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3/8" BRACKETS

Date: 5/7/2019
Job No.: 18-001
Sheet: 6 (1)

Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 3/8" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #33384A - L3" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 1/2"x3-3/4"

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

Bracket Thickness: $th_{bracket}$ = 0.375 in

Nominal Tensile Strength: F_t = 60 ksi *(Carbon Steel)*

Ultimate Strength: F_u = 58 ksi

Anchor Spacing s = 0.00

Anchor Diameter: [redacted] d_b = 0.500 in

Steel strength in tension: N_{sa} = *10.705 kip*

Max distance from bolt to applied force: b = 1.313 in

Tributary length to each bolt: *p = max 2b, but ≤ s* 2.63 in

Adjusted prying distance: $b' = \left[b - \frac{d_b}{2}\right] =$ 1.06 in (Eq. 9-21)

Resistance factor for prying:
Ω_{pr} = *1.67 ASD*
Φ_{pr} = *0.90 LRFD*
Φ_t = *0.75 LRFD*

Maximum bolt force for prying: $T = \frac{\Phi_{pr} * F_u * th_{bracket}{}^2 * p}{2 * b} =$ 7.34 kip (Eq. 9-22a)

Available tensile strength of single bolt: $B = \Phi_t * N_{sa} =$ *8.03 kip* *(per ESR-1917)*

$$t_c = \sqrt{\frac{4 * B * b'}{\Phi_{LRFD} * p * F_u}} = \text{ 0.547 in} \qquad \text{(Eq. 9-30a)}$$

T_{avail} = BQ = 5.72 kip (Eq. 9-31)

Q = 0.712 *a'<, Q = 1*

0 ≤ a' ≤ 1, Q = $\left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') =$ *0.829* (Eq. 9-33)

a'>1, Q = $\left(\frac{t}{t_c}\right)^2 (1 + \delta) =$ *0.829* (Eq. 34)

Width of hole along length of plate: d' = *0.625 in*

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

Additional variables for prying calculation:

$\delta = 1 - \frac{d'}{p} =$ *0.76* *Eq. (9-24)*

$a' = a + \frac{d_b}{2} =$ *1.75 in* $\leq \left(1.25 * b * \frac{d_b}{2}\right) =$ *1.89 in* *Eq. (9-27)*

$\rho = \frac{b'}{a'} =$ *0.61* *Eq. (9-26)*

$\beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) =$ *0.66 in* *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*

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

$$t_{\min} = \sqrt{\frac{4 * T * b'}{\Phi_{pr} * p * F_u * (1 + \delta * \alpha')}} =$$ *0.32 in LRFD* *(Eq. 9-23a)*

Thickness of Bracket is 0.375 in - therefore, Prying OK

Piece #33384A - L3" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 1/2"x3-3/4"

Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness:	$th_{bracket}$ = 0.3750 in		
Yield Strength:	F_y = 36 ksi		*(Table 2-4)*
Ultimate Strength:	F_u = 58 ksi		
Gross Area:	A_g = **(3.0in)*(0.375in)** = 1.13 in2		
	$\phi_{t\text{-}y}$ = 0.9	LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow\text{-}yielding} = \phi_{t\text{-}y} * F_y * A_g$ = 36.45 kip		(Eqn D2-1)
Shear Lag Factor:	U = 1.0		(Table D3.1)
Net Area:	A_n = (3.0in)*(0.375in)-(2*0.3125in)*(0.375in) = 0.89 in2		(Sec B4.3)
Effective Net Area:	A_e = A_n*U = 0.89 in2		(Eqn D3-1)
	ϕ_{tr} = 0.75	LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow\text{-}rupture} = \phi_{t\text{-}r} * F_u * A_e$ = 38.74 kip		(Eqn D2-2)
	$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) = 36.45 kip LRFD		(Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area:	A_{gv} =(3.0in)*(0.375in) = 1.13 in2		
	Φ_{yv} = 1.00	LRFD	*(Eqn J4-3)*
Shear Yielding:	$V_{allow\text{-}yielding} = \phi_{yv} * 0.60 * F_y * A_{gv}$ = 24.30 kip		*(Eqn J4-3)*
	Φ_{rv} = 0.75	LRFD	*(Eqn J4-4)*
Net Area:	A_{nv} = (3.0in)*(0.375in)-(2*0.3125in)*(0.375in) = 0.89 in2		*(Sec B4.3)*
Shear Rupture:	$V_{allow\text{-}rupture} = \phi_{rv} * 0.60 * F_u * A_{nv}$ = 30.99 kip		*(Eqn J4-4)*
	V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) = 24.30 kip LRFD		*(Horizontal Component)*

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

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} =$ 0.2813 in3

ϕ_b = 0.9 LRFD *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = 9.113 kip-in

Moment Arm: $Moment_{arm}$ = 1.313 in *(Eqn F11-1)*

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 6.943 kip LRFD *(Vertical Component)*

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} =$ 0.2813 in3

ϕ_b = 0.9 LRFD *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = 9.113 kip-in *(Eqn F11-1)*

Moment Arm: $Moment_{arm}$ = 1.313 in

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 6.943 kip LRFD *(Horizontal Component)*

Piece #33384A - L3" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 1/2"x3-3/4"

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 (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 3 $V_{allow}*N$	$T_{allow}*N$
0.048-0.048	603 lbf	0.048	210 lbf	**1809 lbf**	**630 lbf**
0.048-0.075	1001 lbf	0.06	331 lbf	**3003 lbf**	**993 lbf**
0.060-0.060	833 lbf	0.075	409 lbf	**2499 lbf**	**1227 lbf**
0.075-0.078	1058 lbf	0.105	548 lbf	**3174 lbf**	**1644 lbf**
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	**3063 lbf**	**2691 lbf**
		3/16"	1439 lbf		**4317 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: $th_{bracket}$ = 0.3750 in

Nominal Shear Strength, Threads Excluded: F_v = 27 ksi *(Table J3.2 AISC 14th)*

Bolt Diameter: d_{bcd} = 0.50 in

Bolt Area: $A_{bcd} = 0.25*\pi*d_{bcd}^2$ = 0.20 in2

Resistance factor for bolt tension or shear: ϕ= 0.75 LRFD

Shear Capacity of single bearing type bolt: $V_{allow\text{-}bolt} = \phi*F_v*A_{bcd}$ = 3.98 kip

Tension Capacity of single bearing type bolt: $T_{allow\text{-}bolt} = \phi*F_t*A_{bcd}$ = 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 = 0.563 in

Clear distance between edge of hole and edge of adjacent hole or edge of plate: L_c = edge-dist -0.5*bh = 1.22 in2

Single end bolt bearing capacity: $Bolt_{br'g} = \min[(1.5*L_c*th_{plate}*F_u),(3.0*d_{bcd}*th_{plate}*F_u)]$ = 32.63 kip

$Bolt_{allow\text{-}bolt} = \phi*Bolt_{bearing}$ 24.47 kip LRFD

Piece #33384A - L3" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 1/2"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 (LRFD)

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

$\phi N_n =$ 370 lbf

$\phi V_n =$ 500 lbf

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

$\phi N_n =$ 1000 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, which allows the conversion of the allowable loads to ASD, requiring input which is specific to the racking system, the allowable load fore this Piece is given in LRFD only.

Overall Capacity of Seismic Load - Piece #33384A - L3" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 1/2"x3-3/4"

$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	**370 lbf**	**Tension**	**Vertical & Horizontal Allowable Load - (LRFD)**
$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	**500 lbf**	**Shear**	
$Load_{allowable\text{-}total\text{-}on\text{-}4"\ min\text{-}concrete\text{-}slab}$ =	**1000 lbf**	**Tension**	
$Load_{allowable\text{-}total\text{-}on\text{-}4"\ min\text{-}concrete\text{-}slab}$ =	**1322 lbf**	**Shear**	

Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 3/8" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #33386A - L3" x 3" x3/8" x 0'-6" Steel Angle Bracket (A36 min) w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" dia Hole for Hilti KB-TZ 1/2" x 3-3/4"

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

Bracket Thickness: $th_{bracket}$ = 0.375 in

Nominal Tensile Strength: F_t = 60 ksi *(Carbon Steel)*

Ultimate Strength: F_u = 58 ksi

Anchor Spacing s = 4.00

Anchor Diameter: [redacted] d_b = 0.500 in

Steel strength in tension: N_{sa} = *10.705 kip*

Max distance from bolt to applied force: b = 1.313 in

Tributary length to each bolt: *p = max 2b, but ≤ s* 2.63 in

Adjusted prying distance: $b' = \left[b - \frac{d_b}{2}\right] =$ 1.06 in (Eq. 9-21)

Resistance factor for prying:
Ω_{pr} = *1.67 ASD*
Φ_{pr} = *0.90 LRFD*
Φ_t = *0.75 LRFD*

Maximum bolt force for prying: $T = \frac{\Phi_{pr} * F_u * th_{bracket}^{\ 2} * p}{2 * b} =$ 7.34 kip (Eq. 9-22a)

Available tensile strength of single bolt: $B = \Phi_t * N_{sa} =$ *8.03 kip* *(per ESR-1917)*

$$t_c = \sqrt{\frac{4 * B * b'}{\Phi_{LRFD} * p * F_u}} = 0.547 \text{ in}$$ (Eq. 9-30a)

T_{avail} = BQ = *6.66 kip* (Eq. 9-31)

Q = *0.829*

a'<, Q = *1*

0 ≤ a' ≤ 1, Q = $\left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') =$ *0.829*
(Eq. 9-33)

a'>1, Q = $\left(\frac{t}{t_c}\right)^2 (1 + \delta) =$ *0.829*
(Eq. 34)

Width of hole along length of plate: d' = *0.625 in*

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

Additional variables for prying calculation:

$\delta = 1 - \frac{d'}{p} =$ 0.76
Eq. (9-24)

$a' = a + \frac{d_b}{2} =$ *1.75 in* $\leq \left(1.25 * b * \frac{d_b}{2}\right) =$ *1.89 in*
Eq. (9-27)

$\rho = \frac{b'}{a'} =$ 0.61
Eq. (9-26)

$\beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) =$ *0.34 in*
Eq. (9-25)

$\alpha' = if\left[\beta => 1, 1, \min\left[1, \frac{1}{\delta} * \left(\frac{\beta}{1-\beta}\right)\right]\right] =$ *0.67*

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

$$t_{min} = \sqrt{\frac{4 * T * b'}{\Phi_{pr} * p * F_u * (1 + \delta * \alpha')}} =$$ *0.37 in LRFD* *(Eq. 9-23a)*

Thickness of Bracket is 0.375 in - therefore, Prying OK

Prying of Piece #33386A - L3" x 3" x 3/8" x 0'-6" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" dia Hole for Hilti KB-TZ 1/2" x 3-3/4"

Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness:	$th_{bracket}$ = 0.375 in		
Yield Strength:	F_y = *36 ksi*		*(Table 2-4)*
Ultimate Strength:	F_u = *58 ksi*		
Gross Area:	A_g = ***(6.0in)*(0.375in)*** = *2.25 in2*		
	$\phi_{t\text{-}y}$ = *0.9*	*LRFD*	(Eqn D2.1)
Tensile Yielding:	$T_{allow\text{-}yielding} = \phi_{t\text{-}y} * F_y * A_g$ = 72.90 kip		(Eqn D2-1)
Shear Lag Factor:	$U =$ *1.0*		(Table D3.1)
Net Area:	A_n = *(6.0in)*(0.375in)-(4*0.3125in)*(0.375in) = 1.78 in2*		(Sec B4.3)
Effective Net Area:	A_e = $A_n * U$ =	*1.78 in2*	(Eqn D3-1)
	ϕ_{tr} = *0.75*	*LRFD*	(Eqn D2-2)
Tensile Rupture:	$T_{allow\text{-}rupture} = \phi_{t\text{-}r} * F_u * A_e$ = 77.48 kip		(Eqn D2-2)
	$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) = 72.90 kip LRFD		(Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area:	A_{gv} =*(6.0in)*(0.375in)* = *1.50 in2*		
	Φ_{yv} = *1.00*	*LRFD*	*(Eqn J4-3)*
Shear Yielding:	$V_{allow\text{-}yielding} = \phi_{yv} * 0.60 * F_y * A_{gv}$ = *32.40 kip*		*(Eqn J4-3)*
	Φ_{rv} = *0.75*	*LRFD*	*(Eqn J4-4)*
Net Area:	A_{nv} = *(6.0in)*(0.375in)-(4*0.3125in)*(0.375in) = 0.59 in2*		*(Sec B4.3)*
Shear Rupture:	$V_{allow\text{-}rupture} = \phi_{rv} * 0.60 * F_u * A_{nv}$ = *20.66 kip*		
	V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) = 20.66 kip LRFD		*(Horizontal Component)*

Capacity: critical case of tensile, bending moment and shear on plate

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

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \dfrac{bd^2}{4}\ \dfrac{6*0.375^2}{4} =$ *0.2109 in3*

ϕ_b = *0.9* *LRFD* *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = *6.834 kip-in*

Moment Arm: $Moment_{arm}$ = *1.313 in* *(Eqn F11-1)*

Allowable Load: $P_{allow} = \dfrac{M_{allow}}{Moment_{arm}} =$ *5.207 kip LRFD* *(Vertical Component)*

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \dfrac{bd^2}{4} = \dfrac{6*0.375^2}{4} =$ *0.2109 in3*

ϕ_b = *0.9* *LRFD* *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = *6.834 kip-in* *(Eqn F11-1)*

Moment Arm: $Moment_{arm}$ = *1.313 in*

Allowable Load: $P_{allow} = \dfrac{M_{allow}}{Moment_{arm}} =$ *5.207 kip LRFD* *(Horizontal Component)*

Prying of Piece #33386A - L3" x 3" x 3/8" x 0'-6" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" dia Hole for Hilti KB-TZ 1/2" 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 (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 4 $V_{allow}*N$	$T_{allow}*N$
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: A307 Gr. A (Common Bolts), bearing type connection

Nominal Tensile Strength: F_t = 45 ksi

Bracket Thickness: $th_{bracket}$ = 0.375 in

Nominal Shear Strength, Threads Excluded: F_v = 27 ksi (Table J3.2 AISC 14th)

Bolt Diameter: d_{bcd} = 0.50 in

Bolt Area: $A_{bcd} = 0.25*\pi*d_{bcd}^2$ = 0.20 in2

Resistance factor for bolt tension or shear: ϕ= 0.75 LRFD

Shear Capacity of single bearing type bolt: $V_{allow\text{-}bolt} = \phi*F_v*A_{bcd}$ = 3.98 kip

Tension Capacity of single bearing type bolt: $T_{allow\text{-}bolt} = \phi*F_t*A_{bcd}$ = 6.63 kip

Bolt bearing strength at bracket connection: (Section J3.10) F_u = 58 ksi

Bolt edge distance: edge-dist = 1.00 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole and edge of adjacent hole or edge of plate: L_c = edge-dist -0.5*bh = 0.72 in2

Single end bolt bearing capacity: $Bolt_{br'g} = \min[(1.5*L_c*th_{plate}*F_u),(3.0*d_{bcd}*th_{plate}*F_u)]$ = 23.45 kip

$2*Bolt_{allow\text{-}bolt} = 2*\phi*Bolt_{bearing}$ 35.17 kip LRFD

Prying of Piece #33386A - L3" x 3" x 3/8" x 0'-6" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" dia Hole for Hilti KB-TZ 1/2" 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)

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

$\phi N_n =$ 670 lbf

$\phi V_n =$ 1100 lbf

1/2" 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, which allows the conversion of the allowable loads to ASD, requiring input which is specific to the racking system, the allowable load fore this Piece is given in LRFD only.

Overall Capacity of Seismic Load - Piece #33386A - L3" x 3" x 3/8" x 0'-6" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	670 lbf	Tension	Vertical & Horizontal Allowable Load - (LRFD)
$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	1100 lbf	Shear	
$Load_{allowable\text{-}total\text{-}on\text{-}4"\ min\text{-}concrete\text{-}slab}$ =	1275 lbf	Tension	
$Load_{allowable\text{-}total\text{-}on\text{-}4"\ min\text{-}concrete\text{-}slab}$ =	2552 lbf	Shear	

Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 3/8" Floor & Wall Brackets (included within this calculation package for reference).

Piece #43384A - L4" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

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

Bracket Thickness: $th_{bracket}$ = 0.375 in

Nominal Tensile Strength: F_t = 60 ksi *(Carbon Steel)*

Ultimate Strength: F_u = 58 ksi

Anchor Spacing s = 0.00

Anchor Diameter: [redacted] d_b = 0.500 in

Steel strength in tension: N_{sa} = *10.705 kip*

Max distance from bolt to applied force: b = 1.313 in

Tributary length to each bolt: *p = max 2b, but ≤ s* 2.63 in

Adjusted prying distance: $b' = \left[b - \frac{d_b}{2}\right] =$ 1.06 in (Eq. 9-21)

Resistance factor for prying:
Ω_{pr} = *1.67 ASD*
Φ_{pr} = *0.90 LRFD*
Φ_t = *0.75 LRFD*

Maximum bolt force for prying: $T = \frac{\Phi_{pr} * F_u * th_{bracket}{}^2 * p}{2 * b} =$ 7.34 kip (Eq. 9-22a)

Available tensile strength of single bolt: $B = \Phi_t * N_{sa} =$ *8.03 kip* *(per ESR-1917)*

$$t_c = \sqrt{\frac{4 * B * b'}{\Phi_{LRFD} * p * F_u}} = 0.547 \text{ in}$$ (Eq. 9-30a)

T_{avail} = BQ = 6.66 kip (Eq. 9-31)

Q = 0.829 *a'<, Q = 1*

0 ≤ a' ≤ 1, Q = $\left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') =$ *0.829* (Eq. 9-33)

a'>1, Q = $\left(\frac{t}{t_c}\right)^2 (1 + \delta) =$ *0.829* (Eq. 34)

Width of hole along length of plate: d' = *0.625 in*

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

Additional variables for prying calculation:

$\delta = 1 - \frac{d'}{p} =$ *0.76* *Eq. (9-24)*

$a' = a + \frac{d_b}{2} =$ *1.75 in* $\leq \left(1.25 * b * \frac{d_b}{2}\right) =$ *1.89 in* *Eq. (9-27)*

$\rho = \frac{b'}{a'} =$ *0.61* *Eq. (9-26)*

$\beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) =$ *0.34 in* *Eq. (9-25)*

$\alpha' = if\left[\beta => 1, 1, \min\left[1, \frac{1}{\delta} * \left(\frac{\beta}{1 - \beta}\right)\right]\right] =$ *0.67*

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

$$t_{\min} = \sqrt{\frac{4 * T * b'}{\Phi_{pr} * p * F_u * (1 + \delta * \alpha')}} =$$ *0.37 in LRFD* *(Eq. 9-23a)*

Thickness of Bracket is 0.375 in - therefore, Prying OK

Piece #43384A - L4" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" 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:	$th_{bracket}$ = 0.375 in		
Yield Strength:	F_y = 36 ksi		(Table 2-4)
Ultimate Strength:	F_u = 58 ksi		
Gross Area:	A_g = **(4.0in)*(0.375in)** = 1.50 in2		
	ϕ_{t-y} = 0.9	LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow-yielding} = \phi_{t-y} * F_y * A_g$ = 48.60 kip		(Eqn D2-1)
Shear Lag Factor:	U = 1.0		(Table D3.1)
Net Area:	A_n = (4.0in)*(0.375in)-(3*0.3125in)*(0.375in) = 1.15 in2		(Sec B4.3)
Effective Net Area:	A_e = $A_n * U$ = 1.15 in2		(Eqn D3-1)
	ϕ_{tr} = 0.75	LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow-rupture} = \phi_{t-r} * F_u * A_e$ = 49.96 kip		(Eqn D2-2)
	$T_{allow-bracket}$ = min($T_{allow-yielding}$, $T_{allow-rupture}$) = 48.60 kip LRFD		(Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area:	A_{gv} =(4.0in)*(0.375in) = 1.50 in2		
	Φ_{yv} = 1.00	LRFD	(Eqn J4-3)
Shear Yielding:	$V_{allow-yielding} = \phi_{yv} * 0.60 * F_y * A_{gv}$ = 32.40 kip		(Eqn J4-3)
	Φ_{rv} = 0.75	LRFD	(Eqn J4-4)
Net Area:	A_{nv} = (4.0in)*(0.375in)-(3*0.3125in)*(0.375in) = 1.15 in2		(Sec B4.3)
Shear Rupture:	$V_{allow-rupture} = \phi_{rv} * 0.60 * F_u * A_{nv}$ = 39.97 kip		
	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 (Side A):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4}$ = 0.1406 in3

ϕ_b = 0.9 LRFD (Sec F1)

$M_{allow} = \phi * F_y * Z$ = 4.556 kip-in

Moment Arm: $Moment_{arm}$ = 2.813 in (Eqn F11-1)

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 1.620 kip LRFD (Vertical Component)

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4}$ = 0.1406 in3

ϕ_b = 0.9 LRFD (Sec F1)

$M_{allow} = \phi * F_y * Z$ = 4.556 kip-in (Eqn F11-1)

Moment Arm: $Moment_{arm}$ = 1.313 in

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 3.471 kip LRFD (Horizontal Component)

Piece #43384A - L4" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" 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 (in)	V_{allow}	*Design Thickness (in)*	T_{allow}	Number of Screws N = 3 V_{allow}*N	T_{allow}*N
0.048-0.048	603 lbf	0.048	210 lbf	**1809 lbf**	**630 lbf**
0.048-0.075	1001 lbf	0.06	331 lbf	**3003 lbf**	**993 lbf**
0.060-0.060	833 lbf	0.075	409 lbf	**2499 lbf**	**1227 lbf**
0.075-0.078	1058 lbf	0.105	548 lbf	**3174 lbf**	**1644 lbf**
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	**3063 lbf**	**2691 lbf**
		3/16"	1439 lbf		4317 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: $th_{bracket}$ = 0.375 in

Nominal Shear Strength, Threads Excluded: F_v = 27 ksi *(Table J3.2 AISC 14th)*

Bolt Diameter: d_{bcd} = 0.50 in

Bolt Area: $A_{bcd} = 0.25*\pi*d_{bcd}^2 =$ 0.20 in2

Resistance factor for bolt tension or shear: ϕ= 0.75 *LRFD*

Shear Capacity of single bearing type bolt: $V_{allow\text{-}bolt} = \phi*F_v*A_{bcd} =$ 3.98 kip

Tension Capacity of single bearing type bolt: $T_{allow\text{-}bolt} = \phi*F_t*A_{bcd} =$ 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 = 0.563 in

Clear distance between edge of hole and edge of adjacent hole or edge of plate: L_c = edge-dist -0.5*bh = 1.22 in2

Single end bolt bearing capacity: $Bolt_{br'g} = \min[(1.5*L_c*th_{plate}*F_u),(3.0*d_{bcd}*th_{plate}*F_u)] =$ 32.63 kip

$Bolt_{allow\text{-}bolt} = \phi*Bolt_{bearing}$ 24.47 kip LRFD

Piece #43384A - L4" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" 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)

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

$\phi N_n =$ *370 lbf*

$\phi V_n =$ *500 lbf*

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

$\phi N_n =$ *1000 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, which allows the conversion of the allowable loads to ASD, requiring input which is specific to the racking system, the allowable load fore this Piece is given in LRFD only.

Overall Capacity of Seismic Load - Piece #43384A - L4" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

$\text{Load}_{\text{allowable-total-on-concrete-ove-metal-deck}} =$ **370 lbf**	**Tension**		
$\text{Load}_{\text{allowable-total-on-concrete-ove-metal-deck}} =$ **500 lbf**	**Shear**	**Vertical & Horizontal Allowable Load - (LRFD)**	
$\text{Load}_{\text{allowable-total-on-4" min-concrete-slab}} =$ **1000 lbf**	**Tension**		
$\text{Load}_{\text{allowable-total-on-4" min-concrete-slab}} =$ **1322 lbf**	**Shear**		

Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 3/8" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #43384B - L4" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min) w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

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Bracket Thickness:	$th_{bracket}$ =	0.375 in	
Nominal Tensile Strength:	F_t =	60 ksi	*(Carbon Steel)*
Ultimate Strength:	F_u =	58 ksi	
Anchor Spacing	s =	0.00	
Anchor Diameter: [redacted]	d_b =	0.500 in	
Steel strength in tension:	N_{sa} =	*10.705 kip*	
Max distance from bolt to applied force:	b =	1.313 in	
Tributary length to each bolt:	*p = max 2b, but ≤ s*	2.63 in	
Adjusted prying distance:	$b' = \left[b - \frac{d_b}{2}\right] =$	1.06 in	(Eq. 9-21)
Resistance factor for prying:	Ω_{pr} =	*1.67 ASD*	
	Φ_{pr} =	*0.90 LRFD*	
	Φ_t =	*0.75 LRFD*	

Maximum bolt force for prying: $T = \frac{\Phi_{pr} * F_u * th_{bracket}{}^2 * p}{2 * b} =$ 7.34 kip (Eq. 9-22a)

Available tensile strength of single bolt: $B = \Phi_t * N_{sa} =$ *8.03 kip* *(per ESR-1917)*

$$t_c = \sqrt{\frac{4 * B * b'}{\Phi_{LRFD} * p * F_u}} = 0.547 \text{ in}$$ (Eq. 9-30a)

T_{avail} *= BQ = 6.66 kip* (Eq. 9-31)

Q = *0.829* *a'<, Q = 1*

0 ≤ a' ≤ 1, Q = $\left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') =$ *0.829* (Eq. 9-33)

a'>1, Q = $\left(\frac{t}{t_c}\right)^2 (1 + \delta) =$ *0.829* (Eq. 34)

Width of hole along length of plate: d' = *0.625 in*

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

Additional variables for prying calculation:

$\delta = 1 - \frac{d'}{p} =$ *0.76* *Eq. (9-24)*

$a' = a + \frac{d_b}{2} =$ *1.75 in* $\leq \left(1.25 * b * \frac{d_b}{2}\right) =$ *1.89 in* *Eq. (9-27)*

$\rho = \frac{b'}{a'} =$ *0.61* *Eq. (9-26)*

$\beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) =$ *0.34 in* *Eq. (9-25)*

$\alpha' = if\left[\beta => 1, 1, \min\left[1, \frac{1}{\delta} * \left(\frac{\beta}{1-\beta}\right)\right]\right] =$ *0.67*

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

$$t_{min} = \sqrt{\frac{4 * T * b'}{\Phi_{pr} * p * F_u * (1 + \delta * \alpha')}} =$$ *0.37 in LRFD* *(Eq. 9-23a)*

Thickness of Bracket is 0.375 in - therefore, Prying OK

Piece #43384B - L4" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" 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: $th_{bracket}$ = 0.375 in

Yield Strength: F_y = *36 ksi* *(Table 2-4)*

Ultimate Strength: F_u = *58 ksi*

Gross Area: A_g = ***(4.0in)*(0.375in)*** = *1.50 in2*

$\phi_{t\text{-}y}$ = *0.9* *LRFD* (Eqn D2.1)

Tensile Yielding: $T_{allow\text{-}yielding}$ = $\phi_{t\text{-}y} * F_y * A_g$ = 48.60 kip (Eqn D2-1)

Shear Lag Factor: $U =$ *1.0* (Table D3.1)

Net Area: A_n = *(4.0in)*(0.375in)-(2*0.3125in)*(0.375in)* = *1.27 in2* (Sec B4.3)

Effective Net Area: A_e = $A_n * U$ = *1.27 in2* (Eqn D3-1)

ϕ_{tr} = *0.75* *LRFD* (Eqn D2-2)

Tensile Rupture: $T_{allow\text{-}rupture}$ = $\phi_{t\text{-}r} * F_u * A_e$ = 55.05 kip (Eqn D2-2)

$T_{allow\text{-}bracket}$ = $\min(T_{allow\text{-}yielding}, T_{allow\text{-}rupture})$ = 48.60 kip LRFD (Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area: A_{gv} *=(4.0in)*(0.375in)* = *1.50 in2*

Φ_{yv} = *1.00* *LRFD* *(Eqn J4-3)*

Shear Yielding: $V_{allow\text{-}yielding}$ = $\phi_{yv} * 0.60 * F_y * A_{gv}$ = *32.40 kip* *(Eqn J4-3)*

Φ_{rv} = *0.75* *LRFD* *(Eqn J4-4)*

Net Area: A_{nv} = *(4.0in)*(0.375n)-(2*0.3125in)*(0.375in)* = *1.27 in2* *(Sec B4.3)*

Shear Rupture: $V_{allow\text{-}rupture}$ = $\phi_{rv} * 0.60 * F_u * A_{nv}$ = *44.04 kip*

$V_{allow} = \min(V_{allow\text{-}yielding}, V_{allow\text{-}rupture})$ = 32.40 kip LRFD *(Horizontal Component)*

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

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \dfrac{bd^2}{4} = \dfrac{4*0.375^2}{4} =$ *0.1406 in3*

ϕ_b = *0.9* *LRFD* *(Sec F1)*

M_{allow} = $\phi * F_y * Z$ = *4.556 kip-in*

Moment Arm: $Moment_{arm}$ = *2.313 in* *(Eqn F11-1)*

Allowable Load: $P_{allow} = \dfrac{M_{allow}}{Moment_{arm}} =$ *1.970 kip LRFD* *(Vertical Component)*

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \dfrac{bd^2}{4} = \dfrac{4*0.375^2}{4} =$ *0.1406 in3*

ϕ_b = *0.9* *LRFD* *(Sec F1)*

M_{allow} = $\phi * F_y * Z$ = *4.556 kip-in* *(Eqn F11-1)*

Moment Arm: $Moment_{arm}$ = *1.313 in*

Allowable Load: $P_{allow} = \dfrac{M_{allow}}{Moment_{arm}} =$ *3.471 kip LRFD* *(Horizontal Component)*

Piece #43384B - L4" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" 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 (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 4 V_{allow}*N	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: *A307 Gr. A (Common Bolts), bearing type connection*

Nominal Tensile Strength: F_t = 45 ksi

Bracket Thickness: $th_{bracket}$ = 0.375 in

Nominal Shear Strength, Threads Excluded: F_v = 27 ksi *(Table J3.2 AISC 14th)*

Bolt Diameter: d_{bcd} = 0.50 in

Bolt Area: $A_{bcd} = 0.25 * \pi * d_{bcd}^2$ = 0.20 in2

Resistance factor for bolt tension or shear: ϕ= 0.75 LRFD

Shear Capacity of single bearing type bolt: $V_{allow\text{-}bolt} = \phi * F_v * A_{bcd}$ = 3.98 kip

Tension Capacity of single bearing type bolt: $T_{allow\text{-}bolt} = \phi * F_t * A_{bcd}$ = 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 = 0.563 in

Clear distance between edge of hole and edge of adjacent hole or edge of plate: L_c = edge-dist -0.5*bh = 1.22 in2

Single bearing capacity: $Bolt_{br'g} = \min[(1.5 * L_c * th_{plate} * F_u),(3.0 * d_{bcd} * th_{plate} * F_u)]$ = 32.63 kip

$Bolt_{allow\text{-}bolt} = \phi * Bolt_{bearing}$ 24.47 kip LRFD

Piece #43384B - L4" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" 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)

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

$\phi N_n =$ 370 lbf

$\phi V_n =$ 500 lbf

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

$\phi N_n =$ 1000 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, which allows the conversion of the allowable loads to ASD, requiring input which is specific to the racking system, the allowable load fore this Piece is given in LRFD only.

Overall Capacity of Seismic Load - Piece #43384B - L4" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	370 lbf	Tension	Vertical & Horizontal Allowable Load - (LRFD)
$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	500 lbf	Shear	
$Load_{allowable\text{-}total\text{-}on\text{-}4"\ min\text{-}concrete\text{-}slab}$ =	1000 lbf	Tension	
$Load_{allowable\text{-}total\text{-}on\text{-}4"\ min\text{-}concrete\text{-}slab}$ =	1322 lbf	Shear	

Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 3/8" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #43386A - L4" x 3" x 3/8" x 0'-6" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

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Bracket Thickness:	$th_{bracket}$ =	0.375 in	
Nominal Tensile Strength:	F_t =	60 ksi	*(Carbon Steel)*
Ultimate Strength:	F_u =	58 ksi	
Anchor Spacing	s =	4.00	
Anchor Diameter: [redacted]	d_b =	0.500 in	
Steel strength in tension:	N_{sa} =	*10.705 kip*	
Max distance from bolt to applied force:	b =	1.313 in	
Tributary length to each bolt:	*p = max 2b, but ≤ s*	2.63 in	
Adjusted prying distance:	$b' = \left[b - \frac{d_b}{2}\right] =$	1.06 in	(Eq. 9-21)

Resistance factor for prying:
Ω_{pr} = *1.67 ASD*
Φ_{pr} = *0.90 LRFD*
Φ_t = *0.75 LRFD*

Maximum bolt force for prying: $T = \dfrac{\Phi_{pr} * F_u * th_{bracket}{}^2 * p}{2*b} =$ 7.34 kip (Eq. 9-22a)

Available tensile strength of single bolt: $B = \Phi_t * N_{sa} =$ *8.03 kip* *(per ESR-1917)*

$$t_c = \sqrt{\frac{4*B*b'}{\Phi_{LRFD} * p * F_u}} = 0.547 \text{ in}$$ (Eq. 9-30a)

T_{avail} = BQ = *6.66 kip* (Eq. 9-31)

Q = *0.829*

a'<, Q = *1*

0 ≤ a' ≤ 1, Q = $\left(\frac{t}{t_c}\right)^2 (1+\delta*\alpha') =$ *0.829* (Eq. 9-33)

a'>1, Q = $\left(\frac{t}{t_c}\right)^2 (1+\delta) =$ *0.829* (Eq. 34)

Width of hole along length of plate:	d' =	*0.625 in*
Distance from bolt centerline to edge of plate:	a =	*1.50 in*

Additional variables for prying calculation:

$\delta = 1 - \frac{d'}{p} =$ *0.76* *Eq. (9-24)*

$a' = a + \frac{d_b}{2} =$ *1.75 in* $\leq \left(1.25*b*\frac{d_b}{2}\right) =$ *1.89 in* *Eq. (9-27)*

$\rho = \frac{b'}{a'} =$ *0.61* *Eq. (9-26)*

$\beta = \frac{1}{\rho}*\left(\frac{B}{T}-1\right) =$ *0.34 in* *Eq. (9-25)*

$\alpha' = if\left[\beta => 1, 1, \min\left[1, \frac{1}{\delta}*\left(\frac{\beta}{1-\beta}\right)\right]\right] =$ *0.67*

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

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr} * p * F_u * (1+\delta*\alpha')}} =$$ *0.37 in LRFD* *(Eq. 9-23a)*

Thickness of Bracket is 0.375 in - therefore, Prying OK

Piece #43386A - L4" x 3" x 3/8" x 0'-6" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" 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:	$th_{bracket}$ = 0.375 in		
Yield Strength:	F_y = *36 ksi*		*(Table 2-4)*
Ultimate Strength:	F_u = *58 ksi*		
Gross Area:	A_g = **(6.0in)*(0.375in)** = *2.25 in2*		
	$\phi_{t\text{-}y}$ = *0.9*	*LRFD*	(Eqn D2.1)
Tensile Yielding:	$T_{allow\text{-}yielding}$ = $\phi_{t\text{-}y}$*F_y*A_g = 72.90 kip		(Eqn D2-1)
Shear Lag Factor:	U = *1.0*		(Table D3.1)
Net Area:	A_n = *(6.0in)*(0.375in)-(4*0.3125in)*(0.375in) = 1.78 in2*		(Sec B4.3)
Effective Net Area:	A_e = A_n*U =	*1.78 in2*	(Eqn D3-1)
	ϕ_{tr} = *0.75*	*LRFD*	(Eqn D2-2)
Tensile Rupture:	$T_{allow\text{-}rupture}$ = $\phi_{t\text{-}r}$*F_u*A_e = 77.48 kip		(Eqn D2-2)
	$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) =	72.90 kip LRFD	(Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area:	A_{gv} *=(6.0in)*(0.375in) = 2.25 in2*		
	Φ_{yv} = *1.00*	*LRFD*	*(Eqn J4-3)*
Shear Yielding:	$V_{allow\text{-}yielding}$ = ϕ_{yv}*0.60*F_y*A_{gv} =	*48.60 kip*	*(Eqn J4-3)*
	Φ_{rv} = *0.75*	*LRFD*	*(Eqn J4-4)*
Net Area:	A_{nv} *=(6.0in)*(0.375in)-(4*0.3125in)*(0.375in) = 1.78 in2*		*(Sec B4.3)*
Shear Rupture:	$V_{allow\text{-}rupture}$ = ϕ_{rv}*0.60*F_u*A_{nv} =	*61.99 kip*	*(Eqn J4-4)*
	V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) =	48.60 kip LRFD	*(Horizontal Component)*

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

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{6*0.375^2}{4} =$ *0.2109 in3*

ϕ_b = *0.9* *LRFD* *(Sec F1)*

M_{allow} = ϕ*F_y*Z = *6.834 kip-in*

Moment Arm: $Moment_{arm}$ = *2.813 in* *(Eqn F11-1)*

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ *2.430 kip LRFD* *(Vertical Component)*

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{6*0.375^2}{4} =$ *0.2109 in3*

ϕ_b = *0.9* *LRFD* *(Sec F1)*

M_{allow} = ϕ*F_y*Z = *6.834 kip-in* *(Eqn F11-1)*

Moment Arm: $Moment_{arm}$ = *1.313 in*

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ *5.207 kip LRFD* *(Horizontal Component)*

Piece #43386A - L4" x 3" x 3/8" x 0'-6" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" 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 (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 4 $V_{allow}*N$	$T_{allow}*N$
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: *A307 Gr. A (Common Bolts), bearing type connection*

Nominal Tensile Strength: F_t = 45 ksi

Bracket Thickness: $th_{bracket}$ = 0.375 in

Nominal Shear Strength, Threads Excluded: F_v = 27 ksi *(Table J3.2 AISC 14th)*

Bolt Diameter: d_{bcd} = 0.50 in

Bolt Area: $A_{bcd} = 0.25*\pi*d_{bcd}^2$ = 0.20 in2

Resistance factor for bolt tension or shear: ϕ= 0.75 LRFD

Shear Capacity of single bearing type bolt: $V_{allow\text{-}bolt} = \phi*F_v*A_{bcd}$ = 3.98 kip

Tension Capacity of single bearing type bolt: $T_{allow\text{-}bolt} = \phi*F_t*A_{bcd}$ = 6.63 kip

Bolt bearing strength at bracket connection: (Section J3.10) F_u = 58 ksi

Bolt edge distance: edge-dist = 1.00 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole and edge of adjacent hole or edge of plate: L_c = edge-dist -0.5*bh = 0.72 in2

Single end bolt bearing capacity: $Bolt_{br'g} = \min[(1.5*L_c*th_{plate}*F_u),(3.0*d_{bcd}*th_{plate}*F_u)]$ = 23.45 kip

$2*Bolt_{allow\text{-}bolt} = 2*\phi*Bolt_{bearing}$ 35.17 kip LRFD

Piece #43386A - L4" x 3" x 3/8" x 0'-6" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" 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)

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

$\phi N_n =$ 670 lbf

$\phi V_n =$ 1100 lbf

1/2" 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, which allows the conversion of the allowable loads to ASD, requiring input which is specific to the racking system, the allowable load fore this Piece is given in LRFD only.

Overall Capacity of Seismic Load - Piece #43386A - L4" x 3" x 3/8" x 0'-6" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck} =$	670 lbf	Tension	Vertical & Horizontal Allowable Load - (LRFD)
$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck} =$	1100 lbf	Shear	
$Load_{allowable\text{-}total\text{-}on\text{-}4''\ min\text{-}concrete\text{-}slab} =$	1275 lbf	Tension	
$Load_{allowable\text{-}total\text{-}on\text{-}4''\ min\text{-}concrete\text{-}slab} =$	2552 lbf	Shear	

Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 3/8" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #43388A - L4" x 3" x 3/8" x 0'-8" Steel Angle Bracket (A36 min)
w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

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Bracket Thickness: $th_{bracket}$ = 0.375 in

Nominal Tensile Strength: F_t = 60 ksi *(Carbon Steel)*

Ultimate Strength: F_u = 58 ksi

Anchor Spacing s = 3.00

Anchor Diameter: [redacted] d_b = 0.500 in

Steel strength in tension: N_{sa} = *10.705 kip*

Max distance from bolt to applied force: b = 1.313 in

Tributary length to each bolt: *p = max 2b, but ≤ s* 2.63 in

Adjusted prying distance: $b' = \left[b - \frac{d_b}{2}\right] =$ 1.06 in (Eq. 9-21)

Resistance factor for prying: Ω_{pr} = *1.67 ASD*
Φ_{pr} = *0.90 LRFD*
Φ_t = *0.75 LRFD*

Maximum bolt force for prying: $T = \frac{\Phi_{pr} * F_u * th_{bracket}{}^2 * p}{2 * b} =$ 7.34 kip (Eq. 9-22a)

Available tensile strength of single bolt: $B = \Phi_t * N_{sa} =$ *8.03 kip* *(per ESR-1917)*

$$t_c = \sqrt{\frac{4 * B * b'}{\Phi_{LRFD} * p * F_u}} = 0.547 \text{ in}$$ (Eq. 9-30a)

T_{avail} = BQ = *6.66 kip* (Eq. 9-31)

Q = *0.829* *a'<, Q =* *1*

0 ≤ a' ≤ 1, Q = $\left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') =$ *0.829* (Eq. 9-33)

a'>1, Q = $\left(\frac{t}{t_c}\right)^2 (1 + \delta) =$ *0.829* (Eq. 34)

Width of hole along length of plate: d' = *0.625 in*

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

Additional variables for prying calculation:

$\delta = 1 - \frac{d'}{p} =$ *0.76* *Eq. (9-24)*

$a' = a + \frac{d_b}{2} =$ *1.75 in* $\leq \left(1.25 * b * \frac{d_b}{2}\right) =$ *1.89 in* *Eq. (9-27)*

$\rho = \frac{b'}{a'} =$ *0.61* *Eq. (9-26)*

$\beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) =$ *0.34 in* *Eq. (9-25)*

$\alpha' = if\left[\beta => 1, 1, \min\left[1, \frac{1}{\delta} * \left(\frac{\beta}{1-\beta}\right)\right]\right] =$ *0.67*

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

$$t_{\min} = \sqrt{\frac{4 * T * b'}{\Phi_{pr} * p * F_u * (1 + \delta * \alpha')}} =$$ *0.37 in LRFD* *(Eq. 9-23a)*

Thickness of Bracket is 0.375 in - therefore, Prying OK

Piece #43388A - L4" x 3" x 3/8" x 0'-8" Steel Angle Bracket (A36 min)
w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.5625" 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:	$th_{bracket}$ = 0.375 in		
Yield Strength:	F_y = 36 ksi		(Table 2-4)
Ultimate Strength:	F_u = 58 ksi		
Gross Area:	A_g = **(8.0in)*(0.375in)** = 3.00 in2		
	$\phi_{t\text{-}y}$ = 0.9	LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow\text{-}yielding} = \phi_{t\text{-}y} {}^*F_y {}^*A_g$ = 97.20 kip		(Eqn D2-1)
Shear Lag Factor:	U = 1.0		(Table D3.1)
Net Area:	A_n = (8.0in)*(0.375in)-(5*0.3125in)*(0.375in) = 2.41 in2		(Sec B4.3)
Effective Net Area:	A_e = A_n*U = 2.41 in2		(Eqn D3-1)
	ϕ_{tr} = 0.75	LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow\text{-}rupture} = \phi_{t\text{-}r} {}^*F_u {}^*A_e$ = 105.01 kip		(Eqn D2-2)
	$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) =	97.20 kip LRFD	(Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area:	A_{gv} =(8.0in)*(0.375in)= 3.00 in2		
	Φ_{yv} = 1.00	LRFD	(Eqn J4-3)
Shear Yielding:	$V_{allow\text{-}yielding} = \phi_{yv} {}^*0.60^*F_y {}^*A_{gv}$ =	64.80 kip	(Eqn J4-3)
	Φ_{rv} = 0.75	LRFD	(Eqn J4-4)
Net Area:	A_{nv} = (8.0in)*(0.375in)-(5*0.3125in)*(0.375in) = 2.41 in2		(Sec B4.3)
Shear Rupture:	$V_{allow\text{-}rupture} = \phi_{rv} {}^*0.60^*F_u {}^*A_{nv}$ =	84.01 kip	
	V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) =	64.80 kip	(Horizontal Component)

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

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \frac{bd^2}{4} : \frac{8*0.375^2}{4} =$ 0.2813 in3

ϕ_b = 0.9 LRFD (Sec F1)

$M_{allow} = \phi^*F_y{}^*Z$ = 9.113 kip-in

Moment Arm: $Moment_{arm}$ = 2.813 in (Eqn F11-1)

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 3.240 kip LRFD (Vertical Component)

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{8*0.375^2}{4} =$ 0.2813 in3

ϕ_b = 0.9 LRFD (Sec F1)

$M_{allow} = \phi^*F_y{}^*Z$ = 9.113 kip-in (Eqn F11-1)

Moment Arm: $Moment_{arm}$ = 1.313 in

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 6.943 kip LRFD (Horizontal Component)

Piece #43388A - L4" x 3" x 3/8" x 0'-8" Steel Angle Bracket (A36 min)
w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" 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 (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 5 $V_{allow}*N$	$T_{allow}*N$
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 (Common Bolts), bearing type connection*

Nominal Tensile Strength: F_t = 45 ksi

Bracket Thickness: $th_{bracket}$ = 0.375 in

Nominal Shear Strength, Threads Excluded: F_v = 27 ksi (Table J3.2 AISC 14th)

Bolt Diameter: d_{bcd} = 0.50 in

Bolt Area: $A_{bcd} = 0.25*\pi*d_{bcd}^2$ = 0.20 in2

Resistance factor for bolt tension or shear: ϕ= 0.75 LRFD

Shear Capacity of single bearing type bolt: $V_{allow\text{-}bolt} = \phi*F_v*A_{bcd}$ = 3.98 kip

Tension Capacity of single bearing type bolt: $T_{allow\text{-}bolt} = \phi*F_t*A_{bcd}$ = 6.63 kip

Bolt bearing strength at bracket connection: (Section J3.10) F_u = 58 ksi

Bolt edge distance: edge-dist = 1.00 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole and edge of adjacent hole or edge of plate: L_c = edge-dist -0.5*bh = 0.72 in2

Single end bolt bearing capacity: $Bolt_{br'g} = \min[(1.5*L_c*th_{plate}*F_u),(3.0*d_{bcd}*th_{plate}*F_u)]$ = 23.45 kip

$3*Bolt_{allow\text{-}bolt} = 3*\phi*Bolt_{bearing}$ 93.80 kip LRFD

Piece #43388A - L4" x 3" x 3/8" x 0'-8" Steel Angle Bracket (A36 min)
w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.5625" 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)

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

$\phi N_n =$ 750 lbf

$\phi V_n =$ 1240 lbf

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

$\phi N_n =$ 1400 lbf

$\phi V_n =$ 3150 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, which allows the conversion of the allowable loads to ASD, requiring input which is specific to the racking system, the allowable load fore this Piece is given in LRFD only.

Overall Capacity of Seismic Load - Piece #43388A - L4" x 3" x 3/8" x 0'-8" Steel Angle Bracket (A36 min)
w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	750 lbf	Tension	Vertical & Horizontal Allowable Load - (LRFD)
$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	1240 lbf	Shear	
$Load_{allowable\text{-}total\text{-}on\text{-}4\text{"}\ min\text{-}concrete\text{-}slab}$ =	1400 lbf	Tension	
$Load_{allowable\text{-}total\text{-}on\text{-}4\text{"}\ min\text{-}concrete\text{-}slab}$ =	3150 lbf	Shear	

Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 3/8" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #63384A - L6" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min) w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

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Bracket Thickness:	$th_{bracket}$ =	0.375 in	
Nominal Tensile Strength:	F_t =	60 ksi	*(Carbon Steel)*
Ultimate Strength:	F_u =	58 ksi	
Anchor Spacing	s =	0.00	
Anchor Diameter: [redacted]	d_b =	0.500 in	
Steel strength in tension:	N_{sa} =	*10.705 kip*	
Max distance from bolt to applied force:	b =	1.313 in	
Tributary length to each bolt:	*p = max 2b, but ≤ s*	2.63 in	
Adjusted prying distance:	$b' = \left[b - \frac{d_b}{2}\right] =$	1.06 in	(Eq. 9-21)

Resistance factor for prying:
Ω_{pr} = *1.67 ASD*
Φ_{pr} = *0.90 LRFD*
Φ_t = *0.75 LRFD*

Maximum bolt force for prying: $T = \dfrac{\Phi_{pr} * F_u * th_{bracket}^{\ 2} * p}{2*b} =$ 7.34 kip (Eq. 9-22a)

Available tensile strength of single bolt: $B = \Phi_t * N_{sa} =$ *8.03 kip* *(per ESR-1917)*

$$t_c = \sqrt{\frac{4*B*b'}{\Phi_{LRFD} * p * F_u}} = 0.547 \text{ in}$$ (Eq. 9-30a)

T_{avail} = BQ = *6.66 kip* (Eq. 9-31)

Q = *0.712* *a'<, Q = 1*

0 ≤ a' ≤ 1, Q = $\left(\frac{t}{t_c}\right)^2 (1+\delta*\alpha') =$ *0.712* (Eq. 9-33)

a'>1, Q = $\left(\frac{t}{t_c}\right)^2 (1+\delta) =$ *0.829* (Eq. 34)

Width of hole along length of plate:	d' =	*0.625 in*
Distance from bolt centerline to edge of plate:	a =	*1.50 in*

Additional variables for prying calculation:

$\delta = 1 - \frac{d'}{p} =$ *0.76* *Eq. (9-24)*

$a' = a + \frac{d_b}{2} =$ *1.75 in* $\le \left(1.25*b*\frac{d_b}{2}\right) =$ *1.89 in* *Eq. (9-27)*

$\rho = \frac{b'}{a'} =$ *0.61* *Eq. (9-26)*

$\beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) =$ *0.34 in* *Eq. (9-25)*

$\alpha' = if\left[\beta => 1, 1, \min\left[1, \frac{1}{\delta} * \left(\frac{\beta}{1-\beta}\right)\right]\right] =$ *0.67*

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

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr} * p * F_u * (1+\delta*\alpha')}} =$$ *0.37 in LRFD* *(Eq. 9-23a)*

Thickness of Bracket is 0.375 in - therefore, Prying OK

Prying of Piece #63384A - L6" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" 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: $th_{bracket}$ = 0.375 in

Yield Strength: F_y = 36 ksi (Table 2-4)

Ultimate Strength: F_u = 58 ksi

Gross Area: A_g = **(4.0in)*(0.375in)** = 1.50 in2

$\phi_{t\text{-}y}$ = 0.9 LRFD (Eqn D2.1)

Tensile Yielding: $T_{allow\text{-}yielding} = \phi_{t\text{-}y} * F_y * A_g$ = 48.60 kip (Eqn D2-1)

Shear Lag Factor: U = 1.0 (Table D3.1)

Net Area: A_n = (4.0in)*(0.375in)-(3*0.3125in)*(0.375in) = 1.15 in2 (Sec B4.3)

Effective Net Area: A_e = $A_n * U$ = 1.15 in2 (Eqn D3-1)

ϕ_{tr} = 0.75 LRFD (Eqn D2-2)

Tensile Rupture: $T_{allow\text{-}rupture} = \phi_{t\text{-}r} * F_u * A_e$ = 49.96 kip (Eqn D2-2)

$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) = 48.60 kip LRFD (Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area: A_{gv} =(4.0in)*(0.25in) = 1.50 in2

Φ_{yv} = 1.00 LRFD (Eqn J4-3)

Shear Yielding: $V_{allow\text{-}yielding} = \phi_{yv} * 0.60 * F_y * A_{gv}$ = 32.40 kip (Eqn J4-3)

Φ_{rv} = 0.75 LRFD (Eqn J4-4)

Net Area: A_{nv} = (4.0in)*(0.375in)-(3*0.3125in)*(0.375in) = 1.15 in2 (Sec B4.3)

Shear Rupture: $V_{allow-rupture} = \dfrac{0.60 * F_u * A_{nv}}{\Omega_v} =$ 39.97 kip

V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) = 32.40 kip LRFD (Horizontal Component)

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

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \dfrac{bd^2}{4} = \dfrac{4*0.375^2}{4} =$ 0.1406 in3

ϕ_b = 0.9 LRFD (Sec F1)

$M_{allow} = \phi * F_y * Z$ = 4.556 kip-in

Moment Arm: $Moment_{arm}$ = 4.813 in (Eqn F11-1)

Allowable Load: $P_{allow} = \dfrac{M_{allow}}{Moment_{arm}} =$ 0.947 kip LRFD (Vertical Component)

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \dfrac{bd^2}{4} = \dfrac{4*0.375^2}{4} =$ 0.1406 in3

ϕ_b = 0.9 LRFD (Sec F1)

$M_{allow} = \phi * F_y * Z$ = 4.556 kip-in (Eqn F11-1)

Moment Arm: $Moment_{arm}$ = 1.313 in

Allowable Load: $P_{allow} = \dfrac{M_{allow}}{Moment_{arm}} =$ 3.471 kip LRFD (Horizontal Component)

Prying of Piece #63384A - L6" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" 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 (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 3 V_{allow}*N	T_{allow}*N
0.048-0.048	603 lbf	0.048	210 lbf	**1809 lbf**	**630 lbf**
0.048-0.075	1001 lbf	0.06	331 lbf	**3003 lbf**	**993 lbf**
0.060-0.060	833 lbf	0.075	409 lbf	**2499 lbf**	**1227 lbf**
0.075-0.078	1058 lbf	0.105	548 lbf	**3174 lbf**	**1644 lbf**
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	**3063 lbf**	2691 lbf
		3/16"	1439 lbf		4317 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: $th_{bracket}$ = 0.375 in

Nominal Shear Strength, Threads Excluded: F_v = 27 ksi (Table J3.2 AISC 14th)

Bolt Diameter: d_{bcd} = 0.50 in

Bolt Area: $A_{bcd} = 0.25*\pi*d_{bcd}^2 =$ 0.20 in2

Resistance factor for bolt tension or shear: ϕ= 0.75 LRFD

Shear Capacity of single bearing type bolt: $V_{allow\text{-}bolt} = \phi*F_v*A_{bcd} =$ 3.98 kip

Tension Capacity of single bearing type bolt: $T_{allow\text{-}bolt} = \phi*F_t*A_{bcd} =$ 6.63 kip

Bolt bearing strength at bracket connection: (Section J3.10) F_u = 58 ksi

Bolt edge distance: edge-dist = 1.00 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole and edge of adjacent hole or edge of plate: L_c = edge-dist -0.5*bh = 0.72 in2

Single end bolt bearing capacity: $Bolt_{br'g} = \min[(1.5*L_c*th_{plate}*F_u),(3.0*d_{bcd}*th_{plate}*F_u)] =$ 23.45 kip

$Bolt_{allow\text{-}bolt} = \phi*Bolt_{bearing}$ 17.59 kip LRFD

Piece #63384A - L6" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" 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)

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

$\phi N_n =$ 370 lbf

$\phi V_n =$ 500 lbf

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

$\phi N_n =$ 1000 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, which allows the conversion of the allowable loads to ASD, requiring input which is specific to the racking system, the allowable load fore this Piece is given in LRFD only.

Overall Capacity of Seismic Load - Piece #63384A - L6" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	370 lbf	Tension	Vertical & Horizontal Allowable Load - (LRFD)
$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	500 lbf	Shear	
$Load_{allowable\text{-}total\text{-}on\text{-}4\text{"}\ min\text{-}concrete\text{-}slab}$ =	1000 lbf	Tension	
$Load_{allowable\text{-}total\text{-}on\text{-}4\text{"}\ min\text{-}concrete\text{-}slab}$ =	1322 lbf	Shear	

Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 3/8" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #63384B - L6" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

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Bracket Thickness: $th_{bracket}$ = 0.375 in

Nominal Tensile Strength: F_t = 60 ksi *(Carbon Steel)*

Ultimate Strength: F_u = 58 ksi

Anchor Spacing s = 0.00

Anchor Diameter: [redacted] d_b = 0.500 in

Steel strength in tension: N_{sa} = *10.705 kip*

Max distance from bolt to applied force: b = 1.313 in

Tributary length to each bolt: *p = max 2b, but ≤ s* 2.63 in

Adjusted prying distance: $b' = \left[b - \frac{d_b}{2}\right] =$ 1.06 in (Eq. 9-21)

Resistance factor for prying:
Ω_{pr} = *1.67 ASD*
Φ_{pr} = *0.90 LRFD*
Φ_t = *0.75 LRFD*

Maximum bolt force for prying: $T = \frac{\Phi_{pr} * F_u * th_{bracket}^2 * p}{2 * b} =$ 7.34 kip (Eq. 9-22a)

Available tensile strength of single bolt: $B = \Phi_t * N_{sa} =$ *8.03 kip* *(per ESR-1917)*

$t_c = \sqrt{\frac{4 * B * b'}{\Phi_{LRFD} * p * F_u}} =$ 0.547 in (Eq. 9-30a)

T_{avail} = BQ = *6.66 kip* (Eq. 9-31)

Q = *0.829*

a'<, Q = *1*

0 ≤ a' ≤ 1, Q = $\left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') =$ *0.829* (Eq. 9-33)

a'>1, Q = $\left(\frac{t}{t_c}\right)^2 (1 + \delta) =$ *0.829* (Eq. 34)

Width of hole along length of plate: d' = *0.625 in*

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

Additional variables for prying calculation:

$\delta = 1 - \frac{d'}{p} =$ *0.76* *Eq. (9-24)*

$a' = a + \frac{d_b}{2} =$ *1.75 in* $\leq \left(1.25 * b * \frac{d_b}{2}\right) =$ *1.89 in* *Eq. (9-27)*

$\rho = \frac{b'}{a'} =$ *0.61* *Eq. (9-26)*

$\beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) =$ *0.34 in* *Eq. (9-25)*

$\alpha' = if\left[\beta => 1, 1, \min\left[1, \frac{1}{\delta} * \left(\frac{\beta}{1 - \beta}\right)\right]\right] =$ *0.67*

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

$t_{\min} = \sqrt{\frac{4 * T * b'}{\Phi_{pr} * p * F_u * (1 + \delta * \alpha')}} =$ *0.37 in LRFD* *(Eq. 9-23a)*

Thickness of Bracket is 0.375 in - therefore, Prying OK

Piece #63384B - L6" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" 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:	$th_{bracket}$ = 0.375 in		
Yield Strength:	F_y = 36 ksi		(Table 2-4)
Ultimate Strength:	F_u = 58 ksi		
Gross Area:	A_g = **(4.0in)*(0.375in)** = 1.50 in2		
	$\phi_{t\text{-}y}$ = 0.9	LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow\text{-}yielding}$ = $\phi_{t\text{-}y}$ *F_y *A_g = 48.60 kip		(Eqn D2-1)
Shear Lag Factor:	U = 1.0		(Table D3.1)
Net Area:	A_n = (4.0in)*(0.375in)-(2*0.3125in)*(0.375in) = 1.27 in2		(Sec B4.3)
Effective Net Area:	A_e = A_n *U = 1.27 in2		(Eqn D3-1)
	ϕ_{tr} = 0.75	LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow\text{-}rupture}$ = $\phi_{t\text{-}r}$ *F_u *A_e = 55.05 kip		(Eqn D2-2)
	$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) = 48.60 kip LRFD		(Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area:	A_{gv} =(4.0in)*(0.375in)= 1.50 in2		
	Φ_{yv} = 1.00	LRFD	(Eqn J4-3)
Shear Yielding:	$V_{allow\text{-}yielding}$ = ϕ_{yv} *0.60*F_y *A_{gv} = 32.40 kip		(Eqn J4-3)
	Φ_{rv} = 0.75	LRFD	(Eqn J4-4)
Net Area:	A_{nv} = (4.0in)*(0.375in)-(2*0.3125in)*(0.375in) = 1.27 in2		(Sec B4.3)
Shear Rupture:	$V_{allow\text{-}rupture}$ = ϕ_{rv} *0.60*F_u *A_{nv} = 44.04 kip		
	V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) = 32.40 kip LRFD		(Horizontal Component)

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

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} =$ 0.1406 in3

ϕ_b = 0.9 LRFD (Sec F1)

M_{allow} = ϕ*F_y *Z = 4.556 kip-in

Moment Arm: $Moment_{arm}$ = 4.313 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):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} =$ 0.1406 in3

ϕ_b = 0.9 LRFD (Sec F1)

M_{allow} = ϕ*F_y *Z = 4.556 kip-in (Eqn F11-1)

Moment Arm: $Moment_{arm}$ = 1.313 in

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 3.471 kip LRFD (Horizontal Component)

Piece #63384B - L6" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" 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 (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 4 V_{allow}*N	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: *A307 Gr. A (Common Bolts), bearing type connection*

Nominal Tensile Strength: F_t = 45 ksi

Bracket Thickness: $th_{bracket}$ = 0.375 in

Nominal Shear Strength, Threads Excluded: F_v = 27 ksi *(Table J3.2 AISC 14th)*

Bolt Diameter: d_{bcd} = 0.50 in

Bolt Area: $A_{bcd} = 0.25*\pi*d_{bcd}^2 =$ 0.20 in2

Resistance factor for bolt tension or shear: $\phi =$ 0.75 LRFD

Shear Capacity of single bearing type bolt: $V_{allow\text{-}bolt} = \phi*F_v*A_{bcd} =$ 3.98 kip

Tension Capacity of single bearing type bolt: $T_{allow\text{-}bolt} = \phi*F_t*A_{bcd} =$ 6.63 kip

Bolt bearing strength at bracket connection: (Section J3.10) F_u = 58 ksi

Bolt edge distance: edge-dist = 1.00 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole and edge of adjacent hole or edge of plate: L_c = edge-dist -0.5*bh = 0.72 in2

Single end bolt bearing capacity: $Bolt_{br'g} = \min[(1.5*L_c*th_{plate}*F_u),(3.0*d_{bcd}*th_{plate}*F_u)] =$ 23.45 kip

$Bolt_{allow\text{-}bolt} = \phi*Bolt_{bearing}$ 31.27 kip LRFD

Piece #63384B - L6" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" 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)

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

$\phi N_n =$ 370 lbf

$\phi V_n =$ 500 lbf

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

$\phi N_n =$ 1000 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, which allows the conversion of the allowable loads to ASD, requiring input which is specific to the racking system, the allowable load fore this Piece is given in LRFD only.

Overall Capacity of Seismic Load - Piece #63384B - L6" x 3" x 3/8" x 0'-4" Steel Angle Bracket (A36 min)
w/ (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	**370 lbf**	**Tension**	**Vertical & Horizontal Allowable Load - (LRFD)**
$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	**500 lbf**	**Shear**	
$Load_{allowable\text{-}total\text{-}on\text{-}4"\ min\text{-}concrete\text{-}slab}$ =	**1000 lbf**	**Tension**	
$Load_{allowable\text{-}total\text{-}on\text{-}4"\ min\text{-}concrete\text{-}slab}$ =	**1322 lbf**	**Shear**	

<u>Design Scope:</u>

Calculations to determine the allowable strength design capacity of the seismic restraint as detailed by 9.0 SeismicCo., 3/8" Floor & Wall Brackets (included within this calculation package for reference).

<u>Prying of Piece #63388A - L6" x 3" x 1/4" x 0'-8" Steel Angle Bracket (A36 min)</u>
<u>w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"</u>

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Bracket Thickness:	$th_{bracket}$ =	0.375 in		
Nominal Tensile Strength:	F_t =	60 ksi	*(Carbon Steel)*	
Ultimate Strength:	F_u =	58 ksi		
Anchor Spacing	s =	3.00		
Anchor Diameter: [redacted]	d_b =	0.500 in		
Steel strength in tension:	N_{sa} =	*10.705 kip*		
Max distance from bolt to applied force:	b =	1.313 in		
Tributary length to each bolt:	*p = max 2b, but ≤ s*	2.63 in		
Adjusted prying distance:	$b' = \left[b - \frac{d_b}{2}\right] =$	1.06 in	(Eq. 9-21)	
Resistance factor for prying:	Ω_{pr} =	*1.67 ASD*		
	Φ_{pr} =	*0.90 LRFD*		
	Φ_t =	*0.75 LRFD*		
Maximum bolt force for prying:	$T = \frac{\Phi_{pr} * F_u * th_{bracket}^2 * p}{2*b} =$	7.34 kip	(Eq. 9-22a)	
Available tensile strength of single bolt:	$B = \Phi_t * N_{sa} =$	*8.03 kip*	*(per ESR-1917)*	

$$t_c = \sqrt{\frac{4*B*b'}{\Phi_{LRFD} * p * F_u}} = 0.547 \text{ in} \qquad \text{(Eq. 9-30a)}$$

T_{avail} = BQ = *6.66 kip* (Eq. 9-31)

Q = *0.712* *a'<, Q = 1*

0 ≤ a' ≤ 1, Q = $\left(\frac{t}{t_c}\right)^2 (1+\delta*\alpha') =$ *0.712* (Eq. 9-33)

a'>1, Q = $\left(\frac{t}{t_c}\right)^2 (1+\delta) =$ *0.829* (Eq. 34)

Width of hole along length of plate:	d' =	*0.625 in*
Distance from bolt centerline to edge of plate:	a =	*1.50 in*

Additional variables for prying calculation:

$\delta = 1 - \frac{d'}{p} =$ *0.76* *Eq. (9-24)*

$a' = a + \frac{d_b}{2} =$ *1.75 in* $\le \left(1.25*b*\frac{d_b}{2}\right) =$ *1.89 in* *Eq. (9-27)*

$\rho = \frac{b'}{a'} =$ *0.61* *Eq. (9-26)*

$\beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) =$ *0.34 in* *Eq. (9-25)*

$\alpha' = if\left[\beta => 1, 1, \min\left[1, \frac{1}{\delta} * \left(\frac{\beta}{1-\beta}\right)\right]\right] =$ *0.67*

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

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr} * p * F_u * (1+\delta*\alpha')}} =$$ *0.37 in LRFD* *(Eq. 9-23a)*

Thickness of Bracket is 0.375 in - therefore, Prying OK

Piece #63388A - L6" x 3" x 3/8" x 0'-8" Steel Angle Bracket (A36 min)
w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.5625" 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:	$th_{bracket}$ = 0.375 in		
Yield Strength:	F_y = 36 ksi		*(Table 2-4)*
Ultimate Strength:	F_u = 58 ksi		
Gross Area:	A_g = **(8.0in)*(0.375in)** = 3.00 in2		
	$\phi_{t\text{-}y}$ = 0.9	LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow\text{-}yielding} = \phi_{t\text{-}y} * F_y * A_g$ = 97.20 kip		(Eqn D2-1)
Shear Lag Factor:	U = 1.0		(Table D3.1)
Net Area:	A_n = (8.0in)*(0.375in)-(5*0.3125in)*(0.375in) = 2.41 in2		(Sec B4.3)
Effective Net Area:	A_e = A_n*U =	2.41 in2	(Eqn D3-1)
	ϕ_{tr} = 0.75	LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow\text{-}rupture} = \phi_{t\text{-}r} * F_u * A_e$ = 105.01 kip		(Eqn D2-2)
	$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) =	97.20 kip LRFD	(Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area:	A_{gv} =(8.0in)*(0.25in) = 3.00 in2		
	Φ_{yv} = 1.00	LRFD	*(Eqn J4-3)*
Shear Yielding:	$V_{allow\text{-}yielding} = \phi_{yv} * 0.60 * F_y * A_{gv}$ =	64.80 kip	*(Eqn J4-3)*
	Φ_{rv} = 0.75	LRFD	*(Eqn J4-4)*
Net Area:	A_{nv} = (8.0in)*(0.375in)-(5*0.3125in)*(0.375in) =	2.41 in2	*(Sec B4.3)*
Shear Rupture:	$V_{allow\text{-}rupture} = \phi_{rv} * 0.60 * F_u * A_{nv}$ =	84.01 kip	*(Eqn J4-4)*
	V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) =	64.80 kip LRFD	*(Horizontal Component)*

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

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{8*0.375^2}{4} =$ 0.2813 in3

ϕ_b = 0.9 LRFD *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = 9.113 kip-in

Moment Arm: $Moment_{arm}$ = 1.313 in *(Eqn F11-1)*

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 6.943 kip LRFD *(Vertical Component)*

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{8*0.375^2}{4} =$ 0.2813 in3

ϕ_b = 0.9 LRFD *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = 9.113 kip-in *(Eqn F11-1)*

Moment Arm: $Moment_{arm}$ = 1.313 in

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 6.943 kip LRFD *(Horizontal Component)*

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} =$ 0.1406 in3

ϕ_b = 0.9 LRFD *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = 0.000 kip-in *(Eqn F11-1)*

Moment Arm: $Moment_{arm}$ = 1.313 in

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ #DIV/0! *(Horizontal Component)*

Piece #63388A - L6" x 3" x 3/8" x 0'-8" Steel Angle Bracket (A36 min)
w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.5625" 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 (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 5 $V_{allow}*N$	$T_{allow}*N$
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 (Common Bolts), bearing type connection*

Nominal Tensile Strength: F_t = 45 ksi

Bracket Thickness: $th_{bracket}$ = 0.375 in

Nominal Shear Strength, Threads Excluded: F_v = 27 ksi (Table J3.2 AISC 14th)

Bolt Diameter: d_{bcd} = 0.50 in

Bolt Area: $A_{bcd} = 0.25*\pi*d_{bcd}^2$ = 0.20 in2

Resistance factor for bolt tension or shear: ϕ= 0.75 LRFD

Shear Capacity of single bearing type bolt: $V_{allow\text{-}bolt} = \phi*F_v*A_{bcd}$ = 3.98 kip

Tension Capacity of single bearing type bolt: $T_{allow\text{-}bolt} = \phi*F_t*A_{bcd}$ = 6.63 kip

Bolt bearing strength at bracket connection: (Section J3.10) F_u = 58 ksi

Bolt edge distance: edge-dist = 1.00 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole and edge of adjacent hole or edge of plate: L_c = edge-dist -0.5*bh = 0.72 in2

Single end bolt bearing capacity: $Bolt_{br'g} = \min[(1.5*L_c*th_{plate}*F_u),(3.0*d_{bcd}*th_{plate}*F_u)]$ = 23.45 kip

$3*Bolt_{allow\text{-}bolt} = 3*\phi*Bolt_{bearing}$ 52.76 kip LRFD

Prying of Piece #63384A - L6" x 3" x 3/8" x 0'-8" Steel Angle Bracket (A36 min)
w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.5625" 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)

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on Concrete over Metal Deck

$\phi N_n =$ 750 lbf

$\phi V_n =$ 1240 lbf

1/2" dia Hilti Kwik Bolt-TZ Expansion Anchors (ESR-1917) w/ 2" Embedment on 4' min Concrete Slab

$\phi N_n =$ 1400 lbf

$\phi V_n =$ 3150 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, which allows the conversion of the allowable loads to ASD, requiring input which is specific to the racking system, the allowable load fore this Piece is given in LRFD only.

Overall Capacity of Seismic Load - Prying of Piece #63384A - L6" x 3" x 3/8" x 0'-8" Steel Angle Bracket (A36 min)
w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	**750 lbf**	**Tension**	**Vertical & Horizontal Allowable Load - (LRFD)**
$Load_{allowable\text{-}total\text{-}on\text{-}concrete\text{-}ove\text{-}metal\text{-}deck}$ =	**1240 lbf**	**Shear**	
$Load_{allowable\text{-}total\text{-}on\text{-}4"\ min\text{-}concrete\text{-}slab}$ =	**1400 lbf**	**Tension**	
$Load_{allowable\text{-}total\text{-}on\text{-}4"\ min\text{-}concrete\text{-}slab}$ =	**3150 lbf**	**Shear**	

Piece #36384A - L3" x 6" x 3/8" x 0'-4" Steel Angle Bracket (A36 min) w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (3) 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: $th_{bracket}$ = 0.375 in

Yield Strength: F_y = 36 ksi (Table 2-4)

Ultimate Strength: F_u = 58 ksi

Gross Area: A_g = **(4.0in)*(0.375in)** = 1.50 in2

$\phi_{t\text{-}y}$ = 0.9 LRFD (Eqn D2.1)

Tensile Yielding: $T_{allow\text{-}yielding} = \phi_{t\text{-}y} * F_y * A_g$ = 48.60 kip (Eqn D2-1)

Shear Lag Factor: U = 1.0 (Table D3.1)

Net Area: A_n = (4.0in)*(0.375in)-(3*0.3125in)*(0.375in) = 1.15 in2 (Sec B4.3)

Effective Net Area: A_e = A_n*U = 1.15 in2 (Eqn D3-1)

ϕ_{tr} = 0.75 LRFD (Eqn D2-2)

Tensile Rupture: $T_{allow\text{-}rupture} = \phi_{t\text{-}r} * F_u * A_e$ = 49.96 kip (Eqn D2-2)

$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) = 48.60 kip

$T_{allow\text{-}bracket\text{-}lbf}$ = T_{allow}*1000= 48,600 lbf (Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area: A_{gv} =(4.0in)*(0.375in) = 1.50 in2

Φ_{yv} = 1.00 LRFD (Eqn J4-3)

Shear Yielding: $V_{allow\text{-}yielding} = \phi_{yv} * 0.60 * F_y * A_{gv}$ = 32.40 kip (Eqn J4-3)

Net Area: A_{nv} = (4.0in)*(0.375in)-(3*0.3125in)*(0.375in) = 1.15 in2 (Sec B4.3)

Φ_{rv} = 0.75 LRFD (Eqn J4-4)

Shear Rupture: $V_{allow\text{-}rupture} = \phi_{rv} * 0.60 * F_u * A_{nv}$ = 39.97 kip (Eqn J4-4)

V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) = 32.40 kip LRFD (Horizontal Component)

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} =$ 0.1406 in3

ϕ_b = 0.9 LRFD (Sec F1)

$M_{allow} = \phi * F_y * Z$ = 4.556 kip-in

Moment Arm: $Moment_{arm}$ = 4.813 in (Eqn F11-1)

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 0.947 kip LRFD (Horizontal Component)

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} =$ 0.1406 in3

ϕ_b = 0.9 LRFD (Sec F1)

$M_{allow} = \phi * F_y * Z$ = 4.556 kip-in

Moment Arm: $Moment_{arm}$ = 1.313 in (Eqn F11-1)

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 3.471 kip LRFD (Vertical Component)

Piece #36384A - L3" x 6" x 3/8" x 0'-4" Steel Angle Bracket (A36 min) w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816

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

Vertical Leg:

Design Thickness (in)	V_{allow}	*Design Thickness (in)*	T_{allow}	Number of Screws *N = 3* V_{allow} *N	T_{allow} *N
0.048-0.048	*603 lbf*	*0.048*	*210 lbf*	***1809 lbf***	***630 lbf***
0.048-0.075	*1001 lbf*	*0.06*	*331 lbf*	***3003 lbf***	***993 lbf***
0.060-0.060	*833 lbf*	*0.075*	*409 lbf*	***2499 lbf***	***1227 lbf***
0.075-0.078	*1058 lbf*	*0.105*	*548 lbf*	***3174 lbf***	***1644 lbf***
1/8" - 3/16"	*1021 lbf*	*1/8"*	*897 lbf*	***3063 lbf***	*2691 lbf*
		3/16"	*1439 lbf*		*4317 lbf*

Horizontal Leg:

Design Thickness (in)	V_{allow}	*Design Thickness (in)*	T_{allow}	Number of Screws *N = 3* V_{allow} *N	T_{allow} *N
0.048-0.048	*603 lbf*	*0.048*	*210 lbf*	***1809 lbf***	***630 lbf***
0.048-0.075	*1001 lbf*	*0.06*	*331 lbf*	***3003 lbf***	***993 lbf***
0.060-0.060	*833 lbf*	*0.075*	*409 lbf*	***2499 lbf***	***1227 lbf***
0.075-0.078	*1058 lbf*	*0.105*	*548 lbf*	***3174 lbf***	***1644 lbf***
1/8" - 3/16"	*1021 lbf*	*1/8"*	*897 lbf*	***3063 lbf***	*2691 lbf*
		3/16"	*1439 lbf*		*4317 lbf*

Piece #36384B - L3" x 8" x 3/8" x 0'-4" Steel Angle Bracket (A36 min) w/ (4) 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:	$th_{bracket}$ = 0.375 in		
Yield Strength:	F_y = *36 ksi*		*(Table 2-4)*
Ultimate Strength:	F_u = *58 ksi*		
Gross Area:	A_g = ***(4.0in)*(0.375in)*** = *1.50 in2*		
	$\phi_{t\text{-}y}$ = *0.9*	*LRFD*	(Eqn D2.1)
Tensile Yielding:	$T_{allow\text{-}yielding} = \phi_{t\text{-}y} * F_y * A_g$ = 48.60 kip		(Eqn D2-1)
Shear Lag Factor:	U = *1.0*		(Table D3.1)
Net Area:	A_n = *(4.0in)*(0.375in)-(2*0.3125in)*(0.375in) = 1.27 in2*		(Sec B4.3)
Effective Net Area:	$A_e = A_n * U$ = *1.27 in2*		(Eqn D3-1)
	ϕ_{tr} = *0.75*	*LRFD*	(Eqn D2-2)
Tensile Rupture:	$T_{allow\text{-}rupture} = \phi_{t\text{-}r} * F_u * A_e$ = 55.05 kip		(Eqn D2-2)
	$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) =	48.60 kip LRFD	(Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area:	A_{gv} *=(4.0in)*(0.375in) = 1.50 in2*		
	Φ_{yv} = *1.00*	*LRFD*	*(Eqn J4-3)*
Shear Yielding:	$V_{allow\text{-}yielding} = \phi_{yv} * 0.60 * F_y * A_{gv}$ =	*32.40 kip*	*(Eqn J4-3)*
Net Area:	A_{nv} *= (4.0in)*(0.375in)-(2*0.3125in)*(0.375in) = 1.27 in2*		*(Sec B4.3)*
	Φ_{rv} = *0.75*	*LRFD*	*(Eqn J4-4)*
Shear Rupture:	$V_{allow\text{-}rupture} = \phi_{rv} * 0.60 * F_u * A_{nv}$ =	*44.04 kip*	*(Eqn J4-4)*
	V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) =	32.40 kip LRFD	*(Horizontal Component)*

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} =$ *0.1406 in3*

ϕ_b = *0.9* *LRFD* *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = *4.556 kip-in*

Moment Arm: $Moment_{arm}$ = *4.313 in* *(Eqn F11-1)*

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ *1.057 kip LRFD* *(Horizontal Component)*

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} =$ *0.1406 in3*

ϕ_b = *0.9* *LRFD* *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = *4.556 kip-in*

Moment Arm: $Moment_{arm}$ = *1.313 in* *(Eqn F11-1)*

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ *3.471 kip LRFD* *(Vertical Component)*

Piece #36384B - L3" x 6" x 3/8" x 0'-4" Steel Angle Bracket (A36 min) w/ (4) 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

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

Vertical Leg:

Design Thickness (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 4 V_{allow}*N	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

Horizontal Leg:

Design Thickness (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 4 V_{allow}*N	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

Piece #36388A - L3" x 6" x 3/8" x 0'-8" Steel Angle Bracket (A36 min) w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (5) 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: $th_{bracket}$ = 0.375 in

Yield Strength: F_y = 36 ksi *(Table 2-4)*

Ultimate Strength: F_u = 58 ksi

Gross Area: A_g = **(8.0in)*(0.375in)** = 3.00 in2

$\phi_{t\text{-}y}$ = 0.9 LRFD (Eqn D2.1)

Tensile Yielding: $T_{allow\text{-}yielding} = \phi_{t\text{-}y} * F_y * A_g$ = 97.20 kip (Eqn D2-1)

Shear Lag Factor: $U =$ 1.0 (Table D3.1)

Net Area: A_n = (8.0in)*(0.375in)-(5*0.3125in)*(0.375in) = 2.41 in2 (Sec B4.3)

Effective Net Area: A_e = $A_n * U$ = 2.41 in2 (Eqn D3-1)

ϕ_{tr} = 0.75 LRFD (Eqn D2-2)

Tensile Rupture: $T_{allow\text{-}rupture} = \phi_{t\text{-}r} * F_u * A_e$ = 105.01 kip (Eqn D2-2)

$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) = 97.20 kip LRFD (Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area: A_{gv} =(8.0in)*(0.375in) = 3.00 in2

Φ_{yv} = 1.00 LRFD *(Eqn J4-3)*

Shear Yielding: $V_{allow\text{-}yielding} = \phi_{yv} * 0.60 * F_y * A_{gv}$ = 64.80 kip *(Eqn J4-3)*

Net Area: A_{nv} = (8.0in)*(0.375in)-(5*0.3125in)*(0.375in) = 2.41 in2 *(Sec B4.3)*

Φ_{rv} = 0.75 LRFD *(Eqn J4-4)*

Shear Rupture: $V_{allow\text{-}rupture} = \phi_{rv} * 0.60 * F_u * A_{nv}$ = 84.01 kip *(Eqn J4-4)*

V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) = 64.80 kip *(Horizontal Component)*

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \dfrac{bd^2}{4} = \dfrac{(8)*0.375^2}{4} =$ 0.2813 in3

ϕ_b = 0.9 LRFD *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = 9.113 kip-in

Moment Arm: $Moment_{arm}$ = 4.813 in *(Eqn F11-1)*

Allowable Load: $P_{allow} = \dfrac{M_{allow}}{Moment_{arm}} =$ 1.894 kip LRFD *(Horizontal Component)*

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \dfrac{bd^2}{4} = \dfrac{(8)*0.375^2}{4} =$ 0.2813 in3

ϕ_b = 0.9 LRFD *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = 9.113 kip-in

Moment Arm: $Moment_{arm}$ = 1.313 in *(Eqn F11-1)*

Allowable Load: $P_{allow} = \dfrac{M_{allow}}{Moment_{arm}} =$ 6.943 kip LRFD *(Vertical Component)*

<u>Piece #36388A - L3" x 6" x 3/8" x 0'-8" Steel Angle Bracket (A36 min) w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816</u>

<u>Screws from Angle Bracket to Steel Sheet:</u>

<u>Hilti Kwik Flex #EAF-816 & #EAF-846 : (Screw Type 6, ASD 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)</u>

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)

Vertical Leg:

Design Thickness (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 5 V_{allow}*N	T_{allow}*N
0.048-0.048	377 lbf	0.048	131 lbf	**1885 lbf**	**655 lbf**
0.048-0.075	626 lbf	0.06	207 lbf	**3130 lbf**	**1035 lbf**
0.060-0.060	520 lbf	0.075	255 lbf	**2600 lbf**	**1275 lbf**
0.075-0.078	661 lbf	0.105	342 lbf	**3305 lbf**	**1710 lbf**
1/8" - 3/16"	638 lbf	1/8"	561 lbf	**3190 lbf**	2805 lbf
		3/16"	899 lbf		4495 lbf

Horizontal Leg:

Design Thickness (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 5 V_{allow}*N	T_{allow}*N
0.048-0.048	377 lbf	0.048	131 lbf	**1885 lbf**	**655 lbf**
0.048-0.075	626 lbf	0.06	207 lbf	**3130 lbf**	**1035 lbf**
0.060-0.060	520 lbf	0.075	255 lbf	**2600 lbf**	**1275 lbf**
0.075-0.078	661 lbf	0.105	342 lbf	**3305 lbf**	**1710 lbf**
1/8" - 3/16"	638 lbf	1/8"	561 lbf	**3190 lbf**	2805 lbf
		3/16"	899 lbf		4495 lbf

Piece #38384A - L3" x 8" x 3/8" x 0'-4" Steel Angle Bracket (A36 min) w/ (3) 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:	$th_{bracket}$ = 0.375 in		
Yield Strength:	F_y = *36 ksi*		*(Table 2-4)*
Ultimate Strength:	F_u = *58 ksi*		
Gross Area:	A_g = ***(4.0in)*(0.375in)*** = *1.50 in2*		
	$\phi_{t\text{-}y}$ = *0.9*	*LRFD*	(Eqn D2.1)
Tensile Yielding:	$T_{allow\text{-}yielding} = \phi_{t\text{-}y} * F_y * A_g$ = 48.60 kip		(Eqn D2-1)
Shear Lag Factor:	$U =$ *1.0*		(Table D3.1)
Net Area:	A_n = *(4.0in)*(0.375in)-(3*0.3125in)*(0.375in) = 1.15 in2*		(Sec B4.3)
Effective Net Area:	$A_e = A_n * U =$	*1.15 in2*	(Eqn D3-1)
	ϕ_{tr} = *0.75*	*LRFD*	(Eqn D2-2)
Tensile Rupture:	$T_{allow\text{-}rupture} = \phi_{t\text{-}r} * F_u * A_e$ = 49.96 kip		(Eqn D2-2)
	$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) =	48.60 kip LRFD	(Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area:	A_{gv} *=(4.0in)*(0.375in) = 1.50 in2*		
	Φ_{yv} = *1.00*	*LRFD*	*(Eqn J4-3)*
Shear Yielding:	$V_{allow\text{-}yielding} = \phi_{yv} * 0.60 * F_y * A_{gv}$ = *32.40 kip*		*(Eqn J4-3)*
Net Area:	A_{nv} *= (4.0in)*(0.375in)-(3*0.3125in)*(0.375in) = 1.15 in2*		*(Sec B4.3)*
	Φ_{rv} = *0.75*	*LRFD*	*(Eqn J4-4)*
Shear Rupture:	$V_{allow\text{-}rupture} = \phi_{rv} * 0.60 * F_u * A_{nv}$ = *39.97 kip*		*(Eqn J4-4)*
	V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) = 32.40 kip LRFD		*(Horizontal Component)*

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4 * 0.375^2}{4} =$ *0.1313 in3*

ϕ_b = *0.9* *LRFD* *(Sec F1)*

$M_{allow} = \phi * F_y * Z =$ *4.254 kip-in*

Moment Arm: $Moment_{arm}$ = *6.313 in* *(Eqn F11-1)*

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ *0.674 kip LRFD* *(Horizontal Component)*

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{4 * 0.375^2}{4} =$ *0.1406 in3*

ϕ_b = *0.9* *LRFD* *(Sec F1)*

$M_{allow} = \phi * F_y * Z =$ *4.556 kip-in*

Moment Arm: $Moment_{arm}$ = *1.313 in* *(Eqn F11-1)*

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ *3.471 kip LRFD* *(Vertical Component)*

Piece #38384A - L3" x 8" x 3/8" x 0'-4" Steel Angle Bracket (A36 min) w/ (3) 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

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

Vertical Leg:

Design Thickness (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 3 V_{allow}*N	T_{allow}*N
0.048-0.048	603 lbf	0.048	210 lbf	**1809 lbf**	**630 lbf**
0.048-0.075	1001 lbf	0.06	331 lbf	**3003 lbf**	**993 lbf**
0.060-0.060	833 lbf	0.075	409 lbf	**2499 lbf**	**1227 lbf**
0.075-0.078	1058 lbf	0.105	548 lbf	**3174 lbf**	**1644 lbf**
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	**3063 lbf**	2691 lbf
		3/16"	1439 lbf		4317 lbf

Horizontal Leg:

Design Thickness (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 4 V_{allow}*N	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

Piece #38384B - L3" x 8" x 3/8" x 0'-4" Steel Angle Bracket (A36 min) w/ (4) 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: $th_{bracket}$ = 0.375 in

Yield Strength: F_y = 36 ksi (Table 2-4)

Ultimate Strength: F_u = 58 ksi

Gross Area: A_g = **(4.0in)*(0.375in)** = 1.50 in2

$\phi_{t\text{-}y}$ = 0.9 LRFD (Eqn D2.1)

Tensile Yielding: $T_{allow\text{-}yielding} = \phi_{t\text{-}y} * F_y * A_g$ = 48.60 kip (Eqn D2-1)

Shear Lag Factor: $U =$ 1.0 (Table D3.1)

Net Area: A_n = (4.0in)*(0.375in)-(2*0.3125in)*(0.375in) = 1.27 in2 (Sec B4.3)

Effective Net Area: A_e = $A_n * U$ = 1.27 in2 (Eqn D3-1)

ϕ_{tr} = 0.75 LRFD (Eqn D2-2)

Tensile Rupture: $T_{allow\text{-}rupture} = \phi_{t\text{-}r} * F_u * A_e$ = 55.05 kip (Eqn D2-2)

$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) = 48.60 kip LRFD (Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area: A_{gv} =(4.0in)*(0.375in) = 1.50 in2

1.50 (Eqn J4-3)

Shear Yielding: $V_{allow-yielding} = \dfrac{0.60 * F_y * A_{gv}}{\Omega_v} =$ 21.60 kip (Eqn J4-3)

Net Area: A_{nv} = (4.0in)*(0.375in)-(2*0.3125in)*(0.375in) = 1.27 in2 (Sec B4.3)

Φ_{rv} = 0.75 LRFD (Eqn J4-4)

Shear Rupture: $V_{allow\text{-}rupture} = \phi_{rv} * 0.60 * F_u * A_{nv}$ = 66.07 kip (Eqn J4-4)

V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) = 21.60 kip LRFD (Horizontal Component)

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \dfrac{bd^2}{4} = \dfrac{4 * 0.375^2}{4} =$ 0.1406 in3

ϕ_b = 0.9 LRFD (Sec F1)

$M_{allow} = \phi * F_y * Z$ = 4.556 kip-in

Moment Arm: $Moment_{arm}$ = 6.313 in (Eqn F11-1)

Allowable Load: $P_{allow} = \dfrac{M_{allow}}{Moment_{arm}} =$ 0.722 kip LRFD (Horizontal Component)

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \dfrac{bd^2}{4} = \dfrac{4 * 0.375^2}{4} =$ 0.1406 in3

ϕ_b = 0.9 LRFD (Sec F1)

$M_{allow} = \phi * F_y * Z$ = 4.556 kip-in

Moment Arm: $Moment_{arm}$ = 1.313 in (Eqn F11-1)

Allowable Load: $P_{allow} = \dfrac{M_{allow}}{Moment_{arm}} =$ 3.471 kip LRFD (Vertical Component)

Piece #38384B - L3" x 8" x 3/8" x 0'-4" Steel Angle Bracket (A36 min) w/ (4) 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

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

Vertical Leg:

Number of Screws *N* = 4

Design Thickness (in)	V_{allow}	*Design Thickness (in)*	T_{allow}	V_{allow} *N	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

Horizontal Leg:

Number of Screws *N* = 4

Design Thickness (in)	V_{allow}	*Design Thickness (in)*	T_{allow}	V_{allow} *N	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

Piece #38388A - L3" x 8" x 3/8" x 0'-8" Steel Angle Bracket (A36 min) w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (10) 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:	$th_{bracket}$ = 0.375 in		
Yield Strength:	F_y = 36 ksi		*(Table 2-4)*
Ultimate Strength:	F_u = 58 ksi		
Gross Area:	A_g = **(8.0in)*(0.375in)** = 3.00 in2		
	$\phi_{t\text{-}y}$ = 0.9	LRFD	(Eqn D2.1)
Tensile Yielding:	$T_{allow\text{-}yielding} = \phi_{t\text{-}y} * F_y * A_g$ = 97.20 kip		(Eqn D2-1)
Shear Lag Factor:	U = 1.0		(Table D3.1)
Net Area:	A_n = (8.0in)*(0.375in)-(5*0.3125in)*(0.375in) = 2.41 in2		(Sec B4.3)
Effective Net Area:	A_e = $A_n * U$ =	2.41 in2	(Eqn D3-1)
	ϕ_{tr} = 0.75	LRFD	(Eqn D2-2)
Tensile Rupture:	$T_{allow\text{-}rupture} = \phi_{t\text{-}r} * F_u * A_e$ = 105.01 kip		(Eqn D2-2)
	$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) =	97.20 kip LRFD	(Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area:	A_{gv} =(8.0in)*(0.375in) = 3.00 in2		
	Φ_{yv} = 1.00	LRFD	*(Eqn J4-3)*
Shear Yielding:	$V_{allow\text{-}yielding} = \phi_{yv} * 0.60 * F_y * A_{gv}$ = 64.80 kip		*(Eqn J4-3)*
Net Area:	A_{nv} = (8.0in)*(0.375in)-(5*0.3125in)*(0.375in) = 2.41 in2		*(Sec B4.3)*
	Φ_{rv} = 0.75	LRFD	*(Eqn J4-4)*
Shear Rupture:	$V_{allow\text{-}rupture} = \phi_{rv} * 0.60 * F_u * A_{nv}$ = 84.01 kip		*(Eqn J4-4)*
	V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) = 64.80 kip LRFD		*(Horizontal Component)*

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{8*0.375^2}{4} =$ 0.2813 in3

ϕ_b = 0.9 LRFD *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = 9.113 kip-in

Moment Arm: $Moment_{arm}$ = 6.313 in *(Eqn F11-1)*

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 1.444 kip LRFD *(Horizontal Component)*

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \frac{bd^2}{4} = \frac{8*0.375^2}{4} =$ 0.2813 in3

ϕ_b = 0.9 LRFD *(Sec F1)*

$M_{allow} = \phi * F_y * Z$ = 9.113 kip-in

Moment Arm: $Moment_{arm}$ = 1.313 in *(Eqn F11-1)*

Allowable Load: $P_{allow} = \frac{M_{allow}}{Moment_{arm}} =$ 6.943 kip LRFD *(Vertical Component)*

Piece #38388A - L3" x 8" x 3/8" x 0'-8" Steel Angle Bracket (A36 min) w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (10) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816

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

Vertical Leg:

Design Thickness (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 5 V_{allow}*N	T_{allow}*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

Horizontal Leg:

Design Thickness (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 10 V_{allow}*N	T_{allow}*N
0.048-0.048	603 lbf	0.048	210 lbf	**6030 lbf**	**2100 lbf**
0.048-0.075	1001 lbf	0.06	331 lbf	**10010 lbf**	**3310 lbf**
0.060-0.060	833 lbf	0.075	409 lbf	**8330 lbf**	**4090 lbf**
0.075-0.078	1058 lbf	0.105	548 lbf	**10580 lbf**	**5480 lbf**
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	**10210 lbf**	8970 lbf
		3/16"	1439 lbf		14390 lbf

Piece #310386A - L3" x 10" x 3/8" 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:	$th_{bracket}$ = 0.375 in		
Yield Strength:	F_y = *36 ksi*		*(Table 2-4)*
Ultimate Strength:	F_u = *58 ksi*		
Gross Area:	A_g = ***(6.0in)*(0.375in)*** = *2.25 in2*		
	$\phi_{t\text{-}y}$ = *0.9*	*LRFD*	(Eqn D2.1)
Tensile Yielding:	$T_{allow\text{-}yielding}$ = $\phi_{t\text{-}y}*F_y*A_g$ = 72.90 kip		(Eqn D2-1)
Shear Lag Factor:	$U =$ *1.0*		(Table D3.1)
Net Area:	A_n = *(6.0in)*(0.375in)-(4*0.3125in)*(0.375in)* = *1.78 in2*		(Sec B4.3)
Effective Net Area:	A_e = A_n*U =	*1.78 in2*	(Eqn D3-1)
	ϕ_{tr} = *0.75*	*LRFD*	(Eqn D2-2)
Tensile Rupture:	$T_{allow\text{-}rupture}$ = $\phi_{t\text{-}r}*F_u*A_e$ = 77.48 kip		(Eqn D2-2)
	$T_{allow\text{-}bracket}$ = min($T_{allow\text{-}yielding}$, $T_{allow\text{-}rupture}$) =	72.90 kip LRFD	(Vertical & Horizontal Component))

Shear on Bracket Vertical Leg:

Gross Area:	A_{gv} =*(6.0in)*(0.375in)* = *2.25 in2*		
	Φ_{yv} = *1.00*	*LRFD*	*(Eqn J4-3)*
Shear Yielding:	$V_{allow\text{-}yielding}$ = $\phi_{yv}*0.60*F_y*A_{gv}$ = *48.60 kip*		*(Eqn J4-3)*
Net Area:	A_{nv} = *(6.0in)*(0.25in)-(4*0.3125in)*(0.375in)* = *1.78 in2*		*(Sec B4.3)*
	Φ_{rv} = *0.75*	*LRFD*	*(Eqn J4-4)*
Shear Rupture:	$V_{allow\text{-}rupture}$ = $\phi_{rv}*0.60*F_u*A_{nv}$ = *61.99 kip*		
	V_{allow} = min($V_{allow\text{-}yielding}$, $V_{allow\text{-}rupture}$) =	48.60 kip LRFD	*(Horizontal Component)*

Bending Moment on Lower Leg (Side B):

Plastic Modulus: $Z = \dfrac{bd^2}{4} = \dfrac{6*0.375^2}{4} =$ *0.2109 in3*

ϕ_b = *0.9* *0.2109 in3* *(Sec F1)*

M_{allow} = $\phi*F_y*Z$ = *6.834 kip-in*

Moment Arm: $Moment_{arm}$ = *8.313 in* *(Eqn F11-1)*

Allowable Load: $P_{allow} = \dfrac{M_{allow}}{Moment_{arm}} =$ *1.216 kip LRFD* *(Horizontal Component)*

Bending Moment on Vertical Leg (Side A):

Plastic Modulus: $Z = \dfrac{bd^2}{4} = \dfrac{6*0.375^2}{4} =$ *0.2109 in3*

ϕ_b = *0.9* *LRFD* *(Sec F1)*

M_{allow} = $\phi*F_y*Z$ = *6.834 kip-in*

Moment Arm: $Moment_{arm}$ = *1.313 in* *(Eqn F11-1)*

Allowable Load: $P_{allow} = \dfrac{M_{allow}}{Moment_{arm}} =$ *5.207 kip LRFD* *(Vertical Component)*

Piece #310386A - L3" x 10" x 3/8" 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

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

Vertical Leg:

Design Thickness (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 4 V_{allow} *N	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

Horizontal Leg:

Design Thickness (in)	V_{allow}	Design Thickness (in)	T_{allow}	Number of Screws N = 8 V_{allow} *N	T_{allow} *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