10x19 (7)~~

use W14x22 (7)

Gravity Beam Design

RAM SBeam v3.0
Camp Williams 3030 phase 2-4
FB-1
Licensed to:

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STEEL CODE: AISC LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (35.00,0.00)

Beam Size (Optimum) = W14X22 Fy = 50.0 ksi
Total Beam Length (ft) = 35.00
Distance to Adjacent Beam on Left (ft) = 6.0
Distance to Adjacent Beam on Right (ft) = 6.0

COMPOSITE PROPERTIES (Not Shored):

		Left			Right
Concrete thickness (in)		2.50			2.50
Unit weight concrete (pcf)		145.00			145.00
f'c (ksi)		4.00			4.00
Decking Orientation		perpendicular			perpendicular
Decking type		ASC B-36			ASC B-36
beff (in)	= 72.00	Y bar(in)		=	14.31
Mnf (kip-ft)	= 275.48	Mn (kip-ft)		=	216.81
C (kips)	= 117.48	PNA (in)		=	11.96
Ieff (in4)	= 485.73	Itr (in4)		=	675.54
Stud length (in)	= 3.50	Stud diam (in)		=	0.75

Stud Capacity (kips) Qn = 19.6
# of studs. Full = 34 Partial = 13 Actual = 13
Number of Stud Rows = 1 Percent of Full Composite Action = 36.20
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	CDL	LL	CLL
1	0.000	0.360	0.252	0.420	0.084
	35.000	0.360	0.252	0.420	0.084

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 19.32 kips 0.90Vn = 85.08 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	66.9	17.5	0.0	1.00	0.90	124.50
	Init DL	1.4DL	54.0	17.5	---	---		
	Max +	1.2DL+1.6LL	169.0	17.5	---	---	0.85	184.29
Controlling		1.2DL+1.6LL	169.0	17.5	---	---	0.85	184.29

REACTIONS (kips):

	Left	Right
Initial reaction	5.88	5.88
DL reaction	6.30	6.30
Max +LL reaction	7.35	7.35
Max +total reaction (factored)	19.32	19.32

DEFLECTIONS: (Camber = 1)

Initial load (in)	at	17.50 ft	=	-1.474	L/D =	285
Live load (in)	at	17.50 ft	=	-1.007	L/D =	417

Gravity Beam Design

RAM SBeam v3.0
Camp Williams 3030 phase 2-4
FB-1
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Post Comp load (in)	at	17.50 ft	=	-1.266	L/D =	332
Net Total load (in)	at	17.50 ft	=	-1.740	L/D =	241

Gravity Beam Design

RAM SBeam v3.0
Camp Williams 3030 phase 2-4
FB-2
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STEEL CODE: AISC LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (20.50,0.00)

Beam Size (Optimum) = W8X10 Fy = 50.0 ksi
Total Beam Length (ft) = 20.50
Cantilever on right (ft) = 6.00
Distance to Adjacent Beam on Left (ft) = 6.0
Distance to Adjacent Beam on Right (ft) = 6.0

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	2.50	2.50
Unit weight concrete (pcf)	145.00	145.00
f'c (ksi)	4.00	4.00
Decking Orientation	perpendicular	perpendicular
Decking type	ASC B-36	ASC B-36

beff (in)	=	43.50	Y bar(in)	=	9.45
Mnf (kip-ft)	=	91.82	Mn (kip-ft)	=	58.68
C (kips)	=	39.16	PNA (in)	=	6.25
Ieff (in4)	=	90.56	Itr (in4)	=	146.97
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 19.6
# of studs: Full = 18 Partial = 5 Actual = 5
Number of Stud Rows = 1 Percent of Full Composite Action = 26.44
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	CDL	LL	CLL
1	0.000	0.360	0.252	0.420	0.084
	14.500	0.360	0.252	0.420	0.084
2	14.500	0.360	0.252	0.420	0.084
	20.500	0.360	0.252	0.420	0.084

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 9.37 kips 0.90Vn = 36.22 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	8.9	6.4	0.0	1.00	0.90	32.92
	PreCmp-	1.2DL+1.6LL	-7.9	14.5	14.5	2.05	0.90	18.97
	Init DL	1.4DL	6.4	6.0	---	---		
	Max +	1.2DL+1.6LL	25.3	6.8	---	---	0.85	49.88
	Max -	1.2DL+1.6LL	-19.9	14.5	14.5	2.92	0.90	26.97
Right	PreCmp-	1.2DL+1.6LL	-7.9	14.5	6.0	1.00	0.90	27.21
	Max -	1.2DL+1.6LL	-19.9	14.5	6.0	1.00	0.90	27.21
Controlling		1.4DL	-9.1	14.5	14.5	1.26	0.90	11.69

REACTIONS (kips):

	Left	Right
Initial reaction	2.02	4.87
DL reaction	2.16	5.22
Max +LL reaction	3.04	6.09
Max -LL reaction	-0.52	0.00
Max +total reaction (factored)	7.47	16.00

DEFLECTIONS:

Center span:

Initial load (in)	at	6.89 ft	=	-0.167	L/D	=	1044
Live load (in)	at	6.89 ft	=	-0.159	L/D	=	1097
Post Comp load (in)	at	6.89 ft	=	-0.183	L/D	=	951
Net Total load (in)	at	6.89 ft	=	-0.350	L/D	=	498

Right cantilever:

Init load (in)	=	0.038	L/D	=	3778
Pos Live load (in)	=	-0.276	L/D	=	522
Neg Live load (in)	=	0.211	L/D	=	684
Pos Post Comp load (in)	=	-0.293	L/D	=	492
Neg Post Comp load (in)	=	0.194	L/D	=	743
Pos Total load (in)	=	-0.255	L/D	=	566
Neg Total load (in)	=	0.232	L/D	=	621

Gravity Beam Design

RAM SBeam v3.0
Camp Wiliams 3030 phase 2-4
FB-3
Licensed to:

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STEEL CODE: AISC LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (34.00,0.00)

Beam Size (Optimum) = W21X55 Fy = 50.0 ksi
Total Beam Length (ft) = 34.00
Cantilever on left (ft) = 4.00
Cantilever on right (ft) = 4.00
Distance to Adjacent Edge on Left (ft) = 7.5
Distance to Adjacent Edge on Right (ft) = 17.5

COMPOSITE PROPERTIES (Not Shored):

		Left			Right
Concrete thickness (in)			2.50		2.50
Unit weight concrete (pcf)			145.00		145.00
f'c (ksi)			4.00		4.00
Decking Orientation			parallel		parallel
Decking type			ASC B-36		ASC B-36
beff (in)	=	78.00	Y bar(in)	=	18.32
Mnf (kip-ft)	=	852.87	Mn (kip-ft)	=	700.81
C (kips)	=	182.75	PNA (in)	=	15.27
Ieff (in4)	=	2032.09	Itr (in4)	=	2839.18
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 26.1
# of studs: Full = 53 Partial = 15 Actual = 15
Number of Stud Rows = 1 Percent of Full Composite Action = 27.56
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	CDL	LL	CLL
1	0.000	1.500	1.050	1.750	0.438
	4.000	1.500	1.050	1.750	0.438
2	4.000	1.500	1.050	1.750	0.438
	30.000	1.500	1.050	1.750	0.438
3	30.000	1.500	1.050	1.750	0.438
	34.000	1.500	1.050	1.750	0.438

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 60.66 kips 0.90Vn = 210.60 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Left	PreCmp-	1.2DL+1.6LL	-15.7	4.0	4.0	1.00	0.90	472.50
	Max -	1.2DL+1.6LL	-36.8	4.0	4.0	1.00	0.90	472.50
Center	PreCmp+	1.2DL+1.6LL	155.6	17.0	26.0	1.15	0.90	173.26
	PreCmp-	1.2DL+1.6LL	-15.7	30.0	26.0	1.16	0.90	175.88
	Init DL	1.4DL	112.5	17.0	---	---		
	Max +	1.2DL+1.6LL	374.3	17.0	---	---	0.85	595.69
	Max -	1.2DL+1.6LL	-36.8	30.0	26.0	1.19	0.90	179.55
Right	PreCmp-	1.2DL+1.6LL	-15.7	30.0	4.0	1.00	0.90	472.50

Gravity Beam Design

RAM SBeam v3.0
Camp Wiliams 3030 phase 2-4
FB-3
Licensed to:

06/28/10 14:25:53

Span	Cond	LoadCombo	Mu	@	Lb	Cb	Phi	Phi*Mn
	Max -	1.2DL+1.6LL	-36.8	30.0	4.0	1.00	0.90	472.50
Controlling		1.2DL+1.6LL	155.6	17.0	26.0	1.15	0.90	173.26

REACTIONS (kips):

	Left	Right
Initial reaction	25.30	25.30
DL reaction	25.50	25.50
Max +LL reaction	30.29	30.29
Max -LL reaction	-0.54	-0.54
Max +total reaction (factored)	79.06	79.06

DEFLECTIONS:

Left cantilever:

Init load (in)	=	0.136	L/D =	705	
Pos Live load (in)	=	-0.024	L/D =	3955	
Neg Live load (in)	=	0.150	L/D =	639	
Neg Post Comp load (in)	=	0.183	L/D =	525	
Neg Total load (in)	=	0.319	L/D =	301	

Center span:

Initial load (in)	at	17.00 ft	=	-0.289	L/D =	1078
Live load (in)	at	17.00 ft	=	-0.305	L/D =	1022
Post Comp load (in)	at	17.00 ft	=	-0.375	L/D =	832
Net Total load (in)	at	17.00 ft	=	-0.664	L/D =	470

Right cantilever:

Init load (in)	=	0.136	L/D =	705
Pos Live load (in)	=	-0.024	L/D =	3955
Neg Live load (in)	=	0.150	L/D =	639
Neg Post Comp load (in)	=	0.183	L/D =	525
Neg Total load (in)	=	0.319	L/D =	301

Gravity Beam Design

RAM SBeam v3.0
Camp Wiliams 3030 phase 2-4
FB-3 without cantilevers
Licensed to:

06/28/10 14:30:44

STEEL CODE: AISC LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (26.00,0.00)

Beam Size (Optimum) = W24X68 Fy = 50.0 ksi
Total Beam Length (ft) = 26.00
Distance to Adjacent Edge on Left (ft) = 7.5
Distance to Adjacent Edge on Right (ft) = 17.5

COMPOSITE PROPERTIES (Not Shored):

		Left			Right
Concrete thickness (in)		2.50			2.50
Unit weight concrete (pcf)		145.00			145.00
f'c (ksi)		4.00			4.00
Decking Orientation		parallel			parallel
Decking type		ASC B-36			ASC B-36
beff (in)	= 78.00		Y bar(in)	=	19.87
Mnf (kip-ft)	= 1139.07		Mn (kip-ft)	=	939.71
C (kips)	= 182.75		PNA (in)	=	16.25
Ieff (in4)	= 3072.45		Itr (in4)	=	4196.53
Stud length (in)	= 3.50		Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 26.1
# of studs: Full = 52 Partial = 14 Actual = 14
Number of Stud Rows = 1 Percent of Full Composite Action = 27.56
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	CDL	LL	CLL
1	0.000	1.500	1.050	1.750	0.438
	26.000	1.500	1.050	1.750	0.438

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 59.80 kips 0.90Vn = 265.56 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	165.7	13.0	26.0	1.14	0.90	270.52
	Init DL	1.4DL	124.2	13.0	---	---		
	Max +	1.2DL+1.6LL	388.7	13.0	---	---	0.85	798.76
Controlling		1.2DL+1.6LL	165.7	13.0	26.0	1.14	0.90	270.52

REACTIONS (kips):

	Left	Right
Initial reaction	19.34	19.34
DL reaction	19.50	19.50
Max +LL reaction	22.75	22.75
Max +total reaction (factored)	59.80	59.80

DEFLECTIONS:

Initial load (in)	at	13.00 ft	=	-0.203	L/D =	1534
Live load (in)	at	13.00 ft	=	-0.202	L/D =	1545

Post Comp load (in)	at	13.00 ft	=	-0.254	L/D =	1229
Net Total load (in)	at	13.00 ft	=	-0.457	L/D =	682

Gravity Beam Design

RAM SBeam v3.0
Camp Wiliams 3030 phase 2-4
FB-3 option 2
Licensed to:

06/29/10 09:02:46

STEEL CODE: AISC LRFD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (17.00,0.00)

Beam Size (Optimum) = W10X19 Fy = 50.0 ksi
Total Beam Length (ft) = 17.00
Cantilever on left (ft) = 4.00
Distance to Adjacent Edge on Left (ft) = 7.5
Distance to Adjacent Edge on Right (ft) = 17.5

COMPOSITE PROPERTIES (Not Shored):

		Left			Right
Concrete thickness (in)			2.50		2.50
Unit weight concrete (pcf)			145.00		145.00
f'c (ksi)			4.00		4.00
Decking Orientation			parallel		parallel
Decking type			ASC B-36		ASC B-36
beff (in)	=	39.00	Y bar(in)	=	10.48
Mnf (kip-ft)	=	188.28	Mn (kip-ft)	=	137.05
C (kips)	=	78.32	PNA (in)	=	8.23
Ieff (in4)	=	225.01	Itr (in4)	=	340.10
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 26.1
# of studs: Full = 23 Partial = 7 Actual = 7
Number of Stud Rows = 1 Percent of Full Composite Action = 27.86
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	CDL	LL	CLL
1	0.000	1.500	1.050	1.750	0.438
	4.000	1.500	1.050	1.750	0.438
2	4.000	1.500	1.050	1.750	0.438
	17.000	1.500	1.050	1.750	0.438

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 32.73 kips 0.90Vn = 68.85 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Left	PreCmp-	1.2DL+1.6LL	-15.7	4.0	4.0	1.00	0.90	77.12
	Max -	1.2DL+1.6LL	-36.8	4.0	4.0	1.00	0.90	77.12
Center	PreCmp+	1.2DL+1.6LL	36.5	10.9	13.0	1.16	0.90	38.32
	PreCmp-	1.2DL+1.6LL	-15.7	4.0	13.0	1.23	0.90	40.65
	Init DL	1.4DL	25.5	11.1	---	---		
	Max +	1.2DL+1.6LL	90.1	10.7	---	---	0.85	116.49
	Max -	1.2DL+1.6LL	-36.8	4.0	13.0	1.99	0.90	65.61
Controlling		1.2DL+1.6LL	36.5	10.9	13.0	1.16	0.90	38.32

REACTIONS (kips):

	Left	Right
Initial reaction	16.54	8.76
DL reaction	16.67	8.83
Max +LL reaction	19.45	11.37
Max -LL reaction	0.00	-1.08
Max +total reaction (factored)	51.13	28.79

DEFLECTIONS:

Left cantilever:

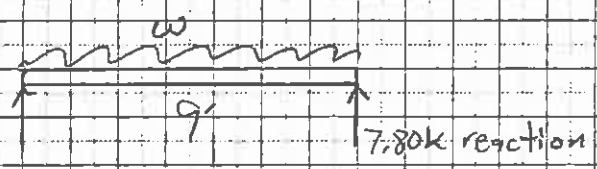
Init load (in)	=	0.127	L/D =	756
Pos Live load (in)	=	-0.099	L/D =	971
Neg Live load (in)	=	0.170	L/D =	566
Pos Post Comp load (in)	=	-0.081	L/D =	1190
Neg Post Comp load (in)	=	0.188	L/D =	511
Neg Total load (in)	=	0.315	L/D =	305

Center span:

Initial load (in)	at	10.63 ft	=	-0.187	L/D =	834
Live load (in)	at	10.63 ft	=	-0.172	L/D =	906
Post Comp load (in)	at	10.63 ft	=	-0.207	L/D =	755
Net Total load (in)	at	10.63 ft	=	-0.394	L/D =	396

10/31

FB-4
New Beam by Elevator



$w = \frac{(20')^2}{2(15')}(60+70)$
$= 1.73 klf$

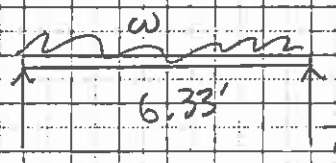
$M = 17.55 k'$

Try 8"x24" CMU w/(2)#5's
$b = 7.63''$, $d = 24'' - 4'' = 20''$, $f'_m = 1500 psi$
$\rightarrow M_{all} = 19.1 k' > 17.6 k'$ ✓
$\rightarrow V = 7.8k - \frac{20''}{12}(1.73 klf) = 4.92k$
$V_{all} = 5.5k > V$ ✓

∴ 8"x24" CMU w/(2)#5's

or may also use W12x14

FB-5
New Beam Along grid A



$w = 18'(60+70)$
$= 2.34 klf$

$M = 11.73 k'$
use 8"x24" w/(2)#5's (see above)
$V = 3.51k < 5.5k$

CALDER RICHARDS
CONSULTING ENGINEERS

JOB TITLE [illegible] BY [illegible] DATE 6-2010

SUBJECT COLUMN DESIGN CHECKED SHEET 12 OF 31

BEAM	END	LOAD (KIPS)	KL (ft)	COLUMN	
FB-3	ends	36.1	11'×1.2 → 14'	HSS 4x4x1/4	
"	middle	42.3	14'	HSS 4x4x5/16	
FB-4	left	9.0	14'	HSS 3x3x3/16	
FB-6	right	4.5	14'	HSS 3x3x3/16	

$Q_{all} = 1500psf$ (Assumed, & to be field verified)

FB-3 (ends)

$Wt = 36.1k$

$b = \left(\frac{36.1k}{1.5ksf}\right)^{1/2} = 4.91' \rightarrow 5'$ ∴ 5'-0" x ☒

FB-3 (middle)

$Wt = 42.3k$

$b = 5.3'$ ∴ 5'-6" x ☒

FB-6 (right)

$Wt = 4.5k$

$b = 1.73'$ ~~2'-0" x ☒~~

CMU Wall along grid C.4

$$Wt = \frac{(20')^2}{2(14.5')} \times (60psf + 70psf) + 11'(52psf)$$

$$2.37klf$$

$$b = \frac{2.37klf}{1.5ksf} = 1.6' \rightarrow 2'$$

2'-0" x Cont

FB-4 (Right)

$Wt = 9k + 3.5'(2.37klf)$

$17.3k$

$b = 3.4'$ 3'-6" x ☒

$$\frac{1.2(\overset{DL}{60psf}) + 1.6(\overset{LL}{70psf})}{60psf + 70psf} = 1.42 \rightarrow 1.5$$

Continuous Footing Design Per ACI 318

Allowable Bearing Pressure (psf)	1500
Average Load Factor	1.5
Factored Bearing Pressure (psf)	2250

fy (psi)	60000
f'c (psi)	2500
phi bending	0.9
phi shear	0.75
phi for plain concrete	0.55
Wall Thickness (in)	8

Notes:

1. Bending checked at face of support
2. One-Way shear checked at d/2 from face of support
3. Depth d used in design is footing depth minus 4"
4. Rho minimum equals 0.0018
5. fr equals 6*f'c^0.5
6. Plane concrete uses the total depth minus 2"

Footing Size & Reinforcement		Transverse	Longitudinal	P allow	One-Way Shear		Bending	
Width (in)	Depth (in)	As min (in^2/ft)	As min (in^2)	(klf)	Vu (k/ft)	phi Vc (k/ft)	Mu (k-ft/ft)	phi Mn (k-ft/ft)
24	12	phi*fr*S > Mu	0.52	3.0	0.75	6.60	0.50	2.75
30	12	phi*fr*S > Mu	0.65	3.8	1.31	6.60	0.95	2.75
36	12	phi*fr*S > Mu	0.78	4.5	1.88	6.60	1.53	2.75
42	12	phi*fr*S > Mu	0.91	5.3	2.44	6.60	2.26	2.75
48	12	0.26	1.04	6.0	3.00	7.20	3.13	8.98
54	12	0.26	1.17	6.8	3.56	7.20	4.13	8.98
60	12	0.26	1.30	7.5	4.13	7.20	5.28	8.98
66	12	0.26	1.43	8.3	4.69	7.20	6.57	8.98
72	12	0.31	1.56	9.0	5.25	7.20	8.00	10.54
78	12	0.32	1.68	9.8	5.81	7.20	9.57	10.87
84	12	0.33	1.81	10.5	6.38	7.20	11.28	11.28
90	12	0.39	1.94	11.3	6.94	7.20	13.13	13.13
96	13	0.39	2.25	12.0	7.41	8.10	15.13	15.13
102	13	0.45	2.39	12.8	7.97	8.10	17.26	17.26
108	14	0.46	2.72	13.5	8.44	9.00	19.53	19.53
114	14	0.52	2.87	14.3	9.00	9.00	21.95	21.95
120	15	0.52	3.24	15.0	9.47	9.90	24.50	24.50
126	16	0.53	3.63	15.8	9.94	10.80	27.20	27.20
132	16	0.59	3.80	16.5	10.50	10.80	30.03	30.03
138	17	0.60	4.22	17.3	10.97	11.70	33.01	33.01
144	17	0.66	4.41	18.0	11.53	11.70	36.13	36.13

Spot Footing Design Per ACI 318

Allowable Bearing Pressure (psf)	1500
Average Load Factor	1.5
Factored Bearing Pressure (psf)	2250

fy (psi)	60000
f'c (psi)	3000
phi bending	0.9
phi shear	0.75

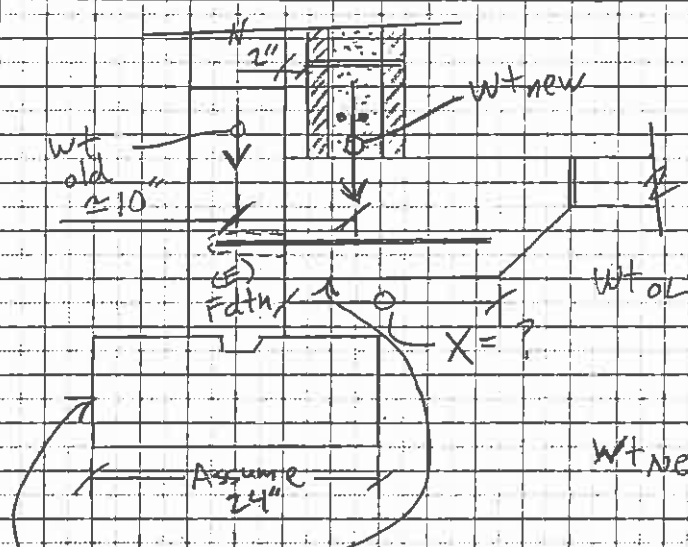
Square pedastal size (in)	6

Notes:

1. Bending checked at face of support
2. Punching and One-Way shear checked at d/2 from face of support
3. Depth d used in design is footing depth minus 4"
4. Rho minimum equals 0.0018

Footing Size & Reinforcement Each Direction			P allow	One-Way Shear		Punching Shear		Bending	
L & W (in)	Depth (in)	As min (In^2)	(kips)	Vu (kips)	phi Vc (kips)	Vu (kips)	phi Vc (kips)	Mu (k-ft)	phi Mn (k-ft)
24	12	0.52	6.0	1.88	15.77	5.94	73.61	1.27	18.07
30	12	0.65	9.4	3.75	19.72	11.00	73.61	2.81	22.59
36	12	0.78	13.5	6.19	23.66	17.19	73.61	5.27	27.10
42	12	0.91	18.4	9.19	27.61	24.50	73.61	8.86	31.62
48	12	1.04	24.0	12.75	31.55	32.94	73.61	13.78	36.14
54	12	1.17	30.4	16.88	35.49	42.50	73.61	20.25	40.66
60	12	1.30	37.5	21.56	39.44	53.19	73.61	28.48	45.17
66	12	1.47	45.4	26.81	43.38	65.00	73.61	38.67	51.12
72	13	1.72	54.0	32.06	53.24	77.48	88.73	51.05	67.50
78	14	1.99	63.4	37.78	64.08	91.06	105.16	65.81	87.05
84	15	2.29	73.5	43.97	75.91	105.73	122.91	83.18	110.05
90	15	2.86	84.4	51.33	81.34	122.05	122.91	103.36	136.58
96	16	3.20	96.0	58.50	94.65	138.94	141.97	126.56	167.29
102	17	3.57	108.4	66.14	108.94	156.92	162.34	153.00	202.30
108	18	3.96	121.5	74.25	124.22	176.00	184.03	182.88	241.86
114	19	4.37	135.4	82.83	140.49	196.17	207.04	216.42	286.27
120	20	4.81	150.0	91.88	157.74	217.44	231.36	253.83	335.81
126	21	5.26	165.4	101.39	175.98	239.80	256.99	295.31	390.75
132	22	5.74	181.5	111.38	195.21	263.25	283.94	341.09	451.37
138	23	6.23	198.4	121.83	215.42	287.80	312.20	391.36	517.96
144	24	6.75	216.0	132.75	236.62	313.44	341.78	446.34	590.79

New CMU Walls adjacent to exterior fdtn (grid A)



conservative

$Wt_{OLD} = 22'(15psf)$

$= 330$ plf (estimate)

$Wt_{New} = \frac{35'}{2}(60+70) + 11'(52psf)$

$= 2850$ plf

Try $X = 18''$

Treat as one ftg w/ regard to soil pressure

$Wt\ ftg + soil \simeq \frac{(24''+10'')}{12} \times (1' \times 150pcf + 1.5' \times 110pcf)$ (Conc, Soil)

$= 900$ plf

$\Sigma P = 330plf + 2850plf + 900plf$

$= 4080$ plf

Try $x = 18'' \rightarrow$ Total width $= 16'' + 18'' = 34''$

$\Sigma M = 2850plf \times 5''/12 - 330plf \times 5''/12 = 1.05$ k'/ft

$e = \frac{\Sigma M}{\Sigma P} = \frac{1.05k'/ft}{4.08k/ft} = 0.26'$ (inside kern)

$q_{max} = 2226psf$ (no good)

Try $X = 24'' \Rightarrow$ Total width $= 16'' + 24'' = 40''$

$\Sigma M = 2850plf \times \frac{2''}{12} - 330plf \times 8''/12 = 0.26$ k'/ft

$\rightarrow q_{max} = 1365psf < 1500psf$ ✓

cont'd

CALDER RICHARDS
CONSULTING ENGINEERS

JOB TITLE C.W. 3030 Phase 2-4 BY IT TU DATE 6-[illegible]
SUBJECT Footings CHECKED SHEET 17 OF 31

New CMU walls adjacent to exterior fdtn (grid A), cont'd

Make sure V can transfer thru shear friction

$V_u = 1.5\,[4080\,plf] = \underline{6.12\,klf}$

Avg Ult Factor, conservative

full load, conservative

Try #5's @ 16" oc

$\phi V_n = 0.75 \cdot \frac{12''}{16''} \cdot 0.31\,in^2\,(60\,ksi)\,(0.6(1.0))$

normal wt

surface not roughened.

$= 6.28\,klf > V_u$ ✓

∴ May pour new 24" x Cont ftg against existing fdtn wall w/ #5 transverse @ 16" oc, doweled in fdtn.

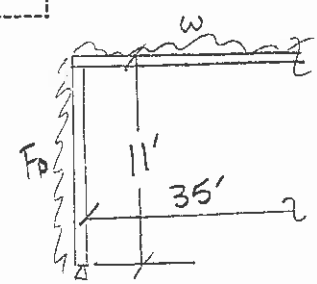
CALDER RICHARDS

MASONRY SLENDER WALL SPREADSHEET

design per 2006 IBC, assumes wall simply supported with roof loading of dead and snow

project: Camp Williams Bldg 3030 phases 2-4 (Typical Wall)

date: 6/30/2010 | 8" wall, with #5 vert @ 32"oc



clear span (ft)	11	f'm (psi)	1500	
parapet (ft)	0	Em (psi)	1350000	
tot. thickness (in)	7.625	fy (psi)	60000	
equiv. thickness (in)	4.9	Es (psi)	29000000	
wall weight (psf)	52	a / c ratio	0.85	
steel depth (in)	3.8	fr (psi)	88	
area stl (in^2/ft)	0.1160	phi	0.9	
I-gross (in^4)	394	εu	0.0025	(CMU =.0025 atlas = .0035)
eccentric dead (plf)	1050	S (in^3)	103.344	
eccentric snow (plf)	1225	Mcr (lb-in)	9094.295	
eccentricity (in)	2	n	21.481	
wind, w/o ω=1.3 (psf)	5	ultimate axial stress (psi)	60.599	60.599
seismic (psf)	12	.05*f'm (psi)	75.000	300.000 =0.2*f'm (psi)
S_{DS} =	0.79	h/t= 17.31148	h/t>30	h/t<30 ok

center in wall

$DL = \frac{35'}{2}(60) = 1050$ plf

$LL = \frac{35'}{2}(70) = 1225$ plf

wind = 5 psf (inside)

Seismic = 0.4(.788)(1.0)×0.7 ×52 psf = 11.5 → 12 psf

service check

	D+S	D+¾S+¾W	D+¾S+¾E
a (in)	0.62229	0.60227	0.60227
c (in)	0.73210	0.70855	0.70855
Ase (in^2)	0.15868	0.15358	0.15358
Icr (in^4)	33.65271	32.95261	32.95261
Ps (plf)	2561.0	2254.8	2254.8
Ms (lb-in)	2295.06	2669.92	3630.18
serv. defl (in)	0.00783	0.00911	0.01239
	ok	ok	ok

rho	0.00254
pbal	0.00988
%pbal	25.7
pmax	0.00584
	ok

serv. defl max (in)	0.924

load factors

cases	1 (D)	2 (D+S)	3 (D+S+W)	4 (D+S+W)	5 (D+S+E)	6 (D+S+E)	7 (D+S+W)
D	1.4	1.2	1.2	1.2	1.36	0.74	0.9
S	0	0.5	1.6	0.5	0.7	0	0
W or E	0	0	0.8	1.6	1.4	1.4	1.6

ultimate check

a (in)	0.57715	0.59972	0.68779	0.59972	0.62949	0.51973	0.53349
c (in)	0.67900	0.70555	0.80917	0.70555	0.74058	0.61145	0.62764
Ase (in^2)	0.14717	0.15293	0.17539	0.15293	0.16052	0.13253	0.13604
Icr (in^4)	32.04723	32.86200	35.82041	32.86200	33.90027	29.85907	30.39910
Pu (plf)	1870.4	2215.7	3563.2	2215.7	2671.3	991.8	1202.4
ult. defl (in)	0.00505	0.00644	0.01363	0.01143	0.01836	0.01311	0.00821
Mu (lb-in)	1479.44	1886.77	3994.57	3349.83	5381.23	3841.72	2406.88
phi*Mn (lb-in)	27906.56	28904.62	32732.35	28904.62	30210.62	25335.55	25955.86
	ok	ok	ok	ok	ok	ok	ok

∴ For typical 8" walls vertical steel may be #5's @ 32"oc

CALDER RICHARDS

MASONRY SLENDER WALL SPREADSHEET

design per 2006 IBC, assumes wall simply supported with roof loading of dead and snow

17/31

project: Camp Williams Bldg 3030 phases 2-4 (Door Jambs)

date: 6/30/2010 — 8" x 8" w/ (2) #4 verts

clear span (ft)	11	f'm (psi)	1500
parapet (ft)	0	Em (psi)	1350000
tot. thickness (in)	7.625	fy (psi)	60000
equiv. thickness (in)	7.625	Es (psi)	29000000
wall weight (psf)	75	a / c ratio	0.85
steel depth (in)	5.25	fr (psi)	88
area stl (in^2/ft)	0.3000	phi	0.9
I-gross (in^4)	437	εu	0.0025 (CMU =.0025 atlas = .0035)
eccentric dead (plf)	6300	S (in^3)	114.623
eccentric snow (plf)	7530	Mcr (lb-in)	10086.820
eccentricity (in)	2	n	21.481
wind, w/o ω=1.3 (psf)	30	ultimate axial stress (psi)	219.705 — 219.705
seismic (psf)	69	.05*f'm (psi)	75.000 — 300.000 =0.2*f'm (psi)
S_{DS} =	0.79	h/t= 17.31148	h/t>30 — h/t<30 ok

equivalent widths:

$= \frac{6.67'/2 + 8''/12}{8''/12} = 6$

DL = 6(1050) = 6300 plf

see prev page

LL = 6(1225) = 7530 plf

w = 6(5psf) = 30 psf

S = 6(11.5psf) = 69 psf

service check

	D+S	D+¾S+¾W	D+¾S+¾E
a (in)	2.10735	1.98431	1.98431
c (in)	2.47924	2.33449	2.33449
Ase (in^2)	0.53738	0.50600	0.50600
Icr (in^4)	149.57746	143.28442	143.28442
Ps (plf)	14242.5	12360.0	12360.0
Ms (lb-in)	14886.44	17245.02	23250.38
serv. defl (in)	0.07418	0.09820	0.15455
	ok	ok	ok

rho	0.00476
pbal	0.00988
%pbal	48.2
pmax	0.00182
	~~no good~~

less than 50% of pbal, okay

serv. defl max (in): 0.924

load factors

cases	1 (D)	2 (D+S)	3 (D+S+W)	4 (D+S+W)	5 (D+S+E)	6 (D+S+E)	7 (D+S+W)
D	1.4	1.2	1.2	1.2	1.36	0.74	0.9
S	0	0.5	1.6	0.5	0.7	0	0
W or E	0	0	0.8	1.6	1.4	1.4	1.6

ultimate check

a (in)	1.79069	1.94902	2.49039	1.94902	2.11659	1.50218	1.57132
c (in)	2.10669	2.29296	2.92987	2.29296	2.49011	1.76727	1.84862
Ase (in^2)	0.45663	0.49700	0.63505	0.49700	0.53973	0.38306	0.40069
Icr (in^4)	134.31580	141.57692	174.03569	141.57692	150.07437	121.88661	124.85182
Pu (plf)	9397.5	11820.0	20103.0	11820.0	14383.9	4983.4	6041.3
ult. defl (in)	0.02794	0.04820	0.16366	0.14140	0.25437	0.17434	0.08266
Mu (lb-in)	9082.60	11894.75	27253.97	21708.31	35015.54	23078.84	14881.40
phi*Mn (lb-in)	107376.04	114745.61	137335.54	114745.61	122169.28	93060.06	96595.44
	ok	ok	ok	ok	ok	ok	ok

∴ Use 8"x8" w/(2) #4's min for all jambs

Camp Williams Bldg 3030 phases 2-4
Mon Jun 28 13:30:25 MDT 2010

Conterminous 48 States
2006 International Building Code
Latitude = 40.436356
Longitude = -111.925302
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.009999999776482582 deg grid spacing

Period (sec)	Sa (g)
0.2	1.127 (Ss, Site Class B)
1.0	0.460 (S1, Site Class B)

Conterminous 48 States
2006 International Building Code
Latitude = 40.436356
Longitude = -111.925302
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.049 ,Fv = 1.54

Period (sec)	Sa (g)
0.2	1.183 (SMs, Site Class D)
1.0	0.709 (SM1, Site Class D)

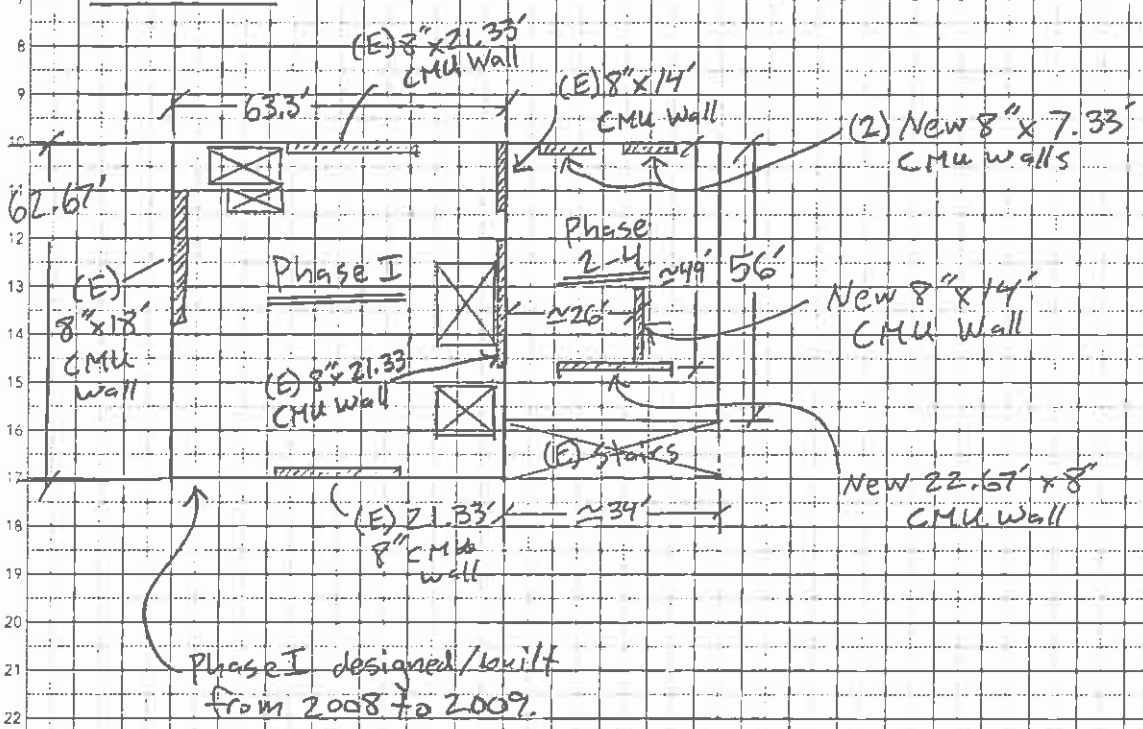
Conterminous 48 States
2006 International Building Code
Latitude = 40.436356
Longitude = -111.925302
Design Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.049 ,Fv = 1.54

Period (sec)	Sa (g)
0.2	0.788 (SDs, Site Class D)
1.0	0.473 (SD1, Site Class D)

Wind

By inspection seismic will govern over wind.

Seismic



$$V = 0.7 \cdot C_s \cdot W_t$$

$$C_s = \frac{(0.788)}{(5.0)/(1.0)} = 0.16$$

SDS

R, Reinf masonry S.W.'s

I_E

$$V = \underline{0.11 \cdot W_t}$$

see next page for weights

Seismic, cont'd

Weights:

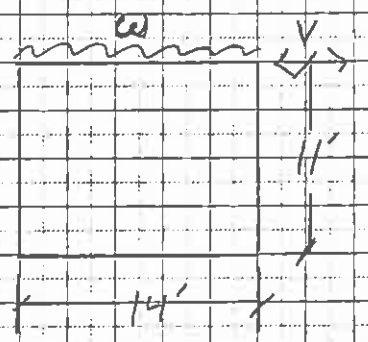
Weight New = 56' x 34' x (60psf + 20psf) (Floor) = 152.3k
(DL; partition)

+ 52psf x 14'/2 x (2x7.33' + 14' + 22.7' + + 1/2(14' + 21.3')) (Walls) = 25.1k
(height assumed; (E) walls)

$\sum$ 177.4k

$V_{New} = 0.11(177.4k)$
$= 19.5k$

Right-Side



account for 5% eccentricity

$V = 19.5k \times (0.55 \times 34') / 26'$

$= \underline{14.03k}$

$v = \frac{14.03k}{14'} = 1.0\,klf$

< 2.13klf (steel reinf)

&

< 2.91klf (masonry)

see next page for allowable shear values

$\frac{M}{V \cdot d} \simeq \frac{h}{d} = 0.78 \Rightarrow 0.8$

Grout vert cells @ 32"oc & provide (2)#4 horiz @ 48"oc

overturning:

$M_{OT} = 14.03k(11') = 154.3k'$

$\omega = (0.6 - 0.14(0.788)) \times [11'(52psf) + 3'(60 + 20)]$

$= 0.49\,(812plf)$

DL Resistance

$= 398plf$

$M_R = 398plf \times \frac{1}{2}(14')^2 = 39.0k'$

$T = C = (M_{OT} - M_R)/14' = 8.24k$

(2)#5 verts @ end of wall

$T_{all} = 2 \times 0.31in^2 \times 24ksi \times 4/3$

$= 19.8k > 8.2k$ ✓

CALDER RICHARDS

Reinforced Masonry Shearwall Quick Design

Design based upon requirements in chapter 2 of ACI 530-05

f'm (psi) = 1500 Allow for stress increase? yes [2] j = 0.9
Fs (ksi) = 24 Horizontal Reinforcing = 2 # 4 bars
Masonry type: CMU

MASONRY TAKES ALL SHEAR

V (kips per linear foot , service loads) assuming masonry takes all of the load [1]

Nominal Thickness	vertical cells grouted at	M/V*d 0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0 +
6 in	solid	2.71	2.64	2.57	2.50	2.43	2.36	2.29	2.22	2.12	1.88
	16	2.18	2.12	2.06	2.01	1.95	1.90	1.84	1.78	1.71	1.51
	24	1.98	1.93	1.88	1.83	1.78	1.73	1.68	1.63	1.55	1.38
	32	1.93	1.88	1.83	1.78	1.74	1.69	1.64	1.59	1.52	1.34
	40	1.84	1.79	1.74	1.70	1.65	1.60	1.55	1.51	1.44	1.28
	48	1.79	1.74	1.70	1.65	1.60	1.56	1.51	1.47	1.40	1.24
8 in	solid	3.67	3.58	3.49	3.39	3.30	3.20	3.11	3.01	2.88	2.55
	16	2.80	2.73	2.66	2.59	2.52	2.44	2.37	2.30	2.20	1.95
	24	2.51	2.45	2.38	2.32	2.26	2.19	2.13	2.06	1.97	1.75
	32	2.37	2.31	2.25	2.19	2.13	2.06	2.00	1.94	1.86	1.65
	40	2.27	2.21	2.16	2.10	2.04	1.98	1.92	1.86	1.78	1.58
	48	2.22	2.17	2.11	2.05	2.00	1.94	1.88	1.82	1.74	1.55

REINFORCEMENT TAKES ALL SHEAR

V (kips per linear foot , service loads) assuming reinforcement takes all of the load [1]

Horizontal spacing of 2 # 4 bars	8 in oc	16 in oc	24 in oc	32 in oc	40 in oc	48 in oc
	12.80	6.40	4.27	3.20	2.56	2.13

Reinforcement shear values above may not exceed Vmax for masonry [1]

Nominal Thickness	vertical cells grouted at	M/V*d 0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0 +
6 in	solid	4.06	3.96	3.85	3.75	3.64	3.54	3.44	3.33	3.23	3.12
	16	3.26	3.18	3.10	3.01	2.93	2.84	2.76	2.68	2.59	2.51
	24	2.97	2.90	2.82	2.74	2.67	2.59	2.52	2.44	2.36	2.29
	32	2.90	2.83	2.75	2.68	2.60	2.53	2.45	2.38	2.31	2.23
	40	2.76	2.68	2.61	2.54	2.47	2.40	2.33	2.26	2.19	2.12
	48	2.68	2.61	2.55	2.48	2.41	2.34	2.27	2.20	2.13	2.06
8 in	solid	5.51	5.37	5.23	5.09	4.95	4.80	4.66	4.52	4.38	4.24
	16	4.21	4.10	3.99	3.88	3.77	3.67	3.56	3.45	3.34	3.23
	24	3.77	3.67	3.58	3.48	3.38	3.29	3.19	3.09	3.00	2.90
	32	3.55	3.46	3.37	3.28	3.19	3.10	3.01	2.91	2.82	2.73
	40	3.41	3.32	3.23	3.15	3.06	2.97	2.88	2.80	2.71	2.62
	48	3.34	3.25	3.16	3.08	2.99	2.91	2.82	2.74	2.65	2.57

For masonry taking all shear :

$$F_v = \left(\tfrac{1}{3}\right)\left(4 - M/Vd\right)\sqrt{f'm} \le 80 - 45 \cdot M/Vd \quad for\ M/Vd < 1.0 \quad (ACI\ 530-05, eqn\ 2-21)$$

$$or \quad \sqrt{f'm} \le 35 psi \quad for\ M/Vd \ge 1.0 \quad (ACI\ 530-05, eqn\ 2-22)$$

For reinforcement taking shear :

$$V = \frac{A_v \cdot F_s \cdot 12''}{s} \quad (ACI\ 530-05, eqn\ 2-26)$$

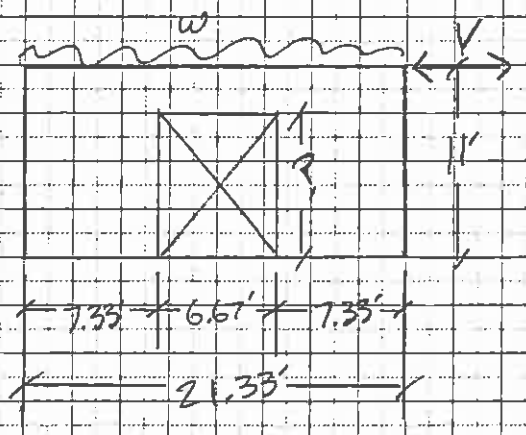
$$F_v = \left(\tfrac{1}{2}\right)\left(4 - M/Vd\right)\sqrt{f'm} \le 120 - 45 \cdot M/Vd \quad for\ M/Vd < 1.0 \quad (ACI\ 530-05, eqn\ 2-24)$$

$$or \quad 1.5\sqrt{f'm} \le 75 psi \quad for\ M/Vd \ge 1.0 \quad (ACI\ 530-05, eqn\ 2-25)$$

Notes:

1. Loads above are allowable service loads and have been divided by 1.5 for seismic loads per IBC section 2106.5.1. Loads may be multiplied by 1.5 for shearwalls governed by wind loads.
2. Stress increase is 4 / 3 for transient loads per ACI 530-05 section 2.1.2.3.

Top Side



$V = 19.5k \cdot (49' - \frac{1}{2}(56') + 0.05(56')) / 49'$

$= 9.5k$

$\frac{M}{V \cdot d} \simeq \frac{h}{d} > 1.0$

$v = \frac{9.5k}{2 \times 7.33'} = 0.65klf < 2.13klf$ ← see previous page.

∴ grout vert cells @ 32" oc & provide (2) #4 horiz @ 48" oc

Check Overturning:

$w = 0.49(17.5'(60+20) + 11'(52))$

$= 0.97klf$

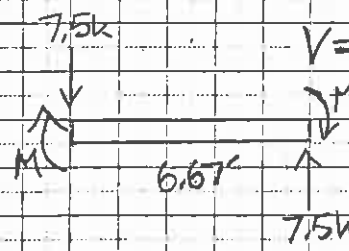
$M_{OT} = 9.5k(11') = \underline{105k'}$

$M_R = 0.97klf \times \frac{1}{2}(21.33')^2$

$= 221k' > M_{OT}$

w/o DL ⇒ $T = C = \frac{\frac{1}{2}(105k')}{7'} = 7.5k$ ← (2) #5 verts

Check V, M in masonry beam



$V = \underline{7.5k}$ (resist uplift of (1) wall)

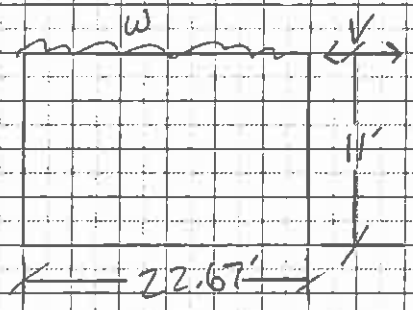
$M = \frac{7.5k(6.67')}{2} = \underline{25.0k'}$

∴ Need 8" x 32" w/ (2) #5's Top & Bottom

$V_{all} = 7.7k$ ✓

$M_{all} = 32.5k'$ ✓

Bottom Side



$V = 19.5k\,(0.55(56')) / 49'$
$= 12.3k$

$\frac{M}{V \cdot d} \simeq \frac{h}{d} = 0.49 \rightarrow 0.50$

$v = \frac{12.3k}{22.67'} = 0.54\,klf < 2.13\,klf$ ✓

$\omega = 0.49\,(13'(80) + 11'(52)) = 790\,plf$

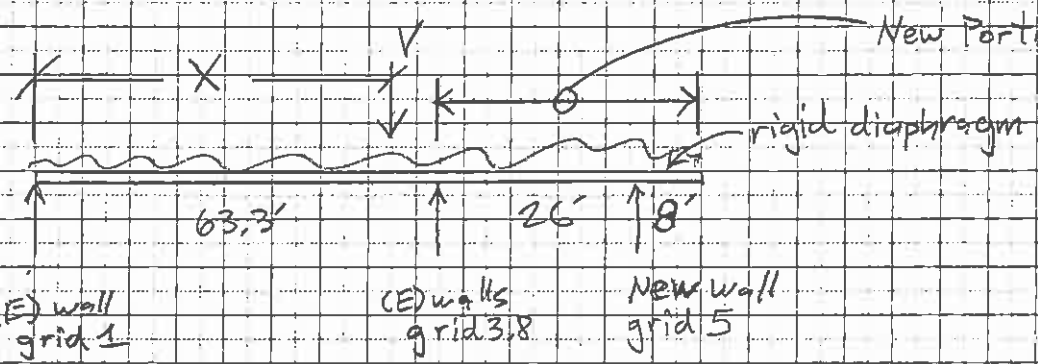
$M_{OT} = 135k'$

$M_R = 203k' > M_{OT}$

Use Typical (2)#5 verts @ end of wall

∴ grout vert cells @ 32" oc & provide (2)#4 horiz @ 48" oc

Check (E) shearwalls (14' long & 21.33')



$Wt_{(Exist)} \simeq 560k$ (see copy from phase 1 calcs on next page).

$V = 0.11(560k) = 61.6k$

$X = 0.55 \times 97.3' = 53.5'$

Relative Rigidities:

grid 1:

$$K \simeq \underbrace{\left[0.4\left(\tfrac{11'}{18'}\right)^3 + 0.3\left(\tfrac{11'}{18'}\right)\right]^{-1}}_{\text{from Amrhein's masonry design manual}} = \underline{3.64\ k/in}$$

grid 3.8:

$$K \simeq \left[0.4\left(\tfrac{11'}{21.33'}\right)^3 + 0.3\left(\tfrac{11'}{18'}\right)\right]^{-1} + \left[0.4\left(\tfrac{11'}{14'}\right)^3 + 0.3\left(\tfrac{11'}{14'}\right)\right]^{-1}$$

$$= \underline{6.53\ k/in}$$

grid 5:

$$K \simeq \left[0.4\left(\tfrac{11'}{14'}\right)^3 + 0.3\left(\tfrac{11'}{14'}\right)\right]^{-1} = 2.33\ k/in$$

Reactions:

Found using RISA:
- grid 1 = 14.9k < 19.5k ✓ } Values used in Phase 1,
- grid 3.8 = 33.7k < 34.5k ✓ } See next two pages.
- grid 5 = 13.0k < 14k (see previous pages)

∴ (E) walls okay for addition

CALDER RICHARDS
CONSULTING ENGINEERS
JOB TITLE Camp [illegible] BY [illegible] DATE [illegible]
SUBJECT Seismic Loads CHECKED SHEET 28 OF 31

$V = \frac{C_s}{1.4} \times W_t$ $\qquad C_s = \frac{(0.788)}{5.0/(1.0)} = 0.16$ (S_{DS}, R, I_E)

$= 0.11 \cdot W_t$

9-4-08
These calcs performed assuming phase 1 & phase 2 would be built at same time

$W_t = 97.33' \times 63' \times (60\,psf + 20\,psf)$ (floor) $= 491k$ (DL, partition)

$+ 52\,psf \times \frac{11'}{2} \times (4 \times 20' + 50' + 23')$ (walls) $= 43.8k$

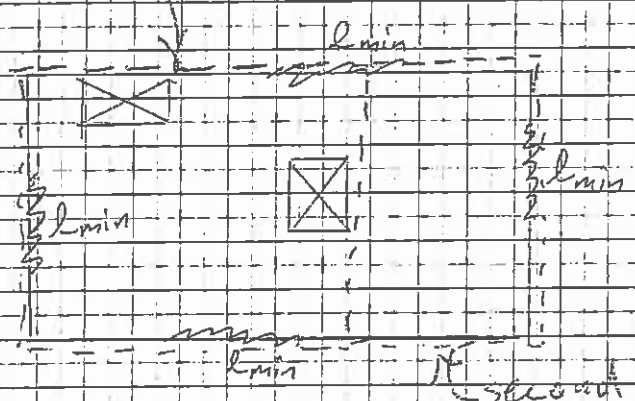
8" wall grouted @32" o.c. — assume 14' high — estimate of outside S.W.'s — stairwell 1 — stairwell

$+ 25.1k$ — New walls

$\Sigma\ 559k \rightarrow 560k$ w/ new phases

9-4-08
first phase

$V = 570k\,(0.11)$
$= 62.7k$



Find l_{min} of 8" CMU wall needed to resist shear

Assume some eccentricity so increase by 5%

$V_{1side} = \frac{62.7k}{2} \times 1.05 = 32.9k$

$F_v = \sqrt{1500\,psi}$
$= 38.7 \Rightarrow 35\,psi\ max$
$\Rightarrow$ increase for seismic
$\Rightarrow 35\,psi \times \frac{4}{3} = 46.6\,psi$

Trial
$\Rightarrow l_{min} = \frac{32900\#\,(1.5)}{0.9(4.9")\times 12 \times 46.6\,psi} = 20'$ ✓

$\Rightarrow \frac{M}{V \cdot d} = \frac{L}{d} = \frac{14'}{20'} = 0.7$

$F_v = \left(\frac{1}{3}\right)(4 - 0.7)\sqrt{1500\,psi} = 42.6\,psi$
$\Rightarrow \times \frac{4}{3} = 56.8\,psi$

$l_{min} = 20' \times \frac{46.6\,psi}{56.8\,psi} = 16.4'$
$\Rightarrow 18'$ ✓

$l_{min} = 18'$ ea side
preliminary for arch's use

Phase I COPIED FROM PHASE 1 CALCS

Walls at top & bottom:

$V = 20.6k$ (from rigid diaphragm analysis)

$f_v = \frac{1.5(20600\#)}{0.9(4.9'')(19.33'(12))} = 30.2\,PSI$ $\frac{M}{V \cdot d} = \frac{h}{d} = \frac{12'}{19.33'} = 0.62$

$F_v = (\frac{1}{3})(4 - 0.62)\sqrt{1500\,PSI} = 43.6\,PSI$

or

$80 - 45(0.62) = 52.1\,PSI$

$= 43.6\,psi \times \frac{4}{3} = 58.1\,PSI > f_v$ ✓

∴ May use 8" walls, grouted @ 32" oc

Left wall: Grid 1

$V = 17.72k$ (from rigid diaphragm)

use $\frac{1}{2} \times 38.5k = 19.5k > 14.9k$

$f_v = 30.7\,psi$, $\frac{h}{d} = \frac{12'}{18'} = 0.67 \Rightarrow F_v = 57.3\,psi$ ✓

∴ May use 8" walls, grouted @ 32" oc

Right walls: Grid 3.8

Assume 21.33' & 14' walls will take $\frac{110\%}{2} \times 62.7k$ ← Phase I ; Phase II

$= 34.5k > 33.7k$ ✓

% V based on rigidity:

21.33' = 71%

14' = 29%

$V_{21.33'} = 0.71(34.5k) = 24.5k$ $f_v = 33\,PSI$ $F_v = 59.1\,PSI$ ✓

$V_{14'} = 0.29(34.5k) = 10.0k$ $F_v = 54.1\,PSI$ ✓ $f_v = 20\,PSI$

∴ May use 8" walls, grouted @ 32" oc

$v_{all} = 2130$ plf (welded to support @ 12" oc, see next page)

$F_{pmin} = 0.2(0.788)(1.0) \times 0.7 \times W_t$
S_{DS} I_E ASD

$= \underline{0.1 \times W_t}$

same as for shear walls. Use forces found in shearwall design

Right Side:

$V = 14.03k$

$ft_{req'd} = \frac{14.03k}{2.13 klf} = 6.6' \rightarrow 7' < \text{wall length} = 14'$ ✓

for diaphragm attachment

Top Side:

$V = 9.5k$

$v = \frac{9.5k}{21.33' \text{ wall}} = 446 \text{ plf} < 2130 \text{ plf}$ ✓

Bottom Side:

$V = 12.3k$

$v = \frac{12.3k}{22.67'} = 543 \text{ plf} < 2130 \text{ plf}$ ✓

Left Side:

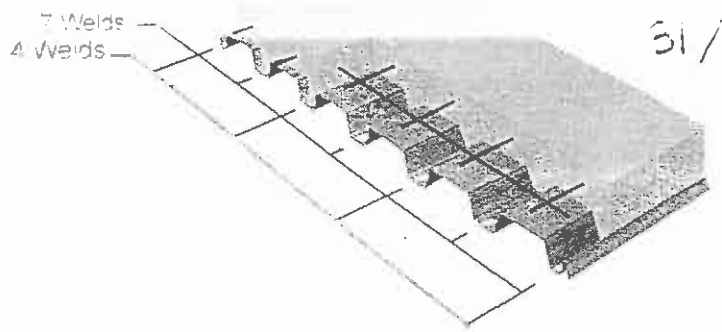
$V = 34.5k$

$ft_{req'd} = \frac{34.5k}{2.13 klf} = 16.2' \rightarrow 17' \text{ min} < 2 \text{ walls} (21.33' + 14')$ ✓

∴ May attach deck to supports w/ 3/4" Ø puddle welds @ 12" oc

51/51

- 4 in. (*102 mm*) TOTAL SLAB DEPTH
- Normal Weight Concrete
  - 145 pcf (*2,320 kg/m³*)
  - 36.6 psf (*1,752 N/m²*)
- Galvanized or Phosphatized/Painted



Deck Weight and Section Properties

Gage	Weight (psf, *N/m²*) Galv G60 *Z180*	Weight (psf, *N/m²*) Phos/ Painted	Properties per ft (*m*) of Width I in.⁴ *mm⁴*	+S in.³ *mm³*	−S in.³ *mm³*	Allowable Reactions per ft (*m*) of Width (lb, *N*) End Bearing 2" *51 mm*	End Bearing 3" *76 mm*	End Bearing 4" *102 mm*	Interior Bearing 3" *76 mm*	Interior Bearing 4" *102 mm*
22	1.9 *91.0*	1.8 *86.2*	0.175 *238,978*	0.187 *10,054*	0.198 *10,645*	487 *7,107*	585 *8,537*	683 *9,968*	1250 *18,242*	1498 *21,862*
20	2.3 *110.1*	2.2 *105.3*	0.216 *294,967*	0.235 *12,634*	0.248 *13,333*	665 *9,705*	784 *11,442*	903 *13,178*	1790 *26,123*	2118 *30,910*

Allowable Superimposed Loads (psf, *kN/m²*)

Gage	Spans	Span (ft-in., *mm*) 6'-0" *1,830*	6'-6" *1,980*	7'-0" *2,130*	7'-6" *2,290*	8'-0" *2,440*	8'-6" *2,590*	9'-0" *2,740*	9'-6" *2,900*	10'-0" *3,050*	10'-6" *3,200*	11'-0" *3,350*
22	1	269 *12.9*	231 *11.1*	200 *9.6*	175 *8.4*	153 *7.3*	135 *6.5*	120 *5.7*	107 *5.1*	95 *4.5*	85 *4.1*	76 *3.6*
	2	300 *14.4*	266 *12.7*	200 *9.6*	175 *8.4*	153 *7.3*	135 *6.5*	120 *5.7*	107 *5.1*	95 *4.5*	85 *4.1*	76 *3.6*
	3	300 *14.4*	266 *12.7*	200 *9.6*	175 *8.4*	153 *7.3*	135 *6.5*	120 *5.7*	107 *5.1*	95 *4.5*	85 *4.1*	76 *3.6*
20	1	300 *14.4*	244 *11.7*	212 *10.2*	185 *8.9*	163 *7.8*	144 *6.9*	127 *6.1*	113 *5.4*	101 *4.8*	91 *4.4*	81 *3.9*
	2	300 *14.4*	279 *13.4*	247 *11.8*	220 *10.5*	163 *7.8*	144 *6.9*	127 *6.1*	113 *5.4*	101 *4.8*	91 *4.4*	81 *3.9*
	3	300 *14.4*	279 *13.4*	247 *11.8*	220 *10.5*	163 *7.8*	144 *6.9*	127 *6.1*	113 *5.4*	101 *4.8*	91 *4.4*	81 *3.9*

Shoring required in shaded areas to right of ***heavy line***

Diaphragm Shear Values, q (plf, *kN/m*) and Flexibility Factors, F ((in./lb)x10⁶, (*mm/N*)x10⁶)

Gage	Welds	Span (ft-in., *mm*) 6'-0" *1,830*	6'-6" *1,980*	7'-0" *2,130*	7'-6" *2,290*	8'-0" *2,440*	8'-6" *2,590*	9'-0" *2,740*	9'-6" *2,900*	10'-0" *3,050*	10'-6" *3,200*	11'-0" *3,350*
22	q4	2065 *30.14*	2025 *29.55*	1995 *29.11*	1965 *28.68*	1940 *28.31*	1915 *27.95*	1900 *27.73*	1880 *27.44*	1865 *27.22*	1850 *27.00*	1835 *26.78*
	F4	0.40 *2.3*	0.41 *2.3*	0.41 *2.3*	0.42 *2.4*	0.42 *2.4*	0.43 *2.5*	0.43 *2.5*	0.44 *2.5*	0.44 *2.5*	0.45 *2.6*	0.45 *2.6*
	q7	2275 *33.20*	2220 *32.40*	2170 *31.67*	2130 *31.09*	2095 *30.57*	2065 *30.14*	2035 *29.70*	2015 *29.41*	1990 *29.04*	1970 *28.75*	1950 *28.46*
	F7	0.36 *2.1*	0.37 *2.1*	0.38 *2.2*	0.39 *2.2*	0.39 *2.2*	0.40 *2.3*	0.40 *2.3*	0.41 *2.3*	0.41 *2.3*	0.42 *2.4*	0.42 *2.4*
20	q4	2130 *31.09*	2085 *30.43*	2045 *29.84*	2010 *29.33*	1980 *28.90*	1955 *28.53*	1930 *28.17*	1910 *27.87*	1890 *27.58*	1875 *27.36*	1860 *27.14*
	F4	0.35 *2.0*	0.36 *2.1*	0.37 *2.1*	0.37 *2.1*	0.38 *2.2*	0.38 *2.2*	0.39 *2.2*	0.39 *2.2*	0.40 *2.3*	0.40 *2.3*	0.40 *2.3*
	q7	2385 *34.81*	2315 *33.78*	2260 *32.98*	2215 *32.33*	2170 *31.67*	2135 *31.16*	2100 *30.65*	2070 *30.21*	2045 *29.84*	2020 *29.48*	1995 *29.11*
	F7	0.32 *1.8*	0.32 *1.8*	0.33 *1.9*	0.34 *1.9*	0.35 *2.0*	0.35 *2.0*	0.36 *2.1*	0.36 *2.1*	0.37 *2.1*	0.37 *2.1*	0.38 *2.2*