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STRUCTURAL CALCULATIONS

FOR

Hardy Fall Protection Saddle

HTB-S12

SDS Connection

PREPARED FOR:

Hardy Fall Protection Systems, Inc.

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STRUCTURAL CALCULATIONS INDEX

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
2018 NDS	NATIONAL DESIGN SPECIFICATION FOR WOOD CONSTRUCTION
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

5) **ANCHORS:**

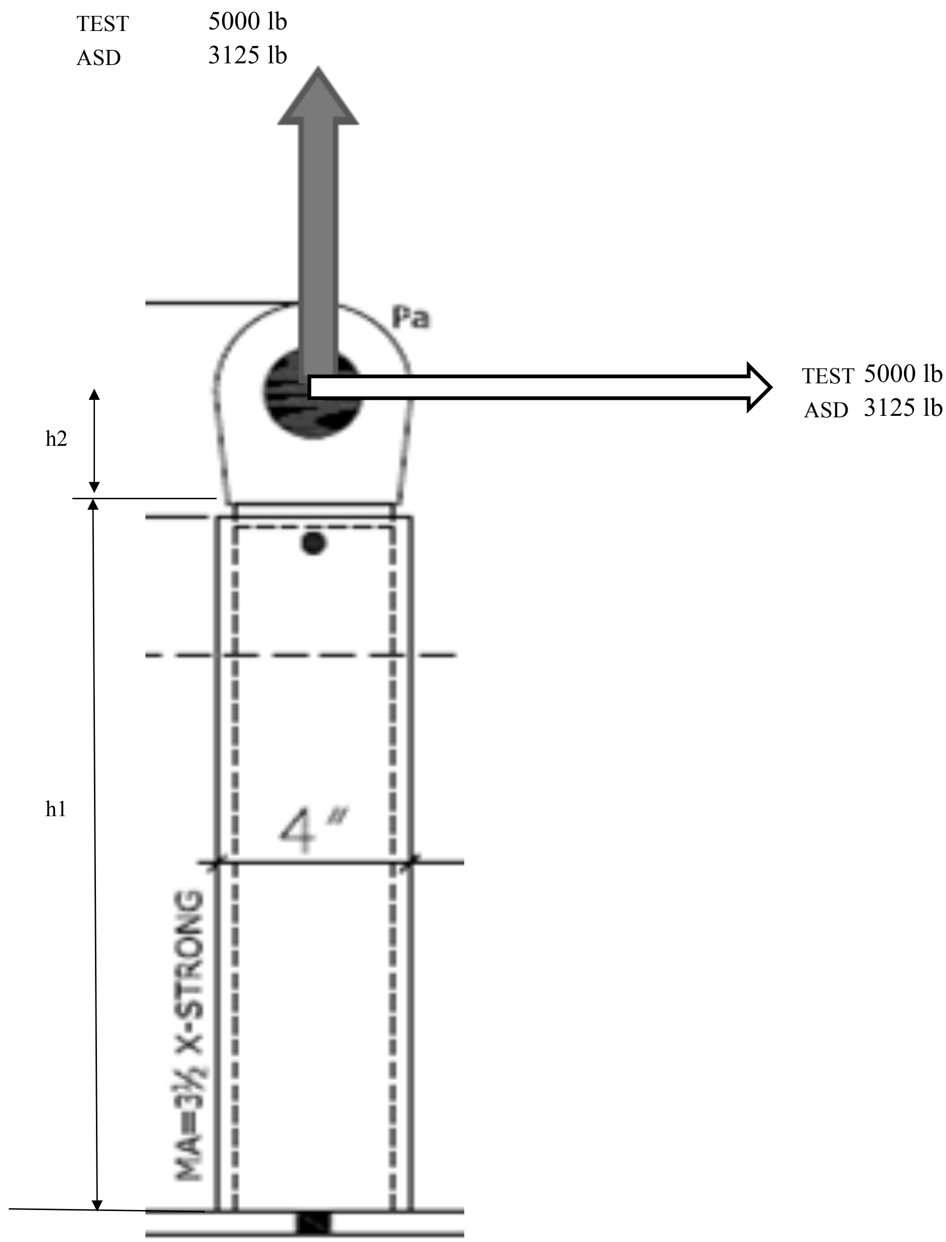
ANCHOR RODS	ASTM F1554 Gr. 36	F_y =	36 ksi

LOADING OF HARDY FALL PROTECTION SADDLE

LOADING CRITERIA:

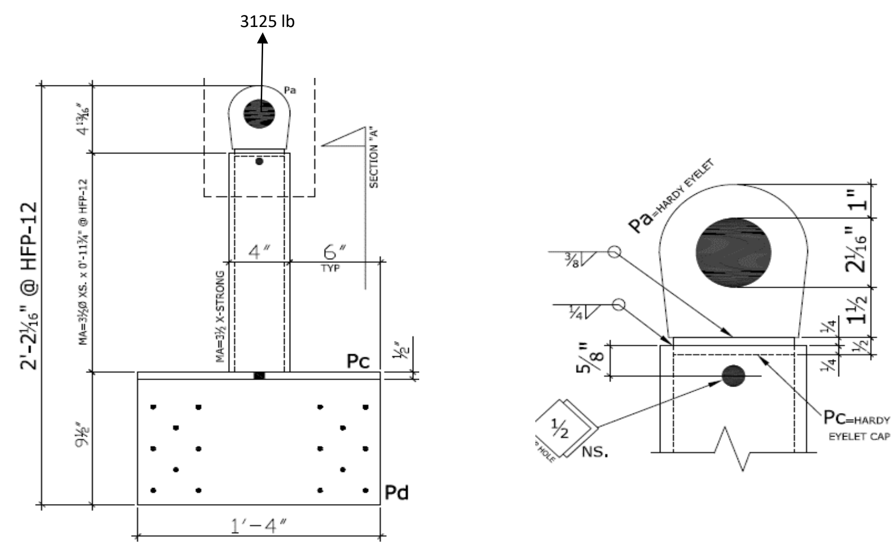
	TEST LOADING	ALLOWABLE STRESS DESIGN LOADING
VERTICAL LOAD =	5.00 kip	3.125 kip
HORIZONTAL LOAD =	5.00 kip	3.125 kip

RUN CALCULATIONS FOR:	ALLOWABLE STRESS DESIGN LOADING
POST HEIGHT h1 =	12.00 in
HEIGHT FROM TOP OF POST TO CENTER OF EYELET h2 =	2.50 in
TOTAL HEIGHT OF HORIZONTAL LOAD FROM BASE OF POST =	14.50 in
VERTICAL LOAD =	3.13 kip
HORIZONTAL LOAD =	3.13 kip
MAXIMUN SHEAR AT POST BASE F_{max} =	3.13 kip
MAXIMUN MOMENT AT POST BASE M_{max} =	45.31 kip-in



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:	4 1/16 in
	ASTM SPECIFICATION	ASTM A36	SHEAR LAG FACTOR U =	1.00
	HOLE DIAMETER =	2 1/16 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 =	4.063 sq.in
	HOLE DIAMETER =	2 1/8 in
	NET AREA OF MEMBER A_n =	1.938 sq.in
	EFFECTIVE NET AREA $A_e = A_n U$ =	1.938 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	$P_n = F_y A_g$ =	146.250 kip
AISC 360-16 EQUATION D2-1		$\Omega = 1.67$
	DESIGN TENSILE STRENGTH ΦP_n OR ALLOWABLE TENSILE STRENGTH P_n/Ω =	87.575 kip
TENSILE RUPTURE IN THE NET SECTION		
AISC 360-16 EQUATION D2-2	$P_n = F_u A_e$ =	112.375 kip
AISC 360-16 EQUATION D2-2		$\Omega = 2.00$
	DESIGN TENSILE STRENGTH ΦP_n OR ALLOWABLE TENSILE STRENGTH P_n/Ω =	56.188 kip
TENSILE RUPTURE IN THE NET SECTION GOVERNS,		
	DESIGN TENSILE STRENGTH ΦP_n OR ALLOWABLE TENSILE STRENGTH P_n/Ω =	**56.188 kip**
CHECK EYELET PLATE STRENGTH IN TENSION:		
	AVAILABLE TENSILE STRENGTH P_n/Ω =	**56.188 kip**
	REQUIRED TENSILE STRENGTH =	**3.125 kip**
	RATIO OF REQUIRED STRENGTH/ AVAILABLE STRENGTH =	**0.056**
		OK

CHECK OF HTB STEEL POST STRENGTH

BY PROVISIONS OF AISC 360-16

MEMBER INPUT

SECTION INPUT:	MEMBER ID	HTB-S12		SHAPE:	PIPE
	SECTION:	Pipe3-1/2XS	ASTM SPEC.	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	1.21 ft	1.21 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 =	1.21 ft

SUMMARY OF RESULTS

AVAILABLE STRENGTH OF SECTION:	**LRFD**		**ASD**	
AVAILABLE COMPRESSIVE STRENGTH:	$\Phi_c P_n$ =	**105.741 kips**	P_n / Ω_c =	**70.353 kips**
AVAILABLE TENSILE STRENGTH:	$\Phi_t P_n$ =	**108.411 kips**	P_n / Ω_t =	**72.130 kips**
AVAILABLE FLEXURAL STRENGTH:	$\Phi_b M_n$ =	**10.675 kip-ft**	M_n / Ω_b =	**7.102 kip-ft**
AVAILABLE SHEAR STRENGTH:	$\Phi_v V_n$ =	**32.523 kips**	V_n / Ω_v =	**21.639 kips**

CALCULATIONS

MATERIAL PROPERTIES

YOUNG'S MODULUS E_s =	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
AISC 360-16 SECTION (B4-2)	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
		λ_p =	N/A
		λ_r = 0.11 E/F_y =	91.143
		CLASSIFICATION FOR UNIFORM COMPRESSION:	NONCOMPACT
2- SLENDERNESS RATIO:	AISC 360-16 SECTION E2	$(L_c/r)_x = (KL/r)_x$ =	22.07
		$(L_c/r)_y = (KL/r)_y$ =	22.07
		$(L_c/r)_{max} = (KL/r)_{max}$ =	22.07
		≤ 200	**OK**
ELASTIC CRITICAL BUCKLING STRESS F_e			
	AISC 360-16 EQUATION (E3-4)	$F_e = \pi^2 E / (L_c / r)^2$ =	587.454 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)		$F_{cr} = [0.658^{Fy/Fe}] F_y$	34.138 ksi
AISC 360-16 EQUATION (E3-3)		$F_{cr} = 0.877 F_e$	N/A
		F_{cr} =	34.138 ksi
AVAILABLE COMPRESSIVE STRENGTH	PROVISIONS OF AISC 360-16 SECTION E3, "MEMBERS WITHOUT SLENDER ELEMENTS"APPLY		
		F_{cr} =	34.138 ksi
AISC 360-16 EQUATION (E3-1)		$P_n = F_{cr} A_g$ =	117.49 kips

	LRFD		**ASD**	
AISC 360-16 SECTION E1	Φ_c =	**0.9**	Ω_c =	**1.67**
AVAILABLE COMPRESSIVE STRENGTH:	$\Phi_c P_n$ =	**105.741 kips**	P_n / Ω_c =	**70.35 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		
	LRFD		ASD	
AISC 360-16 SECTION D2. (a)	$\Phi_t =$	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		
	LRFD		ASD	
AISC 360-16 SECTION D2. (b)	$\Phi_t =$	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 OK

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)	$M_n = (0.021E / (D/t) + F_y)\ S$	N/A
FOR SLENDER SECTIONS:		
AISC 360-16 EQUATION (F8-4)	$F_{cr} = 0.33E / (D/t)$	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		
	LRFD		**ASD**	
AISC 360-16 SECTION F1	$\Phi_b =$	**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

	$L_v =$	1.21 ft		
AISC 360-16 SECTION G6 F_{cr} IS THE LARGER OF:				
AISC 360-16 EQUATION (G5-2a)	$F_{cr} = 1.60\ E / \sqrt{(L_v / D)}\ (D/t)^{5/4} =$	939.564 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 $V_n = F_{cr} A_g / 2 =$	36.14 kips		
	LRFD		**ASD**	
AISC 360-16 SECTION G1	$\Phi_v =$	**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

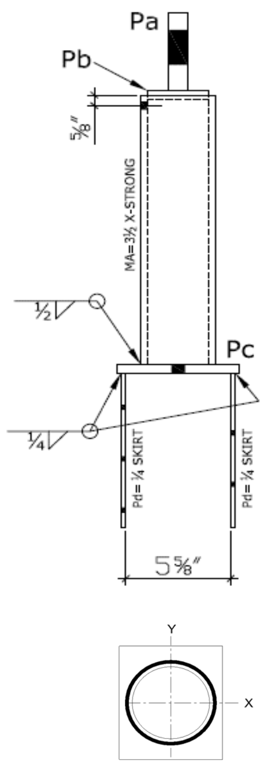
CHECK POST STRENGTH FOR ALLOWABLE STRESS DESIGN LOADING

	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:	45.31 kip-in	85.23 kip-in	0.531657194	**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 =	45.313 kip-in
	APPLIED BENDING MOMENT M_z =	0.000 kip-in

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 ≥ 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

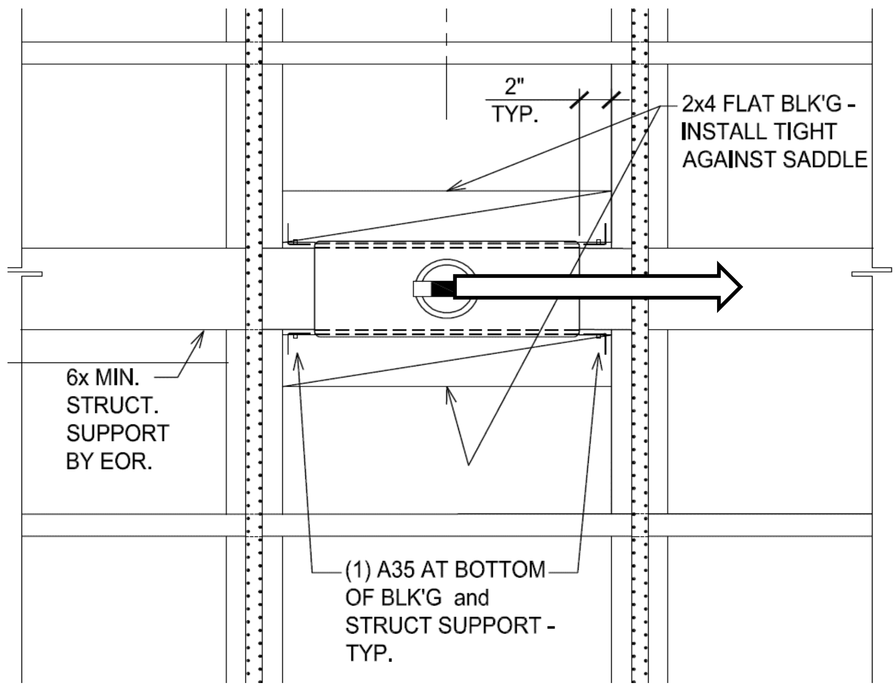
2- TENSION STRESSES:

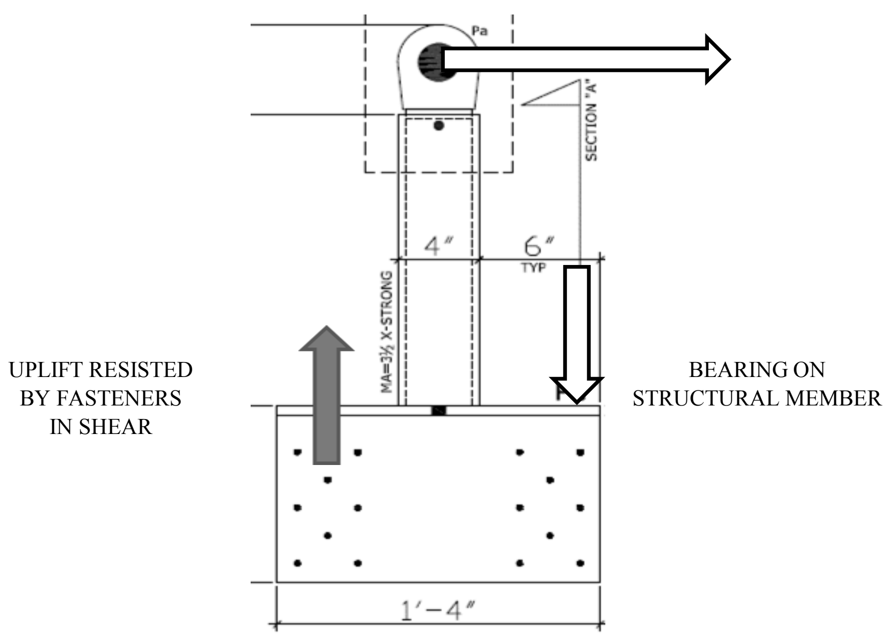
TENSION STRESS DUE TO F_Z	$r_{Faz} = F_Z / A_e$ =	0.646 ksi
CRITICAL TENSILE STRESS DUE TO M_X	$r_{Max} = M_X\ C_y / I_x$ =	0.000 ksi
CRITICAL TENSILE STRESS DUE TO M_Y	$r_{May} = M_Y\ C_x / I_y$ =	9.248 ksi
	$r_a = r_{Faz} + r_{Max} + r_{May}$ =	9.894 ksi

3- COMBINING SHEAR AND TENSILE STRESSES INTO RESULTANT SHEAR STRESS:

	$r_a = \sqrt{r_{a\ SHEAR}^2 + r_{a\ TENSION}^2}$	9.915 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 =	**0.472**
	REQUIRED STRENGTH ≤ AVAILABLE STRENGTH	**OK**

CHECK OF HTB ATTACHMENT (CASE 1)





POST HEIGHT h1 =	12.00 in
HEIGHT FROM TOP OF POST TO CENTER OF EYELET h2 =	2.50 in
TOTAL HEIGHT OF HORIZONTAL LOAD FROM BASE OF POST =	14.50 in
VERTICAL LOAD =	3.13 kip
HORIZONTAL LOAD =	3.13 kip
MAXIMUN MOMENT AT POST BASE M_{max} =	45.31 kip-in
HORIZONTAL DISTANCE BETWEEN BEARING / UPLIFT ON 6X STRUCTURAL MEMBERS =	12.50 in
BEARING / UPLIFT ON 6X STRUCTURAL MEMBERS =	3.625 kip
UPLIFT RESISTED BY FASTENERS ON ONE SIDE OF SKIRT PLATE =	**1.813 kip**

HARDY SADDLE DEVICE UPLIFT CHECK

	TYPE OF FASTENERS	SDS 2 1/2"X1/4"
IAPMO UES 461 SECTION 4.2.2	MINIMUM MAIN MEMBER THICKNESS =	5 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 6X10, A_m =	60.00 sq.in
	A_s = 10 X 1/4 =	2.50 sq.in
	A_m / A_s =	24
	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 =	0.99
	GROUP 2: NUMBER OF FASTENER ROWS =	1
	NUMBER OF FASTENERS IN A ROW =	2
NDS 2018 TABLE 11.3.6C	GROUP ACTION FACTOR C_g =	1.00

Adjustment Factors per NDS 2018 Table 11.3.1

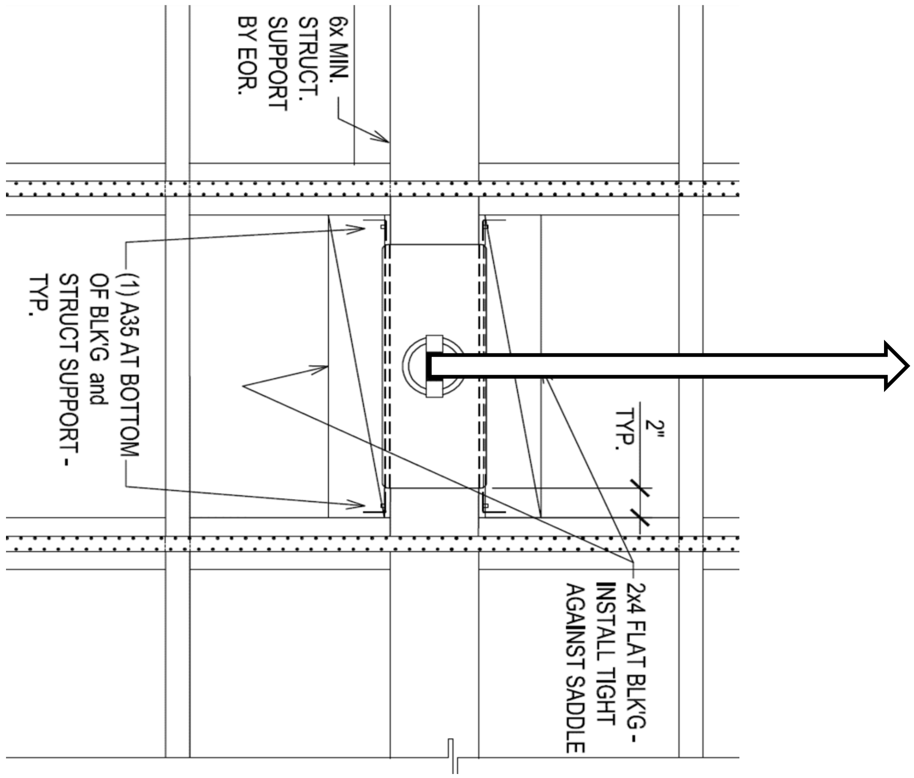
C_D	C_M	C_t	C_g	C_Δ	C_d	C_{st}
1.5	1	1	VARRIES	1	1	1

GROUP 1:	SHEAR DESIGN VALUE OF ONE FASTENER Z' =	624 lb
	SHEAR DESIGN VALUE OF ONE ROW OF FASTENERS =	1871 lb
	SHEAR DESIGN VALUE OF GROUP 1 =	3742 lb
GROUP 2:	SHEAR DESIGN VALUE OF ONE FASTENER Z' =	630 lb
	SHEAR DESIGN VALUE OF ONE ROW OF FASTENERS =	1260 lb
	SHEAR DESIGN VALUE OF GROUP 2 =	1260 lb
	UPLIFT STRENGTH = SHEAR DESIGN VALUE OF CONNECTION =	**5002 lb**
	REQUIRED UPLIFT STRENGTH =	**1813 lb**
	RATIO OF REQUIRED STRENGTH/ AVAILABLE STRENGTH =	**0.362**
		OK

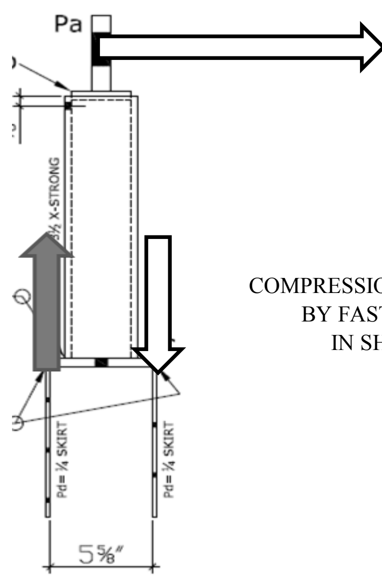
DURATION FACTOR NOTE:

THE DESIGN LOAD AS STATED IN PAGE 4 OF THIS PACKAGE ACCOUNTS FOR THE IMPACT LOAD RESULTING FROM THE EVENT OF FALL.
ACCORDING TO 2018 NDS, SECTION 2.3.2 AND TABLE 2.3.2, DURATION FACTOR FOR IMPACT LOAD = 2.0
A MORE CONSERVATIVE DESIGN APPROACH WOULD BE CONSIDERING A LOAD DURATION FACTOR OF 1.6 TO REPRESENT A 10-MINUTES DURATION OF LOAD
HOWEVER, A MORE CONSERVATIVE DURATION FACTOR OF 1.5 WAS USED TO CALCULATE FASTENER DESIGN VALUES IN THIS PACKAGE.

CHECK OF HTB ATTACHMENT (CASE 2)



UPLIFT RESISTED
BY FASTENERS
IN SHEAR



COMPRESSION RESISTED
BY FASTENERS
IN SHEAR

POST HEIGHT h1 =	12.00 in
HEIGHT FROM TOP OF POST TO CENTER OF EYELET h2 =	2.50 in
TOTAL HEIGHT OF HORIZONTAL LOAD FROM BASE OF POST =	14.50 in
VERTICAL LOAD =	3.13 kip
HORIZONTAL LOAD =	3.13 kip
MAXIMUN MOMENT AT POST BASE M_{max} =	45.31 kip-in
HORIZONTAL DISTANCE BETWEEN BEARING / UPLIFT ON 6X STRUCTURAL MEMBERS =	5.50 in
BEARING / UPLIFT ON 6X STRUCTURAL MEMBERS =	**8.239 kip**

HARDY SADDLE DEVICE UPLIFT CHECK

	TYPE OF FASTENERS	SDS 2 1/2"X1/4"
IAPMO UES 461 SECTION 4.2.2	MINIMUM MAIN MEMBER THICKNESS =	5 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 6X10,	A_m =	60.00 sq.in
	A_s = 10 X 1/4 =	2.50 sq.in
	A_m / A_s =	24
GROUP 1:	NUMBER OF FASTENER ROWS =	4
	NUMBER OF FASTENERS IN A ROW =	3
NDS 2018 TABLE 11.3.6C	GROUP ACTION FACTOR C_g =	0.99
GROUP 2:	NUMBER OF FASTENER ROWS =	2
	NUMBER OF FASTENERS IN A ROW =	2
NDS 2018 TABLE 11.3.6C	GROUP ACTION FACTOR C_g =	1.00

Adjustment Factors per NDS 2018 Table 11.3.1						
C_D	C_M	C_t	C_g	C_Δ	C_d	C_{st}
1.5	1	1	VARRIES	1	1	1

GROUP 1:	SHEAR DESIGN VALUE OF ONE FASTENER Z' =	624 lb
	SHEAR DESIGN VALUE OF ONE ROW OF FASTENERS =	1871 lb
	SHEAR DESIGN VALUE OF GROUP 1 =	7484 lb
GROUP 2:	SHEAR DESIGN VALUE OF ONE FASTENER Z' =	630 lb
	SHEAR DESIGN VALUE OF ONE ROW OF FASTENERS =	1260 lb
	SHEAR DESIGN VALUE OF GROUP 2 =	2520 lb
	UPLIFT STRENGTH = SHEAR DESIGN VALUE OF CONNECTION =	**10004 lb**
	REQUIRED UPLIFT STRENGTH =	**8239 lb**
	RATIO OF REQUIRED STRENGTH/ AVAILABLE STRENGTH =	**0.824**
		OK

DURATION FACTOR NOTE:

THE DESIGN LOAD AS STATED IN PAGE 4 OF THIS PACKAGE ACCOUNTS FOR THE IMPACT LOAD RESULTING FROM THE EVENT OF FALL.
ACCORDING TO 2018 NDS, SECTION 2.3.2 AND TABLE 2.3.2, DURATION FACTOR FOR IMPACT LOAD = 2.0
A MORE CONSERVATIVE DESIGN APPROACH WOULD BE CONSIDERING A LOAD DURATION FACTOR OF 1.6 TO REPRESENT A 10-MINUTES DURATION OF LOAD
HOWEVER, A MORE CONSERVATIVE DURATION FACTOR OF 1.5 WAS USED TO CALCULATE FASTENER DESIGN VALUES IN THIS PACKAGE.