Jimco
SALES AND
MANUFACTURING

Job Title: Tex Best Travel Center #501
20290 IH-37
Elmendorf, TX

Job No. 23-3108R01

Calc'd By: R. Lauer **Date:** 19-Jan-2024

Checked by: V. Herrera, PE **Date:** 19-Jan-2024

STRUCTURAL CALCULATIONS

FOR

Dos Tienolas Inc.

Canopy Structural Steel Quick Reference:

Header Beams:	(6)	W14X22
Purlin Beams:	(3)	W10X12
Columns:	(12)	HSS12X12X1/4

AISC CERTIFIED FABRICATOR

1968 - 2023
55
YEARS OF EXCELLENCE
Jimco Sales & Manufacturing Co.

Digitally signed by Victor Herrera
Date: 2024.01.30 08:19:31 -06'00

STATE OF TEXAS
VICTOR M. HERRERA
141679
LICENSED
PROFESSIONAL ENGINEER

Job No: 23-3108R01

Building Code:

2021 International Building Code
AISC Manual -15th Edition
ASCE 7-16

The ASD method of design has been used unless noted otherwise.

Canopy Specifications:

Length:	**110.0 ft**	Total Canopy Height:	20.00 ft
Width:	**28.0 ft**	Number of Column Rows:	2
Fascia Height:	**3.0 ft**	Number of Columns / Row:	6
Canopy Clearance height:	**17.0 ft**	TTL Columns:	12
Canopy Area =	3080 SqFt		

Dead Loads:

Colateral Load: **0.0 psf**

Total Canopy Dead Load =	**10.33 psf**
Dead Load on Purlins =	5.00 psf
Mansard Dead Load =	0.00 psf

Snow Loads:

Pg-Ground Snow Load =	**5.00 psf**	**ATC**
Pf-Roof Snow Load =	4.20 psf	
Rain on Snow Surcharge =	5.00 psf	
Combined Snow =	9.20 psf	

Roof Live Load: 20 psf

Live Load Reduction:

At = 280.00 Sqft F = 0

R1 = 0.92 R2 = 1.00

Reduced Live Load (Lr) = 18 psf

(for Columns and Footings)

Load Transfer System in Canopy: Loads are transferred from the fascia system to the inter locking Deck Pans. The Deck Pans transfer the load through the Deck Clips to the wide flange Purlin Beams. The Purlin Beams transmit the load to the Header Beams. The Header Beams carry the load to the Cantilevered Columns. Finally, the Columns transmit the load to the Foundations through Baseplates and Anchor Bolts.

Wind Forces:

Wind Speed (Vult) =	**109 mph**	**ATC**
Vasd =	84 mph	
Exposure =	**C**	
Risk Category =	II	

Seismic Forces: Ω= 1.25

12.3.4 - Redundancy For SDC = B,C

ρ = 1.0

Site Class = **D**

Seismic Design Category = **B**

The maximum Considered (Mapped) Spectral Acceleration (ASCE sec. 11.4.1)

Ss-	**0.053 g**	**ATC**	Sds-	**0.130 g**
S1-	**0.020 g**	**ATC**	Sd1-	**0.092 g**
Importance Factor =	1.0		Cs-	**0.10**
Total Canopy Dead Load			DL =	10.33 psf
Seismic W=DL*A =		31.80 kips		
Seismic Force ($V=C_SW$) =		**3.31 kips**		

Seismic Resisting System:

The canopy is classified as cantilevered column system detailed to conform to the requirements for Steel Ordinary Column Systems as per ASCE 7 Table 9.5.2.2 and has been designed using Equivalent Lateral Force Procedure as per Section 9.9.5.

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ASCE 7-16 Wind Forces (Chapter 27 Directional Procedure)

Open Building w/Monoslope roof (Section 27.3.2) w/ Fascia Panels as Parapets (Section 27.3.4)

Gust Effect Factor (Section 26.9) The Canopy's Fundamental natural frequency is greater than 1Hz, and is therefore rigid as defined in Secton 26.2. Therefore, as per Section 26.9.1

as per table (26.6-1) Directional factor G= 0.85

K_d= 0.85

Wind Velocity pressure section 27.3.2, Eq-27.3.2

For Open Buildings: $q_z = 0.00256(K_z)(K_{zt})(K_d)(V^2)$

Terrain Exposure Constrants	α =9.50	Canopy Clear Hgt (H)=	17.00 ft
zg (Table 26.9-1) =	900.0 ft	Fascia Hgt =	3.00 ft
Fascia Hgt (Parapet) =	3.00 ft	**h=**	18.50 ft
K_z =2.01(h/zg^2/α	0.887	Parapet Hgt = Clr Hft + Fascia Hgt =	20.00 ft
Topographic Factor K_{zt} =	1.00	Kp =2.01(h/zg^2/α	0.90
$q_z = 0.00256(K_z)(K_{zt})(K_d)(V^2)$		$q_p = 0.00256(K_p)(K_{pt})(K_d)(V^2)$	
Therefore, q_z =	22.94 psf	Therefore, q_p =	23.32 psf

MWFRS Horizontal Forces (Section 27.4.3)

Wind Design Load for Fascia: $qp = 0.00256(K_p)(K_{pt})(K_d)(V^2)$

Parapet Wind Pressure for MWFRS: $p=q_p(GC_p - GC_{pi})$

From Section (6.5.12.4.4)	Windward GCpn =	1.5
	Leeward GCpn =	-1.0
Wind Shear for maximum Tributary Area to the Column Pw = p_p*A	Windward Fascia P_p =	34.97 psf
	Leeward Fascia P_p =	-23.32 psf

Maximum Tributary Area of Single Column C2, A = hx B — Tributary Width = B= 20.00 ft

Fascia Hgt: h = 3.00 ft — A= 60.0 ft

Thus, Pw = 1.75 kips for Single Column

MWFRS (Section 27.3-4)

When Flat Roof:

Clear Leeward or Windward flow will control design (obstructions always < 50%)

From Figure 27.4.4, Worst case Cn =	-1.10	q = qGCn	U_{plift} =	-21.45 psf
From Figure 27.4.4, Worst case Cn =	1.20		$Q_{downward}$ =	23.40 psf

Uplift Force Puplift = (Uplift Area* Uplift pressure quplift) — P_{uplift} = A* q_{uplift}

Uplift Area= Max Tributary column Length * Max. Cantilever Width

Max Tributary column Length = L 20.00 ft

B 14.00 ft

P_{uplift} = 6.00 psf

Total Wind Force = 19236.11 lbs = 19.24 kips

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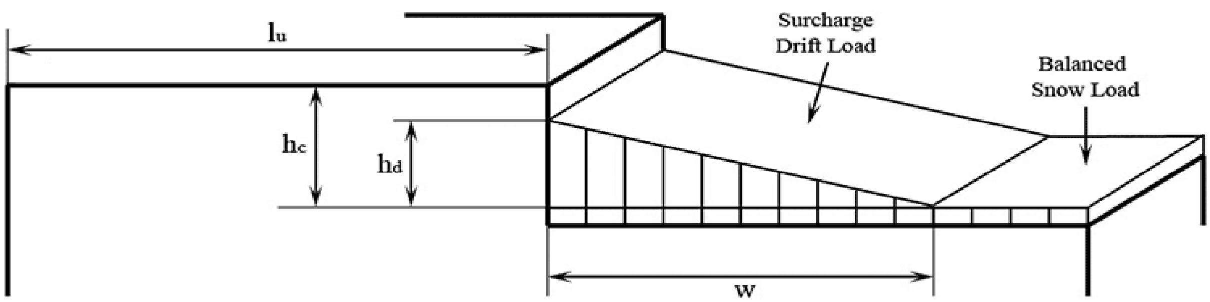
Flat Snow Load:

Pg =	5.00 psf	Important Factor	**Is = 1.00**
Pf =	4.20 psf	Unheated and Open Air Structure (ASCE Table 7-3)	
Rain on Snow =	5.00 psf	Exposure Factor	**Ce = 1.00**
Balanced Snow Load =	9.20 psf	Risk Category II - (ASCE Table 1.5-1 & Table 1.5-2)	
Fascia Hgt =	3.00 ft	Thermal Factor	**Ct = 1.20**
		Terrain Category C - (ASCE Table 7-2)	

Snow Drift:



Snow Density:

γ = 0.13(Pg) + 14 =	14.65 pcf		
Bal. Snow Hgt h_b =	0.63 ft		0.63 ft
Clear Height h_c =	1.96 ft		1.96 ft
Drift hgt h_d =	1.92 ft		0.80 ft
Drift Length W =	7.66 ft		3.21 ft
Length Lu =	110.00 ft	Width=	28.00 ft
Drift Surcharge Pd =	0.00 psf		0.00 psf
Combined Snow =	9.20 psf		9.20 psf

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Jimco Standard - Max Deflection: l/180 Span; l/90 Cantilever

Purlins: Purlin Count: 3 Bwgt= 12 lbs

Use:	**W10X12**	Moment check:	OK	Ix= 53.8 in^4	Bh= 10 in
	fy=50ksi	Weak Axis Check:	OK		

Max Trib Width =	**10.50 ft**		**Simple Span**	
Left Cantilever =	5.00 ft	5	spans @	20.00 ft
Right Cantilever =	5.00 ft	0	spans @	0.00 ft
		0	spans @	0.00 ft

Purlin Spacing: **Length - 110.0 ft**

3.50	10.50	10.50	3.50

Brace Spacing: 3 @ 6.67 ft O.C. Max.

Gravity Loads

D=	0.06 klf	Wdn =	0.25 klf
Cant Lr/S =	0.21 klf	Wup =	-0.23 klf
Span Lr/S =	0.21 klf	Ev =	0.002 klf

Uniform Loads

Wttl Cantilever =	0.33 klf
Wttl Simple Span =	0.33 klf

Vertical Loads

Purlin Cantilever Moment:	M_r =	4.16 ft-k	W_v =	0.023 klf
Purlin Simple Span Moment:	M_r =	16.63 ft-k	E_v =	0.0002 klf
Purlin Weak Axis Moment:	M_{rh} =	1.75 ft-k	S =	0.009 klf

Lateral Loads

W_h =	0.058 klf
E_h=	0.007 klf

Beam Slenderness

Flange	Web
NC	Comp

Beam Flexural Design

	Mp	Mltb	Mflb	Mn	Ma
Strong:	--	37.22	52.1	37.22 k-ft	22.29 k-ft
	1.74				
Weak:	7.25	11.60		7.25 k-ft	4.34 k-ft

ASCE Load Combinations (ASD)

ASCE 7-05? No

Wind Factor 0.6

	STRONG	WEAK	COMBINED	
1. D	0.116	0.000	0.12	
2. D+L	0.116	0.000	0.12	
3. D+(Lr or S)	0.494	0.000	0.49	
4. D + 0.75L + 0.75(Lr or S)	0.400	0.000	0.40	
5a. D + 0.6W	0.377	0.405	0.78	
5b. D + 0.7E	0.119	0.057	0.18	
6a. D + 0.75L +0.75(0.6W) + 0.75(Lr or S)	0.595	0.304	0.90	**Controls**
6b. D + 0.75L +0.75(0.7E) + 0.75(Lr or S)	0.402	0.043	0.44	
7. 0.6D + 0.6W	0.331	0.405	0.74	
8. 0.6D + 0.7E	0.072	0.057	0.13	

Steel Beam

Project File: 23-3108R01 28X110 ELMENDORF, TX MOBIL 2Di.ec6

LIC# : KW-06014602, Build:20.23.07.20 Jimco Sales & Mfg

DESCRIPTION: Purlin Beam 1 Strong Axis

CODE REFERENCES

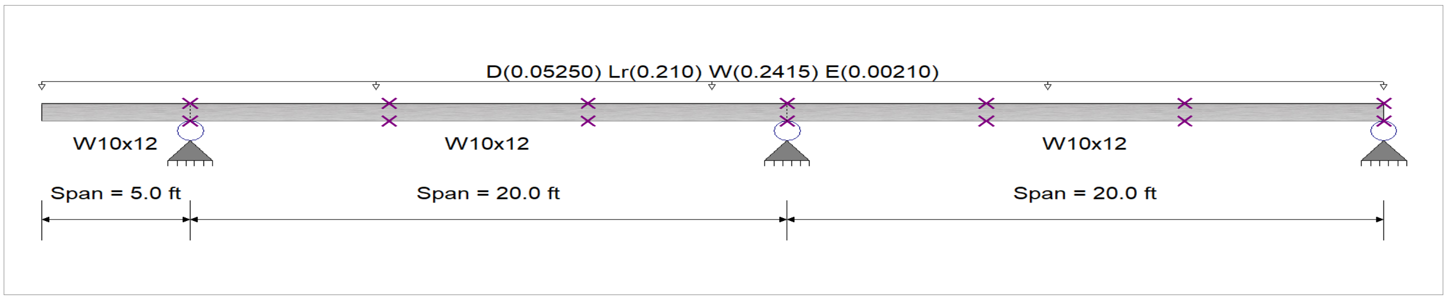
Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined Beam-by-Beam	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

Span # 1, Defined Brace Locations, First Brace at ft, Second Brace at ft, Third Brace at ft
Span # 2, Braced @ 1/3 Points
Span # 3, Braced @ 1/3 Points



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : D = 0.0050, Lr = 0.020, W = 0.0230, E = 0.00020 ksf, Tributary Width = 10.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.497** : 1	Maximum Shear Stress Ratio =	**0.109** : 1
Section used for this span	**W10x12**	Section used for this span	**W10x12**
Ma : Applied	15.500 k-ft	Va : Applied	4.082 k
Mn / Omega : Allowable	31.207 k-ft	Vn/Omega : Allowable	37.506 k
Load Combination	+D+0.750Lr+0.450W	Load Combination	+D+0.750Lr+0.450W
		Location of maximum on span	20.000 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.252 in	Ratio = 953	>=180.	Span: 3 : W Only
Max Upward Transient Deflection	-0.104 in	Ratio = 1,149	>=180.	Span: 3 : W Only
Max Downward Total Deflection	0.344 in	Ratio = 697	>=180	Span: 3 : +D+0.750Lr+0.450W
Max Upward Total Deflection	-0.143 in	Ratio = 840	>=180	Span: 3 : +D+0.750Lr+0.450W

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+0.750Lr+0.450W	-0.1429	0.000
+D+0.750Lr+0.450W	2	0.2288	8.667		0.0000	0.000
+D+0.750Lr+0.450W	3	0.3445	11.467		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions		4.392	7.957	2.532
Max Upward from Load Combinations		4.392	7.957	2.532
Max Upward from Load Cases		3.207	5.811	1.849
D Only		0.857	1.552	0.494
+D+Lr		3.646	6.605	2.102
+D+0.750Lr		2.948	5.342	1.700
+D+0.60W		2.781	5.039	1.603
+D+0.750Lr+0.450W		4.392	7.957	2.532
+D+0.450W		2.300	4.167	1.326
+0.60D+0.60W		2.438	4.418	1.406
+D+0.70E		0.876	1.587	0.505
+D+0.5250E		0.871	1.579	0.502

Steel Beam

Project File: 23-3108R01 28X110 ELMENDORF, TX MOBIL 2Di.ec6

DESCRIPTION: Purlin Beam 1 Strong Axis

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
+0.60D+0.70E		0.534	0.967	0.308
Lr Only		2.789	5.053	1.608
W Only		3.207	5.811	1.849
E Only		0.028	0.051	0.016

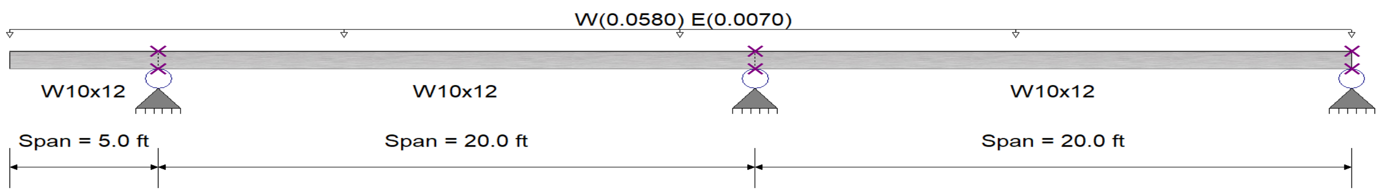
DESCRIPTION: Purlin Beam 1 Weak Axis

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.0580, E = 0.0070 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.380** : 1	Maximum Shear Stress Ratio =	**0.013** : 1
Section used for this span	**W10x12**	Section used for this span	**W10x12**
Ma : Applied	1.631 k-ft	Va : Applied	0.4296 k
Mn / Omega : Allowable	4.296 k-ft	Vn/Omega : Allowable	33.264 k
Load Combination	+0.60W	Load Combination	+0.60W
		Location of maximum on span	20.000 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	1.491 in	Ratio =	160	>=60.0	Span: 3 : W Only
Max Upward Transient Deflection	-0.619 in	Ratio =	193	>=60.0	Span: 3 : W Only
Max Downward Total Deflection	0.895 in	Ratio =	268	>=60.0	Span: 3 : +0.60W
Max Upward Total Deflection	-0.371 in	Ratio =	323	>=60.0	Span: 3 : +0.60W

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.6187	0.000
W Only	2	0.9906	8.667		0.0000	0.000
W Only	3	1.4912	11.467		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions		0.770	1.396	0.444
Max Upward from Load Combinations		0.462	0.837	0.266
Max Upward from Load Cases		0.770	1.396	0.444
+0.60W		0.462	0.837	0.266
+0.450W		0.347	0.628	0.200
E Only * 0.70		0.065	0.118	0.038
E Only * 0.5250		0.049	0.088	0.028
W Only		0.770	1.396	0.444
E Only		0.093	0.168	0.054

Steel Beam

Project File: 23-3108R01 28X110 ELMENDORF, TX MOBIL 2Di.ec6

LIC# : KW-06014602, Build:20.23.07.20 Jimco Sales & Mfg

DESCRIPTION: Purlin Beam 2 Strong Axis

CODE REFERENCES

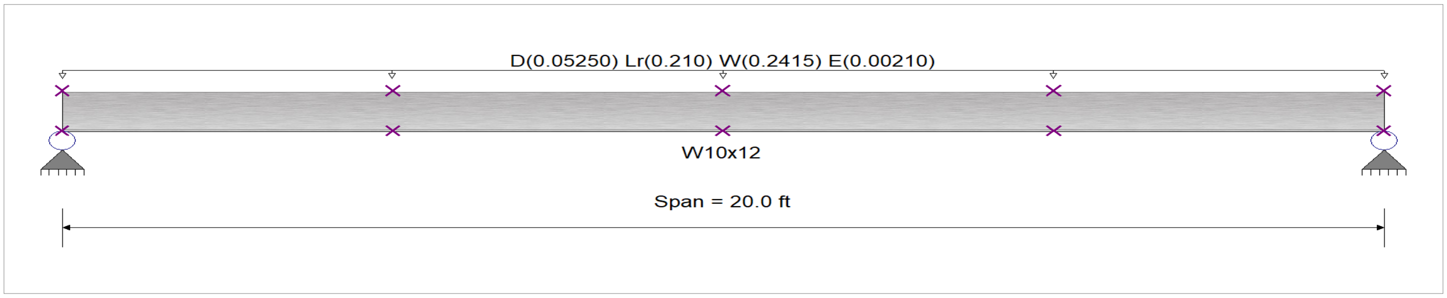
Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined Beam-by-Beam	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

Span # 1, Braced @ 1/4 Points



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : D = 0.0050, Lr = 0.020, W = 0.0230, E = 0.00020 ksf, Tributary Width = 10.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.596** : 1	Maximum Shear Stress Ratio =	**0.088** : 1
Section used for this span	**W10x12**	Section used for this span	**W10x12**
Ma : Applied	16.534 k-ft	Va : Applied	3.307 k
Mn / Omega : Allowable	27.759 k-ft	Vn/Omega : Allowable	37.506 k
Load Combination	+D+0.750Lr+0.450W	Load Combination	+D+0.750Lr+0.450W
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.560 in	Ratio = 428	>=180.	Span: 1 : W Only
Max Upward Transient Deflection	0 in	Ratio = 0	<180.0	n/a
Max Downward Total Deflection	0.766 in	Ratio = 313	>=180	Span: 1 : +D+0.750Lr+0.450W
Max Upward Total Deflection	0 in	Ratio = 0	<180	n/a

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.450W	1	0.7665	10.057		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.307	3.307
Max Upward from Load Combinations	3.307	3.307
Max Upward from Load Cases	2.415	2.415
D Only	0.645	0.645
+D+Lr	2.745	2.745
+D+0.750Lr	2.220	2.220
+D+0.60W	2.094	2.094
+D+0.750Lr+0.450W	3.307	3.307
+D+0.450W	1.732	1.732
+0.60D+0.60W	1.836	1.836
+D+0.70E	0.660	0.660
+D+0.5250E	0.656	0.656
+0.60D+0.70E	0.402	0.402
Lr Only	2.100	2.100
W Only	2.415	2.415
E Only	0.021	0.021

Steel Beam

Project File: 23-3108R01 28X110 ELMENDORF, TX MOBIL 2Di.ec6

LIC# : KW-06014602, Build:20.23.08.30 Jimco Sales & Mfg

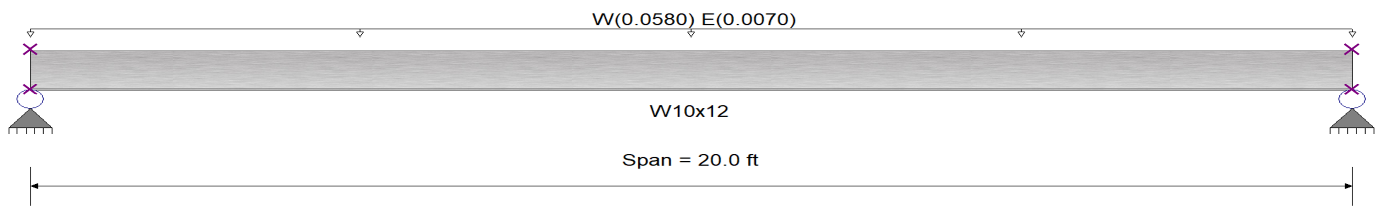
DESCRIPTION: Purlin Beam 2 Weak Axis

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.0580, E = 0.0070 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.405 : 1	Maximum Shear Stress Ratio =	0.010 : 1
Section used for this span	W10x12	Section used for this span	W10x12
Ma : Applied	1.740 k-ft	Va : Applied	0.3480 k
Mn / Omega : Allowable	4.296 k-ft	Vn/Omega : Allowable	33.264 k
Load Combination	+0.60W	Load Combination	+0.60W
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	3.291 in	Ratio = 72	>=60.0	Span: 1 : W Only
Max Upward Transient Deflection	0 in	Ratio = 0	<60.0	n/a
Max Downward Total Deflection	1.991 in	Ratio = 121	>=60.0	Span: 1 : +0.60W
Max Upward Total Deflection	0 in	Ratio = 0	<60.0	n/a

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	3.3179	10.057		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2	
Max Upward from all Load Conditions	0.580	0.580	0.084
Max Upward from Load Combinations	0.348	0.348	0.084
Max Upward from Load Cases	0.580	0.580	0.084
Max Downward from all Load Conditions (Resis			0.084
Max Downward from Load Combinations (Resi:			0.084
Max Downward from Load Cases (Resisting Up			0.084
			0.084
+0.60W	0.348	0.348	0.084
+0.450W	0.261	0.261	0.084
E Only * 0.70	0.049	0.049	0.084
E Only * 0.5250	0.037	0.037	0.084
W Only	0.580	0.580	0.084
E Only	0.070	0.070	0.084

Job No: 23-3108R01

Lateral Bracing Calculations for beams - AISC 360 Appendix 6.3

Angle Bridging for Purlins

The angle bridging spans each purlin and doesn't connect to a seperate fixed point. Therefore the bridging is considered a relative brace.

Section 1.a Relative Bracing

Purlin Beam		W10X12			**Simple Span**			**Cantilever**	
Required flexural Strength of Purlin				Mr=	16.6	k-ft (ASD)	Mr=	4.2	k-ft (ASD)
	Cd=	1							
	ho=	9.66	in						
	Ω=	2							
Required Bridging Axial Strength				Prb=	0.165	kips	Prb=	0.041	kips
Required Bridging Axial Stiffness				βrb=	2.07	k/in (ASD)	βrb=	2.58	k/in (ASD)

Bridging Angle		**L2x2x3/16**		**Deck and Deck Clip**	**Deck Capacity - 40ksi -20ga**		
Unbraced Length of Purlin				(Only used as bridging for cantilever)			
Angle Spacing	Lb=	6.67	ft	Deck Spacing	Lb=	1.33	ft
	K=	1			K=	1	
Max Purlin Spacing	L=	10.50	ft	Max Purlin Spacing	L=	10.50	ft
	r=	0.389	in		r=	0.761	in
	b=	2	in		b=	16	in
	t=	0.188	in		t=	0.0359	in
	A=	0.722	in^2		A=	0.855	in^2
	E=	29000	ksi		E=	29000	ksi
	Fy=	36	ksi		Fy=	40	ksi

Limiting Width to Thickness Ratio λr= 12.77

b/t= 10.64 < 12.77 | Clip Shear Strength 282 lbs

Angle is Not Slender for Compression | (See Decking Sheet)

KL/r= 323.9 > 133.68

Fe= 2.73 ksi

Fcr= 2.39 ksi

Ω= 1.67

Pn/Ω=	1.03 kips	>	0.165 kips		0.282 kips	>	0.041 kips	
	Bridging Angle has adequate strength.				Clip has adequate strength.			
AE/L=	166.17 k/in	>	2.07 k/in		196.79 k/in	>	2.58 k/in	
	Bridging Angle has adequate stiffness.				Decking has adequate stiffness.			

Job No: 23-3108R01

Jimco Standard - Max Deflection: l/180 Span; l/90 Cantilever

Header Beam:

Use:	**W14X22**	Moment check:	OK	Header Count:	6	Bwgt=	22 lbs
fy=50ksi		Weak Axis Check:	OK	Ix=	199 in^4	Bh=	14 in

Max Trib Width =	20.00 ft
Left Cantilever =	9.00 ft
Right Cantilever =	9.00 ft
Canopy Width =	28.00 ft
Max Unbraced Length =	10.50 ft

Simple Span

1	Bay(s) @	10.00 ft
0	Bay(s) @	0.00 ft
0	Bay(s) @	0.00 ft

Gravity Loads

DL =	0.21 klf	Wdn =	0.47 klf
Cant Lr/S =	0.40 klf	Wup =	-0.43 klf
Span Lr/S =	0.40 klf	Ev =	0.01 klf

Uniform Loads

Wttl Cantilever =	0.717 klf
Wttl Simple Span =	0.72 klf

Vertical Moments

Cantilever Moment:	M_r =	29.04 ft-k	E_v=	0.0006 klf
Simple Span Moment:	M_r =	8.96 ft-k		
Weak Axis Moment:	M_{rh} =	1.14 ft-k		

Lateral Loads

W_h =	0.029 klf
E_h=	0.010 klf

Beam Slenderness

Flange	Web
Comp	Comp

Beam Flexural Design

	Mp	Mltb	Mn (Flange)	Mflb	Mn	Ma
Strong:	138.3	83.5		--	83.5 ft-k	50.0 ft-k
Weak:	18.3	18.666667			18.3 ft-k	11.0 ft-k

ASCE Load Combinations (ASD)

ASCE 7-05? No

Wind Factor 0.6

	STRONG	**WEAK**	**COMBINED**	
1. D	0.127	0.000	0.13	
2. D+L	0.127	0.000	0.13	
3. D+(Lr or S)	0.502	0.000	0.50	
4. D + 0.75L + 0.75(Lr or S)	0.408	0.000	0.41	
5a. D + 0.6*W	0.386	0.064	0.45	
5b. D + 0.7E	0.130	0.026	0.16	
6a. D + 0.75L +0.75(0.6W) + 0.75(Lr or S)	0.603	0.048	0.65	**Controls**
6b. D + 0.75L +0.75(0.7E) + 0.75(Lr or S)	0.410	0.019	0.43	
7. 0.6D + 0.6*W	0.335	0.064	0.40	
8. 0.6D + 0.7E	0.079	0.026	0.10	

Steel Beam

Project File: 23-3108R01 28X110 ELMENDORF, TX MOBIL 2Di.ec6

LIC# : KW-06014602, Build:20.23.08.30 Jimco Sales & Mfg

DESCRIPTION: Header Beam Strong Axis

CODE REFERENCES

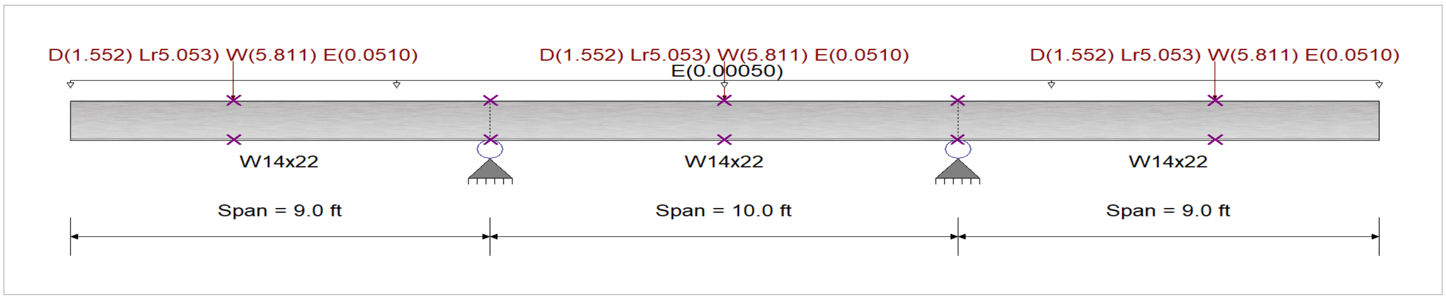
Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam bracing is defined Beam-by-Beam	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		

Unbraced Lengths

Span # 1, Defined Brace Locations, First Brace at 3.50 ft, Second Brace at ft, Third Brace at ft
Span # 2, Defined Brace Locations, First Brace at 5.0 ft, Second Brace at ft, Third Brace at ft
Span # 3, Defined Brace Locations, First Brace at 5.50 ft, Second Brace at ft, Third Brace at ft



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : E = 0.00050 k/ft

Load(s) for Span Number 1
Point Load : D = 1.552, Lr = 5.053, W = 5.811, E = 0.0510 k @ 3.50 ft, (From Purlin)

Load(s) for Span Number 2
Point Load : D = 1.552, Lr = 5.053, W = 5.811, E = 0.0510 k @ 5.0 ft, (From Purlin)

Load(s) for Span Number 3
Point Load : D = 1.552, Lr = 5.053, W = 5.811, E = 0.0510 k @ 5.50 ft, (From Purlin)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.603** : 1	Maximum Shear Stress Ratio =	**0.129** : 1
Section used for this span	**W14x22**	Section used for this span	**W14x22**
Ma : Applied	44.653 k-ft	Va : Applied	8.155 k
Mn / Omega : Allowable	74.053 k-ft	Vn/Omega : Allowable	63.020 k
Load Combination	+D+0.750Lr+0.450W	Load Combination	+D+0.750Lr+0.450W
		Location of maximum on span	10.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.522 in	Ratio = 413	>=180.	Span: 3 : W Only
Max Upward Transient Deflection	-0.085 in	Ratio = 1,418	>=180.	Span: 3 : W Only
Max Downward Total Deflection	0.730 in	Ratio = 296	>=180.	Span: 3 : +D+0.750Lr+0.450W
Max Upward Total Deflection	-0.118 in	Ratio = 1014	>=180.	Span: 3 : +D+0.750Lr+0.450W

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.450W	1	0.7299	0.000		0.0000	0.000
	2	0.0000	0.000	+D+0.750Lr+0.450W	-0.1184	5.000
+D+0.750Lr+0.450W	3	0.7277	9.000		0.0000	5.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions		12.243	12.243	

Steel Beam

Project File: 23-3108R01 28X110 ELMENDORF, TX MOBIL 2Di.ec6

LIC# : KW-06014602, Build:20.23.08.30 Jimco Sales & Mfg

DESCRIPTION: Header Beam Strong Axis

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from Load Combinations		12.243	12.243	
Max Upward from Load Cases		8.717	8.717	
D Only		2.636	2.636	
+D+Lr		10.216	10.216	
+D+0.750Lr		8.321	8.321	
+D+0.60W		7.866	7.866	
+D+0.750Lr+0.450W		12.243	12.243	
+D+0.450W		6.558	6.558	
+0.60D+0.60W		6.812	6.812	
+D+0.70E		2.694	2.694	
+D+0.5250E		2.680	2.680	
+0.60D+0.70E		1.640	1.640	
Lr Only		7.580	7.580	
W Only		8.717	8.717	
E Only		0.084	0.084	

Steel Beam

Project File: 23-3108R01 28X110 ELMENDORF, TX MOBIL 2Di.ec6

LIC# : KW-06014602, Build:20.23.07.20 Jimco Sales & Mfg (c) ENERCALC INC 1983-2023

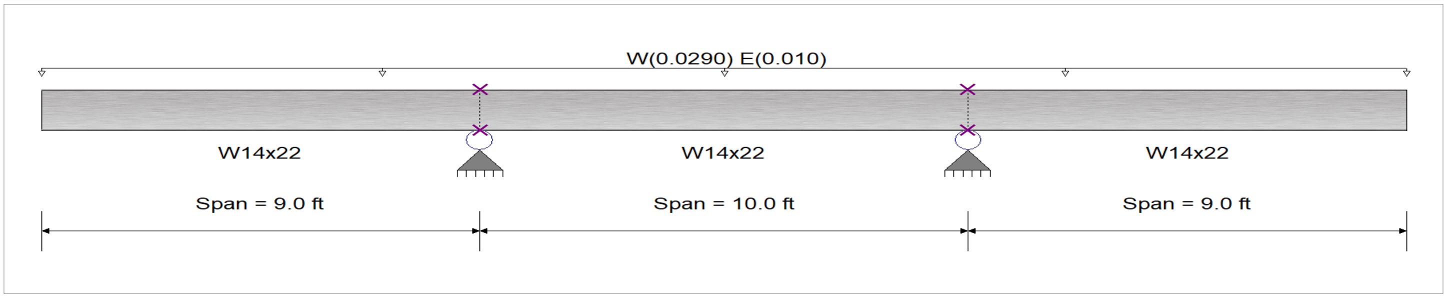
DESCRIPTION: Header Beam Weak Axis

CODE REFERENCES

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Completely Unbraced	E: Modulus :	29,000.0 ksi
Bending Axis :	Minor Axis Bending		



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Loads on all spans...
Uniform Load on ALL spans : W = 0.0290, E = 0.010 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.064** : 1	Maximum Shear Stress Ratio =	**0.002** : 1
Section used for this span	**W14x22**	Section used for this span	**W14x22**
Ma : Applied	0.705 k-ft	Va : Applied	0.1566 k
Mn / Omega : Allowable	10.953 k-ft	Vn/Omega : Allowable	67.0 k
Load Combination	+0.60W	Load Combination	+0.60W
		Location of maximum on span	10.000 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.561 in	Ratio = 385	>=60.0	Span: 3 : W Only
Max Upward Transient Deflection	-0.094 in	Ratio = 1,274	>=60.0	Span: 3 : W Only
Max Downward Total Deflection	0.336 in	Ratio = 642	>=60.0	Span: 3 : +0.60W
Max Upward Total Deflection	-0.056 in	Ratio = 2124	>=60.0	Span: 3 : +0.60W

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
W Only	1	0.5607	0.000		0.0000	0.000
	2	0.0000	0.000	W Only	-0.0942	5.000
W Only	3	0.5589	9.000		0.0000	5.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions		0.406	0.406	
Max Upward from Load Combinations		0.244	0.244	
Max Upward from Load Cases		0.406	0.406	
+0.60W		0.244	0.244	
+0.450W		0.183	0.183	
E Only * 0.70		0.098	0.098	
E Only * 0.5250		0.074	0.074	
W Only		0.406	0.406	
E Only		0.140	0.140	

Job No: 23-3108R01

Ponding:

Ponding Check: (AISC 15th - App X 2.1)

Lp=	10.00 ft		Wp=	W14X22				
Ls=	20.00 ft		Ws=	W10X12				
S=	10.50 ft							
ip=	199.00	in^4						
is=	53.80	in^4			Metal Works, LLC Deck		Deck Capacity - 40ksi -20ga	
id=	0.30	in^4/ft	<		Ix Pos =	0.802	in^4	**OK**
					Ix Neg =	0.411	in^4	

Cs=	0.10				
Cp=	0.00				
Cp+0.9Cs=		0.09	<	0.25	**OK**

Job No: 23-3108R01

Column:	**HSS**	**SQR**	Material - A500 Gr. C		
Column Size:	**HSS12X12X1/4**		Fy = 50ksi		
	Ix=	248 in^4			

W_h Lateral @ Columns =	1.75 kip	Length =	18.49 ft
E_h Lateral @ Columns =	0.35 kip	Z =	47.60 in^3
Total Effective Col Height =	20.99 ft	Col Weight =	39.43 plf
Max Column Vertical Trib =	280.00 SqFt	k=	2.10
Column Trib Width =	14.00 ft	kl/r=	97.27 in
Column Trib Length =	20.00 ft		
Column Lateral Trib =	10.00 ft/col		
	2.33 ft/col		

Column Properties:		**Cutout Properties:**	
D=	12.00 in	D=	5.50 in
t=	0.23 in	t=	0.23 in
A=	10.8 in^2	A=	0.84 in^2
I=	248 in^4	I=	46.14 in^3
Z=	47.60 in^3		

Column Properties with Cutout:

A=	9.5185 in^2	**Max allowable Stress Ratio =**	**0.84**
I=	201.86 in^4	An Increase of 1.2 is allowed for members designed using overstrength as per ASCE 7-16 12.4.3.3	
S=	33.64 in^3		
R=	4.61 in		
Z=	39.91 in^3	**Max allowable stress Ratio=**	**1.01**

Seismic Load Combination w/Overstrength (Section 12.4.5)

Ω = 1.25

Sds = 0.130 g

	D	**S**	**E**
5. (1.0+0.14Sds)D+0.7ΩQe	3.43 k	0	0.31 kips
6b. (1.0+0.105Sds)D+0.525ΩQe+0.75S	3.41 k	0.0	0.23 kips
8. (0.6-0.14Sds)D+0.7ΩQe	1.96 k	0	0.31 kips

ASCE 7-16 12.2.5.2 Axial Strength Requirement

Prc = 3.43 kips < 0.15*Pc = 23.36 kips OK

Job No: 23-3108R01

Columns:

HSS12X12X1/4

Cantilevered Column

ASD Overstrength Factor Included

Wv=	1.05	k/col		**Table B4.1**			
0.7pQeΩ=	0.24	k/col		Typ	Shape	Flanges	Webs
Ω=	1.25			HSS	SQR	Slender	Comp
	Bending				**Compression**		
Mpx	Mflbx	Mwlbx	Max	Fe (ksi)	Qa	Fcr (ksi)	Pa (k)
119.00	86.13	--	86.13	30.25	0.89	14.42	155.76

ASD Load Combination: Multiple Rows of Columns

D+(Lr or S)

P =	14.0 x	20.0 x	0.0287	=	8 k	
Pa =	155.76 k					
	P/Pa =	0.05	<=	0.84	OK	

D+.6W Controls

P =	14.0 x	20.0 x	0.024	=	7 k				
M =	10.0 x	3.0 x	0.035 x	18.49	=	19.40 ft-k			
Pa =	156 k		Ma =	86.13 ft-k		Pdelta (ft-k) =			0.30
	AISC H1.1	0.02	+	0.23	= 0.25	< =	0.84	OK	

D+0.75(.6W+(Lr orS))

P=	14.0 x	20.0 x	0.04	=	10 k			
M =	19.4 x	0.75	=	14.55 ft-k				
Pa =	156 k		Ma =	86.13 ft-k				
	AISC H1.1	0.03	+	0.17	= 0.20	< =	0.84	OK

D+.7E

P =	14.0 x	20.0 x	0.011	=	3 k			
M =	0.2 x	18.49	=	4.46 ft-k				
Pa =	156 k		Ma =	86.13 ft-k				
	AISC H1.1	0.01	+	0.05	= 0.06	< =	1.01	OK

D+.75(.7E+(Lr orS))

P =	14.0 x	20.0 x	0.02	=	6 k			
M =	0.2 x	18.49	=	3.34 ft-k				
Pa =	156 k		Ma =	86.13 ft-k				
	AISC H1.1	0.02	+	0.04	= 0.06	< =	1.01	OK

Steel Column

DESCRIPTION: Column

Code References

Calculations per AISC 360-16, IBC 2021, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name : **HSS12x12x1/4**

Analysis Method :	Allowable Strength	
Steel Stress Grade		
Fy : Steel Yield	50.0	ksi
E : Elastic Bending Modulus	29,000.0	ksi

Overall Column Height	18.490 ft
Top & Bottom Fixity	Top Free, Bottom Fixed
Brace condition :	

Unbraced Length for buckling ABOUT X-X Axis = 18.490 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 18.490 ft, K = 2.1

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 729.06 lbs * Dead Load Factor
AXIAL LOADS . . .
Header Beam: Axial Load at 18.490 ft, Xecc = 1.0 in, Yecc = 1.0 in, D = 2.636, LR = 6.974, W = 8.717, E = 0.0840 k
BENDING LOADS . . .
Lat. Point Load at 18.490 ft creating Mx-x, W = 1.749, E = 0.3450 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.2655**	: 1
	Load Combination	+D+0.60W	
	Location of max.above base	0.0	ft
	At maximum location values are . . .		
	Pa : Axial	8.595	k
	Pn / Omega : Allowabl	159.938	k
	Ma-x : Applied	-20.059	k-ft
	Mn-x / Omega : Allowable	86.793	k-ft
	Ma-y : Applied	-0.6555	k-ft
	Mn-y / Omega : Allowable	86.793	k-ft
PASS	Maximum Shear Stress Rati	**0.01109**	: 1
	Load Combination	+D+0.60W	
	Location of max.above base	0.0	ft
	At maximum location values are . . .		
	Va : Applied	1.049	k
	Vn / Omega : Allowable	94.604	k

Maximum Load Reactions . .

Top along X-X	0.0 k
Bottom along X-X	0.0 k
Top along Y-Y	0.0 k
Bottom along Y-Y	1.749 k

Maximum Load Deflections . . .

Along Y-Y 0.9106 in at 18.490 ft above base
for load combination : W Only

Along X-X 0.04008 in at 18.490 ft above base
for load combination : +D+0.750Lr+0.450W

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only	3.365					-0.220		-0.220	
+D+Lr	10.339					-0.801		-0.801	
+D+0.750Lr	8.596					-0.656		-0.656	
+D+0.60W	8.595			1.049		-20.059		-0.656	
+D+0.750Lr+0.450W	12.518			0.787		-15.535		-0.982	
+D+0.450W	7.288			0.787		-15.099		-0.547	
+0.60D+0.60W	7.249			1.049		-19.971		-0.568	
+D+0.70E	3.424			0.242		-4.690		-0.225	
+D-0.70E	3.306			-0.242		4.251		-0.215	
+D+0.5250E	3.409			0.181		-3.572		-0.223	
+D-0.5250E	3.321			-0.181		3.133		-0.216	
+0.60D+0.70E	2.078			0.242		-4.602		-0.137	
+0.60D-0.70E	1.960			-0.242		4.338		-0.127	
Lr Only	6.974					-0.581		-0.581	
W Only	8.717			1.749		-33.065		-0.726	
E Only	0.084			0.345		-6.386		-0.007	
E Only * -1.0	-0.084			-0.345		6.386		0.007	

Steel Column

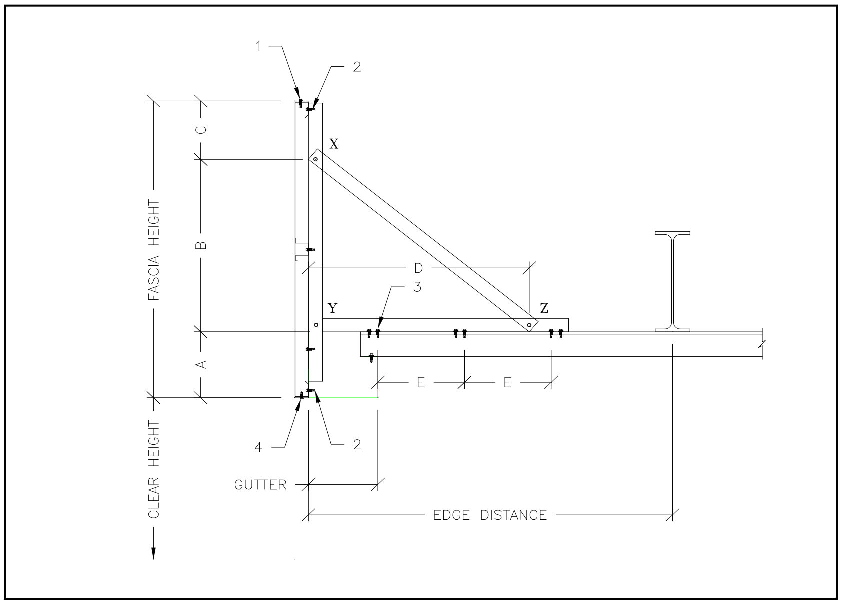
DESCRIPTION: Column

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top (k)	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top (k-ft)	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	12.518			0.787		-15.535		-0.982	
"	Minimum	-0.084			-0.345		6.386		0.007	
Reaction, X-X Axis Base	Maximum	3.365					-0.220		-0.220	
"	Minimum	3.365					-0.220		-0.220	
Reaction, Y-Y Axis Base	Maximum	8.717			1.749		-33.065		-0.726	
"	Minimum	-0.084			-0.345		0.007		6.386	
Reaction, X-X Axis Top	Maximum	3.365					-0.220		-0.220	
"	Minimum	3.365					-0.220		-0.220	
Reaction, Y-Y Axis Top	Maximum	3.365					-0.220		-0.220	
"	Minimum	3.365					-0.220		-0.220	
Moment, X-X Axis Base	Maximum	-0.084	6.386		-0.345		6.386		0.007	
"	Minimum	8.717	-33.065		1.749		-33.065		-0.726	
Moment, Y-Y Axis Base	Maximum	-0.084			-0.345		-0.220		0.007	
"	Minimum	12.518			0.787		-0.220		-0.982	
Moment, X-X Axis Top	Maximum	3.365					-0.220		-0.220	
"	Minimum	3.365					-0.220		-0.220	
Moment, Y-Y Axis Top	Maximum	3.365					-0.220		-0.220	
"	Minimum	3.365					-0.220		-0.220	

Fascia Frame Check

Clear Height:	17.00 ft	A=	8.00 in	E=	10.00 in	"@ Sides"
Fascia Height:	3.00 ft	B=	22.00 in	E=	16.00 in	"@ Ends"
Frame Spacing:	48 in o.c.	B1=	0.00 in			
Edge Distance:	3.50 ft	C=	6.00 in			
Gutter Width:	8.00 in	D=	24.44 in			



Forces: (Due to the light weight of the fasica, wind shall control over Seismic.)

Fascia weight (incl frames) = 4 psf

p_p = 34.97 psf

Fascia Frame Design:

C Section: **20ga x 1 3/4" x 1 5/8" C** M/Ω= 0.89 k-in P/Ω= 1.82 k

Frames @ 4.00 ft O.C.

Single or Double Brace?

Vertical Member XY: **Brace XZ:** Length = 27.28 in

M = 0.4 k-in < 0.89 k-in OK P = 0.1 k < 1.82 k OK

Screw Capacity:

#8 Tek = 174 lb Shear --------- ESR-2196

#12 Tek = 190 lb Tension (Lapped 20ga)

#12 Tek = 200 lb Shear (Lapped 20ga)

#12 Tek = 315 lb Shear (20ga to 1/4") ESR-1976

#1/4 Tek = 430 lb Shear (Lapped 20ga)

#1/4 Tek = 115 lb Tension (Pullout) 20ga

Screw:		Trib. Area	Fa		Fc	
1	#1/4 Tek @ 30in o.c. Shear:	1.88 SqFt	26.2 lb	<	215 lb	OK
2	#1/4 Tek @ ea. frame (T&B) Tensile:	1.88 SqFt	26.2 lb	<	115 lb	OK
	Shear:	3.75 SqFt	15.0 lb	<	215 lb	OK
3	(6) #12 Tek @ ea. frame Shear:	12.00 SqFt	251.8 lb	<	1200 lb	OK
	Tensile:	4.00 ft	28.7 lb	<	190 lb	OK
4	#8 ex @ 30in o.c.	1.88 SqFt	26.2 lb	<	174 lb	OK

Job No: 23-3108R01

Deck Spans:

Lc =	3.50 ft	Ls =	10.50 ft	**Design Loads:**	
EWAc =	4.67 Sqft	EWAs =	36.75 Sqft	D -	5.0 psf
a =	3.00 ft	a^2 =	9.00 ft	Lr OR S -	20.0 psf
				W-q_z -	22.9 psf

Materials Specifications: Pan Deck by Metal Works; 3"x16" 20 Ga

Deck Capacity - 40ksi -20ga

Width =	16.00 in
Mapos =	7.93 k-in
Maneg =	-6.51 k-in

Check Deck at 10'-6" Span	Span Length-	10.50 ft
For Gravity (Downward) Check	Cn	1.20
Zone 3 Net Pressure Coefficient Figure 30.8-1	$W_{total\ down}$	30.5 psf
ASD Total Load = D+.75(Lr or S)+.75*0.6*W =	Mu	4.27 k-in
Moment Required = w_u*l^2/8 =	Mn/Ω > Mu	OK
For Uplift Check	Cn	-1.70
Zone 3 Net Pressure Coefficient Figure 30.8-1	$W_{total\ uplift}$	-16.9 psf
ASD total Load = 0.6D+0.6*W =	Mu	-3.72 k-in
Moment Required = w_u*l^2/8 =	Mn/Ω > Mu	OK

Check Deck at 3'-6" Cantilever		
Zone 3 Net Pressure Coefficient Figure 30.8-1	L	3.50 ft
For Gravity (Downward) Check		
ASD Total Load = D+.75(Lr or S)+.75*0.6*W =	Cn	2.40
Moment Required = w_u*l^2/2 =	$W_{total\ down}$	41.1 psf
	Mu	-4.00 k-in
For Uplift Check	Mn/Ω > Mu	OK
Zone 3 Net Pressure Coefficient Figure 30.8-1		
ASD total Load = 0.6D+0.6*W =	Cn	-3.30
Moment Required = w_u*l^2/2 =	$W_{total\ uplift}$	-35.6 psf
	Mu	3.49 k-in
	Mn/Ω > Mu	OK

Roof Wind Design Pressures (psf) C&C Figure 30.8-1

Z3		Z2		Z1	
23	-21	23	-21	23	-21

Metal Works Deck Clip:

Beam/Deck Clip Capacity Checks:

Deck clip yield capacity	**941 lbs**
Deck Allowable Capacity Ω=1.67	**563 lbs**
Single or Double Clamped	**Single**

Seismic Load Calculations:

Total Dead Load		5.0 psf	
Seismic Response Coefficient	Cs	0.10	
Horizontal Shear on Deck	V = Cs*W	0.5 psf	
Deck Clip Spacing	L1	1.33 ft	16 in o.c.
Deck Span	L2	10.50 ft	

Clip Shear Resistance Calculation:

Assume lateral capacity of clip is 20% of clamp load.

Seismic Load Transfer to Bolt = V*L1*L2 =	5.1 lb	<	282 lbs	OK
Wind Transfer to Bolt = Pp*Hf*L1/#purlins	46.5 lb	<	282 lbs	OK
Uplift Check Wu = Wtotal * L1*L2	137.8 lb	<	563 lbs	OK
Gravity Load Wu = Wtotal * L1*L2	206.6 lb	<	563 lbs	OK

Job No: 23-3108R01

Column Cap Plate

Top Plate Properties:				**Bolt Design**			
Fy =	50ksi	Uplift -	-2.42 k	db:	**3/4 Ø**	**A307 BOLTS**	
Plate Thickness =	**0.75 in**	Mreq -	0.0 ft-k	# of Bolts:	**4**	Total	
Plate Width =	**12.0 in**	Applied Safety Factor=	1.50	V/Bolt=	1.57	k =	0.39 k/bolt
Plate Length =	**19.0 in**	Shear -	1.05 k	T/Bolt=	-3.62	k =	0.12 k/bolt
				G.F. (Shear) =	5.96	k =	OK
				G.F. (Tension) =	9.94	k =	OK
				Combined UC=	0.00	< 1	OK

Column:
HSS12X12X1/4

Col Trib =	280.00 SqFt	Column Width =	12 in
Header:	W14X22	Bolt Spacing =	3 in
Purlin:	W10X12	Distance from Col =	2 in
Depth:	9.9 in	Bolt Gage =	16 in

Weld Design (Cap Plate to Column)

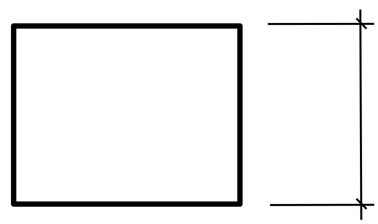
Column Typ =	HSS	A =	9.00 in^2
Column Width =	12.0 in	S =	1.13 in^3
Column Length =	12.0 in	Z =	1.69 in^3

Combined Properties

d1= 12 in
b= 12 in

L weld= 48.0 in
S weld= 192 in^2

d1



b

T Weld Strength Req'd =

$$\frac{4 \text{ k-in}}{48 \text{ in}^2} = \mathbf{0.08\ k/in}$$

S Weld Strength Req'd =

$$\frac{1.57 \text{ k}}{48.0 \text{ in}^4} = \mathbf{0.03\ k/in}$$

Combined = **0.09 k/in**

USE: **2 /16" Fillet Weld**
min. 1/8" for weld per J2.4 AISC
G.F. = 1.86 ASD OK

Cap Plate - PJP = 2 /16"
min. 1/8" for weld per J2.3 AISC

Job No: 23-3108R01

Anchor Bolt (ASTM F1554, Gr. 55)

No of Anchor Blots =	4			
Mean Anchor Bolt Spacing =	16 in			
# of Bolts in Tension =	2		# of Bolts in Shear =	4
Concrete Fc=	3000 Psi			
Trod @ Drilled Pier =	5.16 k/rod		Trod @ Spread Ftg =	5.68 k/rod
Vrod =	0.39 k/rod			
Nu =	10.32 k		Nu =	11.35 k
Vu=	1.57 k			

Anchor Bolts Capacities Futa = 75ksi

Area of Anchor Bolt	A_{se}	0.61 sqin		
Steel Strength of Anchor in Tension ACI D.5.1	ΦN_{sa} =	Φ *Ase*Futa	Φ =	0.75
	ΦN_{sa}	$\boxed{34.07}$ k/bolt		
Steel Strength of Anchor in Shear ACI D.6.1	ΦVs = Φ*.60 *Ase*Futa		Φ =	0.65
	ΦVs =	$\boxed{17.72}$ k/bolt		

For Shallow Spread

Anchor Bolt Diameter = **1.00 Dia x 36.00"** **hef=** 26.00 in

For DRILLED PIERS

Anchor Bolt Diameter = **1.00 Dia x 36.00"** **hef=** 26.00 in

Job No: 23-3108R01

Column: **HSS12X12X1/4**

Base Plate Design:	***CONTROLS***				
Seismic Base Shear =	3.31 k	Size:	N =	**22.00 in**	n' = 3.00
W lateral @ Column =	1.75 k		B =	**22.00 in**	m = 5.30
E lateral @ Column =	0.35 k		Col d =	12.00 in	X = 0.01
Ω =	1.25		Col bf =	12.00 in	L = 0.10

Load Combination:

Base Plate Reactions

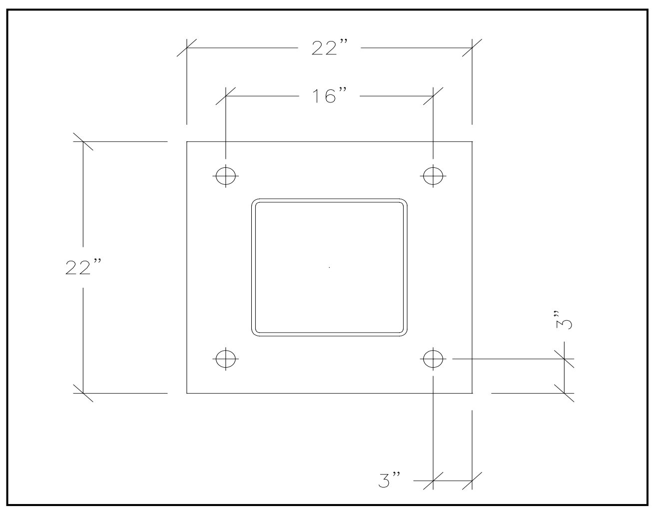
D+.6W		**D+0.75(.6W+(Lr orS))**		**Base Plate Properties**	
P=	6.82 k	P=	10.04 k	Base Plate Thickness =	1.00
M=	22.02 k-ft	M=	16.52 k-ft	Fyp=	50ksi
ecc=	38.74 in	e=	19.75 in	Seismic Shear =	0.44 k
ecrit=	10.85	ecrit=	10.78 in	Seimic Moment =	5.66 k-ft
Y =	0.76 in	Y =	0.66 in	Wind M @ Base =	22.55 k-ft
T=	10.30 k	T=	4.88 k		
Trod =	5.15 k	Trod =	2.44 k		
tpmin=	0.71 in	tpmin=	0.67 in		

.6(D+W)		**D+(Lr or S)**	
P=	2.09 k	P=	8.04 k
Trod=	5.68 k	l=	11.40 in
tpmin=	0.37 in	tpmin=	0.38 in

Base Plate Thickness:

Use: **1.00 x 22 x 22**

tp min = 0.71 in



Sweld =	192.00 sqin
Lweld =	48.00 in
S.F. =	1.5
M=	22.02 k-ft
V=	1.05 k
Pu=	-3.21 k

T Weld Strength Req'd = $\frac{4.82 \text{ k}}{48.00 \text{ in}} + \frac{396 \text{ k-in}}{192 \text{ sqin}}$ = **2.2 k/in**

S Weld Strength Req'd = $\frac{1.57 \text{ k}}{48.00 \text{ in}}$ = **0.03 k/in**

Combined Weld Strength Required = **2.2 k/in**

Column to Base Plate Weld:

min. 1/8" for weld per J2.4 AISC

Use: **3** **/16"** Min. fillet All Around per J2.4 AISC or CJP

ASD G.F. = 2.78 k-in > 2.17 k-in OK

Steel Base Plate

Project File: 23-3108R01 28X110 ELMENDORF, TX MOBIL 2Di.ec6

DESCRIPTION: Baseplate and Anchor Bolt

Code References

Calculations per AISC Design Guide # 1, IBC 2021, ASCE 7-16, AISC 360-16
Load Combination Set : ASCE 7-16

General Information

Material Properties

AISC Design Method	Allowable Strength Design			
Steel Plate Fy	= 50.0 ksi			
Concrete Support f'c	= 3.0 ksi	Ω c	: ASD Safety Facto	2.31
Assumed Bearing Area :	Bearing Area = P / Fp	Nominal Bearing Fp per J8		2.550 ksi

Column & Plate

Column Properties

Steel Section HSS12x12x1/4

Depth	12 in	Area	10.8 in^2
Width	12 in	Ixx	in^4
Flange Thickness	0.233 in	Iyy	in^4
Web Thicknes	in		

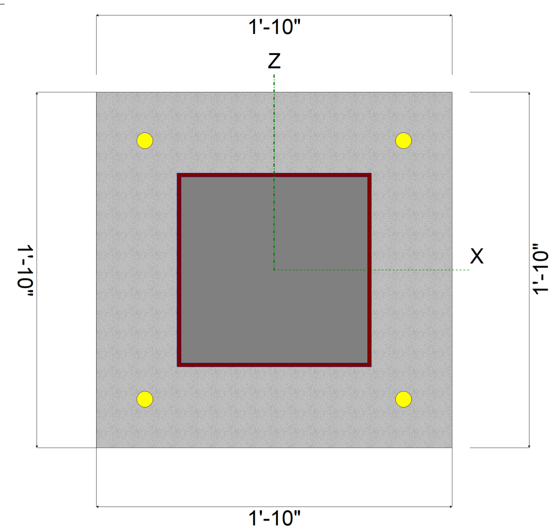
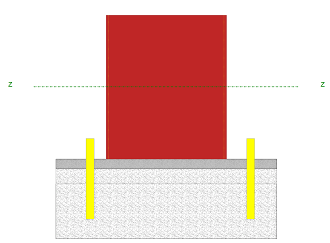
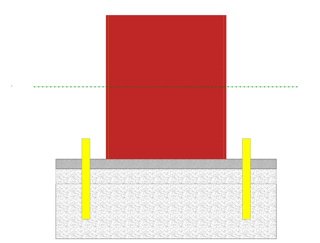


Plate Dimensions		**Support Dimensions**	
N : Length	22.0 in	Width along "X"	22.0 in
B : Width	22.0 in	Length along "Z'	22.0 in
Thickness	1.0 in		

Column assumed welded to base plate

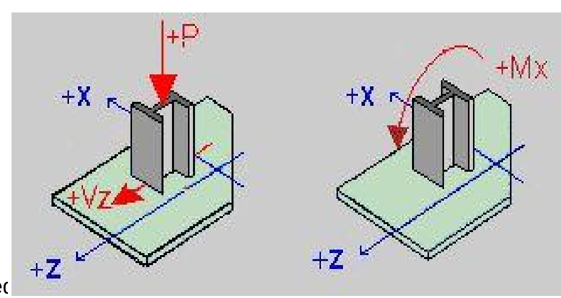




Applied Loads

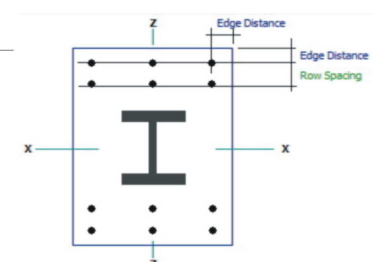
	P-Y	V-Z	M-X
D : Dead Load	3.365 k	k	k-ft
L : Live	k	k	k-ft
Lr : Roof Live	6.974 k	k	k-ft
S : Snow	k	k	k-ft
W : Wind	8.717 k	1.749 k	-33.065 k-ft
E : Earthquake	0.0840 k	0.3450 k	-6.386 k-ft
H : Lateral Earth	k	k	k-ft

" P " = Gravity load, "+" sign is downward
"+" Moments create higher soil pressure at +Z edge
"+" Shears push plate towards +Z edge



Anchor Bolts

Anchor Bolt or Rod Description 1		
Max of Tension or Pullout Capacity......	34.070	k
Shear Capacity....................................	17.720	k
Edge distance : bolt to plate.................	3.0	in
Number of Bolts in each Row...............	2	
Number of Bolt Rows.......................	1	



Steel Base Plate

Project File: 23-3108R01 28X110 ELMENDORF, TX MOBIL 2Di.ec6

LIC# : KW-06014602, Build:20.23.08.30 Jimco Sales & Mfg

DESCRIPTION: Baseplate and Anchor Bolt

GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method	**Allowable Strength Design**
Governing Load Combinat	**+D+0.60W**
Governing Load Case Typ	**Axial + Moment, L/2 < Eccentricity, Tensioı**
Governing STRESS RATI(	**1.0**
Design Plate Size	**1'-10" x 1'-10" x 1"**

Pa : Axial Load	7.249	k
Ma : Moment	19.839	k-ft

Ma : Max. Moment	3.644	k-in
fb : Max. Bending Stress	21.865	ksi
Fb : Allowable : Fy / Omega	29.940	ksi
Bending Stress Rati	0.730	
	Bending Stress OK	
fu : Max. Plate Bearing Stress	1.104	ksi
Fp : Allowable :	1.104	ksi
Bearing Stress Ratic	1.000	
	Bearing Stress OK	
Tension in each Bolt	4.350	
Allowable Bolt Tension	34.070	
Tension Stress Rati	0.128	
	Tension Stress OK	

Job No: 23-3108R01

Spread Footing:

Geotech Engineer:	
Project No.:	
Date:	

Soil properties are presumptive from the Building Code (1806.2). Where the building official has reason to doubt the classification, strength or compressionability of the soil, the building official may require a geotechnical investigation per 1803.5.2.

Soil Properties:

Concrete: f'c=	**3000 psi**
Rebar =	60000 psi
Soil Bearing Pressure =	**1500 psf**
Lateral Pressure =	**100 psf/ft**

Constrained at Top

Assume: L = **6.5 ft** W = **6.5 ft**

A= 42.25 SqFt
3.91 CuYds

Depth of FTG= **2.5** ft

Depth of Soil = **2.5** ft

6" - 8" Slab on Grade over Ftg

Reinforcement:

Quantity: **7**

Rebar Mat Top & Bottom
#5 Each Way

Asmin = 4.21 sqin
Ac= 2340 sqin

Density - 145 lbs
Soil - 110 lbs

Weight of Footing=	15.32 k	Wind Uplift=	-2.42 k		
Weight of Soil =	11.62 k	Dead Load at FND=	30.30 k	Check:	OK

Overturning and Bearing Checked in Enercalc

General Footing

Project File: 23-3108R01 28X110 ELMENDORF, TX MOBIL 2Di.ec6

DESCRIPTION: Spread Footing

Code References

Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.750	

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00090
Min. Overturning Safety Factor	=	1.50 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	1.50	ksf
Soil Density	=	110.0	pcf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	100.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

Footing base depth below soil surface	=	5.0	ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=		ksf
when max. length or width is greater than	=		ft

Dimensions

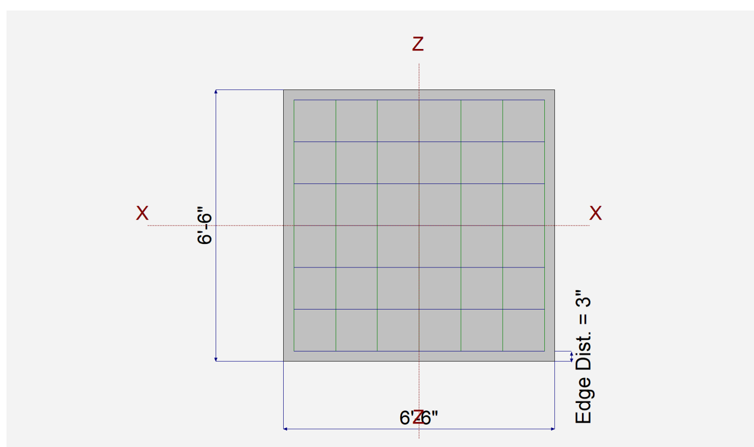
Width parallel to X-X Axis	=	6.50	ft
Length parallel to Z-Z Axis	=	6.50	ft
Footing Thickness	=	30.0	in

Pedestal dimensions...

px : parallel to X-X Axis	=		in
pz : parallel to Z-Z Axis	=		in
Height	=		in

Rebar Centerline to Edge of Concrete...

at Bottom of footing	=	3.0	in

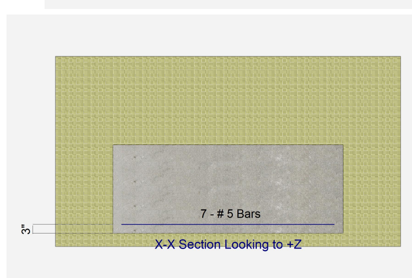


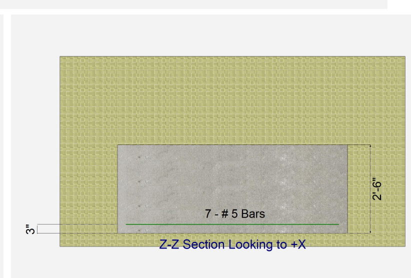
Reinforcing

Bars parallel to X-X Axis			
Number of Bars	=		7.0
Reinforcing Bar Size	=	#	5
Bars parallel to Z-Z Axis			
Number of Bars	=		7.0
Reinforcing Bar Size	=	#	5

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	3.365	6.974			8.717	0.0840		k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=					33.065	6.386		k-ft
V-x	=					1.749	0.3450		k
V-z	=								k

General Footing

Project File: 23-3108R01 28X110 ELMENDORF, TX MOBIL 2Di.ec6

LIC# : KW-06014602, Build:20.23.08.30 Jimco Sales & Mfg

DESCRIPTION: Spread Footing

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.8847	Soil Bearing	1.327 ksf	1.50 ksf	+D+0.60W about Z-Z axis
PASS	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
PASS	3.387	Overturning - Z-Z	22.463 k-ft	76.082 k-ft	+0.60D+0.60W
PASS	12.499	Sliding - X-X	1.049 k	13.117 k	+0.60D+0.60W
PASS	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.1225	Z Flexure (+X)	4.910 k-ft/ft	40.071 k-ft/ft	+1.20D+0.50Lr+W
PASS	0.04741	Z Flexure (-X)	1.90 k-ft/ft	40.071 k-ft/ft	+1.20D+1.60Lr
PASS	0.0610	X Flexure (+Z)	2.444 k-ft/ft	40.071 k-ft/ft	+1.20D+1.60Lr+0.50W
PASS	0.0610	X Flexure (-Z)	2.444 k-ft/ft	40.071 k-ft/ft	+1.20D+1.60Lr+0.50W
PASS	0.03955	1-way Shear (+X)	3.249 psi	82.158 psi	+1.20D+0.50Lr+W
PASS	0.01528	1-way Shear (-X)	1.256 psi	82.158 psi	+0.90D+W
PASS	0.01695	1-way Shear (+Z)	1.393 psi	82.158 psi	+1.20D+1.60Lr+0.50W
PASS	0.01695	1-way Shear (-Z)	1.393 psi	82.158 psi	+1.20D+1.60Lr+0.50W
PASS	0.03609	2-way Punching	5.931 psi	164.317 psi	+1.20D+1.60Lr+0.50W

Top reinforcing mat required (see 'Bending' tab).

Hand check required for anchor pullout.

Anchor Bolt Design for Spread Footing

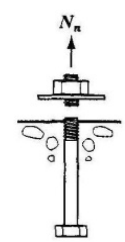
λ_a = 1.0 for cast-in-place Anchors in normal weight concrete

Tensile Force on Anchors (Nu) =	Nu	22.70	kips
Shear force on Anchors (Vu) =	Vu	1.57	kips
Seismic Design Category =		B	
Number of Anchors (n) =		4	
Number of Anchors in Tension (n) =		2	

Anchor Diameter (da) = **1.00 inch** Anchor Area (Ase) = 0.61 in^2

Anchor Spacing perpendicular to load (S1)=	16	inch
Anchor Spacing Parallel to load (S2)=	16	inch
Spacing of Outer Anchors (So) =	16	inch
Embedment Depth of Anchor on Concrete (h_{EF})	**26**	**inch**
Yield Strength of Anchors (Fy) =	55	ksi
Tensile Strength of Anchors (Futa)	75	ksi
Edge distance in Load Direction (Ca1)	31	inch
Edge distance perpendicular to Load (Ca2)	31	inch
Concrete strength (f'c) =	3000	psi
Axial Eccentricity (e'N)=	8	inch
Shear Eccentricity (e'v)=	0	inch
Footing Width =	**6.50**	ft
Embed Washer with Nut =	**NO**	

I. Steel Strength of Anchors in Tension (Eq. 17.4.1)


Steel failure

F_{uta} = < 1.9 Fy or 125 ksi = 104.50 ksi

F_{uta} = Min(1.9 Fy, F_{uta}, 125) 75.00 ksi

Thus $N_{sa} = n*A_{SE}*f_{uta}$ 90.86 kips

Φ = 0.75 $\Phi\ N_{sa}$ = 68.15 kips OK

II. Concrete Breakout Strength of Anchors in Tension (17.4.2)

A_{Nc} 4836 in^2 $A_{Nco} = 9*h_{ef}^2$ = 3844 in^2

$\psi_{ec,N} = 1/(1+2e'N/3*h_{EF})$=<1.0 $\psi_{ec,N}$ 0.79

Ca(min) > 1.5hEF 39.00 in YES $\psi_{ed,N} = 0.7 + (0.3*C_a\min/1.5\ h_{EF})$ $\psi_{ed,N}$ 1.00

Cracking Service Loads? YES $\psi_{c,N}$ 1.25

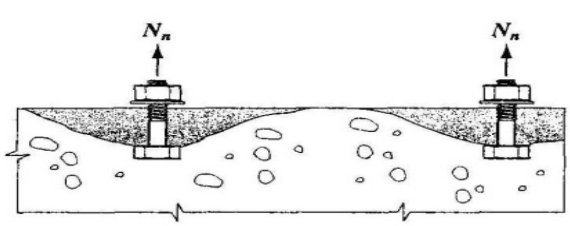
Headed anchors used, Therefore - $\psi_{cp,N}$ 1.00

h'_{EF} >= 11in? 20.67 in YES

N_b = $16\lambda_a f'c^{0.5} h_{EF}^{5/3}$ = 136.39 kips For a Single anchor

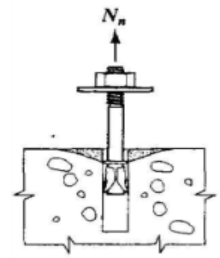
$N_{cbg} = A_{Nc}/A_{nco}*\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}*N$ = 170.49 kips For a group anchors

ΦN_{cbg} 127.87 kips OK


Concrete breakout

Concrete breakout of anchors in Tension will be resisted by the mat of reinforcing at the top of the footing, therefore, concrete breakout will not control the tensile capacity of the anchor rods.

III. Pullout strength of Anchors in Tension (17.4.3)



Pullout

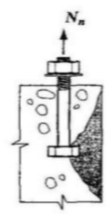
A_{BRG} =	1.50 in^2			
Np = 8* A_{BRG}*f'c =	36.03 kips			
Cracked at service Loads?		YES	$\Psi_{c,p}$ =	1
ΦN_{Pn} (Group) =	$n\Psi_{c,p}N_p$ =		72.07 kips	
Φ = 0.7	ΦN_{Pn} (Group) =		50.45 kips	
			OK	

IV. Side-face Blowout of Anchors in Tension (17.4.4)

h_{EF}>2.5Ca1	77.50	No	Side Face Blowout Will Not Control		
S1<6Ca1	186	YES	R=	0.5	
Ca2<3Ca1	0.5	YES			
N_{sb} = 160*C_{a1}*(A_{BRG})^0.5 *λa*f'c^.5*R	124.83	kips	Thus-0.75*N_{sb} =	93.62	kips
Therefore - $N_{sbg} = (1+s_o/6c_{a1})*N_{sb}$	101.68	kips			

s_0 = distance between outer anchors along the edge

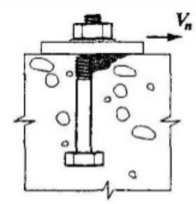
Φ = 0.75 Thus-ΦN_{sbg} = 76.26 kips OK



Side-face blowout

V. Steel Strength of Anchor in Shear (17.5.1)

Built-up grout pads used?	Yes			
$V_{sa} = 0.6nA_{SE}F_{uta}$	54.52	kips		
Thus ΦV_{sa} =	35.44	kips >	1.57 kips	OK
Φ = 0.65				



Steel failure preceeded by concrete spall

VI. Concrete Break out Strength of Anchors in Shear (17.5.2)

A_{vc}= 3627 in^2 = 4.5*$(C_{a1})^2$ = 4325 in^2 A_{vc}=< nA_{vco} OK

A_{vc} =< n*A_{vco} OK

$\Psi_{ec,V}$ = 1/(1+2e'V/3Ca1)=<1.0		$\psi_{ec,V}$	1.00
Ca2 >= 1.5*Ca1 ?	NO	$\psi_{ed,V}$	0.90
Cracking at Service Loads?	YES	$\psi_{c,V}$	1.20
		$\psi_{h,V}$	1.00

Thickness: h_a = 30.0 in

$V_b = \{7(l_e/d_a)0.2(d_a^{0.5})\}\lambda_a f'c^{0.5}*(c_{a1})^{1.5}$ Eq. (17.5.2.3 = 100.30 kips

$V_b = 9\lambda_a f'c^{0.5}*(c_{a1})^{1.5}$ Eq. (17.5.2.2 = 85.08 kips

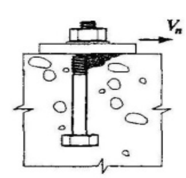
$V_{cbg} = (A_{vc}/A_{vco})Y_{ec,V}\,Y_{ed,V}\,Y_{c,V}\,Y_{h,V,*}V_b$ = 77.07 kips

Φ = 0.7 Thus-ΦVc_{bg} = 53.95 kips

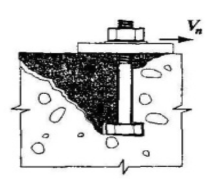
Ca1=Ca2 per design

Thus ΦVcbg= (Avc/Avco)* $Y_{ec,v}\,Y_{ed,v}\,Y_{c,v}$ Yh,v,*Vb

53.95 kips > 1.6 kips OK

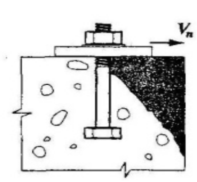


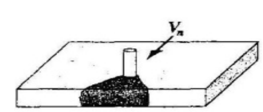
Steel failure preceeded by concrete spall



Concrete pryout for anchors far from a free edge

Tie Size	= # 4 bar	Bar area	Av	0.2	sq. inch
Number of ties at top	= 3		Fy	60	ksi
Number of shear effects = 3 *2				6	
Tensile Capacities of Ties			$T = 6A_vF_y$	72	kips
Therefore			V_{cbg}	72	kips
			Φ =	0.7	
ΦVcbg		50.4 kips >		1.6 kips	OK



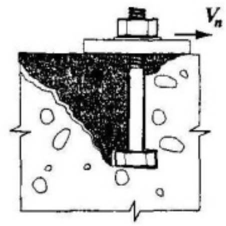


VII. Concrete pryout strength of Anchor in Shear (17.5.3)

$V_{cpg} = K_{cp} * N_{cpg}$ Eq. 17.5.3.1b

$h_{EF} =$ 26 inch.

Thus K_{cp} 2



Concrete pryout for anchors far from a free edge

Recalling:	N_{cpg}	127.87	kips		
	V_{cpg}	255.74	kips		
	Φ =	0.7			
Thus	ΦV_{cpg}	179.02	kips >	1.57 kips	OK

VIII. Anchor Rods Group Capacity

Allow. Tension	ΦN_n	50.4 kips	>	22.7 kips	OK
Allow. Shear	ΦV_n	35.4 kips	>	1.6 kips	OK

IX. Check Shear / Tension Interaction

If $\underline{N_{ua}/\Phi N_n <= 0.2}$ 0.45 No

Therefor full shear design strength is not permitted

If $\underline{V_{ua}/\Phi V_n < = 0.2}$ 0.04 Yes

Full Tension design is permitted

$\underline{\mathbf{N_{ua}/\Phi N_n + V_{ua}/\Phi V_n =}}$ 0.49 < 1.2 OK

Job No: 23-3108R01

Straight Shaft Pier:

Geotech Engineer:	
Project No.:	
Dated:	

IBC - 1810.3.3.1.4

Soil properties are presumptive from the Building Code (1806.2). Where the building official has reason to doubt the classification, strength or compressibility of the soil, the building official may require a geotechnical investigation per 1803.5.2.

Soil Properties:

Concrete: f'c=	3000 psi
Rebar =	60000 psi
Soil Bearing Pressure =	1500.0 psf
Lateral Pressure =	250 psf/ft
Adhesion =	130 psf
Concrete Density =	145 psf
C(u) =	0 psf

Depths:

	BGS =	1.0 ft
	Pier =	**6.5 ft**
(Neglected Top Skin Friction)	Za =	0.0 ft
	La =	6.5 ft
(Neglected Bot Skin Friction)	N1 =	0.0 ft
	Bearing =	7.5 ft

Assume: Shaft Dia= **3.0 ft** A= 7.07 SqFt

Check Gravity Loads:

Soil Bearing =	14.14 k	Axial Load =	9.24 k	
Pier Weight =	6.66 k			
Adhesion =	7.96 k			
Total Allowable load =	22.10 k	>	9.24 k	OK
Unity Check =	0.42	<	1.0	OK

Check for Uplift:

Trib =	280.0 SqFt	S.F.	1.33				
Gross Uplift =	-21.45 psf	Pier Weight =	6.66 k				
Dead Load =	10.33 psf	Adhesion =	7.96 k				
Net Uplift =	-1.87 k	Net Uplift Resistance =		3.74 k	<	12.64 k	OK
Uplift from Soil =	-1.87 k	Unity Check =		0.30	<	1.0	OK

Check - Lateral Loads:

Column Height =	h	18.5 ft	
Shear =	V	1.7	Kip
Moment =	M	20.45	Kip-ft
Applied Force =	Mma/h	1106	lbs
Lateral Pressure =	S3	1625	

Constrained at Top

Diameter:	b	3.0 ft
Depth of Embedment:	d	6.5 ft

Check Depth: Req'd Depth = 5.2 ft OK

Reinforcement:	Quantity:	Vertical Rebar:	Rebar Ties: #4	
Long. Reinforcement :	12	#6	(2) _ in top 5"; Ties @ 12" O.C.	ACI 318-14

Ac=	1017.9 sqin				
Asmin=	1.83 sqin	As Provided =	5.30 sqin	> 2 sqin	OK
Asmin Req'd=	0.16 sqin	Steel Ratio =	0.52%	> 0.50 %	OK

Pier Anchor Check

Load Information		Pier Information		Anchor Information	
Nu=	10.32 kip/anchor	D=	3.00 ft, dia	da=	1.00 in
Vu=	0.39 kip/anchor	f'c=	3.0 ksi	Lemb=	26.00
		# bar=	12	Fya=	55 ksi
		bar size=	#6	PhiCv=	0.70
		db=	0.75 in	Abrg=	1.50 in
		fy=	60 ksi		

Tension (with anchor reinforcement):

Development Length of Hooked Bars (ACI 318) **Hooked Bars Req'd -** **NO**

Ldh= .02*Psi*fy*db / lambda*(f'c)^.5 *(0.70) = 11.5 in

y= tan(35)*Dist = 2.0 in

Lact= Lemb-2"(clr)-4db-y = 19.0 in

dv=	3"(clr)+0.5"(tie)+db/2=	3.88 in
rv=	rp-dv	14.13 in
ra=	8*(2)^.5	11.31 in
rp=	D*12/2	18.00 in
Dist=	rp-dv-ra	2.81 in

Tension Capacity (Lact/Ldh) = 1.00 x 100 percent

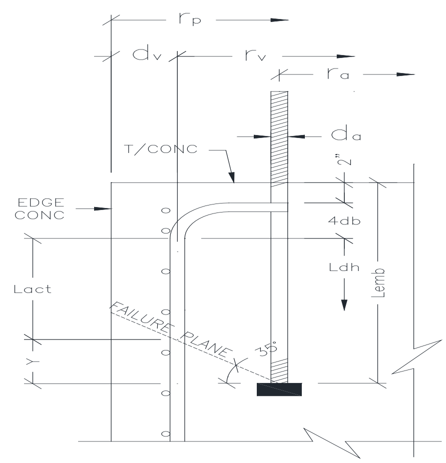
Spacing of Vertical Bars

s= pi(D-2dv)/#bar 7.40 c/c of bars

arc= CL of reinf within Lemb/2 = 29.7 in

#bars within Lemb/2 = 4.0 use 4.00 bars

PhiNn=Phi*Ab*Num*fy*TC(%) = 79.5 k/anchor OK



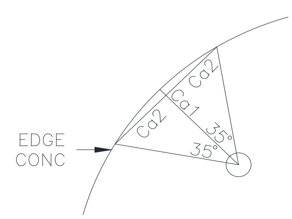
Shear (without anchor reinforcement):

Ca1= rp-ra = 6.69 in

Ca2= (rp-ra)*sin(35)= 3.84 in

Avc= (2*Ca2)*(1.5*Ca1)/2= 38 in^2

Avco= 4.5*Ca1 = 201 in^2



Vb= 7*(Lemb/da)^.2*(da^.5)*lambda*(f'c^.5)*Ca1^1.5= 10.05 k

Psi ed,v= 1 in

Psi h,v= 1 in

Psi c,v= 1 in due to #4 ties

phiVcb= phi*(Avc/Avco)*Psi ed,v*Psi h,v*Psi c,v*Vb= 1.53 k/anchor OK

Tension Side Face Blowout:

Nsb= 55.53 k PhiNsb= 38.87 k OK

Pole Footing Embedded in Soil

Project File: 23-3108R01 28X110 ELMENDORF, TX MOBIL 2Di.ec6

LIC# : KW-06014602, Build:20.23.07.20 | Jimco Sales & Mfg |

DESCRIPTION: Drilled Pier

Code References

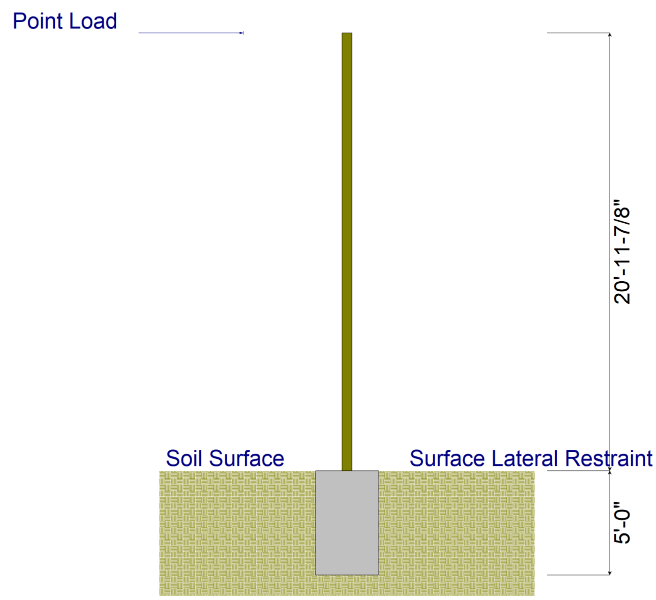
Calculations per IBC 2021 1807.3, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Pole Footing Shape	Circular
Pole Footing Diameter	36.0 in
Calculate Min. Depth for Allowable Pressures	
Lateral Restraint at Ground Surface	
Allow Passive	250.0 pcf
Max Passive	1,500.0 psf

Controlling Values

Governing Load Combination	+D+0.60W
Lateral Load	1.049 k
Moment	22.027 k-ft
Restraint @ Ground Surface	
Pressure at Depth	
Actual	**1,248.19** psf
Allowable	**1,250.0** psf
Surface Retraint Force	10,410.8 lbs
Minimum Required Depth	**5.0 ft**
Footing Base Area	7.069 ft^2
Maximum Soil Pressure	1.771 ksf



Applied Loads

Lateral Concentrated Load (k)		Lateral Distributed Loads (k	Applied Moment (kft)	Vertical Load (k)
D : Dead Load	k	k/ft	k-ft	3.365 k
Lr : Roof Live	k	k/ft	k-ft	6.974 k
L : Live	k	k/ft	k-ft	k
S : Snow	k	k/ft	k-ft	k
W : Wind	1.749 k	k/ft	k-ft	8.717 k
E : Earthquake	0.3450 k	k/ft	k-ft	0.0840 k
H : Lateral Earth	k	k/ft	k-ft	k
Load distance above ground surface	20.990 ft	TOP of Load above ground surface ft		
		BOTTOM of Load above ground surface ft		

Load Combination Results

	Forces @ Ground Surface		Required	Pressure at Depth		Soil Increase
Load Combination	Loads - (k)	Moments - (ft-k)	Depth - (ft)	Actual - (psf)	Allow - (psf)	Factor
D Only	0.000	0.000	0.13	0.0	31.3	1.000
+D+Lr	0.000	0.000	0.13	0.0	31.3	1.000
+D+0.750Lr	0.000	0.000	0.13	0.0	31.3	1.000
+D+0.60W	1.049	22.027	5.00	1,248.2	1,250.0	1.000
+D+0.750Lr+0.450W	0.787	16.520	4.63	1,094.1	1,156.3	1.000
+D+0.450W	0.787	16.520	4.63	1,094.1	1,156.3	1.000
+0.60D+0.60W	1.049	22.027	5.00	1,248.2	1,250.0	1.000
+D+0.70E	0.242	5.069	3.13	735.4	781.3	1.000
+D+0.5250E	0.181	3.802	2.88	651.6	718.8	1.000

Pole Footing Embedded in Soil

DESCRIPTION: Drilled Pier

+0.60D+0.70E	0.242	5.069	3.13	735.4	781.3	1.000

Anchor Bolt Design for Drilled Pier

Anchor Bolt (ASTM F1554, Gr. 55), ƛa = 1.0 for cast -in-place anchors in normal weight concrete (ACI 318 Section 8.6 and D3.6)

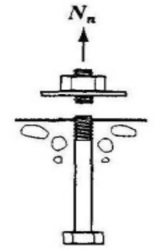
ANCHOR ROD GROUP CHECK (Per ACI318-14 Chapt. 17 Anchoring to Concrete)

Tensile Force on Anchors (Nu) =	Nu	10.32	kips	Footing Diameter =	**3.0 ft**
Shear force on Anchors (Vu) =	Vu	1.57	kips	Concrete strength (f'c) =	**3000 psi**
Seismic Design Category =		B			
Number of Anchors (n)	n	4			
Number of Anchors in Tension (n) =		2			
Anchor Diameter (da) =	**da**	**1.00**	**inch**	Anchor Area (Ase) =	0.61 sqin
Embedment Depth of Anchor on Concrete (h_{EF})		**26**	**inch**		
Embed Washer with Nut =		**NO**		Yield Strength of Anchors (Fy) =	55ksi
Anchor Spacing perpendicular to load (S1)=		16	inch	Tensile Strength of Anchors (Futa)	75ksi
Anchor Spacing Parallel to load (S2)=		16	inch		
Spacing of Outer Anchors (So) =		16	inch		
Edge distance in Load Direction (Ca1)		8.12	inch		
Edge distance perpendicular to Load (Ca2)		8.12	inch		
Axial Eccentricity (e'N)=		8	inch		
Shear Eccentricity (e'v)=		0	inch		

I. Steel Strength of Anchors in Tension (Eq. 17.4.1)

Nsa =n*Ase*futa =< 1.9*Fy or 125 ksi

futa = < 1.9 Fy or 125 ksi =		104.50	ksi	
futa = Min(1.9 Fy,Futa,125)		75.00	ksi	
Thus Nsa = n*Ase*futa		90.86	kips	
Φ = 0.75	Φ Nsa =	<u>68.15</u>	kips	OK



Steel failure

II. Concrete Breakout Strength of Anchors in Tension (17.4.2) ΦNcbg Φ=.75

A_{Nc} = projected Concrete failure area of a single anchor or group of anchors, for calculation of strength in tension

A_{Nc} 1040.00 sq. inch

A_{Nco} = projected Concrete failure area of a single anchor, for calculation of strength in tension if not limited by edge distance or spacing. $A_{Nco} = 9*h_{EF}$^2 = 2327.97 sq. inch

$y_{ec,N}$= Factor used to modify tensile strength of anchors based on eccentricity of applied loads

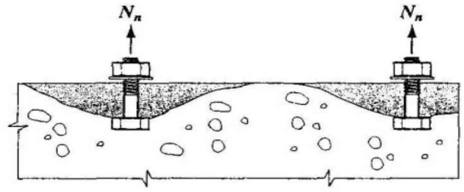
$y_{ec,N} = 1/(1+2e'N/3*h_{EF)}$=<1.0		0.830	
Ca(min) < 1.5hEF	$1.5h_{EF}$	39.000	in
Adjusted hEF	h_{EF}	16.083	in

$y_{ed,N =}$ Factored used to modify tensile strength of anchors based on proximity to edges of concrete members

yed,N = 0.7 +(0.3* Ca,min/1.5 hEF)	$y_{ed,N}$	0.801		Kc	24	
	$y_{c,N}$	1.00		N_b	85	kips
	$y_{cp,N}$	1.00				
Then $N_{cbg} = A_{Nc}/A_{Nco}$ *	$Y_{ec,N}Y_{ed,N}Y_{c,N}Y_{cp,N}*N_b$	25.18	kips	For a group anchors		
	ΦNcbg	18.88	kips	OK		

Since, ΦNcbg < Nu

Rebar cage is required to tie anchor bolts to the footing, so concrete breakout will be limited by the tensile strength of the rebar vertical in the cage



Concrete breakout

Number of vertical bars w/ Hooks 12

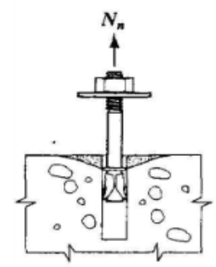
Size of bar #6

Cross Section Area of Single Bar — As — 0.44 sq. inch

Yield Strength of Reinforcement — Fy — 60 ksi

Tensile strength of Reinforcement — ΦNcbg — 238.6 kips

III. Pullout strength of Anchors in Tension (17.4.3) — ΦN_{Pn} = in group



Pullout

Bearing Area of Hevy Hex Anchor Bol A_{BRG} — 1.50 sq. inch

$Np = 8* A_{BRG}*f'c$ — Np — 36.03 kips

Cracking at Service Loads? Yes — $Y_{c,p}$ — 1

Number of tensile Anchors — n — 2

$N_{PN} = n*Y_{c,p}*N_p$ — N_{PN} = — 72 kips

Φ = 0.75 — $\Phi N_{PN(group)}$ — 54.05 kips — OK

IV. Side-face Blowout of Anchors in Tension (17.4.4) — ΦN_{sbg} = — Φ = 0.75

h_{EF}>2.5Ca1 — 2.5Ca1= — 20.31 — Yes

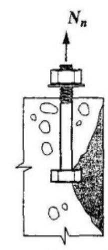
S1<6Ca1 — 6Ca1= — 48.75 — Yes

So that Side-Face Blowout will not Control

Ca2<3Ca1 Yes — R =(1+Ca2/Ca1)/4 — 0.50

$N_{sb} = 160*C_{a1}*\lambda a*(A_{BRG})^{0.5} *f'c^{.5}*R$

Thus — N_{sb} = — 43.62 — 43.62 kips



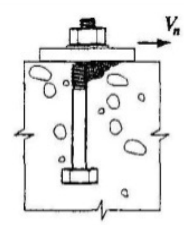
Side-face blowout

Therefore Nsbg = $(1+s_0/6ca1)$*Nsb — 57.94 kips

s_0 = distance between outer anchors along the edge

Thus — ΦNsbg = — 43.45 kips — OK

V. Steel Strength of Anchor in Shear (Eq.17.5.1.2l ΦVsa = — Φ = 0.65



Steel failure preceeded by concrete spall

For cast -in headed bolts where sleeves do not extended through the shear plane

Built-up grout pads used? — Yes

V_s = 0.6 * n*Ase* Fut — 54.52 kips

Thus — ΦVsa — 35.44 kips > — 1.57 kips — OK

VI. Concrete Break out Strength of Anchors in Shear (17.5.2)

Vcbg= (Avc/Avco)* $Y_{ec,v}$ $Y_{ed,v}$ $Y_{c,v}$ Yh,v*Vb — Eq 17.5.2.1b)

For shear force perpendicular to the edge on a group of anchors

A_{vc} = Projected concrete Failure Area group of anchors for calculation of strength of shear

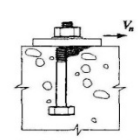
A_{vc}= — $(C_{a2} +s +C_{a2})*(1.5C_{a1})$ — Avc — 393.01 Sq. inch

A_{vco} = Projected concrete failure area of a single anchor for calculation of strength in shear.

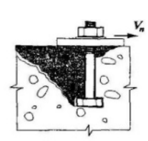
if not limited by corner influences, spacing, or member thickness

A_{vco} = — $4.5*(C_{a1})^2$ — A_{vco} — 297.03 Sq. inch

A_{vc}=< $n*A_{vco}$ — OK



Steel failure preceeded by concrete spall

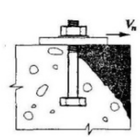


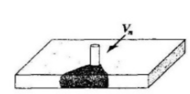
Concrete pryout for anchors far from a free edge

$Y_{ec,v}$ = Modification factor for Anchor group loaded eccentrically in shear

$Y_{ec,v}$ = — 1/(1+2e'v/3ca1)<=1.0 — $Y_{ec,v}$ = — 1.00

Ca2 >= 1.5*Ca1 ? — No





Since ca2 = ca1 = 16 inch.

$Y_{ed,v}$ = modification factor edge effects for anchors loaded in shear

$Y_{ed,v}$ = 0.7 + 0.3 *(ca2/1.5ca1) $Y_{ed,v}$ = 0.90

Cracking and Service loads? Yes

$Y_{c,v}$ = modification factor for concrete condition at service load level: pedestal ties resist cracking and act as supplemental reinforcement

$Y_{c,v}$ = 1.20

$Y_{h,v}$ = Modification factor for member thickness in relation to anchor embedment depth

ha > h_{EF} $Y_{h,v}$ = 1.00

Vb = the basic concrete break out strength in shear of single anchor in cracked concrete

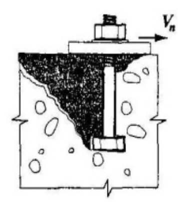
Vb1= [7{le/da)^0.2* (da^0.5)}*ƛa*f'c^0.5*(ca1)^1 Eq. 17.5.2.2a)

le = loaded bearing length of the anchor for shear

le = h_{EF} le<= 8*da	le	8	inch
Vb(a) = 7*(le/da)^0.2*(da)^0.5*(f'c)^	$V_{b(a)}$	13.46	kips
Vb(b) = 9*ƛa*(f'c^0.5)*(ca1^1.5)	$V_{b(b)}$	11.42	kips
Thus Vb = Min(Vb1, Vb2)	Vb	11.42	kips
	Φ =	0.7	

Thus ΦVcbg= (Avc/Avco)* $Y_{ec,v}$ $Y_{ed,v}$ $Y_{c,v}$ Yh,v,*Vb <u>11.42</u> kips > 1.57 kips OK

VII. Concrete pryout strength of Anchor in Shear (Vcpg) = (17.5.3)



Concrete pryout for anchors far from a free edge

Vcpg = Kcp*Ncpg Eq. 17.5.3.1b

When	hEF	26	inch.	
Thus	Kcp	2		

Ncpg = basic concrete pryout strength of a group of a anchors

Recallin	Ncpg	25.18	kips	
	Vcpg	50.35	kips	
	Φ =	0.7		
Thus	ΦVcpg	35.25	kips >	1.57 kips OK

VIII Anchor Rods Group Capacity

Allow. Tension	ΦNn	18.88	kips >	10.32 kips OK
Allow. Shear	ΦVn	35.25	kips >	1.57 kips OK

IX Check Shear / Tension Interaction

If <u>Nua/ΦNn <= 0.2</u> 0.55 No

Therefor full shear design strength is not permitted

If <u>Vua/ΦVn < = 0.2</u> 0.04 Yes

Full Tension design is permitted

<u>**Nua/ΦNn + Vua/ΦVn =**</u> 0.59 < 1.2 OK