



STRUCTURAL CALCULATIONS

FOR

Hardy Tie Back Device

HTB-16

SDS Connection

PREPARED FOR:

Hardy Fall Protection Systems, Inc.



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DESIGN CRITERIA AND ASSUMPTIONS

BUILDING CODES AND MATERIAL STANDARDS

STRUCTURAL DESIGN MEETS OR EXCEEDS PROVISIONS OF THE FOLLOWING BUILDING CODES AND MATERIAL STANDARDS

2018 IBC	CALIFORNIA BUILDING CODE
2019 IRC	CALIFORNIA RESIDENTIAL CODE
ASCE 7-16	MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES
AISC 360-16	STEEL CONSTRUCTION MANUAL, FOURTEENTH EDITION
AISC 341-16	SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS
ACI 318-14	BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE
AWS D1.1 / D1.1M 2015	STRUCTURAL WELDING CODE

MATERIAL SPECIFICATIONS

UNLESS OTHERWISE NOTED ON THE DRAWINGS, MATERIALS SHALL CONFORM TO THE FOLLOWING SPECIFICATIONS

1) **STRUCTURAL STEEL:**

STRUCTURAL STEEL SHALL CONFORM TO THE ASTM DESIGNATION AS FOLLOWS:

W SHAPE	ASTM A992	$F_y =$	50 ksi
PIPE	ASTM A53 - Gr. B	$F_y =$	35 ksi
RECTANGULAR HSS	ASTM A500 - Gr. B	$F_y =$	46 ksi
CIRCULAR HSS	ASTM A500 - Gr. B	$F_y =$	42 ksi
ANGLES	ASTM A36	$F_y =$	36 ksi
CHANNELS	ASTM A36	$F_y =$	36 ksi
STEEL PLATES	ASTM A572 GRADE 50	$F_y =$	50 ksi

2) **CONNECTIONS:**

BOLTS	ASTM A325 - N
WELDS	E70XX

3) **CONCRETE:**

CONCRETE USED FOR FOUNDATION SHALL DEVELOP A MINIMUM COMPRESSIVE STRENGTH OF 2500 psi IN 28 DAYS"

4) **REINFORCING STEEL:**

REINFORCING STEEL SHALL CONFORM TO ASTM A615	$F_y =$	60 ksi
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5) **ANCHORS:**

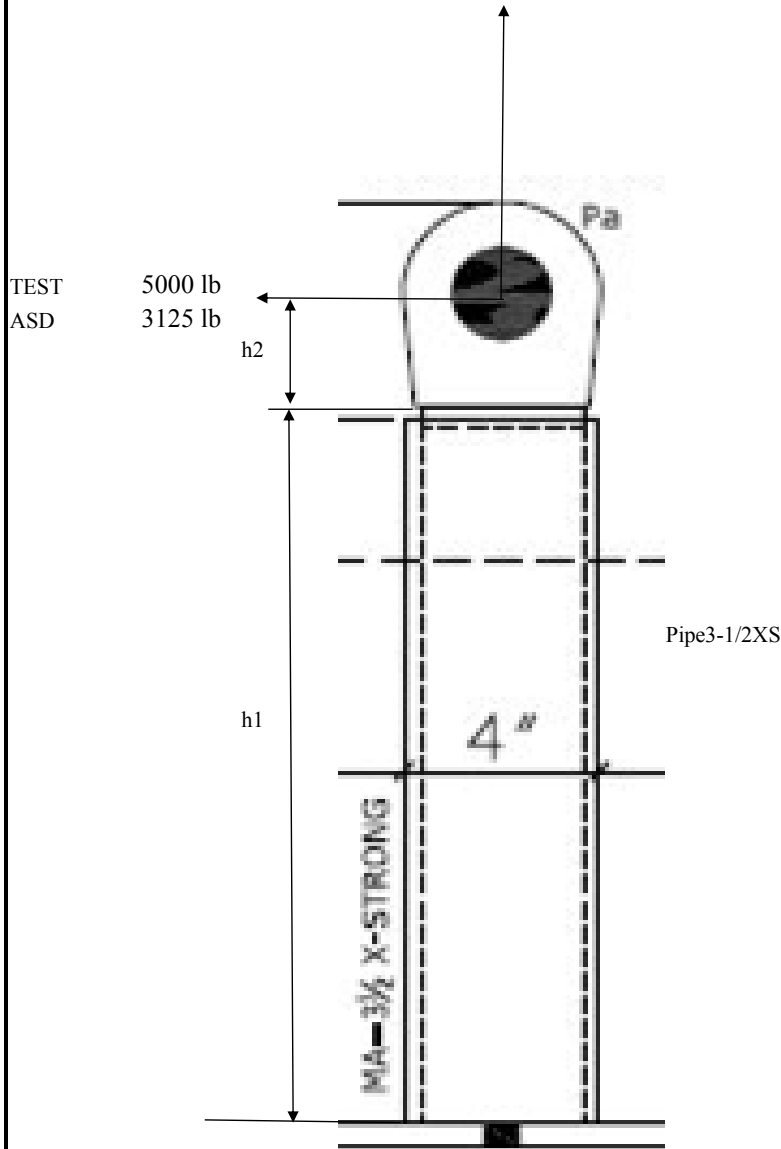
ANCHOR RODS	ASTM F1554 Gr. 36	$F_y =$	36 ksi
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HARDY TIE BACK DEVICE HORIZONTAL LOADING

LOADING CRITERIA:

	TEST LOADING	ALLOWABLE STRESS DESIGN LOADING
VERTICAL LOAD =	5.00 kip	3.125 kip
HORIZONTAL LOAD =	5.00 kip	3.125 kip
RUN CALCULATIONS FOR:		ALLOWABLE STRESS DESIGN LOADING
POST HEIGHT h_1 =		16.00 in
HEIGHT FROM TOP OF POST TO CENTER OF EYELET h_2 =		2.50 in
TOTAL HEIGHT OF HORIZONTAL LOAD FROM BASE OF POST =		18.50 in
VERTICAL LOAD =		3.13 kip
HORIZONTAL LOAD =		3.13 kip
MAXIMUM SHEAR AT POST BASE F_{max} =		3.13 kip
MAXIMUM MOMENT AT POST BASE M_{max} =		57.81 kip-in

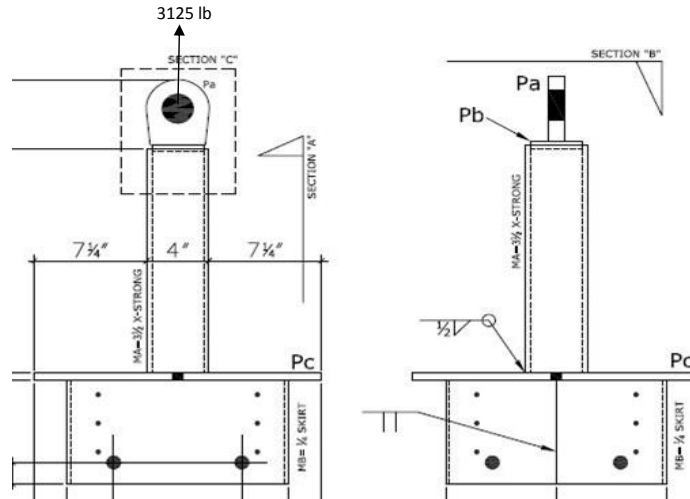
TEST 5000 lb
ASD 3125 lb



CHECK OF EYELET PLATE AS TENSION MEMBER

BY PROVISIONS OF ANSI/AISC 360-16

(STEEL CONSTRUCTION MANUAL- FIFTEENTH EDITION)



INPUT

GENERAL:	MEMBER ID	EYELET PLATE			
STRAP INPUT:	PLATE THICKNESS	1	in	PLATE WIDTH:	3.50 in
	ASTM SPECIFICATION	ASTM A36		SHEAR LAG FACTOR U =	1.00
	HOLE DIAMETER =	2	in	NUMBER OF HOLES IN STRAP PLATE SECTION n =	1
	ADDED WIDTH FOR HOLES =	0	in	NEGLECTED WIDTH DUE TO HOLE PUNCHING =	1/16 in

CALCULATIONS

EYELET PLATE STRENGTH IN TENSION

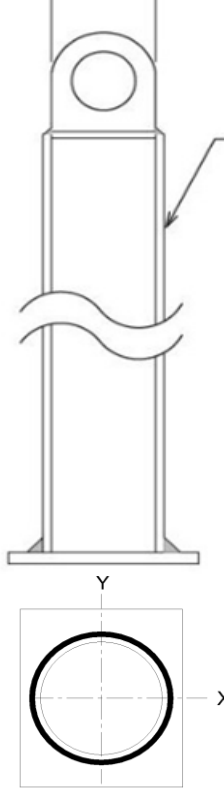
GROSS, NET AND EFFECTIVE AREA:	PLATE THICKNESS t =	1.000 in
	GROSS AREA OF MEMBER A_g =	3.500 sq.in
	HOLE DIAMETER =	2 1/16 in
	NET AREA OF MEMBER A_n =	1.438 sq.in
	EFFECTIVE NET AREA $A_e = A_n U$ =	1.438 sq.in
MATERIAL PROPERTIES:	TENSION MEMBER YIELD STRESS F_y =	36 ksi
	TENSION MEMBER ULTIMATE TENSILE STRESS F_u =	58 ksi
TENSILE YIELDING IN THE GROSS SECTION		
AISC 360-16 EQUATION D2-1	$P_n = F_y A_g$ =	126.000 kip
AISC 360-16 EQUATION D2-1	$\Omega =$	1.67
	DESIGN TENSILE STRENGTH ΦP_n OR ALLOWABLE TENSILE STRENGTH P_n/Ω =	75.449 kip
TENSILE RUPTURE IN THE NET SECTION		
AISC 360-16 EQUATION D2-2	$P_n = F_u A_e$ =	83.375 kip
AISC 360-16 EQUATION D2-2	$\Omega =$	2.00
	DESIGN TENSILE STRENGTH ΦP_n OR ALLOWABLE TENSILE STRENGTH P_n/Ω =	41.688 kip
TENSILE RUPTURE IN THE NET SECTION GOVERNS,		
	DESIGN TENSILE STRENGTH ΦP_n OR ALLOWABLE TENSILE STRENGTH P_n/Ω =	41.688 kip
CHECK EYELET PLATE STRENGTH IN TENSION:		
	AVAILABLE TENSILE STRENGTH P_n/Ω =	41.688 kip
	REQUIRED TENSILE STRENGTH =	3.125 kip
	RATIO OF REQUIRED STRENGTH/ AVAILABLE STRENGTH =	0.075
		OK

CHECK OF HTB STEEL POST STRENGTH				
BY PROVISIONS OF AISC 360-16				
MEMBER INPUT				
SECTION INPUT:	MEMBER ID	HTB-16	SHAPE:	PIPE
	SECTION:	Pipe3-1/2XS	SELECT PREFERRED SPECIFICATION	ASTM A53 Grade B
EFFECTIVE LENGTH FOR DESIGN FOR COMPRESSION: AISC 360-16 SECTION E2			FOR X AXIS	FOR Y AXIS
LATERALLY UNBRACED LENGTH L			1.54 ft	1.54 ft
K			2.00	2.00
AISC 360-16 SECTION (B4-2)		USE DESIGN WALL THICKNESS = 0.93 NOMINAL WALL THICKNESS?	YES	
DISTANCE FROM MAXIMUM TO ZERO SHEAR FORCE:			$L_v =$	1.54 ft
SUMMARY OF RESULTS				
AVAILABLE STRENGTH OF SECTION:		LRFD		ASD
AVAILABLE COMPRESSIVE STRENGTH:		$\Phi_c P_n =$	104.098 kips	$P_n / \Omega_c =$ 69.260 kips
AVAILABLE TENSILE STRENGTH:		$\Phi_t P_n =$	108.411 kips	$P_n / \Omega_t =$ 72.130 kips
AVAILABLE FLEXURAL STRENGTH:		$\Phi_b M_n =$	10.675 kip-ft	$M_n / \Omega_b =$ 7.102 kip-ft
AVAILABLE SHEAR STRENGTH:		$\Phi_v V_n =$	32.523 kips	$V_n / \Omega_v =$ 21.639 kips
CALCULATIONS				
MATERIAL PROPERTIES				
YOUNG'S MODULUS $E_c =$		29000 ksi		
$F_y =$		35 ksi		
$F_u =$		60 ksi		
SECTION PROPERTIES				
OUTSIDE DIAMETER $D =$		4 in		
NOMINAL WALL THICKNESS $t_{nom} =$		0.318 in		
DESIGN WALL THICKNESS $t_{des} =$		0.296 in		
CROSS SECTION AREA $A =$		3.442 sq.in		
$D/t =$		13.53		
MOMENT OF INERTIA I (in ⁴) =		5.94		
SECTION MODULUS $S =$		2.97		
RADIUS OF GYRATION $r =$		1.314 cu.in		
PLASTIC SECTION MODULUS $Z =$		4.067 in		
AVAILABLE COMPRESSIVE STRENGTH				
1- CLASSIFICATION OF SECTION FOR UNIFORM COMPRESSION: (AISC 360-16 TABLE B.4.1a)				
$D/t =$		13.53		
$\lambda_p =$		N/A		
$\lambda_r = 0.11 E/F_y =$		91.143		
CLASSIFICATION FOR UNIFORM COMPRESSION:		NONCOMPACT		
2- SLENDERNESS RATIO: AISC 360-16 SECTION E2				
$(L_c/r)_x = (KL/r)_x =$		28.16		
$(L_c/r)_y = (KL/r)_y =$		28.16		
$(L_c/r)_{max} = (KL/r)_{max} =$		28.16		
		≤ 200 OK		
ELASTIC CRITICAL BUCKLING STRESS F_c				
AISC 360-16 EQUATION (E3-4)		$F_c = \pi^2 E / (L_c / r)^2 =$	360.883 ksi	
MEMBERS WITHOUT SLENDER ELEMENTS BY PROVISIONS OF AISC 360-16 SECTION E3 (ASD)				
LIMIT STATE OF FLEXURE BUCKLING		$4.71 \sqrt{E/F_y} =$	135.58	
AISC 360-16 EQUATION (E3-2)		$F_{cr} = [0.658^{F_y/F_c}] F_y =$	33.608 ksi	
AISC 360-16 EQUATION (E3-3)		$F_{cr} = 0.877 F_c =$	N/A	
		$F_{cr} =$	33.608 ksi	
AVAILABLE COMPRESSIVE STRENGTH PROVISIONS OF AISC 360-16 SECTION E3, "MEMBERS WITHOUT SLENDER ELEMENTS" APPLY				
		$F_{cr} =$	33.608 ksi	
AISC 360-16 EQUATION (E3-1)		$P_n = F_{cr} A_g =$	115.66 kips	
		LRFD		ASD
AISC 360-16 SECTION E1		$\Phi_c =$	0.9	$\Omega_c =$ 1.67
AVAILABLE COMPRESSIVE STRENGTH:		$\Phi_c P_n =$	104.098 kips	$P_n / \Omega_c =$ 69.26 kips

AVAILABLE TENSILE STRENGTH					
(ASSUMING $A_g = A_n = A_e$)		$A_g = A_n = A_e =$	3.442 sq.in		
TENSILE YIELDING IN THE GROSS SECTION: AISC 360-16 EQUATION (D2-1)		$P_n = F_y A_g =$	120.46 kips		
AISC 360-16 SECTION D2. (a)		LRFD		ASD	
		$\Phi_t =$	0.9	$\Omega_t =$	1.67
AVAILABLE TENSILE STRENGTH:		$\Phi_t P_n =$	108.411 kips	$P_n / \Omega_t =$	72.13 kips
TENSILE RUPTURE IN THE NET SECTION: AISC 360-16 EQUATION (D2-2)		$P_n = F_u A_e =$	206.50 kips		
AISC 360-16 SECTION D2. (b)		LRFD		ASD	
		$\Phi_t =$	0.75	$\Omega_t =$	2.00
AVAILABLE TENSILE STRENGTH:		$\Phi_t P_n =$	154.872 kips	$P_n / \Omega_t =$	103.25 kips
TENSILE YIELDING IN THE GROSS SECTION GOVERNS		LRFD		ASD	
AVAILABLE TENSILE STRENGTH:		$\Phi_t P_n =$	108.411 kips	$P_n / \Omega_t =$	72.13 kips
AVAILABLE FLEXURE STRENGTH					
CLASSIFICATION OF SECTION FOR LOCAL BUCKLING IN FLEXURE: (AISC 360-16 TABLE B.4.1b)					
		$\lambda = D/t =$	13.53		
		$\lambda_p = 0.07 E/F_y =$	58.000 cu.in		
		$\lambda_r = 0.31 E/F_y =$	256.857 in		
SECTION CLASSIFICATION FOR LOCAL BUCKLING IN FLEXURE:		COMPACT			
LIMIT STATE OF YIELDING:					
CHECK $D/t < 0.45 E/F_y$:		13.52539393	<	372.86	
AISC 360-16 EQUATION (F8-1)		$M_n = M_p = F_y Z =$	142 kip-in		
LIMIT STATE OF FLANGE LOCAL BUCKLING:					
FOR NONCOMPACT SECTIONS:					
AISC 360-16 EQUATION (F8-2)		$M_n = (0.021 E / (D/t) + F_y) S$	N/A		
FOR SLENDER SECTIONS:					
AISC 360-16 EQUATION (F8-4)		$F_{cr} = 0.33 E / (D/t)$	N/A		
AISC 360-16 EQUATION (F8-3)		$M_n = F_{cr} S$	N/A		
DESIGN FLEXURE STRENGT THE LIMIT STATE OF YIELDING GOVERNS,					
		NOMINAL FLEXURAL STRENGTH OF THE SECTION $M_n =$	142 kip-in		
AISC 360-16 SECTION F1		LRFD		ASD	
		$\Phi_b =$	0.9	$\Omega_b =$	1.67
AVAILABLE FLEXURAL STRENGTH:		$\Phi_b M_n =$	128.099 kip-in	$M_n / \Omega_b =$	85.229 kip-in
AVAILABLE SHEAR STRENGTH					
		$L_v =$	1.54 ft		
AISC 360-16 SECTION G6 F_{cr} IS THE LARGER OF:					
AISC 360-16 EQUATION (G5-2a)		$F_{cr} = 1.60 E / \sqrt{(L_v / D) (D/t)^{5/4}} =$	831.811 ksi		
AISC 360-16 EQUATION (G5-2b)		$F_{cr} = 0.78 E / (D/t)^{3/2} =$	454.745 ksi		
		F_{cr} UPPER LIMIT $= 0.6 F_y =$	21.000 ksi		
		$F_{cr} =$	21.000 ksi		
AISC 360-16 EQUATION (G5-1)		NOMINAL SHEAR STRENGTH $V_n = F_u A_g / 2 =$	36.14 kips		
AISC 360-16 SECTION G1		LRFD		ASD	
		$\Phi_v =$	0.9	$\Omega_v =$	1.67
AVAILABLE SHEAR STRENGTH:		$\Phi_v V_n =$	32.523 kips	$V_n / \Omega_v =$	21.64 kips
CHECK POST STRENGTH					
CHECK POST STRENGTH FOR ALLOWABLE STRESS DESIGN LOADING					
	REQUIRED	AVAILABLE	REQUIRED / AVAILABLE	CHECK	
TENSILE STRENGTH:	3.13 kip	72.13 kip	0.043324848	OK	
SHEAR STRENGTH:	3.13 kip	21.64 kip	0.144416159	OK	
FLEXURAL STRENGTH:	57.81 kip-in	85.23 kip-in	0.678321248	OK	

CHECK OF FILLET WELD AT POST BASE (ASD)
BY PROVISIONS OF ANSI/AISC 360-16

INPUT



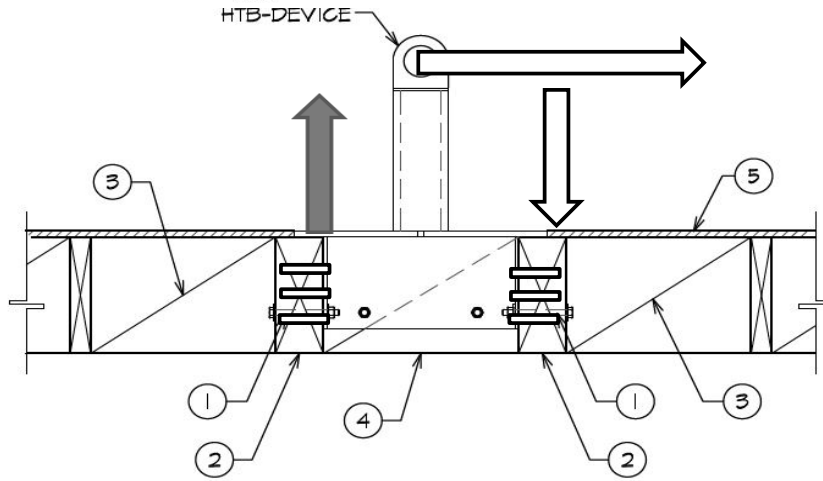
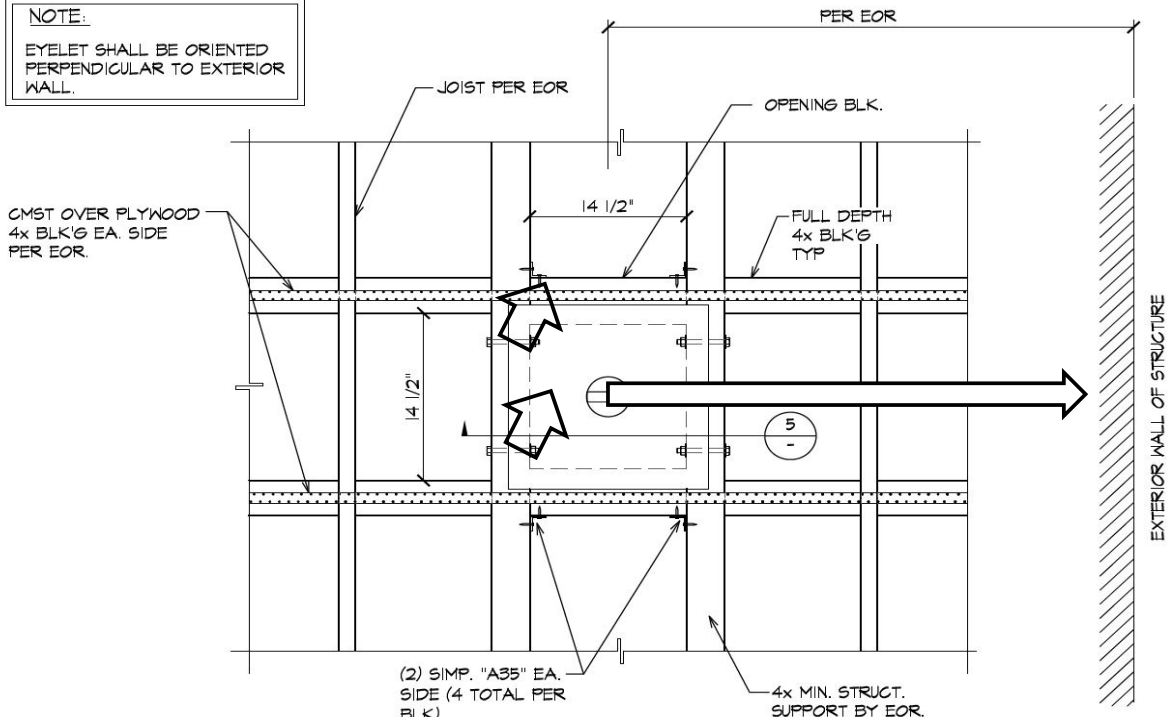
POST:		SECTION:	Pipe3-1/2XS
		ASTM SPECIFICATION	ASTM A53 Grade B
BASE PLATE:	THICKNESS t =	0.500 in	ASTM SPECIFICATION
			ASTM A36
FILLET WELD PROPERTIES:		ELECTRODE CLASSIFICATION	E70XX
		FILLET WELD LEG SIZE AT FLANGE =	1/2 in
APPLIED LOADS		APPLIED AXIAL FORCE F_z =	3.125 kip
		APPLIED SHEAR FORCE F_x =	3.125 kip
		APPLIED SHEAR FORCE F_y =	0.000 kip
		APPLIED BENDING MOMENT M_x =	0.000 kip-in
		APPLIED BENDING MOMENT M_y =	57.813 kip-in
		APPLIED BENDING MOMENT M_z =	0.000 kip-in

Hardy Tie Back HTB 16 SDS Connection 200505
POST BASE WELD (P)

CALCULATIONS		
POST PROPERTIES:	OUTSIDE DIAMETER D =	4.000 in
	t_{des} =	0.296 in
	F_y =	35 ksi
	F_u =	60 ksi
BASE PLATE PROPERTIES:	F_y =	36 ksi
	F_u =	58 ksi
FILLET WELD PROPERTIES:	AISC 360-16 TABLE J2.5 $F_y = 0.6 F_{EXX}$ =	42.0 ksi
	TOTAL LENGTH OF WELD L =	13.68 in
	$I_x = I_y = \pi d_1^4 / 64 - \pi d_2^4 / 64$ =	11.53 in ⁴
	$C_x = C_y$ =	2.35 in
	$S_x = S_y$ =	4.90 cu.in
AISC 360-16 TABLE J2.4	MINIMUM WELD SIZE =	3/16 in
	CHECK PROVIDED WELD SIZE \geq MINIMUM ALLOWABLE	OK
	EFFECTIVE LENGTH	13.68 in
	EFFECTIVE THROAT	0.35 in
	EFFECTIVE AREA A_w =	4.84 sq.in
WELD REQUIRED STRENGTH		
1- SHEAR STRESSES:		
	SHEAR STRESS DUE TO F_x $r_{Fax} = F_x / A_e$ =	0.646 ksi
	SHEAR STRESS DUE TO F_y $r_{Fay} = F_y / A_e$ =	0.000 ksi
MOST CRITICAL SHEAR STRESS DUE TO TORSION M_z	$r_{Max} = M_z C_y / I_p$ =	0.000 ksi
	$r_{May} = M_z C_x / I_p$ =	0.000 ksi
RESULTANT SHEAR STRESS:	$r_a = \sqrt{(r_{Fax} + r_{Max})^2 + (r_{Fay} + r_{May})^2}$	0.646 ksi
2- TENSION STRESSES:		
	TENSION STRESS DUE TO F_z $r_{Faz} = F_z / A_e$ =	0.646 ksi
	CRITICAL TENSILE STRESS DUE TO M_x $r_{Max} = M_x C_y / I_x$ =	0.000 ksi
	CRITICAL TENSILE STRESS DUE TO M_y $r_{May} = M_y C_x / I_y$ =	11.799 ksi
	$r_a = r_{Faz} + r_{Max} + r_{May}$ =	12.445 ksi
3- COMBINING SHEAR AND TENSILE STRESSES INTO RESULTANT SHEAR STRESS:		
	$r_a = \sqrt{r_{a\ SHEAR}^2 + r_{a\ TENSION}^2}$	12.462 ksi
AISC 360-16 TABLE J2.5	WELD NOMINAL STRESS $F_{nw} = 0.6 F_{EXX}$ =	42.00 ksi
AISC 360-16 TABLE J2-5	Ω =	2
WELD AVAILABLE STRESS =	WELD AVAILABLE STRESS = F_{nw} / Ω =	21.000 ksi
	RATIO OS REQUIRED STRENGTH/ AVAILABLE STRENGTH =	0.593
	REQUIRED STRENGTH \leq AVAILABLE STRENGTH	OK

CHECK OF HTB ATTACHMENT (CASE 1)

NOTE:
EYELET SHALL BE ORIENTED PERPENDICULAR TO EXTERIOR WALL.



A-A ATTACHMENT OPTION #2

1. (2) 7/8"Ø THRU-BOLTS INTO EA. STRUCTURAL SUPPORT MEMBERS PER MANUFACTURER (2 EA. SIDE, 4 TOTAL)
2. STRUCTURAL SUPPORT MEMBER BY EOR, 4x MIN.
3. MIN. 4x BLK'G w/ STRAP PER EOR.
4. MIN. 4x OPENING BLK'G w/ (2) SIMP. "A35" EA. SIDE TO STRUCTURAL SUPPORT MEMBER.
5. ROOF SHEATHING PER DESIGN PROFESSIONAL

POST HEIGHT h ₁ =	16.00 in
HEIGHT FROM TOP OF POST TO CENTER OF EYELET h ₂ =	2.50 in
TOTAL HEIGHT OF HORIZONTAL LOAD FROM BASE OF POST =	18.50 in
VERTICAL LOAD =	3.13 kip
HORIZONTAL LOAD =	3.13 kip
MAXIMUM MOMENT AT POST BASE M _{max} =	57.81 kip-in
HORIZONTAL DISTANCE BETWEEN BEARING / UPLIFT ON 6X STRUCTURAL MEMBERS =	14.25 in
BEARING / UPLIFT ON 6X STRUCTURAL MEMBERS =	4.057 kip

HARDY TIE BACK DEVICE UPLIFT CHECK						
		TYPE OF FASTENERS	SDS 3"x1/4"			
IAPMO UES 461 SECTION 4.2.2		MINIMUM MAIN MEMBER THICKNESS =	3 1/2 in			
IAPMO UES 461 SECTION 4.2.2		MAIN MEMBER SPECIFIC GRAVITY =	0.50 in			
ICC ESR-2236 TABLE 2		REFERENCE SHEAR DESIGN VALUE Z =	420 lb			
GROUP ACTION FACTOR C_g	NDS 2018 SECTION 11.3.6	ASSUMING A MINIMUM STRUCTURAL MEMBER 4X10,	A _m =	40.00 sq.in		
			A _s = 10 X 1/4 =	2.50 sq.in		
			A _m / A _s =	16		
GROUP 1:		NUMBER OF FASTENER ROWS =	2			
		NUMBER OF FASTENERS IN A ROW =	3			
NDS 2018 TABLE 11.3.6C		GROUP ACTION FACTOR C _g =	0.987			
GROUP 2:		NUMBER OF FASTENER ROWS =	2			
		NUMBER OF FASTENERS IN A ROW =	3			
NDS 2018 TABLE 11.3.6C		GROUP ACTION FACTOR C _g =	0.987			
Adjustment Factors per NDS 2018 Table 11.3.1						
		C _D	C _M	C _t	C _g	C _Δ
		1.5	1	1	VARRIES	1
GROUP 1:		SHEAR DESIGN VALUE OF ONE FASTENER Z' =				622 lb
		SHEAR DESIGN VALUE OF ONE ROW OF FASTENERS =				1865 lb
		SHEAR DESIGN VALUE OF GROUP =				3730 lb
GROUP 2:		SHEAR DESIGN VALUE OF ONE FASTENER Z' =				622 lb
		SHEAR DESIGN VALUE OF ONE ROW OF FASTENERS =				1865 lb
		SHEAR DESIGN VALUE OF GROUP =				3730 lb
		UPLIFT STRENGTH = SHEAR DESIGN VALUE OF CONNECTION =				7459 lb
		REQUIRED UPLIFT STRENGTH =				4057 lb
		RATIO OF REQUIRED STRENGTH/ AVAILABLE STRENGTH =				0.544
						OK