R. F. NELSON		Date:	5/7/2019
& ASSOCIATES	1/4" BRACKETS	Job No.:	18-001
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Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/4" Floor & Wall Brackets (included within this calculation package for reference).

#### Prying of Piece #33143A - L3" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min) w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

AISC 14 Edition Part 9, p.9-10 of Specification

Bracket Thickness:	th <sub>bracket</sub> =	0.25 in		
Nominal Tensile Strength:	$F_t =$	60 ksi	(Carbon Steel)	
Ultimate Strength:	F <sub>u</sub> =	58 ksi		
Anchor Spacing	s =	0.00		
Anchor Diameter:	<i>d</i> <sub>b</sub> =	0.375 in		
Steel strength in tension:	N <sub>sa</sub> =	6.500 kip		
Max distance from bolt to applied force	e: b =	1.375 in		
Tributary length to each bolt:	<i>p = max 2b,</i> but <u>&lt;</u> s	2.75 in		
Adjusted prying distance:	$b' = \left[b - \frac{d_b}{2}\right] =$	1.19 in	(Eq. 9-21)	
Resistance factor for prying:	$\Phi_{\rm pr} =$	1.67 ASD 0.90 LRFD 0.75 LRFD		
Maximum bolt force for prying:	$T = \frac{\Phi_{pr} * F_u * th}{2}$	$h_{bracket}^{2} * p =$	3.26 kip	(Eq. 9-22a)

Maximum bolt force for prying:  $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = 3.26 \text{ kip}$ 

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 4.88 \text{ kip}$  (per ESR-1917)

$$t_{c} = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_{u}}} = 0.440 \text{ in}$$
 (Eq. 9-30a)

 $T_{avail} = BQ = 2.86 kip$ 

(Eq. 9-31)

$$Q = 0.587 \qquad a' < Q = 1$$

$$0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta^* \alpha') = 0.587 \qquad a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.587 \qquad (Eq. 9-33) \qquad (Eq. 34)$$

Width of hole along length of plate: d' = 0.500 in

Distance from bolt centerline to edge of plate: a = 1.50 in

Additional variables for prying calculation:

$$\begin{split} \delta &= 1 - \frac{d}{\rho} = 0.82 & a' = a + \frac{d}{2} = 1.69 \text{ in } \leq \left(1.25 * b * \frac{d_b}{2}\right) = 1.91 \text{ in } \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \rho &= \frac{b}{\alpha'} = 0.70 & \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = 1.00 \text{ in } \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ \alpha' &= tr \left[ \rho => 1, 1, \min \left[ 1, \frac{1}{\delta} * \left( \frac{\rho}{1 - \rho} \right) \right] \right] = 1.00 \end{split}$$

Required bracket thickness to ensure an acceptable combination of fitting strength, stiffness, and bolt strength:

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr}*p*F_{u}*(1+\delta*\alpha')}} = 0.23 \text{ in LRFD} \quad (Eq. 9-23a)$$

$$Thickness of Bracket is 0.25 in - therefore, Prying OK$$

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& ASSOCIATES	1/4" BRACKETS	Job No.:	18-001
Structural Engineers		Sheet:	1 (2)

#### Piece #33143A - L3" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min). w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickn	ness: th <sub>bracket</sub> = 0.2500 in	
Yield Strength:	$F_y = 36  ksi$	(Table 2-4)
Ultimate Streng	gth: $F_u = 58 \text{ ksi}$	
Gross Area:	A g = (3.0in)*(0.25in) = 0.75 in2	
	$\phi_{t-y} = 0.9$ LRFD	(Eqn D2.1)
Tensile Yielding	$T_{allow-yielding} = \phi_{ty} * F_y * A_g = 24.30 \text{ kip}$	(Eqn D2-1)
Shear Lag Fac	U = 1.0	(Table D3.1)
Net Area:	$A_n = (3.0in)^*(0.25in) - (2^*0.3125in)^*(0.25in) = 0.59 in2$	(Sec B4.3)
Effective Net A	Area: $A_e = A_n^* U = 0.59 \text{ in } 2$	(Eqn D3-1)
	$\phi_{tr} = 0.75$ LRFD	(Eqn D2-2)
Tensile Rupture	re: $T_{allow-rupture} = \phi_{tr} * F_u * A_e = 25.83 \text{ kip}$	(Eqn D2-2)
	$T_{allow-bracket} = min(T_{allow-yledding}, T_{allow-rupture}) = 24.30 kip LRFD$	(Vertical & Horizontal Component))
<u>Shear on Bracket Verti</u>		
Gross Area:	$A_{gv} = (3.0in)^*(0.25in) = 0.75 in2$	
	$     \Phi_{yy} = 1.00 $ <i>LRFD</i>	(Eqn J4-3)
Shear Yielding	$V_{allow-yielding} = \phi_{yv} * 0.60 * F_y * A_{gv} = 16.20 \text{ kip}$	(Eqn J4-3)
	$     \Phi_{\rm rv} = 0.75 $ LRFD	(Eqn J4-4)
Net Area:	$A_{nv} = (3.0in)^* (0.25in) - (2^* 0.3125in)^* (0.25in)$	= 0.59 in2
Shear Rupture.	V allow-rupture = $\phi_{rv} * 0.60 * F_u * A_{nv} = 20.66  kip$	(Eqn J4-4)
	$V_{allow}$ = min( $V_{allow-yielding}$ , $V_{allow-rupture}$ ) = 16.20 kip LRFD	(Horizontal Component)
**Note: Bending	ig of Bracket is Considered within the Prying Calculation	
Bending Moment on Ve	ertical Leg (Side A):	
Plastic Modulus	is: $Z = \frac{bd^2}{4} = \frac{3^* 0.25^2}{4} = 0.0469 \text{ in } 3$	
	$\phi_b = 0.9$ LRFD	(Sec F1)
	$M_{allow} = \phi *F_y *Z = 1.519$ kip-in	
Moment Arm:	Moment <sub>arm</sub> = 1.375 in	(Eqn F11-1)
Allowable Load	d: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 1.105 \text{ kip LRFD}$ (Vertical Composition)	nent)
Bending Moment on Lo	ower Leg (Side B):	
Plastic Modulus	is: $Z = \frac{bd^2}{4} = \frac{3^* 0.25^2}{4} = 0.0469 \text{ in 3}$	
	$\phi_b = 0.9$ LRFD	(Sec F1)
	$M_{allow} = \phi^* F_y^* Z = 1.519  kip-in$	(Eqn F11-1)

Moment Arm:	Moment arm =	1 375 in

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 1.105 \text{ kip LRFD}$  (Horizontal Component)

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<u>Piece #33143A - L3" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min)</u> w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

#### Screws from Angle Bracket to Steel Sheet:

 Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6, LRFD Shear (Bearing) & Tension (Pull-Out) Capacity

 (ICC-ESR-3332) Based on a Steel Member (min yield strength, Fy = 33 ksi & min tensile strength Fu = 45 ksi)

 Capacity of (1) Screw from 1/4", 3/8" & 1/2" Brackets to various design thickness steel sheets

 Shear Bearing Capacity first number is the minimum thickness of the steel in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet).

 Tensile Pull-out Capacity the number is for the steel sheet not in contact with the screw head (bottom sheet).

 Screw Capacities (Shear Bearing per ESR-3332, Table 3 & Tensile Pull-out per Table 5 per ESR-3332) LRFD

Design Thickness	V allow	Design Thickness	T <sub>allow</sub>	Number of Screws	N = 2
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	1206 lbf	420 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	2002 lbf	662 lbf
0.060-0.060	833 lbf	0.075	409 lbf	1666 lbf	818 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	2116 lbf	1096 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	2042 lbf	1794 lbf
		3/16"	1439 lbf		2878 lbf

# Bolts thru Angle Bracket to Concrete Slab or Concrete-Filled Profile Steel Deck Failure Modes:

Hilti Kwik Bolt-TZ anchors may be installed in cracked or uncracked concrete or concrete-filled steel deck

	Bolt type:	A307 Gr. A (Commo	on Bolts), b	earing type connection		
	Nominal Tensile St	rength:		$F_t =$	45 ksi	
	Bracket Thickness:			th <sub>bracket</sub> =	0.25 in	
	Nominal Shear Stre	ength, Threads Exclud Excluded:	ded:	F <sub>v</sub> =	27 ksi (Tabla 12 2	AISC 14th)
	Bolt Diameter:	Excluded.		d <sub>bcd</sub> =	(Table J3.2 0.50 in	AISC 14(II)
	Bolt Area:			A <sub>bcd</sub> =	$0.25^* \pi * d_{bcd}^2 =$	0.20 in2
	Resistance factor f	or bolt tension or shea	ar:	$\phi \!=$	0.75	LRFD
	Shear Capacity of s	single bearing type bo	olt:	V <sub>allow-bolt</sub> =	$\phi * F_v * A_{bcd} =$	3.98 kip
	Tension Capacity c	of single bearing type	bolt:	$T_{allow-bolt} =$	$\phi * F_t * A_{bcd} =$	6.63 kip
Bolt bea	aring strength at brac	ket connection: (Secti	ion J3.10)	F <sub>u</sub> =	58 ksi	
	Bolt edge distance:	÷		edge-dist =	1.50 in	
	Bolt hole diameter:			bh =	0.625 in	
	Clear distance betv and edge of adjace	ween edge of hole ent hole or edge of pla	te:	$L_c = edge-dist - 0.5*bh =$	1.19 in2	
	Single end bolt bea	aring capacity:	Bolt <sub>br'g</sub> =n	nin[(1.5*L <sub>c</sub> *th <sub>plate</sub> *F <sub>u</sub> ),(3.0	)*d <sub>bcd</sub> *th <sub>plate</sub> *F <sub>u</sub> )] =	21.75 kip

Bolt allow-bolt =  $\phi$  \*Bolt bearing 16.31 kip LRFD

#### <u>Piece #33143A - L3" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min)</u> w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

See Hilti Excel output for Allowable Combined Tension and Shear Loads for Concrete Over Metal Deck (LRFD) See Hilti Profis output for Allowable Combined Tension and Shear Loads for Concrete Slab (LRFD)

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

 $\phi N_n = 390 \, lbf$ 

 $\phi V_n = 440 \, lbf$ 

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

 $\phi N_n = 950 \, lbf$ 

 $\phi V_n = 1,322 \, lbf$ 

Given that the Load and Resistance Factor Design calculated above for the angle brackets and bolts far outweigh the capacity of the concrete anchors, the allowable loading to the concrete anchors govern.
Note also that the capacity of the concrete anchors shown here is based on utilizing Section D.3.3.4.3 (d) of ACI 318-11, which requires the inclusion of the Omega factor when determining the loads applied to the anchorage.
Do to the complication of the requirement (per ACI 318-11) to determine the concrete anchorage capacity utilizing LRFD as well as Section 4.2 in ESR-1917 the allowable load for this Piece is given in LRFD only.

#### <u>Overall Capacity of Seismic Load - Piece #33143A - 3" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min)</u> w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

Load allowable-total-on-concrete-ove-metal-deck = 390 lbf	Tension	
Load allowable-total-on-concrete-ove-metal-deck = 440 lbf	Shear	Vertical & Horizontal Allowable Load -
Load allowable-total-on-4" min-concrete-slab = 950 lbf	Tension	(LRFD)
Load allowable-total-on-4" min-concrete-slab = 1,322 lbf	Shear	

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Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/4" Floor & Wall Brackets (included within this calculation package for reference).

#### Prying of Piece #33144A - L3" x 3" x 1/4" x 0'-4" Steel Angle Bracket (A36 min) w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

AISC 14 Edition Part 9, p.9-10 of Specification

Bracket Thickness:	th <sub>bracket</sub> =	0.25 in		
Nominal Tensile Strength:	$F_t =$	60 ksi	(Carbon Steel)	
Ultimate Strength:	F <sub>u</sub> =	58 ksi		
Anchor Spacing	s =	0.00		
Anchor Diameter:	<i>d</i> <sub>b</sub> =	0.375 in		
Steel strength in tension:	N <sub>sa</sub> =	6.500 kip		
Max distance from bolt to applied force	e: b =	1.375 in		
Tributary length to each bolt:	<i>p = max 2b,</i> but <u>&lt;</u> s	2.75 in		
Adjusted prying distance:	$b' = \left[b - \frac{d_b}{2}\right] =$	1.19 in	(Eq. 9-21)	
Resistance factor for prying:	$\Phi_{\rm pr}$ =	1.67 ASD 0.90 LRFD 0.75 LRFD		
Maximum bolt force for prying:	$T = \frac{\Phi_{pr} * F_u * t}{2}$	$h_{bracket}^{2} * p =$	3.26 kip	(Eq. 9-22a)

 $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = -3.26 \text{ kip}$ Maximum bolt force for prying:

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 4.88 \text{ kip}$  (per ESR-1917)

$$t_{c} = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_{u}}} = 0.440 \text{ in}$$
 (Eq. 9-30a)

(Eq. 9-31)

(Eq. 34)

 $T_{avail} = BQ = 2.86 kip$ 

$$Q = 0.587 \qquad a' \le Q = 1$$

$$0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta^* \alpha') = 0.587 \qquad a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.587$$

(Eq. 9-33) Width of hole along length of plate: d' = 0.500 in

Distance from bolt centerline to edge of plate: a = 1.50 in

Additional variables for prying calculation:

$$\begin{split} \delta &= 1 - \frac{d'}{p} = 0.82 & a' = a + \frac{d}{b} = 1.69 \text{ in } \leq \left(1.25 * b * \frac{d_b}{2}\right) = 1.91 \text{ in } \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \rho &= \frac{b'}{a'} = 0.70 & \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = 1.00 \text{ in } \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ \alpha' &= if \left[\beta => 1, 1, \min\left[1, \frac{1}{\delta} * \left(\frac{\beta}{1 - \beta}\right)\right]\right] = 1.00 \end{split}$$

Required bracket thickness to ensure an acceptable combination of fitting strength, stiffness, and bolt strength:

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{\rho r}*p*F_{u}*(1+\delta*\alpha')}} = 0.23 \text{ in LRFD} \quad (Eq. 9-23a)$$

$$Thickness of Bracket is 0.25 \text{ in - therefore, Prying OK}$$

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& ASSOCIATES	1/4" BRACKETS	Job No.:	18-001
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# Piece #33144A - L3" x 3" x 1/4" x 0'-4" Steel Angle Bracket (A36 min). w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

	Bracket Thickness:	$th_{bracket} = 0.2500$ in				
	Yield Strength:	$F_y = 36 ksi$	(Table 2-4)			
	Ultimate Strength:	$F_u = 58 \text{ ksi}$				
	Gross Area:	A <sub>g</sub> = (4.0in)*(0.25in) = 1.00 in2				
		$\phi_{t\cdot y} = 0.9$ LRFD	(Eqn D2.1)			
	Tensile Yielding:	$T_{allow-yielding} = \phi_{t-y} * F_y * A_g = 32.40 \text{ kip}$	(Eqn D2-1)			
	Shear Lag Factor:	<i>U</i> = 1.0	(Table D3.1)			
	Net Area:	$A_n = (4.0in)^*(0.25in) - (2^*0.3125in)^*(0.25in) = 0.84 in2$	(Sec B4.3)			
	Effective Net Area:	$A_{e} = A_{n}^{*}U = 0.84 \text{ in } 2$	(Eqn D3-1)			
		$\phi_{tr} = 0.75$ LRFD	(Eqn D2-2)			
	Tensile Rupture:	$T_{allow-rupture} = \phi_{t-r} * F_u * A_e = 36.70 \text{ kip}$	(Eqn D2-2)			
		$T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-supture}) = 32.40 kip LRFD$	(Vertical & Horizontal Component))			
Shear on Bracket Vertical Leg:						
	Gross Area:	$A_{gv} = (4.0in)^*(0.25in) = 1.00 in2$				
		$\Phi_{yv}$ = 1.00 LRFD	(Eqn J4-3)			
	Shear Yielding:	$V_{allow-yielding} = \phi_{yy} * 0.60 * F_y * A_{gv} = 21.60 \ kip$	(Eqn J4-3)			
		$     \Phi_{rv} = 0.75 $ LRFD	(Eqn J4-4)			
	Net Area:	$A_{nv} = (4.0in)^*(0.25in) - (2^*0.3125in)^*(0.25in) = 0.84 in2$	(Sec B4.3)			
	Shear Rupture:	$V_{allow-nupture} = \phi_{rv} * 0.60 * F_u * A_{rv} = 29.36 \ kip$	(Eqn J4-4)			
		$V_{allow}$ = min( $V_{allow-yielding}$ , $V_{allow-rupture}$ ) = 21.60 kip LRFD	(Horizontal Component)			
**Note: Bending of Bracket is Considered within the Prying Calculation						
Bending Moment on Vertical Leg:						
	Plastic Modulus:	$Z = \frac{bd^2}{4} : \frac{4*0.25^2}{4} = 0.0625 \text{ in } 3$				
		$\phi_b = 0.9$ LRFD	(Sec F1)			
		$M_{allow} = \phi *F_y *Z = 2.025 kip-in$				
	1 4	Mamont = 4.075 in				

Moment Arm: Moment arm = 1.375 in (Eqn F11-1) Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 1.473 \text{ kip LRFD}$  (Vertical Component)

Bending Moment on Lower Leg (Side B):

iding moment on Lower Lo	eg (Side D).		
Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{4*0.25^2}{4} =$	0.0625 in3	
	$\phi_b = 0.9$	LRFD	(Sec F1)
	$M_{allow} = \phi * F_y * Z =$	2.025 kip-in	(Eqn F11-1)
Moment Arm:	Moment <sub>arm</sub> = 1.375 in		

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 1.473 \text{ kip LRFD}$  (Horizontal Component)

R. F. NELSON		Date:	5/7/2019
& ASSOCIATES Structural Engineers	1/4" BRACKETS	Job No.: Sheet:	18-001 2 (3)
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## <u>Piece #33144A - L3" x 3" x 1/4" x 0'-4" Steel Angle Bracket (A36 min)</u> w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

#### Screws from Angle Bracket to Steel Sheet:

<u>Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6. LRFD Shear (Bearing) & Tension (Pull-Out) Capacity</u> (ICC-ESR-3332) Based on a Steel Member (min yield strength. Fy = 33 ksi & min tensile strength Fu = 45 ksi) Capacity of (1) Screw from 1/4", 3/8" & 1/2" Brackets to various design thickness steel sheets Shear Bearing Capacity first number is the minimum thickness of the steel in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet). Tensile Pull-out Capacity the number is for the steel sheet not in contact with the screw head (bottom sheet). Screw Capacities (Shear Bearing per ESR-3332, Table 3 & Tensile Pull-out per Table 5 per ESR-3332) LRFD

Design Thickness	V allow	Design Thickness	T <sub>allow</sub>	Number of Screws	
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	1206 lbf	420 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	2002 lbf	662 lbf
0.060-0.060	833 lbf	0.075	409 lbf	1666 lbf	818 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	2116 lbf	1096 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	2042 lbf	1794 lbf
		3/16"	1439 lbf		2878 lbf

#### Bolts thru Angle Bracket to Concrete Slab or Concrete-Filled Profile Steel Deck Failure Modes:

Hilti Kwik Bolt-TZ anchors may be installed in cracked or uncracked concrete or concrete-filled steel deck

Bolt type: A307 Gr. A (Common Bolt	s), bearing type connection		
Nominal Tensile Strength:	$F_t =$	45 ksi	
Bracket Thickness:	th <sub>bracket</sub> =	0.25 in	
Nominal Shear Strength, Threads Excluded:	F <sub>v</sub> =	27 ksi (Tabla 12 2	AISC 14th)
Bolt Diameter:	d <sub>bcd</sub> =	(Table J3.2. 0.50 in	AISC 14(11)
Bolt Area:	A <sub>bcd</sub> =	$0.25^{*}\pi^{*}d_{bcd}^{2} =$	0.20 in2
Resistance factor for bolt tension or shear:	$\phi =$	0.75	LRFD
Shear Capacity of single bearing type bolt:	V <sub>allow-bolt</sub> =	$\phi * F_v * A_{bcd} =$	6.63 kip
Tension Capacity of single bearing type bolt:	T <sub>allow-bolt</sub> =	$\phi * F_t * A_{bcd} =$	3.98 kip
Bolt bearing strength at bracket connection: (Section J3	10) F <sub>u</sub> =	58 ksi	
Bolt edge distance:	edge-dist =	2.00 in	
Bolt hole diameter:	bh =	0.625 in	
Clear distance between edge of hole and edge of adjacent hole or edge of plate:	$L_c$ = edge-dist -0.5*bh =	1.69 in2	
Single end bolt bearing capacity: Bolt b	<sub>r'g</sub> =min[(1.5*L <sub>c</sub> *th <sub>plate</sub> *F <sub>u</sub> ),(3.0	$d_{bcd}$ $th_{plate}$ $F_{u}] =$	21.75 kip

 $Bolt_{allow-bolt} = \phi *Bolt_{bearing}$  16.31 kip LRFD

#### <u>Piece #33144A - L3" x 3" x 1/4" x 0'-4" Steel Angle Bracket (A36 min)</u> w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

See Hilti Excel output for Allowable Combined Tension and Shear Loads for Concrete Over Metal Deck (LRFD) See Hilti Profis output for Allowable Combined Tension and Shear Loads (LRFD)

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

 $\phi N_n = 390 \, lbf$ 

 $\phi V_n = 440 \, lbf$ 

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

 $\phi N_n = 950 \, lbf$ 

 $\phi V_n = 1322 \, lbf$ 

Given that the Load and Resistance Factor Design calculated above for the angle brackets and bolts far outweigh the capacity of the concrete anchors, the allowable loading to the concrete anchors govern.
Note also that the capacity of the concrete anchors shown here is based on utilizing Section D.3.3.4.3 (d) of ACI 318-11, which requires the inclusion of the Omega factor when determining the loads applied to the anchorage.
Do to the complication of the requirement (per ACI 318-11) to determine the concrete anchorage capacity utilizing LRFD as well as Section 4.2 in ESR-1917 the allowable load for this Piece is given in LRFD only.

# <u>Overall Capacity of Seismic Load - Piece #33143A - 3" x 3" x 1/4" x 0'-4" Steel Angle Bracket (A36 min)</u> w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

Load allowable-total-on-concrete-ove-metal-deck = 390 lbf	Tension	
Load allowable-total-on-concrete-ove-metal-deck = 440 lbf	Shear	Vertical & Horizontal Allowable Load -
Load allowable-total-on-4" min-concrete-slab = 950 lbf	Tension	(LRFD)
Load allowable-total-on-4" min-concrete-slab = 1322 lbf	Shear	

R. F. NELSON		Date:	5/7/2019
& ASSOCIATES	1/4" BRACKETS	Job No.:	18-001
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Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/4" Floor & Wall Brackets (included within this calculation package for reference).

#### Prying of Piece #33146A - L3" x 3" x 1/4" x 0'-6" Steel Angle Bracket (A36 min) w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

AISC 14 Edition Part 9, p.9-10 of Specification

Bracket Thickness:	th <sub>bracket</sub> =	0.25 in		
Nominal Tensile Strength:	$F_t =$	60 ksi	(Carbon Steel)	
Ultimate Strength:	F <sub>u</sub> =	58 ksi		
Anchor Spacing	s =	4.00		
Anchor Diameter:	<i>d</i> <sub>b</sub> =	0.375 in		
Steel strength in tension:	N <sub>sa</sub> =	6.500 kip		
Max distance from bolt to applied force	: b =	1.375 in		
Tributary length to each bolt:	p = <i>max 2b,</i> but <u>&lt;</u> s	2.75 in		
Adjusted prying distance:	$b' = \left[b - \frac{d_b}{2}\right] =$	1.19 in	(Eq. 9-21)	
Resistance factor for prying:	$\Phi_{\rm pr} =$	1.67 ASD 0.90 LRFD 0.75 LRFD		
Maximum bolt force for prying:	$T = \frac{\Phi_{pr} * F_u * th}{2}$	$\frac{p_{bracket}^2 * p}{p} =$	3.26 kip	(Eq. 9-22a)

 $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = 3.26 \text{ kip}$ Maximum bolt force for prying:

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 4.88 \text{ kip}$  (per ESR-1917)

$$t_{c} = \sqrt{\frac{4^{*}B^{*}b'}{\Phi_{LRFD}^{*}p^{*}F_{u}}} = 0.440 \text{ in}$$
 (Eq. 9-30a)

(Eq. 9-31)

 $T_{avail} = BQ = 2.86 kip$ 

$$Q = 0.587 \qquad a' \leq Q = 1$$

$$0 \leq a' \leq 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta^* \alpha') = 0.587 \qquad a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.587$$
(Eq. 9-33)
(Eq. 34)

Width of hole along length of plate: d' = 0.500 in

Distance from bolt centerline to edge of plate: a = 1.50 in

Additional variables for prying calculation:

$$\begin{split} \delta &= 1 - \frac{d'}{p} = 0.82 & a' = a + \frac{d_{\pm}}{2} = 1.69 \text{ in } \leq \left(1.25 * b * \frac{d_{b}}{2}\right) = 1.91 \text{ in } \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \mathcal{P} &= \frac{b'}{\alpha'} = 0.70 & \beta = \frac{1}{p} * \left(\frac{B}{T} - 1\right) = 1.00 \text{ in } \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ \alpha' &= d' \left[\beta => 1, 1, \min\left[1, \frac{1}{\delta} * \left(\frac{\beta}{1 - \beta}\right)\right]\right] = 1.00 \end{split}$$

Required bracket thickness to ensure an acceptable combination of fitting strength, stiffness, and bolt strength:

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{\rho r}*p*F_{\mu}*(1+\delta*\alpha')}} = 0.23 \text{ in LRFD} \quad (Eq. 9-23a)$$

$$Thickness of Bracket is 0.25 in - therefore, Prying OK$$

R. F. NELSON		Date:	1/23/2020
& ASSOCIATES	1/4" BRACKETS	Job No.:	18-001
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#### Piece #33146A - L3" x 3" x 1/4" x 0'-6" Steel Angle Bracket (A36 min)\_ w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

#### Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness:	<i>th</i> <sub>bracket</sub> = 0.2500 in	
Yield Strength:	$F_{\gamma} = 36 \text{ ksi}$	(Table 2-4)
Ultimate Strength:	$F_u = 58  ksi$	
Gross Area:	A g = (6.0in)*(0.25in) = 1.50 in2	
	$\phi_{t-y} = 0.9$ LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow-yielding} = \phi_{ty} * F_y * A_g = 48.60 \text{ kip}$	(Eqn D2-1)
Shear Lag Factor:	U= 1.0	(Table D3.1)
Net Area:	$A_n = (6.0in)^*(0.25in) - (4^*0.3125in)^*(0.25in) = 1.19 in2$	(Sec B4.3)
Effective Net Area:	$A_e = A_n * U = 1.19 in2$	(Eqn D3-1)
	$\phi_{tr} = 0.75 \qquad LRFD$	(Eqn D2-2)
Tensile Rupture:	$T_{allow-rupture} = \phi_{t,r} * F_u * A_e = 51.66 \text{ kip}$	(Eqn D2-2)
	$T_{allow-brackef} = min(T_{allow-yielding}, T_{allow-rupture}) = 48.60 \text{ kip LRFD}$	(Vertical & Horizontal Component))

#### Shear on Bracket Vertical Leg:

Gross Area:	$A_{gv} = (6.0in)^*(0.25in) = 1.50 in2$			
	$     \Phi_{yv} = 1.00 $ LRFD	(Eqn J4-3)		
Shear Yielding:	$V_{allow-yielding} = \phi_{yv} * 0.60 * F_{y} * A_{gv} = 32.40 \text{ kip}$	(Eqn J4-3)		
	$     \Phi_{\rm rv} = 0.75 $ LRFD	(Eqn J4-4)		
Net Area:	$A_{nv} = (6.0in)^*(0.25in) - (4^*0.3125in)^*(0.25in) = 1.19 in2$	(Sec B4.3)		
Shear Rupture:	$V_{allow-cupture} = \phi_{rv} * 0.60 * F_u * A_{nv} = 41.33  kip$	(Eqn J4-4)		
	$V_{allow} = min(V_{allow-yielding}, V_{allow-rupture}) = 32.40 kip LRFD$	(Horizontal Component)		

\*\*Note: Bending of Bracket is Considered within the Prying Calculation

#### Bending Moment on Vertical Leg:

Plastic Modulus:	$Z = \frac{bd^2}{4} : \frac{6*0.25^2}{4} =$	= 0.0938 in3	
	$\phi_b = 0.9$	LRFD	(Sec F1)
	$M_{allow} = \phi * F_y * Z$	= 3.038 kip-in	
Moment Arm:	Moment <sub>arm</sub> = 1.375 in		(Eqn F11-1)

# Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 2.209 \text{ kip LRFD}$ (Vertical Component)

# Bending Moment on Lower Leg (Side B):

Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{6*0.25^2}{4} =$	0.0938 in3	
	$\phi_b = 0.9$	LRFD	(Sec F1)
	$M_{allow} = \phi * F_y * Z =$	3.038 kip-in	(Eqn F11-1)
Moment Arm:	Moment <sub>arm</sub> = 1.375 in		

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 2.209 \text{ kip LRFD}$  (Horizontal Component)

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& ASSOCIATES	1/4" BRACKETS	Job No.:	18-001
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<u>Piece #33146A - L3" x 3" x 1/4" x 0'-6" Steel Angle Bracket (A36 min)</u> w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

#### Screws from Angle Bracket to Steel Sheet:

<u>Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6, LRFD Shear (Bearing) & Tension (Pull-Out) Capacity</u> (ICC-ESR-3332) Based on a Steel Member (min yield strength, Fy = 33 ksi & min tensile strength Fu = 45 ksi) Capacity of (1) Screw from 1/4", 3/8" & 1/2" Brackets to various design thickness steel sheets Shear Bearing Capacity first number is the minimum thickness of the steel in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet). Tensile Pull-out Capacity the number is for the steel sheet not in contact with the screw head (bottom sheet). Screw Capacities (Shear Bearing per ESR-3332, Table 3 & Tensile Pull-out per Table 5 per ESR-3332) LRFD

Design Thickness (in)	V <sub>allow</sub>	Design Thickness (in)	T <sub>allow</sub>	Number of Screws V <sub>allow</sub> *N	$N = 4$ $T_{allow} *N$
0.048-0.048	603 lbf	0.048	210 lbf	2412 lbf	840 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	4004 lbf	1324 lbf
0.060-0.060	833 lbf	0.075	409 lbf	3332 lbf	1636 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	4232 lbf	2192 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	4084 lbf	3588 lbf
		3/16"	1439 lbf		5756 lbf

#### Bolts thru Angle Bracket to Concrete Slab or Concrete-Filled Profile Steel Deck Failure Modes:

Hilti Kwik Bolt-TZ anchors may be installed in cracked or uncracked concrete or concrete-filled steel deck

Bolt type	Bolt type: A307 Gr. A (Common Bolts), bearing type connection				
Nominal	Tensile Strength:	$F_t =$	45 ksi		
Bracket	Thickness:	th <sub>bracket</sub> =	0.25 in		
Nominal	Shear Strength, Threads Excluded:	F <sub>v</sub> =	27 ksi (Tabla 12.0	A 100 d dth)	
Bolt Diar	neter:	d <sub>bcd</sub> =	(Table J3.2 / 0.50 in	AISC 14th)	
Bolt Area	a:	A <sub>bcd</sub> =	$0.25^* \pi^* d_{bcd}^2 =$	0.20 in2	
Resistan	ce factor for bolt tension or shear:	$\phi =$	0.75	LRFD	
Shear Ca	apacity of single bearing type bolt:	V <sub>allow-bolt</sub> =	$\phi * F_v * A_{bcd} =$	3.98 kip	
Tension	Capacity of single bearing type bolt:	$T_{allow-bolt} =$	$\phi * F_t * A_{bcd} =$	6.63 kip	
Bolt bearing streng	gth at bracket connection: (Section J3.	10) $F_u =$	58 ksi		
Bolt edge	e distance:	edge-dist =	1.00 in		
Bolt hole	diameter:	bh =	0.625 in		
	stance between edge of hole e of adjacent hole or edge of plate:	$L_c$ = edge-dist -0.5*bh =	0.69 in2		
Single er	nd bolt bearing capacity: Bolt b	<sub>r'g</sub> =min[(1.5*L <sub>c</sub> *th <sub>plate</sub> *F <sub>u</sub> ),(3.0 <sup>-</sup>	$d_{bcd} th_{plate} F_u] =$	14.95 kip	

 $2*Bolt_{allow-bolt} = 2*\phi *Bolt_{bearing}$  22.43 kip LRFD

#### <u>Piece #33146A - L3" x 3" x 1/4" x 0'-6" Steel Angle Bracket (A36 min)</u> w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

See Hilti Excel output for Allowable Combined Tension and Shear Loads for Concrete Over Metal Deck (LRFD) See Hilti Profis output for Allowable Combined Tension and Shear Loads for Concrete Slab (LRFD)

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

 $\phi N_n = 570 \, lbf$ 

 $\phi V_n = 1100 \, lbf$ 

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

 $\phi N_n = 1275 \, lbf$ 

 $\phi V_n = 2552 \, lbf$ 

Given that the Load and Resistance Factor Design calculated above for the angle brackets and bolts far outweigh the capacity of the concrete anchors, the allowable loading to the concrete anchors govern.
Note also that the capacity of the concrete anchors shown here is based on utilizing Section D.3.3.4.3 (d) of ACI 318-11, which requires the inclusion of the Omega factor when determining the loads applied to the anchorage.
Do to the complication of the requirement (per ACI 318-11) to determine the concrete anchorage capacity utilizing LRFD as well as Section 4.2 in ESR-1917 the allowable load for this Piece is given in LRFD only.

#### <u>Overall Capacity of Seismic Load - Piece #33146A - 3" x 3" x 1/4" x 0'-6" Steel Angle Bracket (A36 min)</u> w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

Load allowable-total-on-concrete-ove-metal-deck = 570 lbf	Tension	
Load allowable-total-on-concrete-ove-metal-deck = 1100 lbf	Shear	Vertical & Horizontal Allowable Load -
Load allowable-total-on-4" min-concrete-slab = 1275 lbf	Tension	(LRFD)
Load allowable-total-on-4" min-concrete-slab = 2552 lbf	Shear	

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& ASSOCIATES	1/4" BRACKETS	Job No.:	18-001
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Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/4" Floor & Wall Brackets (included within this calculation package for reference).

#### Prying of Piece #33148A - L3" x 3" x 1/4" x 0'-8" Steel Angle Bracket (A36 min) w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

AISC 14 Edition Part 9, p.9-10 of Specification

Bracket Thickness:	th <sub>bracket</sub> =	0.25 in	
Nominal Tensile Strength:	$F_t =$	60 ksi	(Carbon Steel)
Ultimate Strength:	F <sub>u</sub> =	58 ksi	
Anchor Spacing	s =	3.00	
Anchor Diameter:	<i>d</i> <sub>b</sub> =	0.375 in	
Steel strength in tension:	N <sub>sa</sub> =	6.500 kip	
Max distance from bolt to applied force:	b =	1.375 in	
Tributary length to each bolt:	<i>p = max 2b,</i> but <u>&lt;</u> s	2.75 in	
Adjusted prying distance:	$b' = \left[b - \frac{d_b}{2}\right] =$	1.19 in	(Eq. 9-21)
Resistance factor for prying:	Φ <sub>pr</sub> =	1.67 ASD 0.90 LRFD 0.75 LRFD	
	$\Psi_t = \Phi_t + E_t$		

Maximum bolt force for prying:  $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = 3.26 \text{ kip}$ 

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 4.88 \text{ kip}$  (per ESR-1917)

$$t_{c} = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_{u}}} = 0.440 \text{ in}$$
 (Eq. 9-30a)

(Eq. 9-22a)

(Eq. 9-31)

 $T_{avail} = BQ = 2.86 kip$ 

$$Q = 0.587 \qquad a' < Q = 1$$
  
$$0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta^* \alpha') = 0.587 \qquad a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.587$$

(Eq. 9-33) (Eq. 34) Width of hole along length of plate: d' = 0.500 in

Distance from bolt centerline to edge of plate: a = 1.50 in

Additional variables for prying calculation:

$$\begin{split} \delta &= 1 - \frac{d'}{p} = 0.82 & a' = a + \frac{d}{2} = 1.69 \text{ in } \leq \left(1.25 * b * \frac{d_b}{2}\right) = 1.91 \text{ in } \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \rho &= \frac{b'}{\alpha} = 0.70 & \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = 1.00 \text{ in } \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ \alpha' &= if \left[\beta => 1, 1, \min\left[1, \frac{1}{\delta} * \left(\frac{\beta}{1 - \beta}\right)\right]\right] = 1.00 \end{split}$$

Required bracket thickness to ensure an acceptable combination of fitting strength, stiffness, and bolt strength:

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr}*p*F_{u}*(1+\delta*\alpha')}} = 0.23 \text{ in LRFD} \quad (Eq. 9-23a)$$

$$Thickness of Bracket is 0.25 in - therefore, Prying OK$$

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& ASSOCIATES	1/4" BRACKETS	Job No.:	18-001
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#### Piece #33148A - L3" x 3" x 1/4" x 0'-8" Steel Angle Bracket (A36 min)\_ w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

#### Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness:	<i>th</i> <sub>bracket</sub> = 0.2500 in	
Yield Strength:	$F_{y} = 36  ksi$	(Table 2-4)
Ultimate Strength:	F <sub>u</sub> = 58 ksi	
Gross Area:	A g = (8.0in)*(0.25in) = 2.00 in2	
	$\phi_{ty} = 0.9$ LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow-yialding} = \phi_{t-y} *F_y *A_g = 64.80 \text{ kip}$	(Eqn D2-1)
Shear Lag Factor:	U= 1.0	(Table D3.1)
Net Area:	$A_n = (8.0in)^*(0.25in) - (5^*0.3125in)^*(0.25in) = 1.61 in2$	(Sec B4.3)
Effective Net Area:	$A_e = A_n * U = 1.61 in2$	(Eqn D3-1)
	$\phi_{tr} = 0.75$ LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow-cupture} = \phi_{t.r} * F_u * A_e = 70.01 \text{ kip}$	(Eqn D2-2)
	$T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 64.80 \text{ kip LRFD}$	(Vertical & Horizontal

#### Shear on Bracket Vertical Leg:

Gross Area:	$A_{gv} = (8.0in)^*(0.25in) = 2.00 in2$		
	$     \Phi_{yy} = 1.00 $ LRFD	(Eqn J4-3)	
Shear Yielding:	$V_{allow-yielding} = \phi_{yy} *0.60 * F_y * A_{gy} = 43.20 kip$	(Eqn J4-3)	
	$     \Phi_{rr} = 0.75 $ LRFD	(Eqn J4-4)	
Net Area:	$A_{nv} = (8.0in)^*(0.25in) \cdot (5^*0.3125in)^*(0.25in) = 1.61 in2$	(Sec B4.3)	
Shear Rupture:	$V_{ellow-rupture} = \phi_{rv} * 0.60 * F_u * A_{rv} = 56.01  kip$	(Eqn J4-4)	
	$V_{allow} = min(V_{allow-yielding}, V_{allow-rupture}) = 43.20 \text{ kip LRFD}$	(Horizontal Component)	

Component))

\*\*Note: Bending of Bracket is Considered within the Prying Calculation

#### Bending Moment on Vertical Leg (Side A):

Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{8*0.25^2}{4} = \frac{1}{4}$	= 0.1250 in3		
	$\phi_{b} = 0.9$	LRFD		(Sec F1)
	$M_{allow} = \phi * F_y * Z =$	= 4.050 kip-in		
Moment Arm:	Moment <sub>arm</sub> = 1.375 in			(Eqn F11-1)
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$	= 2.945 kip LRFD	(Vertical Compone	ent)

#### Bending Moment on Lower Leg (Side B):

Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{8*0.}{4}$	$\frac{25^2}{2} = 0.1250 \text{ in } 3$	
	$\phi_b = 0.9$	LRFD	(Sec F1)
	$M_{allow} = \phi * F$	<sub>y</sub> *Z = 4.050 kip-in	(Eqn F11-1)

Moment Arm: Moment arm = 1.375 in	Moment Arm:	Moment <sub>arm</sub> =	1.375 in
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Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 2.945 \text{ kip LRFD}$  (Horizontal Component)

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<u>Piece #33148A - L3" x 3" x 1/4" x 0'-8" Steel Angle Bracket (A36 min)</u> w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

#### Screws from Angle Bracket to Steel Sheet:

 Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6, LRFD Shear (Bearing) & Tension (Pull-Out) Capacity

 (ICC-ESR-3332) Based on a Steel Member (min yield strength, Fy = 33 ksi & min tensile strength Fu = 45 ksi)

 Capacity of (1) Screw from 1/4", 3/8" & 1/2" Brackets to various design thickness steel sheets

 Shear Bearing Capacity first number is the minimum thickness of the steel in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet).

 Tensile Pull-out Capacity the number is for the steel sheet not in contact with the screw head (bottom sheet).

 Screw Capacities (Shear Bearing per ESR-3332, Table 3 & Tensile Pull-out per Table 5 per ESR-3332) LRFD

Design Thickness	V allow	Design Thickness	T <sub>allow</sub>	Number of Screws I	V = 5
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	3015 lbf	1050 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	5005 lbf	1655 lbf
0.060-0.060	833 lbf	0.075	409 lbf	4165 lbf	2045 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	5290 lbf	2740 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	5105 lbf	4485 lbf
		3/16"	1439 lbf		7195 lbf

# Bolts thru Angle Bracket to Concrete Slab or Concrete-Filled Profile Steel Deck Failure Modes:

Hilti Kwik Bolt-TZ anchors may be installed in cracked or uncracked concrete or concrete-filled steel deck

Bolt type: A307 Gr. A (Com	mon Bolts), bearing type connection		
Nominal Tensile Strength:	F <sub>t</sub> =	45 ksi	
Bracket Thickness:	th <sub>bracket</sub> =	0.25 in	
Nominal Shear Strength, Threads Exc	luded: F <sub>v</sub> =	27 ksi (Table J3.2 )	AISC 14th)
Bolt Diameter:	d <sub>bcd</sub> =	0.50 in	AISC 14(1)
Bolt Area:	A <sub>bcd</sub> =	$0.25^{*}\pi^{*}d_{bcd}^{2} =$	0.20 in2
Resistance factor for bolt tension or sh	<i>ear:</i> $\phi =$	0.75	LRFD
Shear Capacity of single bearing type	bolt: V <sub>allow-bolt</sub> =	$\phi * F_v * A_{bcd} =$	7.07 kip
Tension Capacity of single bearing typ	e bolt: T <sub>allow-bolt</sub> =	$\phi * F_t * A_{bcd} =$	11.78 kip
Bolt bearing strength at bracket connection: (Se	ction J3.10) $F_u =$	58 ksi	
Bolt edge distance:	edge-dist =	1.00 in	
Bolt hole diameter:	bh =	0.625 in	
Clear distance between edge of hole and edge of adjacent hole or edge of p	$L_c$ = edge-dist -0.5*bh = late:	0.69 in2	
Single bolt bearing capacity:	Bolt $_{brg}$ =min[(1.5*L $_{c}$ *th $_{plate}$ *F $_{u}$ ),(3.0	$d_{bcd} *th_{plate} *F_u)] =$	14.95 kip

 $3*Bolt_{allow-bolt} = 3*\phi *Bolt_{bearing}$  33.64 kip LRFD

#### <u>Piece #33148A - L3" x 3" x 1/4" x 0'-8" Steel Angle Bracket (A36 min)</u> w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

See Hilti Excel output for Allowable Combined Tension and Shear Loads for Concrete Over Metal Deck (LRFD) See Hilti Profis output for Allowable Combined Tension and Shear Loads for Concrete Slab (LRFD)

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

 $\phi N_n = 650 \, lbf$ 

 $\phi V_n = 1220 \, lbf$ 

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

 $\phi N_n = 1400 \, lbf$ 

 $\phi V_n = 3130 \, lbf$ 

Given that the Load and Resistance Factor Design calculated above for the angle brackets and bolts far outweigh the capacity of the concrete anchors, the allowable loading to the concrete anchors govern.
Note also that the capacity of the concrete anchors shown here is based on utilizing Section D.3.3.4.3 (d) of ACI 318-11, which requires the inclusion of the Omega factor when determining the loads applied to the anchorage.
Do to the complication of the requirement (per ACI 318-11) to determine the concrete anchorage capacity utilizing LRFD as well as Section 4.2 in ESR-1917 the allowable load for this Piece is given in LRFD only.

#### Overall Capacity of Seismic Load - Piece #33148A - 3" x 3" x 1/4" x 0'-8" Steel Angle Bracket (A36 min) w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

Load allowable-total-on-concrete-ove-metal-deck = 650 lbf	Tension	
Load allowable-total-on-concrete-ove-metal-deck = 1220 lbf	Shear	Vertical & Horizontal Allowable Load -
Load allowable-total-on-4" min-concrete-slab = 1400 lbf	Tension	(LRFD)
Load allowable-total-on-4" min-concrete-slab = 3130 lbf	Shear	

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Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/4" Floor & Wall Brackets (included within this calculation package for reference).

#### Prying of Piece #43143A - L4" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min) w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

AISC 14 Edition Part 9, p.9-10 of Specification

Bracket Thickness:	th <sub>bracket</sub> =	0.25 in		
Nominal Tensile Strength:	$F_t =$	60 ksi	(Carbon Steel)	
Ultimate Strength:	F <sub>u</sub> =	58 ksi		
Anchor Spacing	s =	0.00		
Anchor Diameter:	<i>d</i> <sub>b</sub> =	0.375 in		
Steel strength in tension:	N <sub>sa</sub> =	6.500 kip		
Max distance from bolt to applied force	e: b =	1.375 in		
Tributary length to each bolt:	p = <i>max 2b,</i> but <u>&lt;</u> s	2.75 in		
Adjusted prying distance:	$b' = \left[b - \frac{d_b}{2}\right] =$	1.19 in	(Eq. 9-21)	
Resistance factor for prying:	$\Phi_{\rm pr} =$	1.67 ASD 0.90 LRFD 0.75 LRFD		
Maximum bolt force for prying:	$T = \frac{\Phi_{pr} * F_u * th}{2}$	$\frac{p_{bracket}^2 * p}{p} =$	3.26 kip	(Eq. 9-22a)

Maximum bolt force for prying:  $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = -3.26 \text{ kip}$ 

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 4.88 \text{ kip}$  (per ESR-1917)

$$t_{c} = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_{u}}} = 0.440 \text{ in}$$
 (Eq. 9-30a)

 $T_{avail} = BQ = 2.86 kip$ 

(Eq. 9-31)

$$Q = 0.587 \qquad a' < Q = 1$$

$$0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta^* \alpha') = 0.587 \qquad a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.587 \qquad (Eq. 9-33) \qquad (Eq. 34)$$

Width of hole along length of plate: d' = 0.500 in

Distance from bolt centerline to edge of plate: a = 1.50 in

Additional variables for prying calculation:

$$\begin{split} \delta &= 1 - \frac{d}{\rho} = 0.82 & a' = a + \frac{d}{2} = 1.69 \text{ in } \leq \left( 1.25 * b * \frac{d}{2} \right) = 1.91 \text{ in } \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \rho &= \frac{b}{\alpha'} = 0.70 & \beta = \frac{1}{\rho} * \left( \frac{B}{T} - 1 \right) = 1.00 \text{ in } \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ \alpha' &= if \left[ \rho => 1, 1, \min \left[ 1, \frac{1}{\delta} * \left( \frac{\rho}{1 - \rho} \right) \right] \right] = 1.00 \end{split}$$

Required bracket thickness to ensure an acceptable combination of fitting strength, stiffness, and bolt strength:

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr}*p*F_{u}*(1+\delta*\alpha')}} = 0.23 \text{ in LRFD} \quad (Eq. 9-23a)$$

$$Thickness of Bracket is 0.25 in - therefore, Prying OK$$

R. F. NELSON		Date:	1/23/2020
& ASSOCIATES	1/4" BRACKETS	Job No.:	18-001
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# Piece #43143A - L4" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min) w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

#### Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness:	<i>th</i> <sub>bracket</sub> = 0.2500 in	
Yield Strength:	$F_{\gamma} = 36  ksi$	(Table 2-4)
Ultimate Strength:	F <sub>u</sub> = 58 ksi	
Gross Area:	A g = (3.0in)*(0.25in) = 0.75 in2	
	$\phi_{t-y} = 0.9$ LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow-yialding} = \phi_{ty} * F_y * A_g = 24.30 \text{ kip}$	(Eqn D2-1)
Shear Lag Factor:	U= 1.0	(Table D3.1)
Net Area:	$A_n = (3.0in)^*(0.25in) \cdot (2^*0.3125in)^*(0.25in) = 0.59 in2$	(Sec B4.3)
Effective Net Area:	$A_{e} = A_{n} * U = 0.59 in2$	(Eqn D3-1)
	$\phi_{tr} = 0.75$ LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow-rupture} = \phi_{t-r} *F_u *A_e = 25.83 \text{ kip}$	(Eqn D2-2)
	$T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 24.30 kip LRFD$	(Vertical & Horizontal

#### Shear on Bracket Vertical Leg:

Gross Area:	$A_{gv} = (3.0in)^*(0.25in) = 0.75 in2$	
	$     \Phi_{yv} = 1.00 $ LRFD	(Eqn J4-3)
Shear Yielding:	$V_{allow-yielding} = \phi_{yv} * 0.60 * F_y * A_{gv} = 16.20 \ kip$	(Eqn J4-3)
	$ \Phi_{_{IV}} = 0.75 $ LRFD	(Eqn J4-4)
Net Area:	$A_{nv} = (3.0in)^*(0.25in) - (2^*0.3125in)^*(0.25in) = 0.59 in2$	(Sec B4.3)
Shear Rupture:	$V_{allow-rupture} = \phi_{rv} * 0.60 * F_{u} * A_{nv} = 20.66 \ kip$	(Eqn J4-4)
	$V_{allow} = min(V_{allow-yielding}, V_{allow-rupture}) = 16.20 \text{ kip LRFD}$	(Horizontal Component)

Component))

\*\*Note: Bending of Bracket is Considered within the Prying Calculation

#### Bending Moment on Vertical Leg:

Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{3*0.25^2}{4} =$	0.0469 in3		
	$\phi_b = 0.9$	LRFD		(Sec F1)
	$M_{allow} = \phi * F_y * Z =$	1.519 kip-in		
Moment Arm:	Moment <sub>arm</sub> = 2.875 in			(Eqn F11-1)
Allowable Load:	$P_{allow} = \frac{M_{allow}}{M_{allow}} =$	0.528 kip LRFD	(Vertical Compone	nt)

# $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 0.528 \text{ kip LRFD} \quad (\text{Vertical Component})$

#### Bending Moment on Lower Leg (Side B):

Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{3*0.25^2}{4}$	-= 0.0469 in3	
	$\phi_{b} = 0.9$	LRFD	(Sec F1)
	$M_{allow} = \phi * F_y *.$	Z = 1.519 kip-in	(Eqn F11-1)

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = -1.105 \text{ kip LRFD}$  (Horizontal Component)

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	/4" BRACKETS	

## <u>Piece #43143A - L4" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min)</u> w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

#### Screws from Angle Bracket to Steel Sheet:

<u>Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6. LRFD Shear (Bearing) & Tension (Pull-Out) Capacity</u> (ICC-ESR-3332) Based on a Steel Member (min yield strength. Fy = 33 ksi & min tensile strength Fu = 45 ksi) Capacity of (1) Screw from 1/4", 3/8" & 1/2" Brackets to various design thickness steel sheets Shear Bearing Capacity first number is the minimum thickness of the steel in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet). Tensile Pull-out Capacity the number is for the steel sheet not in contact with the screw head (bottom sheet). Screw Capacities (Shear Bearing per ESR-3332, Table 3 & Tensile Pull-out per Table 5 per ESR-3332) LRFD

Design Thickness	V allow	Design Thickness	T <sub>allow</sub>	Number of Screws	N = 2
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	1206 lbf	420 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	2002 lbf	662 lbf
0.060-0.060	833 lbf	0.075	409 lbf	1666 lbf	818 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	2116 lbf	1096 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	2042 lbf	1794 lbf
		3/16"	1439 lbf		2878 lbf

#### Bolts thru Angle Bracket to Concrete Slab or Concrete-Filled Profile Steel Deck Failure Modes:

Hilti Kwik Bolt-TZ anchors may be installed in cracked or uncracked concrete or concrete-filled steel deck

Bolt type: A307 Gr. A (	Common Bolts), bearing type conr	nection		
Nominal Tensile Strength:		$F_t =$	45 ksi	
Bracket Thickness:		th <sub>bracket</sub> =	0.25 in	
Nominal Shear Strength, Threads	Excluded:	$F_v =$		AISC 1 4th)
Bolt Diameter:		$d_{bcd} =$	(Table J3.2 ) 0.50 in	4130 14(11)
Bolt Area:		A <sub>bcd</sub> =	$0.25^* \pi * d_{bcd}^2 =$	0.20 in2
Resistance factor for bolt tension	or shear:	$\phi =$	0.75	LRFD
Shear Capacity of single bearing	type bolt:	/ <sub>allow-bolt</sub> =	$\phi * F_v * A_{bcd} =$	3.98 kip
Tension Capacity of single bearin	g type bolt:	T <sub>allow-bolt</sub> =	$\phi *F_t *A_{bcd} =$	6.63 kip
Bolt bearing strength at bracket connection	: (Section J3.10)	$F_u =$	58 ksi	
Bolt edge distance:	ec	dge-dist =	1.50 in	
Bolt hole diameter:		bh =	0.625 in	
Clear distance between edge of h and edge of adjacent hole or edge	• •	:-0.5*bh =	1.19 in2	
Single end bolt bearing capacity:	Bolt <sub>br'g</sub> =min[(1.5*L <sub>c</sub> *th <sub>plat</sub>	<sub>e</sub> *F <sub>u</sub> ),(3.0*	$d_{bcd} * th_{plate} * F_u)] =$	21.75 kip

 $Bolt_{allow-bolt} = \phi *Bolt_{bearing}$  16.31 kip LRFD

#### <u>Piece #43143A - L4" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min)</u> w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

See Hilti Excel output for Allowable Combined Tension and Shear Loads for Concrete Over Metal Deck (LRFD) See Hilti Profis output for Allowable Combined Tension and Shear Loads for Concrete Slab (LRFD)

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

 $\phi N_n = 390 \, lbf$ 

 $\phi V_n = 440 \, lbf$ 

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

 $\phi N_n = 950 \, lbf$ 

 $\phi V_n = 1322 \, lbf$ 

Given that the Load and Resistance Factor Design calculated above for the angle brackets and bolts far outweigh the capacity of the concrete anchors, the allowable loading to the concrete anchors govern.
Note also that the capacity of the concrete anchors shown here is based on utilizing Section D.3.3.4.3 (d) of ACI 318-11, which requires the inclusion of the Omega factor when determining the loads applied to the anchorage.
Do to the complication of the requirement (per ACI 318-11) to determine the concrete anchorage capacity utilizing LRFD as well as Section 4.2 in ESR-1917 the allowable load for this Piece is given in LRFD only.

#### <u>Overall Capacity of Seismic Load - Piece #43143A - 4" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min)</u> w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

Load allowable-total-on-concrete-ove-metal-deck = 390 lbf	Tension	
Load allowable-total-on-concrete-ove-metal-deck = 440 lbf	Shear	Vertical & Horizontal Allowable Load -
Load allowable-total-on-4" min-concrete-slab = 950 lbf	Tension	(LRFD)
Load allowable-total-on-4" min-concrete-slab = 1322 lbf	Shear	

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& ASSOCIATES	1/4" BRACKETS	Job No.:	18-001
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Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/4" Floor & Wall Brackets (included within this calculation package for reference).

#### Prying of Piece #43146A - L4" x 3" x 1/4" x 0'-6" Steel Angle Bracket (A36 min) w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

AISC 14 Edition Part 9, p.9-10 of Specification

Bracket Thickness:	th <sub>bracket</sub> =	0.25 in		
Nominal Tensile Strength:	$F_t =$	60 ksi	(Carbon Steel)	
Ultimate Strength:	F <sub>u</sub> =	58 ksi		
Anchor Spacing	s =	4.00		
Anchor Diameter:	<i>d</i> <sub>b</sub> =	0.375 in		
Steel strength in tension:	N <sub>sa</sub> =	6.500 kip		
Max distance from bolt to applied force	e: b =	1.375 in		
Tributary length to each bolt:	<i>p = max 2b,</i> but <u>&lt;</u> s	2.75 in		
Adjusted prying distance:	$b' = \left[b - \frac{d_b}{2}\right] =$	1.19 in	(Eq. 9-21)	
Resistance factor for prying:	$\Phi_{\rm pr} =$	1.67 ASD 0.90 LRFD 0.75 LRFD		
Maximum bolt force for prying:	$T = \frac{\Phi_{pr} * F_u * th}{2}$	$h_{bracket}^{2} * p =$	3.26 kip	(Eq. 9-22a)

Maximum bolt force for prying:  $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = 3.26 \text{ kip}$ 

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 4.88 \text{ kip}$  (per ESR-1917)

$$t_{c} = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_{u}}} = 0.440 \text{ in}$$
 (Eq. 9-30a)

 $T_{avail} = BQ = 2.86 kip$ 

(Eq. 9-31)

$$Q = 0.587 \qquad a' < Q = 1$$

$$0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.587 \qquad a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.587 \qquad (Eq. 9-33) \qquad (Eq. 34)$$

Width of hole along length of plate: d' = 0.500 in

Distance from bolt centerline to edge of plate: a = 1.50 in

Additional variables for prying calculation:

$$\begin{split} \delta &= 1 - \frac{d}{\rho} = 0.82 & a' = a + \frac{d}{2} = 1.69 \text{ in } \leq \left( 1.25 * b * \frac{d}{2} \right) = 1.91 \text{ in } \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \rho &= \frac{b}{\alpha'} = 0.70 & \beta = \frac{1}{\rho} * \left( \frac{B}{T} - 1 \right) = 1.00 \text{ in } \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ \alpha' &= if \left[ \rho => 1, 1, \min \left[ 1, \frac{1}{\delta} * \left( \frac{\rho}{1 - \rho} \right) \right] \right] = 1.00 \end{split}$$

Required bracket thickness to ensure an acceptable combination of fitting strength, stiffness, and bolt strength:

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr}*p*F_u*(1+\delta*\alpha')}} = 0.23 \text{ in LRFD} \quad (Eq. 9-23a)$$

$$Thickness of Bracket is 0.25 in - therefore, Prying OK$$

R. F. NELSON		Date:	1/23/2020
& ASSOCIATES	1/4" BRACKETS	Job No.:	18-001
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#### Piece #43146A - L4" x 3" x 1/4" x 0'-6" Steel Angle Bracket (A36 min). w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

# Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness:	$th_{bracket} = 0.2500$ in	
Yield Strength:	$F_y = 36  ksi$	(Table 2-4)
Ultimate Strength:	$F_u = 58  ksi$	
Gross Area:	A <sub>g</sub> = (6.0in)*(0.25in) = 1.50 in2	
	$\phi_{t-y} = 0.9$ LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow-yielding} = \phi_{t,y} * F_y * A_g = 48.60 \text{ kip}$	(Eqn D2-1)
Shear Lag Factor:	<i>U</i> = 1.0	(Table D3.1)
Net Area:	$A_n = (6.0in)^*(0.25in) - (4^*0.3125in)^*(0.25in) = 1.19in2$	(Sec B4.3)
Effective Net Area:	$A_e = A_n^* U = 1.19 in2$	(Eqn D3-1)
	$\phi_{tr} = 0.75$ LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow-cupture} = \phi_{tr} * F_u * A_e = 51.66 \text{ kip}$	(Eqn D2-2)
	$T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 48.60 \text{ kip LRFD}$	(Vertical & Horizontal Component))
Shear on Bracket Vertical Le		Horizontal
<u>Shear on Bracket Vertical Le</u> Gross Area:		Horizontal
	<u>a:</u>	Horizontal
	a: A <sub>gv</sub> =(6.0in)*(0.25in) = 1.50 in2	Horizontal Component))
Gross Area:	$\frac{g}{a_{gv}} = (6.0in)^* (0.25in) = 1.50 in2$ $\Phi_{yv} = 1.00 \qquad LRFD$	Horizontal Component)) (Eqn J4-3)
Gross Area:	$g:$ $A_{gv} = (6.0in)^* (0.25in) = 1.50 in2$ $\Phi_{yv} = 1.00 \qquad LRFD$ $V_{allow-yielding} = \phi_{yv} * 0.60^* F_y * A_{gv} = 32.40 kip$	Horizontal Component)) (Eqn J4-3) (Eqn J4-3)
Gross Area: Shear Yielding:	$g:$ $A_{gv} = (6.0in)^* (0.25in) = 1.50 in2$ $\Phi_{yv} = 1.00 \qquad LRFD$ $V_{allow-yideling} = \phi_{yv} * 0.60^* F_y * A_{gv} = 32.40 kip$ $\Phi_{rv} = 0.75 \qquad LRFD$	Horizontal Component)) (Eqn J4-3) (Eqn J4-3) (Eqn J4-4)
Gross Area: Shear Yielding: Net Area:	$g:$ $A_{gv} = (6.0in)^* (0.25in) = 1.50 in2$ $\Phi_{yv} = 1.00 \qquad LRFD$ $V_{allow-yideling} = \phi_{yv} * 0.60^* F_y * A_{gv} = 32.40 kip$ $\Phi_{rv} = 0.75 \qquad LRFD$ $A_{nv} = (6.0in)^* (0.25in) - (4^* 0.3125in)^* (0.25in) = 1.19 in2$	Horizontal Component)) ( <i>Eqn J4-3</i> ) ( <i>Eqn J4-3</i> ) ( <i>Eqn J4-4</i> ) (Sec B4.3)

#### Bending Moment on Vertical Leg:

Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{6^* 0.25^2}{4} =$	0.0938 in3	
	$\phi_b = 0.9$	LRFD	(Sec F1)
	$M_{allow} = \phi * F_y * Z =$	= 3.038 kip-in	
Moment Arm:	Moment <sub>arm</sub> = 2.875 in		(Eqn F11-1)
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$	= 1.057 kip LRFD	(Vertical Component)

#### Bending Moment on Lower Leg (Side B):

Allowable Load:

Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{6*0.25^2}{4} =$	0.0938 in3	
	$\phi_b = 0.9$	LRFD	(Sec F1)
	$M_{allow} = \phi * F_y * Z =$	= 3.038 kip-in	(Eqn F11-1)
Moment Arm:	Moment <sub>arm</sub> = 1.375 in		

 $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 2.209 \ \text{kip LRFD} \qquad (\text{Horizontal Component})$ 

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## <u>Piece #43146A - L4" x 3" x 1/4" x 0'-6" Steel Angle Bracket (A36 min)</u> w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

#### Screws from Angle Bracket to Steel Sheet:

<u>Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6. LRFD Shear (Bearing) & Tension (Pull-Out) Capacity</u> (ICC-ESR-3332) Based on a Steel Member (min yield strength. Fy = 33 ksi & min tensile strength Fu = 45 ksi) Capacity of (1) Screw from 1/4", 3/8" & 1/2" Brackets to various design thickness steel sheets Shear Bearing Capacity first number is the minimum thickness of the steel in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet). Tensile Pull-out Capacity the number is for the steel sheet not in contact with the screw head (bottom sheet). Screw Capacities (Shear Bearing per ESR-3332, Table 3 & Tensile Pull-out per Table 5 per ESR-3332) LRFD

Design Thickness	V <sub>allow</sub>	Design Thickness	T <sub>allow</sub>	Number of Screws	N = 4
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	2412 lbf	840 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	4004 lbf	1324 lbf
0.060-0.060	833 lbf	0.075	409 lbf	3332 lbf	1636 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	4232 lbf	2192 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	4084 lbf	3588 lbf
		3/16"	1439 lbf		5756 lbf

#### Bolts thru Angle Bracket to Concrete Slab or Concrete-Filled Profile Steel Deck Failure Modes:

Hilti Kwik Bolt-TZ anchors may be installed in cracked or uncracked concrete or concrete-filled steel deck

Bolt type:	A307 Gr. A (Commor	Bolts), bearing type connection		
Nominal Tens	ile Strength:	F <sub>t</sub>	= 45 ksi	
Bracket Thick	ness:	th <sub>bracket</sub>	= 0.25 in	
Nominal Shea	r Strength, Threads Exclude	ed: F <sub>v</sub>	= 27 ksi	e J3.2 AISC 14th)
Bolt Diameter	:	d <sub>bcd</sub>	= 0.50 in	55.2 AISC 14(1)
Bolt Area:		A <sub>bcd</sub>	= 0.25* π *d <sub>bcd</sub>	<sup>2</sup> = 0.20 in2
Resistance fa	ctor for bolt tension or shear	: ¢	b= 0.75	LRFD
Shear Capaci	ty of single bearing type boli	V allow-bolt	$= \phi * F_v * A_{bc}$	<sub>d</sub> = 3.98 kip
Tension Capa	city of single bearing type b	olt: T <sub>allow-bolt</sub>	$= \phi * F_t * A_{bcc}$	<sub>d</sub> = 6.63 kip
Bolt bearing strength at	bracket connection: (Section	n J3.10) F <sub>u</sub>	= 58 ksi	
Bolt edge dist	ance:	edge-dist	= 1.00 in	
Bolt hole diam	neter:	bh	= 0.625 in	
	e between edge of hole djacent hole or edge of plate	$L_c = edge-dist - 0.5*bh$	= 0.69 in2	
Single bolt be	aring capacity:	Bolt $_{brg}$ = min[(1.5*L $_{c}$ *th $_{plate}$ *F $_{u}$ ),(3	8.0*d <sub>bcd</sub> *th <sub>plate</sub> *F	= <sub>u</sub> )] =  14.95 kip

2\*Bolt<sub>allow-bolt</sub> = 2\*\$\$\$\$ \*Bolt\_bearing 22.43 kip LRFD

#### <u>Piece #43146A - L4" x 3" x 1/4" x 0'-6" Steel Angle Bracket (A36 min)</u> w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

See Hilti Excel output for Allowable Combined Tension and Shear Loads for Concrete Over Metal Deck (LRFD) See Hilti Profis output for Allowable Combined Tension and Shear Loads for Concrete Slab (LRFD)

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

 $\phi N_n = 570 \, lbf$ 

 $\phi V_n = 1100 \, lbf$ 

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

 $\phi N_n = 1275 \, lbf$ 

 $\phi V_n = 2552 \, lbf$ 

Given that the Load and Resistance Factor Design calculated above for the angle brackets and bolts far outweigh the capacity of the concrete anchors, the allowable loading to the concrete anchors govern.
Note also that the capacity of the concrete anchors shown here is based on utilizing Section D.3.3.4.3 (d) of ACI 318-11, which requires the inclusion of the Omega factor when determining the loads applied to the anchorage.
Do to the complication of the requirement (per ACI 318-11) to determine the concrete anchorage capacity utilizing LRFD as well as Section 4.2 in ESR-1917 the allowable load for this Piece is given in LRFD only.

#### <u>Overall Capacity of Seismic Load - Piece #43146A - 4" x 3" x 1/4" x 0'-6" Steel Angle Bracket (A36 min)</u> w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

Load allowable-total-on-concrete-ove-metal-deck = 570 lbf	Tension	
Load allowable-total-on-concrete-ove-metal-deck = 1100 lbf	Shear	Vertical & Horizontal Allowable Load -
Load allowable-total-on-4" min-concrete-slab = 1275 lbf	Tension	(LRFD)
Load allowable-total-on-4" min-concrete-slab = 2552 lbf	Shear	

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Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/4" Floor & Wall Brackets (included within this calculation package for reference).

#### Prying of Piece #63143A - L6" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min) w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8"x3-3/4"

AISC 14 Edition Part 9, p.9-10 of Specification

Bracket Thickness:	th <sub>bracket</sub> =	0.25 in		
Nominal Tensile Strength:	$F_t =$	60 ksi	(Carbon Steel)	
Ultimate Strength:	F <sub>u</sub> =	58 ksi		
Anchor Spacing	s =	0.00		
Anchor Diameter:	<i>d</i> <sub>b</sub> =	0.375 in		
Steel strength in tension:	N <sub>sa</sub> =	6.500 kip		
Max distance from bolt to applied force	e: b =	1.375 in		
Tributary length to each bolt:	<i>p = max 2b,</i> but <u>&lt;</u> s	2.75 in		
Adjusted prying distance:	$b' = \left[b - \frac{d_b}{2}\right] =$	1.19 in	(Eq. 9-21)	
Resistance factor for prying:	$\Phi_{\rm pr}$ =	1.67 ASD 0.90 LRFD 0.75 LRFD		
Maximum bolt force for prying:	$T = \frac{\Phi_{pr} * F_u * t}{2}$	$h_{bracket}^{2} * p =$	3.26 kip	(Eq. 9-22a)

 $T = \frac{\Phi_{pr} * F_{u} * t h_{bracket}^{2} * p}{2 * b} = -3.26 \text{ kip}$ 

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 4.88$  kip (per ESR-1917)

$$t_c = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_u}} = 0.440 \text{ in}$$
 (Eq. 9-30a)

 $T_{avail} = BQ = 2.86 kip$ 

(Eq. 9-31)

$$Q = 0.587 \qquad a' < Q = 1$$
  

$$0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.587 \qquad a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.587$$
  
(Eq. 9-33) (Eq. 34)

Width of hole along length of plate: d' = 0.500 in

Distance from bolt centerline to edge of plate: a = 1.50 in

Additional variables for prying calculation:

$$\begin{split} \delta &= 1 - \frac{d'}{\rho} = 0.82 & a' = a + \frac{d_{\pm}}{2} = 1.69 \text{ in } \leq \left(1.25 * b * \frac{d_{b}}{2}\right) = 1.91 \text{ in } \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \rho &= \frac{b'}{\alpha'} = 0.70 & \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = 1.00 \text{ in } \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ \alpha' &= if' \left[ \beta => 1, 1, \min\left[1, \frac{1}{\delta} * \left(\frac{\beta}{1 - \beta}\right)\right] \right] = 1.00 \end{split}$$

Required bracket thickness to ensure an acceptable combination of fitting strength, stiffness, and bolt strength:

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{\rho r}*p*F_{u}*(1+\delta*\alpha')}} = 0.23 \text{ in LRFD} \quad (Eq. 9-23a)$$

$$Thickness of Bracket is 0.25 in - therefore, Prying OK$$

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#### Piece #63143A - L6" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min) w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8"x3-3/4"

# Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

	Bracket Thickness:	<i>th</i> <sub>bracket</sub> = 0.2500 in	
	Yield Strength:	F <sub>y</sub> = 36 ksi	(Table 2-4)
	Ultimate Strength:	F <sub>u</sub> = 58 ksi	
	Gross Area:	A <sub>g</sub> = (3.0in)*(0.25in) = 0.75 in2	
		$\phi_{t-y} = 0.9$ LRFD	(Eqn D2.1)
	Tensile Yielding:	$T_{allow-yielding} = \phi_{t-y} * F_y * A_g = 24.30 \text{ kip}$	(Eqn D2-1)
	Shear Lag Factor:	<i>U</i> = 1.0	(Table D3.1)
	Net Area:	$A_n = (3.0in)^*(0.25in) - (2^*0.3125in)^*(0.25in) = 0.59 in2$	(Sec B4.3)
	Effective Net Area:	$A_e = A_n * U = 0.59 in2$	(Eqn D3-1)
		$\phi_{tr} = 0.75$ LRFD	(Eqn D2-2)
	Tensile Rupture:	$T_{allow-rupture} = \phi_{tr} * F_u * A_e = 25.83 \text{ kip}$	(Eqn D2-2)
Shoar	on Bracket Vertical Leg:	$T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 24.30 kip LRFD$	(Vertical & Horizontal Component))
<u>Silear</u>	Gross Area:	A <sub>av</sub> =(3.0in)*(0.25in) = 0.75 in2	
	GIUSS AIEa.	$\Phi_{yy} = (3.011) (0.2311) = 0.75112$ $\Phi_{yy} = 1.00$ LRFD	(Ean 14.2)
			(Eqn J4-3)
	Shear Yielding:	$V_{allow-yielding} = \phi_{yv} * 0.60 * F_y * A_{gv} = 16.20 \text{ kip}$	(Eqn J4-3)
		$     \Phi_{\rm rv} = 0.75 $ LRFD	(Eqn J4-4)
	Net Area:	$A_{nv} = (3.0in)^*(0.25in) \cdot (2^*0.3125in)^*(0.25in) = 0.59 in2$	(Sec B4.3)
	Shear Rupture:	$V_{allow-rupture} = \phi_{rv} * 0.60 * F_{u} * A_{nv} = 20.66  kip$	(Eqn J4-4)
		$V_{allow} = min(V_{allow-yielding}, V_{allow-rupture}) = 16.20 kip LRFD$	(Horizontal Component)
	**Note: Bending of Brad	cket is Considered within the Prying Calculation	
<u>Bendin</u>	g Moment on Vertical L	eg (Side A):	
	Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{3*0.25^2}{4} = 0.0469 \text{ in 3}$	
		$\phi_b = 0.9$ LRFD	(Sec F1)

$M_{\text{allow}} = \phi^* F_y^* Z = 1.519 \text{ kip-in}$				
Moment Arm:	Moment <sub>arm</sub> = 4.375 in	(Eqn F11-1)		
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 0.347 \text{ kip LRFD}$	(Vertical Component)		

#### Bending Moment on Lower Leg (Side B):

Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{3*0.25^2}{4} = 0.$	.0469 in3	
	$\phi_b = 0.9 \qquad \qquad Ll$	RFD	(Sec F1)
	$M_{allow} = \phi * F_y * Z = 1.$	.519 kip-in	(Eqn F11-1)
Moment Arm:	Moment <sub>arm</sub> = 1.375 in		
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = -1.$	.105 kip LRFD (Horizor	ntal Component)

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## <u>Piece #63143A - L6" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min)</u> w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8"x3-3/4"

#### Screws from Angle Bracket to Steel Sheet:

<u>Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6. LRFD Shear (Bearing) & Tension (Pull-Out) Capacity</u> (ICC-ESR-3332) Based on a Steel Member (min yield strength, Fy = 33 ksi & min tensile strength Fu = 45 ksi) Capacity of (1) Screw from 1/4", 3/8" & 1/2" Brackets to various design thickness steel sheets Shear Bearing Capacity first number is the minimum thickness of the steel in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet). Tensile Pull-out Capacity the number is for the steel sheet not in contact with the screw head (bottom sheet). Screw Capacities (Shear Bearing per ESR-3332, Table 3 & Tensile Pull-out per Table 5 per ESR-3332) LRFD

Design Thickness	V <sub>allow</sub>	Design Thickness	T <sub>allow</sub>	Number of Screws	N = 4
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	2412 lbf	840 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	4004 lbf	1324 lbf
0.060-0.060	833 lbf	0.075	409 lbf	3332 lbf	1636 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	4232 lbf	2192 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	4084 lbf	3588 lbf
		3/16"	1439 lbf		5756 lbf

#### Bolts thru Angle Bracket to Concrete Slab or Concrete-Filled Profile Steel Deck Failure Modes:

Hilti Kwik Bolt-TZ anchors may be installed in cracked or uncracked concrete or concrete-filled steel deck

Bolt type: A307 Gr. A (Common Bolts),	bearing type connection		
Nominal Tensile Strength:	$F_t =$	45 ksi	
Bracket Thickness:	th <sub>bracket</sub> =	0.25 in	
Nominal Shear Strength, Threads Excluded:	F <sub>v</sub> =	27 ksi (Tabla 12 2	AISC 14th)
Bolt Diameter:	d <sub>bcd</sub> =	(Table J3.2 ) 0.50 in	AISC 14th)
Bolt Area:	A <sub>bcd</sub> =	$0.25^{*}\pi^{*}d_{bcd}^{2} =$	0.20 in2
Resistance factor for bolt tension or shear:	$\phi =$	0.75	LRFD
Shear Capacity of single bearing type bolt:	V <sub>allow-bolt</sub> =	$\phi * F_v * A_{bcd} =$	3.98 kip
Tension Capacity of single bearing type bolt:	$T_{allow-bolt} =$	$\phi * F_t * A_{bcd} =$	6.63 kip
Bolt bearing strength at bracket connection: (Section J3.10)	$F_u =$	58 ksi	
Bolt edge distance:	edge-dist =	1.50 in	
Bolt hole diameter:	bh =	0.625 in	
Clear distance between edge of hole and edge of adjacent hole or edge of plate:	L <sub>c</sub> = edge-dist -0.5*bh =	1.19 in2	
Single end bolt bearing capacity: Bolt <sub>br'g</sub> =	min[(1.5*L <sub>c</sub> *th <sub>plate</sub> *F <sub>u</sub> ),(3.0	$d_{bcd}$ $th_{plate}$ $F_{u}] =$	21.75 kip

 $Bolt_{allow-bolt} = \phi *Bolt_{bearing}$  16.31 kip LRFD

#### <u>Piece #63143A - L6" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min)</u> w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8"x3-3/4"

See Hilti Excel output for Allowable Combined Tension and Shear Loads for Concrete Over Metal Deck (LRFD) See Hilti Profis output for Allowable Combined Tension and Shear Loads for Concrete Slab (LRFD)

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

 $\phi N_n = 390 \, lbf$ 

 $\phi V_n = 440 \, lbf$ 

3/8" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

 $\phi N_n = 950 \, lbf$ 

 $\phi V_n = 1322 \, lbf$ 

Given that the Load and Resistance Factor Design calculated above for the angle brackets and bolts far outweigh the capacity of the concrete anchors, the allowable loading to the concrete anchors govern.
Note also that the capacity of the concrete anchors shown here is based on utilizing Section D.3.3.4.3 (d) of ACI 318-11, which requires the inclusion of the Omega factor when determining the loads applied to the anchorage.
Do to the complication of the requirement (per ACI 318-11) to determine the concrete anchorage capacity utilizing LRFD as well as Section 4.2 in ESR-1917 the allowable load for this Piece is given in LRFD only.

#### <u>Overall Capacity of Seismic Load - Piece #63143A - L6" x 3" x 1/4" x 0'-3" Steel Angle Bracket (A36 min)</u> w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8"x3-3/4"

Load allowable-total-on-concrete-ove-metal-deck = 390 lbf	Tension	
Load allowable-total-on-concrete-ove-metal-deck = 440 lbf	Shear	Vertical & Horizontal Allowable Load -
Load allowable-total-on-4" min-concrete-slab = 950 lbf	Tension	(LRFD)
Load allowable-total-on-4" min-concrete-slab = 1322 lbf	Shear	

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#### Piece #36143A - L3" x 6" x 1/4" x 0-3" Steel Angle Bracket (A36 min) w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816

#### Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness:	$th_{bracket} = 0.2500$ in	
Yield Strength:	$F_{\gamma} = 36  ksi$	(Table 2-4)
Ultimate Strength:	$F_u = 58  ksi$	
Gross Area:	A <sub>g</sub> = (3.0in)*(0.25in) = 0.75 in2	
	$\phi_{ty} = 0.9$ LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow-yielding} = \phi_{ty} *F_y *A_g = 24.30 \text{ kip}$	(Eqn D2-1)
Shear Lag Factor:	<i>U</i> = 1.0	(Table D3.1)
Net Area:	$A_n = (3.0in)^*(0.25in) - (2^*0.3125in)^*(0.25in) = 0.59 in2$	(Sec B4.3)
Effective Net Area:	$A_e = A_n^* U = 0.59 in2$	(Eqn D3-1)
	$\phi_{tr}$ = 0.75 LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow-rupture} = \phi_{tr} * F_u * A_e = 25.83 \text{ kip}$	(Eqn D2-2)
	$T_{allow-brackef} = min(T_{allow-yelding}, T_{allow-rupture}) = 24.30 \text{ kip LRFD}$	(Vertical & Horizontal Component))
Shear on Bracket Vertical Le	<u>a:</u>	
Gross Area:	A <sub>gv</sub> =(3.0in)*(0.25in) = 0.75 in2	
	$     \Phi_{yv} = 1.00 $ LRFD	(Eqn J4-3)
Shear Yielding:	$V_{allow-yielding} = \phi_{yy} *0.60*F_y *A_{gy} = 16.20 \text{ kip}$	(Eqn J4-3)

Net Area:	$A_{nv} = (3.0in)^*(0.25in) - (2^*0.3125in)^*(0.25in) = 0.59 in2$	(Sec B4.3)
	$     \Phi_{\rm rv} = 0.75 $ LRFD	(Eqn J4-4)
Shear Rupture:	$V_{allow-rupture} = \phi_{rv} * 0.60 * F_u * A_{nv} = 20.66  kip$	(Eqn J4-4)

$V_{allow} = min(V_{allow-yielding},$	V <sub>allow-rupture</sub> ) =	16.20 kip LRFD	(Horizontal Component)
			component)

Bending Moment on Lower Leg (Side B):

Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{3*0}{4}$	$\frac{0.25^2}{4} = 0.0469 \text{ in 3}$	
	$\phi_b = 0.9$	LRFD	(Sec F1)
	$M_{allow} = \phi^*$	F <sub>y</sub> *Z = 1.519 kip-in	(Eqn F11-1)

Moment Arm: Moment <sub>arm</sub> = 4.875 in	
--	--

 $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 0.312 \text{ kip LRFD} \quad (\text{Horizontal Component})$ 

#### Bending Moment on Vertical Leg (Side A):

Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{3*0.25^2}{4} = 0.0469 \text{ in 3}$	
	$\phi_b = 0.9$ LRFD	(Sec F1)
	$M_{allow} = \phi *F_y *Z = 1.519$ kip-in	
Moment Arm:	Moment <sub>arm</sub> = 1.375 in	(Eqn F11-1)
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 1.105 \text{ kip LRFL}$	D (Vertical Component)

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# <u>Piece #36143A - L3" x 6" x 1/4" x 0'-3" Steel Angle Bracket (A36 min) w/ (2) 0.3125"</u> <u>dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816</u>

# Screws from Angle Bracket to Steel Sheet:

<u>Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6, LRFD Shear (Bearing) & Tension (Pull-Out) Capacity</u> (ICC-ESR-3332) Based on a Steel Member (min yield strength, Fy = 33 ksi & min tensile strength Fu = 45 ksi) Capacity of (1) Screw from 1/4", 3/8" & 1/2" Brackets to various design thickness steel sheets Shear Bearing Capacity first number is the minimum thickness of the steel in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet). Tensile Pull-out Capacity the number is for the steel sheet not in contact with the screw head (bottom sheet). Screw Capacities (Shear Bearing per ESR-3332, Table 3 & Tensile Pull-out per Table 5 per ESR-3332) LRFD

# Vertical Leg:

Design Thickness	V <sub>allow</sub>	Design Thickness	T <sub>allow</sub>	Number of Screws A	= 2
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	1206 lbf	420 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	2002 lbf	662 lbf
0.060-0.060	833 lbf	0.075	409 lbf	1666 lbf	818 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	2116 lbf	1096 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	2042 lbf	1794 lbf
		3/16"	1439 lbf		2878 lbf

# Horizontal Leg:

Design Thickness	V <sub>allow</sub>	Design Thickness	T <sub>allow</sub>	Number of Screws /	V = 2
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	1206 lbf	420 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	2002 lbf	662 lbf
0.060-0.060	833 lbf	0.075	409 lbf	1666 lbf	818 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	2116 lbf	1096 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	2042 lbf	1794 lbf
		3/16"	1439 lbf		2878 lbf

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#### Piece #36143B - L3" x 6" x 1/4" x 0'-3" Steel Angle Bracket (A36 min) w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816

#### Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness:	th <sub>bracket</sub> = 0.2500 in	
Yield Strength:	$F_{y} = 36  ksi$	(Table 2-4)
Ultimate Strength:	$F_u = 58  \text{ksi}$	(1000 2 4)
Gross Area:	$A_a = (3.0in)^*(0.25in) = 0.75 in2$	
0,000,100	$\phi_{ty} = 0.9 \qquad LRFD$	(Eap D2 1)
		(Eqn D2.1)
Tensile Yielding:	$T_{allow-yielding} = \phi_{t-y} * F_y * A_g = 24.30 \text{ kip}$	(Eqn D2-1)
Shear Lag Factor:	U = 1.0	(Table D3.1)
Net Area:	$A_n = (3.0in)^*(0.25in) - (2^*0.3125in)^*(0.25in) = 0.59 in2$	(Sec B4.3)
Effective Net Area:	$A_e = A_n * U = 0.59 in2$	(Eqn D3-1)
	$\phi_{tr} = 0.75$ LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow-rupture} = \phi_{tr} * F_u * A_e = 25.83 \text{ kip}$	(Eqn D2-2)
Shear on Bracket Vertical Le	<i>T</i> <sub>allow-bracket</sub> = min(T <sub>allow-yielding</sub> , T <sub>allow-rupture</sub> ) = 24.30 kip Li	RFD (Vertical & Horizontal Component))
Gross Area:	A <sub>av</sub> =(3.0in)*(0.25in) = 0.75 in2	
Closs Alea.	$\Phi_{yy} = 1.00 \qquad LRFD$	(Eqn J4-3)
Shear Yielding:	$V_{allow-yielding} = \phi_{yv} * 0.60 * F_y * A_{gv} = 16.20 \text{ kip}$	(Eqn J4-3)
Net Area:	$A_{nv} = (3.0in)^{*}(0.25in) \cdot (2^{*}0.3125in)^{*}(0.25in) = 0.59 in2$	(Sec B4.3)
	$     \Phi_{rv} = 0.75 $ LRFD	(Eqn J4-4)
Shear Rupture:	$V_{allow-rupture} = \phi_{rv} * 0.60 * F_u * A_{rv} = 20.66 kip$	(Eqn J4-4)
	V <sub>allow</sub> = min(V <sub>allow-yielding</sub> , V <sub>allow-rupture</sub> ) = 16.20 kip Ll	RFD (Horizontal Component)
Bending Moment on Lower I	Leg (Side B):	component)
Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{3*0.25^2}{4} = 0.0469 \text{ in}3$	
	$\phi_b = 0.9$ LRFD	(Sec F1)
	$M_{allow} = \phi * F_y * Z = 1.519 $ kip-in	
Moment Arm:	Moment <sub>arm</sub> = $4.375$ in	(Eqn F11-1)
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 0.347 \text{ kip LRFD}$ (Horizontal	Component)
Bending Moment on Vertical		
Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{3*0.25^2}{4} = 0.0469 \text{ in } 3$	
	$\phi_b = 0.9$ LRFD	(Sec F1)
	$M_{\text{allow}} = \phi * F_y * Z = 1.519 \text{ kip-in}$	
Moment Arm:	Moment <sub>arm</sub> = 1.375 in	(Egn F11-1)
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 1.105  kip  LRFD \qquad (Vertical  Comparison Compared on Comparison Compared on Comp$	

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# <u>Piece #36143B- L3" x 6" x 1/4" x 0'-3" Steel Angle Bracket (A36 min) w/ (2) 0.3125"</u> <u>dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816</u>

#### Screws from Angle Bracket to Steel Sheet:

<u>Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6, LRFD Shear (Bearing) & Tension (Pull-Out) Capacity</u> (ICC-ESR-3332) Based on a Steel Member (min yield strength, Fy = 33 ksi & min tensile strength Fu = 45 ksi) Capacity of (1) Screw from 1/4", 3/8" & 1/2" Brackets to various design thickness steel sheets Shear Bearing Capacity first number is the minimum thickness of the steel in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet). Tensile Pull-out Capacity the number is for the steel sheet not in contact with the screw head (bottom sheet). Screw Capacities (Shear Bearing per ESR-3332, Table 3 & Tensile Pull-out per Table 5 per ESR-3332) LRFD

# Vertical Leg:

Design Thickness	V <sub>allow</sub>	Design Thickness	T <sub>allow</sub>	Number of Screws A	1= 2
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	1206 lbf	420 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	2002 lbf	662 lbf
0.060-0.060	833 lbf	0.075	409 lbf	1666 lbf	818 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	2116 lbf	1096 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	2042 lbf	1794 lbf
		3/16"	1439 lbf		2878 lbf

# Horizontal Leg:

Design Thickness	V <sub>allow</sub>	Design Thickness	T <sub>allow</sub>	Number of Screws /	V = 4
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	2412 lbf	840 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	4004 lbf	1324 lbf
0.060-0.060	833 lbf	0.075	409 lbf	3332 lbf	1636 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	4232 lbf	2192 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	4084 lbf	3588 lbf
		3/16"	1439 lbf		5756 lbf

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#### <u>Piece #36143C - L3" x 6" x 1/4" x 0'-3" Steel Angle Bracket (A36 min) w/ (4) 0.3125"</u> dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816

# Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness:	$th_{bracket} = 0.2500$ in	
Yield Strength:	$F_y = 36  ksi$	(Table 2-4)
Ultimate Strength:	$F_u = 58  ksi$	
Gross Area:	$A_g = (3.0in)^*(0.25in) = 0.75 in2$	
	$\phi_{t-y} = 0.9$ LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow-yielding} = \phi_{t-y} * F_y * A_g = 24.30 \text{ kip}$	(Eqn D2-1)
Shear Lag Factor:	<i>U</i> = 1.0	(Table D3.1)
Net Area:	$A_n = (3.0in)^*(0.25in) - (2^*0.3125in)^*(0.25in) = 0.59 in2$	(Sec B4.3)
Effective Net Area:	$A_e = A_n^* U = 0.59 in2$	(Eqn D3-1)
	$\phi_{tr} = 0.75$ LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow-rupture} = \phi_{t-r} * F_u * A_e = 25.83 \text{ kip}$	(Eqn D2-2)
	$T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 24.30 kip$	LRFD (Vertical & Horizontal Component))
<u>Shear on Bracket Vertical L</u>		
Gross Area:	A <sub>gv</sub> =(3.0in)*(0.25in) = 0.75 in2	
	$ \Phi_{yy} = 1.00 $ LRFD	(Eqn J4-3)
Shear Yielding:	$V_{allow-yielding} = \phi_{yv} * 0.60 * F_y * A_{gv} = 16.20 \ kip$	(Eqn J4-3)
Net Area:	$A_{nv} = (3.0in)^*(0.25in) \cdot (2^*0.3125in)^*(0.25in) = 0.59 in2$	(Sec B4.3)
	$     \Phi_{rv} = 0.75 $ <i>LRFD</i>	(Eqn J4-4)
Shear Rupture:	$V_{allow-rupture} = \phi_{rv} * 0.60 * F_u * A_{nv} = 20.66 kip$	(Eqn J4-4)
	$V_{allow} = min(V_{allow-yielding}, V_{allow-rupture}) = 16.20 kip$	(Horizontal Component)
Bending Moment on Lower	Leg (Side B):	
Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{3*0.25^2}{4} = 0.0469 \text{ in 3}$	
	$\phi_b = 0.9$ LRFD	(Sec F1)
	$M_{allow} = \phi *F_y *Z = 1.519 \text{ kip-in}$	
Moment Arm:	Moment <sub>arm</sub> = 4.375 in	(Eqn F11-1)
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{argum}} = 0.347 \text{ kip LRFD}$ (Horizonta	al Component)
Bending Moment on Vertica	547 F/B	
Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{3*0.25^2}{4} = 0.0469 \text{ in } 3$	
	$\phi_b = 0.9$ LRFD	(Sec F1)
	$M_{allow} = \phi^* F_y^* Z = 1.519 kip-in$	
Moment Arm:	Moment <sub>arm</sub> = 1.375 in	(Eqn F11-1)
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 1.105 \text{ kip LRFD}  \text{(Vertical O})$	Component)

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# <u>Piece #36143C- L3" x 6" x 1/4" x 0'-3" Steel Angle Bracket (A36 min) w/ (4) 0.3125"</u> <u>dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816</u>

#### Screws from Angle Bracket to Steel Sheet:

<u>Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6, LRFD Shear (Bearing) & Tension (Pull-Out) Capacity</u> (ICC-ESR-3332) Based on a Steel Member (min yield strength, Fy = 33 ksi & min tensile strength Fu = 45 ksi) Capacity of (1) Screw from 1/4", 3/8" & 1/2" Brackets to various design thickness steel sheets Shear Bearing Capacity first number is the minimum thickness of the steel in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet). Tensile Pull-out Capacity the number is for the steel sheet not in contact with the screw head (bottom sheet). Screw Capacities (Shear Bearing per ESR-3332, Table 3 & Tensile Pull-out per Table 5 per ESR-3332) LRFD

# Vertical Leg:

Design Thickness	V <sub>allow</sub>	Design Thickness	T <sub>allow</sub>	Number of Screws	/= 4
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	2412 lbf	840 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	4004 lbf	1324 lbf
0.060-0.060	833 lbf	0.075	409 lbf	3332 lbf	1636 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	4232 lbf	2192 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	4084 lbf	3588 lbf
		3/16"	1439 lbf		5756 lbf

# Horizontal Leg:

Design Thickness	V <sub>allow</sub>	Design Thickness	T <sub>allow</sub>	Number of Screws /	V = 4
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	2412 lbf	840 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	4004 lbf	1324 lbf
0.060-0.060	833 lbf	0.075	409 lbf	3332 lbf	1636 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	4232 lbf	2192 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	4084 lbf	3588 lbf
		3/16"	1439 lbf		5756 lbf

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#### Piece #36144A - L3" x 6" x 1/4" x 0'-4" Steel Angle Bracket (A36 min) w/ (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816

#### Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness:	$th_{bracket} = 0.2500 \text{ in}$	
Yield Strength:	$F_{\gamma} = 36  ksi$	(Table 2-4)
Ultimate Strength:	$F_u = 58  \text{ksi}$	(14516 2-4)
Gross Area:	$A_{a} = (4.0in)^{*}(0.25in) = 1.00 in2$	
Gross Area.	$\phi_{ty} = 0.9 \qquad LRFD$	(Eap D2 1)
		(Eqn D2.1)
Tensile Yielding:	$T_{allow-yielding} = \phi_{ty} * F_y * A_g = 32.40 \text{ kip}$	(Eqn D2-1)
Shear Lag Factor:	<i>U</i> = 1.0	(Table D3.1)
Net Area:	$A_n = (4.0in)^*(0.25in) - (2^*0.3125in)^*(0.25in) = 0.84 in2$	(Sec B4.3)
Effective Net Area:	$A_e = A_n * U = 0.84 in2$	(Eqn D3-1)
	$\phi_{tr} = 0.75$ LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow-rupture} = \phi_{t\tau} *F_{u} *A_{e} = 36.70 \text{ kip}$	(Eqn D2-2)
	$T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 32.40 kip LRF$	D (Vertical & Horizontal Component))
Shear on Bracket Vertical I		
Gross Area:	$A_{gv} = (4.0in)^*(0.25in) = 1.00 in2$	
	$     \Phi_{yv} = 1.00 $ LRFD	(Eqn J4-3)
Shear Yielding:	$V_{allow-yielding} = \phi_{yv} * 0.60 * F_y * A_{gv} = 21.60 \ kip$	(Eqn J4-3)
Net Area:	$A_{nv} = (4.0in)^*(0.25in) - (2^*0.3125in)^*(0.25in) = 0.84 in2$	(Sec B4.3)
	$     \Phi_{rv} = 0.75 $ LRFD	(Eqn J4-4)
Shear Rupture:	$V_{allow-rupture} = \phi_{rv} * 0.60^{\circ} F_{u} * A_{nv} = 29.36  kip$	
	$V_{allow} = min(V_{allow-yielding}, V_{allow-rupture}) = 21.60 kip LRF$	D (Horizontal Component)
Bending Moment on Lowe	<u>r Leg (Side B):</u>	
Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{4*0.25^2}{4} = 0.0625 \text{ in } 3$	
	$\phi_b = 0.9$ LRFD	(Sec F1)
	$M_{allow} = \phi *F_y *Z = 2.025 kip-in$	(Eqn F11-1)
Moment Arm:	Moment <sub>arm</sub> = 4.875 in	(Eqn F11-1)
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 0.415 \text{ kip LRFD}$ (Horizontal Co	omponent)
Bending Moment on Vertic	cal Leg (Side A):	
Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{6^{*0.25^2}}{4} = 0.0625 \text{ in } 3$	
	$\phi_b = 0.9$ LRFD	(Sec F1)
	$M_{allow} = \phi^* F_y^* Z = 2.025 \text{ kip-in}$	
Moment Arm:	Moment <sub>arm</sub> = 1.375 in	(Eqn F11-1)
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 1.473 \text{ kip LRFD}  (\text{Vertical Com}$	ponent)

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# <u>Piece #36144A- L3" x 6" x 1/4" x 0'-4" Steel Angle Bracket (A36 min) w/ (2) 0.3125"</u> <u>dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (2) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816</u>

#### Screws from Angle Bracket to Steel Sheet:

<u>Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6, LRFD Shear (Bearing) & Tension (Pull-Out) Capacity</u> (ICC-ESR-3332) Based on a Steel Member (min yield strength, Fy = 33 ksi & min tensile strength Fu = 45 ksi) Capacity of (1) Screw from 1/4", 3/8" & 1/2" Brackets to various design thickness steel sheets Shear Bearing Capacity first number is the minimum thickness of the steel in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet). Tensile Pull-out Capacity the number is for the steel sheet not in contact with the screw head (bottom sheet). Screw Capacities (Shear Bearing per ESR-3332, Table 3 & Tensile Pull-out per Table 5 per ESR-3332) LRFD

# Vertical Leg:

Design Thickness	V <sub>allow</sub>	V <sub>allow</sub> Design Thickness		Number of Screws $N = 2$	
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	1206 lbf	420 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	2002 lbf	662 lbf
0.060-0.060	833 lbf	0.075	409 lbf	1666 lbf	818 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	2116 lbf	1096 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	2042 lbf	1794 lbf
		3/16"	1439 lbf		2878 lbf

# Horizontal Leg:

Design Thickness	V <sub>allow</sub>	Design Thickness	T <sub>allow</sub>	Number of Screws A	1 = 2
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	1206 lbf	420 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	2002 lbf	662 lbf
0.060-0.060	833 lbf	0.075	409 lbf	1666 lbf	818 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	2116 lbf	1096 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	2042 lbf	1794 lbf
		3/16"	1439 lbf		2878 lbf

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#### Piece #36146A - L3" x 6" x 1/4" x 0'-6" Steel Angle Bracket (A36 min) w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (8) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816

#### Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness:	$th_{bracket} = 0.2500$ in	
Yield Strength:	$F_y = 36  ksi$	(Table 2-4)
Ultimate Strength:	$F_u = 58 \text{ ksi}$	
Gross Area:	694 1.50 in2	
	$\phi_{t-y} = 0.9$ LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow-yielding} = \phi_{ty} * F_y * A_g = 48.60 \text{ kip}$	(Eqn D2-1)
Shear Lag Factor:	U= 1.0	(Table D3.1)
Net Area:	$A_n = (6.0in)^*(0.25in) - (4^*0.3125in)^*(0.25in) = 1.19in2$	(Sec B4.3)
Effective Net Area:	$A_{e} = A_{n} * U = 1.19 in2$	(Eqn D3-1)
	$\phi_{tr} = 0.75$ LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow-rupture} = \phi_{tr} * F_u * A_e = 51.66 \text{ kip}$	(Eqn D2-2)
	$T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 48.60 kip LF$	RFD (Vertical & Horizontal Component))
Shear on Bracket Vertical L		
Gross Area:	$A_{gv} = (6.0in)^*(0.25in) = 1.50 in2$	
	$     \Phi_{yy} = 1.00 $ LRFD	(Eqn J4-3)
Shear Yielding:	$V_{allow-yielding} = \phi_{yv} * 0.60 * F_y * A_{gv} = 32.40 \text{ kip}$	(Eqn J4-3)
Net Area:	$A_{nv} = (6.0in)^*(0.25in) - (4^*0.3125in)^*(0.25in) = 1.19 in2$	(Sec B4.3)
	$     \Phi_{rv} = 0.75 $ LRFD	(Eqn J4-4)
Shear Rupture:	$V_{allow-rupture} = \phi_{rv} * 0.60 * F_u * A_{rv} = 41.33 kip$	(Eqn J4-4)
	$V_{allow} = min(V_{allow-yielding}, V_{allow-rupture}) = 32.40 kip Lf$	RFD (Horizontal Component)
Bending Moment on Lower		
Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{6*0.25^2}{4} = 0.0938 \text{ in}3$	
	$\phi_b = 0.9$ LRFD	(Sec F1)
	$M_{allow} = \phi *F_y *Z = 3.038$ kip-in	
Moment Arm:	Moment <sub>arm</sub> = 4.375 in	(Eqn F11-1)
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 0.694 \text{ kip LRFD}  (\text{Horizontal})$	Component)
Bending Moment on Vertica	al Leg (Side A):	
Plastic Modulus:	$Z = \frac{bd^2}{4} = \frac{6*0.25^2}{4} = 0.0938 \text{ in}3$	
	$\phi_b = 0.9$ LRFD	(Sec F1)
	$M_{\text{allow}} = \phi *F_y *Z = 3.038 \text{ kip-in}$	
Moment Arm:	Moment <sub>arm</sub> = 1.375 in	(Eqn F11-1)
Allowable Load:	$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 2.209 \text{ kip LRFD} \qquad (\text{Vertical Co}$	mponent)

R. F. NELSON		Date:	5/7/2019
& ASSOCIATES	1/4" BRACKETS	Job No.:	18-001
Structural Engineers		Sheet:	12 (2)

# <u>Piece #3614A- L3" x 6" x 1/4" x 0'-6" Steel Angle Bracket (A36 min) w/ (4) 0.3125"</u> <u>dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (8) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816</u>

#### Screws from Angle Bracket to Steel Sheet:

<u>Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6, LRFD Shear (Bearing) & Tension (Pull-Out) Capacity</u> (ICC-ESR-3332) Based on a Steel Member (min yield strength, Fy = 33 ksi & min tensile strength Fu = 45 ksi) Capacity of (1) Screw from 1/4", 3/8" & 1/2" Brackets to various design thickness steel sheets Shear Bearing Capacity first number is the minimum thickness of the steel in contact with the screw head (top sheet). The second number is the thickness of the steel sheet not in contact with the screw head (bottom sheet). Tensile Pull-out Capacity the number is for the steel sheet not in contact with the screw head (bottom sheet). Screw Capacities (Shear Bearing per ESR-3332, Table 3 & Tensile Pull-out per Table 5 per ESR-3332) LRFD

# Vertical Leg:

Design Thickness	V <sub>allow</sub> Design Thickness		T <sub>allow</sub>	Number of Screws $N = 4$	
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	2412 lbf	840 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	4004 lbf	1324 lbf
0.060-0.060	833 lbf	0.075	409 lbf	3332 lbf	1636 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	4232 lbf	2192 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	4084 lbf	3588 lbf
		3/16"	1439 lbf		5756 lbf

# Horizontal Leg:

Design Thickness	V <sub>allow</sub>	Design Thickness	T <sub>allow</sub>	Number of Screws /	V = 8
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	4824 lbf	1680 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	8008 lbf	2648 lbf
0.060-0.060	833 lbf	0.075	409 lbf	6664 lbf	3272 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	8464 lbf	4384 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	8168 lbf	7176 lbf
		3/16"	1439 lbf		11512 lbf