UNIT BASE FOUNDATION SUMMARY

Parelell Infrastructure **U- 26.0 250** v 4.4
PIWA513- Darrington-Christian Camp Road, WA **A- 540603**

Foundation Dimensions

Pad width, **W**:	**32.50**	ft
Depth, **D**:	**7.50**	ft
Ext. above grade, **E**:	**0.50**	ft
Pier diameter, $\mathbf{d_i}$:	**3.00**	ft
Pad thickness, **T**:	**1.75**	ft
Depth neglected, **N**:	**7.50**	ft
Volume, $\mathbf{V_o}$:	**73.37**	cy

Reinforcement Design

pad rebar qty., m _p:	**45**	bars *
size, **s _p**:	**7**	
pier vertical qty, m _c:	**13**	verticals/pier
size, **s _c**:	**8**	**2.5' cage**
Horizontal Rebar in top 6in of pier for temp. & shrinkage?:	**no**	per TIA-222-H 9.6
pier tie qty., m _t:	**12**	ties/pier
size, **s _t**:	**4**	**default hook**

* Rebar to be equally spaced, both ways, top & bottom, for a total of 180 bars
* Use standees to support top rebar above bottom rebar in mat

Soil Information Per:

Geotechnical Engineering Evaluation by Black Mountain Consulting, Project No. 210093-GEO, dated 06/09/2021and Addendum dated February 22, 2022

Soil Parameters

Soil unit weight, γ:	**100**	pcf
Ultimate Bearing, $\mathbf{B_c}$:	**7.500**	ksf
Cohesion, $\mathbf{C_o}$:	**0.000**	ksf
Friction angle, φ:	**0.0**	degrees
Ult. Passive P., $\mathbf{P_p}$:	**0.100**	pcf
Base sliding, **μ**:	**0.72**	
Seismic Design Cat.:	**D**	
Water at:	**none**	ft

Anchor Steel Selection

Part Number, **P/N**:	**113823**	Dia = 1.5 Length = 80"

Material Properties

Steel tensile str, $\mathbf{F_y}$:	**60000**	psi
Conc. Comp. str, $\mathbf{F'_c}$:	**4500**	psi
Conc. Density, δ:	**150**	pcf
Clear cover, **cc**:	**3.00**	in

Backfill Compaction

Lift thickness:	**8**	in
Compaction:	**92**	%
Modified Proctor:	ASTM	**D1557**

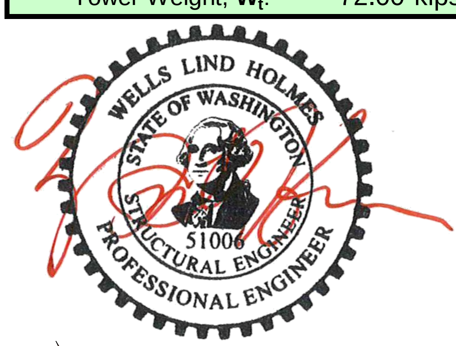
Tower design conforms to the following:

* *International Building Code (IBC)*
* *ANSI TIA-222-H*
* *Building Code Requirements for Reinforced Concrete (ACI 318-14)*

Note: **The centroid of the tower is offset from the centroid of the foundation**

Foundation Loading

			stress ratio: 98.6%	mark up: 1.4%
Shear (Per Leg), $\mathbf{S_l}$:	38.00	kips	x 1.01 =	38.53 kips
Shear (total), **S**:	56.00	kips	x 1.01 =	56.78 kips
Moment, **M**:	9310.00	ft-kips	x 1.01 =	9440.34 ft-kips
Compression/Leg, **C**:	436.00	kips	x 1.01 =	442.10 kips
Uplift/Leg, **U**:	389.00	kips	x 1.01 =	394.45 kips
Tower Weight, $\mathbf{W_t}$:	72.00	kips	=	72.00 kips



02/25/2022

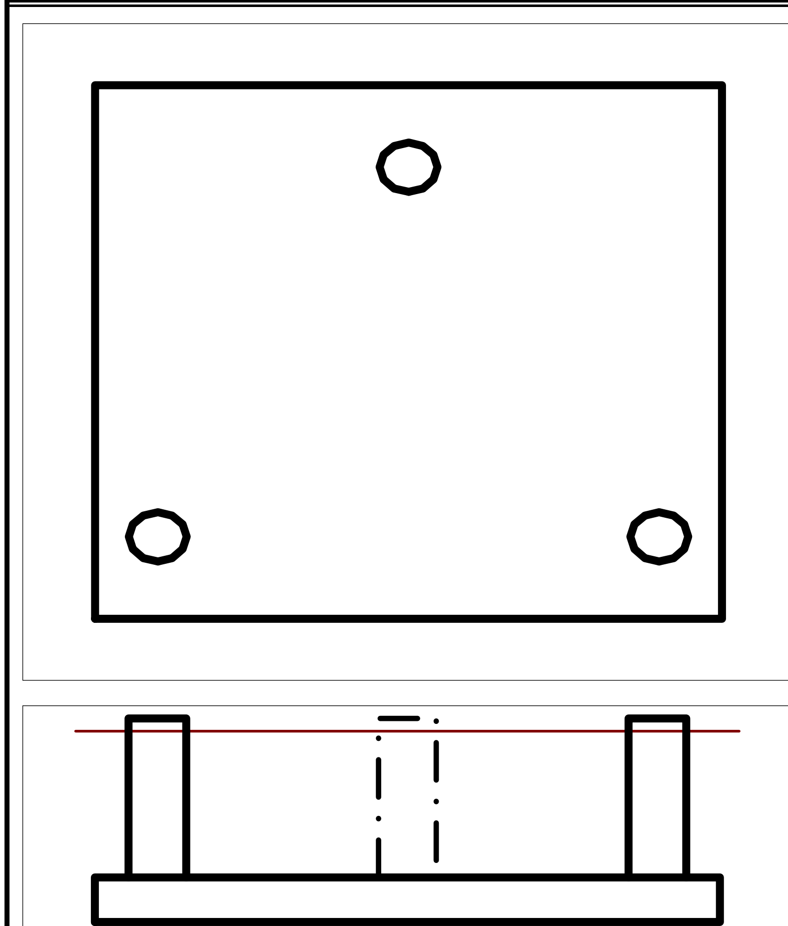
VECTOR ENGINEERS

651 W. Galena Park Blvd. Ste. 101 Draper, Utah 84020 Phone (801) 990-1775 www.vectorse.com

VECTOR PROJECT #: U2309-015-221

Views of Foundation

(not to scale)



Additional Notes:

* No foundation modifications listed.
* See attached "Foundation Notes" for further information.

FOUNDATION NOTES

1 THE ON-SITE GEOTECHNICAL ENGINEER SHALL CONFIRM THAT THE INSITU SOIL STRENGTHS MEET OR EXCEED THOSE PARAMETERS GIVEN IN THE SOIL REPORT.

2 SEE GEOTECHNICAL REPORT FOR ADDITIONAL CONSTRUCTION RECOMMENDATIONS, BACKFILL COMPACTION DETAIL, SUBGRADE PREPARATION ETC.

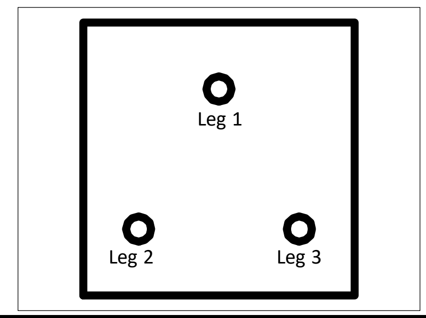
3 ALL FOOTING OVER-EXCAVATIONS SHOULD EXTEND HORIZONTALLY OUTWARD FROM THE FOOTING EDGE A DISTANCE EQUAL TO ONE HALF THE OVEREXCAVATION DEPTH FOR THE STRUCTURAL BACKFILL

UNIT BASE FOUNDATION (DL - 0.9)

Parellel Infrastructure U- 26.0 250 V 4.4
PIWA513- Darrington-Christian Camp Road, WA A- 540603

Reactions	stress ratio	98.6%	mark up:	1.4%
Shear (Per Leg), **S_l**:	38.00 kips	x 1.01 =	38.53 kips	
Shear (total), **S**:	56.00 kips	x 1.01 =	56.78 kips	
Moment, **M**:	9310.00 ft-kips	x 1.01 =	9440.34 ft-kips	
Compression / leg, **C**:	436.00 kips	x 1.01 =	442.10 kips	
Uplift / leg, **U**:	389.00 kips	x 1.01 =	394.45 kips	
Tower weight, **W_t**:	72.00 kips	=	72.00 kips	

Soil per: Geotechnical Engineering Evaluation by Black Mountain Consulting, Project No. 210093-GEO, dated 06/09/2021and



Physical Parameters:

Concrete volume:	$V = T * W^2 + 3 * (di^2 / 4 * \pi) * (D + E - T)$	$V =$	73.4	cy
Concrete weight:	$W_c = V * \delta$	$W_c =$	297.1	kips
Soil weight:	$W_s = (D - T) * (W^2 - 3 * (di^2 / 4 * \pi)) * \gamma$	$W_s =$	595.2	kips
Total weight:	$P = Wc + Ws + Wt$	$P =$	964.30	kips

Passive Pressure:

Pp coefficient:	$K_p = TAN(45 + \varphi / 2)^2$	$K_p =$	1.000	
	$P_{pn} = Kp * \gamma * N + 2 * Co * \sqrt{(Kp)}$	$P_{pn} =$	0.750	ksf
	$P_{pt} = Kp * \gamma * (D - T) + 2 * Co * \sqrt{(Kp)}$	$P_{pt} =$	0.575	ksf
	$P_{pb} = Kp * \gamma * D + 2 * Co * \sqrt{(Kp)}$	$P_{pb} =$	0.750	ksf
	$P_{ptop} = IF(N < (D - T), Ppt, Ppn)$	$P_{ptop} =$	0.8	ksf
	$Pp' = (Pptop + Ppb) / 2$	$Pp' =$	0.750	ksf
Shear area:	$T_{pp} = 0$	$T_{pp} =$	0.0	ft
	$A_{pp} = Tpp * W$	$A_{pp} =$	0.00	ft^2
Shear Capacity: *φr = 0.75*	$S_{actual} = (Pp' * App + \mu * P) * \varphi r$	$S_{actual} =$	520.720	kips

Check	$S_{actual} =$ 520.72 kips	>=	$S =$ 56.78 kips	**OK**

Overturning Moment Resistance at Toe:

Wt of soil wedge:	$W_{sw} = D * (D * TAN(\varphi)) / 2 * W * \gamma$	$W_{sw} =$	0.0	kips
Dist. from leg to edge:	$O = (W - 0.866 * w') / 2$	$O =$	4.992	ft
Additional offset of Wt:	$O_a = W / 2 - (1 / 3 * 0.866 * w' + O)$	$O_a =$	3.753	ft
Resisting moments:	$M_{rwt} = P * 0.9 * W / 2 - Wt * 1.2 * Oa$	$M_{rwt} =$	13778.61	ft-kips
	$M_{rp} = Pp' * App * (D - N) / 3 * \varphi r$	$M_{rp} =$	0.00	ft-kips
	$M_{rsw} = Wsw * (W + D * TAN(\varphi) / 3) * \varphi r$	$M_{rsw} =$	0.00	ft-kips
Total resisting: *φr = 0.75*	$M_{rt} = (Mrwt + Mrp + Mrsw)$	$M_{rt} =$	13778.61	ft-kips
Total overturning:	$M_o = M + S * (D + E)$	$M_o =$	9894.61	ft-kips

Check	$M_{rt} =$ 13778.61 ft-kips	>=	$M_o =$ 9894.61 ft-kips	**OK**

Bearing Resistance due to Pressure Distribution

Area of mat:	$area = W^2$	$area =$	1056.3	ft^2
Section modulus:	$SM = W^3 / 6$	$SM =$	5721.4	ft^3
Factored total weight:	$P' = (Wt / 1.2 + Wc + Ws) * 0.9$	$P' =$	857.1	kip
Pressure exerted:	$P_{pos} = P' / area + Mo / SM$	$P_{max} =$	2.541	ksf
	$P_{neg} = P' / area - Mo / SM$	$P_{min} =$	-0.918	ksf

Note: The stress resultant is NOT within the kern. Bearing area has been adjusted below.

Load eccentricity:	$e_c = Mo / P'$	$e_c =$	11.54	ft
In Parallel Direction	$P_{adj} = 2 * P' / (3 * W * (W / 2 - ec))$	$P_{adj} =$	3.736	ksf
In Diagonal Direction	P_{adj_diag} *see Diagonal Bearing Sheet (attached)*	$P_{adj\_diag} =$	4.915	ksf
Adj. applied pressure:	$q_a = IF(Pneg >= 0, Ppos, Padj)$	$q_a =$	3.736	ksf
Overburden Pressure: (factored)	q_{obp} = NA- Gross Bearing Provided	$q_{obp} =$	0.000	ksf

φr = 0.75 ***Check***	$q_a - q_{obp} =$ 3.736 ksf	<=	$B_c * \varphi r =$ 5.625 ksf	**OK**

Concrete Shear Strength:

One way beam action at d_i from tower

Effective depth:	$d_c = T - cc - db\_p / 2$	$d_c =$	17.563	in
Distance from edge of pad to pier face:	$d' = O - di / 2$	$d' =$	3.492	ft
Distance from edge of pad to dc	$d'' = d' - dc$	$d'' =$	2.028	ft
Bearing Pressure Slop	$q_s = qa / Weff$	$q_s =$	0.2647	kcf
Required shear:	$V_{n1} = [(qa - d'' * qs)+(d'' * qs /2)]*d''*W - [0.9*(D - T)*\gamma*d''*W]$	$V_{n1} =$	194.51	kips
Available shear: *[ACI 22.5.5.1] φs = 0.75 [ACI 21.2.1]*	$V_{c1} = \varphi s * 2 * \lambda * \sqrt{(F'c)} * W * dc$	$V_{c1} =$	689.21	kips

Check	$V_{c1} =$ 689.21 kips	>=	$V_{n1} =$ 194.51 kips	**OK**

Two way beam action at d_i / 2 from tower (ACI 22.6.5)

Eq. Square Column (ACI 8.10.1.3 & 22.6.4.1.2)	d_{eq} =di / 2 * √π	deq =	31.90	in	
Mat effective width in bearing	W_{eff} = Min (W, 3 * (W / 2 - ec))	W_{eff} =	14.115785	ft	
Ratio of long side to short side of Pier	β = 1 (for square or round piers)	β =	1.00		
Critical Section — *Length:*	b_1 = dc / 2 + deq / 2 + (W - w\) / 2	b_1 =	63.73	in	
Width:	b_2 = (dc + deq + W - SIN(60) * w\) / 2	b_2 =	84.63	in	
Perimeter:	b_o = b1 + b2	b_o =	148.37	in	
Centroid	c = (b1 * dc * b1 / 2) / (b1* dc + b2 * dc)	c =	13.689	ft	
Eccentricity:	e_c = (deq + dc) / 2 - c	e_c =	11.0444888	in	
Polar MOI	J_c = [(dc * b1^3 / 12) + (b1 * dc^3 / 12) + (b1 * dc * (b1 / 2 - c)^2] + (b1	J_c =	1.056E+06	in^4	
Moment Fraction transferred by — *flexure:*	$γ_f$ = 1 / (1 + 2 / 3 * √ (b1 / b2))	$γ_f$ =	0.63		
eccentricity of shear:	$γ_v$ = 1 - γf	$γ_v$ =	0.37		
Bearing Pressure Slope:	q_s = qa / Weff	q_s =	0.265	kcf	
Average Bearing Pressure:	$q_{a.pl}$ = ((Weff - b1) * qs + qa) / 2	$q_{a.pl}$ =	3.034	ksf	
Shear Force at Section:	V_{n_pier} = C - qa.pl * (b1 * b2)	V_{n_pier} =	328.475	kips	
Slab Moment:	M_{sc} = Sl * (D - T + E) + Vn_pier * e	M_{sc} =	543.14	ft-kips	
Required shear: φs = 0.75 [ACI 21.2.1]	= (Vn_pier / b0 * dc) + (γv * Msc* c / Jc)		157.02	psi	
Available shear: [ACI 22.6.5.2]	= φs * MIN(4*λ*√(Fc) , (2+(4/β))*λ*√(Fc) , (2+(αs*dc/bo))*λ*√(Fc))		201.246	psi	
	Check V_{c2} = 201.25 psi >=	V_{n2} =	157.02	psi	**OK**
Moment transfered: (Pier 1)	M_{n1} = γf * Msc	M_{n1} =	231.578	ft-kips	
Effective Beam Width:	w_{eff1} = deq + 1.5 * T + MIN(1.5 * T , (W - w\ * SIN(60) - deq) / 2)	w_{eff1} =	7.909	ft	
	A_{st_p1}' = Mn1 / (0.9 * Fy * dc)	A_{st_p1}' =	2.930	in^2	
	a_{p1} = Ast_p1' * Fy / (β * F'c * weff1)	a_{p1} =	0.499	in	
Required steel:	$A_{st\_p\_st1}$ = Mn1 / (Fy * (dc - ap1 / 2))	$A_{st\_p\_st1}$ =	2.675	in^2	
Required steel in entire mat:	$A_{st\_p\_ste1}$ = Ast_p_st1 * W / weff1	$A_{st\_p\_ste1}$ =	10.993	in^2	
Moment transfered: (Pier 2 or 3)	M_{n2} = γf * Msc (Controlling Case: Corner.)	M_{n2} =	344.084	ft-kips	
Effective Beam Width:	w_{eff2} = deq + 1.5 * T + MIN(1.5 * T , (W - w\ - deq) / 2)	w_{eff2} =	7.204	ft	
	A_{st_p2}' = Mn2 / (0.9 * Fy * dc)	A_{st_p2}' =	4.354	in^2	
	a_{p2} = Ast_p2' * Fy / (β * F'c * weff2)	a_{p2} =	0.814	in	
Required steel:	$A_{st\_p\_st2}$ = Mn2 / (Fy * (dc - ap2 / 2))	$A_{st\_p\_st2}$ =	4.011	in^2	
Required steel in entire mat:	$A_{st\_p\_ste2}$ = Ast_p_st2 * W / weff2	$A_{st\_p\_ste2}$ =	18.096	in^2	
		Controlling Case			*Pier 2: Corner*

Two way beam action at d_i / 2 from tower (ACI 22.6.5)- Uplift

Pier Reinforcement Dia	di_T =di -2*cc-2*db_t - 1*db_c	di_T =	28.000	in	
Eq. Square Column (ACI 8.10.1.3 & 22.6.4.1.2)	d_{eq_T} =dprebar / 2 * √π	d_{eq_T} =	24.81	in	
Critical Section Length:	b_{1_T} = deq_T + dc	b_{1_T} =	42.377	in	
Critical Section Perimeter:	b_{o_T} = 4 * (deq + dc)	b_{o_T} =	169.51	in	
Polar MOI	J_{c_T} = (b1_T^3 * dc / 6)+ (b1_T * d ^3 / 6)+(dc * b1_T * b2_T^2 / 2)	J_{c_T} =	929266.449	in^4	
Shear Force at Section:	$V_{n\_pier\_T}$ = U	$V_{n\_pier\_T}$ =	394.446	kips	
Required shear: φs = 0.75 [ACI 21.2.1]	= (Vn_pier_T / b1_T * dc) + (γv * Msc* c_T / Jc_T)		158.856	psi	
Available shear: [ACI 22.6.5.2]	' = φs * MIN(4*λ*√(Fc) , (2+(4/β))*λ*√(Fc) , (2+(αs*dc/bo))*λ*√(Fc))		201.25	psi	
	Check V_{t2} = 201.25 psi >=	V_{nt2} =	158.86	psi	**OK**

Column Compression Capacity:

Compression reaction: *φc = 0.65 [ACI 21.2.2.2]*	P_c = φc * 0.85 * F'c * (di² / 4 * π)	P_c =	2530.7	kips	
	Check P_c = 2530.69 kips >=	C =	442.10	kips	**OK**

Pier Reinforcement:

Cross-sectional area:	A_g = di² * π / 4	A_g =	1017.88	in^2	
Min. area of steel (pier): *[ACI 10.6.1.1] & [ACI 10.3.1.2]*	A_{st_c} =Ag * 0.01	A_{st_c} =	10.18	in^2	
Cage circle:	d_o = di - 2 * cc - db_c - 2 * db_t	d_o =	28.00	in	
Rebar:	$s_{\_c}$ = 8 — d_{b_c} = 1 in				
	$m_{\_c}$ = 13 — A_{b_c} = 0.79 in^2				
	A_{s_c} = Ab_c * m_c	A_{s_c} =	10.27	in^2	
	Check A_{s_c} = 10.27 in^2 >=	A_{st_c} =	10.18	in^2	**OK**
Actual moment:	M_{max} = (D - T + E) * S / 2	M_{max} =	177.45	ft-kips	
Pier moment capacity:	M_{allow} *per Maxmomnt.xls (see attached)*	M_{allow} =	184.27	ft-kips	
	Check M_{allow} = 184.27 ft-kips >=	M_{max} =	177.45	ft-kips	**OK**
Bar separation:	B_{s_c} = (do * π) / m_c - db_c	B_{s_c} =	5.77	in	
	Check 17 >= B_{s_c} = 5.77 in	>=	4"		**OK**

Vertical Rebar Development Length:

Reinforcement location: *[ACI 25.4.2.4]*	ψ_{t_c} = if the space under the rebar > 12 in, use 1.3, else use 1.0	ψ_{t_c} =	1.3	
Epoxy coating: *[ACI 25.4.2.4]*	ψ_{e_c} = if epoxy-coated bars are not used, use 1.0; but if epoxy-coated bars are used, then if Bs < 6 * db or cc < 3 * db, use 1.5, else 1.2	ψ_{e_c} =	1.0	
Max term: *[ACI 25.4.2.4]*	$\psi_t\psi_{e\_c}$ = the product of ψt & ψe, need not be taken larger than 1.7	$\psi_t\psi_{e\_c}$ =	1.3	
Reinforcement size: *[ACI 25.4.2.4]*	ψ_{s_c} = if the bar size is 6 or less, then use 0.8, else use 1.0	ψ_{s_c} =	1	
Light weight concrete: *[ACI 25.4.2.4]*	$\lambda_{\_c}$ = if lightwieght concrete is used, 0.75, else use 1.0	$\lambda_{\_c}$ =	1.0	
Spacing/cover: *[ACI 25.4.2.4]*	$c_{\_c}$ the smaller of: half the bar spacing or the concrete edge distace	$c_{\_c}$ =	3.38	in
Transverse bars: *[ACI 25.4.2.3]*	k_{tr_c} = 0 in (per simplification)	k_{tr_c} =	0	in
Max term: *[ACI 25.4.2.3]*	$c_{\_c}'$ = MIN(2.5, (c_c + ktr_c) / db_c)	$c_{\_c}'$ =	2.500	
Excess reinforcement: *[ACI 25.4.10.1]*	R_c = 1 *(excess reinforcement reduction is not used)*	$R_{\_c}$ =	1.00	
Development (tensile): [ACI 25.4.2.2]	$L_{dt}'_{\_c}$ = (3 / 40) * (Fy / λ_c * √(F'c)) * (ψtψe_c * ψs_c * R_c / c_c') * db_c	$L_{dt}'_{\_c}$ =	34.88	in
Minimum length: *[ACI 25.4.2.1]*	L_{d_min} = 12 inches	L_{d_min} =	12.0	in
Development length:	L_{dt_c} = MAX(Ld_min, Ldt'_c)	L_{dt_c} =	34.88	in
Confining Reinforcement: *[ACI 25.4.9.3]*	ψ_{r_c} = 1	ψ_{r_c} =	1.00	
Development (comp.): [ACI 25.4.9.2]	$L_{dc}'_{\_c}$ = Fy *ψr_c * db_c * R_c / (50 * λ_c * √(F'c))	$L_{dc}'_{\_c}$ =	17.89	in
	$L_{dc}''_{\_c}$ = 0.0003 * db_c * Fy * ψr_c * R_c	$L_{dc}''_{\_c}$ =	18.00	in
Development length:	L_{dc_c} = MAX(8, Ldc'_c, Ldc''_c)	L_{dc_c} =	18.00	in
Length available in pier:	L_{vc} = D - T + E - cc	L_{vc} =	72.0	in

Check	L_{vc} =	72.0	in	>=	L_{dt_c} =	34.9	in	**OK**
Check	L_{vc} =	72.0	in	>=	L_{dc_c} =	18.0	in	**OK**

Length available in pad: L_{vp} = T - cc — L_{vp} = 18.0 in

Check	L_{vp} =	18.0	in	>=	L_{dt_c} =	34.9	in	**HOOKS**
Check	L_{vp} =	18.0	in	>=	L_{dc_c} =	18.0	in	**OK**

Vertical Rebar Hook Ending:

Bar size & clear cover: [ACI 25.4.3.2]	ψ_{t_h} = if the bar size <= 11 and side cc >= 2.5", use 0.7, else use 1.0	ψ_{t_h} =	0.7	
Epoxy coating: [ACI 25.4.3.1]	ψ_{e_h} = if epoxy-coated bars are used, use 1.2, else use 1.0	ψ_{e_h} =	1.0	
Light weight concrete: [ACI 25.4.3.1]	λ_h if lightwieght concrete is used, 0.75, else use 1.0	$\lambda_{\_h}$ =	1.0	
Confining Reinforcement: *[ACI 25.4.3.2]*	ψ_{r_h} = 1	ψ_{r_h} =	1.00	
Development (hook): [ACI 25.4.3.1]	L_{dh}' = (Fy * ψt_h * ψe_h * ψr_h * R_c / (50 * λ_h * √(F'c))) * db_c	L_{dh}' =	12.5	in
Minimum length: [ACI 25.4.3.1]	L_{dh_min} the larger of: 8 * db or 6 in	L_{dh_min} =	8.0	in
Development length:	L_{dh} = MAX(Ldh_min, Ldh')	L_{dh} =	12.5	in

Check	L_{vp} =	18.0	in	>=	L_{dh} =	12.5	in	**OK**

Hook tail length:	L_{h_tail} 12 * db beyond the bend radius	L_{h_tail} =	16.0	in
Length available in pad:	L_{h_pad} = (W - w' - di) / 2	L_{h_pad} =	21	in

Check	L_{h_pad} =	21.0	in	>=	L_{dh_tail} =	16.0	in	**OK**

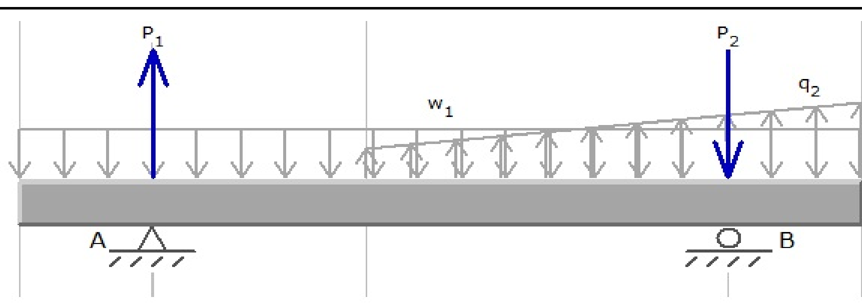
Pier Ties:

Minimum size: [ACI 25.7.2.2]	$s_{\_t\_min}$ =IF(s_c <= 10, 3, 4)	$s_{\_t\_min}$ =	3	
z factor:	z = 0.5 if the seismic zone is less than 2, else 1.0	z =	1	
Tie parameters:	$s_{\_t}$ = 4; d_{b_t} = 0.5 in			
	$m_{\_t}$ = 12; A_{b_t} = 0.2 in^2			
Allowable tie spacing:				
per vertical rebar [ACI 25.7.2.1] & [ACI 18.4.3.3]	$B_{s\_t\_max1}$ = 8 * db_c	$B_{s\_t\_max1}$ =	8	in
per tie size [ACI 25.7.2.1] & [ACI 18.4.3.3]	$B_{s\_t\_max2}$ = 24 * db_t	$B_{s\_t\_max2}$ =	12	in
per pier diameter [ACI 25.7.2.1] & [ACI 18.4.3.3]	$B_{s\_t\_max3}$ = di / 4	$B_{s\_t\_max3}$ =	9	in
per seismic zone [ACI 25.7.2.1] & [ACI 18.4.3.3]	$B_{s\_t\_max4}$ = 12" in active seismic zones, else 18"	$B_{s\_t\_max4}$ =	12	in
	$B_{s\_t\_max}$ = MIN(Bs_t_max1, Bs_t_max2, Bs_t_max3, Bs_t_max4)	$B_{s\_t\_max}$ =	8	in
	$m_{\_t\_min}$ = (D - T + E) / Bs_t_max + 2	$m_{\_t\_min}$ =	11.4	
	Check $m_{\_t}$ = 12.0 >=	$m_{\_t\_min}$ =	11.4	**OK**

Anchor Steel:

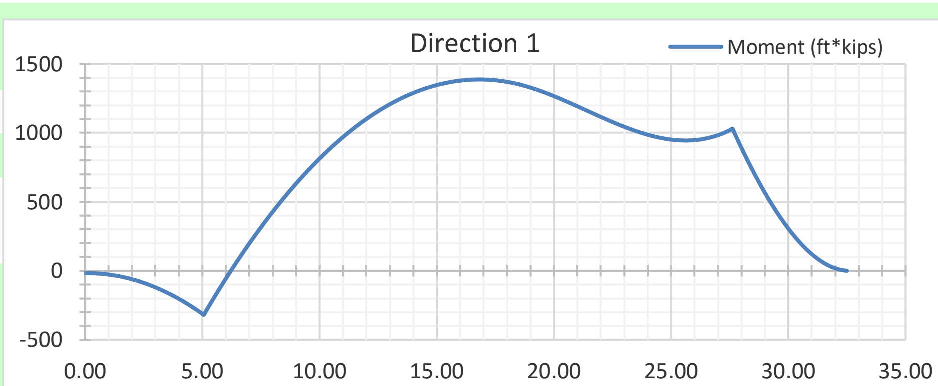
A/S parameters:	P_{as} = 113823; L_{as} = 80 in				
	d_{as} = 1.5 in; E_{as} = 71.50 in				
Development available:	L_{das} *per Anchor Bolts (see attached)*	L_{das} =	61.63	in	
Required development:	L_{das_min} *per Anchor Bolts (see attached)*	L_{das_min} =	34.88	in	
	Check L_{das} = 61.63 in >=	L_{das_min} =	34.88	in	**OK**
To bottom rebar grid:	E_{as_max} =D + E - cc - 2 * db_p	E_{as_max} =	91.25	in	
	Check E_{as} = 71.50 in <=	E_{as_max} =	91.25	in	**OK**
To top rebar grid:	rebar @ = D + E - T + cc	rebar @	78.00	in	
	Check 78 + 6 in >= E_{as} = 71.50 in **or**	<=	78 in		**OK**
Min. cage dia:	d_{o_min} *per ancsteel.xls (see attached)*	d_{o_min} =	24.25	in	
	Check d_o = 28.00 in >=	d_{o_min} =	24.25	in	**OK**

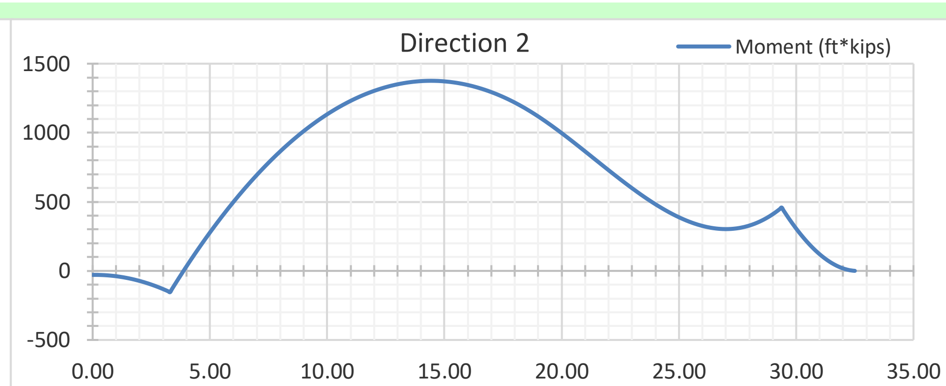
Pad Reactions:



Effective length in bearing:	14.12 ft
Effective length not bearing:	18.38 ft

Total Beam Length:	B_{L2_1} =W	B_{L2_1}=	32.5	ft
Location of Left Support:	S_{L2_1} =O	S_{L2_1}=	4.992	ft
Location of Right Support:	S_{R2_1} =W-O	S_{R2_1}=	27.51	ft
MDSolids Geometry Input (Option 2)				
Total Beam Length:	B_{L2_2} =W	B_{L2_2}=	32.5	ft
Location of Left Support:	S_{L2_2} =(W - w\) / 2	S_{L2_2}=	3.25	ft
Location of Right Support:	S_{R2_2} =S_{L1_2} + w\	S_{R2_2}=	29.25	ft





MDSolids Design Result				
Direction 1:	M_{max2_1} =M_{max2_1}	M_{max2_1}=	1386.78	ft*kips
Direction 2:	M_{max2_2} =M_{max2_2}	M_{max2_2}=	1376.01	ft*kips
Diagonal :	M_{max2_diag} =M_{max1_diag}	M_{max2_diag}=	2042.10	ft*kips
Max moment:	M_{maxp} =Max(Mmax2_1,Mmax2_2, Mmax2_diag)	M_{maxp}=	2042.10	ft*kips
Required moment: *φt = 0.9 [ACI 21.2.2.2]*	M_n = Mmaxp / φt	M_n =	2269.00	ft*kips

Pad Reinforcement:

	β = IF(F'c <= 4000, 0.85, IF(F'c >= 8000, 0.65, 0.85 - (F'c - 4000) * 0.05))	β =	0.825	
Effective width:	W_e = W	W_e =	32.500	ft
	A_{st_p}' = Mn / (0.9 * Fy * dc)	A_{st_p}' =	28.710	in^2
	a_p = Ast_p' * Fy / (β * F'c * We)	a_p =	1.19	in
Required steel:	$A_{st\_p\_st}$ = Mn / (Fy * (dc - ap / 2)) * (W / We)	$A_{st\_p\_st}$ =	26.745	in^2
Shrinkage:	ρ_{sh} = IF(Fy >= 60000, 0.0018, 0.002)	ρ_{sh} =	0.0018	
	$A_{st\_p\_sh}$ = ρsh * W * T / 2	$A_{st\_p\_sh}$ =	7.371	in^2
	A_{st_p} = MAX(Ast_p_st, Ast_p_sh, Ast_p_ste1, Ast_p_ste2)	A_{st_p} =	26.745	in^2

Rebar:	$s_{\_p}$ = 7	*Equally spaced, top and*	d_{b_p} =	0.875	in
	$m_{\_p}$ = 45	*bottom, both directions.*	A_{b_p} =	0.6	in^2

	A_{s_p} = Ab_p * m_p						A_{s_p} =	27.00	in^2	
		Check	A_{s_p} =	27.00	in^2	>=	A_{st_p} =	26.75	in^2	**OK**
Bar separation:	B_{s_p} = (W - 2 * cc - db_p) / (m_p - 1) - db_p						B_{s_p} =	7.83	in	
		Check	17.13	>=	B_{s_p} =	7.83	in	>=	4"	**OK**

Pad Development Length:

Reinforcement location: *[ACI 25.4.2.4]*	ψ_{t_p} = if the space under the rebar > 12 in, use 1.3, else use 1.0	ψ_{t_p} =	1.3	
Epoxy coating: *[ACI 25.4.2.4]*	ψ_{e_p} = if epoxy-coated bars are not used, use 1.0; but if epoxy-coated bars are used, then if Bs < 6 * db or cc < 3 * db, use 1.5, else 1.2	ψ_{e_p} =	1.0	
Max term: *[ACI 25.4.2.4]*	$\psi_t\psi_{e\_p}$ = the product of ψt & ψe, need not be taken larger than 1.7	$\psi_t\psi_{e\_p}$ =	1.3	
Reinforcement size: *[ACI 25.4.2.4]*	ψ_{s_p} = if the bar size is 6 or less, then use 0.8, else use 1.0	ψ_{s_p} =	1	
Light weight concrete: *[ACI 25.4.2.4]*	$\lambda_{\_p}$ = if lightwieght concrete is used, 0.75, else use 1.0	$\lambda_{\_p}$ =	1.0	
Spacing/cover: *[ACI 25.4.2.4]*	$c_{\_p}$ = the smaller of: half the bar spacing or the concrete edge distace	$c_{\_p}$ =	3.44	in
Transverse bars: *[ACI 25.4.2.3]*	k_{tr_p} = 0 in (per simplification)	k_{tr_p} =	0	in
Max term: *[ACI 25.4.2.3]*	c_p' = MIN(2.5, (c_p + ktr_p) / db_p)	$c_{\_p}$' =	2.500	
Excess reinforcement: *[ACI 25.4.10.1]*	R_p = 1 *(excess reinforcement reduction is not used)*	$R_{\_p}$ =	1.00	
Development (tensile): [ACI 25.4.2.2]	L_d = (3 / 40) * (Fy / λ_p * √(F'c)) * ψtψe_p * ψs_p * R_p * db_p / c_p'_u	L_{dp}' =	30.5	in
Minimum length: *[ACI 25.4.2.1]*	L_{d_min} = 12 inches	L_{d_min} =	12.0	in
Development length:	L_{dp} = MAX(Ld_min, Ldp')	L_{dp} =	30.5	in
Length available in pad:	L_{pad} = (W / 2 - w' / 2) - cc	L_{pad} =	36.0	in

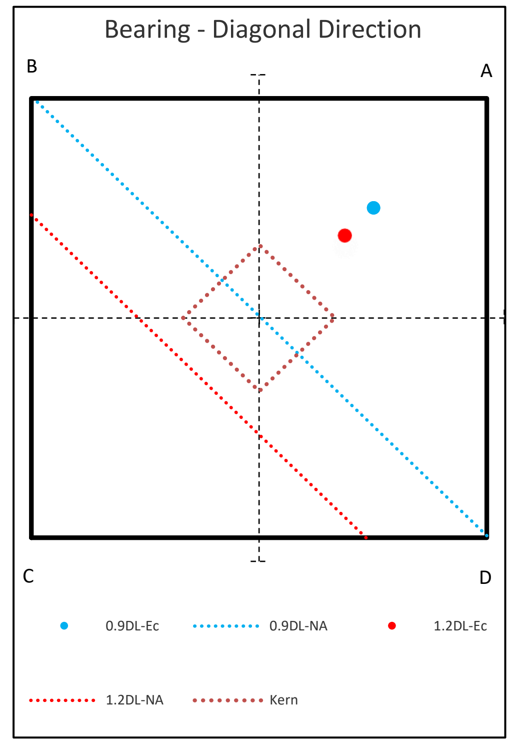
Check	L_{pad} =	36.00	in	>=	L_{dp} =	30.52	in	**OK**

UNIT BASE FOUNDATION DIAGONAL BEARING CHECK

Parellel Infrastructure **U- 26.0 250**

PIWA513- Darrington-Christian Camp Road, W **A- 540603**

		Load Case - DL 1.2	Load Case - DL 0.9	
Moment of Inertia of Mat	MOI	92972.01	92972.01	ft^4
Total Factored Weight	P'	1142.76	857.07	kips
Load Eccentricity	e	8.66	11.54	ft
Bearing at Corner A	B_{c_a}	3.53	3.26	ksf
Bearing at Corner B	B_{c_b}	1.08	0.81	ksf
Bearing at Corner C	B_{c_c}	-1.36	-1.63	ksf
Bearing at Corner D	B_{c_d}	1.08	0.81	ksf
Initial Location of Neutral Axis from C	$NA_{\_c\_ini}$	12.82	15.36	ft
Calculated Location of Neutral Axis from C	$NA_{\_c\_cal}$	16.89	23.09	ft
MOI for Effective Bearing Area	MOI	118663.65	45614.28	ft^4
Distance to Point Load from NA	$L_{p'}$	14.75	11.44	ft
Effective Length in Bearing along AB & AD	W_{eff}	32.50	32.35	ft
Total Vol.	Vol_{tot}	1142.76	857.07	kips
Difference		0.0000	0.0001	kips
		ok	ok	
Adjusted Bearing at A	$B_{c\_a\_adj}$	4.1312	4.9148	ksf
Adjusted Bearing at B & D	$B_{c\_bd\_adj}$	0.87	0.00	ksf
Maximum Diagonal Bearing Pressure	$B_{c\_dia\_max}$	4.1312	4.9148	ksf
Bearing Available	$B_c * \varphi r$	5.6250	5.6250	ksf
Check		**OK**	**OK**	



THIS SPREADSHEET IS SET UP FOR A MAXIMUM OF 56 BARS.
MAXIMUM FACTORED MOMENT OF A CIRCULAR SECTION

Loading (negative for compression)		
Axial load =	394.45	kips

Foundation		
Concrete		
Pier diameter =	3.00	ft
Pier area =	1017.9	in^2
Reinforcement		
Clear cover =	3.00	in
Cage diameter =	2.33	ft
Bar size =	8	
Bar diameter =	1.000	in
Bar area =	0.785	in^2
Number of bars =	13	

Material Strengths			
Concrete compressive strength =	4500	psi	
Reinforcement yield strength =	60000	psi	
Modulus of elasticity =	29000	ksi	
Reinforcement yield strain =	0.00207		(per ACI 10.3.5 - OK)
Limiting compressive strain =	0.003		

458.04

Seismic	
SDC =	D
Are hooks required?	yes

Minimum Area of Steel

Required area of steel =	10.18	in^2	
Actual area of steel =	10.21	in^2	OK
Bar spacing =	6.25	in	

Axial Loading

Load factor =	1.00		
Reduction factor =	0.65575	(per ACI 9.3.1 & 2)	0.6557471
Factored axial load =	394.45	kips	

Neutral Axis

Distance from extreme edge to neutral axis =	3.65	in	
Equivalent compression zone factor =	0.825		(per ACI 10.2.7.3)
Distance from extreme edge to Equivalent compression zone factor =	3.01	in	
Distance from centroid to neutral axis =	14.35	in	

Compression Zone

Area of steel in compression zone =	0.00	in^2	
Angle from centroid of pier to intersection of equivalent compression zone and edge of pier =	33.63	deg	
Area of concrete in compression =	40.76	in^2	40.759193
Force in concrete = 0.85 * f`c * (Acc - steel in comp zone) =	155.90	kips	(per ACI 10.3.6.2)
Total reinforcement forces =	-550.35	kips	
Factored axial load =	394.45	kips	
Force in concrete =	-155.90	kips	
Sum of the forces in concrete =	0.00	kips	OK

Maximum Moment

First moment of the concrete area in compression about the centoid =	660.37	in^3
Distance between centroid of concrete in compression and centroid of pier =	16.20	in

Moment of concrete in compression =	2525.91	in-kips		
Total reinforcement moment =	846.25	in-kips		
Nominal moment strength of column =	3372.16	in-kips		
Factored moment strength of column =	2211.28	in-kips	184.27	ft-kips

Maximum allowable moment of the pier =	**184.27**	**ft-kips**

Individual Bars

Bar #	Angle from first bar (deg)	Distance to centroid (in)	Distance to neutral axis (in)	Distance to equivalent comp. zone (in)	Strain	Area of steel in compression (in^2)	Axial force (kips)	Moment (in-kips)
1	0.00	0.00	-14.35	-14.99	-0.01179	0.00	-47.12	0.00
2	27.69	6.51	-7.84	-8.48	-0.00644	0.00	-47.12	-306.59
3	55.38	11.52	-2.83	-3.47	-0.00232	0.00	-47.12	-542.95
4	83.08	13.90	-0.45	-1.09	-0.00037	0.00	-8.44	-117.24
5	110.77	13.09	-1.26	-1.90	-0.00103	0.00	-23.55	-308.29
6	138.46	9.28	-5.06	-5.70	-0.00416	0.00	-47.12	-437.48
7	166.15	3.35	-11.00	-11.64	-0.00904	0.00	-47.12	-157.88
8	193.85	-3.35	-17.70	-18.34	-0.01454	0.00	-47.12	157.88
9	221.54	-9.28	-23.63	-24.27	-0.01942	0.00	-47.12	437.48
10	249.23	-13.09	-27.44	-28.08	-0.02254	0.00	-47.12	616.86
11	276.92	-13.90	-28.25	-28.89	-0.02321	0.00	-47.12	654.92
12	304.62	-11.52	-25.87	-26.51	-0.02126	0.00	-47.12	542.95
13	332.31	-6.51	-20.85	-21.49	-0.01713	0.00	-47.12	306.59

DEVELOPMENT LENGTH CHECK OF PIER REINFORCEMENT

Foundation:	Pier diameter =	3.0	ft	Cover between side of pier and cage =	3.00	in.
	Cage diameter =	2.33	ft	Cover between top of pier and cage =	3.00	in.
	Rebar size =	8		Compressive strength of concrete =	4500	psi
	Number of bars =	13		Rebar yield strength =	60000	psi
	Clear spacing =	5.77	in.			
	Are there hooks?	n				
	Check Compression?	n				
Anchor Steel:	Part number:	113823				
	Embedment length =	71.5	in.			
	Bolt Diameter =	1.5				
Anchor Plate:	Part number:	281260				
	Plate width =	18.25	in.			
	Required development length (compression) =	999.00	in.	Min. Anchor Bolt Embedment per TIA-222-H 9.6 =	15	in
	Required development length (tension) =	34.88	in.	Actual Anchor Bolt Embedment =	68.5	in
	Available development length =	61.625	in.			

OK

The length available in the pier for the development of the vertical reinforcement exceeds the required length (ACI 318-14, section 25.4).

CHECK EMBEDMENT PLATE CLEARANCE IN THE PIER

Foundation:	Pier diameter =	3.0	ft	Cover between side of pier and cage =	3.00	in.
	Cage diameter =	2.333333	ft	Minimum cover between A/S and cage =	3.00	in.
Anchor Steel:	Part number:	113823		Angle of anchor steel in foundation =	0	degrees
	Embedment length =	71.5	in.			
Anchor Plate:	Part number:	281260				
	Largest plate width =	18.25	in.			
	Bolt Diameter =	1.5	in.			
	Minimum cage diameter =	24.25	in.			
	Actual cage diameter =	28	in.			

OK

The available space exceeds the minimum cage diameter required for anchor steel installed in the pier at an angle.