G0911-011

STRUCTURAL CALCULATIONS

FOR

Gila County Public Administration
Russell Road and Hope Lane
Globe, AZ 85501

DLR PROJECT NO. **30-09115-00**
DATE **10/30/2009**

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PERMIT #
DATE:

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Project:	Gila Country Public Administration	Project Number:	30-09115-00
Subject:	Typical Dead Loads	Computed by:	EDN
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Roof Dead Load (For Foundations only)

Per PEMB manufacturer		5	
Collaterall		5	psf
	Total	10	psf

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-7-1

Project: Gila Country Public Administration
Subject: Min. Roof Live Loads - IBC 2006 Section 4.9.1
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Roof Live Load = $L_R = L_O R_1 R_2$
Where $12 <= L_R <= 20$
L_R = Reduced roof live load horiz projected area
L_O= Roof live load from Table 4-1

$R_1 = 1$, for At <=200 sq.ft.
$R_1 = 1.2 - 0.001At$ for 200 sq.ft.< At < 600 sq.ft.
$R_1 = 0.6$ for At >= 600 sq.ft.

At = Tributary area in square feet

$R_2 = 1$ for F<= 4
$R_2 = 1.2 - 0.05F$ for 4< F<= 12
$R_2 = 0.6$ for F>=12

F= For sloped roofs, the number inches of rise per foot
F= For arch or dome roofs, rise-to-span ratio multiplied by 32

Roof Slope	Roof Live Load Tributary Loaded Area In Square Feet For Any Structural Member				
	0 to 300	301 to 400	401 to 500	501 to 600	Over 600
	Pounds per square foot (psf)				
Flat or rise up to 4 in per foot. Arch, dome with rise-to-span ratio multiplied by 32	20	18	16	14	12
Rise 5 in per foot to less than 6 in per foot. Arch or dome with rise-to-span ratio x 32, from 4 to 6	19	17	15	13	12
Rise 6 in per foot to less than 8 in per foot. Arch or dome with rise-to-span ratio x 32, from 6 to 8	18	16	14	13	12
Rise 8 in per foot to less than 10 in per foot. Arch or dome with rise-to-span ratio x 32, from 8 to 10	16	14	13	12	12
Rise 10 in per foot to less than 12 in per foot. Arch or dome with rise-to-span ratio x 32, from 10 to 12	14	13	12	12	12
Rise 12 in per foot and greater per foot. Arch or dome with rise-to-span ratio x 32, greater than 12	12	12	12	12	12
Awnings & Canopies	5	5	5	5	5
Greenhouses	10	10	10	10	10

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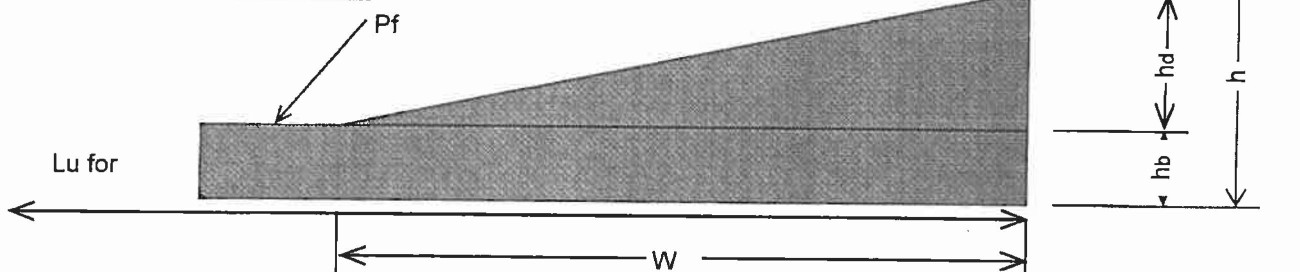
Pf = .7CeCt I Pg = 20 PSF — Minimum Roof Snow Load, Pf_{MIN}=Pg*I for Pg<=20psf, Pf_{MIN}=20*I for Pg>20psf

Unbalanced snow loads shall be determined per section 7.6 ASCE 7-05

20 PSF — Snow Load, for sloped roofs (Cs Pf)
40 PSF — Snow Load at overhangs(2.0 Pf)

Ce	1.0	Snow Exposure Coefficient, Table 7-2, ASCE 7-05
Ct	1.0	Thermal Factor, Table 7-3 ASCE 7-02
Cs	1.0	Roof Slope Factor, Figure 7-2, ASCE 7-05
I	1.00	Importance Factor, Table 7-4, ASCE 7-05
Pg (psf)	25.0	Ground Snow Load
hr (ft)=	7.00	Roof Height Difference
hb (ft)=	1.2	=Pf/D, Balanced Snow Height



Density of Snow:
D = 0.13Pg + 14.0 <=30 (pcf)

D (pcf) = 17.25 Use: 17.25

hdl = 0.43(Lu^0.33)*(Pg + 10)^0.25 - 1.5 = Height of Leeward Drift
hdw =3/4[0.43(Lu^0.33)*(Pg + 10)^0.25 - 1.5] = Height of Windward Drift
hd=Max(hdl,hdw), If hd<=hr-hb then W = 4hd, If hd>hr-hb then hd=hr-hb &
W=4*hd^2 /(hr-hb), but W need not exceed 8(hr-hb)
Drift Loads Need to be considered only when: (hr - hb)/hb > 0.2
or when pg > 5psf.
(Pd + Pf)max = D(hd + hb)<= D*hr

Leeward Lu (ft)	Windward Lu (ft)	Leeward hdl (ft)	Windward hdw (ft)	hd (ft)	W (ft)	Pd (psf)	Pf+Pd (psf)
25	20	1.6	1.0	1.6	6.2	27	47
40	40	2.1	1.6	2.1	8.3	36	56
50	50	2.3	1.8	2.3	9.4	41	61
60	90	2.6	2.4	2.6	10.4	45	65
80	180	3.0	3.3	3.3	13.2	57	77
100	100	3.3	2.5	3.3	13.4	58	78
120	120	3.7	2.7	3.7	14.6	63	83
140	140	3.9	2.9	3.9	15.7	68	88
160	160	4.2	3.1	4.2	16.7	72	92
180	180	4.4	3.3	4.4	17.6	76	96
200	200	4.6	3.5	4.6	18.4	79	99
250	250	5.1	3.8	5.1	20.3	88	108
300	300	5.5	4.1	5.5	22.0	95	115
350	80	5.9	2.3	5.8	23.5	101	121
400	400	6.2	4.6	5.8	26.2	101	121
450	450	6.5	4.9	5.8	28.9	101	121
500	500	6.8	5.1	5.8	31.5	101	121
550	550	7.1	5.3	5.8	34.1	101	121
600	600	7.3	5.5	5.8	36.5	101	121

For drifting snow due to parapet walls and roof projections, use Lu equal to length of upwind roof and 0.75Pd
For additional information, refer to section 7.8, ASCE 7-05, min, length of roof projection for consideration of drifting load is 15'. See section 7.9 ASCE 7-05 for sliding snow.

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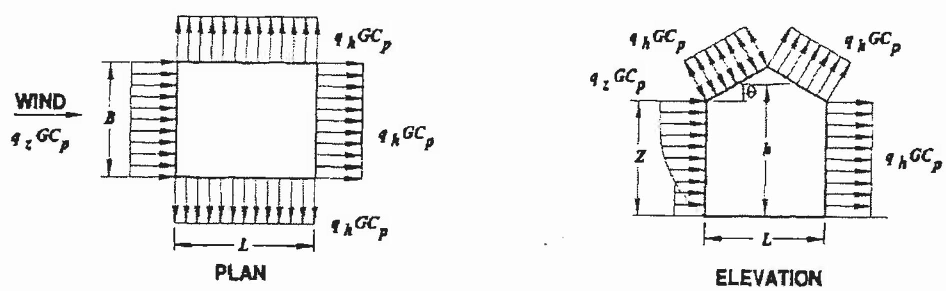
Wind Anlaysis (All Heights) - Method 2 Analytical Procedure (ASCE 7-05 , 6-5) *[ROOFS < 10 DEGREES ONLY]*

h	14.00 '	mean roof height (ft)	
$h_{parapet}$	14.00 '	Top of parapet above grade (ft)	
	C	Exposure Cat. & Case(B1,B2; C; D)..sec 6.5.6, ASCE 7-05	
K_h	0.85	Velocity Pressure Exposure Coefficient at mean roofheight "h",.............. Table 6-3, ASCE 7-05	
$K_{z\ parapet}$	0.85	Velocity Pressure Exposure Coefficient at top of parapet "hparapet",.............. Table 6-3, ASCE 7-05	
K_{zt}	1.0	Topographic Factor,1.0 Default Value, Calculate for Hills, Ridges & Escarpments, Sec 6.5.7.2, ASCE7-02	
K_d	0.85	Wind Directionality Factor, 0.85 for All Buildings, Table 6-4, ASCE 7-05	
V	90	Basic Wind Speed (mph).............. Fig 6-1, ASCE 7-05	
I_w	1.00	Importance Factor, Category I =0.87(non-hurricane), II =1.0, III =1.15, IV = 1.15, Table 6-1, ASCE 7-05	
q_h	14.98	Velocity Pressure (psf) at mean roof height (h) q_h=.00256$K_hK_{zt}K_dV^2I_w$	
Class	ENCL	Exposure Classification (open, par-encl, encl).............Fig 6-5, ASCE 7-05	
G	0.85	Gust Factor, 0.85 for Rigid Buildings, for Flexible Bld. see Sec 6.5.8, ASCE7-02	
Length (L)	200.0 '	0.30 B/L	Length to Width Ratio$_{normal}$ (B=bldg depth in wind direction; L=bldg width transverse to wind direction)
Width (B)	60.0 '	3.33 L/B	Length to Width Ratio$_{parallel}$ (L=bldg depth in wind direction; B=bldg width transverse to wind direction)
Area$_{normal}$	6,000 sf		
Area$_{parallel}$	6,000 sf		
h/B Ratio	0.23 h/B	Height to Width Ratio$_{normal}$ (h=bldg height; B=bldg depth in wind direction)	
h/L Ratio	0.07 h/L	Height to Width Ratio$_{parallel}$ (h=bldg height; L=bldg depth in wind direction)	
$R_{ww\ normal}$	30.0 '	Windward Roof$_{normal}$ (horiz distance from windward edge to ridge) = B/2 if centered	
$R_{ww\ parallel}$	100.0 '	Windward Roof$_{parallel}$ (horiz distance from windward edge to theoretical ridge) = L/2 if centered	
Φ, Roof Slope	9.50 Deg.	Roof Slope in Degrees, Less than 10 Degrees only.	



GABLE, HIP ROOF

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Main Wind Force Resisting System

	Windward	Leeward		Sidewall	Roof		
C_p	0.8	-0.50 normal	-0.23 parall	-0.7	-0.30 norml	-0.30 parall	ExternalPressure Coefficient,...... Fig 6-6, ASCE 7-05
GC_{pi}	-0.18	-0.18		0.18	0.18		Internal Pressure Coefficient,..... Fig 6-5, ASCE 7-05

$p_{total} = (q_zGC_p\text{-}q_i(GC_{pi}))_{windward}$ - $(q_hGC_p\text{-}q_i(GC_{pi}))_{leeward}$ **(Combined Windward & Leeward)**

Design Wind Loads:

	Windward			LeeWard		LeeWard	
height (z)	K_z	q_z	p_{ww}	$p_{lw\,normal}$	p_{normal}	$p_{lw\,parrallel}$	$p_{parallel}$
15 '	0.850	14.98	12.88	-3.67	**16.6**	-0.27	**13.2**
20 '	0.900	15.86	13.48	-3.67	**17.2**	-0.27	**13.8**
25 '	0.940	16.57	13.96	-3.67	**17.6**	-0.27	**14.2**
30 '	0.980	17.27	14.44	-3.67	**18.1**	-0.27	**14.7**
40 '	1.040	18.33	15.16	-3.67	**18.8**	-0.27	**15.4**
50 '	1.090	19.21	15.76	-3.67	**19.4**	-0.27	**16.0**
60 '	1.130	19.92	16.24	-3.67	**19.9**	-0.27	**16.5**
70 '	1.170	20.62	16.72	-3.67	**20.4**	-0.27	**17.0**
80 '	1.210	21.33	17.20	-3.67	**20.9**	-0.27	**17.5**
90 '	1.240	21.86	17.56	-3.67	**21.2**	-0.27	**17.8**
100 '	1.260	22.21	17.80	-3.67	**21.5**	-0.27	**18.1**
120 '	1.310	23.09	18.40	-3.67	**22.1**	-0.27	**18.7**
140 '	1.360	23.97	19.00	-3.67	**22.7**	-0.27	**19.3**
160 '	1.390	24.50	19.36	-3.67	**23.0**	-0.27	**19.6**
180 '	1.430	25.20	19.84	-3.67	**23.5**	-0.27	**20.1**
200 '	1.530	26.97	21.03	-3.67	**24.7**	-0.27	**21.3**
250 '	1.530	26.97	21.03	-3.67	**24.7**	-0.27	**21.3**
300 '	1.590	28.02	21.75	-3.67	**25.4**	-0.27	**22.0**
350 '	1.590	28.02	21.75	-3.67	**25.4**	-0.27	**22.0**
400 '	1.690	29.79	22.95	-3.67	**26.6**	-0.27	**23.2**
450 '	1.730	30.49	23.43	-3.67	**27.1**	-0.27	**23.7**
500 '	1.770	31.20	23.91	-3.67	**27.6**	-0.27	**24.2**

	+/- inter. pressure		Design Pressure
	$p_{(+)}$	$p_{(-)}$	
SideWall	-11.61	-6.22	**11.61**
$Roof_{normal}$	-6.52	-1.12	**10.00**
$Roof_{parallel}$	-6.52	-1.12	**10.00**

Top of parapet

14 '	0.850	14.98	**22.47**	**-14.98**
				37.45

Pp +/- = qp(GCpn....ASCE 7-05 6.5.12.2.4, Eq 6-20
Pww+Plw combined when diaphragm supports two ext. parpapets (ASCE 7-05 Commentary 6.5.11.5)

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COMPONENTS AND CLADDING

For A > 700 sq.ft. Use Main Wind ForceResisting System Loads

$P_{c\&c(-)} = q_h(GC_p) - q_h(GC_{pi})$ Outward under Pos. Inter. Pressure.......ASCE 7-05, Eq 6-23

$P_{c\&c(+)} = q_z(GC_p) - q_h(-GC_{pi})$ Inward under Neg. Inter Pressure.......ASCE 7-05, Eq 6-23

WALLS		Outward under (+) Inter. Pressure (leeward)					Inward under (-) Inter. Pressure (winward)			Design Pressure	
area	height h_z	q_h	Zone $4_{(-)}$ $GC_{p(-)}$	$p_{(-)}$	Zone $5_{(-)}$ $GC_{p(-)}$	$p_{(-)}$	q_z	Zone $4,5_{(+)}$ $GC_{p(+)}$	$p_{(+)}$	Zone 4	Zone 5
10	80	14.98	-0.90	**-16.18**	-1.800	**-29.66**	21.33	0.90	**21.89**	**21.89**	**29.66**
20	80	14.98	-0.90	**-16.18**	-1.800	**-29.66**	21.33	0.90	**21.89**	**21.89**	**29.66**
50	80	14.98	-0.85	**-15.43**	-1.560	**-26.07**	21.33	0.80	**19.76**	**19.76**	**26.07**
100	50	14.98	-0.80	**-14.68**	-1.400	**-23.67**	19.21	0.75	**17.11**	**17.11**	**23.67**
500	80	14.98	-0.70	**-13.18**	-1.000	**-17.68**	21.33	0.60	**15.49**	**15.49**	**17.68**

ROOFS		Outward under (+) Inter. Pressure						
area	height h_{mean}	q_h	**Zone $1_{(-)}$** $GC_{p(-)}$	**Design** $p_{(-)}$	**Zone $2_{(-)}$** $GC_{p(-)}$	**Design** $p_{(-)}$	**Zone $3_{(-)}$** $GC_{p(-)}$	**Design** $p_{(-)}$
20	14	14.98	-1.32	**-22.47**	-2.180	**-35.36**	-3.050	**-48.39**
100	14	14.98	-1.1	**-19.18**	-1.900	**-31.16**	-2.670	**-42.70**
500	14	14.98	-0.9	**-16.18**	-1.600	**-26.67**	-2.300	**-37.15**

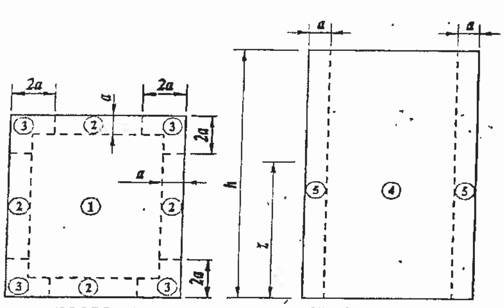
14.00' par   Parapet Height = 0.00 ft   Parapet is< 3' therefore higher pressures at Zone 3

Pc&c CASE A = qp(GCp +wall -GCp -roof); interior and corner zones....ASCE 7-05 6.5.12.4.4, Eq 6-24
Pc&c CASE B = qp(GCp +wall -GCp -wall); interior and corner zones....ASCE 7-05 6.5.12.4.4, Eq 6-24

PARAPETS										
area	$K_{z\ parapet}$	q_p	case "A" pressure towards bldg				case "B" pressure away from bldg			
20	0.850	14.98	3.08 int	**46.1 int**	3.95 crn	**59.2 crn**	-1.8 int	**-27.0 int**	-2.7	**-40.5 crn**
100	0.850	14.98	2.65 int	**39.7 int**	3.42 crn	**51.2 crn**	-1.6 int	**-23.2 int**	-2.15	**-32.2 crn**
500	0.850	14.98	2.20 int	**33.0 int**	2.90 crn	**43.4 crn**	-1.3 int	**-19.5 int**	-1.6	**-24.0 crn**

14.00' par   Parapet Height = 0.00 ft   Parapet is< 3' therefore higher pressures at Zone 3

a =	6.0 '
Roof Zone 3 (2a) =	12.0 '



ROOF PLAN   WALL ELEVATION

Notes:

1. Vertical scale denotes GC_p to be used with appropriate q_z or q_h.
2. Horizontal scale denotes effective wind area A, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Use q_z with positive values of GC_p and q_h with negative values of GC_p.
5. Each component shall be designed for maximum positive and negative pressures.
6. Coefficients are for roofs with angle $\theta \le 10°$. For other roof angles and geometry, use GC_p values from Fig. 6-11 and attendant q_h based on exposure defined in 6.5.6.
7. If a parapet equal to or higher than 3 ft (0.9m) is provided around the perimeter of the roof with $\theta \le 10°$, Zone 3 shall be treated as Zone 2.
8. Notation:
   - a: 10 percent of least horizontal dimension, but not less than 3 ft (0.9 m).
   - h: Mean roof height, in feet (meters), except that eave height shall be used for $\theta \le 10°$.
   - z: height above ground, in feet (meters).
   - θ: Angle of plane of roof from horizontal, in degrees.

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SEISMIC:

	II	OCCUPANCY CATEGORY (I, II, III & IV) TABLE 1-1 ASCE 7-05
$I_E=$	1.00	IMPORTANCE FACTOR (TABLE 11.5-1)
	C	SITE CLASSIFICATION (A, B, C, D,E,or F, TABLE 20.3-1
$S_S=$	37.4	%,MAPPED EQ SPECTRAL RESPONSE ACCELERATION, FIGURE 22-1
$S_1=$	10.2	%,MAPPED EQ SPECTRAL RESPONSE ACCELERATION, FIGURE 22-2
$F_a=$	1.20	SITE COEFFICIENT (TABLE 11.4-1)
$F_v=$	1.70	SITE COEFFICIENT (TABLE11.4-2)
$S_{MS}=$	0.45	S_S * F_a, MAX. EARTH QUAKE RESPONSE ADJUSTED FOR SITE CLASS EFFECTS
$S_{M1}=$	0.17	S_1 * Fv, MAX. EARTH QUAKE RESPONSE ADJUSTED FOR SITE CLASS EFFECTS
$S_{DS}=$	0.30	2/3 * S_{MS}, DESIGN EARTH QUAKE SPECTRAL ACCELERATION, SHORT PERIOD
$S_{D1}=$	0.12	2/3* S_{M1},DESIGN EARTH QUAKE SPECTRAL ACCELERATION, 1 SEC. PERIOD
	B	SEISMIC DESIGN CATEGORY, SHORT PERIOD
	B	SEISMIC DESIGN CATEGORY, 1 SECOND PERIOD
$\Omega_O=$	3	SYSTEM OVER STRENGTH FACTOR, TABLE 12.2-1
		Per PEMB Manufacuter, likely not detailed for seismic resistance
$R=$	3	RESPONSE MODIFICATION FACTOR FROM TABLE12.2-1
$C_S=$	0.100	$S_{DS}/(R/I_E)$, C_{SMAX} = $S_{D1}/(R/I_E)T$ for T< =T_L OR
		C_{SMAX} = $S_{D1}*T_L/(R/I_E)T^2$ for T>TL
$T_L=$	6	Long- period transition period Per ASCE 7-05 Figure 22-15 to 22-20
T=	0.246	Fundamental period determined by analysis, or Default Value of T<Cu Ta
C_S min (S_{1M}>0.6) =	0.010	0.5*$S_1/(R/I_E)$ WHERE S_{M1}>0.6g.
$h_n=$	14	HEIGHT IN FEET TO UPPER MOST LEVEL FROM BASE
$T_a=$	0.145	APPROXIMATE PERIOD OF VIBRATION OF THE BUILDING (SECONDS) = $C_t h_n^x$
$C_t=$	0.02	VIBRATION PERIOD COEFFICIENT (= 0.028 for steel MRF, 0.016 for concrete MRF, 0.03 for eccentrically braced steel frames, 0.02 for all other structural systems)
x=	0.75	VIBRATION PERIOD EXPONENT (= 0.8 for SMRF, 0.9 for CMRF, 0.75 for all other)
W=		TOTAL WEIGHT OF STRUCTURE

C_{SMIN} =	0.010
C_{SMAX} =	0.156
$S_{DS}/(R/I_E)$ =	0.100

DESIGN FORMULA		BASE SHEAR	REMARKS
V= C_S * W	=	0.100 * W	

$E= \rho*Q_E$ +- $0.2S_{DS}D$ E= SEISMIC EFFECT FOR USE IN LOAD COMBINATIONS

$E_m= \Omega_o*Q_E$ +- $0.2S_{DS}D$ E_m= SEISMIC EFFECT FOR USE IN SPECIAL LOAD COMBINATIONS

Q_E=	EFFECT OF HORIZONTAL SEISMIC FORCES, V or Fp		
D=	EFFECT OF DEAD LOAD FORCES		
ρ=	RELIABILITY FACTOR =	1.00	
	USE ρ =1.3 FOR SDC= D, E or F UNLESS ONE OF TWE CONDITIONS ARE MET, PER 12.3.4.2	= 1 FOR SEISMIC DESIGN CATEGORIES A,B,& C	

E=	1.00	* Q_E + -	0.06	*D
E_m=	3	* Q_E + -	0.06	*D

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STRUCTURAL COMPONENTS (PART OF SEISMIC FORCE-RESISTING SYSTEM):

Collectors Elements in Seismic Design Categories C through F : Section 12.10.2.1

Collector elements splices and their connections shall be designed with load combinations with overstrength.

Structural walls and their Anchorage: Section 12.11, (All Seismic Design Categories)

Perpendicular to wall loads:

$F_P = 0.4\ I_E\ S_{DS}\ W_W$, or $F_p = 0.10\ W_W = \boxed{0.120}\ W_W$ W_w = WEIGHT OF WALL

Anchorage of walls: F_P ABOVE OR 400 $S_{DS}\ I_E$ >= 280 PLF

$400\ S_{DS}\ I_E = \boxed{120}$ PLF >= 280 PLF

Seismic Design Category C and above : In accord with Category B except as follows:

Anchorage of walls: Out-of-Plane Wall Anchorage to Flexible Diaphragms:

$= 0.8\ S_{DS}\ I_E\ W_p = \boxed{0.239}\ W_p$, PLF >= 280 PLF

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NON-STRUCTURAL COMPONENTS:

$F_p = 0.40*a_p*S_{DS}*I_p* W_p/(Rp)*(1+2*z/h)$

$F_p >= .3*S_{DS}*I_p*W_p =$ 0.08976 I_p*W_p

$F_p <= 1.6*S_{DS}*I_p*W_p =$ 0.479 I_p*W_p

I_p = COMPONENT IMPORTANCE FACTOR (SECTION 13.1.3)

z = HEIGHT OF COMPONENT (FT), z<= h

h = 17 AVERAGE ROOF HEIGHT (FT)

*Per Section 9.6.1.6.1, R_p = 1.5 for shallow anchors (embedment length/diameter < 8)

**Per Section 13.4.2, increase design load by 1.3 for anchors embedded in concrete or masonry

DESCRIPTION OF ELEMENT (FROM TABLES 13.5-1 OR 13.6-1	a_p	R_p*	z	Ip**	F_p	REMARKS
INTERIOR NON-STRUCTURAL UNREINFORCED MAS. PARTITIONS	1.0	1.5	14	1.0	0.211 $*W_p$	
ALL OTHER INTERIOR NON-STRUCTURAL PARTITION WALLS	1.0	2.5	14	1.0	0.127 $*W_p$	
ALL OTHER INTERIOR NON-STRUCTURAL PARTITION WALLS	1	2.5	14	1.0	0.127 $*W_p$	
EXTERIOR NON-STRUCTURAL WALLS AND CONNECTIONS						
WALL ELEMENT	1	2.5	14	1.0	0.127 $*W_p$	
BODY OF CONNECTION	1	2.5	14	1.0	0.127 $*W_p$	
CONNECTION FASTENERS	1.25	1.0	14	1.0	0.396 $*W_p$	
CANTILEVERED PARAPETS	2.5	2.5	14	1.0	0.317 $*W_p$	
INTERIOR NON-STRUCTURAL UNREINFORCED MAS. PARTITIONS	1.0	1.5	20	1.0	0.268 $*W_p$	
ALL OTHER INTERIOR NON-STRUCTURAL PARTITION WALLS	1.0	2.5	20	1.0	0.161 $*W_p$	
ALL OTHER INTERIOR NON-STRUCTURAL PARTITION WALLS	1	2.5	20	1.0	0.161 $*W_p$	
EXTERIOR NON-STRUCTURAL WALLS AND CONNECTIONS						
WALL ELEMENT	1	2.5	20	1.0	0.161 $*W_p$	
BODY OF CONNECTION	1	2.5	20	1.0	0.161 $*W_p$	
CONNECTION FASTENERS	1.25	1.0	20	1.0	0.479 $*W_p$	
					#DIV/0! $*W_p$	
					#DIV/0! $*W_p$	
					#DIV/0! $*W_p$	

Project Gila Country Public Administration
Subject: Concrete Reinforcing Splice Lengths
Date: ######

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Reinforcing Splice Length Table Per ACI 318-05 and 2006 IBC - (Inches)

Rebar Size				Concrete member								
In-LB	Soft Metric	Area (in^2)	Diam. (in)	Footing (BOTT	FOOTING ~~Grade~~ Beam (top)	Grade Beam (bott.)	Wall Horiz. (top)	Wall Vert.	Slab	Beam (top)	Beam (bott.)	Column
#3	#10	0.11	0.375	16	17	16	12	16	16	16	16	*
#4	#13	0.20	0.500	19	23	18	19	19	20	20	16	15
#5	#16	0.31	0.625	24	28	22	28	28	30	25	19	19
#6	#19	0.44	0.750	29	34	26	37	37	40	29	23	23
#7	#22	0.60	0.875	41	49	38	60	60	64	48	37	27
#8	#25	0.79	1.000	47	56	43	74	74	80	61	47	30
#9	#29	1.00	1.128	53	69	54	90	90	96	75	58	34
#10	#32	1.27	1.270	60	85	66	108	108	116	91	70	39
#11	#36	1.56	1.410	66	103	79	127	127	136	109	84	43
#14	#43	2.25	1.690	*	*	*	*	*	*	*	*	*
#18	#57	4.00	2.257	*	*	*	*	*	*	*	*	*

* Generally not permitted

<u>Design Variables</u>

	Footings	Grade Beam		Wall		Slab	Beam		Columns
		Top	Bottom	Horiz.	Vert.		Top	Bottom	
fy (psi)	60000	60000	60000	60000	60000	60000	60000	60000	60000
f'c (psi)	2500	3000	3000	4000	4000	3500	4000	4000	3000
ψ t	1	1.3	1	1.3	1	1	1.3	1	
ψ e	1	1	1	1	1	1	1	1	
ψ s(3–6)	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	
ψ s(7+)	1	1	1	1	1	1	1	1	
λ	1	1	1	1	1	1	1	1	
Clear Cover c_b	3	2	2	0.75	0.75	0.75	1.5	1.5	
Bar spacing	7.5	5.5	5.5	3	3	12	4.5	4.5	
Ktr	0	0	0	0	0	0	0	0	
Class	1.3	1.3	1.3	1	1.3	1.3	1.3	1.3	1

ACI Section 12.2.3, Eqn. (12-1): l_d * Class=

$$\frac{3 * f_y * \psi t \psi e \psi s \lambda * d_b * CLASS}{40*(f'c)^{1/2} \; ((c_b+K_{tr})/d_b)} = \text{Splice length (see table)}$$

$(c_b+K_{tr})/db$ limited to <= 2.5

ψ_t – Reinforcing location factor (= 1.3 for top, = 1.0 for bottom)

ψ_e – Coating factor = 1.0 for no coating (increase to 1.2 or 1.5 for epoxy coating). See section 12.2.4.
Note: $\psi_t * \psi_e$ can be limited to a maximum value of 1.7, but is not in this spreadsheet.

ψ_s – Reinforcing size factor (0.8 up to #6, 1.0 for #7 and up)

λ – Aggregate Factor (= 1.0 for normal weight, =1.3 for lightweight)

c_b - Cover factor = the lesser of:
smallest clear cover to any edge + $d_b/2$, or one-half the center to center spacing of bars.

K_{tr} - Transverse reinf. index . A reduction factor if the bars being spliced are enclosed within stirrups. It is conservative to consider Ktr = 0. See section 12.2.4.

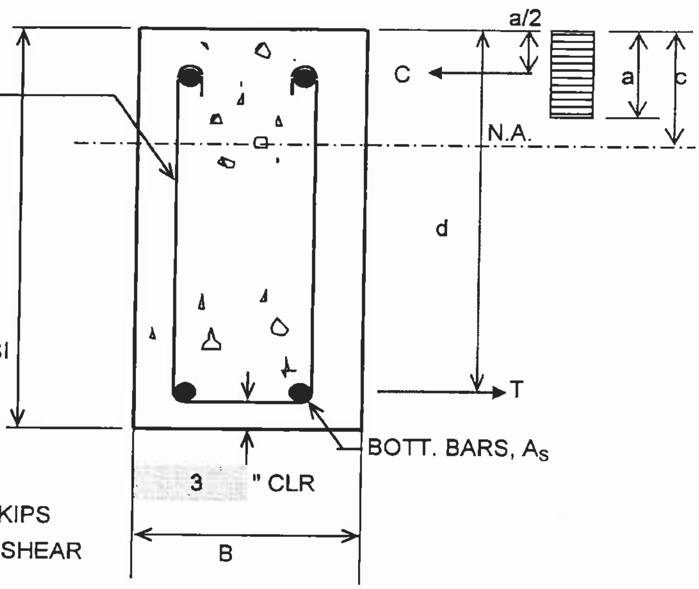
Class - = 1.0 for Class A or = 1.3 for Class B tension lap splices. Use Class B as typical unless you are sure that the bars will be less than 50% stressed in flexure and half or less of reinforcing is spliced within the required lap length.

SPLICES OFCOMPRESSION REINFORCING: (ACI 318-05 12.16)

l_{dc}= 0.0005 * $f_y \cdot d_b$ for f_y =< 60,000 PSI, or (0.0009 * f_y - 24) * d_b for f_y > 60,000 PSI
12" MINIMUM, IF f 'c < 3000 PSI than increase length of lap by 1/3

Project:	Gila Country Public Administration	Project Number:	30-09115-00
Subject:	CONCRETE BEAM DESIGN	Computed by:	EDN
Date:	10/20/09	Page:	

GENERAL DESIGN INFORMATION:



B = BEAM WIDTH, IN.
H = BEAM HEIGHT, IN.
d = DEPTH TO REINF. STEEL, IN.
= H - CLR - .375" - .5"
A_S = AREA OF BOTT. STEEL, IN^2.
A_V = AREA OF SHEAR REINF., IN^2.

f'_C = SPECIFIED COMPRESSIVE STRENGTH OF CONCRETE, PSI.
f_Y = SPECIFIED YIELD STRENGTH OF REINF.,PSI
Φ = STRENGTH REDUCTION FACTOR = 0.90
a = DEPTH OF EQUIVALENT RECTANGULAR BLOCK = β_1*c
M_N = NOMINAL MOMENT STRENGTH, KIP-FEET
V_C = NOMINAL CONCRETE SHEAR STRENGTH, KIPS
V_S = NOMINAL SHEAR STRENGTH PROVIDE BY SHEAR REINFORCEMENT, KIPS
V_N = NOMINAL SHEAR STRENGTH, KIPS = Vc + Vs
β_1 = 0.85 FOR f 'c <= 4000 PSI OR
= 0.85 - 0.05 x (f'_C -4000) / 1000
S = SPACING OF SHEAR REINF.
c = DISTANCE TO NEUTRAL AXIS,IN.
E_S = MODULAS OF ELASTICITY OF REINF., 29,000 KSI
E_C = MODULAS OF ELASTICITY OF CONCRETE,
= 57,000 $\sqrt{f'_C}$/1000, KSI
n = MODULAR RATIO, E_S / E_C

BENDING: C = T C = 0.85 * f'_C * B *a T = A_S * f_Y a = A_S* f_Y / (0.85 * f'_C * B)

$\Phi M_N = \Phi$ [A_S *f_Y (d - a/2)* 1/12000] Φ = 0.9 ρ_{MIN} = 200 / f_Y

ρ_{BAL} = 0.85 f'_C β_1 / f_Y * [87,000 / (87000 + f_Y)]

SHEAR: $V_C = 2\sqrt{f'_C}$ B * d

V_S = A_V * f_Y* d / S S_{MAX} = d/2 V_{SMAX} = 8 $\sqrt{f'_C}$ B * d

$\Phi V_N = \Phi V_C + \Phi V_S$ Φ = 0.75

DEFLECTION: I_{CR} = MOMENT OF INERTIA OF TRANSFORMED CRACKED SECTION

I_{CR} = 1/3 * B * c^2 + n * A_S * $(d - c)^2$

SOLVE FOR X : $c^2 + 2n\,A_S/B\ c - 2nA_S\ d/B = 0$

$$c = \frac{-(2nA_S/B) \pm \sqrt{(2nA_S/B)^2 + 4(2nA_S d/B)}}{2}$$

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TABLE OF CONCRETE BEAM CAPACITIES:

f'_C = 3,000 PSI
β_1 = 0.85

f_Y = 60,000 PSI
n = 9.3

ρ_{MIN} = 0.0033
$\rho_{MAX} = 0.75\rho_{BAL}$ = 0.0160

MARK	B	H	BOTT. BARS, A_S		SHEAR REINF, A_V		ΦM_N	ΦV_N	I_{CR}
	IN.	IN.	NO.	SIZE	SIZE	SPACING	FT*K	K	IN.^{4}
GB-1	24	36	AS< ASmin=2.57 3	6	4	48	187.7	63.3	9112
GB-2	24	42	AS< ASmin=3.05 3	6	4	12	223.5	131.3	13160
GB-3	24	48	AS< ASmin=3.53 3	6	4	12	259.3	152.0	17986
GB-4	36	36	AS< ASmin=3.855 4	6	4	12	250.9	142.3	12396
GB-5	36	42	AS< ASmin=4.575 4	6	4	12	298.6	168.9	17874
GB-6	36	48	AS< ASmin=5.295 4	6	4	12	346.3	195.5	24398
GB-1	24	36	5	7	4	12	414.7	110.7	17309
GB-1	24	42	AS< ASmin=3.05 5	7	4	12	495.9	131.3	25352
GB-1	24	48	AS< ASmin=3.53 5	7	4	12	577.1	152.0	35038
GB-1	36	36	AS< ASmin=3.855 6	7	4	12	502.4	142.3	21959
GB-1	36	42	6	7	4	12	599.8	168.9	32022
GB-1	36	48	AS< ASmin=5.295 6	7	4	12	697.3	195.5	44102

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MINIMUM BEAM WIDTH (INCHES) ACCORDING TO THE ACI CODE

B = BEAM WIDTH (IN.) $B = 2\ CLR + 2\ D_V + 2(0.293\ (0.75 - 0.5D_B)) + ND_B + (N-1)\ [D_B \text{ OR } 1\text{"MIN}]$

D_B = BAR DIAMETER (IN.)

CLR = SIDE CLEAR DISTANCE (IN.)

D_V = STIRRUP BAR DIAMETER (IN.)

N = NUMBER OF BARS

CLR = 3

D_V = 0.375

MINIUM BEAM WIDTH (B, INCHES)									
SIZE OF BAR	NUMBER OF BARS IN SINGLE LAYER OF REINFORCEMENT (N) 2	3	4	5	6	7	8	9	ADD FOR EACH ADDED BAR
4	9.0	10.5	12.0	13.5	15.0	16.5	18.0	19.5	1.5
5	9.3	10.9	12.5	14.1	15.8	17.4	19.0	20.6	1.625
6	9.5	11.2	13.0	14.7	16.5	18.2	20.0	21.7	1.75
7	9.7	11.6	13.4	15.3	17.2	19.1	20.9	22.8	1.875
8	9.9	11.9	13.9	15.9	17.9	19.9	21.9	23.9	2.00
9	10.2	12.5	14.7	17.0	19.2	21.5	23.7	26.0	2.25
10	10.6	13.1	15.6	18.1	20.6	23.1	25.6	28.1	2.50
11	10.9	13.7	16.4	19.2	21.9	24.7	27.4	30.2	2.75

Project:	Gila Country Public Administration	Project Number:	30-09115-00
Subject:	ANCHOR DESIGN ACI 318-05 APPENDIX D	Computed by:	EDN
Date:	10/20/09	Page:	

GENERAL INPUT

Loads are calculated for Load Combinations of ACI 318 Section 9.2 , Thus this spread sheet uses the strength reduction factors of section D4.4

This spread sheet does not apply to adhesive anchors, or grouted anchors.

When anchors design includes seismic loads, additional requirements of D.3.3.3 -D.3.3.5 shall be applied.
(Apply an additional strength reduction factor of 0.75 for anchors resisting moderate or high seismic loads.)

Anchor Bolts Per ASTM F 1554:

Grade	Tensile Strength (KSI)	Yield Strength (KSI)	Size Range	
36	58 - 80	36	1/4" - 2"	Ductile
55	75- -95	55	1/4" - 2"	Ductile
105	125-150	105	1/4" - 2"	Ductile

Per Standard See Table 2, and definition of Ductile steel element per ACI 318 Appendix D

Anchor Bolts Per ASTM A307, A325, A490, Table 7-10 & Table 7-14 AISC 3rd Edition

ASTM	ΦFv (kips)	ΦFt	
A307	18	33.8	Ductile
A325	36	67.5	
A490	45	84.8	

Threads included in shear plane

Project: Gila Country Public Administration
Subject: Spread Footing Design
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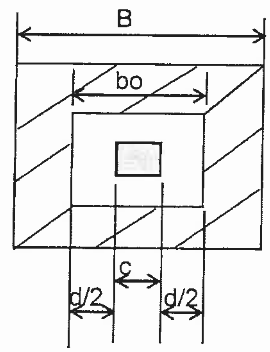
SPREAD FOOTING DESIGN: PER ACI 318-05 and 2006 IBC

Pa = SPREAD FOOTING CAPACITY, KIPS
f 'c = CONCRETE COMPRESSIVE STRENGTH AT 28 DAYS, PSI
Fy = REINFORCING STEEL YIELD STRENGTH, PSI
Qa = SAFE NET ALLOWABLE SOIL BEARING PRESSURE, PSF
B = WIDTH OF FOOTING, FT.
A = BEARING AREA OF FOOTING, BxB, SQ.FT.
c = MINIUM COLUMN DIMENSION, IN.
h = DEPTH OF FOOTING,IN.
CLR = CLR DISTANCE, 3"
d = DEPTH TO REINFORCEMENT, d = h - CLR. - 1/2", IN.
LF = LOAD FACTOR FOR CONCRETE DESIGN
Qu = FOOTING BEARING PRESSURE UNDER OVERLOAD, Qa x LF, PSF

FOOTING CAPACITY BASED ON SOIL BEARING PRESSURE:

Pa = A x Qa

TWO- WAY SHEAR: CRITICAL SECTION d/2 FROM FACE OF COLUMN



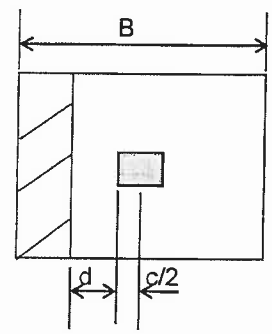
DETERMINE d REQUIRED FOR Pa ABOVE.
vc = 4 x SQRT(f 'c)

Vu = Qu (A - (bo/12)^2)
ΦVc = Φ * vc *4*bo* d
Vu = Φ Vc

bo = c + d,(USE hmin. INITIALLY, THEN USE d FROM PREVOIUS FOOTING,TO CALC. d).
Φ = 0.75

d_2 = Qu (A - (bo/12)^2) / (Φ*vc*4*bo)

ONE WAY SHEAR: ON CRITICAL SECTION d FROM FACE OF COLUMN

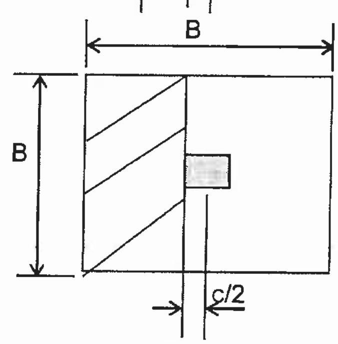


DETERMINE d REQUIRED FOR Pa ABOVE
vc = 2 x SQRT(f 'c)

Vu = Qu (B x(B/2 - d/12 -c/(2*12)))
ΦVc = Φ* vc*B*12*d   Φ = 0.75
Vu = Φ Vc
d_1 = Qu (B x(B/2 - d/12 -c/(2*12))) / (Φ*vc*B*12)

(USE hmin. INITIALLY, THEN USE d FROM PREVOIUS FOOTING,TO CALC. d).

AREA OF STEEL REQUIRED: CRITICAL SECTION AT FACE OF COLUMN



Mu = 1/2*Qu*B(B/2 - c/24)^2   Φ = 0.90
$Ru_{REQD.}$ =Mu / (Φ*B*d^2)   m = Fy/(.85*f 'c)
$As_{REQD.}$ =B*12*d*(1/m)*(1- SQRT(1 - 2*m*Ru / Fy))
ΦMn = Φ*As*Fy*(d - a/2)/12   a = As*Fy/(.85*f 'c*B*12)
Mu <= ΦMn

Project: Gila Country Public Administration
Subject: Spread Footing Design
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SPREAD FOOTING DESIGN: PER ACI 318-02 AND 2003 IBC

f 'c = 3000 PSI
Fy = 60,000 PSI
Qa = 2500 PSF
c = 5 IN.

LF = 1.5
CLR = 3 IN.
h_{MIN} = 12 IN.

SPREAD FOOTING BEARING CAPACITY & REINFORCEMENT

SIZE	B FT.	h IN.	Pa KIPS	REINFORCEMENT (NO. BARS) # 4	# 5	# 6	# 7	# 8	As_{REQD} IN.^{2}
2'-0" SQ.	2	12	10.0	3	2	2	1	1	0.518
3'-0" SQ.	3	12	22.5	4	3	2	2	1	0.778
4'-0" SQ.	4	12	40.0	6	4	3	2	2	1.037
5'-0" SQ.	5	14	62.5	8	5	4	3	2	1.512
6'-0" SQ.	6	17	90.0	12	8	5	4	3	2.203
7'-0" SQ.	7	18	122.5	14	9	7	5	4	2.722
8'-0" SQ.	8	22	160.0	20	13	9	7	5	3.802
9'-0" SQ.	9	23	202.5	23	15	11	8	6	4.471
10'-0" SQ.	10	26	250.0	29	19	13	10	8	5.616
11'-0" SQ.	11	28	302.5	34	22	16	12	9	6.653
12'-0" SQ.	12	31	360.0	41	27	19	14	11	8.035

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Project: Gila County Public Adminstration
Subject: MAXIUM HEIGHTS STEEL WALL FRAMING
Date: ########

Project Number: 30-09115-00
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MAXIMUM HEIGHT OF INTERIOR NON-LOAD BEARING STEEL STUDS IN GYPBOARD WALLS, 5 PSF, L/240, NON-COMPOSITE, PER SSMA

STUD SIZE	MAXIMUM HEIGHT, FEET SPACING , INCHES			DESIGN BASED ON SSMA SIZE
	12	16	24	
Non-structural	17'-1" 18'-5" 21'-0"	15'-6" 16'-9" 19'-1"	13'-6" 14'-7" 16'-8"	362S125-33 362S137-33 362S162-43
Non-structural	18'-11" 21'-8" 22'-8"	17'-3" 19'-8" 20'-7"	15'-0" 17'-2" 18'-0"	400S125-33 400S137-33 400S162-43
Non-structural	26'-3" 27'-5" 31'-2"	23'-11" 24'-11" 28'-4"	20'-10" 21'-9" 24'-9"	600S125-33 600S137-33 600S162-43
Non-structural	36'-5" 37'-10" 41'-1"	33'-1" 34'-4" 37'-4"	28'-11" 30'-0" 32'-7"	800S125-43 800S137-43 800S200-43

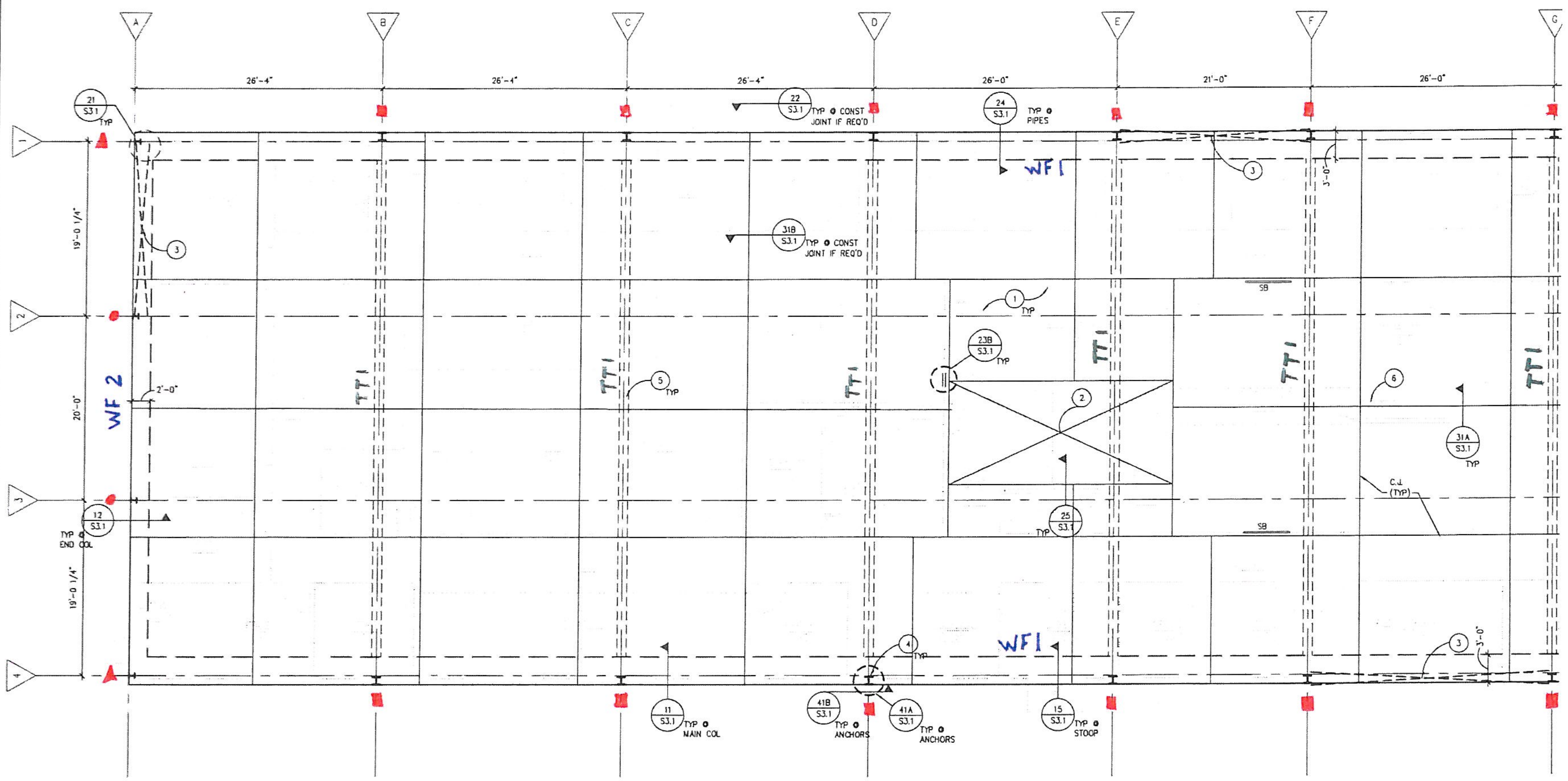
STEEL STUD FRAMING INSTALLATION SHALL COMPLY WITH ASTM C 754

CONSULT SSMA PRODUCT TRECHNICAL INFORMATION FOR HEIGHT GREATER THAN SHOWN IN TABLE.

■ AB1 GROUP MOMENT FRAME COLUMNS (ONE EDGE)

● AB2 GROUP END WALL COLUMNS (ONE EDGE)

▲ AB3 GROUP END WALL COLUMNS (TWO EDGE)



FOUNDATION DESIGN BASED ON THE FOLLOWING:
1. EAVE HEIGHT 14'-0"
2. 2 TO 12 ROOF SLOPE
3. FRAME LAYOUT AS SHOWN

* CONTACT FOUNDATION ENGINEER ANY OF THESE ITEMS ARE REVISED TO DETERMINE FOUNDATION CAPACITY.

B2

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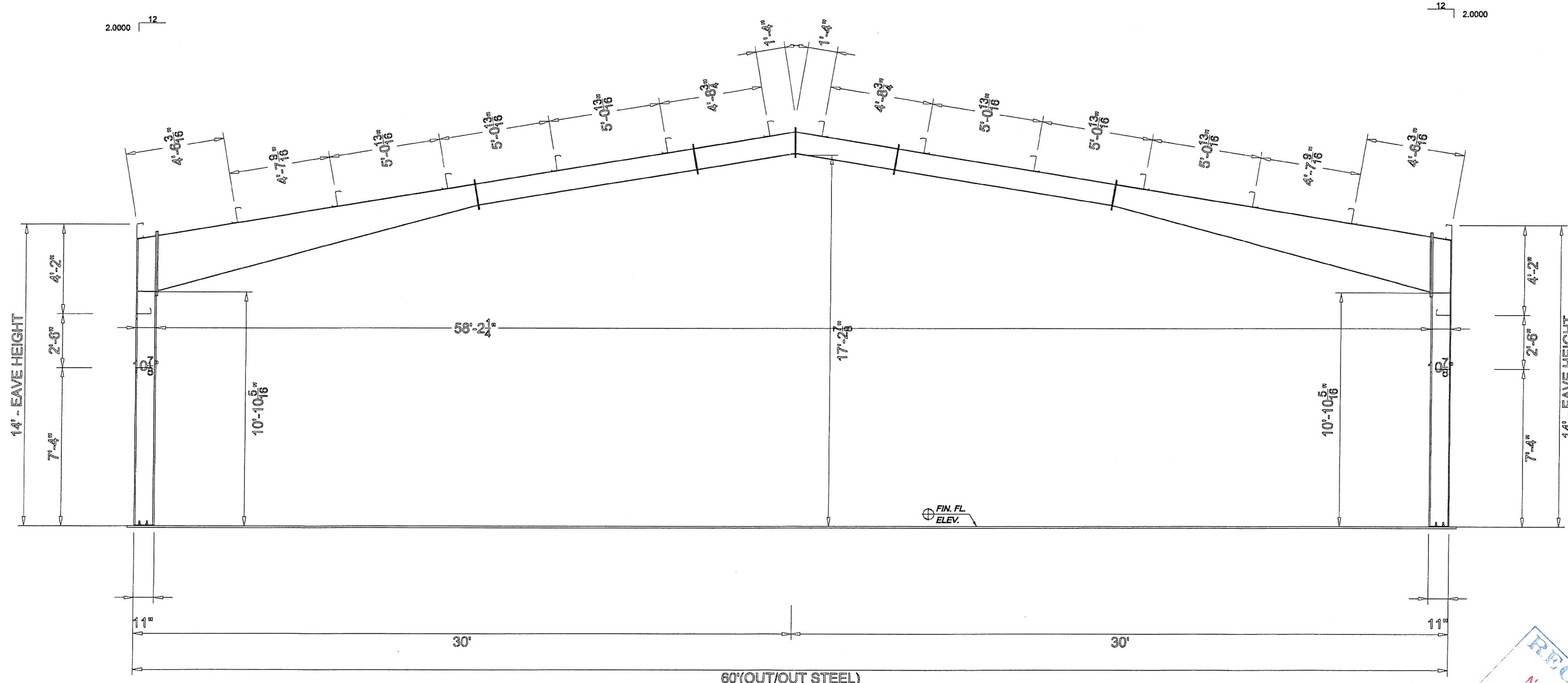

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CROSS SECTION AT FRAME LINE "5" - (A) ADMIN BLDG

C
B D
A

DESIGNS SHOWN ARE BASED ON THE BASIC BUILDING ITSELF, THEY DO NOT INCLUDE LOADS FROM ANY BUILDING OPTIONS OR ANY OTHER MATERIAL

FRAME CLEARANCES SHOWN ARE APPROXIMATE AND MAY VARY DUE TO FIELD CONDITIONS AND LOADS.
VERTICAL CLEARANCE DIMENSIONS ARE FROM FINISHED FLOOR REFERENCE ELEVATION.

DESIGN DATA/ANCHOR ROD DESIGN BASED ON

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CONTACT SALES ENGINEER FOR REVIEW BEFORE USING THIS INFORMATION FOR CONSTRUCTION

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JOB NAME	PUBLIC WORKS BLDG
VERSION	Optima 1.3.2
PAPER SIZE	22x34
ENTERED BY	Wynn
SCALE	Best Fit
DATE	10/20/09

METALLIC BUILDING COMPANY

BUILDER: Architectural Building Systems, LLC
1720 W. Lincoln Street
Phoenix, AZ 85007
COUNTY:
CONTACT: Wynn Pratt

CUSTOMER: Gila County
Globe, AZ

JOB ID
MEMBER MBMA
REVISION

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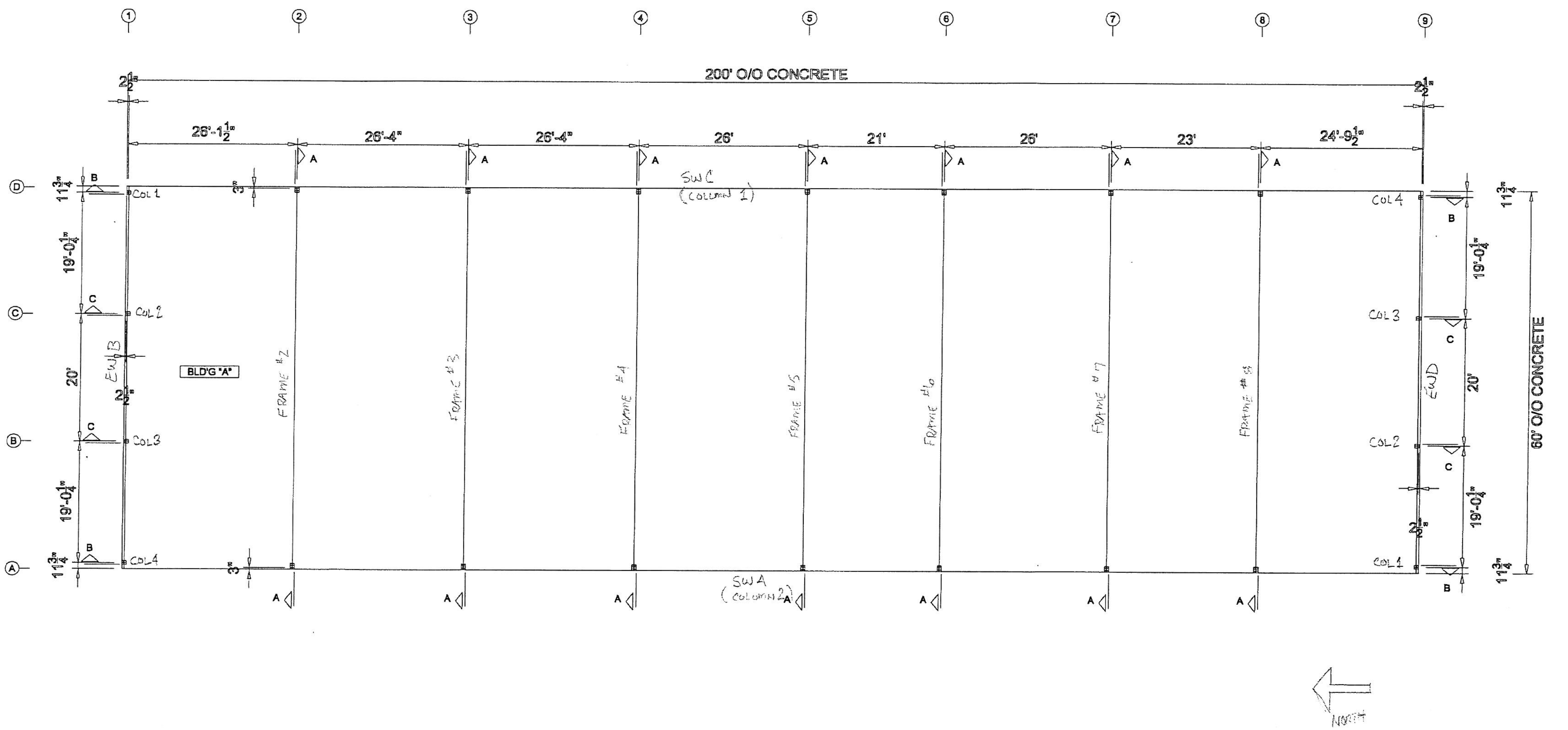
ACCESSORY SCHEDULE

Mark	Description	Qty.



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ANCHOR ROD PLAN - (A) ADMIN BLDG

C
B D
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FOUNDATION MUST BE SQUARE AND LEVEL. ALL ANCHOR RODS MUST BE TRUE IN SIZE, LOCATION, AND PROJECTION. ANCHOR ROD PROJECTIONS MUST BE HELD TO KEEP THREADS CLEAR OF FINISHED CONCRETE.

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THIS DRAWING IS FOR ANCHOR ROD PLACEMENT ONLY AND IS NOT A SUBSTITUTE FOR FOUNDATION DESIGN.

CONTACT SALES ENGINEER FOR REVIEW BEFORE USING THIS INFORMATION FOR CONSTRUCTION

MANUFACTURER RESERVES THE RIGHT TO CHANGE THE FINAL DESIGN. IF DESIGN INFORMATION (IE. CLEARANCES, BASE PLATE/ANCHOR ROD DESIGN) IS TO BE USED FOR CONSTRUCTION MANUFACTURER MUST BE NOTIFIED PRIOR TO ACCEPTANCE OF ORDER.

IT IS THE BUILDERS RESPONSIBILITY TO COMMUNICATE TO MANUFACTURER THE NEED TO HOLD TO ANY PRELIMINARY DESIGN INFORMATION PROVIDED BY MANUFACTURER! MANUFACTURER WILL NOT BE LIABLE FOR ANY CHANGES IN FINAL DESIGN IF THE BUILDER DOES NOT COMMUNICATE TO MANUFACTURER!

JOB NAME	PUBLIC WORKS BLDG
VERSION	Optima 1.3.2
PAPER SIZE	22x34
ENTERED BY	Wynn
SCALE	Best Fit
DATE	10/20/09

METALLIC BUILDING COMPANY

BUILDER: Architectural Building Systems, LLC
1720 W. Lincoln Street
Phoenix, AZ 85007
COUNTY:
CONTACT: Wynn Pratt

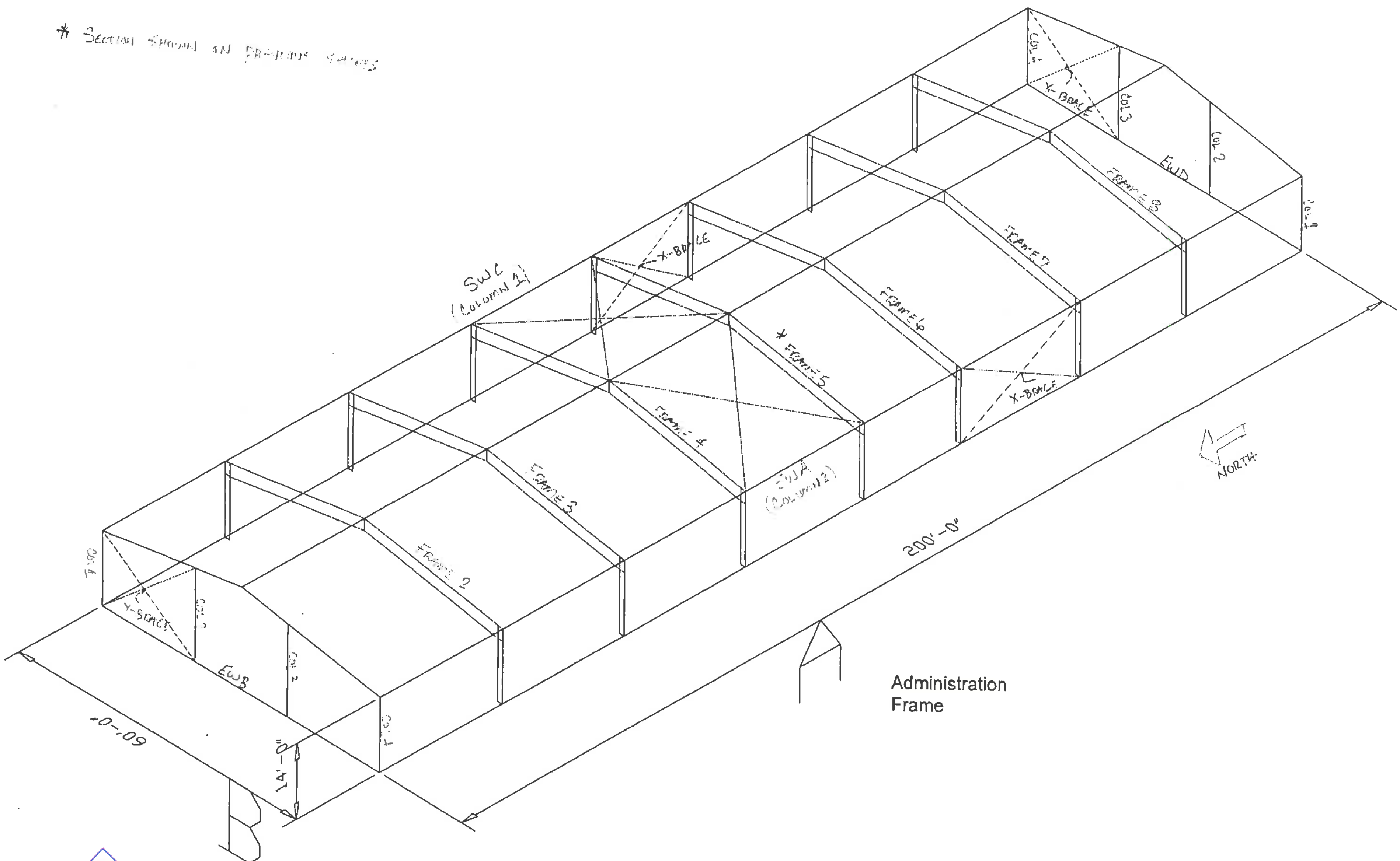
CUSTOMER: Gila County
Globe, AZ

JOB ID

MEMBER: MBMA

REVISION

* Section shown in [illegible] [illegible]



Administration
Frame

C1

METALLIC BUILDING COMPANY

Design Report
Optima 1.3.2
38315

Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

Architectural Building Systems, LLC

Note: All design information provided is preliminary, including but not limited to "Designed", "System Standard" and "Default" design criteria. The Manufacturer will not be responsible for conditions resulting from changes in the final design unless that specific requirement is noted on the Purchase Order.

BUILDING CODE

Project Use Category:	Commercial	**Jobsite State:**	AZ
Building Code:	2006 IBC	**Jobsite County:**	Gila
		Jobsite City:	Globe

Live/Wind

Live Load:	20.000 psf		
Reduction:	No		
Wind Load:	90.00 mph	**Wind Category:**	N/A
Wind Exposure:	Exposure C	**Miles From Coastline:**	N/A
Hurricane Coastline:	No	**Rain Intensity:**	5.0000 in/hr

Snow

Ground Snow Load:	0.000 psf	**Snow Exposure:**	N/A
Min Roof Snow Load:	0.000 psf	**Rain Load:**	N/A
Thermal Condition:	N/A	**Sea Level Elevation:**	N/A

SNOW LOAD = 20 PSF SAME AS UNREDUCED LIVE LOAD

Seismic

Spectral Response(Ss):	37.40 %	**% of Snow Load for Seismic:**	Normal
Spectral Response(Sh):	N/A	**Seismic Zone:**	N/A
Spectral Response(S1):	10.20 %	**Near Source Factor:**	N/A
Spectral Response(S2):	N/A	**Design Seismic For Schools:**	N/A
Velocity Coefficient(Aa):	N/A	**Site Class/Soil Type:**	(D) Stiff Soil
Accelerated Coefficient(Av):	N/A		

METALLIC BUILDING COMPANY

Design Report
Optima 1.3.2
38315

Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

BUILDING A - ADMIN BLDG

Label:	A	**Type:**	Stand Alone
Structure:	New	**Frame Type:**	Symmetrical
		Elevation A:	Sidewall

GEOMETRY, SIDEWALLS & ENDWALLS

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PERMIT

DATE:

Width: 60'-0"

Length: 200'-0"

SWA

Eave Height:	14'-0"
Roof Slope:	2.000000 / 12
Dist. to Ridge:	30'-0"
Girts:	8.0" - Flush

SWC

Eave Height:	14'-0"
Roof Slope:	2.000000 / 12
Dist. to Ridge:	30'-0"
Girts:	8.0" - Flush

EWB

Type:	Bearing Frame with Cold-Form Rafter
Girts:	8.0" - Flush
Setback:	System Standard

EWD

Type:	Bearing Frame with Cold-Form Raft
Girts:	8.0" - Flush
Setback:	System Standard

Purlins:	8.0" Z	**Pregalv. Secondary:**	No
nary Steel Shop Coat:	Red	**Frame Bolt Washers:**	No

METALLIC BUILDING COMPANY

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Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

ꞮUILDING A - ADMIN BLDG

ꞮPACINGS:

Bay Spacing (EWB-EWD):	3@26'-4", 26'-0", 21'-0", 26'-0", 23'-0", 25'-0"	EWB South End
EWB COL. Spacing (SWC-SWA):	3@20'-0"	EWD North End
EWD COL. Spacing (SWA-SWC):	3@20'-0"	

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SWA Girt Locations (Base to Eave): System Standard: 7'-4", 9'-10"
SWC Girt Locations (Base to Eave): System Standard: 7'-4", 9'-10"
EWB Girt Locations (Base to Peak): System Standard: 7'-4", 12'-11"
EWD Girt Locations (Base to Peak): System Standard: 7'-4", 12'-11"

Purlin Spacing: System Standard
Purlin Locations (SWA - Eave to Peak): 4'-7 9/16", 9'-3 1/16", 14'-3 15/16", 19'-4 3/4", 24'-5 9/16", 29'-2 5/16"

Purlin Locations (SWC - Eave to Peak): 4'-7 9/16", 9'-3 1/16", 14'-3 15/16", 19'-4 3/4", 24'-5 9/16", 29'-2 5/16"

*Note: Purlin and girt depths and locations are supplied for reference only, and may be changed at Manufacturer's discretion without notice unless specifically stated otherwise in the "Notes" section of this document.

ꞮRAME GROUPS

Group Number: 1

Frame Lines: 2, 3, 4, 5, 6, 7, 8

SWA

Column:	Straight Required
Max Col. Web Depth:	11.00"
Max Raf. Web Depth:	68.00"
Ext Col. Elevation:	At Finished Floor

SWC

Column:	Straight Required
Max Col. Web Depth:	11.00"
Max Raf. Web Depth:	68.00"
Ext Col. Elevation:	At Finished Floor

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Design Report
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Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

Building A - ADMIN BLDG

LOADS, WIND ENCLOSURE, DEFLECTIONS & DRIFTS

Building Loads

Roof Snow Load By Design:	0.000 psf
Occupancy Category:	II - Normal
Thermal Condition:	N/A
Seismic Design Category:	C

Importance Factors

Snow Is:	1.00
Wind Iw:	1.00
Seismic Ie:	1.00
Designed Snow Exposure:	

Wind Enclosure

Wind Enclosure:	Calculated - Enclosed
Are all Framed Openings enclosed with materials designed to resist building wind loads:	Yes
Are all Open Areas for Other enclosed with materials designed to resist building wind loads:	Yes

Uniform Collateral Loads

Ceiling Load:	0.000 psf	Other:	5.000 psf
Plaster/Sheetrock Ceiling:	No		
Brittle/Dryvit:	No		

Deflections

Purlins Live:	L/150 - Default	Rafters Live:	L/150 - Default
Purlins Snow:	L/180 - Default	Rafters Snow:	L/180 - Default
Purlins Wind:	L/180 - Default	Rafters Wind:	L/180 - Default
Purlins Total Gravity:	L/120 - Default	Rafters Total Gravity:	L/120 - Default
Purlins Total Uplift:	L/N/A - Default	Rafters Total Uplift:	L/N/A - Default
Girts:	L/120 - Default	Endwall Columns:	L/120 - Default

Drifts

Portal Frame Wind:	H/60 - Default
Portal Frame Seismic:	H/50 - Default
Crane:	H/100 - Default
Frame Live:	H/60 - Default
Frame Snow:	H/60 - Default
Frame Wind:	H/60 - Default
Frame Seismic:	H/50 - Default
Frame Total Gravity:	H/60 - Default
Frame Total Wind:	H/60 - Default
Frame Total Seismic:	H/50 - Default

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Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

BUILDING A - ADMIN BLDG

BRACING

SWA	Cable	(EWB to EWD) @ Bays:	6
Roof:	Cable	(EWB to EWD) @ Bays:	4
SWC:	Cable	(EWD to EWB) @ Bays:	4
EWB:	Cable	(SWC to SWA) @ Bays:	1
EWD:	Cable	(SWA to SWC) @ Bays:	3
Purlin:	Angles		
Girt:	None		

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B

ROOF PANEL (12,166 sqft)

Type:	Ultra-Dek
Gauge:	24
Thickness:	N/A
Color:	SIG - 300 TBD*
Finish Warranty:	No
Interior Panel:	N/A

Options

SS Clip Type:	N/A
Thermal Blocks:	N/A
UL90:	N/A
UL Letter:	N/A
IBL Tools:	No
	N/A
	No
	N/A

Fastener Information

Type:	Self-Drilling
Head Finish:	Long-Life
Length:	Standard

WALL PANEL (7,430 sqft)

e:	PBR
Gauge:	26
Color:	SIG - 200 TBD
Thickness:	N/A
Finish Warranty:	N/A
Interior Panel:	

Options

Concrete Notch:	No

Fastener Information

	Self-Drilling
Type:	Standard
Head Finish:	1-1/4"
Length:	

METALLIC BUILDING COMPANY

Design Report
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Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

BUILDING A - ADMIN BLDG

DESIGN DATA FRAME(S): 2

Inside Clearance:	58'-0 3/4"	**Peak Clearance:**	16'-11 7/8"
		Peak Rafter Depth:	15.06"

Column 1 (SWC)

Column Depth		**Knee**	
Base:	10.88"	Rafter Depth:	30.69"
Knee:	10.88"	Clearance:	10'-10 1/4"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	8.00"
Gauge:	4.00"	Thickness:	0.38"

Maximum Reactions		
Vertical:	22.91 Kips	-9.76 Kips
Horizontal:	16.01 Kips	-6.19 Kips
Longitudinal:	0.00 Kips	0.00 Kips

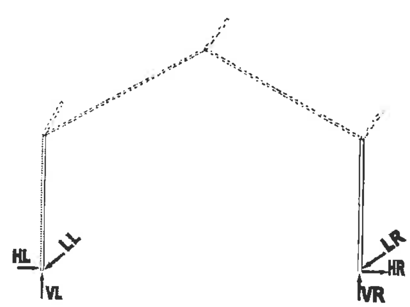
Column 2 (SWA)

Column Depth		**Knee**	
Base:	10.88"	Rafter Depth:	30.69"
Knee:	10.88"	Clearance:	10'-10 1/4"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	8.00"
Gauge:	4.00"	Thickness:	0.38"

Maximum Reactions		
Vertical:	22.91 Kips	-9.76 Kips
Horizontal:	6.19 Kips	-16.01 Kips
Longitudinal:	0.00 Kips	0.00 Kips

```
*****************************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which   *
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.            *
* Anchor rods are not supplied by Manufacturer.                                           *
*****************************************************************************************
```

Column 1 (SWC) -->



ꞁUILDING A - ADMIN BLDG

ndividual Loads - Unfactored

	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Lateral Primary Wind Load 1	-9.726 Kips	-7.322 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-5.503 Kips	-5.475 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-7.287 Kips	-3.690 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-3.063 Kips	-1.843 Kips	-0.000 Kips
Lateral Seismic Load	-0.409 Kips	-1.002 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-11.723 Kips	-5.273 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-10.430 Kips	-5.563 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.091 Kips	-2.623 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-6.547 Kips	-2.745 Kips	-0.000 Kips
Roof Collateral Load	3.900 Kips	2.753 Kips	-0.000 Kips
Roof Dead Load	3.264 Kips	1.890 Kips	-0.000 Kips
Roof Live Load	15.600 Kips	11.013 Kips	-0.000 Kips
Column 2 (SWA)			
Lateral Primary Wind Load 1	-7.287 Kips	3.690 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-3.063 Kips	1.843 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-9.726 Kips	7.322 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-5.503 Kips	5.475 Kips	-0.000 Kips
Lateral Seismic Load	0.409 Kips	-1.002 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-10.430 Kips	5.563 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-11.723 Kips	5.273 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-6.547 Kips	2.745 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.091 Kips	2.623 Kips	-0.000 Kips
Roof Collateral Load	3.900 Kips	-2.753 Kips	-0.000 Kips
Rɔof Dead Load	3.264 Kips	-1.890 Kips	-0.000 Kips
f Live Load	15.600 Kips	-11.013 Kips	-0.000 Kips

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METALLIC BUILDING COMPANY

Design Report
Optima 1.3.2
38315

Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

)ESIGN DATA FRAME(S): 3

Inside Clearance:	58'-0 3/4"	**Peak Clearance:**	17'-2 7/8"
		Peak Rafter Depth:	12.13"

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Column 1 (SWC)

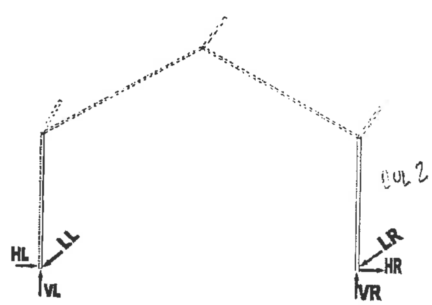
Column Depth		**Knee**	
Base:	10.88"	**Rafter Depth:**	30.69"
Knee:	10.88"	**Clearance:**	10'-10 1/4"
Anchor Rods		**Base Plate:**	
Quantity:	4	**Length:**	11.00"
Diameter:	0.75"	**Width:**	8.00"
Gauge:	4.00"	**Thickness:**	0.38"
Maximum Reactions			
Vertical:	23.23 Kips	-9.89 Kips	
Horizontal:	16.70 Kips	-6.41 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

Column 2 (SWA)

Column Depth		**Knee**	
'ase:	10.88"	**Rafter Depth:**	30.69"
,nee:	10.88"	**Clearance:**	10'-10 1/4"
Anchor Rods		**Base Plate:**	
Quantity:	4	**Length:**	11.00"
Diameter:	0.75"	**Width:**	8.00"
Gauge:	4.00"	**Thickness:**	0.38"
Maximum Reactions			
Vertical:	23.23 Kips	-9.89 Kips	
Horizontal:	6.41 Kips	-16.70 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

```
******************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which  *
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.            *
* Anchor rods are not supplied by Manufacturer.                                         *
******************************************************************************
```

Column 1 (SWC) -->



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Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

ndividual Loads - Unfactored

	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Lateral Primary Wind Load 1	-9.855 Kips	-7.581 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-5.575 Kips	-5.643 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-7.384 Kips	-3.903 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-3.104 Kips	-1.965 Kips	-0.000 Kips
Lateral Seismic Load	-0.415 Kips	-1.017 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-11.878 Kips	-5.519 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-10.568 Kips	-5.813 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.185 Kips	-2.757 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-6.634 Kips	-2.881 Kips	-0.000 Kips
Roof Collateral Load	3.950 Kips	2.870 Kips	-0.000 Kips
Roof Dead Load	3.320 Kips	1.950 Kips	-0.000 Kips
Roof Live Load	15.800 Kips	11.478 Kips	-0.000 Kips
Column 2 (SWA)			
Lateral Primary Wind Load 1	-7.384 Kips	3.903 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-3.104 Kips	1.965 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-9.855 Kips	7.581 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-5.575 Kips	5.643 Kips	-0.000 Kips
Lateral Seismic Load	0.415 Kips	-1.017 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-10.568 Kips	5.813 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-11.878 Kips	5.519 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-6.634 Kips	2.881 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.185 Kips	2.757 Kips	-0.000 Kips
Roof Collateral Load	3.950 Kips	-2.870 Kips	-0.000 Kips
Roof Dead Load	3.320 Kips	-1.950 Kips	-0.000 Kips
f Live Load	15.800 Kips	-11.478 Kips	-0.000 Kips

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)ESIGN DATA FRAME(S): 4

Inside Clearance:	58'-0 3/4"	**Peak Clearance:**	17'-2 7/8"
		Peak Rafter Depth:	12.13"

Column 1 (SWC)

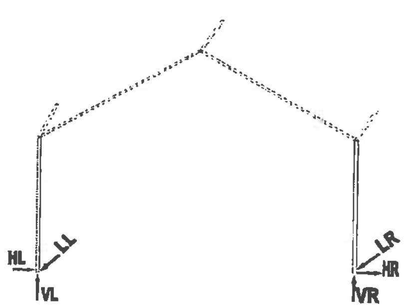
Column Depth		**Knee**	
Base:	10.88"	Rafter Depth:	30.69"
Knee:	10.88"	Clearance:	10'-10 1/4"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	8.00"
Gauge:	4.00"	Thickness:	0.38"
Maximum Reactions			
Vertical:	23.09 Kips	-9.82 Kips	
Horizontal:	16.59 Kips	-6.37 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

Column 2 (SWA)

Column Depth		**Knee**	
'ase:	10.88"	Rafter Depth:	30.69"
.nee:	10.88"	Clearance:	10'-10 1/4"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	8.00"
Gauge:	4.00"	Thickness:	0.38"
Maximum Reactions			
Vertical:	23.09 Kips	-9.82 Kips	
Horizontal:	6.37 Kips	-16.59 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

```
*******************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which  *
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.            *
* Anchor rods are not supplied by Manufacturer.                                          *
*******************************************************************************
```

Column 1 (SWC) -->



METALLIC BUILDING COMPANY

Design Report
Optima 1.3.2
38315

Architectual Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

ndividual Loads - Unfactored

ndividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Lateral Primary Wind Load 1	-9.793 Kips	-7.533 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-5.540 Kips	-5.608 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-7.338 Kips	-3.878 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-3.085 Kips	-1.953 Kips	-0.000 Kips
Lateral Seismic Load	-0.412 Kips	-1.012 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-11.803 Kips	-5.485 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-10.502 Kips	-5.776 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-7.140 Kips	-2.740 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-6.592 Kips	-2.863 Kips	-0.000 Kips
Roof Collateral Load	3.925 Kips	2.851 Kips	-0.000 Kips
Roof Dead Load	3.307 Kips	1.941 Kips	-0.000 Kips
Roof Live Load	15.700 Kips	11.406 Kips	-0.000 Kips
Column 2 (SWA)			
Lateral Primary Wind Load 1	-7.338 Kips	3.878 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-3.085 Kips	1.953 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-9.793 Kips	7.533 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-5.540 Kips	5.608 Kips	-0.000 Kips
Lateral Seismic Load	0.412 Kips	-1.012 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-10.502 Kips	5.776 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-11.803 Kips	5.485 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-6.592 Kips	2.863 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-7.140 Kips	2.740 Kips	-0.000 Kips
Roof Collateral Load	3.925 Kips	-2.851 Kips	-0.000 Kips
P~of Dead Load	3.307 Kips	-1.941 Kips	-0.000 Kips
f Live Load	15.700 Kips	-11.406 Kips	-0.000 Kips

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METALLIC BUILDING COMPANY

Design Report
Optima 1.3.2
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Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

)ESIGN DATA FRAME(S): 5

Inside Clearance:	58'-0 3/4"	**Peak Clearance:**	17'-2 7/8"
		Peak Rafter Depth:	12.13"

Column 1 (SWC)

Column Depth		**Knee**	
Base:	10.88"	Rafter Depth:	30.63"
Knee:	10.88"	Clearance:	10'-10 5/16"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	8.00"
Gauge:	4.00"	Thickness:	0.38"
Maximum Reactions			
Vertical:	20.80 Kips	-10.78 Kips	
Horizontal:	14.83 Kips	-5.68 Kips	
Longitudinal:	0.00 Kips	-5.07 Kips	

Column 2 (SWA)

Column Depth		**Knee**	
'ase:	10.88"	Rafter Depth:	30.63"
.(nee:	10.88"	Clearance:	10'-10 5/16"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	8.00"
Gauge:	4.00"	Thickness:	0.38"
Maximum Reactions			
Vertical:	20.80 Kips	-8.77 Kips	
Horizontal:	5.68 Kips	-14.83 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

```
*************************************************************************************
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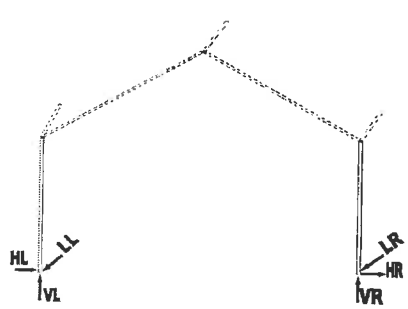
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.        *
* Anchor rods are not supplied by Manufacturer.                                     *
*************************************************************************************
```

Column 1 (SWC) -->



ndividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Brace Downward forces due to Longitudinal Wind	2.000 Kips	-0.014 Kips	-0.000 Kips
Brace Downward forces due to Seismic	4.848 Kips	-0.033 Kips	-0.000 Kips
Brace Upward forces due to Longitudinal Wind	-2.000 Kips	0.015 Kips	-2.988 Kips
Brace Upward forces due to Seismic	-4.848 Kips	0.036 Kips	-7.244 Kips
Lateral Primary Wind Load 1	-8.793 Kips	-6.737 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-4.974 Kips	-5.021 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-6.588 Kips	-3.454 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-2.770 Kips	-1.738 Kips	-0.000 Kips
Lateral Seismic Load	-0.375 Kips	-0.918 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-10.597 Kips	-4.893 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-9.429 Kips	-5.154 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-6.410 Kips	-2.441 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-5.919 Kips	-2.551 Kips	-0.000 Kips
Roof Collateral Load	3.525 Kips	2.547 Kips	-0.000 Kips
Roof Dead Load	3.036 Kips	1.767 Kips	-0.000 Kips
Roof Live Load	14.100 Kips	10.188 Kips	-0.000 Kips
Column 2 (SWA)			
Brace Downward forces due to Longitudinal Wind	-0.008 Kips	0.014 Kips	-0.000 Kips
Brace Downward forces due to Seismic	-0.019 Kips	0.033 Kips	-0.000 Kips
Brace Upward forces due to Longitudinal Wind	0.008 Kips	-0.015 Kips	-0.000 Kips
Brace Upward forces due to Seismic	0.019 Kips	-0.036 Kips	-0.000 Kips
Lateral Primary Wind Load 1	-6.588 Kips	3.454 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-2.770 Kips	1.738 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-8.793 Kips	6.737 Kips	-0.000 Kips
ral Primary Wind Load 4	-4.974 Kips	5.021 Kips	-0.000 Kips
eral Seismic Load	0.375 Kips	-0.918 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-9.429 Kips	5.154 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-10.597 Kips	4.893 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-5.919 Kips	2.551 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-6.410 Kips	2.441 Kips	-0.000 Kips
Roof Collateral Load	3.525 Kips	-2.547 Kips	-0.000 Kips
Roof Dead Load	3.036 Kips	-1.767 Kips	-0.000 Kips
Roof Live Load	14.100 Kips	-10.188 Kips	-0.000 Kips

METALLIC BUILDING COMPANY

Design Report
Optima 1.3.2
38315

Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

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PERMIT #
DATE:

)ESIGN DATA FRAME(S): 6

Inside Clearance:	58'-0 3/4"	**Peak Clearance:**	17'-2 7/8"
		Peak Rafter Depth:	12.13"

Column 1 (SWC)

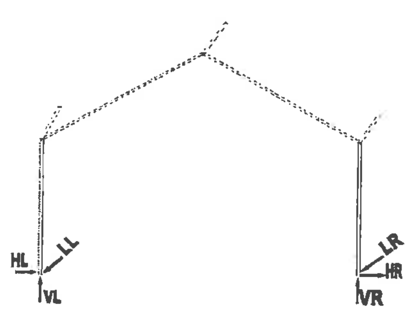
Column Depth		**Knee**	
Base:	10.88"	Rafter Depth:	30.63"
Knee:	10.88"	Clearance:	10'-10 5/16"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	8.00"
Gauge:	4.00"	Thickness:	0.38"
Maximum Reactions			
Vertical:	20.80 Kips	-10.77 Kips	
Horizontal:	14.83 Kips	-5.68 Kips	
Longitudinal:	0.00 Kips	-5.07 Kips	

Column 2 (SWA)

Column Depth		**Knee**	
`ase:	10.88"	Rafter Depth:	30.63"
.nee:	10.88"	Clearance:	10'-10 5/16"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	8.00"
Gauge:	4.00"	Thickness:	0.38"
Maximum Reactions			
Vertical:	20.80 Kips	-10.38 Kips	
Horizontal:	5.68 Kips	-14.83 Kips	
Longitudinal:	0.00 Kips	-5.07 Kips	

```
**************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which  *
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.            *
* Anchor rods are not supplied by Manufacturer.                                          *
**************************************************************************
```

Column 1 (SWC) -->



ndividual Loads - Unfactored

	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Brace Downward forces due to Longitudinal Wind	1.993 Kips	-0.024 Kips	-0.000 Kips
Brace Downward forces due to Seismic	4.833 Kips	-0.059 Kips	-0.000 Kips
Brace Upward forces due to Longitudinal Wind	-1.993 Kips	0.027 Kips	-2.988 Kips
Brace Upward forces due to Seismic	-4.833 Kips	0.065 Kips	-7.244 Kips
Lateral Primary Wind Load 1	-8.793 Kips	-6.737 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-4.974 Kips	-5.021 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-6.588 Kips	-3.454 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-2.770 Kips	-1.738 Kips	-0.000 Kips
Lateral Seismic Load	-0.375 Kips	-0.918 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-10.597 Kips	-4.893 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-9.429 Kips	-5.154 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-6.410 Kips	-2.441 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-5.919 Kips	-2.551 Kips	-0.000 Kips
Roof Collateral Load	3.525 Kips	2.547 Kips	-0.000 Kips
Roof Dead Load	3.036 Kips	1.767 Kips	-0.000 Kips
Roof Live Load	14.100 Kips	10.188 Kips	-0.000 Kips
Column 2 (SWA)			
Brace Downward forces due to Longitudinal Wind	1.608 Kips	0.024 Kips	-0.000 Kips
Brace Downward forces due to Seismic	3.897 Kips	0.059 Kips	-0.000 Kips
Brace Upward forces due to Longitudinal Wind	-1.608 Kips	-0.027 Kips	-2.988 Kips
Brace Upward forces due to Seismic	-3.897 Kips	-0.065 Kips	-7.244 Kips
Lateral Primary Wind Load 1	-6.588 Kips	3.454 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-2.770 Kips	1.738 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-8.793 Kips	6.737 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-4.974 Kips	5.021 Kips	-0.000 Kips
Lateral Seismic Load	0.375 Kips	-0.918 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-9.429 Kips	5.154 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-10.597 Kips	4.893 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-5.919 Kips	2.551 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-6.410 Kips	2.441 Kips	-0.000 Kips
Roof Collateral Load	3.525 Kips	-2.547 Kips	-0.000 Kips
Roof Dead Load	3.036 Kips	-1.767 Kips	-0.000 Kips
Roof Live Load	14.100 Kips	-10.188 Kips	-0.000 Kips

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METALLIC BUILDING COMPANY

Design Report Optima 1.3.2 38315

Architectual Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

)ESIGN DATA FRAME(S): 7

Inside Clearance:	58'-0 3/4"	Peak Clearance:	17'-2 15/16"
		Peak Rafter Depth:	12.00"

Column 1 (SWC)

Column Depth		**Knee**	
Base:	10.88"	Rafter Depth:	30.56"
Knee:	10.88"	Clearance:	10'-10 3/8"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	8.00"
Gauge:	4.00"	Thickness:	0.38"

Maximum Reactions

Vertical:	21.62 Kips	-9.19 Kips
Horizontal:	15.53 Kips	-5.96 Kips
Longitudinal:	0.00 Kips	0.00 Kips

Column 2 (SWA)

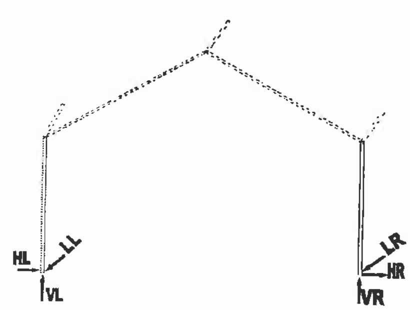
Column Depth		**Knee**	
'ase:	10.88"	Rafter Depth:	30.56"
.nee:	10.88"	Clearance:	10'-10 3/8"
Anchor Rods		**Base Plate:**	
Quantity:	4	Length:	11.00"
Diameter:	0.75"	Width:	8.00"
Gauge:	4.00"	Thickness:	0.38"

Maximum Reactions

Vertical:	21.62 Kips	-10.81 Kips
Horizontal:	5.96 Kips	-15.53 Kips
Longitudinal:	0.00 Kips	-5.07 Kips

```
*******************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which  *
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.            *
* Anchor rods are not supplied by Manufacturer.                                          *
*******************************************************************************
```

Column 1 (SWC) -->



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ndividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Brace Downward forces due to Longitudinal Wind	-0.006 Kips	-0.011 Kips	-0.000 Kips
Brace Downward forces due to Seismic	-0.015 Kips	-0.027 Kips	-0.000 Kips
Brace Upward forces due to Longitudinal Wind	0.006 Kips	0.012 Kips	-0.000 Kips
Brace Upward forces due to Seismic	0.015 Kips	0.029 Kips	-0.000 Kips
Lateral Primary Wind Load 1	-9.170 Kips	-7.054 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-5.187 Kips	-5.251 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-6.871 Kips	-3.631 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-2.888 Kips	-1.829 Kips	-0.000 Kips
Lateral Seismic Load	-0.386 Kips	-0.947 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-11.052 Kips	-5.135 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-9.833 Kips	-5.408 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-6.686 Kips	-2.565 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-6.172 Kips	-2.680 Kips	-0.000 Kips
Roof Collateral Load	3.675 Kips	2.670 Kips	-0.000 Kips
Roof Dead Load	3.093 Kips	1.821 Kips	-0.000 Kips
Roof Live Load	14.700 Kips	10.680 Kips	-0.000 Kips
Column 2 (SWA)			
Brace Downward forces due to Longitudinal Wind	1.615 Kips	0.011 Kips	-0.000 Kips
Brace Downward forces due to Seismic	3.916 Kips	0.027 Kips	-0.000 Kips
Brace Upward forces due to Longitudinal Wind	-1.615 Kips	-0.012 Kips	-2.988 Kips
Brace Upward forces due to Seismic	-3.916 Kips	-0.029 Kips	-7.244 Kips
Lateral Primary Wind Load 1	-6.871 Kips	3.631 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-2.888 Kips	1.829 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-9.170 Kips	7.054 Kips	-0.000 Kips
ral Primary Wind Load 4	-5.187 Kips	5.251 Kips	-0.000 Kips
eral Seismic Load	0.386 Kips	-0.947 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-9.833 Kips	5.408 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-11.052 Kips	5.135 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-6.172 Kips	2.680 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-6.686 Kips	2.565 Kips	-0.000 Kips
Roof Collateral Load	3.675 Kips	-2.670 Kips	-0.000 Kips
Roof Dead Load	3.093 Kips	-1.821 Kips	-0.000 Kips
Roof Live Load	14.700 Kips	-10.680 Kips	-0.000 Kips

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)ESIGN DATA FRAME(S): 8

Inside Clearance:	58'-0 3/4"	**Peak Clearance:**	17'-2 7/8"
		Peak Rafter Depth:	12.13"

Column 1 (SWC)

Column Depth		**Knee**	
Base:	10.88"	**Rafter Depth:**	30.69"
Knee:	10.88"	**Clearance:**	10'-10 1/4"
Anchor Rods		**Base Plate:**	
Quantity:	4	**Length:**	11.00"
Diameter:	0.75"	**Width:**	8.00"
Gauge:	4.00"	**Thickness:**	0.38"
Maximum Reactions			
Vertical:	20.92 Kips	-8.85 Kips	
Horizontal:	14.97 Kips	-5.73 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

Column 2 (SWA)

Column Depth		**Knee**	
`ase:	10.88"	**Rafter Depth:**	30.69"
ʻnee:	10.88"	**Clearance:**	10'-10 1/4"
Anchor Rods		**Base Plate:**	
Quantity:	4	**Length:**	11.00"
Diameter:	0.75"	**Width:**	8.00"
Gauge:	4.00"	**Thickness:**	0.38"
Maximum Reactions			
Vertical:	20.92 Kips	-8.85 Kips	
Horizontal:	5.73 Kips	-14.97 Kips	
Longitudinal:	0.00 Kips	0.00 Kips	

```
**************************************************************************************
* These reactions control the design of the anchor rods.  The load combinations which  *
* these reactions may not be the controlling combinations required for the design of the *
* produced foundation.  It is the responsibility of the foundation engineer to determine *
* the load combinations which are required for the design of the foundation.           *
* Anchor rods are not supplied by Manufacturer.                                        *
**************************************************************************************
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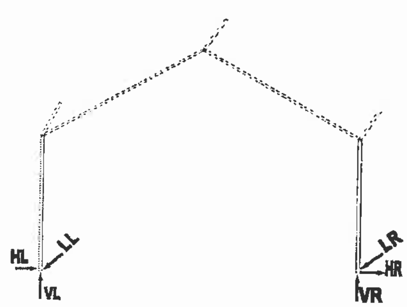
Column 1 (SWC) -->



ndividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 1 (SWC)			
Lateral Primary Wind Load 1	-8.857 Kips	-6.799 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-5.011 Kips	-5.064 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-6.637 Kips	-3.493 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-2.790 Kips	-1.758 Kips	-0.000 Kips
Lateral Seismic Load	-0.376 Kips	-0.922 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-10.676 Kips	-4.944 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-9.499 Kips	-5.207 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-6.458 Kips	-2.468 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-5.962 Kips	-2.579 Kips	-0.000 Kips
Roof Collateral Load	3.550 Kips	2.572 Kips	-0.000 Kips
Roof Dead Load	3.038 Kips	1.775 Kips	-0.000 Kips
Roof Live Load	14.200 Kips	10.287 Kips	-0.000 Kips
Column 2 (SWA)			
Lateral Primary Wind Load 1	-6.637 Kips	3.493 Kips	-0.000 Kips
Lateral Primary Wind Load 2	-2.790 Kips	1.758 Kips	-0.000 Kips
Lateral Primary Wind Load 3	-8.857 Kips	6.799 Kips	-0.000 Kips
Lateral Primary Wind Load 4	-5.011 Kips	5.064 Kips	-0.000 Kips
Lateral Seismic Load	0.376 Kips	-0.922 Kips	-0.000 Kips
Longitudinal Primary Wind Load 1	-9.499 Kips	5.207 Kips	-0.000 Kips
Longitudinal Primary Wind Load 2	-10.676 Kips	4.944 Kips	-0.000 Kips
Longitudinal Primary Wind Load 3	-5.962 Kips	2.579 Kips	-0.000 Kips
Longitudinal Primary Wind Load 4	-6.458 Kips	2.468 Kips	-0.000 Kips
Roof Collateral Load	3.550 Kips	-2.572 Kips	-0.000 Kips
Roof Dead Load	3.038 Kips	-1.775 Kips	-0.000 Kips
f Live Load	14.200 Kips	-10.287 Kips	-0.000 Kips

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ᴉUILDING A - ADMIN BLDG

)ESIGN DATA ENDWALL(s): EWB

Column 1 (Cee)

Anchor Rods:	2	Base Plate Width:	7.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.00"
Column Depth:	8.00"	Base Plate Thickness:	0.25"
Flange Width:	3.50"		

Column 2 (Double Cee)

Anchor Rods:	2	Base Plate Width:	7.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.00"
Column Depth:	8.00"	Base Plate Thickness:	0.25"
Flange Width:	3.50"		

Column 3 (Double Cee)

Anchor Rods:	2	Base Plate Width:	7.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.00"
Column Depth:	8.00"	Base Plate Thickness:	0.25"
Flange Width:	3.50"		

Column 4 (Cee)

Anchor Rods:	2	Base Plate Width:	7.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.00"
Column Depth:	8.00"	Base Plate Thickness:	0.25"
Flange Width:	3.50"		

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ual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 1			
Collateral Load	0.573 Kips	0.000 Kips	0.042 Kips
Dead Load	0.414 Kips	0.000 Kips	0.027 Kips
Live Load	2.293 Kips	0.000 Kips	0.170 Kips
Seismic Force Left	-0.841 Kips	-0.970 Kips	0.000 Kips
Seismic Force Right	0.679 Kips	0.000 Kips	0.000 Kips
Wind Force Left	-3.597 Kips	-1.083 Kips	-0.197 Kips
Wind Force Right	-1.900 Kips	0.000 Kips	-0.197 Kips
Wind Load as Inward Pressure	-2.658 Kips	0.000 Kips	-0.197 Kips
Wind Load as Outward Pressure	-2.658 Kips	0.000 Kips	-0.197 Kips
Column 2			
Collateral Load	1.451 Kips	0.000 Kips	-0.087 Kips
Dead Load	1.048 Kips	0.000 Kips	-0.055 Kips
Live Load	5.806 Kips	0.000 Kips	-0.349 Kips
Seismic Force Left	0.841 Kips	0.000 Kips	0.000 Kips
Seismic Force Right	-0.679 Kips	0.970 Kips	0.000 Kips
Wind Force Left	-4.195 Kips	0.000 Kips	0.308 Kips
Wind Force Right	-5.892 Kips	1.083 Kips	0.308 Kips
Wind Load as Inward Pressure	-5.134 Kips	0.000 Kips	2.397 Kips
Wind Load as Outward Pressure	-5.134 Kips	0.000 Kips	-2.005 Kips
Column 3			
Collateral Load	1.451 Kips	0.000 Kips	-0.087 Kips
Dead Load	1.048 Kips	0.000 Kips	-0.055 Kips
Live Load	5.806 Kips	0.000 Kips	-0.349 Kips
Seismic Force Left	0.000 Kips	0.000 Kips	0.000 Kips
Seismic Force Right	0.000 Kips	0.000 Kips	0.000 Kips
Wind Force Left	-5.134 Kips	0.000 Kips	0.308 Kips
Wind Force Right	-5.134 Kips	0.000 Kips	0.308 Kips
·d Load as Inward Pressure	-5.134 Kips	0.000 Kips	2.397 Kips
.d Load as Outward Pressure	-5.134 Kips	0.000 Kips	-2.005 Kips
Column 4			
Collateral Load	0.573 Kips	0.000 Kips	-0.042 Kips

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Design Report
Optima 1.3.2
38315

Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

ᴉUILDING A - ADMIN BLDG

ıdividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 4			
Dead Load	0.414 Kips	0.000 Kips	-0.027 Kips
Live Load	2.293 Kips	0.000 Kips	-0.170 Kips
Seismic Force Left	0.000 Kips	0.000 Kips	0.000 Kips
Seismic Force Right	0.000 Kips	0.000 Kips	0.000 Kips
Wind Force Left	-2.658 Kips	0.000 Kips	0.197 Kips
Wind Force Right	-2.658 Kips	0.000 Kips	0.197 Kips
Wind Load as Inward Pressure	-2.658 Kips	0.000 Kips	0.197 Kips
Wind Load as Outward Pressure	-2.658 Kips	0.000 Kips	0.197 Kips

#	Rafter Type	Rafter Depth
1	Cee	12.00"
2	Cee	12.00"

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METALLIC BUILDING COMPANY

Design Report
Optima 1.3.2
38315

Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

)ESIGN DATA ENDWALL(s): EWD

Column 1 (Cee)

Anchor Rods:	2	Base Plate Width:	7.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.00"
Column Depth:	8.00"	Base Plate Thickness:	0.25"
Flange Width:	3.50"		

Column 2 (Double Cee)

Anchor Rods:	2	Base Plate Width:	7.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.00"
Column Depth:	8.00"	Base Plate Thickness:	0.25"
Flange Width:	3.50"		

Column 3 (Double Cee)

Anchor Rods:	2	Base Plate Width:	7.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.00"
Column Depth:	8.00"	Base Plate Thickness:	0.25"
Flange Width:	3.50"		

Column 4 (Cee)

Anchor Rods:	2	Base Plate Width:	7.00"
Anchor Rods Diameter:	0.63"	Base Plate Length:	8.00"
Column Depth:	8.00"	Base Plate Thickness:	0.25"
Flange Width:	3.50"		

ual Loads - Unfactored

	Vertical	Horizontal	Longitudinal
Column 1			
Collateral Load	0.545 Kips	0.000 Kips	0.040 Kips
Dead Load	0.399 Kips	0.000 Kips	0.026 Kips
Live Load	2.180 Kips	0.000 Kips	0.162 Kips
Seismic Force Left	0.000 Kips	0.000 Kips	0.000 Kips
Seismic Force Right	0.000 Kips	0.000 Kips	0.000 Kips
Wind Force Left	-2.548 Kips	0.000 Kips	-0.189 Kips
Wind Force Right	-2.548 Kips	0.000 Kips	-0.189 Kips
Wind Load as Inward Pressure	-2.548 Kips	0.000 Kips	-0.189 Kips
Wind Load as Outward Pressure	-2.548 Kips	0.000 Kips	-0.189 Kips
Column 2			
Collateral Load	1.379 Kips	0.000 Kips	-0.083 Kips
Dead Load	1.010 Kips	0.000 Kips	-0.053 Kips
Live Load	5.519 Kips	0.000 Kips	-0.331 Kips
Seismic Force Left	0.000 Kips	0.000 Kips	0.000 Kips
Seismic Force Right	0.000 Kips	0.000 Kips	0.000 Kips
Wind Force Left	-4.912 Kips	0.000 Kips	0.295 Kips
Wind Force Right	-4.912 Kips	0.000 Kips	0.295 Kips
Wind Load as Inward Pressure	-4.912 Kips	0.000 Kips	2.384 Kips
Wind Load as Outward Pressure	-4.912 Kips	0.000 Kips	-2.018 Kips
Column 3			
Collateral Load	1.379 Kips	0.000 Kips	-0.083 Kips
Dead Load	1.010 Kips	0.000 Kips	-0.053 Kips
Live Load	5.519 Kips	0.000 Kips	-0.331 Kips
Seismic Force Left	-0.799 Kips	-0.922 Kips	0.000 Kips
Seismic Force Right	0.646 Kips	0.000 Kips	0.000 Kips
Wind Force Left	-5.816 Kips	-1.044 Kips	0.295 Kips
Wind Force Right	-4.181 Kips	0.000 Kips	0.295 Kips
'd Load as Inward Pressure	-4.912 Kips	0.000 Kips	2.384 Kips
.d Load as Outward Pressure	-4.912 Kips	0.000 Kips	-2.018 Kips
Column 4			
Collateral Load	0.545 Kips	0.000 Kips	-0.040 Kips

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C23

METALLIC BUILDING COMPANY

Design Report
Optima 1.3.2
38315

Architectural Building Systems, LLC
Project ID: PUBLIC WORKS BLDG

ıdividual Loads - Unfactored	Vertical	Horizontal	Longitudinal
Column 4			
Dead Load	0.399 Kips	0.000 Kips	-0.026 Kips
Live Load	2.180 Kips	0.000 Kips	-0.162 Kips
Seismic Force Left	0.799 Kips	0.000 Kips	0.000 Kips
Seismic Force Right	-0.646 Kips	0.922 Kips	0.000 Kips
Wind Force Left	-1.644 Kips	0.000 Kips	0.189 Kips
Wind Force Right	-3.279 Kips	1.044 Kips	0.189 Kips
Wind Load as Inward Pressure	-2.548 Kips	0.000 Kips	0.189 Kips
Wind Load as Outward Pressure	-2.548 Kips	0.000 Kips	0.189 Kips

#	Rafter Type	Rafter Depth
1	Cee	12.00"
2	Cee	12.00"

D C
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C24

)esign Notes

Buyer is responsible for selecting the appropriate thermal blocks and clips for standing seam roofs for use with the insulation used on the project.

Buyer is responsible for determining the correct fastener length for use with the insulation used on the project.
See the Help file or contact the Manufacturer for documents regarding the proper selection of fasteners, clips and thermal blocks.

OPY
PMENT

2

Calculations

DLR Group

Date	10/26/09
Project Name	GILA COUNTY
Project No.	30-09115-00
Subject	FOUNDATIONS
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Checked	
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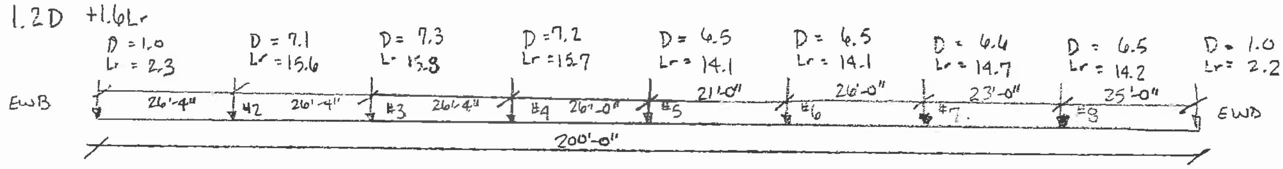
AF
GIL
PEF
DA

WF 1 SWA (COLUMN 2)

DOWNWARD

LOAD COMBO: 1.2D + 1.6Lr



EWB $P_u = 1.2(1) + 1.6(2.3) = 4.9K$

#2 $P_u = 1.2(7.1) + 1.6(15.6) = 33.5K$

#3 $P_u = 1.2(7.3) + 1.6(15.8) = 34.0K$

#4 $P_u = 1.2(7.2) + 1.6(15.7) = 33.8K$

#5 $P_u = 1.2(6.5) + 1.6(14.1) = 30.4K$

#6 $P_u = 1.2(6.5) + 1.6(14.1) = 30.4K$

#7 $P_u = 1.2(6.6) + 1.6(14.7) = 31.4K$

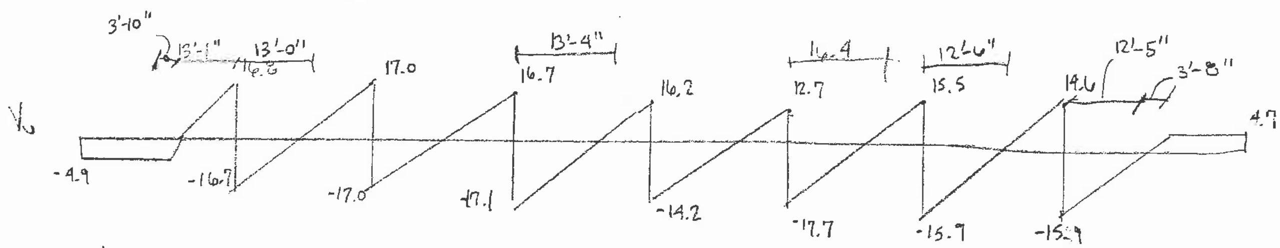
#8 $P_u = 1.2(6.5) + 1.6(14.2) = 30.5K$

EWD $P_u = 1.2(1.0) + 1.6(2.2) = \underline{4.7K}$

$\Sigma P_u = 238.3K$

$$q_u = \frac{(4.9 + 33.5 + 34.0 + 33.8 + 30.4 + 30.4 + 31.4 + 30.5 + 4.7)}{200'\text{-}0"} = 1.17\,KLF\;/\;2' = 584\,psf$$

FOR BALANCED MOMENT / SHEAR SAY $33.8/26.33 = 1.28\,KLF$ TRANSFERRED TO FOUNDATION OVER 186'-2"



$V_u = 17.1K$

$M_u = -55.5\,K\text{-}FT$

$= \pm 111\,K\text{-}FT$

FOOTING DESIGN:

TRY 2x2 $b = 24"$ $d = 20.63"$

$\phi V_n = 39.7K$ $\phi V_c = \phi 2\sqrt{f'c}\,bd = 40.6K$

$\frac{1}{2}V_c > V_u$ ∴ OK W/OUT SHEAR REINF.

$\phi M_n = 153.1\,K\text{-}FT > M_u$

4-#6 REBAR @ TOP & BOTTOM FLEXURE #4 STIRRUP @ 48" OC FOR SUPPORT

Calculations

DLR Group

Date	10/26/09
Project Name	Gila County Public Works
Project No.	30-09115-00
Subject	Foundations
Computed	EDN
Checked	
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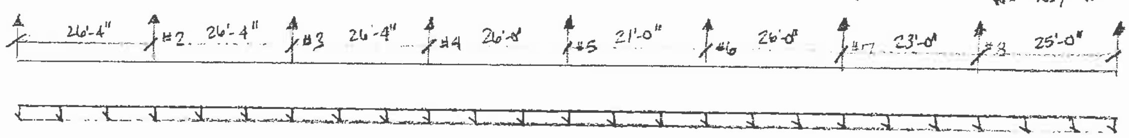
PRO
COUNT
MIT #:
E:

WF 1   SWA (Column ?)

* VERTICAL UPLIFT : LOAD CASE : LONGITUDINAL WIND 2 : + COLLATERAL NOT INCLUDED.

D = 0.4, W = -2.7 | D = 3.3, W = -11.7 | D = 3.3, W = -11.9 | D = 3.3, W = -11.8 | D = 3.0, W = -10.6 | D = 3.0, W = -10.6 | D = 3.1, W = -11.1 | D = 3.0, W = -10.7 | D = 0.4, W = -2.5

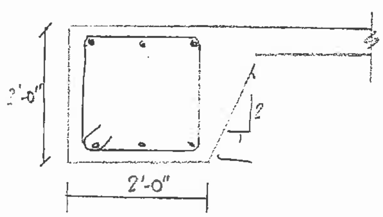
LOAD COMBO:
0.9D + 1.6W   FACTURED
0.6D + W   UNFACTORED



UNFACTORED UPLIFT : 0.6 $D_{REQ'D}$ = $\dfrac{0.6(0.4\times2 + 3.3\times3 + 3.0\times3 + 3.1) - 1.0(2.7 + 11.7 + 11.9 + 11.8 + 10.6 + 10.6 + 11.1 + 10.7 + 2.5)}{200'\text{-}0''}$

$W_{D\,REQ'D}$ = -0.583 KLF

TRY 2' DEEP x 2' WIDE



W = .145(2x2 + 0.5(2x1)) = 0.725 KLF

∴ W > $W_{DREQ'D}$ 2x2 UPLIFT   FS = $\dfrac{.725}{.583}$ = 1.2

SLAB-ON-GRADE NOT INCLUDED

SIZE OK FOR UPLIFT.

FOOTING DESIGN FOR UPLIFT NOT COMPUTED SINCE DOWNWARD FORCE WILL CONTROL DESIGN.

DLR Group

Calculations

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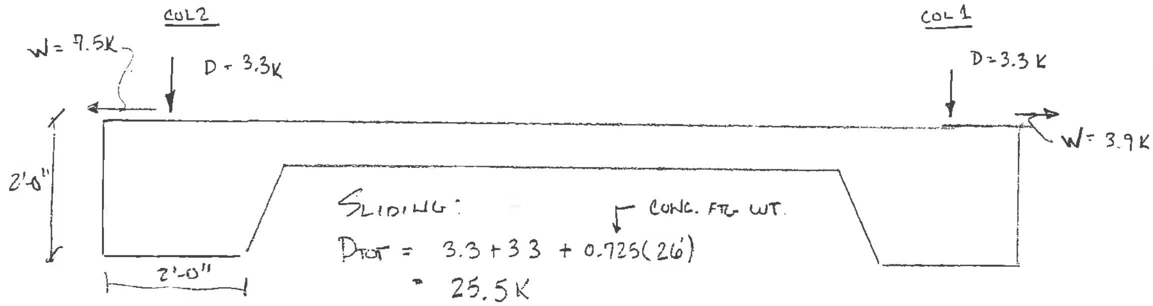
WF 1 HORIZONTAL / SLIDING



SLIDING:
= 7.5 - 3.9 = 3.6K / FRAME

SLIDING:

CONC. FTG WT.

D_{TOT} = 3.3 + 3.3 + 0.725(26')
= 25.5 K

H_{RESIST} = 0.4(25.5) = 10.2 K / FRAME

FS = $\frac{10.2}{3.6}$ = 2.8 > 1.5 SLIDING OK, PASSIVE PRESSURE NOT REQ'D.

OVERTURN: → VERIFIED w/ UPLIFT CALCS

BRG PRESS: → UNIFORM LOAD UNDER FOUNDATION, CHECKED w/ FOOTING DESIGN.

DLR Group

Calculations

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BY: Sol

WF 2 EWB

UPWARD:

Column loads 1 & 4 go to WF1, incorporated into that design

LOAD COMBO: $0.6D + W$

D = 1.0
W = -5.8

D = 1.0
W = -5.8

$$0.6\,D_{REQ'D} = \frac{0.6(1.0+1.0) - (5.8+5.8)}{60'} \approx -0.233 \text{ KLF}$$

ENTIRE LENGTH

$$0.6\,D_{REQ'D} = \frac{0.6(1.0+1.0) - (5.8+5.8)}{40'} \approx -0.433 \text{ KLF}$$

USE 2x2 FTG

$W = 0.725$ KLF $\therefore$ $W > W_{REQ'D}$ $FS = \frac{0.725}{0.433} = 1.67$

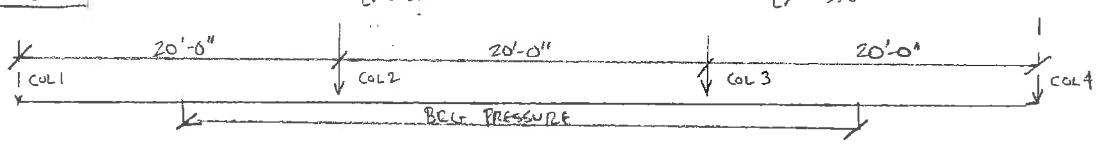
SLAB-ON-GRADE NOT INCLUDED, SIZE OK FOR UPLIFT

DOWNWARD:

$1.2D + 1.6L_r$

D = 2.4
Lr = 5.8

D = 2.4
Lr = 5.8



COL 2

$P_u = 1.2(2.4) + 1.6(5.8) = 12.2K$

$P_u = 1.2(2.4) + 1.6(5.8) = 12.2K$

$q_u = 12.2/20' = 0.61$ KLF

$$L_{REQ'D} = \frac{(12.2 + 12.2)}{0.61} \approx 40'\text{-}0"$$

$V_u = 0.61(10) = 6.1K$

$M_u = 30.5$ K-FT

2x2 FTG

$\phi V_N = 39.7K$ $\tfrac{1}{2}\phi V_N > V_u$ NO SHEAR REINF REQ'D

$\phi M_N = 153.1$ K-FT $> M_u$

4-#6 REBAR @ TOP & BOTTOM FLEXURAL
#4 STIRRUP @ 48" OC FOR SUPPORT.

Calculations

DLR Group

Date
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)PY
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TT1

COMPRESSION:

D w/ COLLATERAL : 4.6
L_r = 11.0

$P_u = 1.2(4.6)(2) + 1.6(11.0 \cdot 2) = 46.2K$

$\phi P_{BRG} = \phi 0.85 f'_c A_1$

$= 0.65(0.85)(3000)(12\times12) = 238.7K > P_u$

$\phi P_{AXIAL} = \phi 0.80[0.85f'_c(A_G - A_{ST}) + F_y A_{ST}]$

$(0.80)(0.80)[0.85(3)(144-0.88) + 60(.88)] = 267.4K > P_u$

2-#6 OK

FOR ELEMENT TO BE COLUMN $A_{ST} = .01 A_G$ MIN $= 1.44 in^2$

4-#6 BARS w/ #3 TIES @ 12" OC. REQ'D.

TENSION:

W =	-7.6	-5.8
W =	3.9	5.5
	11.5 K	11.3

CONTROLS

$T_u = 1.6(11.5) = 18.4K$

$\phi T = \phi A_s f_y = 0.9(.88)(60) = 47.5K$

2-#6 BARS OK

4-#6 BARS USED FOR COMPRESSION

$\boxed{\text{\#4-\#6 BARS w/ \#3 TIES @ 12"OC} \approx \text{\#2 TIES OK ALSO}}$

Project Gila Country Public Administration
Subjec ANCHOR DESIGN ACI 318-05 APPENDIX D
Date: 10/30/2009

Project Number: 30-09115-00
Computed by: EDN
Page:

Vertical (Tension)

Gridline	Group Type	D	Dc	L	Lr/S	W1	W2	W3	W4	W5	W6	W7	W8	E
I	Frame2, Col2	3.3	3.9	0	15.6	-7.3	-3.1	-9.7	-5.5	-10.4	-11.7	-6.5	-7.1	0.4
B4	Frame2, Col1	3.3	3.9	0	15.6	-9.7	-5.5	-7.3	-3.1	-11.7	-10.4	-7.1	-6.5	-0.4
C1	Frame3, Col2	3.3	4	0	15.8	-7.3	-3.1	-9.9	-5.6	-10.6	-11.9	-6.6	-7.2	0.4
C4	Frame3, Col1	3.3	4	0	15.8	-9.9	-5.6	-7.4	-3.1	-11.9	-10.6	-7.2	-6.6	-0.4
D1	Frame4, Col2	3.3	3.9	0	15.7	-7.4	-3.1	-9.8	-5.5	-10.5	-11.8	-6.6	-7.1	0.4
D4	Frame4, Col1	3.3	3.9	0	15.7	-9.8	-5.5	-7.3	-3.1	-11.8	-10.5	-7.1	-6.6	-0.4
E1	Frame5, Col2	3	3.5	0	14.1	-6.6	-2.8	-8.8	-5	-9.4	-10.6	-5.9	-6.4	0.4
E4	Frame5, Col1	3	3.5	0	14.1	-8.8	-5	-6.6	-2.8	-12.6	-11.4	-8.4	-7.9	-5.2
F1	Frame6, Col2	3	3.5	0	14.1	-6.6	-2.8	-8.8	-5	-9.4	-10.6	-5.9	-6.4	-3.5
F4	Frame6, Col1	3	3.5	0	14.1	-8.8	-5	-6.6	-2.8	-10.6	-9.4	-6.4	-5.9	-5.2
G1	Frame7, Col2	3.1	3.7	0	14.7	-6.9	-2.9	-9.2	-5.2	-11.4	-12.7	-7.8	-8.3	-1.2
G4	Frame7, Col1	3.1	3.7	0	14.7	-9.2	-5.2	-6.9	-2.9	-11.1	-9.8	-6.7	-6.2	-0.4
H1	Frame8, Col2	3	3.6	0	14.2	-8.9	-5	-6.6	-2.8	-10.7	-9.5	-6.5	-6	-0.4
H4	Frame8, Col1	3	3.6	0	14.2	-8.9	-5	-6.6	-2.8	-10.7	-9.5	-6.5	-6	-0.4
A1	EWB, Col1	0.4	0.6	0	2.3	-3.6	-3.6	-1.9	-1.9	-2.7	-2.7	-2.7	-2.7	-0.8
A2	EWB, Col2	1	1.5	0	5.8	-4.2	-4.2	-5.9	-5.9	-5.1	-5.1	-5.1	-5.1	-0.7
A3	EWB, Col3	1	1.5	0	5.8	-5.1	-5.1	-5.1	-5.1	-5.1	-5.1	-5.1	-5.1	0
A4	EWB, Col4	0.4	0.6	0	2.3	-2.7	-2.7	-2.7	-2.7	-2.7	-2.7	-2.7	-2.7	0
J4	EWD, Col1	0.4	0.5	0	2.2	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5	-2.5	0
J3	EWD, Col2	1	1.4	0	5.5	-4.9	-4.9	-4.9	-4.9	-4.9	-4.9	-4.9	-4.9	0
J2	EWD, Col3	1	1.4	0	5.5	-5.8	-5.8	-4.2	-4.2	-4.9	-4.9	-4.9	-4.9	-0.8
J1	EWD, Col4	0.4	0.5	0	2.2	-1.6	-1.6	-3.3	-3.3	-2.5	-2.5	-2.5	-2.5	-0.6

Horizontal (Shear +/- y) to Footing

Gridline	D	Dc	L	Lr/S	W1	W2	W3	W4	W5	W6	W7	W8	E
I	-1.9	-2.8	0	-11	3.7	1.8	7.3	5.5	5.6	5.3	2.7	2.6	-1
B4	-1.9	-2.8	0	-11	7.3	5.5	3.7	1.8	5.3	5.6	2.6	2.8	1
C1	-2	-2.9	0	-11.5	3.9	2	7.6	5.6	5.8	5.5	2.9	2.8	-1
C4	-2	-2.9	0	-11.5	7.6	5.6	3.9	2	5.5	5.8	2.8	2.9	1
D1	-1.9	-2.9	0	-11.4	3.9	2	7.5	5.6	5.8	5.5	2.9	2.7	-1
D4	-1.9	-2.9	0	-11.4	7.5	5.6	3.9	2	5.5	5.8	2.7	2.9	1
E1	-1.8	-2.5	0	-10.2	3.5	1.7	6.7	5	5.2	4.9	2.6	2.4	-0.9
E4	-1.8	-2.5	0	-10.2	6.7	5	3.5	1.7	4.9	5.2	2.4	2.6	0.9
F1	-1.8	-2.5	0	-10.2	3.5	1.7	6.7	5	5.2	4.9	2.6	2.4	-0.9
F4	-1.8	-2.5	0	-10.2	6.7	5	3.5	1.7	4.9	5.2	2.4	2.6	0.9
G1	-1.8	-2.7	0	-10.7	3.6	1.8	7	5.3	5.4	5.1	2.7	2.6	-0.9
G4	-1.8	-2.7	0	-10.7	7.1	5.3	3.6	1.8	5.1	5.4	2.6	2.7	0.9
H1	-1.8	-2.6	0	-10.3	3.5	1.8	6.8	5.1	5.2	4.9	2.6	2.5	-0.9
H4	-1.8	-2.6	0	-10.3	6.8	5.1	3.5	1.8	4.9	5.2	2.5	2.6	0.9
A1	0	0	0	0	1	1	0	0	0	0	0	0	-0.9
A2	0	0	0	0	0	0	1	1	0	0	0	0	1
A3	0	0	0	0	0	0	0	0	0	0	0	0	0
A4	0	0	0	0	0	0	0	0	0	0	0	0	0
J4	0	0	0	0	0	0	0	0	0	0	0	0	0
J3	0	0	0	0	0	0	0	0	0	0	0	0	0
J2	0	0	0	0	1	1	0	0	0	0	0	0	-0.9
J1	0	0	0	0	0	0	1	1	0	0	0	0	0.9

Longitudnal (Shear +/- x) to Footing

Gridline	D	Dc	L	Lr/S	W1	W2	W3	W4	W5	W6	W7	W8	E
I	0	0	0	0	0	0	0	0	0	0	0	0	0
B4	0	0	0	0	0	0	0	0	0	0	0	0	0
C1	0	0	0	0	0	0	0	0	0	0	0	0	0
C4	0	0	0	0	0	0	0	0	0	0	0	0	0
D1	0	0	0	0	0	0	0	0	0	0	0	0	0
D4	0	0	0	0	0	0	0	0	0	0	0	0	0
E1	0	0	0	0	0	0	0	0	0	0	0	0	0
E4	0	0	0	0	0	0	0	0	3	3	3	3	7.2
F1	0	0	0	0	0	0	0	0	-3	-3	-3	-3	-7.2
F4	0	0	0	0	0	0	0	0	3	3	3	3	7.2
G1	0	0	0	0	0	0	0	0	-3	-3	-3	-3	-7.2
G4	0	0	0	0	0	0	0	0	0	0	0	0	0
H1	0	0	0	0	0	0	0	0	0	0	0	0	0
H4	0	0	0	0	0	0	0	0	0	0	0	0	0
A1	0	0	0	-0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0
A2	0	0	0	-0.3	0.3	0.3	0.3	0.3	2.4	2.4	2	2	0
A3	0	0	0	-0.3	0.3	0.3	0.3	0.3	2.4	2.4	2	2	0
A4	0	0	0	-0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0
J4	0	0	0	-0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0
J3	0	0	0	-0.3	0.3	0.3	0.3	0.3	2.4	2.4	2	2	0
J2	0	0	0	-0.3	0.3	0.3	0.3	0.3	2.4	2.4	2	2	0
J1	0	0	0	-0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0

Anchor Bolt Combo's

Gridline	Group Type	Vertical Load Combo: Nu max 1	Nu max 2	Nu max 3	Nu min 4	Nu min 5	Horiz Load Combo: -Vuy 1	-Vuy 2	-Vuy 3	+Vuy 4	+Vuy 5	Long Load Combo (Vux): -Vux 1	-Vux 2	-Vux 3	+Vux 4	+Vux 5
IBC 2006 Load Cases		1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
B1	Frame2, Col2	16.4	33.6	11.5	-15.8	3.4	-11.1	-23.2	-8.3	10.0	-2.7	0.0	0.0	0.0	0.0	0.0
i	Frame2, Col1	16.4	33.6	11.5	-15.8	2.6	-11.1	-23.2	-8.3	10.0	-0.7	0.0	0.0	0.0	0.0	0.0
C1	Frame3, Col2	16.7	34.0	11.7	-16.1	3.4	-11.6	-24.3	-8.4	10.4	-2.8	0.0	0.0	0.0	0.0	0.0
C4	Frame3, Col1	16.7	34.0	11.7	-16.1	2.6	-11.6	-24.3	-8.4	10.4	-0.8	0.0	0.0	0.0	0.0	0.0
D1	Frame4, Col2	16.5	33.8	11.5	-15.9	3.4	-11.5	-24.0	-8.3	10.3	-2.7	0.0	0.0	0.0	0.0	0.0
D4	Frame4, Col1	16.5	33.8	11.5	-15.9	2.6	-11.5	-24.0	-8.3	10.3	-0.7	0.0	0.0	0.0	0.0	0.0
E1	Frame5, Col2	14.9	30.4	10.4	-14.3	3.1	-10.3	-21.5	-7.5	9.1	-2.5	0.0	0.0	0.0	0.0	0.0
E4	Frame5, Col1	14.9	30.4	10.4	-17.5	-2.5	-10.3	-21.5	-7.5	9.1	-0.7	0.0	0.0	0.0	4.8	7.2
F1	Frame6, Col2	14.9	30.4	10.4	-14.3	-0.8	-10.3	-21.5	-7.5	9.1	-2.5	0.0	-2.4	-4.8	0.0	-7.2
F4	Frame6, Col1	14.9	30.4	10.4	-14.3	-2.5	-10.3	-21.5	-7.5	9.1	-0.7	0.0	0.0	0.0	4.8	7.2
G1	Frame7, Col2	15.5	31.7	10.9	-17.5	1.6	-10.8	-22.5	-7.9	9.6	-2.5	0.0	-2.4	-4.8	0.0	-7.2
G4	Frame7, Col1	15.5	31.7	10.9	-15.0	2.4	-10.8	-22.5	-7.9	9.7	-0.7	0.0	0.0	0.0	0.0	0.0
H1	Frame8, Col2	15.0	30.6	10.5	-14.4	2.3	-10.4	-21.8	-7.6	9.3	-2.5	0.0	0.0	0.0	0.0	0.0
H4	Frame8, Col1	15.0	30.6	10.5	-14.4	2.3	-10.4	-21.8	-7.6	9.3	-0.7	0.0	0.0	0.0	0.0	0.0
A1	EWB, Col1	2.4	4.9	-0.7	-5.4	-0.4	0.0	0.0	0.0	1.6	-0.9	-0.1	-0.3	0.2	0.3	0.0
A2	EWB, Col2	5.9	12.3	-0.8	-8.5	0.2	0.0	0.0	0.0	1.6	1.0	-0.2	-0.5	0.3	3.8	0.0
A3	EWB, Col3	5.9	12.3	-2.3	-7.3	0.9	0.0	0.0	0.0	0.0	0.0	-0.2	-0.5	0.3	3.8	0.0
A4	EWB, Col4	2.4	4.9	-2.0	-4.0	0.4	0.0	0.0	0.0	0.0	0.0	-0.1	-0.3	0.2	0.3	0.0
J4	EWD, Col1	2.2	4.6	-1.8	-3.6	0.4	0.0	0.0	0.0	0.0	0.0	-0.1	-0.3	0.2	0.3	0.0
J3	EWD, Col2	5.6	11.7	-2.2	-6.9	0.9	0.0	0.0	0.0	0.0	0.0	-0.2	-0.5	0.3	3.8	0.0
J2	EWD, Col3	5.6	11.7	-1.1	-8.4	0.1	0.0	0.0	0.0	1.6	-0.9	-0.2	-0.5	0.3	3.8	0.0
J1	EWD, Col4	2.2	4.6	-0.4	-4.9	-0.2	0.0	0.0	0.0	1.6	0.9	-0.1	-0.3	0.2	0.3	0.0

Gridline	Tension øNmin	Shear y -øVy	+øVy	Shear x -øVx	+øVx	Load Combination (Tension + Horiz Shear or Longitudnal Shear): A	B	C	D	E	F	G	H	I	Shear	Tension	Combo
B1	-27.3	-30.2	5.3	-15.8	15.8	0.77	1.88	0.00	0.00	0.58	1.35	2.46	0.58	0.58			
i	-27.3	-30.2	5.3	-15.8	15.8	0.77	1.88	0.00	0.00	0.58	1.35	2.46	0.58	0.58			
C1	-27.3	-30.2	5.3	-15.8	15.8	0.80	1.95	0.00	0.00	0.59	1.39	2.54	0.59	0.59			
C4	-27.3	-30.2	5.3	-15.8	15.8	0.80	1.95	0.00	0.00	0.59	1.39	2.54	0.59	0.59			
D1	-27.3	-30.2	5.3	-15.8	15.8	0.79	1.94	0.00	0.00	0.58	1.38	2.52	0.58	0.58			
D4	-27.3	-30.2	5.3	-15.8	15.8	0.79	1.94	0.00	0.00	0.58	1.38	2.52	0.58	0.58			
E1	-27.3	-30.2	5.3	-15.8	15.8	0.71	1.72	0.00	0.00	0.52	1.23	2.24	0.52	0.52			
E4	-27.3	-30.2	5.3	-15.8	15.8	0.71	1.72	0.00	0.46	0.64	1.35	2.36	0.64	1.10			
F1	-27.3	-30.2	5.3	-15.8	15.8	0.71	1.72	0.30	0.00	0.52	1.23	2.24	0.83	0.52			
F4	-27.3	-30.2	5.3	-15.8	15.8	0.71	1.72	0.00	0.46	0.52	1.23	2.24	0.52	0.98			
G1	-27.3	-30.2	5.3	-15.8	15.8	0.75	1.81	0.30	0.00	0.64	1.39	2.45	0.95	0.64			
G4	-27.3	-30.2	5.3	-15.8	15.8	0.75	1.84	0.00	0.00	0.55	1.29	2.39	0.55	0.55			
H1	-27.3	-30.2	5.3	-15.8	15.8	0.72	1.75	0.00	0.00	0.53	1.25	2.28	0.53	0.53			
H4	-27.3	-30.2	5.3	-15.8	15.8	0.72	1.75	0.00	0.00	0.53	1.25	2.28	0.53	0.53			
A1	-11.6	-10.2	3.8	-10.2	2.7	0.00	0.42	0.03	0.12	0.47	0.47	0.89	0.50	0.58			
A2	-16.2	-10.2	4.8	-10.2	10.2	0.00	0.33	0.05	0.38	0.53	0.53	0.86	0.57	0.90			
A3	-16.2	-10.2	4.8	-10.2	10.2	0.00	0.00	0.05	0.38	0.45	0.45	0.45	0.50	0.82			
A4	-11.6	-10.2	3.8	-10.2	2.7	0.00	0.00	0.03	0.12	0.34	0.34	0.34	0.37	0.46			
J4	-11.6	-10.2	3.8	-10.2	2.7	0.00	0.00	0.03	0.12	0.31	0.31	0.31	0.35	0.43			
J3	-16.2	-10.2	4.8	-10.2	10.2	0.00	0.00	0.05	0.38	0.43	0.43	0.43	0.48	0.80			
J2	-16.2	-10.2	4.8	-10.2	10.2	0.00	0.33	0.05	0.38	0.52	0.52	0.85	0.56	0.89			
J1	-11.6	-10.2	3.8	-10.2	2.7	0.00	0.42	0.03	0.12	0.42	0.42	0.85	0.46	0.54			

APPRO
GILA COUNT
PERMIT #:
DATE:

Given: Indicates X-bracing loads

Vertical - PEMB Manufacturer
+ is downward load (compression)
- is upward load (tension)

Column 2 - PEMB Manufacturer
+ is toward the concrete edge
- is away from the concrete edge

*Column 1 - PEMB Manufacturer
+ is away from concrete edge
- is toward the concrete edge

*End Wall - PEMB Manufacturer
+ is away from concrete edge
- is toward the concrete edge

Anchor Bolt design
+ is toward the concrete edge (exterior)
- is away from the concrete edge (interior)

B	+Vuy / +øVy < 1.0
C	-Vux / -øVx < 1.0
D	+Vux / +øVx < 1.0
E	Nu min / øNmin < 1.0
F	Nu min / øNmin + -Vuy / -øVy < 1.2
G	Nu min / øNmin + Vuy / +øVy < 1.2
H	Nu min / øNmin + -Vux / -øVx < 1.2
I	Nu min / øNmin + +Vux / +øVx < 1.2

*Based on this all the Column 1 signs for shear are switched from the PEMB given loadsto align with anchor bolt design sheets.

*Based on this all the End Wall column signs for shear are switched from PEMB given loads to align with anchor bolt design sheets.

Also the End Wall horizontal and longitudnal loads are switched to align with anchor bolt design direction. for the footing design direction

DLR Group

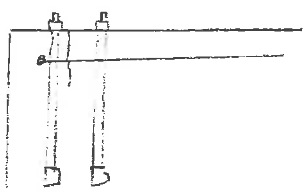
Calculations

Date
Project Name
Project No.
Subject
Computed
Checked
Page (of pages)

ANCHOR DESIGN

FRAME COL.

+V_{uy} = 10.4K



$\phi V_y = 5.3K$

HAIRPIN: #5 REBAR $A_{vf} = 0.31 \times 2 = 0.62 \text{ in}^2$

SHEAR FRICTION: $\phi V_N = \phi A_{vf} f_y \mu$
$= 0.75(0.62)(60)(1.4)$
$= 39.1K$

$\boxed{\text{\#5 HAIRPIN OK FOR ALL LOADING}}$

ALTHOUGH ANCHORS IN TENSION ARE OK PLACE STIRRUP BARS

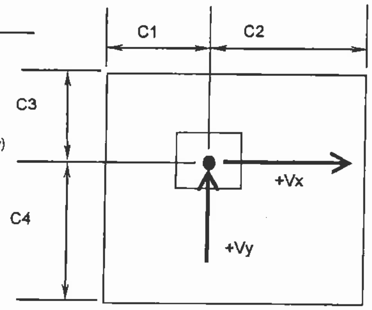
F3

Project:	Gila Country Public Administration	Project Number:	30-09115-00
Subject:	ANCHOR DESIGN ACI 318-05 APPENDIX D	Computed by:	EDN
Date:	########	Page:	

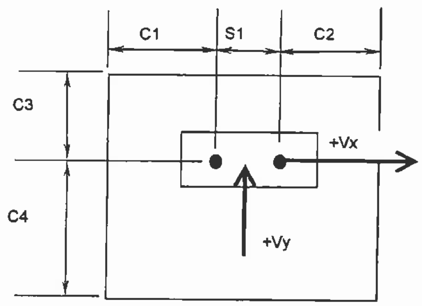
INPUT: Description: Frame Column Bolt Group

No. of Anchors = 4 C1 = 100 IN. S1 = 4 IN.
C2 = 100 IN. S2 = 4 IN.
Anchor Diameter = 0.750 IN. C3 = 4 IN.
C4 = 100 IN.
ANCHOR STEEL DESIGNATION = A307 (A307, A325, A490, or leave blank for ACI design only)
fya = 36,000 YIELD STRENGTH (PSI) PER ASTM F 1554
futa = 58,000 TENSILE STRENGTH (PSI) PER ASTM F 1554
Anchor Embed. = 14 IN. (hef)

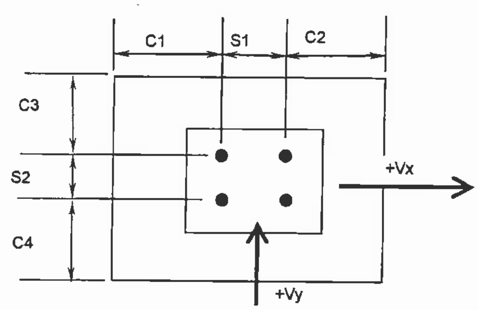
Anchor Type = CI (CI = Cast-In place , PI = Post Installed)
Ductile? DUCTILE DUCTILE or BRITTLE?
Built-Up Grout Pad = NO (YES or NO)
Anchor Head: H (H = headed anchor, J = J or L bolt)
Concrete : NW (NW = Normal weight, LW = All Light weight Concrete, SLW = Sand Light weight Concrete)
f'c = 3,000 PSI (28 Day Compressive Strength)
UNCRACKEI CRACKED or UNCRACKED Concrete
h = 24 IN, (Thickness of Concrete Member)
Condition: B A or B, ("A" applies when supplementary reinf. Is provided, per D.4.4)
Condition for P.I. Anchors = ______ (1, 2, or 3)



ONE ANCHOR



TWO ANCHORS



FOUR ANCHORS

TENSION:

Steel Strength: Nsa = n Ase futa Φns = 0.75

:I 318 0.0 Ase = 0.334 IN² futa = 1.9 fya or 125,000 PSI = 58,000 PSI
ΦNsa = 58.1 KIPS
AISC 7-14 ΦNs 59.7 KIPS ΦFt = 33.8 KSI, AISC TABLE 7-14

ΦNsa = 58.1 Kips/GROUP ΦNsa = 14.5 Kips/BOLT

Concrete Break-out Strength : Φcb = 0.70 NW?= 1

ANco = 1764.00 IN² (D-6) Anc = 1334.0 IN.(

Nb = 71,267.9 (LB) ACI D-7 & D-8

If three or four edge distances are less than 1.5 hef, (Ca,max<= 1.5 hef) then embedment is limited to the greater of Cmax/1.5, (hef = Ca,max/1.5) or 1/3 of the max spacing between anchors For use in eq. D-4 thru D-11
hef = 14.000 IN.

Ψec,N = 1.00 NO ECCENTRICITY e'nx = 0
e'ny = 0

Ψed,N = 0.76 Modification for edge effects

Ψc,N = 1.25 Where anchor is located in a region of a concrete member where analysis indicated no cracking (ft < fr) at service load levels, the modification is as follows:
Ψc,N = 1.25 for cast-in anchors
Ψc,N = 1.4 for Post-installed anchors
Ψc,N = 1.0 Cracking, post-installed, or cast-in anchors

ΦNcb = 35.71 KIPS (D-4, D-5)

Pullout Strength in Tension: Φpn = 0.70

Abrg = 0.911 eh= 4.5do for J or L bolts

Np = 21,864.00 LB, D-15, D-16

Ψc,p = 1.4 Where anchor is located in a region of a concrete member where analysis indicated no cracking (ft < fr) at service load levels, the modification is as follows:
Ψ4 = 1.4 for cast-in anchors
Ψ4 = 1.0 Cracking, cast-in anchors

ΦNpn = 21.43 KIPS, Single Anchor (D-14)

ΦNpng = 85.71 KIPS, Group of Anchors (D-14)

Concrete side-face blowout strength of a headed anchor in tension:

Φnc = 0.7 Not considered when Ca,min > 0.4 hef

Nsb= 39,034.3

C = 4 IN.
C2 = 100 IN.
C2/C = 3

C is equal to the minimum edge distance and C2 is equal to the minimum edge distance perpendicular to C.
If C2 < 3 C then multiply by an additional factor per D.5.4.2.

Φnc = 27.32 KIPS

SHEAR:

Steel Strength of Anchor in Shear: D6.1 $\Phi vsa =$ 0.65

CI318 0.0, Ase = 0.334 IN^2 futa = 1.9 fya or 125,000 PSI = 58,000.0 PSI

Post-Installed Anchor Sleeves:
Asl = ______ IN2 futsl = ______ PSI

$\Phi Vsa =$ 30.2 KIPS, D-18 or D-19
AISC 7-10 $\Phi Vsa =$ 31.79 KIPS $\Phi Fs =$ 18 KSI, AISC TABLE 7-10
Assume threads included in shear plane & single shear application

$\Phi Vsa =$ 30.2 Kips/GROUP	$\Phi Vsa =$ 7.6 Kips/BOLT

Concrete Breakout Strength of Anchor in Shear, per D.6.2: $\Phi vs =$ 0.7

Avco+y =	72.0 IN^2	Avco+x =	1152.0 IN^2
Avco-y =	1152.0 IN^2	Avco-x =	1152.0 IN^2
Avc+y =	96.0 IN^2	Avc+x =	768.0 IN^2
Avc-y =	1248.0 IN^2	Avc-x =	768.0 IN^2
vb+y =	4026.2 LB	vb+x =	32209.7 LB
vb-y =	32209.7 LB	vb-x =	32209.7 LB
$\Psi ec,v$ +y =	1.0	$\Psi ec,v$ +x =	1.0
$\Psi ec,v$ -y =	1.0	$\Psi ec,v$ -x =	1.0
Ψed +y =	1.0	Ψed +x =	0.8
Ψed -y =	1.0	Ψed -x =	0.8

$\Psi c,v =$ 1.4

Are the Anchors welded to the plate? NO
If yes, then anchor capacity is determined based on the row farthest from the edge and, the center to center spacing of the anchor is not less than 2.5 in.; and supplementary reinforcement is provided at the corners if C2 <= 1.5 hef

If three or more edge distances are less than 1.5 C1, (C2-4 or h<= 1.5 C1) then edge distance C1 is limited to C1 = h/1.5,
For use in eq. D-22 thru D-26

C'1+y = 4.0 IN C'1+x = 16.0
C'1-Y = 16.0 IN C'1-x = 16.0

e'vx = 0 IN., Eccentricity of shear force on a group of anchors, X axis
e'vy = 0 IN., Eccentricity of shear force on a group of anchors, Y axis

$\Psi c,v$: D6.2.7, For anchors located in a region of concrete where analysis indicates no cracking (ft < fr) at service loads, then $\Psi e,v$ = 1.4

$\Psi e,v$ = 1.0, Anchors in cracked concrete with no supplementary reinf.
$\Psi e,v$ = 1.2, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & the edge.
$\Psi e,v$ = 1.4, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & edge & #4 stirrups.

$\Phi Vcb(g)+y$ =	5.26 KIPS	$\Phi Vcb(g)+x$ =	15.78 KIPS
$\Phi Vcb(g)-y$ =	34.20 KIPS	$\Phi Vcb(g)-x$ =	15.78 KIPS

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR: D.6.3 : $\Phi cp =$ 0.7

Ncb = 51.0 KIPS From D-4 & Above

$\Phi cpVcp =$ 71.4 KIPS	D-28

RESULTS:

TENSION KIPS	
$\Phi Nsa =$	58.1
$\Phi Ncb =$	35.7
$\Phi Npng =$	85.7
$\Phi nc =$	27.3
$\Phi Nna =$	27.3

SHEAR KIPS			
$\Phi Vsa =$	30.2		
$\Phi Vcb(g)+y$ =	5.26	$\Phi Vcb(g)+x$ =	15.78
$\Phi Vcb(g)-y$ =	34.20	$\Phi Vcb(g)-x$ =	15.78
$\Phi cpVcp =$	71.4		
$\Phi Vn+y=$	5.26	$\Phi Vn+x=$	15.78
$\Phi Vn-y=$	30.22	$\Phi Vn-x=$	15.78
HORIZONTAL		LONGITUDINAL	

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MENT

SPACING & EDGE DISTANCES:

SPACING:

Minimum center to center spacing of untorqued cast-in-place anchors = 4do = 3.0 IN.

Minimum o.c. spacing of torqued cast-in-place & post-installed anchors = 6do = 4.5 IN.

EDGE DISTANCE:

Minimum edge distance for cast-in headed anchors, un-torqued shall be based on minimum cover requirements for reinforcement of ACI 7.7:

Concrete cast against and permanently exposed to soil: 3 IN.

Concrete exposed to earth or weather:
#6-#18 bars 2 IN.
#5 bar & smaller: 1 1/2 IN.

Concrete not exposed to weather or in contact with ground:
#14 & #18 bars: 1 1/2 IN.
#11 bars & smaller: 3/4 IN.

Minimum edge distance for post-installed anchors shall be based on the greater of the minimum cover requirements for reinforcement in ACI 7.7, or the minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2 and shall not be less than 2.0 times the maximum aggregate size. In the absence of product-specific ACI 355.2 test information, the minimum edge distance shall be taken as not less than:

Undercut anchors, 6do = 4.5 IN.

Torqued-controlled anchors, 8do = 6 IN.

Displacement - controlled anchors, 10do = 7.5 IN.

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F6

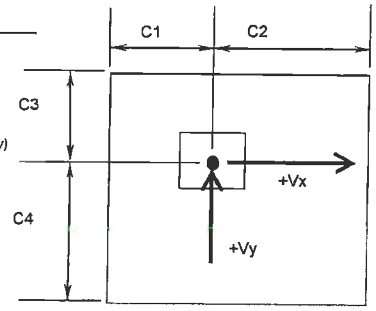
Project: Gila Country Public Administration
Subject: ANCHOR DESIGN ACI 318-05 APPENDIX D
Date: ########

Project Number: 30-09115-00
Computed by: EDN
Page:

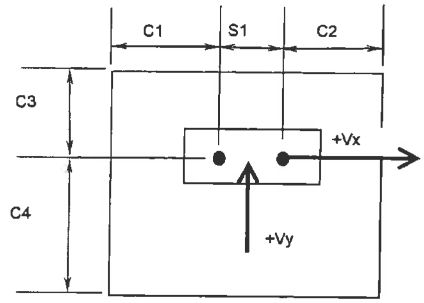
INPUT: Description: End Wall Column Bolt Group

No. of Anchors = 2
C1 = 100 IN.
C2 = 100 IN.
C3 = 4 IN.
C4 = 100 IN.
S1 = 4 IN.
S2 = 0 IN.
Anchor Diameter = 0.625 IN.
ANCHOR STEEL DESIGNATION = A307 (A307, A325, A490, or leave blank for ACI design only)
fya = 36,000 YIELD STRENGTH (PSI) PER ASTM F 1554
futa = 58,000 TENSILE STRENGTH (PSI) PER ASTM F 1554
Anchor Embed. = 8 IN. (hef)

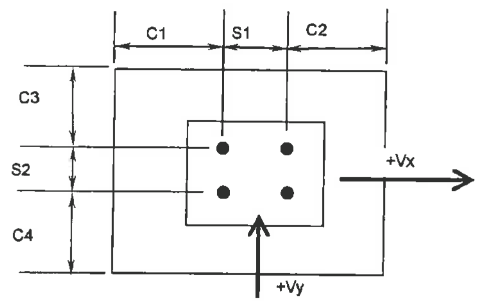
Anchor Type = CI (CI = Cast-In place , PI = Post Installed)
Ductile? DUCTILE DUCTILE or BRITTLE?
Built-Up Grout Pad = NO (YES or NO)
Anchor Head: H (H = headed anchor, J = J or L bolt)
Concrete : NW (NW = Normal weight, LW = All Light weight Concrete, SLW = Sand Light weight Concrete)
f'c = 3,000 PSI (28 Day Compressive Strength)
UNCRACKEI CRACKED or UNCRACKED Concrete
h = 24 IN, (Thickness of Concrete Member)
Condition: B A or B, ("A" applies when supplementary reinf. Is provided, per D.4.4)
Condition for P.I. Anchors = (1, 2, or 3)



ONE ANCHOR



TWO ANCHORS



FOUR ANCHORS

TENSION:

Steel Strength: Nsa = n Ase futa   Φns = 0.75

] 318 0.0 Ase = 0.226 IN^2 futa = 1.9 fya or 125,000 PSI = 58,000 PSI
ΦNsa = 19.7 KIPS
AISC 7-14 ΦNs 20.7 KIPS   ΦFt = 33.8 KSI, AISC TABLE 7-14

ΦNsa =	19.7	Kips/GROUP	ΦNsa =	9.8 Kips/BOLT

Concrete Break-out Strength :   Φcb = 0.70   NW?= 1

ANco = 576.00 IN^2 (D-6)   Anc = 448.0 IN.(

Nb = 29,744.5 (LB) ACI D-7 & D-8

If three or four edge distances are less than 1.5 hef, (Ca,max<= 1.5 hef) then embedment is limited to the greater of Cmax/1.5, (hef = Ca,max/1.5) or 1/3 of the max spacing between anchors For use in eq. D-4 thru D-11
hef = 8.000 IN.

Ψec,N = 1.00 NO ECCENTRICITY   e'nx = 0
e'ny = 0

Ψed,N = 0.80 Modification for edge effects

Ψc,N = 1.25 Where anchor is located in a region of a concrete member where analysis indicated no cracking (ft < fr) at service load levels, the modification is as follows:
Ψc,N = 1.25 for cast-in anchors
Ψc,N = 1.4 for Post-installed anchors
Ψc,N = 1.0 Cracking, post-installed, or cast-in anchors

ΦNcb =	16.19 KIPS

(D-4, D-5)

Pullout Strength in Tension: Φpn = 0.70

Abrg = 0.671   eh= 4.5do for J or L bolts

Np = 16,104.00 LB, D-15, D-16

Ψc,p = 1.4 Where anchor is located in a region of a concrete member where analysis indicated no cracking (ft < fr) at service load levels, the modification is as follows:
Ψ4 = 1.4 for cast-in anchors
Ψ4 = 1.0 Cracking, cast-in anchors

ΦNpn =	15.78	KIPS, Single Anchor (D-14)
ΦNpng =	31.56	KIPS, Group of Anchors (D-14)

Concrete side-face blowout strength of a headed anchor in tension:

Φnc = 0.7   Not considered when Ca,min > 0.4 hef

Nsb= NOT CONSIDERED
C = 4 IN.
C2 = 100 IN.
C2/C = 3

C is equal to the minimum edge distance and C2 is equal to the minimum edge distance perpendicular to C.
If C2 < 3 C then multiply by an additional factor per D.5.4.2.

Φnc = N/A

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IT #:

P11

SHEAR:

Steel Strength of Anchor in Shear: D6.1 Φ_{vsa} = 0.65

CI318 0.0, A_{se} = 0.226 IN² futa = 1.9 fya or 125,000 PSI = 58,000.0 PSI

Post-Installed Anchor Sleeves:
Asl = ______ IN2 futsl = ______ PSI

ΦV_{sa} = 10.2 KIPS, D-18 or D-19
AISC 7-10 ΦV_{sa}= 11.04 KIPS ΦF_s = 18 KSI, AISC TABLE 7-10
Assume threads included in shear plane & single shear application

ΦV_{sa} = **10.2** Kips/GROUP	ΦV_{sa} = 5.1 Kips/BOLT

Concrete Breakout Strength of Anchor in Shear, per D.6.2: Φ_{vs} = 0.7

Avco+y = 72.0 IN² Avco+x = 1152.0 IN²
Avco-y = 1152.0 IN² Avco-x = 1152.0 IN²
Avc+y = 96.0 IN² Avc+x = 672.0 IN²
Avc-y = 1248.0 IN² Avc-x = 672.0 IN²

vb+y= 3675.4 LB vb+x= 29403.3 LB
vb-y= 29403.3 LB vb-x= 29403.3 LB

Ψec,v +y = 1.0 Ψec,v +x = 1.0
Ψec,v -y = 1.0 Ψec,v -x = 1.0
Ψed+y = 1.0 Ψed+x = 0.8
Ψed-y = 1.0 Ψed-x = 0.8
Ψc,v = 1.4

Are the Anchors welded to the plate? NO
If yes, then anchor capacity is determined based on the row farthest from the edge and, the center to center spacing of the anchor is not less than 2.5 in.; and supplementary reinforcement is provided at the corners if C2 <= 1.5 hef

If three or more edge distances are less than 1.5 C1, (C2-4 or h<= 1.5 C1) then edge distance C1 is limited to C1 = h/1.5,
For use in eq. D-22 thru D-26

C'1+y = 4.0 IN C'1+x = 16.0
C'1-Y = 16.0 IN C'1-x = 16.0

e'vx = 0 IN., Eccentricity of shear force on a group of anchors, X axis
e'vy = 0 IN., Eccentricity of shear force on a group of anchors, Y axis

Ψc,v : D6.2.7, For anchors located in a region of concrete where analysis indicates no cracking (ft < fr) at service loads, then Ψc,v = 1.4

Ψe,v = 1.0, Anchors in cracked concrete with no supplementary reinf.
Ψe,v = 1.2, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & the edge.
Ψe,v = 1.4, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & edge & #4 stirrups.

ΦVcb(g)+y =	**4.80** KIPS	ΦVcb(g)+x =	**12.61** KIPS
ΦVcb(g) -y =	**31.22** KIPS	ΦVcb(g) -x =	**12.61** KIPS

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR: D.6.3 : Φ_{cp} = 0.7

Ncb = 23.1 KIPS From D-4 & Above

$\Phi_{cp}V_{cp}$ = **32.4 KIPS**	D-28

RESULTS:

TENSION KIPS	
ΦNsa =	19.7
ΦNcb =	16.2
ΦNpng =	31.6
Φnc =	N/A
ΦNna =	16.2

SHEAR KIPS			
ΦVsa =	10.2		
ΦVcb(g)+y =	4.80	ΦVcb(g)+x =	12.61
ΦVcb(g) -y =	31.22	ΦVcb(g) -x =	12.61
ΦcpVcp =	32.4		
ΦVn+y =	4.80	ΦVn+x =	10.22
ΦVn - y =	10.22	ΦVn- x =	10.22

HORIZONTAL LONGITUDINAL

F10

SHEAR:

Steel Strength of Anchor in Shear: D6.1 Φvsa = 0.65

CI318 0.0, Ase = 0.226 IN² futa = 1.9 fya or 125,000 PSI = 58,000.0 PSI

Post-Installed Anchor Sleeves: Asl = ______ IN2 futsl = ______ PSI

ΦVsa = 10.2 KIPS, D-18 or D-19
AISC 7-10 ΦVsa = 11.04 KIPS ΦFs = 18 KSI, AISC TABLE 7-10
Assume threads included in shear plane & single shear application

ΦVsa =	10.2	Kips/GROUP	ΦVsa =	5.1	Kips/BOLT

Concrete Breakout Strength of Anchor in Shear, per D.6.2: Φvs = 0.7

Avco+y =	72.0 IN²	Avco+x =	72.0 IN²
Avco-y =	1152.0 IN²	Avco-x =	1152.0 IN²
Avc+y =	84.0 IN²	Avc+x =	60.0 IN²
Avc-y =	768.0 IN²	Avc-x =	672.0 IN²
vb+y=	3675.4 LB	vb+x=	3675.4 LB
vb-y=	29403.3 LB	vb-x=	29403.3 LB
Ψec,v +y =	1.0	Ψec,v +x =	1.0
Ψec,v -y =	1.0	Ψec,v -x =	1.0
Ψed+y =	0.9	Ψed+x =	0.9
Ψed-y =	0.8	Ψed-x =	0.8
Ψc,v =	1.4		

Are the Anchors welded to the plate? NO
If yes, then anchor capacity is determined based on the row farthest from the edge and, the center to center spacing of the anchor is not less than 2.5 in.; and supplementary reinforcement is provided at the corners if C2 <= 1.5 hef

If three or more edge distances are less than 1.5 C1, (C2-4 or h<= 1.5 C1) then edge distance C1 is limited to C1 = h/1.5,
For use in eq. D-22 thru D-26

C'1+y =	4.0 IN	C'1+x =	4.0
C'1-Y =	16.0 IN	C'1-x =	16.0

e'vx = 0 IN., Eccentricity of shear force on a group of anchors, X axis
e'vy = 0 IN., Eccentricity of shear force on a group of anchors, Y axis

Ψc,v : D6.2.7, For anchors located in a region of concrete where analysis Indicates no cracking (ft < fr) at service loads, then Ψe,v = 1.4

Ψe,v = 1.0, Anchors in cracked concrete with no supplementary reinf.
Ψe,v = 1.2, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & the edge.
Ψe,v = 1.4, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & edge & #4 stirrups.

ΦVcb(g)+y =	3.78 KIPS	ΦVcb(g)+x =	2.70 KIPS
ΦVcb(g) -y =	14.41 KIPS	ΦVcb(g) -x =	12.61 KIPS

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR: D.6.3 :

Φcp = 0.7

Ncb = 16.5 KIPS From D-4 & Above

ΦcpVcp =	23.1 KIPS

D-28

RESULTS:

TENSION KIPS	
ΦNsa =	19.7
ΦNcb =	11.6
ΦNpng =	31.6
Φnc =	N/A
ΦNna =	11.6

SHEAR KIPS			
ΦVsa =	10.2		
ΦVcb(g)+y =	3.78	ΦVcb(g)+x =	2.70
ΦVcb(g) -y =	14.41	ΦVcb(g) -x =	12.61
ΦcpVcp =	23.1		
ΦVn+y=	3.78	ΦVn+x=	2.70
ΦVn - y =	10.22	ΦVn- x =	10.22

HORIZONTAL LONGITUDNAL

SHEAR:

Steel Strength of Anchor in Shear: D6.1 Φvsa = 0.65

CI318 0.D. Ase = 0.226 IN² futa = 1.9 fya or 125,000 PSI = 58,000.0 PSI

Post-installed Anchor Sleeves: Asl = ____ IN2 futsl = ____ PSI

ΦVsa = 10.2 KIPS, D-18 or D-19
AISC 7-10 ΦVsa = 11.04 KIPS ΦFs = 18 KSI, AISC TABLE 7-10
Assume threads included in shear plane & single shear application

ΦVsa = 10.2 Kips/GROUP	ΦVsa = 5.1 Kips/BOLT

Concrete Breakout Strength of Anchor in Shear, per D.6.2: Φvs = 0.7

Avco+y =	72.0 IN²	Avco+x =	72.0 IN²
Avco-y =	1152.0 IN²	Avco-x =	1152.0 IN²
Avc+y =	84.0 IN²	Avc+x =	60.0 IN²
Avc-y =	768.0 IN²	Avc-x =	672.0 IN²
vb+y=	3675.4 LB	vb+x=	3675.4 LB
vb-y=	29403.3 LB	vb-x=	29403.3 LB
Ψec,v +y =	1.0	Ψec,v +x =	1.0
Ψec,v -y =	1.0	Ψec,v -x =	1.0
Ψed+y =	0.9	Ψed+x =	0.9
Ψed-y =	0.8	Ψed-x =	0.8

Ψc,v = 1.4

Are the Anchors welded to the plate? NO
If yes, then anchor capacity is determined based on the row farthest from the edge and, the center to center spacing of the anchor is not less than 2.5 in., and supplementary reinforcement is provided at the corners if C2 <= 1.5 hef

If three or more edge distances are less than 1.5 C1, (C2-4 or h<= 1.5 C1) then edge distance C1 is limited to C1 = h/1.5,
For use in eq. D-22 thru. D-26

C'1+y =	4.0 IN	C'1+x =	4.0
C'1-Y =	16.0 IN	C'1-x =	16.0

e'vx = 0 IN., Eccentricity of shear force on a group of anchors, X axis
e'vy = 0 IN., Eccentricity of shear force on a group of anchors, Y axis

Ψc,v : D6.2.7, For anchors located in a region of concrete where analysis indicates no cracking fit < ft at service loads, then Ψc,v = 1.4

Ψc,v = 1.0, Anchors in cracked concrete with no supplementary reinf.
Ψc,v = 1.2, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & the edge.
Ψc,v = 1.4, Anchors in cracked concrete with supplementary reinf. #4 or greater between the anchor & edge & #4 stirrups.

ΦVcb(g)+y =	3.78 KIPS	ΦVcb(g)+x =	2.70 KIPS
ΦVcb(g) -y =	14.41 KIPS	ΦVcb(g) -x =	12.61 KIPS

CONCRETE PRYOUT STRENGTH OF ANCHOR IN SHEAR: D.6.3 : Φcp = 0.7

Ncb = 16.5 KIPS From D-4 & Above

ΦcpVcp = 23.1 KIPS	D-23

RESULTS:

TENSION KIPS	
ΦNsa =	19.7
ΦNcb =	11.6
ΦNpng =	31.6
Φnc =	N/A
ΦNna =	11.6

SHEAR KIPS			
ΦVsa =	10.2		
ΦVcb(g)+y =	3.78	ΦVcb(g)+x =	2.70
ΦVcb(g) -y =	14.41	ΦVcb(g) -x =	12.61
ΦcpVcp =	23.1		
ΦVn+y =	3.78	ΦVn+x =	2.70
ΦVn-y =	10.22	ΦVn-x =	10.22

HORIZONTAL LONGITUDINAL

COPY
OPMENT

F II

SPACING & EDGE DISTANCES:

SPACING:

Minimum center to center spacing of untorqued cast-in-place anchors = 4do = **2.5** IN.

Minimum o.c. spacing of torqued cast-in-place & post-installed anchors = 6do = **3.8** IN.

EDGE DISTANCE:

Minimum edge distance for cast-in headed anchors, un-torqued shall be based on minimum cover requirements for reinforcement of ACI 7.7:

Concrete cast against and permanently exposed to soil: **3** IN.

Concrete exposed to earth or weather:
#6-#18 bars **2** IN.
#5 bar & smaller: **1 1/2** IN.

Concrete not exposed to weather or in contact with ground:
#14 & #18 bars: **1 1/2** IN.
#11 bars & smaller: **3/4** IN.

Minimum edge distance for post-installed anchors shall be based on the greater of the minimum cover requirements for reinforcement in ACI 7.7, or the minimum edge distance requirements for the products as determined by tests in accordance with ACI 355.2 and shall not be less than 2.0 times the maximum aggregate size. In the absence of product-specific ACI 355.2 test information, the minimum edge distance shall be taken as not less than:

Undercut anchors, 6do = **3.75** IN.

Torqued-controlled anchors, 8do = **5** IN.

Displacement - controlled anchors, 10do = **6.25** IN.

APP
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PERM
DATE:

BEAM LOADING DATA

Span		Channel	Max. Allowable Uniform Load		Deflection at Uniform Load		Uniform Loading at Deflections					
							Span/180		Span/240		Span/360	
In	mm		Lbs	kN	In	mm	Lbs	kN	Lbs	kN	Lbs	kN
24	610	P3000	1290	5.7	0.07	2	1290	5.7	1290	5.7	1290	5.7
		P3001	2660*	11.8	0.03	1	2660*	11.8	2660*	11.8	2660*	11.8
36	914	P3000	860	3.8	0.15	4	860	3.8	860	3.8	590	2.6
		P3001	2410	10.7	0.08	2	2410	10.7	2410	10.7	2410	10.7
48	1219	P3000	650	2.9	0.26	7	650	2.9	500	2.2	330	1.5
		P3001	1810	8.1	0.15	4	1810	8.1	1810	8.1	1620	7.2
60	1524	P3000	520	2.3	0.41	10	420	1.9	320	1.4	210	0.9
		P3001	1450	6.4	0.23	6	1450	6.4	1450	6.4	1040	4.6
72	1829	P3000	430	1.9	0.59	15	290	1.3	220	1.0	150	0.7
		P3001	1200	5.3	0.33	8	1200	5.3	1080	4.8	720	3.2
84	2134	P3000	370	1.6	0.80	20	220	1.0	160	0.7	110	0.5
		P3001	1030	4.6	0.45	12	1030	4.6	790	3.5	530	2.4
96	2438	P3000	320	1.4	1.03	26	170	55 point	120	0.5	80	0.4
		P3001	900	4.0	0.59	15	810	405 point	610	2.7	400	1.8
108	2743	P3000	290	1.3	1.33	34	130	0.6	100	0.4	70	0.3
		P3001	800	3.6	0.75	19	640	2.8	480	2.1	320	1.4
120	3048	P3000	260	1.2	1.64	42	110	0.5	80	0.4	50	0.2
		P3001	720	3.2	0.93	24	520	2.3	390	1.7	260	1.2
144	3658	P3000	220	1.0	2.40	61	70	0.3	60	0.3	40	0.2
		P3001	600	2.7	1.33	34	360	1.6	270	1.2	180	0.8
168	4267	P3000	180	0.8	3.11	79	50	0.2	40	0.2	30	0.1
		P3001	520	2.3	1.84	47	260	1.2	200	0.9	130	0.6
192	4877	P3000	160	0.7	4.13	105	40	0.2	30	0.1	NR	NR
		P3001	450	2.0	2.37	60	200	0.9	150	0.7	100	0.4
216	5486	P3000	140	0.6	5.15	131	NR	NR	NR	NR	NR	NR
		P3001	400	1.8	3.00	76	160	0.7	120	0.5	80	0.4
240	6096	P3000	130	0.6	6.56	167	NR	NR	NR	NR	NR	NR
		P3001	360	1.6	3.70	94	130	0.6	100	0.4	60	0.3

*Load limited by spot weld shear.

NR = Not Recommended

Notes:

1. Above loads include the weight of the member. This weight must be deducted to arrive at the net allowable load the beam will support.
2. Long span beams should be supported in such a manner as to prevent rotation and twist.
3. Allowable uniformly distributed loads are listed for various simple spans, that is, a beam on two supports. If load is concentrated at the center of the span, multiply load from the table by 0.5 and corresponding deflection by 0.8.
4. See page 66 for lateral bracing load reduction charts.

BEAM LOADING DATA

Span in	Span mm	Channel	Max. Allowable Uniform Load Lbs	Max. Allowable Uniform Load kN	Deflection at Uniform Load in	Deflection at Uniform Load mm	Span/180 Lbs	Span/180 kN	Span/240 Lbs	Span/240 kN	Span/360 Lbs	Span/360 kN
24	610	P5000 P5001	5260† 6170*†	23.4 27.4	0.03 0.01	1 0	5260 6170*†	23.4 27.4	5260 6170*†	23.4 27.4	5260 6170*†	23.4 27.4
36	914	P5000 P5001	3510 6170*†	15.6 27.4	0.07 0.02	2 1	3510 6170*†	15.6 27.4	3510 6170*†	15.6 27.4	3510 6170*†	15.6 27.4
48	1219	P5000 P5001	2630 5650†	11.7 25.1	0.12 0.05	3 1	2630 5650†	11.7 25.1	2630 5650†	11.7 25.1	2630 5650†	11.7 25.1
60	1524	P5000 P5001	2110 4520†	9.4 20.1	0.18 0.08	5 2	2110 4520†	9.4 20.1	2110 4520†	9.4 20.1	1920 4520†	8.5 20.1
72	1829	P5000 P5001	1750 3770	7.8 16.8	0.26 0.11	7 3	1750 3770	7.8 16.8	1750 3770	7.8 16.8	1330 3770	5.9 16.8
84	2134	P5000 P5001	1500 3230	6.7 14.4	0.36 0.15	9 4	1500 3230	6.7 14.4	1470 3230	6.5 14.4	980 3230	4.4 14.4
96	2438	P5000 P5001	1320 2830	5.9 12.6	0.47 0.20	12 5	1320 2830	[illegible] Point 1415 Point	1130 2830	5.0 12.6	750 2830	3.3 12.6
108	2743	P5000 P5001	1170 2510	5.2 11.2	0.59 0.25	15 6	1170 2510	5.2 11.2	890 2510	4.0 11.2	590 2510	2.6 11.2
120	3048	P5000 P5001	1050 2260	4.7 10.1	0.73 0.31	19 8	960 2260	4.3 10.1	720 2260	3.2 10.1	480 2260	2.1 10.1
144	3658	P5000 P5001	880 1880	3.9 8.4	1.06 0.44	27 11	670 1880	3.0 8.4	500 1880	2.2 8.4	330 1690	1.5 7.5
168	4267	P5000 P5001	750 1610	3.3 7.2	1.43 0.60	36 15	490 1610	2.2 7.2	370 1610	1.6 7.2	250 1240	1.1 5.5
192	4877	P5000 P5001	660 1410	2.9 6.3	1.88 0.79	48 20	380 1410	1.7 6.3	280 1410	1.2 6.3	190 950	0.8 4.2
216	5486	P5000 P5001	580 1260	2.6 5.6	2.35 1.00	60 26	300 1260	1.3 5.6	220 1130	1.0 5.0	150 750	0.7 3.3
240	6096	P5000 P5001	530 1130	2.4 5.0	2.94 1.24	75 31	240 1130	1.1 5.0	180 910	0.8 4.0	120 610	0.5 2.7

*Load limited by spot weld shear. †Bearing load may govern capacity. See page 67.

Notes:

1. Above loads include the weight of the member. This weight must be deducted to arrive at the net allowable load the beam will support.
2. Long span beams should be supported in such a manner as to prevent rotation and twist.
3. Allowable uniformly distributed loads are listed for various simple spans, that is, a beam on two supports. If load is concentrated at the center of the span, multiply load from the table by 0.5 and corresponding deflection by 0.8.
4. See page 66 for lateral bracing load reduction charts.

G3

Bentley DLR Group

MECHANICAL SUPPORT ANGLES
≤ 2000 LBS, CENTERED

Current Date: 10/20/2009 3:37 PM
Units system: English
File name: \

Design Results

Continuous Beam
Design code ANSI/AISC 360-05 LRFD

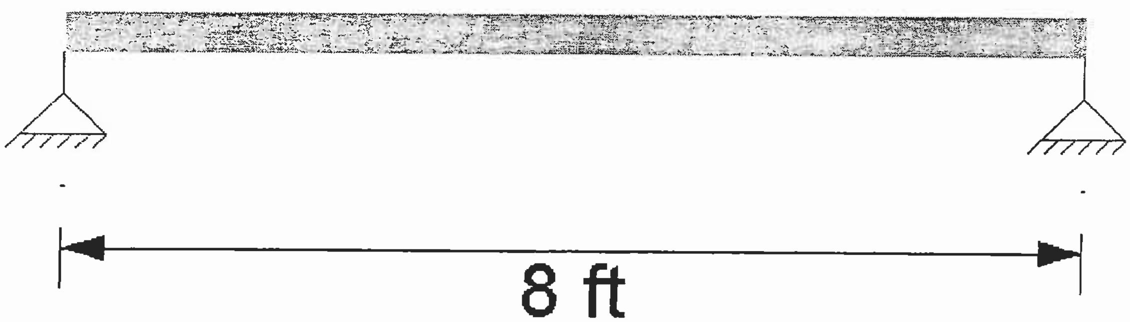
GENERAL INFORMATION:

Spans:

Span	Span length [ft]	Section	Material
1	8.00	T2LU 4X3X1_4LLBB	A36

Nodes:

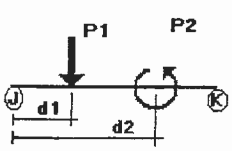
Distance [ft]	Restraint	Tx	Ty	Rz
0.00	Pinned	1	1	0
8.00	Pinned	1	1	0



Load conditions:

Condition	Description	Comb.	Category	Duration
DL	Dead Load	No	DL	--
LL	Live Load	No	LL	--
S1	DL+LL	Yes	Service	--
D1	1.4DL	Yes	Design	--
D2	1.2DL+1.6LL	Yes	Design	--

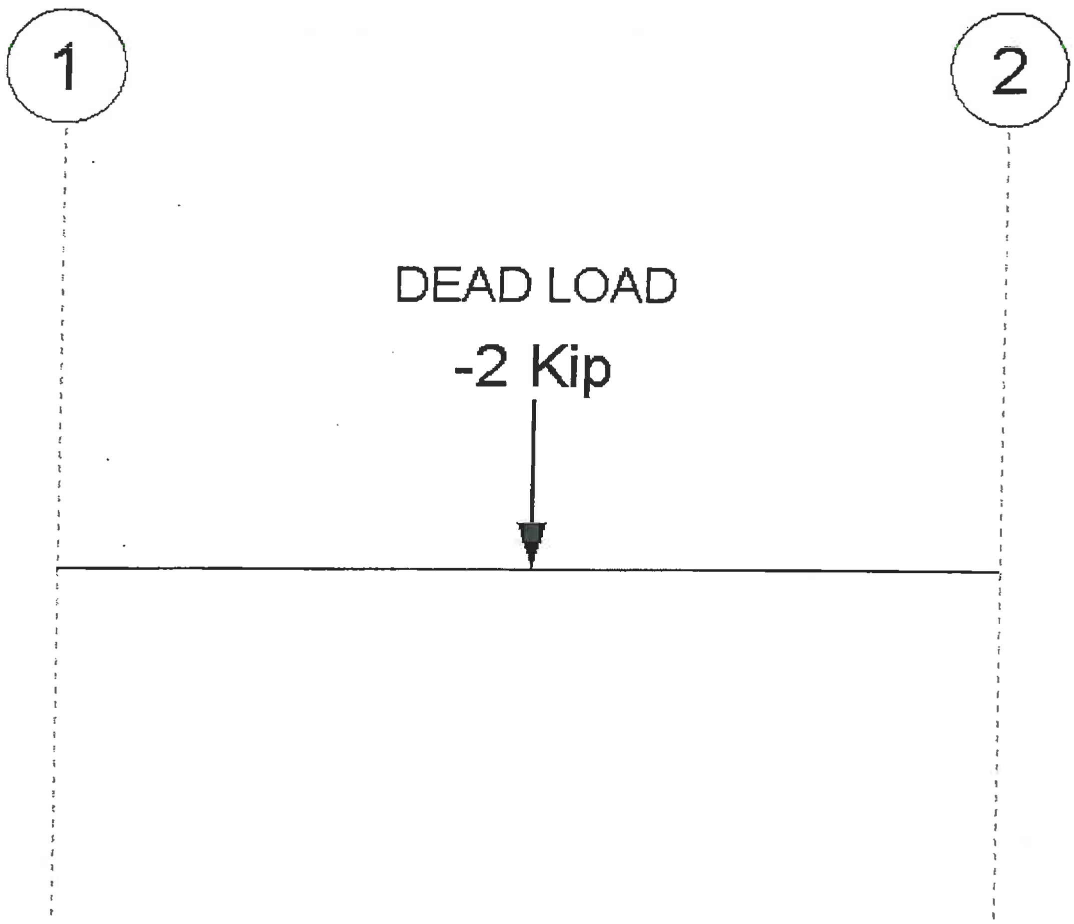
Concentrated forces and moments



G4

Condition	Span	Dist [ft]	P [Kip]	M [Kip*ft]
DL	1	4.00	-2.00	0.00

Loads summary



Reactions:

Nodes	Load condition	Rx [Kip]	Ry [Kip]	Mz [Kip*ft]
1	D1	0.00	1.46	0.00
2	D1	0.00	1.46	0.00
1	D2	0.00	1.26	0.00
2	D2	0.00	1.26	0.00
1	Min.	0.00	1.26	0.00
2	Min.	0.00	1.26	0.00
1	Max.	0.00	1.46	0.00
2	Max.	0.00	1.46	0.00

G5

Member forces and Inflection points

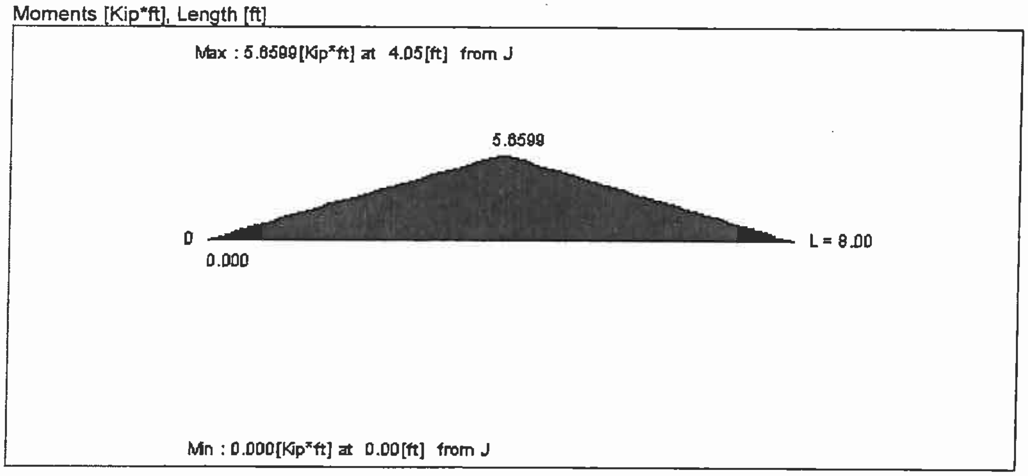
Station [%]	Condition	Distance [ft]	Shear V [Kip]	Moment M [Kip*ft]
0	D1	0.00	-1.46	0.00
50	D1	4.00	1.40	5.73
100	D1	8.00	1.46	0.00
0	D2	0.00	-1.26	0.00
50	D2	4.00	1.20	4.91
100	D2	8.00	1.26	0.00

Critical deflections

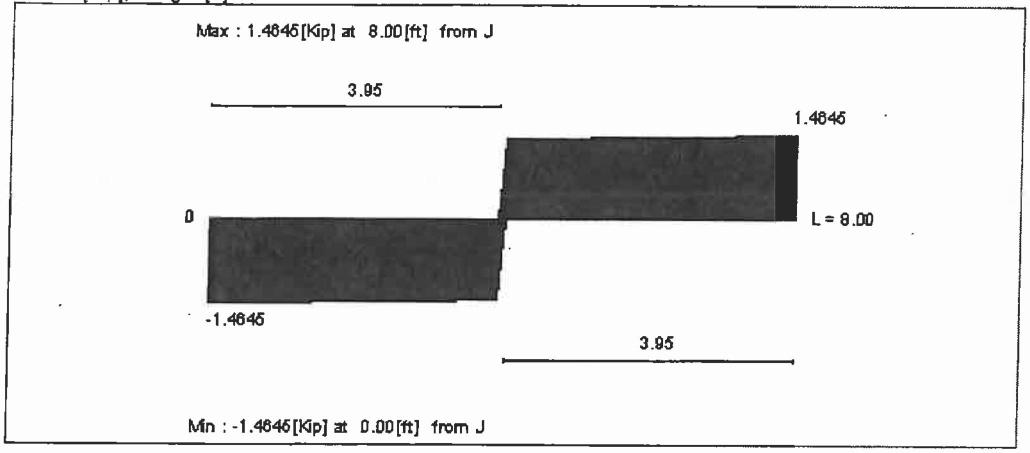
Condition	Span	Distance [ft]	@ [%]	Deflection [in]	Deflection f(L)	Allowable [in]
S1	1	4.00	50.00	0.23973	(L/400)	0.53333

Envelopes :

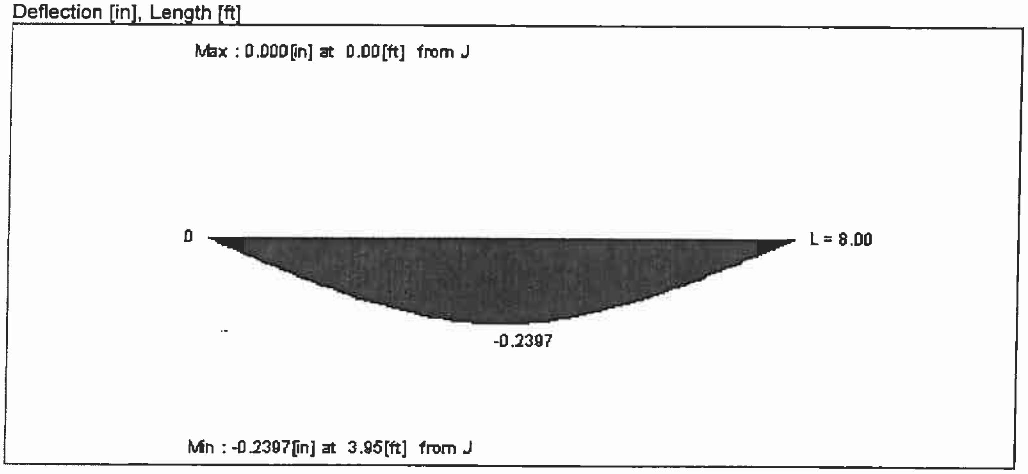
M33 bending moment
Moments [Kip*ft], Length [ft]



V2 shear forces:
Forces [Kip], Length [ft]



Vertical Translation
Deflection [in], Length [ft]



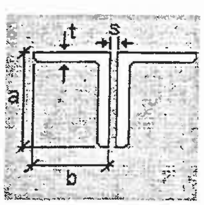
PRO
COUNT
MIT #:
E:

DESIGN:

Span	**:**	**1 (T2LU 4X3X1_4LLBB_A36)**
Design status	**:**	**OK**

PROPERTIES

Section : T2LU 4X3X1_4LLBB



Height (a)	4.00	[in]
Width (b)	3.00	[in]
Separation (s)	0.00	[in]
Thickness (t)	0.25	[in]

Section properties	**Unit**	**Major axis**	**Minor axis**
Full unreduced cross-sectional area (A)	[in2]	3.38	
Moment of Inertia (local axes) (I)	[in4]	5.49	4.47
Moment of Inertia (principal axes) (I')	[in4]	5.49	4.47
Bending constant for moments (principal axis) (J')	[in]	-1.13	0.00
Radius of gyration (local axes) (r)	[in]	1.27	1.15
Radius of gyration (principal axes) (r')	[in]	1.27	1.15
Saint-Venant torsion constant (J)	[in4]	0.07	
Warping constant of the cross-section (Cw)	[in6]	0.15	
Distance from centroid to shear center (principal axis) (xo, yo)	[in]	0.00	-0.76
Top elastic section modulus of the section (local axis) (S top)	[in3]	4.50	1.51
Bottom elastic section modulus of the section (local axis) (S bot)	[in3]	1.98	1.51
Top elastic section modulus of the section (principal axis) (S' top)	[in3]	4.50	1.51
Bottom elastic section modulus of the section (principal axis) (S' bot)	[in3]	1.98	1.51
Plastic section modulus (local axis) (Z)	[in3]	3.55	2.49
Plastic section modulus (principal axis) (Z')	[in3]	3.55	2.49
Polar radius of gyration (ro)	[in]	2.03	
Area for shear (Aw)	[in2]	1.50	2.00
Torsional modulus (1/C)	--	3.56	

Material : A36

Properties	**Unit**	**Value**
Yield stress (Fy):	[Kip/in2]	36.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11507.94

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DESIGN CRITERIA

Description	**Unit**	**Major axis**	**Minor axis**
Top unbraced length between lateral supports (LbTop)	[ft]	8.00	
Bottom unbraced length between lateral supports (LbBop)	[ft]	8.00	
Effective length factor (K)	--	1.00	1.00
Effective length factor for torsion	--	1.00	
Length for axial tension (L)	[ft]	8.00	
Unbraced compression length (Lx, Ly)	[ft]	8.00	8.00
Length for torsion and lateral-torsional buckling	[ft]	8.00	
Clear distance between longitudinal connectors	[ft]	0.00	

Additional hypotheses	
Continuous lateral torsional restraint	No
Tension field action	No

SERVICE CONDITIONS

Verification	**Unit**	**Value**	**Ctrl EQ**	**Reference**
Tension				
Maximum geometric slenderness (L/r)	--	83.48		(Sec. D1)
Compression				
Geometric critical slenderness (KL/r)	--	83.48		(Sec. E2)

Compression and flexure

Deflection	[in]	-0.24	S1 at 50.00%

DESIGN CHECKS

DESIGN FOR FLEXURE(ϕ= 0.90) ✔

Bending about major axis, M33

Ratio	:	0.67			
Capacity	:	8.53 [Kip*ft]	Ctrl Eq.	:	D1 at 50.00%
Demand	:	5.73 [Kip*ft]	Reference	:	(Sec. F)

Intermediate results	Unit	Value	Reference
Yielding (Mp)	[Kip*ft]	9.48	(Sec. F)
Lateral-torsional buckling (LTB Mn)	[Kip*ft]	56.49	(Sec. F)
Modification factor for lateral-torsional buckling (Cb)	–	1.31	(Sec. F1)
Lateral-torsional factor (c)	–	1.00	(Sec. F2.2)
Factor for lateral-torsional buckling in tees and 2L (B)	–	0.76	(Eq. F9-5)
Web local buckling (WLB Mn)	–	N/A	(Sec. F)
Local buckling (LB Mn)	–	N/A	(Sec. F)
Flange local buckling (FLB Mn)	–	N/A	(Sec. F)
Slenderness parameter for flange (λ)	–	12.00	(Sec. B4)
Limiting slenderness parameter for compact flange (λp)	–	15.33	(Sec. B4)
Limiting slenderness parameter for noncompact flange (λr)	–	25.83	(Sec. B4)
Tension flange yielding (TFY Mn)	–	N/A	(Sec. F)

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	6.72 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%
Demand	:	0.00 [Kip*ft]	Reference	:	(Sec. F)

Intermediate results	Unit	Value	Reference
Yielding (Mp)	[Kip*ft]	7.46	(Sec. F)
Flange local buckling (FLB Mn)	–	N/A	(Sec. F)
Slenderness parameter for flange (λ)	–	12.00	(Sec. B4)
Limiting slenderness parameter for compact flange (λp)	–	15.33	(Sec. B4)
Limiting slenderness parameter for noncompact flange (λr)	–	25.83	(Sec. B4)

DESIGN FOR SHEAR ✔

Shear parallel to major axis, V3 (ϕ= 0.90)

Ratio	:	0.00			
Capacity	:	29.16 [Kip]	Ctrl Eq.	:	D1 at 0.00%
Demand	:	0.00 [Kip]	Reference	:	(Sec. G)

Intermediate results	Unit	Value	Reference
Web Shear coefficient (Cv)	–	1.00	
Web plate buckling coefficient (kv)	–	1.20	(Sec. G2)

Shear parallel to minor axis, V2 (ϕ= 0.90)

Ratio	: 0.04			
Capacity	: 38.88 [Kip]	Ctrl Eq.	: D1 at 0.00%	
Demand	: -1.46 [Kip]	Reference	: (Sec. G)	

Intermediate results	Unit	Value	Reference
Web Shear coefficient (Cv)	--	1.00	
Web plate buckling coefficient (kv)	--	1.20	(Sec. G2)

DESIGN FOR TENSION ϕ= 0.90) ✔

Tension

Ratio	: 0.00		
Capacity	: 109.51 [Kip]	Ctrl Eq.	: D1 at 0.00%
Demand	: 0.00 [Kip]	Reference	: (Sec. D)

DESIGN FOR COMPRESSION ϕ= 0.90) ✔

Compression

Ratio	: 0.00		
Capacity	: 63.75 [Kip]	Ctrl Eq.	: D1 at 0.00%
Demand	: 0.00 [Kip]	Reference	: (Sec. E)

Intermediate results	Unit	Value	Reference
Slenderness parameter for web (λw)	--	16.00	(Sec. B4)
Limiting slenderness parameter for noncompact web (λrw)	--	15.89	(Sec. B4)
Slenderness parameter for flange (λf)	--	12.00	(Sec. B4)
Limiting slenderness parameter for noncompact flange (λrf)	--	12.77	(Sec. B4)
Elastic flexural stress (Fex)	[Kip/in2]	50.44	(Eq. E4-9)
Elastic flexural stress (Fey)	[Kip/in2]	41.07	(Ec. E4-10)
Elastic torsional buckling stress (Fez)	[Kip/in2]	58.09	(Eq. E4-11)
Critical elastic flexural-torsional buckling stress (Fe)	[Kip/in2]	30.63	(Sec.E4)
Critical flexural buckling stress (Fcr)	[Kip/in2]	23.49	(Sec.E)
Critical flexural-torsional buckling stress (FcrTor)	[Kip/in2]	20.96	(Sec.E4)
Stress reduction factor in unstiffened elements (Qs)	--	0.91	(Sec.E7)
Effective section reduction factor in stiffened elements (Qa)	--	1.00	(Sec.E7)
Effective area at a uniform stress (Aeff)	[in2]	3.38	(Sec.E7)

DESIGN FOR TORSION ϕ= 0.90) ✔

Torsion

Ratio	: 0.00		
Capacity	: 0.46 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%
Demand	: 0.00 [Kip*ft]	Reference	: (Sec. H3)

Intermediate results	Unit	Value	Reference
Critical stress (Fcr)	[Kip/in2]	21.60	(Sec. H)

INTERACTION ✔

G10

Combined axial and flexure interaction value

Ratio	:	0.67			
Ctrl Eq	:	D1 at 50.00%	Reference	:	(H1-1b)

Combined shear and torsion interaction value

Ratio	:	0.04			
Ctrl Eq.	:	D1 at 0.00%	Reference	:	(Ec. 4.9) DG 9

CRITICAL STRENGTH RATIO ✔

Ratio	:	0.67			
Ctrl Eq	:	D1 at 50.00%	Reference	:	(Sec. F)

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