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STRUCTURAL CALCULATIONS

FOR

Hardy Fall Protection

Hardy Wall Mount

10X10 PLATE

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
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 A36	F_y =	36 ksi

2) **CONNECTIONS:**

BOLTS	ASTM A307
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:**

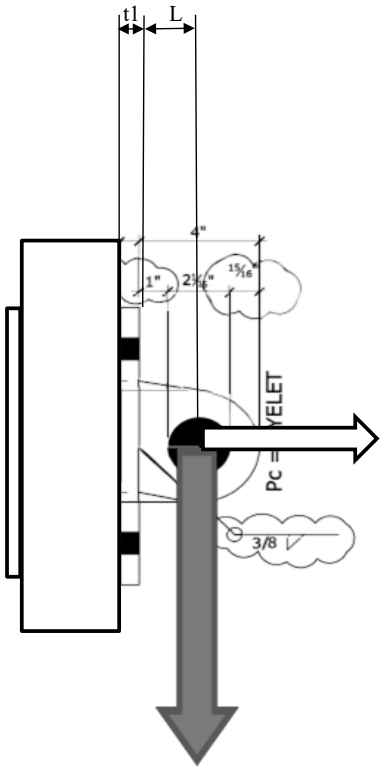
ANCHOR RODS	ASTM F1554 Gr. 36	F_y =	36 ksi

LOADING OF HARDY WALL MOUNT

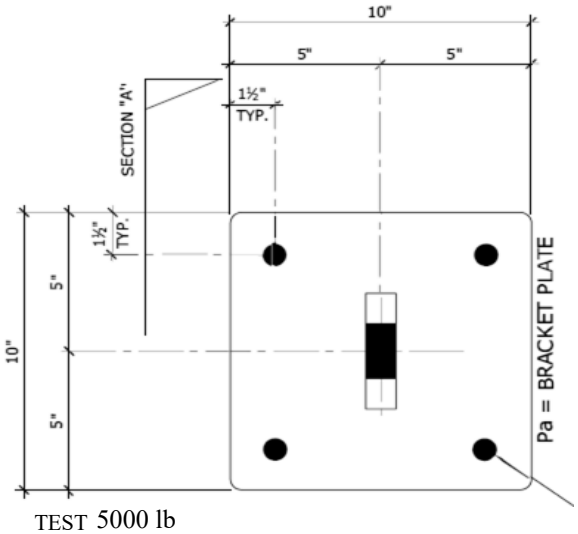
LOADING CRITERIA:

	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
PLATE THICKNESS t1 =	0.50 in
DISTANCE FROM TOP OF PLATE TO CENTER OF EYELET L =	2.00 in
TOTAL DISTANCE OF VERTICAL LOAD FROM FACE OF PLATE =	2.50 in
VERTICAL LOAD =	3.13 kip
HORIZONTAL LOAD =	3.13 kip
MAXIMUN SHEAR AT FACE OF PLATE F_{max} =	3.13 kip
MAXIMUN MOMENT AT FACE OF PLATE M_{max} =	7.81 kip-in



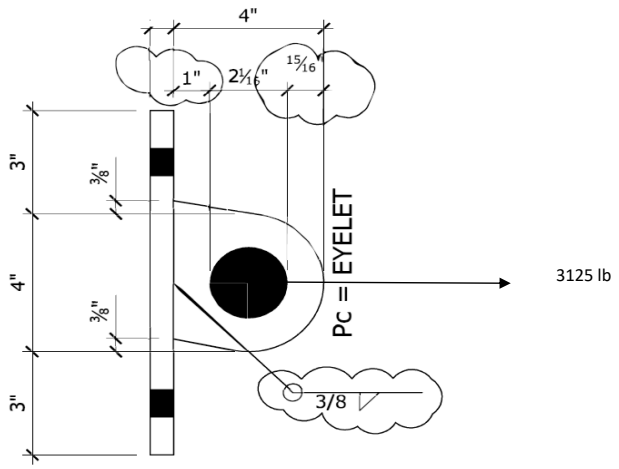
TEST 5000 lb
ASD 3125 lb



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:	4 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

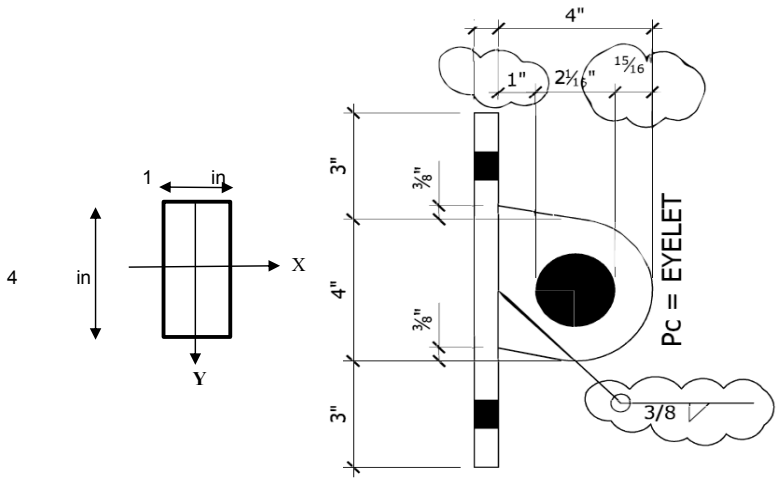
EYELET PLATE STRENGTH IN TENSION

GROSS, NET AND EFFECTIVE AREA:	PLATE THICKNESS t =	1.000 in
	GROSS AREA OF MEMBER A_g =	4.000 sq.in
	HOLE DIAMETER =	2 1/8 in
	NET AREA OF MEMBER A_n =	1.875 sq.in
	EFFECTIVE NET AREA $A_e = A_n U$ =	1.875 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	$P_n = F_y A_g$ =	144.000 kip
AISC 360-16 EQUATION D2-1		Ω = 1.67
	DESIGN TENSILE STRENGTH ΦP_n OR ALLOWABLE TENSILE STRENGTH P_n/Ω =	86.228 kip
TENSILE RUPTURE IN THE NET SECTION		
AISC 360-16 EQUATION D2-2	$P_n = F_u A_e$ =	108.750 kip
AISC 360-16 EQUATION D2-2		Ω = 2.00
	DESIGN TENSILE STRENGTH ΦP_n OR ALLOWABLE TENSILE STRENGTH P_n/Ω =	54.375 kip
	TENSILE RUPTURE IN THE NET SECTION GOVERNS,	
	DESIGN TENSILE STRENGTH ΦP_n OR ALLOWABLE TENSILE STRENGTH P_n/Ω =	**54.375 kip**
CHECK EYELET PLATE STRENGTH IN TENSION:		
	AVAILABLE TENSILE STRENGTH P_n/Ω =	**54.375 kip**
	REQUIRED TENSILE STRENGTH =	**3.125 kip**
	RATIO OF REQUIRED STRENGTH/ AVAILABLE STRENGTH =	**0.057**
		OK

CHECK OF FILLET WELD AT EYELET PLATE BASE (ASD)

BY PROVISIONS OF ANSI/AISC 360-16

INPUT



EYELET PLATE:		SECTION:	RECTANGULAR
		ASTM SPECIFICATION	ASTM A36
BASE PLATE:	THICKNESS t = 0.500 in	ASTM SPECIFICATION	ASTM A36
FILLET WELD PROPERTIES:		ELECTRODE CLASSIFICATION	E70XX
		FILLET WELD LEG SIZE AT FLANGE =	3/8 in
APPLIED LOADS		APPLIED AXIAL FORCE F_Z =	3.125 kip
		APPLIED SHEAR FORCE F_x =	0.000 kip
		APPLIED SHEAR FORCE F_y =	3.125 kip
		APPLIED BENDING MOMENT M_x =	7.813 kip-in
		APPLIED BENDING MOMENT M_y =	0.000 kip-in
		APPLIED BENDING MOMENT M_z =	0.000 kip-in

CALCULATIONS

EYELET PLATE PROPERTIES:		F_y =	36 ksi
		F_u =	58 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 =	10.00 in
		I_x =	10.30 in^4
		C_y =	2.00 in
		S_x =	5.15 cu.in
AISC 360-16 TABLE J2.4		MINIMUM WELD SIZE =	3/16 in
		CHECK PROVIDED WELD SIZE ≥ MINIMUM ALLOWABLE	OK
		EFFECTIVE LENGTH	10.00 in
		EFFECTIVE THROAT	0.27 in
		EFFECTIVE AREA A_w =	2.65 sq.in

WELD REQUIRED STRENGTH

1- SHEAR STRESSES:

SHEAR STRESS DUE TO F_X	$r_{Fax} = F_X / A_e$ =	0.000 ksi
SHEAR STRESS DUE TO F_Y	$r_{Fay} = F_Y / A_e$ =	1.179 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: $r_a = \sqrt{(r_{Fax} + r_{Max})^2 + (r_{Fay} + r_{May})^2}$ = 1.179 ksi

2- TENSION STRESSES:

TENSION STRESS DUE TO F_Z	$r_{Faz} = F_Z / A_e$ =	1.179 ksi
CRITICAL TENSILE STRESS DUE TO M_X	$r_{Max} = M_X\ C_y / I_x$ =	1.518 ksi
CRITICAL TENSILE STRESS DUE TO M_Y	$r_{May} = M_Y\ C_x / I_y$ =	0.000 ksi
	$r_a = r_{Faz} + r_{Max} + r_{May}$ =	2.696 ksi

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

$r_a = \sqrt{r_{a\ SHEAR}{}^2 + r_{a\ TENSION}{}^2}$ = 2.942 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 =	**0.140**
REQUIRED STRENGTH ≤ AVAILABLE STRENGTH	**OK**

WALL MOUNT ATTACHMENT LOADING

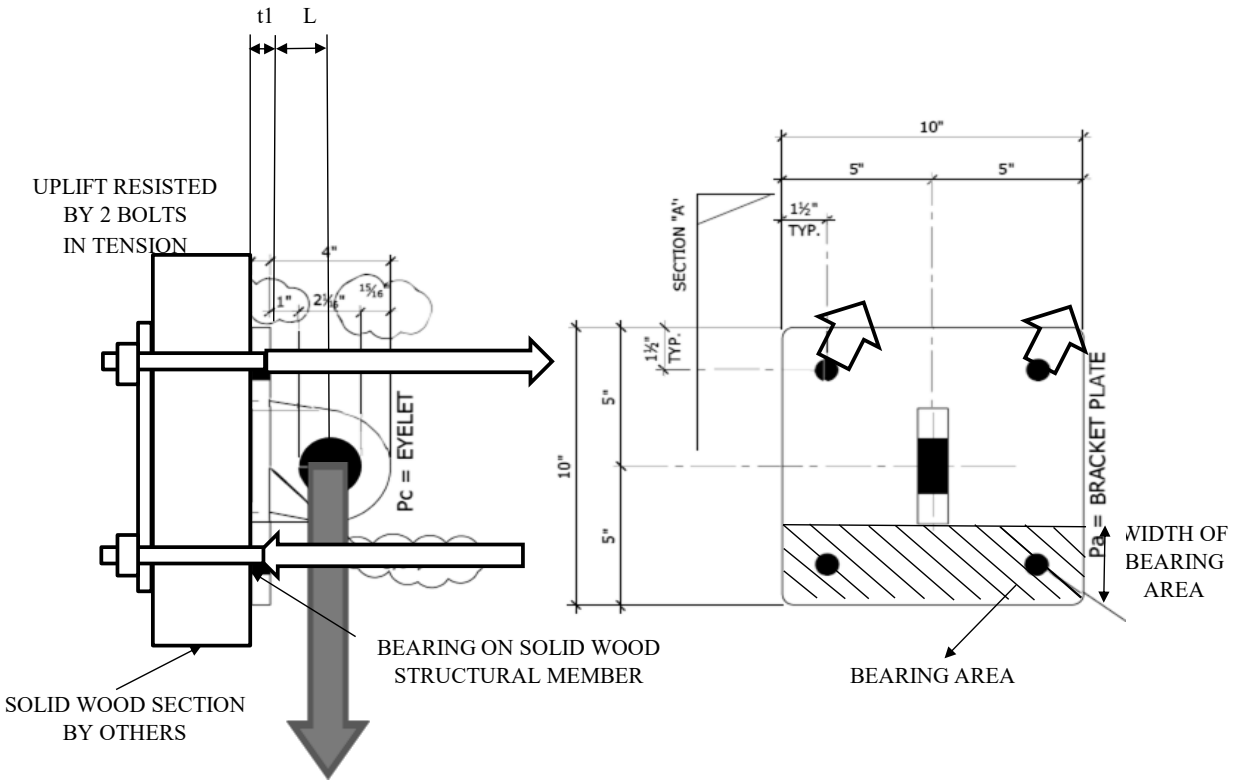


PLATE THICKNESS t1 =	0.50 in
DISTANCE FROM TOP OF PLATE TO CENTER OF EYELET L =	2.00 in
TOTAL DISTANCE OF VERTICAL LOAD FROM FACE OF PLATE =	2.50 in
VERTICAL LOAD =	3.13 kip
HORIZONTAL LOAD =	3.13 kip
MAXIMUN MOMENT AT FACE OF PLATE Mmax =	7.81 kip-in

LICENSED TO: NOT FOR COMMERCIAL USE

LICENSED No.: TRIAL VERSION

BASE PLATE FOR SOLID RECTANGULAR COLUMN WITH LARGE MOMENT - ASD

BY PROVISIONS OF AISC 360-16, AISC DESIGN GUIDE 1 SECOND EDITION AND 2018 NDS

INPUT

COLUMN INPUT:

SHAPE:	SOLID RECTANGULAR
SECTION:	4X1
COLUMN LOADS: P_r =	0.100 kip
M_{xr} =	7.81 kip-in

BASE PLATE INPUT:

LENGTH PARALLEL TO COLUMN WEB (N) =	10.00 in.
WIDTH PARALLEL TO COLUMN FLANGE (B) =	10.00 in.
ASTM SPECIFICATION	ASTM A36

ANCHOR BOLTS INPUT:

BOLT DIAMETER =	1/2 in.
No. OF ANCHORS IN THE GROUP (n)	4
No. OF ANCHORS IN ONE (X) ROW (nx)	2
No. OF ANCHORS IN (Y) ROW (ny)	2
SPACING S_x	7 in
SPACING S_y	7 in

WOOD DIMENSIONS TO CALCULATE A2 PER AISC 360-16 SECTION J8

DISTANCE TO CLOSEST WOOD EDGE MEASURED PARALLEL TO X DIRECTION b2 =	0.00 in.
DISTANCE TO CLOSEST WOOD EDGE MEASURED PARALLEL TO Y DIRECTION n2 =	0.00 in.

WOOD INPUT: (ASSUMING DF-L #2) **DURATION FACTOR (SEE NOTE AT END OF CALCULATIONS) =** 1.5

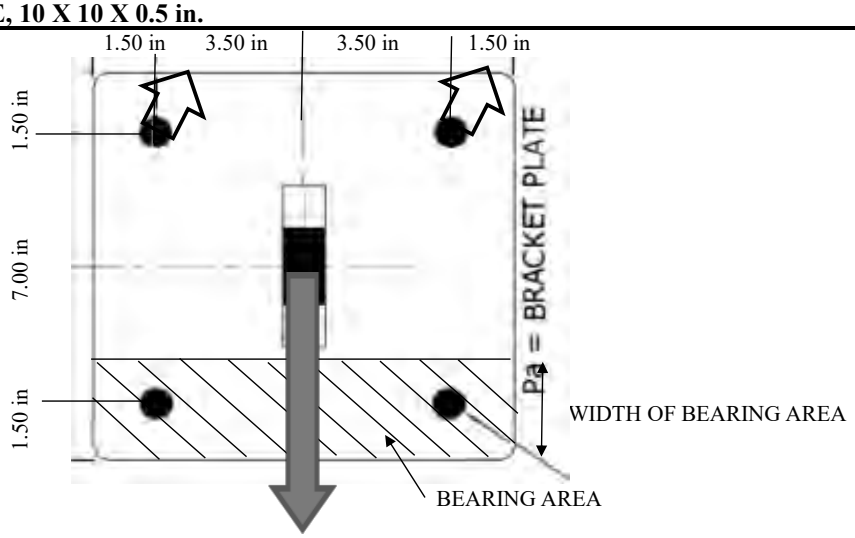
VALUES FOR COMPRESSION PERPENDICULAR TO GRAIN WILL CONSERVATIVELY BE USED

	BASIC STRESS (psi)	WET SERVICE FACTOR C_M	TEMPERATURE FACTOR C_t	SIZE FACTOR C_F	FLAT USE FACTOR C_{fu}	INCISING FACTOR C_i	REPETITIVE MEMBER FACTOR C_r	Bearing Area Factor C_b	ALLOWABLE STRESS / (C_D*C_L) (psi)
Fb	1500 psi	1	1	1.00	1	1	1	-	1500 psi
Ft	1000 psi	1	1	0.9	-	1	-	-	900 psi
Fv	180 psi	1	1	-	-	1	-	-	180 psi
Fc ⊥	625 psi	1	1	-	-	1	-	1	625 psi
Fc	1700 psi	1	1	0.9	-	1	-	-	1530 psi
E	1900000 psi	1	1	-	-	1	-	-	1900000 psi
Emin	690000 psi	1	1	-	-	1	-	-	690000 psi

SUMMARY OF RESULTS

ASSUMPTION OF LARGE MOMENT CONDITION:	**OK**
TRIAL BASE PLATE DIMENSIONS N X B:	**OK**

USE ASTM A36 BASE PLATE, 10 X 10 X 0.5 in.



CALCULATIONS

BASE PLATE MATERIAL PROPERTIES:	F_y =	36 ksi
	F_u =	58 ksi
BASE PLATE, COLUMN AND BOLTS DIMENSIONS:		
	COLUMN WIDTH W =	4.000 in
	COLUMN THICKNESS (t1) =	1.000 in
	e_x =	1.50 in.
	e_y =	1.50 in.
	DISTANCE PARALLEL TO FLANGE BETWEEN ANCHOR BOLTS (d_x) =	7.00 in.
	DISTANCE PARALLEL TO WEB BETWEEN ANCHOR BOLTS (d_y) =	7.00 in.
	0.9 W =	3.600 in
	0.9 t1 =	0.900 in
	m = (N - 0.9 W) / 2 =	3.200 in
	n = (B - 0.9 t1) / 2 =	4.550 in
	f = d_y / 2 =	3.500 in
	DISTANCE FROM C.L OF BOLT TO CRITICAL SECTION x = MAX{ (m - e_y) , (n - e_x)} =	3.050 in
WOOD BEARING STRENGTH (2018 NDS 3.10.1, 3.10.2, ASCE 360-16 J8)	A_1 = N B =	100.00 sq.in
CALCULATION OF A2:	ASPECT RATIO OF A1 = N / B =	1.000
	B2 (IF b2 GOVERNS) =	10.000 in
	CORRESPONDING N2 = B2 * ASPECT RATIO =	10.000 in
	CORRESPONDING A2 = B2 * N2 =	100.00 sq.in
	N2 (IF n2 GOVERNS) =	10.000 in
	CORRESPONDING B2 = N2 / ASPECT RATIO =	10.000 in
	CORRESPONDING A2 = B2 * N2 =	100.00 sq.in
	b2 GOVERNS, A2 =	100.00 sq.in
AISC 360-16 EQUATION J8-2 (CONVERTED TO STRESS FORMAT)	$f_{p\,max} = 0.85\, f'_c \sqrt{A_2 / A_1}$	797 psi
DOES NOT APPLY FOR WOOD DESIGN	$\leq 1.7\, f'_c$	N/A
	$f_{p\,max}$ =	797 psi
	Ω_c =	2.31
	AVAILABLE BEARING STRESS $f_{pa\,max} = f_{p\,max} / \Omega_c$ =	345 psi
DESIGN GUIDE 1 EQUATION 3.3.4	$q_{max} = f_{pa\,max} * B$ =	3450 lb / in
CHECK ECCENTRICITY	ECCENTRICITY e = M_r / P_r =	78.125 in
DESIGN GUIDE 1 EQUATION 3.4.1	RITICAL ECCENTRICITY $e_{crit} = N/2 - P_r / 2\, q_{max}$ =	4.986 in
ECCENTRICITY EXCEEDS CRITICAL ECCENTRICITY, ASSUMPTION OF LARGE MOMENT CONDITION IS VALID		
CHECK PLATE DIMENSIONS N AND B	(f + N / 2) =	8.500 in
	$(f + N / 2)^2$ =	72.25 sq.in
	$2P_r (e + f) / q_{max}$ =	4.73 sq.in
DESIGN GUIDE 1 EQUATION 3.4.4 IS VERIFIED		
BEARING AND TENSILE FORCES	BEARING AREA WIDTH = Y	
DESIGN GUIDE 1 EQUATION 3.4.3	$Y = \left[f + \frac{N}{2}\right] \pm \sqrt{\left(f + \frac{N}{2}\right)^2 - \frac{2P_r(e+f)}{q_{max}}}$ =	0.283 in
DESIGN GUIDE 1 EQUATION 3.4.2	TENSION IN BOLTS ON ONE SIDE $T_a = q_{max}$ Y - P_r =	877 lb
CALCULATIONS OF REQUIRED PLATE THICKNESS	(IF n > m, n WILL BE USED INSTEAD OF m IN THIS SECTION)	
1) YIELDING LIMIT AT BEARING INTERFACE		
DESIGN GUIDE 1 EQUATION 3.3.14b-2	IF Y ≥ m , $t_{p\,req} = 1.83\, m \sqrt{\frac{f_{p\,max}}{F_y}}$ =	N/A
DESIGN GUIDE 1 EQUATION 3.3.15b-2	IF Y < m , $t_{p\,req} = 2.58 \sqrt{\frac{f_{p\,max} Y \left(m - \frac{Y}{2}\right)}{F_y}}$ =	0.282 in
MINIMUM REQUIRED PLATE THICKNESS FOR YIELDING LIMIT AT BEARING INTERFACE =		0.282 in
2) YIELDING LIMIT AT TENSION INTERFACE		
DESIGN GUIDE 1 EQUATION 3.4.5b	$M_{pl} = T_a$ x / B =	0.267 kip.in / in
DESIGN GUIDE 1 EQUATION 3.4.7b	$t_{p\,req} = 2.58 \sqrt{\frac{T_a x}{B F_y}}$ =	0.222 in
YIELDING LIMIT AT BEARING INTERFACE GOVERS, REQUIRED PLATE THICKNESS =		0.282 in
	USE BASE PLATE THICKNESS =	**1/2 in**

DURATION FACTOR NOTE:

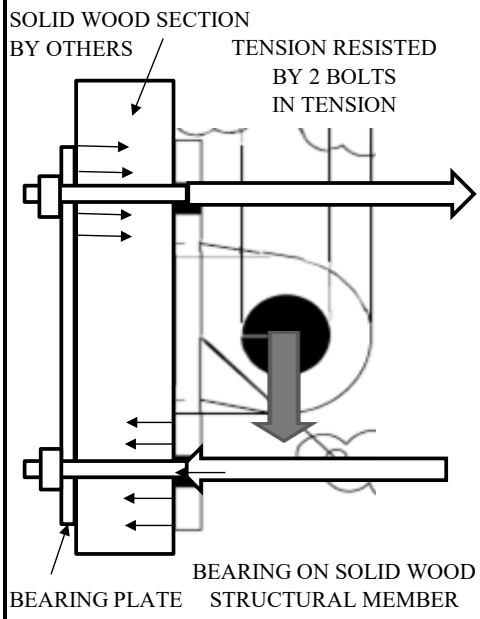
THE DESIGN LOAD AS STATED IN PAGE 4 OF THIS PACKAGE ACCOUNTS FOR THE IMPACT LOAD RESULTING FROM THE EVENT OF FALL.

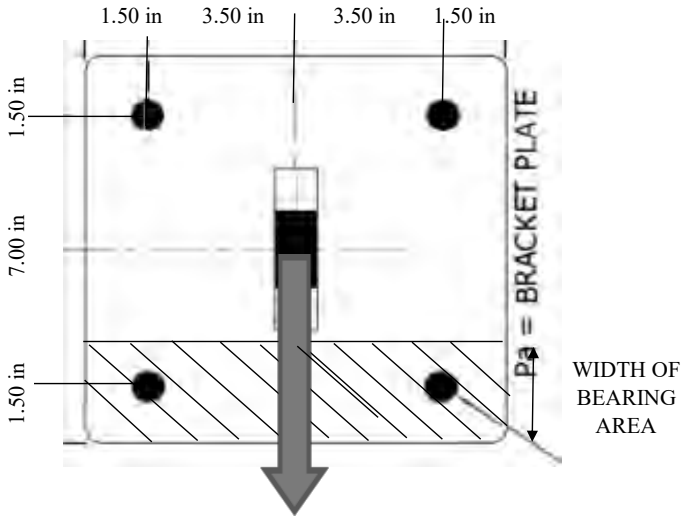
ACCORDING TO 2018 NDS, SECTION 2.3.2 AND TABLE 2.3.2, DURATION FACTOR FOR IMPACT LOAD = 2.0

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.

CONNECTION CHECK





CONNECTION CONFIGURATION	**TYPE OF FASTENERS**	**BOLTS**
	FASTENER DIAMETER =	**1/2 in**
	BOLT ASTM SPECIFICATION:	A307
	No. OF BOLTS IN THE GROUP (n)	4
	No. OF BOLTS IN (X) ROW (nx) =	2
	No. OF BOLTS IN (Y) ROW (ny) =	2
	SPACING S_X =	7 in
	SPACING S_Y =	7 in
	MINIMUM MAIN MEMBER THICKNESS =	3 1/2 in
	MAIN MEMBER SPECIFIC GRAVITY =	0.50 in
	BEARING PLATE SHAPE	RECTANGLE
	BEARING PLATE DIMENSION	3 in
	BEARING PLATE THICKNISS	1/4 in
CONNECTION FORCES	MOMENT AT BASE PLATE LEVEL =	7.81 kip-in
	REQUIRED TENSILE STRENGTH (T) ON ONE SIDE =	0.877 kip
	REQUIRED TENSILE STRENGTH (T) ON ONE BOLT =	**0.438 kip**
	WIDTH OF BEARING AREA =	0.283 in
	MOMENT ARM (DISTACCE BETWEEN TENSION BOLTS AND CENTER OF BEARING AREA) =	8.358 in
	BEARING ON ONE BEARING PLATE =	0.438 kip
CHECK TENSION BOLTS (AISC 360-16 Table J3.2)	NOMINAL TENSILE STRESS F_{nt} =	45 ksi
	F_{nv} =	27 ksi
	HOLE DIAMETER	9/16 in
	BOLT NOMINAL AREA A_b =	0.196 sq.in
(AISC 360-16 EQUATION J3-1, J3-2)	**Ω =**	2
(AISC 360-16 EQUATION J3-1)	$R_{nt} = F_{nt} A_b$ =	8.836 kip
	ALLOWABLE TENSION STRENGTH R_{nt} / Ω =	**4.418 kip**
RATIO OF REQUIRED STRENGTH TO AVAILABLE STRENGTH =	**0.099**	**OK**

CHECK BEARING (CRITICAL CASE: FROM BEARING PLATE)

VALUES FOR COMPRESSION PERPENDICULAR TO GRAIN WILL CONSERVATIVELY BE USED

AREA OF BEARING PLATE A_b =		30.000 sq.in
APPLIED BEARING STRESS =		**15 psi**
DESIGN VALUE FOR COMPRESSION PERPENDICULAR TO GRAIN $Fc^{\perp}$ =		625 psi
WET SERVICE FACTOR C_M =		1
TEMPERATURE FACTOR C_t =		1
INCISING FACTOR C_i =		1
BEARING AREA FACTOR C_b =		1
DURATION FACTOR (SEE NOTE AT END OF CALCULATIONS) =		1.5
ALLOWABLE BEARING STRESS =		**938 psi**
RATIO OF APPLIED TO ALLOWABLE BEARING STRESS =	**0.016**	**OK**

DESIGN OF BEARING PLATE

PLATE ASTM SPECIFICATION		ASTM A36 Grade 36
PLATE WIDTH (b) =		3.000 in
PLATE THICKNESS (t) =		0.2500 in
F_y =		36 ksi
Ω =		1.67
APPLIED BEARING STRESS =		**15 psi**
MOMENT ARM (a) FOR PLATE BENDING =		0.75 in
REQUIRED FLEXURE STRENGTH OF PLATE =		0.049 kip-in
$Z = b\ t^2/4$ =		0.047 in^3
$M_n = Z\ F_y$ =		1.688 kip-in
END PLATE AVAILABLE STRENGTH M_n / Ω =		**1.010 kip-in**
DEMAND / CAPACITY RATIO =	**0.049**	**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.

SOLID WOOD SECTION DISCLAIMER:

DESIGN OF SOLID WOOD SECTION IS NOT IN THE SCOPE OF THIS CALCULATION PACKAGE. THE DESIGN OF WOOD SECTION WILL BE THE RESPONSIBILITY OF THE ENGINEER OF RECORDS OF THE PROJECT WHERE THE HARDY WALL MOUNT IS TO BE INSTALLED.
THE FOLLOWING MINIMUM REQUIREMENTS WERE ASSUMED AND NEED TO BE CHECKED BY THE ENGINEER OF RECORDS:
WOOD GRADE AND SPECIES: DOUGLAS FIR-LARCH #2.
SPECIFIC GRAVITY: 0.5.
DESIGN VALUE FOR COMPRESSION PERPENDICULAR TO GRAIN = 625psi