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(Eq. 9-22a)

### 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

Max distance from bolt to applied force:

Resistance factor for prying:

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

 $F_{t} = 60 \text{ ksi}$ Nominal Tensile Strength: (Carbon Steel)

Ultimate Strength:  $F_{y} = 58 \text{ ksi}$ 

s = 0.00Anchor Spacing

 $d_b = 0.500 \text{ in}$ Anchor Diameter:

 $N_{sa} = 10.705 \, kip$ Steel strength in tension:

p = max 2b, Tributary length to each bolt: 2.63 in

but <u><</u> s

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

 $\Omega_{\rm pr}$  = 1.67 ASD  $\Phi_{pr}$  = 0.90 LRFD  $\Phi_t = 0.75 LRFD$ 

b = 1.313 in

 $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * P}{2 * b} = 7.34 \text{ kip}$ Maximum bolt force for prying:

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

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

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

Q = 0.712

 $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.829$   $a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.829$ 

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:

 $\delta = 1 - \frac{d'}{\rho} = 0.76 \qquad a' = a + \frac{d}{\frac{b}{\rho}} = 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_b}{2}\right) = 1.89 \text{ in}$   $Eq. (9-24) \qquad Eq. (9-27)$   $\rho = \frac{b'}{a'} = 0.61 \qquad \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = 0.66 \text{ in}$   $Eq. (9-26) \qquad Eq. (9-25)$ 

 $\alpha' = if \left[ \beta => 1, 1, \min \left[ 1, \frac{1}{\delta} * \left( \frac{\beta}{1 - \beta} \right) \right] \right] = 1.00$ 

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

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

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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"

### Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness:  $th_{bracket} = 0.3750 \text{ in}$ 

 $F_{v} = 36 \, \text{ksi}$ Yield Strenath: (Table 2-4)

F., = 58 ksi Ultimate Strength:

Gross Area:  $A_g = (3.0in)*(0.375in) = 1.13 in2$ 

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

Tensile Yielding:  $T_{allow-yielding} = \phi_{t-y} *F_y *A_g = 36.45 \text{ kip}$ (Eqn D2-1)

Shear Lag Factor: (Table D3.1)

 $A_n = (3.0in)^*(0.375in) - (2^*0.3125in)^*(0.375in) = 0.89 in2$ Net Area: (Sec B4.3)

 $A_e = A_n *U = 0.89 in 2$ Effective Net Area: (Eqn D3-1)

> $\phi_{tr} = 0.75$ LRFD (Eqn D2-2)

 $T_{allow-rupture} = \phi_{t-r} *F_u *A_e = 38.74 \text{ kip}$ Tensile Rupture: (Eqn D2-2)

> $T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 36.45 \text{ kip LRFD}$ (Vertical & Horizontal

Shear on Bracket Vertical Leg:

 $A_{gv} = (3.0in)*(0.375in) = 1.13 in2$ Gross Area:

> $\Phi_{vv} = 1.00$ LRFD (Egn J4-3)

 $V_{allow-yielding} = \phi_{yv} *0.60 *F_y *A_{gv} = 24.30 \text{ kip}$ Shear Yielding: (Eqn J4-3)

> $\Phi_{ni} = 0.75$ LRFD (Egn J4-4)

Net Area:  $A_{nv} = (3.0in)*(0.375in)-(2*0.3125in)*(0.375in) = 0.89 in2$ (Sec B4.3)

(Eqn J4-4)  $V_{allow-rupture} = \phi_{rv} *0.60 *F_{u} *A_{rv} = 30.99 kip$ Shear Rupture:

> V<sub>allow</sub> = min(V<sub>allow-yielding</sub>, V<sub>allow-rupture</sub>) = 24.30 kip LRFD (Horizontal . Component)

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

Plastic Modulus

$$Z = \frac{bd^2}{4} = -\frac{4*0.375^2}{4} = -0.2813 \ in 3$$
 
$$\phi_b = -0.9 \qquad \qquad LRFD \qquad \qquad \text{(Sec F1)}$$

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

Moment <sub>arm</sub> = 1.313 in Moment Arm: (Eqn F11-1)

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

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

 $Z = \frac{bd^2}{4} = \begin{array}{cc} 4*0.375^2 \\ \hline 4 \end{array} = \begin{array}{cc} \text{0.2813 in3} \end{array}$ Plastic Modulus:

$$\phi_b = 0.9$$
 LRFD (Sec F1)

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

Moment arm = 1.313 in Moment Arm:

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

<sup>\*\*</sup>Note: Bending of Bracket is Considered within the Prying Calculation

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Sheet:

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	$V_{\it allow}$	Design Thickness	$T_{allow}$	Number of Screws	N = 3
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *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:  $F_t = 45 \text{ ksi}$ 

Bracket Thickness:  $th_{bracket} = 0.3750 \text{ in}$ 

Nominal Shear Strength, Threads  $F_v = 27 \text{ ksi}$ 

Excluded: (Table J3.2 AISC 14th)

Bolt Diameter:  $d_{bcd} = 0.50 \text{ in}$ 

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

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

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

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

Bolt bearing strength at bracket connection: (Section J3.10)  $F_u = 58 \text{ ksi}$ 

Bolt edge distance: edge-dist = 1.50 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole  $L_c = \text{edge-dist } -0.5 \text{*bh} = 1.22 \text{ in } 2$ 

and edge of adjacent hole or edge of plate:

Single end bolt bearing capacity: Bolt  $_{brg}$  =min[(1.5\*L  $_{c}$  \*th  $_{plate}$  \*F  $_{u}$ ),(3.0\*d  $_{bcd}$  \*th  $_{plate}$  \*F  $_{u}$ )] = 32.63 kip

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

### 3/8" BRACKETS

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0.5625

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## 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)

 $\phi N_n = 370 \, lbf$ 

 $\phi V_n = 500 \text{ lbf}$ 

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 <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 370 lbf Tension

Load <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 500 lbf Shear Horizontal
Allowable Load -

Load <sub>allowable-total-on-4" min-concrete-slab</sub> = 1000 lbf Tension (LRFD)

Load <sub>allowable-total-on-4" min-concrete-slab</sub> = 1322 lbf Shear

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### Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 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:  $th_{bracket} = 0.375 \text{ in}$ 

Nominal Tensile Strength:  $F_{+} = 60 \text{ ksi}$  (Carbon Steel)

Ultimate Strength:  $F_{\mu} = 58 \text{ ksi}$ 

Anchor Spacing s = 4.00

Anchor Diameter:  $d_b = 0.500 \text{ in}$ 

Steel strength in tension:  $N_{sa} = 10.705 \text{ kip}$ 

•

Tributary length to each bolt: p = max 2b, 2.63 in

Max distance from bolt to applied force:

Resistance factor for prying:

but <u><</u> s

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

Adjusted prying distance:  $b = \left[b = \frac{1}{2}\right] = 1.06 \text{ in}$  (Eq. 9-21)

 $Ω_{pr}$  = 1.67 ASD  $Φ_{pr}$  = 0.90 LRFD

b = 1.313 in

 $\Phi_{\rm t} = 0.75 \, LRFD$ 

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

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

 $t_c = \sqrt{\frac{4*8*b'}{\Phi_{LRFD}*p*F_u}} = 0.547 \text{ in}$  (Eq. 9-30a)

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

Q = 0.829 a' < Q = 0.829

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

 $\delta = 1 - \frac{d'}{p} = 0.76 \qquad a' = a + \frac{d}{2} = 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_b}{2}\right) = 1.89 \text{ in}$   $Eq. (9-24) \qquad Eq. (9-27)$   $P = \frac{b'}{a'} = 0.61 \qquad \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = 0.34 \text{ in}$   $Eq. (9-26) \qquad Eq. (9-25)$   $\alpha' = if \left[\beta = > 1, 1, \min \left[1, \frac{1}{\delta} * \left(\frac{\beta}{1 - \beta}\right)\right]\right] = 0.67$ 

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

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

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## 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:  $F_{v} = 36 \, \text{ksi}$ (Table 2-4)

F., = 58 ksi Ultimate Strength:

 $A_g = (6.0in)*(0.375in) = 2.25 in2$ Gross Area:

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

 $T_{allow-yielding} = \phi_{t-y} *F_y *A_g = 72.90 \text{ kip}$ Tensile Yielding: (Eqn D2-1)

Shear Lag Factor: (Table D3.1)

 $A_n = (6.0in)^*(0.375in)-(4^*0.3125in)^*(0.375in) = 1.78 in2$ Net Area: (Sec B4.3)

 $A_e = A_n *U = 1.78 in 2$ Effective Net Area: (Eqn D3-1)

> $\phi_{tr} = 0.75$ LRFD (Eqn D2-2)

Tensile Rupture:  $T_{allow-rupture} = \phi_{t-r} *F_u *A_e = 77.48 \text{ kip}$ (Eqn D2-2)

> $T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 72.90 \text{ kip LRFD}$ (Vertical &

Horizontal Component))

## Shear on Bracket Vertical Leg:

 $A_{gv} = (6.0in)*(0.375in) = 1.50 in2$ Gross Area:

> $\Phi_{vv} = 1.00$ LRFD (Egn J4-3)

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

> $\Phi_{ni} = 0.75$ LRFD (Egn J4-4)

Net Area:  $A_{nv} = (6.0in)*(0.375in)-(4*0.3125in)*(0.375in) = 0.59 in2$ (Sec B4.3)

 $V_{allow-rupture} = \phi_{rv} *0.60*F_{u} *A_{nv} = 20.66 kip$ Shear Rupture:

> V<sub>allow</sub> = min(V<sub>allow-yielding</sub>, V<sub>allow-rupture</sub>) = 20.66 kip LRFD (Horizontal

. Component)

(Eqn F11-1)

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

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

Plastic Modulus:

$$Z = \frac{bd^2}{4} \, \frac{6*0.375^2}{4} = \, \frac{0.2109 \, \text{in3}}{4}$$
 
$$\phi_b = \, 0.9 \qquad \qquad \text{LRFD} \qquad \qquad \text{(Sec F1)}$$

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

Moment arm = 1.313 in Moment Arm:

 $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = \quad \text{5.207 kip LRFD} \quad \text{(Vertical Component)}$ 

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

Plastic Modulus: 
$$Z=\frac{bd^2}{4}= \quad \frac{6*0.375^2}{4}= \quad 0.2109 \text{ in 3}$$
 
$$\phi_b=0.9 \qquad \qquad \text{LRFD} \qquad \qquad \text{(Sec F1)}$$

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

Moment Arm: Moment arm = 1.313 in

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

<sup>\*\*</sup>Note: Bending of Bracket is Considered within the Prying Calculation

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Sheet: 2 (3)

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

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

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

Bolt type: A307 Gr. A (Common Bolts), bearing type connection

Nominal Tensile Strength:  $F_t = 45 \text{ ksi}$ 

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

Nominal Shear Strength, Threads  $F_v = 27 \text{ ksi}$ 

Excluded: (Table J3.2 AISC 14th)

Bolt Diameter:  $d_{bcd} = 0.50 \text{ in}$ 

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

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

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

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

Bolt bearing strength at bracket connection: (Section J3.10)  $F_u = 58 \text{ ksi}$ 

Bolt edge distance: edge-dist = 1.00 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole  $L_c = \text{edge-dist -0.5*bh} = 0.72 \text{ in } 2$ 

and edge of adjacent hole or edge of plate:

Single end bolt bearing capacity: Bolt  $_{brg}$  =min[(1.5\*L  $_{c}$  \*th  $_{plate}$  \*F  $_{u}$ ),(3.0\*d  $_{bcd}$  \*th  $_{plate}$  \*F  $_{u}$ )] = 23.45 kip

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

### 3/8" BRACKETS

Date: 5/7/2019

Job No.: 18-001

Sheet: 2 (4)

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

Date: 5/7/2019 Job No.: 18-001 Sheet: 8 (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).

### 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

Resistance factor for prying:

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

Nominal Tensile Strength:  $F_{+} = 60 \text{ ksi}$  (Carbon Steel)

Ultimate Strength:  $F_{\mu} = 58 \text{ ksi}$ 

Anchor Spacing s = 0.00

Anchor Diameter:  $d_b = 0.500 \text{ in}$ 

Steel strength in tension:  $N_{sa} = 10.705 \text{ kip}$ 

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

Tributary length to each bolt: p = max 2b, 2.63 in

but <u><</u> s

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

r -1

 $\Omega_{\rm pr}$  = 1.67 ASD  $\Phi_{\rm pr}$  = 0.90 LRFD

 $\Phi_{\rm t} = 0.75 \, LRFD$ 

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

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 8.03 \text{ kip}$  (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  $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.829$   $a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.829$ 

(Eq. 9-33) (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:

 $\delta = 1 - \frac{d'}{p} = 0.76 \qquad a' = a + \frac{d}{2} = 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_b}{2}\right) = 1.89 \text{ in}$   $Eq. (9-24) \qquad Eq. (9-27)$   $P = \frac{b'}{a'} = 0.61 \qquad \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = 0.34 \text{ in}$   $Eq. (9-26) \qquad Eq. (9-25)$   $\alpha' = if \left[\beta = > 1, 1, \min \left[1, \frac{1}{\delta} * \left(\frac{\beta}{1 - \beta}\right)\right]\right] = 0.67$ 

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

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

Sheet: 3 (2)

# 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:  $F_{v} = 36 \, \text{ksi}$ (Table 2-4)

F., = 58 ksi Ultimate Strength:

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

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

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

Shear Lag Factor: (Table D3.1)

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

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

> $\phi_{tr} = 0.75$ LRFD (Ean D2-2)

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

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

### Shear on Bracket Vertical Leg:

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

> $\Phi_{yv} = 1.00$ LRFD (Eqn J4-3)

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

> $\Phi_{rv} = 0.75$ LRFD (Eqn J4-4)

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

 $V_{allow-rupture} = \phi_{rv} *0.60 *F_{u} *A_{rv} = 39.97 kip$ Shear Rupture:

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

Component)

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

Plastic Modulus:

$$Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} = 0.1406 \ in 3$$
 
$$\phi_b = 0.9 \qquad \text{LRFD} \qquad \text{(Sec F1)}$$

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

Moment Arm: Moment arm = 2.813 in (Eqn F11-1)

 $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = \begin{array}{c} \text{1.620 kip LRFD} & \text{(Vertical Component)} \\ \end{array}$ Allowable Load:

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

 $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} = 0.1406 \text{ in 3}$ Plastic Modulus:  $\phi_b = 0.9$ LRFD (Sec F1)

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

Moment arm = 1.313 in Moment Arm:

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

<sup>\*\*</sup>Note: Bending of Bracket is Considered within the Prying Calculation

Date: 5/7/2019 Job No.: 18-001

lob No.: 18-001 Sheet: 3 (3)

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	$V_{allow}$	Design Thickness	$T_{allow}$	Number of Screws	N = 3
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *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:  $F_t = 45 \text{ ksi}$ 

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

Nominal Shear Strength, Threads  $F_v = 27 \text{ ksi}$  (Table J3.2 AISC 14th)

Excluded:

Bolt Diameter:  $d_{bcd} = 0.50 \text{ in}$ 

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

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

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

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

Bolt bearing strength at bracket connection: (Section J3.10)  $F_u = 58 \text{ ksi}$ 

Bolt edge distance: edge-dist = 1.50 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole  $L_c = \text{edge-dist } -0.5 \text{*bh} = 1.22 \text{ in } 2$ 

and edge of adjacent hole or edge of plate:

Single end bolt bearing capacity: Bolt  $_{brg}$  =min[(1.5\*L  $_{c}$  \*th  $_{plate}$  \*F  $_{u}$ ),(3.0\*d  $_{bcd}$  \*th  $_{plate}$  \*F  $_{u}$ )] = 32.63 kip

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

### 3/8" BRACKETS

Date: 5/7/2019

Job No.: 18-001

Sheet: 3 (4)

## 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

 $\phi N_n = 370 \, lbf$ 

 $\phi V_n = 500 \text{ lbf}$ 

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"

Load <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 370 lbf Tension

Load <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 500 lbf Shear Horizontal
Allowable Load -

Load <sub>allowable-total-on-4" min-concrete-slab</sub> = 1000 lbf Tension (LRFD)

Load allowable-total-on-4" min-concrete-slab = 1322 lbf Shear

5/7/2019 Date: 18-001 Job No · Sheet: 4 (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 #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"

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

Resistance factor for prying:

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

 $F_{t} = 60 \text{ ksi}$ Nominal Tensile Strength: (Carbon Steel)

Ultimate Strength:  $F_{y} = 58 \text{ ksi}$ 

s = 0.00Anchor Spacing

 $d_b = 0.500 \text{ in}$ Anchor Diameter:

 $N_{sa} = 10.705 \, kip$ Steel strength in tension:

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

p = max 2b, Tributary length to each bolt: 2.63 in

but <u><</u> s

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

 $\Omega_{\rm pr}$  = 1.67 ASD  $\Phi_{pr}$  = 0.90 LRFD

 $\Phi_{\rm t} = 0.75 \, LRFD$ 

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

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 8.03 \text{ kip}$ (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

 $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.829$   $a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.829$ 

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:

 $\delta = 1 - \frac{d'}{p} = 0.76 \qquad a' = a + \frac{d}{\frac{b}{2}} = 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_b}{2}\right) = 1.89 \text{ in}$   $Eq. (9-24) \qquad Eq. (9-27)$   $\rho = \frac{b'}{a'} = 0.61 \qquad \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = 0.34 \text{ in}$   $Eq. (9-26) \qquad Eq. (9-25)$ 

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

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

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

Date:

1/23/2020

#### 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 Fiex #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 <sub>bracket</sub> = 0.375 in

Yield Strength:  $F_y = 36 \text{ ksi}$  (Table 2-4)

Ultimate Strength:  $F_u = 58 \text{ ksi}$ 

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

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

Tensile Yielding:  $T_{\text{allow-yielding}} = \phi_{t-y} *F_y *A_g = 48.60 \text{ kip}$  (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:  $A_e = A_n * U = 1.27 \text{ in } 2$  (Eqn D3-1)

 $\phi_{tr} = 0.75 \qquad LRFD \qquad (Ean D2-2)$ 

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

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

## Shear on Bracket Vertical Leg:

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

 $\Phi_{yy} = 1.00$  LRFD (Eqn J4-3)

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

 $\Phi_{rr} = 0.75$  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:  $V_{allow-rupture} = \phi_{rv} *0.60 *F_{u} *A_{nv} = 44.04 \text{ kip}$ 

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

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

Plastic Modulus:

$$Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} = \frac{0.1406 \text{ in 3}}{4}$$
 
$$\phi_b = 0.9 \qquad LRFD \qquad \text{(Sec F1)}$$

(Eqn F11-1)

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

Moment Arm: Moment <sub>arm</sub> = 2.313 in

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = \quad ^{1.970~kip~LRFD} \quad \text{(Vertical Component)}$ 

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

Plastic Modulus:  $Z = \frac{bd^2}{4} = -\frac{4*0.375^2}{4} = 0.1406 \text{ in 3}$ 

$$\phi_b = 0.9 \qquad LRFD \qquad (Sec F1)$$

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

Moment Arm: Moment <sub>arm</sub> = 1.313 in

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = \phantom{-}^{3.471~kip~LRFD} \qquad \textit{(Horizontal Component)}$ 

<sup>\*\*</sup>Note: Bending of Bracket is Considered within the Prying Calculation

Date: 5/7/2019

Job No.: 18-001

Sheet: 4 (3)

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

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

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

Bolt type: A307 Gr. A (Common Bolts), bearing type connection

Nominal Tensile Strength:  $F_t = 45 \text{ ksi}$ 

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

Nominal Shear Strength, Threads  $F_v = 27 \text{ ksi}$  (Table J3.2 AISC 14th)

Excluded:

Bolt Diameter:  $d_{bcd} = 0.50 \text{ in}$ 

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

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

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

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

Bolt bearing strength at bracket connection: (Section J3.10)  $F_u = 58 \text{ ksi}$ 

Bolt edge distance: edge-dist = 1.50 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole  $L_c = \text{edge-dist } -0.5 \text{*bh} = 1.22 \text{ in } 2$ 

and edge of adjacent hole or edge of plate:

Single bearing capacity: Bolt  $_{brg}$  =min[(1.5\*L  $_{c}$  \*th  $_{plate}$  \*F  $_{u}$ ),(3.0\*d  $_{bcd}$  \*th  $_{plate}$  \*F  $_{u}$ )] = 32.63 kip

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

### 3/8" BRACKETS

Date: 5/7/2019

Job No.: 18-001

Sheet: 9 (4)

## 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

 $\phi N_n = 370 \, lbf$ 

 $\phi V_n = 500 \text{ lbf}$ 

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 <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 370 lbf Tension

Load <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 500 lbf Shear Horizontal
Allowable Load -

Load <sub>allowable-total-on-4" min-concrete-slab</sub> = 1000 lbf Tension (LRFD)

Load <sub>allowable-total-on-4" min-concrete-slab</sub> = 1322 lbf Shear

Date: 5/7/2019 Job No.: 18-001 Sheet: 5 (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 #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"

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

Max distance from bolt to applied force:

Resistance factor for prying:

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

Nominal Tensile Strength:  $F_{+} = 60 \text{ ksi}$  (Carbon Steel)

Ultimate Strength:  $F_{ii} = 58 \text{ ksi}$ 

Anchor Spacing s = 4.00

Anchor Diameter:  $d_b = 0.500 \text{ in}$ 

Steel strength in tension:  $N_{sa} = 10.705 \text{ kip}$ 

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Tributary length to each bolt: p = max 2b, 2.63 in

but <u><</u> s

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

r -1

 $\Omega_{\rm pr}$  = 1.67 ASD  $\Phi_{\rm pr}$  = 0.90 LRFD

b = 1.313 in

 $\Phi_{\rm t} = 0.75 \, LRFD$ 

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

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 8.03 \text{ kip}$  (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  $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.829$   $a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.829$ (Fig. 9.33)

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:

$$\begin{split} \delta &= 1 - \frac{d^{+}}{p} = \quad 0.76 & a^{+} = a + \frac{d_{+}}{2} = \quad 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_{b}}{2}\right) = \quad 1.89 \text{ in} \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \mathcal{P} &= \frac{b^{+}}{\alpha^{+}} = \quad 0.61 & \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = \quad 0.34 \text{ in} \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ & \alpha^{+} = if \left[\beta = > 1, 1, \min \left[1, \frac{1}{\delta} * \left(\frac{B}{1 - \beta}\right)\right]\right] = \quad 0.67 \end{split}$$

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

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

1/23/2020 Date: 18-001 Job No.: 5 (2)

## 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 Fiex #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:  $th_{bracket} = 0.375 \text{ in}$ 

Yield Strenath:  $F_v = 36 \, \text{ksi}$ (Table 2-4)

 $F_u = 58 \, \text{ksi}$ Ultimate Strength:

 $A_g = (6.0in)*(0.375in) = 2.25 in2$ Gross Area:

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

 $T_{allow-yielding} = \phi_{t-y} *F_y *A_g = 72.90 \text{ kip}$ Tensile Yielding: (Eqn D2-1)

U = 10 Shear Lag Factor: (Table D3.1)

Net Area:  $A_n = (6.0in)^*(0.375in)-(4^*0.3125in)^*(0.375in) = 1.78 in2$ (Sec B4.3)

 $A_e = A_n * U = 1.78 in 2$ Effective Net Area: (Eqn D3-1)

> $\phi_{tr} = 0.75$ LRFD (Ean D2-2)

Tensile Rupture:  $T_{allow-rupture} = \phi_{t-r} * F_u * A_e = 77.48 \text{ kip}$ (Eqn D2-2)

> $T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 72.90 \text{ kip LRFD}$ (Vertical & Horizontal Component))

## Shear on Bracket Vertical Leg:

 $A_{gv} = (6.0in)*(0.375in) = 2.25 in2$ Gross Area:

> $\Phi_{vv} = 1.00$ LRFD (Eqn J4-3)

Shear Yielding:  $V_{allow-yielding} = \phi_{yv} *0.60 *F_y *A_{gv} = 48.60 kip$ (Eqn J4-3)

> $\Phi_{rv} = 0.75$ LRFD (Eqn J4-4)

 $A_{nv} = (6.0in)*(0.375in)-(4*0.3125in)*(0.375in) = 1.78 in2$ Net Area: (Sec B4.3)

(Eqn J4-4) Shear Rupture:  $V_{allow-rupture} = \phi_{rv} *0.60 *F_{u} *A_{rv} = 61.99 kip$ 

> $V_{allow} = min(V_{allow-yielding}, V_{allow-rupture}) = 48.60 \text{ kip LRFD}$ (Horizontal

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

Plastic Modulus: 
$$Z = \frac{bd^2}{4} = -\frac{6*0.375^2}{4} = 0.2109 \text{ in 3}$$
 
$$\phi_b = 0.9 \qquad \text{LRFD}$$

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

Moment arm = 2.813 in Moment Arm: (Eqn F11-1)

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

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

 $Z = \frac{bd^2}{4} = \frac{6*0.375^2}{4} = 0.2109 \text{ in 3}$ Plastic Modulus:  $\phi_b = 0.9$ LRFD (Sec F1)

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

Moment <sub>arm</sub> = 1.313 in Moment Arm

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

<sup>\*\*</sup>Note: Bending of Bracket is Considered within the Prying Calculation

Date: 5/7/2019 Job No.: 18-001 Sheet: 5 (3)

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

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

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

Bolt type: A307 Gr. A (Common Bolts), bearing type connection

Nominal Tensile Strength:  $F_t = 45 \text{ ksi}$ 

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

Nominal Shear Strength, Threads  $F_v = 27 \text{ ksi}$ 

Excluded: (Table J3.2 AISC 14th)

Bolt Diameter:  $d_{bot} = 0.50 \text{ in}$ 

Bolt Diameter.  $u_{bcd} = 0.50 \, \text{II}$ 

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

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

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

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

Bolt bearing strength at bracket connection: (Section J3.10)  $F_u = 58 \text{ ksi}$ 

Bolt edge distance: edge-dist = 1.00 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole  $L_c = \text{edge-dist -0.5*bh} = 0.72 \text{ in } 2$ 

and edge of adjacent hole or edge of plate:

Single end bolt bearing capacity: Bolt  $_{brg}$  =min[(1.5\*L  $_{c}$  \*th  $_{plate}$  \*F  $_{u}$ ),(3.0\*d  $_{bcd}$  \*th  $_{plate}$  \*F  $_{u}$ )] = 23.45 kip

2\*Bolt <sub>allow-bolt</sub> = 2\*φ \*Bolt <sub>bearing</sub> 35.17 kip LRFD

### 3/8" BRACKETS

Date: 5/7/2019

Job No.: 18-001

Sheet: 10 (4)

## 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 <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 670 lbf	Tension	
Load <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 1100 lbf	Shear	Vertical & Horizontal Allowable Load -
Load allowable-total-on-4" min-concrete-slab = 1275 lbf	Tension	(LRFD)
Load allowable-total-on-4" min-concrete-slab = 2552 lbf	Shear	

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 #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"

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

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

Nominal Tensile Strength:  $F_{+} = 60 \text{ ksi}$  (Carbon Steel)

Ultimate Strength:  $F_{\mu} = 58 \text{ ksi}$ 

Anchor Spacing s = 3.00

Anchor Diameter:  $d_b = 0.500 \text{ in}$ 

Steel strength in tension:  $N_{sa} = 10.705 \text{ kip}$ 

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

Tributary length to each bolt: p = max 2b, 2.63 in

but <u><</u> s

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

Resistance factor for prying:  $\Omega_{\rm pr}$  = 1.67 ASD

 $\Phi_{pr}$  = 0.90 LRFD  $\Phi_{t}$  = 0.75 LRFD

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

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 8.03 \text{ kip}$  (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  $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.829 a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.829$ 

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:

$$\begin{split} \delta &= 1 - \frac{d^{+}}{p} = \quad 0.76 & a^{+} = a + \frac{d_{+}}{2} = \quad 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_{b}}{2}\right) = \quad 1.89 \text{ in} \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \mathcal{P} &= \frac{b^{+}}{\alpha^{+}} = \quad 0.61 & \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = \quad 0.34 \text{ in} \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ & \alpha^{+} = if \left[\beta = > 1, 1, \min \left[1, \frac{1}{\delta} * \left(\frac{B}{1 - \beta}\right)\right]\right] = \quad 0.67 \end{split}$$

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

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

ob No.: 18-00 Sheet: 6 (2)

1/23/2020

#### 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 Fiex #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 <sub>bracket</sub> = 0.375 in

Yield Strength:  $F_y = 36 \text{ ksi}$  (Table 2-4)

Ultimate Strength:  $F_u = 58 \text{ ksi}$ 

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

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

Tensile Yielding:  $T_{\text{allow-yielding}} = \phi_{t\cdot y} * F_y * A_g = 97.20 \text{ kip}$  (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:  $A_e = A_n * U = 2.41 \text{ in } 2$  (Eqn D3-1)

 $\phi_{tr} = 0.75$  LRFD (Eqn D2-2)

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

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

## Shear on Bracket Vertical Leg:

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

 $\Phi_{_{YV}}$  = 1.00 LRFD (Eqn J4-3)

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

 $\Phi_{\text{rv}} = 0.75$  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:  $V_{allow-rupture} = \phi_{rv} *0.60*F_u *A_{nv} = 84.01 \text{ kip}$ 

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

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

Plastic Modulus:  $Z = \frac{bd^2}{4} : \frac{8*0.375^2}{4} = 0.2813 \text{ in 3}$ 

4  $\phi_b = 0.9 \qquad LRFD \qquad (Sec F1)$ 

 $M_{allow} = \phi *F_{y} *Z = 9.113 \, kip-in$ 

Moment Arm: Moment <sub>arm</sub> = 2.813 in (Eqn F11-1)

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

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

Plastic Modulus:  $Z = \frac{bd^2}{4} = \frac{8*0.375^2}{4} = 0.2813 \text{ in 3}$   $\phi_b = 0.9$  LRFD

(Sec F1)

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

Moment Arm: Moment <sub>arm</sub> = 1.313 in

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

<sup>\*\*</sup>Note: Bending of Bracket is Considered within the Prying Calculation

Date: 5/7/2019

Job No.: 18-001 Sheet: 6 (3)

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

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

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

Bolt type: A307 Gr. A (Common Bolts), bearing type connection

Nominal Tensile Strength:  $F_t = 45 \text{ ksi}$ 

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

Nominal Shear Strength, Threads  $F_v = 27 \text{ ksi}$ 

Excluded: (Table J3.2 AISC 14th)

Bolt Diameter:  $d_{bcd} = 0.50 \text{ in}$ 

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

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

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

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

Bolt bearing strength at bracket connection: (Section J3.10)  $F_u = 58 \text{ ksi}$ 

Bolt edge distance: edge-dist = 1.00 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole  $L_c = \text{edge-dist -0.5*bh} = 0.72 \text{ in } 2$ 

and edge of adjacent hole or edge of plate:

Single end bolt bearing capacity: Bolt  $_{brg}$  =min[(1.5\*L  $_{c}$  \*th  $_{plate}$  \*F  $_{u}$ ),(3.0\*d  $_{bcd}$  \*th  $_{plate}$  \*F  $_{u}$ )] = 23.45 kip

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

### 3/8" BRACKETS

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## 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-total-on-concrete-ove-metal-deck = 750 lbf	Tension	
Load <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 1240 lbs	f Shear	Vertical & Horizontal Allowable Load -
Load allowable-total-on-4" min-concrete-slab = 1400 lbi	f Tension	(LRFD)
Load allowable-total-on-4" min-concrete-slab = 3150 lbi	f Shear	

Date: 5/7/2019

Job No.: 18-001

Sheet: 7 (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 #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"

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

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

Nominal Tensile Strength:  $F_{+} = 60 \text{ ksi}$  (Carbon Steel)

Ultimate Strength:  $F_{\mu} = 58 \text{ ksi}$ 

Anchor Spacing s = 0.00

Anchor Diameter:  $d_b = 0.500 \text{ in}$ 

Steel strength in tension:  $N_{sa} = 10.705 \text{ kip}$ 

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

Tributary length to each bolt: p = max 2b, but  $\leq s$  2.63 in

=

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

Resistance factor for prying:  $\Omega_{\rm pr}$  = 1.67 ASD

 $\Phi_{pr}$  = 0.90 LRFD  $\Phi_{t}$  = 0.75 LRFD

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

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 8.03 \text{ kip}$  (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  $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.712 a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.829$ (Eq. 9-33) (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:

$$\begin{split} \delta &= 1 - \frac{d^{+}}{p} = \quad 0.76 & a^{+} = a + \frac{d_{+}}{2} = \quad 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_{b}}{2}\right) = \quad 1.89 \text{ in} \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \mathcal{P} &= \frac{b^{+}}{\alpha^{+}} = \quad 0.61 & \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = \quad 0.34 \text{ in} \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ & \alpha^{+} = if \left[\beta = > 1, 1, \min \left[1, \frac{1}{\delta} * \left(\frac{B}{1 - \beta}\right)\right]\right] = \quad 0.67 \end{split}$$

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

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

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:  $F_{v} = 36 \, \text{ksi}$ (Table 2-4)

F., = 58 ksi Ultimate Strength:

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

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

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

Shear Lag Factor: (Table D3.1)

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

 $A_e = A_n *U = 1.15 in 2$ Effective Net Area: (Eqn D3-1)

> $\phi_{tr} = 0.75$ (Eqn D2-2) LRFD

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

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

### Shear on Bracket Vertical Leg:

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

> $\Phi_{yv} = 1.00$ LRFD (Eqn J4-3)

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

> $\Phi_{rv} = 0.75$ LRFD (Eqn J4-4)

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

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

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

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

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

 $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} = 0.1406 \text{ in 3}$ Plastic Modulus:

LRFD (Sec F1)

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

Moment Arm: Moment arm = 4.813 in (Eqn F11-1)

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

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

 $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} = 0.1406 \text{ in 3}$ Plastic Modulus:  $\phi_b = 0.9$ (Sec F1) LRFD

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

Moment Arm:

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

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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	$V_{\it allow}$	Design Thickness	$T_{allow}$	Number of Screws	N = 3
(in)		(in)		V <sub>allow</sub> *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:  $F_t = 45 \text{ ksi}$ 

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

Nominal Shear Strength, Threads  $F_v = 27 \text{ ksi}$ 

Excluded: (Table J3.2 AISC 14th)

Bolt Diameter:  $d_{bcd} = 0.50 \text{ in}$ 

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

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

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

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

Bolt bearing strength at bracket connection: (Section J3.10)  $F_u = 58 \text{ ksi}$ 

Bolt edge distance: edge-dist = 1.00 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole  $L_c = \text{edge-dist } -0.5 \text{*bh} = 0.72 \text{ in } 2$ 

and edge of adjacent hole or edge of plate:

Single end bolt bearing capacity: Bolt  $_{brg}$  =min[(1.5\*L  $_{c}$  \*th  $_{plate}$  \*F  $_{u}$ ),(3.0\*d  $_{bcd}$  \*th  $_{plate}$  \*F  $_{u}$ )] = 23.45 kip

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

### 3/8" BRACKETS

Date: 5/7/2019

Job No.: 18-001

Sheet: 7 (4)

## 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

 $\phi N_n = 370 \, lbf$ 

 $\phi V_n = 500 \text{ lbf}$ 

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

 $\phi N_{p} = 1000 \, lbf$ 

 $\phi V_n = 1322 \, lbf$ 

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 <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 370 lbf Tension

Vertical &

Load <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 500 lbf Shear Horizontal

Allowable Load -

Load  $_{allowable-total-on-4" min-concrete-slab} = 1000 lbf$  Tension (LRFD)

Load <sub>allowable-total-on-4" min-concrete-slab</sub> = 1322 lbf Shear

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### Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 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"

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

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

 $F_{t} = 60 \text{ ksi}$ Nominal Tensile Strength: (Carbon Steel)

Ultimate Strength:  $F_{y} = 58 \text{ ksi}$ 

s = 0.00Anchor Spacing

 $d_b = 0.500 \text{ in}$ Anchor Diameter:

 $N_{sa} = 10.705 \, kip$ Steel strength in tension:

b = 1.313 in

p = max 2b, Tributary length to each bolt: 2.63 in

Max distance from bolt to applied force:

Resistance factor for prying:

but <u><</u> s

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

 $\Omega_{\rm pr}$  = 1.67 ASD  $\Phi_{pr}$  = 0.90 LRFD

 $\Phi_{\rm t} = 0.75 \, LRFD$ 

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

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 8.03 \text{ kip}$ (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 $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.829$   $a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.829$ 

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:

$$\begin{split} \delta &= 1 - \frac{d^{+}}{p} = \quad 0.76 & a^{+} = a + \frac{d_{+}}{2} = \quad 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_{b}}{2}\right) = \quad 1.89 \text{ in} \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \mathcal{P} &= \frac{b^{+}}{\alpha^{+}} = \quad 0.61 & \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = \quad 0.34 \text{ in} \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ & \alpha^{+} = if \left[\beta = > 1, 1, \min \left[1, \frac{1}{\delta} * \left(\frac{B}{1 - \beta}\right)\right]\right] = \quad 0.67 \end{split}$$

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

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

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#### 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 Fiex #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 <sub>bracket</sub> = 0.375 in

Yield Strength:  $F_y = 36 \text{ ksi}$  (Table 2-4)

Ultimate Strength:  $F_u = 58 \text{ ksi}$ 

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

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

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

Shear Lag Factor: 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:  $A_e = A_n *U = 1.27 \text{ in } 2$  (Eqn D3-1)

 $\phi_{tr} = 0.75$  LRFD (Eqn D2-2)

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

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

### Shear on Bracket Vertical Leg:

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

 $\Phi_{yv}$  = 1.00 LRFD (Eqn J4-3)

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

 $\Phi_{rr} = 0.75$  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:  $V_{allow-rupture} = \phi_{rv} *0.60 *F_{u} *A_{riv} = 44.04 \text{ kip}$ 

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

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

Plastic Modulus:  $Z=\frac{bd^2}{4}=\frac{4*0.375^2}{4}=0.1406 \ \text{in3}$   $\phi_b=0.9 \qquad \text{LRFD} \qquad \text{(Sec F1)}$ 

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

Moment Arm: Moment <sub>arm</sub> = 4.313 in (Eqn F11-1)

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = \quad \text{1.057 kip LRFD} \qquad \text{(Vertical Component)}$ 

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

Plastic Modulus:  $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} = 0.1406 \text{ in 3}$   $\phi_b = 0.9$  LRFD (Sec F1)

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

Moment Arm: Moment <sub>arm</sub> = 1.313 in

<sup>\*\*</sup>Note: Bending of Bracket is Considered within the Prying Calculation

R. F. NELSO N.75 LRFD

& ASSOCIATES Structural Engineers

### 3/8" BRACKETS

Date: 5/7/2019

Job No.: 18-001

Sheet: 8 (3)

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

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

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

Bolt type: A307 Gr. A (Common Bolts), bearing type connection

Nominal Tensile Strength:  $F_t = 45 \text{ ksi}$ 

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

Nominal Shear Strength, Threads  $F_v = 27 \text{ ksi}$ 

Excluded: (Table J3.2 AISC 14th)

Bolt Diameter:  $d_{bcd} = 0.50 \text{ in}$ 

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

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

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

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

Bolt bearing strength at bracket connection: (Section J3.10)  $F_u = 58 \text{ ksi}$ 

Bolt edge distance: edge-dist = 1.00 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole  $L_c = \text{edge-dist -0.5*bh} = 0.72 \text{ in 2}$ 

and edge of adjacent hole or edge of plate:

Single end bolt bearing capacity: Bolt  $_{brg}$  = min[(1.5\*L  $_{c}$  \*th  $_{plate}$  \*F  $_{u}$ ),(3.0\*d  $_{bcd}$  \*th  $_{plate}$  \*F  $_{u}$ )] = 23.45 kip

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

### 3/8" BRACKETS

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## 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

 $\phi N_n = 370 \, lbf$ 

 $\phi V_n = 500 \text{ lbf}$ 

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

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### Design Scope:

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).

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

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

Resistance factor for prying:

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

 $F_{t} = 60 \text{ ksi}$ Nominal Tensile Strength: (Carbon Steel)

Ultimate Strength:  $F_{y} = 58 \text{ ksi}$ 

s = 3.00Anchor Spacing

 $d_b = 0.500 \text{ in}$ Anchor Diameter:

 $N_{sa} = 10.705 \, kip$ Steel strength in tension:

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

p = max 2b, Tributary length to each bolt: 2.63 in

but <u><</u> s

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

 $\Omega_{\rm pr}$  = 1.67 ASD  $\Phi_{pr}$  = 0.90 LRFD

 $\Phi_{\rm t} = 0.75 \, LRFD$ 

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

Available tensile strength of single bolt:  $B = \Phi_t * N_{sa} = 8.03 \text{ kip}$ (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

 $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.712$   $a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 0.829$ 

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:

 $\delta = 1 - \frac{d'}{p} = 0.76 \qquad a' = a + \frac{d}{\frac{b}{2}} = 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_b}{2}\right) = 1.89 \text{ in}$   $Eq. (9-24) \qquad Eq. (9-27)$   $\rho = \frac{b'}{a'} = 0.61 \qquad \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = 0.34 \text{ in}$   $Eq. (9-26) \qquad Eq. (9-25)$  $\alpha' = if \left[ \beta => 1, 1, \min \left[ 1, \frac{1}{\delta} * \left( \frac{\beta}{1 - \beta} \right) \right] \right] = 0.67$ 

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

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

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(Eqn J4-4)

#### 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<sub>u</sub> = 58 ksi

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

 $\phi_{bv} = 0.9$  LRFD (Eqn D2.1)

Tensile Yielding:  $T_{\text{allow-yielding}} = \phi_{t\cdot y} *F_y *A_g = 97.20 \text{ kip}$  (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:  $A_e = A_n * U = 2.41 \text{ in 2}$  (Eqn D3-1)

 $\phi_{tr} = 0.75$  LRFD (Eqn D2-2)

 $T_{\text{allow-rupture}} = \phi_{tr} *F_u *A_e = 105.01 \text{ kip}$  (Eqn D2-2)

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

#### Shear on Bracket Vertical Leg:

Tensile Rupture:

Gross Area:  $A_{gv} = (8.0 \text{in})^*(0.25 \text{in}) = 3.00 \text{ in} 2$ 

 $\Phi_{yv} = 1.00$  LRFD (Eqn J4-3)

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

LRFD

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

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

V<sub>allow</sub> = min(V<sub>allow-yielding</sub>, V<sub>allow-rupture</sub>) = 64.80 kip LRFD (Horizontal Component)

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

 $\Phi_{rv} = 0.75$ 

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

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

$$\phi_b = 0.9$$
 LRFD (Sec F1)

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

Moment Arm: Moment <sub>arm</sub> = 1.313 in (Eqn F11-1)

Allowable Load:  $P_{\rm allow} = \frac{M_{\rm allow}}{Moment_{\rm orm}} = \quad \text{6.943 kip LRFD} \quad \text{(Vertical Component)}$ 

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

Plastic Modulus: 
$$Z=\frac{bd^2}{4}= \qquad \frac{8*0.375^2}{4}= \ \ 0.2813 \ \ \text{in} 3$$

 $\phi_b$  = 0.9 LRFD (Sec F1)

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

Moment Arm: Moment <sub>arm</sub> = 1.313 in

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = \begin{array}{cc} \text{6.943 kip LRFD} & \textit{(Horizontal Component)} \end{array}$ 

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

Plastic Modulus:  $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} = 0.1406 \text{ in 3}$   $\phi_b = 0.9$  LRFD (Sec F1)

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

Moment Arm: Moment <sub>arm</sub> = 1.313 in

Allowable Load:  $P_{\it allow} = \frac{M_{\it allow}}{Moment_{\it arm}} = \begin{tabular}{c} \# {\it DIV/0!} & \# {\it DIV/0!} \\ \hline \end{tabular} \end{tabular} \label{eq:partial}$ 

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

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

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

Bolt type: A307 Gr. A (Common Bolts), bearing type connection

Nominal Tensile Strength:  $F_t = 45 \text{ ksi}$ 

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

Nominal Shear Strength, Threads  $F_v = 27 \text{ ksi}$ 

Excluded: (Table J3.2 AISC 14th)

Bolt Diameter:  $d_{bcd} = 0.50 \text{ in}$ 

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

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

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

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

Bolt bearing strength at bracket connection: (Section J3.10)  $F_u = 58 \text{ ksi}$ 

Bolt edge distance: edge-dist = 1.00 in

Bolt hole diameter: bh = 0.563 in

Clear distance between edge of hole  $L_c = \text{edge-dist -0.5*bh} = 0.72 \text{ in 2}$ 

and edge of adjacent hole or edge of plate:

Single end bolt bearing capacity: Bolt  $_{brg}$  = min[(1.5\*L  $_{c}$  \*th  $_{plate}$  \*F  $_{u}$ ),(3.0\*d  $_{bcd}$  \*th  $_{plate}$  \*F  $_{u}$ )] = 23.45 kip

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

### 3/8" BRACKETS

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## 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-total-on-concrete-ove-metal-deck = 750 lbf	Tension	
Load <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 1240 lbf	Vertical & Shear Horizontal Allowable Load :	
Load allowable-total-on-4" min-concrete-slab = 1400 lbf	Tension	(LRFD)
Load allowable-total-on-4" min-concrete-slab = 3150 lbf	Shear	

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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:  $F_{v} = 36 \, \text{ksi}$ (Table 2-4)

F., = 58 ksi Ultimate Strength:

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

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

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

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

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

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

> $\phi_{tr} = 0.75$ LRFD (Eqn D2-2)

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

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

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

#### Shear on Bracket Vertical Leg:

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

> $\Phi_{vv} = 1.00$ (Eqn J4-3)

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

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

> $\Phi_{rv} = 0.75$ LRFD (Eqn J4-4)

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

(Horizontal

V<sub>allow</sub> = min(V<sub>allow-yielding</sub>, V<sub>allow-rupture</sub>) = 32.40 kip LRFD

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

 $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} = 0.1406 \text{ in 3}$ Plastic Modulus:

> $\phi_b = 0.9$ (Sec F1)

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

Moment Arm: Moment <sub>arm</sub> = 4.813 in (Eqn F11-1)

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

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

 $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} = 0.1406 \text{ in 3}$ Plastic Modulus:  $\phi_b = 0.9$ LRFD (Sec F1)

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

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

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

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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	$V_{\it allow}$	Design Thickness	$T_{allow}$	Number of Screws $N = 3$	
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *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

Design Thickness	$V_{\it allow}$	Design Thickness	$T_{allow}$	Number of Screws $N = 3$	
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *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

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# 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-816 & (4) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816

## Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket Thickness: th <sub>bracket</sub> = 0.375 in

Yield Strength:  $F_y = 36 \text{ ksi}$  (Table 2-4)

Ultimate Strength:  $F_u = 58 \text{ ksi}$ 

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

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

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

Shear Lag Factor: 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:  $A_e = A_n^* U = 1.27 \text{ in } 2$  (Eqn D3-1)

 $\phi_{tr} = 0.75 \qquad LRFD \qquad (Eqn D2-2)$ 

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

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

#### Shear on Bracket Vertical Leg:

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

 $\Phi_{yv}$  = 1.00 LRFD (Eqn J4-3)

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

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

 $\Phi_{N} = 0.75$  LRFD (Eqn J4-4)

Shear Rupture:  $V_{allow-rupture} = \phi_{rv} *0.60 *F_{u} *A_{nv} = 44.04 kip$  (Eqn J4-4)

V<sub>allow</sub> = min(V<sub>allow-yielding</sub>, V<sub>allow-rupture</sub>) = 32.40 kip LRFD (Horizontal

# Bending Moment on Lower Leg (Side B):

Plastic Modulus:  $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} = 0.1406 \text{ in 3}$ 

 $\phi_b$  = 0.9 LRFD (Sec F1)

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

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

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

# Bending Moment on Vertical Leg (Side A):

Plastic Modulus:  $Z=\frac{bd^2}{4}=-\frac{4*0.375^2}{4}=-0.1406 \ \text{in} 3$   $\phi_b=0.9 \qquad \qquad \text{LRFD} \qquad \qquad \text{(Sec F1)}$ 

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

Moment Arm: Moment <sub>arm</sub> = 1.313 in (Eqn F11-1)

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

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

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

LIVID

& ASSOCIATES Structural Engineers 3/8" BRACKETS

Date: 1/23/2020 Job No.: 18-001 Sheet: 12 (1)

# 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:  $th_{bracket} = 0.375 \text{ in}$ 

Yield Strength:  $F_y = 36 \text{ ksi}$  (Table 2-4)

Ultimate Strength:  $F_u = 58 \text{ ksi}$ 

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

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

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

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

Net Area:  $A_n = (8.0\text{in})^*(0.375\text{in}) - (5^*0.3125\text{in})^*(0.375\text{in}) = 2.41\text{ in}2$  (Sec B4.3)

Effective Net Area:  $A_e = A_n^* U = 2.41 \text{ in 2}$  (Eqn D3-1)

 $\phi_{tr} = 0.75$  LRFD (Eqn D2-2)

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

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

#### Shear on Bracket Vertical Leg:

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

 $\Phi_{yv} = 1.00$  LRFD (Eqn J4-3)

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

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

 $\Phi_{rv} = 0.75 \qquad LRFD \qquad (Eqn J4-4)$ 

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

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

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

Plastic Modulus:  $Z = \frac{bd^2}{4} = \frac{(8)*0.375^2}{4} = 0.2813 \text{ in 3}$ 

 $\phi_b = 0.9$  LRFD (Sec F1)

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

Moment Arm: Moment <sub>arm</sub> = 4.813 in (Eqn F11-1)

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

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

Plastic Modulus:  $Z=\frac{bd^2}{4}= \quad \frac{(8)^*0.375^2}{4}= \text{ 0.2813 in3}$   $\phi_b=\text{ 0.9} \qquad \text{LRFD} \qquad \text{(Sec F1)}$ 

 $M_{allow} = \phi *F_{y} *Z = 9.113 \text{ kip-in}$ 

Moment Arm: Moment <sub>arm</sub> = 1.313 in (Eqn F11-1)

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

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1710 lbf

2805 lbf

4495 lbf

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

## Screws from Angle Bracket to Steel Sheet:

0.075-0.078

1/8" - 3/16"

661 lbf

638 lbf

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)

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 1	Thickness	$V_{\it allow}$	Design Thickness	$T_{allow}$	Number of Screws /	V = 5
	(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.04	8-0.048	377 lbf	0.048	131 lbf	1885 lbf	655 lbf
0.04	8-0.075	626 lbf	0.06	207 lbf	3130 lbf	1035 lbf
0.06	0-0.060	520 lbf	0.075	255 lbf	2600 lbf	1275 lbf
0.07	5-0.078	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:						
•	Thickness	$V_{\it allow}$	Design Thickness	$T_{\it allow}$	Number of Screws A	
	(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.04	8-0.048	377 lbf	0.048	131 lbf	1885 lbf	655 lbf
0.04	8-0.075	626 lbf	0.06	207 lbf	3130 lbf	1035 lbf
0.06	0-0.060	520 lbf	0.075	255 lbf	2600 lbf	1275 lbf

0.105

1/8"

3/16"

342 lbf

561 lbf

899 lbf

3305 lbf

3190 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-816 & (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:  $th_{bracket} = 0.375 \text{ in}$ 

Yield Strength:  $F_{y} = 36 \text{ ksi}$  (Table 2-4)

Ultimate Strength:  $F_u = 58 \text{ ksi}$ 

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

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

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

Shear Lag Factor: 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:  $A_e = A_n^* U = 1.15 \text{ in } 2$  (Eqn D3-1)

 $\phi_{tr} = 0.75$  LRFD (Eqn D2-2)

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

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

#### Shear on Bracket Vertical Leg:

Gross Area:  $A_{ov} = (4.0in)^*(0.375in) = 1.50 in2$ 

 $\Phi_{yv}$  = 1.00 LRFD (Eqn J4-3)

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

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

 $\Phi_{N} = 0.75 \qquad LRFD \qquad (Eqn J4-4)$ 

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

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

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

Plastic Modulus:  $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} = 0.1313 \text{ in 3}$ 

 $\phi_b$  = 0.9 LRFD (Sec F1)

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

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

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

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

Plastic Modulus:  $Z = \frac{bd^2}{4} = \quad \frac{4*0.375^2}{4} = \quad 0.1406 \ \text{in3}$   $\phi_b = 0.9 \qquad \qquad \text{LRFD} \qquad \text{(Sec F1)}$ 

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

Moment Arm: Moment <sub>arm</sub> = 1.313 in (Eqn F11-1)

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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	V <sub>allow</sub> Design Thickness		$T_{allow}$	Number of Screws $N = 3$	
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *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

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

Date: 1/23/2020 18-001 Job No.:

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# 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 <sub>bracket</sub> = 0.375 in

 $F_{y} = 36 \, \text{ksi}$ Yield Strenath: (Table 2-4)

 $F_u = 58 \, \text{ksi}$ Ultimate Strength:

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

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

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

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

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

 $A_e = A_n *U = 1.27 in 2$ (Eqn D3-1) Effective Net Area:

> $\phi_{tr} = 0.75$  LRFD (Eqn D2-2)

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

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

#### Shear on Bracket Vertical Leg:

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

(Eqn J4-3)

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

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

> $\Phi_{rv} = 0.75$ LRFD (Egn J4-4)

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

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

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

 $Z = \frac{bd^2}{4} = \frac{4*0.375^2}{4} = 0.1406 \text{ in 3}$ Plastic Modulus:

> $\phi_h = 0.9$ LRFD (Sec F1)

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

Moment arm = 6.313 in Moment Arm: (Ean F11-1)

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

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

Plastic Modulus:  $Z = \frac{bd^2}{4} = -\frac{4*0.375^2}{4} = -0.1406 \text{ in 3}$  $\phi_b = 0.9$  LRFD (Sec F1)

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

Moment arm = 1.313 in Moment Arm: (Eqn F11-1)

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

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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:

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

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

# 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:

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

 $F_{y} = 36 \, \text{ksi}$ Yield Strenath: (Table 2-4)

F<sub>u</sub> = 58 ksi Ultimate Strength:

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

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

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

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

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

 $A_e = A_n *U = 2.41 \text{ in } 2$ (Eqn D3-1) Effective Net Area:

> $\phi_{tr} = 0.75$  LRFD (Eqn D2-2)

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

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

#### Shear on Bracket Vertical Leg:

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

> $\Phi_{vv} = 1.00$ LRFD (Eqn J4-3)

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

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

> $\Phi_{rv} = 0.75$ LRFD (Eqn J4-4)

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

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

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

 $Z = \frac{bd^2}{4} = \frac{8*0.375^2}{4} = 0.2813 \text{ in 3}$ Plastic Modulus:

> $\phi_b = 0.9$  LRFD (Sec F1)

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

Moment Arm: Moment arm = 6.313 in (Egn F11-1)

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

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

 $Z = \frac{bd^2}{4} = \frac{8*0.375^2}{4} = 0.2813 \text{ in } 3$ Plastic Modulus:  $\phi_b = 0.9$ LRFD (Sec F1)

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

Moment arm = 1.313 in Moment Arm: (Eqn F11-1)

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

Date: 5/7/2019 Job No.: 18-001 Sheet: 15 (2)

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	$V_{\it allow}$	Design Thickness	$T_{allow}$	Number of Screws A	<i>l</i> = 5
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	3015 lbf	1050 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	5005 lbf	1655 lbf
0.060-0.060	833 lbf	0.075	409 lbf	4165 lbf	2045 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	5290 lbf	2740 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	5105 lbf	4485 lbf
		3/16"	1439 lbf		7195 lbf
contal Leg:					
Design Thickness	$V_{\it allow}$	Design Thickness	$T_{\it allow}$	Number of Screws A	V = 10

# Horizo

Design Thickness	$V_{\it allow}$	Design Thickness	$T_{allow}$	Number of Screws $N = 10$	
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *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

Date:

1/23/2020

18-001 16 (1) Sheet:

# 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-816 & (8) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816

## Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

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

 $F_{y} = 36 \, \text{ksi}$ Yield Strenath: (Table 2-4)

F<sub>u</sub> = 58 ksi Ultimate Strength:

Gross Area:  $A_g = (6.0in)*(0.375in) = 2.25 in2$ 

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

Tensile Yieldina:  $T_{allow-yielding} = \phi_{t-y} *F_y *A_g = 72.90 \text{ kip}$ (Eqn D2-1)

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

Net Area:  $A_n = (6.0in)*(0.375in)-(4*0.3125in)*(0.375in) = 1.78 in2$ (Sec B4.3)

 $A_e = A_n *U = 1.78 \text{ in } 2$ (Eqn D3-1) Effective Net Area:

> $\phi_{tr} = 0.75$  LRFD (Eqn D2-2)

Tensile Rupture:  $T_{allow-rupture} = \phi_{t-r} *F_u *A_e = 77.48 \text{ kip}$ (Eqn D2-2)

> $T_{allow\text{-}bracket} = min(T_{allow\text{-}yielding}, T_{allow\text{-}rupture}) = 72.90 \text{ kip LRFD}$ (Vertical & Horizontal Component))

#### Shear on Bracket Vertical Leg:

Gross Area:  $A_{qv} = (6.0in)*(0.375in) = 2.25 in2$ 

> $\Phi_{yy} = 1.00$  LRFD (Eqn J4-3)

Shear Yielding:  $V_{allow-yielding} = \phi_{yv} *0.60 *F_y *A_{gv} = 48.60 kip$ (Eqn J4-3)

 $A_{nv} = (6.0in)*(0.25in)-(4*0.3125in)*(0.375in) = 1.78 in2$ Net Area: (Sec B4.3)

> $\Phi_{rv} = 0.75$ LRFD (Eqn J4-4)

Shear Rupture:  $V_{allow-rupture} = \phi_{rv} *0.60 *F_{u} *A_{nv} = 61.99 kip$ 

> $V_{allow} = min(V_{allow-yielding}, V_{allow-rupture}) = 48.60 \text{ kip LRFD}$ (Horizontal

(Sec F1)

# Bending Moment on Lower Leg (Side B):

 $Z = \frac{bd^2}{4} = \frac{6*0.375^2}{4} = \text{ 0.2109 in 3}$ Plastic Modulus:

> $\phi_b = 0.9$  0.2109 in3 (Sec F1)

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

Moment arm = 8.313 in Moment Arm: (Eqn F11-1)

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

# Bending Moment on Vertical Leg (Side A):

 $Z = \frac{bd^2}{4} = \frac{6*0.375^2}{4} = 0.2109 \text{ in 3}$ Plastic Modulus:  $\phi_b = 0.9$  LRFD

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

Moment Arm: Moment arm = 1.313 in (Eqn F11-1)

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

Date: 5/7/2019 Job No.: 18-001 Sheet: 16 (2)

<u>Piece #310386A - L3" x 10" x 3/8" x 0'-6" Steel Angle Bracket (A36 min) w/ (4) 0.3125"</u> <u>dia Holes for 1/4" Hilti Kwik Flex #EAF-846 & (8) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816</u>

## Screws from Angle Bracket to Steel Sheet:

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

Design Thickness $V_{allow}$ Design Thickness $T_{allow}$ Number of Screws $N = 8$	T *N
	T <sub>allow</sub> *N 1680 lbf
0.040-0.040 000 lbi 0.040 210 lbi <b>4024 lbi</b>	1000 151
0.048-0.075 1001 lbf 0.06 331 lbf <b>8008 lbf</b>	2648 lbf
0.000.000	
0.060-0.060 833 lbf 0.075 409 lbf <b>6664 lbf</b>	3272 lbf
0.075-0.078 1058 lbf 0.105 548 lbf <b>8464 lbf</b>	4384 lbf
1/8" - 3/16" 1021 lbf 1/8" 897 lbf <b>8168 lbf</b>	7176 lbf
3/16" 1439 lbf	11512 lbf