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., 1/2" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #33124 A - L3" x 3" x 1/2" 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:  $th_{bracket} = 0.500 \text{ in}$ 

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

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

Anchor Spacing s = 0.00

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

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

-- '

Tributary length to each bolt: p = max 2b, 2.50 in

but <u><</u> s 2.50 in

Adjusted prying distance:  $b' = \left\lceil b - \frac{d_b}{2} \right\rceil = 1.00 \text{ in}$  (Eq. 9-21)

[ 2]

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

b = 1.250 in

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

Maximum bolt force for prying:  $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = 13.05 \text{ kip}$  (Eq. 9-22a)

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

 $t_c = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_u}} = 0.543 \text{ in}$  (Eq. 9-30a)

 $T_{avail} = BQ = 8.03 \, kip$  (Eq. 9-31)

Q = 0.632 a' <, Q = 1  $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.847 a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 1.481$ (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:

 $\delta = 1 - \frac{d'}{\rho} = 0.75 \qquad a' = a + \frac{d_b}{2} = 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_b}{2}\right) = 1.81 \text{ in}$   $Eq. (9-24) \qquad Eq. (9-27)$   $\rho = \frac{b'}{\alpha'} = 0.57 \qquad \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = 0.00 \text{ in}$   $Eq. (9-26) \qquad Eq. (9-25)$   $\alpha' = if \left[\beta = > 1, 1, \min \left[1, \frac{1}{\delta} * \left(\frac{\beta}{1 - \beta}\right)\right]\right] = 0.00$ 

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr}*p*F_{u}*(1+\delta*\alpha')}} = 0.50 \text{ in LRFD} \quad \text{ (Eq. 9-23a)}$$

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

# Piece #33124 A - L3" x 3" x 1/2" 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 <sub>bracket</sub> = 0.500 in

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

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

Gross Area:  $A_g = (3.0in)*(0.5in) = 1.50 in2$ 

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

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

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

Net Area:  $A_n = (3.0in)^*(0.5in) - (3^*0.3125in)^*(0.5in) = 1.03 in2$  (Sec B4.3)

Effective Net Area:  $A_e = A_n * U = 1.03 \text{ in } 2$  (Eqn D3-1)

 $\phi_t = 0.75$  LRFD (Eqn D2-2)

Tensile Rupture:  $T_{allow-rupture} = \phi_{t-r} *F_u *A_e = 44.86 \text{ kip}$  (Eqn D2-2)

 $T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 44.86 \text{ kip LRFD}$  (Vertical & Horizontal Component))

#### Shear on Bracket Vertical Leg:

Gross Area:  $A_{gv} = (3.0in)^*(0.3in) = 1.50 in2$ 

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

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

Net Area:  $A_{nv} = (3.0in)^*(0.5in) - (3^*0.3125in)^*(0.5in) = 1.03 in2$  (Sec B4.3)

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

Shear Rupture:  $V_{allow-rupture} = \phi_{rv} *0.60*F_u *A_{nv} = 35.89 \text{ kip}$ 

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

# Bending Moment on Vertical Leg (Side A):

Plastic Modulus: 
$$Z = \frac{bd^2}{4} = \frac{4*0.5^2}{4} = 0.2500 \text{ in 3}$$

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

$$M_{allow} = \phi *F_y *Z = 8.100 \text{ kip-in}$$

Moment Arm: 
$$Moment_{arm} = 1.250 \text{ in}$$
 (Eqn F11-1)

Allowable Load: 
$$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = \text{ 6.480 kip LRFD} \quad \text{(Vertical Component)}$$

# Bending Moment on Lower Leg (Side B):

Plastic Modulus: 
$$Z = \frac{bd^2}{4} = \frac{4*0.5^2}{4} = 0.2500 \text{ in 3}$$

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

$$M_{allow} = \phi *F_y *Z = 8.100 \text{ kip-in}$$
 (Eqn F11-1)

Moment Arm:  $Moment_{arm} = 1.250 in$ 

Allowable Load: 
$$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = ~~6.480~{\it kip}~{\it LRFD}~~{\it (Horizontal Component)}$$

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

Date: 5/7/2019

Job No.: 18-001 Sheet: 1 (3)

# Piece #33124 A - L3" x 3" x 1/2" 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:  $th_{bracket} = 0.500 \text{ in}$ 

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:  $\Omega \phi = 0.75$  LRFD

Shear Capacity of single bearing type bolt:  $V_{allow-bolt} = \phi *F_v *A_{bcd} = 5.30 \text{ kip}$ 

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  $L_c = \text{edge-dist -0.5*bh} = 1.22 \text{ in 2}$ 

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  $_{u}$ )] = 43.50 kip

 $Bolt_{allow-bolt} = \phi *Bolt_{bearing} 32.63 kip$ 

#### 1/2" BRACKETS

Date: 5/7/2019

Job No.: 18-001

Sheet: 1 (4)

# Piece #33124 A - L3" x 3" x 1/2" x 0'-4" Steel Angle Bracket (A36 min) w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 1/2"x3-3/4"

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

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

 $\phi N_n = 370 \, lbf$ 

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

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

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

 $\phi V_n = 1322 \, lbf$ 

Load allowable-total-on-4" min-concrete-slab = 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.

Shear

# Overall Capacity of Seismic Load - Piece #33124 A - L3" x 3" x 1/2" x 0'-4" Steel Angle Bracket (A36 min) w/ (3) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.5625" dia Hole for Hilti KB-TZ 1/2"x3-3/4"

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

Date: 5/7/2019 Job No.: 18-001 Sheet: 2 (1)

#### Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/2" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #33126A - L3" x 3" x 1/2" x 0'-6" 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 1/2" x 3-3/4"

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

Resistance factor for prying:

Bracket Thickness:  $th_{bracket} = 0.500 \text{ in}$ 

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:  $N_{sa} = 10.705 \text{ kip}$ 

Max distance from bolt to applied force: b = 1.250 in

Tributary length to each bolt: p = max 2b, 2.50 in

but <u><</u> s

Adjusted prying distance:  $b' = \left\lceil b - \frac{d_b}{2} \right\rceil = 1.00 \text{ in}$  (Eq. 9-21)

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

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

Maximum bolt force for prying:  $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = 13.05 \text{ kip}$  (Eq. 9-22a)

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

 $t_c = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_u}} = 0.543 \text{ in}$  (Eq. 9-30a)

 $T_{avail} = BQ = 5.08 \, kip$  (Eq. 9-31)

Q = 0.847 a' < Q = 1  $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.847 a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta') = 1.481$ 

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:

$$\begin{split} \delta &= 1 - \frac{d^{+}}{p} = \quad 0.75 & a^{+} = a + \frac{d_{+}}{2} = \quad 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_{b}}{2}\right) = \quad 1.81 \text{ in} \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \mathcal{P} &= \frac{b^{+}}{\alpha^{+}} = \quad 0.57 & \beta &= \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = \quad 1.02 \text{ in} \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ & \alpha^{+} &= if \left[\beta &=> 1, 1, \min \left[1, \frac{1}{\delta} * \left(\frac{B}{1 - \beta}\right)\right]\right] = \quad 1.00 \end{split}$$

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr}*p*F_{u}*(1+\delta*\alpha')}} = \text{0.30 in LRFD} \quad \text{(Eq. 9-23a)}$$

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

#### Piece #33126A - L3" x 3" x 1/2" x 0'-6" Steel Angle Bracket (A36 min) w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Fiex #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 <sub>bracket</sub> = 0.500 in

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

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

Gross Area:  $A_g = (6.0in)*(0.5in) = 3.00 in2$ 

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

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

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

Net Area:  $A_n = (6.0in)^*(0.5in) - (5^*0.3125in)^*(0.5in) = 2.22 in2$  (Sec B4.3)

Effective Net Area:  $A_e = A_n * U = 2.22 in2$  (Eqn D3-1)

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

Tensile Rupture:  $T_{allow-rupture} = \phi_{t-r} *F_u *A_e = 96.52 \text{ kip}$  (Eqn D2-2)

#### Shear on Bracket Vertical Leg:

Gross Area:  $A_{gv} = (6.0in)*(0.5in) = 3.00 in2$ 

 $\Phi_{yy}$  = 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)

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

Net Area:  $A_{nv} = (6.0in)^*(0.5in) - (5^*0.3125in)^*(0.5in) = 2.22 in2$  (Sec B4.3)

Shear Rupture:  $V_{allow-rupture} = \phi_{rv} *0.60*F_{u} *A_{nv} = 77.21 \text{ kip}$  (Eqn J4-4)

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

(Eqn F11-1)

# Bending Moment on Vertical Leg (Side A):

Plastic Modulus:

$$M_{allow} = \phi^*F_y^*Z = 12.150 \text{ kip-in}$$

Moment Arm: Moment <sub>arm</sub> = 1.250 in

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = ~9.720~{\it kip}~{\it LRFD}~~({\it Vertical Component})$ 

# Bending Moment on Lower Leg (Side B):

Plastic Modulus:  $Z = \frac{bd^2}{dt} = \frac{6*0.5^2}{4}$ 

$$Z = \frac{bd^2}{4} = \frac{6*0.5^2}{4} = 0.3750 \, in3$$
 
$$\phi_b = 0.9 \qquad LRFD \qquad (Sec F1)$$

$$M_{\text{allow}} = \phi^* F_y^* Z = 12.150 \text{ kip-in}$$
 (Eqn F11-1)

Moment Arm: Moment <sub>arm</sub> = 1.250 in

Allowable Load:  $P_{\it allow} = \frac{M_{\it allow}}{Moment_{\it arm}} = ~~9.720~kip~LRFD~~(Horizontal~Component)$ 

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

Date: 5/7/2019

Job No.: 18-001

Sheet: 2 (3)

<u>Piece #33126A - L3" x 3" x 1/2" x 0'-6" Steel Angle Bracket (A36 min)</u> w/ (5) 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 = 5
(in)		(in)		V <sub>allow</sub> *N	T allow *N
0.048-0.048	603 lbf	0.048	210 lbf	3015 lbf	1050 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	5005 lbf	1655 lbf
0.060-0.060	833 lbf	0.075	409 lbf	4165 lbf	2045 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	5290 lbf	2740 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	5105 lbf	4485 lbf
		3/16"	1439 lbf		7195 lbf

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

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

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

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

Bracket Thickness:  $th_{bracket} = 0.500 \text{ in}$ 

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:  $\Omega \phi = 0.75$  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: Boltbr'g=min[(1.5\*Lc\*thplate\*Fu),(3.0\*dbcd\*thplate\*Fu)] = 31.27 kip

 $2*Bolt_{allow-bolt} = 2*\phi *Bolt_{bearing}$  46.90 kip LRFD

#### 1/2" BRACKETS

Date: 5/7/2019

Job No.: 18-001

Sheet: 2 (4)

# Piece #33126A - L3" x 3" x 1/2" x 0'-6" 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 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. LRFD

# Overall Capacity of Seismic Load - Piece #33126A - L3" x 3" x 1/2" x 0'-6" 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 1/2" 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	

5/7/2019 Date: 18-001 Job No · Sheet: 8 (1)

#### Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/2" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #63124A - L6" x 3" x 1/2" 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

Max distance from bolt to applied force:

Resistance factor for prying:

Bracket Thickness:  $th_{bracket} = 0.500 \text{ in}$ 

 $F_{t} = 60 \text{ ksi}$ 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 = 1.250 in

p = max 2b, Tributary length to each bolt: 2.50 in

but <u><</u> s

 $b' = \left[ b - \frac{d_b}{2} \right] = 1.00 \text{ in}$ Adjusted prying distance: (Eq. 9-21)

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

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

 $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = 13.05 \text{ kip}$ Maximum bolt force for prying: (Eq. 9-22a)

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

> $t_c = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_u}} = \text{ 0.543 in}$ (Eq. 9-30a)

 $T_{avail} = BQ = 6.80 \text{ kip}$ (Eq. 9-31)

Q = 0.847 $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 1.481$   $a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 1.481$ 

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:

 $\delta = 1 - \frac{d'}{\rho} = 0.75 \qquad a' = a + \frac{d}{\frac{b}{2}} = 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_b}{2}\right) = 1.81 \text{ in}$   $Eq. (9-24) \qquad Eq. (9-27)$   $\rho = \frac{b'}{a'} = 0.57 \qquad \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = 0.32 \text{ in}$   $Eq. (9-26) \qquad Eq. (9-25)$  $\alpha' = if \left[ \beta => 1, 1, \min \left[ 1, \frac{1}{\delta} * \left( \frac{\beta}{1 - \beta} \right) \right] \right] = 0.62$ 

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr}*p*F_{u}*(1+\delta*\alpha')}} = \text{0.38 in LRFD} \quad \text{(Eq. 9-23a)}$$

Date: 1/23/2020 Job No.: 18-001 Sheet: 3 (2)

# Piece #63124A - L6" x 3" x 1/2" 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:  $th_{bracket} = 0.500 \text{ in}$ 

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

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

Gross Area:  $A_g = (4.0in)*(0.5in) = 2.00 in2$ 

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

Tensile Yielding:  $T_{\text{allow-yielding}} = \phi_{t\cdot y} *F_y *A_g = 64.80 \text{ kip}$  (Eqn D2-1)

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

Net Area:  $A_n = (4.0in)^*(0.5in) - (3^*0.3125in)^*(0.5in) = 1.53 in2$  (Sec B4.3)

Effective Net Area:  $A_e = A_n *U = 1.53 \text{ in 2}$  (Eqn D3-1)

 $\phi_t = 0.75$  LRFD (Eqn D2-2)

Tensile Rupture:  $T_{allow-rupture} = \phi_{t-r} * F_{u} * A_{e} = 66.61 \text{ kip}$  (Eqn D2-2)

 $T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 64.80 kip LRFD$  (Vertical & Horizontal Component))

#### Shear on Bracket Vertical Leg:

Gross Area:  $A_{qq} = (4.0in)^*(0.5in) = 2.00 in2$ 

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

Shear Yielding:  $V_{allow-yielding} = \phi_{yv} *0.60 *F_{y} *A_{gv} = 43.20 \text{ kip}$  (Eqn J4-3)

Net Area:  $A_{ny} = (4.0in)^*(0.5in) - (3^*0.3125in)^*(0.5in) = 1.53 in2$  (Sec B4.3)

Shear Rupture:  $V_{allow-rupture} = \phi_{rv} *0.60 *F_{u} *A_{pv} = 53.29 \text{ kip}$  (Eqn J4-4)

 $V_{allow} = min(V_{allow-yielding}, V_{allow-rupture}) = 43.20 \text{ kip LRFD}$  (Horizontal Component)

# Bending Moment on Vertical Leg (Side A):

Plastic Modulus:  $Z = \frac{bd^2}{4} = \frac{4*0.5^2}{4} = 0.5000 \text{ in 3}$ 

 $\phi_b = 0.9$  LRFD (Sec F1)

 $M_{allow} = \phi *F_y *Z = 16.200 \text{ kip-in}$ 

Moment Arm: Moment <sub>arm</sub> = 4.750 in (Eqn F11-1)

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 3.411 \ \textit{kip LRFD} \quad \textit{(Vertical Component)}$ 

# Bending Moment on Lower Leg (Side B):

Plastic Modulus:  $Z=\frac{bd^2}{4}= \ \frac{6*0.5^2}{4}= \ 0.3750 \ \text{in3}$   $\phi_b=0.9 \qquad \qquad \text{LRFD} \qquad \qquad \text{(Sec F1)}$ 

$$M_{allow} = \phi^* F_y^* Z = 12.150 \text{ kip-in}$$
 (Eqn F11-1)

Moment Arm: Moment <sub>arm</sub> = 1.250 in

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = \phantom{-} 9.720 \ \textit{kip LRFD} \quad \textit{(Horizontal Component)}$ 

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

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

Sheet: 3 (3)

Piece #63124A - L6" x 3" x 1/2" 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 <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:  $th_{bracket} = 0.500 \text{ in}$ 

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  $_{u}$ )] = 43.50 kip

 $Bolt_{allow-bolt} = \phi *Bolt_{bearing}$  32.63 kip LRFD

#### 1/2" BRACKETS

Date: 5/7/2019

Job No.: 18-001

Sheet: 3 (4)

# Piece #63124A - L6" x 3" x 1/2" 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 #63124A - L6" x 3" x 1/2" 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"

 $Load_{\it allowable-total-on-concrete-ove-metal-deck} = 370 \, lbf \qquad Tension$   $Load_{\it allowable-total-on-concrete-ove-metal-deck} = 500 \, lbf \qquad Shear \qquad Horizontal \\ Load_{\it allowable-total-on-4" min-concrete-slab} = 1000 \, lbf \qquad Tension \qquad (LRFD)$ 

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

Date: 5/7/2019 Job No.: 18-001 Sheet: 4 (1)

#### Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/2" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #63126A - L6" x 3" x 1/2" x 0'-6" 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"

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

Bracket Thickness:  $th_{bracket} = 0.500 \text{ in}$ 

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, 2.50 in

Max distance from bolt to applied force:

Resistance factor for prying:

but <u><</u> s

Adjusted prying distance:  $b' = \left\lceil b - \frac{d_b}{2} \right\rceil = 1.00 \text{ in}$  (Eq. 9-21)

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

b = 1.250 in

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

Maximum bolt force for prying:  $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = 13.05 \text{ kip}$  (Eq. 9-22a)

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

 $t_c = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_u}} = 0.543 \text{ in}$  (Eq. 9-30a)

 $T_{avail} = BQ = 6.80 \text{ kip}$  (Eq. 9-31)

Q = 1.481 a', Q = 1  $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 1.240 a' \ge 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 1.481$ (Fq. 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:

$$\begin{split} \delta &= 1 - \frac{d^{+}}{\rho} = & 0.75 & a^{+} = a + \frac{d_{+}}{2} = & 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_{b}}{2}\right) = & 1.81 \text{ in} \\ & & Eq. \ (9 - 24) & Eq. \ (9 - 27) \\ \rho &= \frac{b^{+}}{\alpha^{+}} = & 0.57 & \beta &= \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = & 0.32 \text{ in} \\ & Eq. \ (9 - 26) & Eq. \ (9 - 25) \\ \alpha^{+} &= & if \left[\beta &=> 1, 1, \min \left[1, \frac{1}{\delta} * \left(\frac{B}{1 - \beta}\right)\right]\right] = & 0.62 \end{split}$$

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr}*p*F_{u}*(1+\delta*\alpha')}} = \text{0.38 in LRFD} \quad \text{(Eq. 9-23a)}$$

& ASSOCIATES Structural Engineers

#### 1/2" BRACKETS

Date: 1/23/2020 Job No.: 18-001 Sheet: 4 (2)

#### Piece #63126A - L6" x 3" x 1/2" x 0'-6" Steel Angle Bracket (A36 min) w/ (5) 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:  $th_{bracket} = 0.500 \text{ in}$ 

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

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

Gross Area:  $A_g = (6.0in)*(0.5in) = 3.00 in2$ 

 $\phi_{t-y} = 0.9$  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:  $A_n = (460in)^*(0.5in) - (2^*0.3125in)^*(0.5in) = 2.69 in2$  (Sec B4.3)

Effective Net Area:  $A_e = A_n * U = 2.69 \text{ in } 2$  (Eqn D3-1)

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

Tensile Rupture:  $T_{\text{allow-rupture}} = \phi_{t-r} *F_u *A_e = 116.91 \text{ kip}$  (Eqn D2-2)

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

#### Shear on Bracket Vertical Leg:

Gross Area:  $A_{gv} = (6.0in)*(0.5in) = 3.00 in2$ 

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

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

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

Net Area:  $A_{nv} = (6.0in)^*(0.5n) - (2^*0.3125in)^*(0.5in) = 2.69 in2$  (Sec B4.3)

Shear Rupture:  $V_{allow-nupture} = \phi_{rv} *0.60 *F_{u} *A_{nv} = 93.53 kip$  (Eqn J4-4)

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

(Eqn F11-1)

# Bending Moment on Vertical Leg (Side A):

Plastic Modulus:

$$Z=rac{bd^2}{4}=rac{6*0.5^2}{4}=~0.3750\, \mbox{in3}$$
  $\phi_b=0.9~$  LRFD (Sec F1)

$$M_{allow} = \phi^*F_y^*Z = 12.150 \text{ kip-in}$$

Moment Arm: Moment <sub>arm</sub> = 4.750 in

 $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 2.558 \, kip \, LRFD$  (Vertical Component)

# Bending Moment on Lower Leg (Side B):

Plastic Modulus:

Allowable Load:

$$Z = \frac{bd^2}{4} = \frac{6*0.5^2}{4} = \frac{0.3750 \, \text{in3}}{4}$$
 
$$\phi_b = 0.9 \qquad \text{LRFD} \qquad \text{(Sec F1)}$$

$$M_{\text{allow}} = \phi^* F_y^* Z = 12.150 \text{ kip-in}$$
 (Eqn F11-1)

Moment Arm: Moment <sub>arm</sub> = 1.250 in

 $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = \begin{array}{c} 9.720 \ \textit{kip LRFD} \end{array} \hspace{0.5cm} \textit{(Horizontal Component)}$ 

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

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

4 (3)

Piece #63126A - L6" x 3" x 1/2" x 0'-6" Steel Angle Bracket (A36 min) w/ (5) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

## Screws from Angle Bracket to Steel Sheet:

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

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

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

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

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

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

Bracket Thickness:  $th_{bracket} = 0.500 \text{ in}$ 

Nominal Shear Strength, Threads

(Table J3.2 AISC 14th) Excluded:

Bolt Diameter:  $d_{hcd} = 0.50 in$ 

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

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

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:  $T_{allow-bolt} = \phi *F_t *A_{bcd} = 6.63 \text{ kip}$ 

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

Bolt edge distance: edge-dist = 1.50 in

bh = 0.563 inBolt hole diameter:

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

and edge of adjacent hole or edge of plate:

Single bearing capacity: Boltbr'g=min[(1.5\*Lc\*thplate\*Fu),(3.0\*dbcd\*thplate\*Fu)] = 43.50 kip

2\*Bolt allow-bolt = 3\*\phi \*Bolt bearing 65.25 kip LRFD

#### 1/2" BRACKETS

Date: 5/7/2019

Job No.: 18-001

Sheet: 9 (4)

# Piece #63126A - L6" x 3" x 1/2" x 0'-6" 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"

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 #63126A - L6" x 3" x 1/2" x 0'-6" 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"

Load <sub>allowable-total-on-concrete-ove-metal-deck</sub> = 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: 5 (1)

#### Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/2 Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #63128A - L6" x 3" x 1/2" x 0'-8" Steel Angle Bracket (A36 min) w/ (10) 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:  $th_{bracket} = 0.500 \text{ in}$ 

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:  $N_{sa} = 10.705 \text{ kip}$ 

Max distance from bolt to applied force: b = 1.250 in

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

but <u><</u> s

Adjusted prying distance:  $b' = \left\lceil b - \frac{d_b}{2} \right\rceil = 1.00 \text{ in}$  (Eq. 9-21)

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

 $\Phi_{pr}$ = 0.90 LRFD

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

Maximum bolt force for prying:  $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = 13.05 \text{ kip}$  (Eq. 9-22a)

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

 $t_c = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_u}} = 0.543 \text{ in}$  (Eq. 9-30a)

 $T_{avail} = BQ = 11.89 \, kip$  (Eq. 9-31)

Q = 1.240 a' <, Q = 1  $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 1.240 a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 1.481$ (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:

$$\begin{split} \delta &= 1 - \frac{d^{+}}{\rho} = & 0.75 & a^{+} = a + \frac{d_{+}}{2} = & 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_{b}}{2}\right) = & 1.81 \text{ in} \\ & & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \rho &= \frac{b^{+}}{\alpha^{+}} = & 0.57 & \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = & -0.57 \text{ in} \\ & & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ \alpha^{+} &= & \text{if} \left[\beta = > 1, 1, \min\left[1, \frac{1}{\delta} * \left(\frac{B}{1 - \beta}\right)\right]\right] = & -0.48 \end{split}$$

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr}*p*F_{u}*(1+\delta*\alpha')}} = \text{0.76 in LRFD} \quad \text{(Eq. 9-23a)}$$

R. F. NELSON'5	LRFD	(Eqn J4-4)	Date:	1/23/2020
& ASSOCIATES		1/2" BRACKETS	Job No.:	18-001
Structural Engineers			Sheet:	5 (2)

#### Piece #63128A - L6" x 3" x 1/2" x 0'-8" Steel Angle Bracket (A36 min) w/ (10) 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 <sub>bracket</sub> = 0.500 in

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

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

Gross Area:  $A_g = (8.0in)*(0.5in) = 4.00 in2$ 

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

Tensile Yielding:  $T_{allow-yielding} = \phi_{t-y} *F_y *A_g = 129.60 \text{ kip}$  (Eqn D2-1)

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

Net Area:  $A_n = (8.0in)^*(0.5in) - (5^*0.3125in)^*(0.5in) = 3.22 in2$  (Sec B4.3)

Effective Net Area:  $A_e = A_n * U = 3.22 \text{ in } 2$  (Eqn D3-1)

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

Tensile Rupture:  $T_{allow-rupture} = \phi_{t-r} *F_u *A_e = 140.02 \text{ kip}$  (Eqn D2-2)

 $T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-ruphure}) = 129.60 \text{ kip LRFD}$  (Vertical & Horizontal Component))

#### Shear on Bracket Vertical Leg:

Gross Area:  $A_{gv} = (8.0in)^*(0.5in) = 4.00 in2$ 

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

Shear Yielding:  $V_{allow-yielding} = \phi_{yy} *0.60 *F_y *A_{gy} = 86.40 \text{ kip}$  (Eqn J4-3)

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

Net Area:  $A_{nv} = (8.0in)^*(0.5in) - (5^*0.3125in)^*(0.5in) = 3.22 in2$  (Sec B4.3)

Shear Rupture:  $V_{allow-rupture} = \phi_{rv} *0.60*F_u *A_{riv} = 112.01 kip$  (Eqn J4-4)

 $V_{allow} = min(V_{allow-yielding}, V_{allow-rupture}) = 86.40 \text{ kip LRFD}$  (Horizontal Component)

# Bending Moment on Vertical Leg (Side A):

Plastic Modulus: 
$$Z=\frac{bd^2}{4}=\frac{8*0.5^2}{4}=\ 0.5000\ \text{in3}$$
 
$$\phi_b=0.9 \qquad \text{LRFD} \qquad \text{(Sec F1)}$$

$$M_{allow} = \phi^* F_y^* Z = 16.200 \text{ kip-in}$$

Allowable Load: 
$$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = \ 3.812 \, kip \, \text{LRFD} \qquad \text{(Vertical Component)}$$

# Bending Moment on Lower Leg (Side B):

Plastic Modulus: 
$$Z = \frac{bd^2}{4} = \frac{8*0.5^2}{4} = \frac{0.5000 \text{ in 3}}{4}$$
 
$$\phi_b = 0.9 \qquad \text{LRFD} \qquad \text{(Sec F1)}$$

$$M_{\text{allow}} = \phi^* F_y^* Z = 16.200 \text{ kip-in}$$
 (Eqn F11-1)

Moment Arm: Moment <sub>arm</sub> = 1.250 in

$$P_{allow} = \frac{M_{allow}}{Moment_{arm}} = \begin{array}{c} 12.960 \ \textit{kip LRFD} & \textit{(Horizontal Component)} \\ \end{array}$$

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

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Sheet: 5 (3)

Piece #63128A - L6" x 3" x 1/2" x 0'-8" Steel Angle Bracket (A36 min) w/ (10) 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	$V_{allow}$	Design Thickness	$T_{allow}$	Number of Screws	N = 10
(in)		(in)		V <sub>allow</sub> *N	T allow *N
0.048-0.048	603 lbf	0.048	210 lbf	6030 lbf	2100 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	10010 lbf	3310 lbf
0.060-0.060	833 lbf	0.075	409 lbf	8330 lbf	4090 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	10580 lbf	5480 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	10210 lbf	8970 lbf
		3/16"	1439 lbf		14390 lbf

# 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:  $th_{bracket} = 0.500 \text{ in}$ 

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:  $V_{allow-bolt} = \phi *F_v *A_{bcd} = 3.98 \ kip$ 

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  $_{brg} = min[(1.5 * L_c * th_{plate} * F_u), (3.0 * d_{bcd} * th_{plate} * F_u)] = 31.27 \ kip$ 

 $3*Bolt_{allow-bolt} = 3*\phi *Bolt_{bearing} 70.35 kip LRFD$ 

#### 1/2" BRACKETS

Date: 5/7/2019 Job No.: 18-001 Sheet: 10 (4)

# Piece #63128A - L6" x 3" x 1/2" x 0'-8" Steel Angle Bracket (A36 min) w/ (10) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (3) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

See Hilti Excel output for Allowable Combined Tension and Shear Loads for Concrete Over Metal Deck (LRFD) See Hilti Profis output for Allowable Combined Tension and Shear Loads (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 #63128A - L6" x 3" x 1/2" x 0'-8" Steel Angle Bracket (A36 min) w/ (10) 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	Shear	Vertical & Horizontal Allowable Load -
Load allowable-total-on-4" min-concrete-slab = 1400 lbf	Tension	(LRFD)
Load allowable-total-on-4" min-concrete-slab = 3150 lbf	Shear	

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

#### Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/2" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #83124A - L8" x 3" x 1/2" x 0'-4" Steel Angle Bracket (A36 min) w/ (6) 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

Max distance from bolt to applied force:

Bracket Thickness:  $th_{bracket} = 0.500 \text{ in}$ 

 $F_{t} = 60 \text{ ksi}$ 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 = 1.250 in

p = max 2b, Tributary length to each bolt: 2.50 in

but <u><</u> s

 $b' = \left[ b - \frac{d_b}{2} \right] = 1.00 \text{ in}$ Adjusted prying distance: (Eq. 9-21)

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

 $\Phi_{pr}$ = 0.90 LRFD

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

 $T = \frac{\Phi_{pr} * F_{u} * t h_{bracket}^{2} * P}{2*b} = 13.05 \text{ kip}$ Maximum bolt force for prying: (Eq. 9-22a)

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

> $t_c = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_u}} = 0.543 \text{ in}$ (Eq. 9-30a)

 $T_{avail} = BQ = 9.95 kip$ (Eq. 9-31)

Q = 1.000 $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.540$   $a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 1.481$ 

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:

$$\begin{split} \delta &= 1 - \frac{d^{+}}{p} = \quad 0.75 & a^{+} = a + \frac{d_{+}}{2} = \quad 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_{b}}{2}\right) = \quad 1.81 \text{ in} \\ & \text{Eq. (9-24)} & \text{Eq. (9-27)} \\ \mathcal{P} &= \frac{b^{+}}{\alpha^{+}} = \quad 0.57 & \beta &= \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = \quad -0.34 \text{ in} \\ & \text{Eq. (9-26)} & \text{Eq. (9-25)} \\ & \alpha^{+} &= if \left[\beta &=> 1, 1, \min \left[1, \frac{1}{\delta} * \left(\frac{B}{1 - \beta}\right)\right]\right] = \quad -0.34 \end{split}$$

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr}*p*F_{u}*(1+\delta*\alpha')}} = 0.64 \, \text{in LRFD} \quad \text{(Eq. 9-23a)}$$

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#### Prying of Piece #83124A - L8" x 3" x 1/2" x 0"-4" Steel Angle Bracket (A36 min) w/ (6) 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> = 0.500 in

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

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

Gross Area:  $A_g = (4.0in)*(0.5in) = 2.00 in2$ 

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

Tensile Yielding:  $T_{\text{allow-yielding}} = \phi_{t-y} *F_y *A_g = 64.80 \text{ kip}$  (Eqn D2-1)

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

Net Area:  $A_n = (4.0in)^*(0.5in) - (3^*0.3125in)^*(0.5in) = 1.53 in2$  (Sec B4.3)

Effective Net Area:  $A_e = A_n * U = 1.53 \text{ in } 2$  (Eqn D3-1)

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

Tensile Rupture:  $T_{allow-rupture} = \phi_{t-r} *F_u *A_e = 66.61 \text{ kip}$  (Eqn D2-2)

 $T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-ruphure}) = 64.80 \text{ kip LRFD}$  (Vertical & Horizontal Component))

#### Shear on Bracket Vertical Leg:

Gross Area:  $A_{gv} = (4.0in)^*(0.5in) = 2.00 in2$ 

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

Shear Yielding:  $V_{allow-yielding} = \phi_{yv} *0.60 *F_y *A_{gv} = 43.20 \text{ kip}$  (Eqn J4-3)

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

Net Area:  $A_{nv} = (4.0in)^*(0.5in) - (3^*0.3125in)^*(0.5in) = 1.53 in2$  (Sec B4.3)

Shear Rupture:  $V_{allow-rupture} = \phi_{rv} *0.60*F_u *A_{rv} = 53.29 \text{ kip}$  (Eqn J4-4)

V<sub>allow</sub> = min(V<sub>allow-yielding</sub>, V<sub>allow-rupture</sub>) = 43.20 kip (Horizontal Component)

(Sec F1)

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

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

Plastic Modulus:  $Z=\frac{bd^2}{4}=\frac{4*0.5^2}{4}=~0.2500~\text{in}3$   $\phi_0=~0.9~~\text{LRFD}$ 

 $M_{allow} = \phi * F_y * Z = 8.100 \text{ kip-in}$ 

Moment <sub>arm</sub> = 6.250 in (Ean F11-1)

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = 1.296 \, \mathit{kip} \, \mathsf{LRFD}$  (Vertical Component)

# Bending Moment on Lower Leg (Side B):

Moment Arm:

Plastic Modulus:  $Z = \frac{bd^2}{4} = \frac{4*0.5^2}{4} = \frac{\text{0.2500 in3}}{\text{LRFD}}$  (Sec F1)

$$M_{\text{allow}} = \phi^* F_y^* Z = 8.100 \text{ kip-in}$$
 (Eqn F11-1)

Moment Arm: Moment <sub>arm</sub> = 1.250 in

Allowable Load:  $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = ~~6.480~kip~LRFD~~(Horizontal~Component)$ 

R. F. NELSON 75 LRFD

& ASSOCIATES Structural Engineers

## 1/2" BRACKETS

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

Sheet: 6 (3)

Prying of Piece #83124A - L8" x 3" x 1/2" x 0'-4" Steel Angle Bracket (A36 min) w/ (6) 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 = 6
(in)		(in)		V <sub>allow</sub> *N	T <sub>allow</sub> *N
0.048-0.048	603 lbf	0.048	210 lbf	3618 lbf	1260 lbf
0.048-0.075	1001 lbf	0.06	331 lbf	6006 lbf	1986 lbf
0.060-0.060	833 lbf	0.075	409 lbf	4998 lbf	2454 lbf
0.075-0.078	1058 lbf	0.105	548 lbf	6348 lbf	3288 lbf
1/8" - 3/16"	1021 lbf	1/8"	897 lbf	6126 lbf	5382 lbf
		3/16"	1439 lbf		8634 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:  $th_{bracket} = 0.500 \text{ in}$ 

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  $_{u}$ )] = 31.27 kip

 $Bolt_{allow-bolt} = \phi *Bolt_{bearing} 23.45 \text{ kip LRFD}$ 

#### 1/2" BRACKETS

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# Prying of Piece #83124A - L8" x 3" x 1/2" x 0'-4" Steel Angle Bracket (A36 min) w/ (6) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (1) 0.4375" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

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

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 -Prying of Piece #83124A - L8" x 3" x 1/2" x 0'-4" Steel Angle Bracket (A36 min) w/ (6) 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

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

#### Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/2" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #83126A - L8" x 3" x 1/2" x 0'-6" Steel Angle Bracket (A36 min) w/ (10) 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

Bracket Thickness:  $th_{bracket} = 0.500 \text{ in}$ 

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

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

s = 4.00Anchor Spacing

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

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

b = 1.250 in

p = max 2b, Tributary length to each bolt: 2.50 in

Max distance from bolt to applied force:

Resistance factor for prying:

but <u><</u> s

 $b' = \left[ b - \frac{d_b}{2} \right] = 1.00 \text{ in}$ Adjusted prying distance: (Eq. 9-21)

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

 $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = 13.05 \text{ kip}$ Maximum bolt force for prying: (Eq. 9-22a)

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

> $t_c = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_u}} = \text{ 0.543 in}$ (Eq. 9-30a)

 $T_{avail} = BQ = 8.03 kip$ (Eq. 9-31)

Q = 1.000 $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.632$   $a' > 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 1.481$ 

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:

$$\begin{split} \delta &= 1 - \frac{d^{\, +}}{p} = \quad 0.75 & a^{\, +} = a + \frac{d_{\, +}}{2} = \quad 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_{\, b}}{2}\right) = \quad 1.81 \text{ in} \\ & \qquad \qquad Eq. \ (9 - 24) & \qquad \qquad Eq. \ (9 - 27) \\ \wp &= \frac{b^{\, +}}{a^{\, +}} = \quad 0.57 & \qquad \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = \quad 0.00 \text{ in} \\ & \qquad \qquad Eq. \ (9 - 26) & \qquad \qquad Eq. \ (9 - 25) \\ \alpha' &= if \left[\beta = > 1, 1, \min \left[1, \frac{1}{\delta} * \left(\frac{B}{1 - \beta}\right)\right]\right] = \quad 0.00 \end{split}$$

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr}*p*F_{u}*(1+\delta*\alpha')}} = \text{0.50 in LRFD} \quad \text{(Eq. 9-23a)}$$

Job No.: 18-001 Sheet: 7 (2)

1/23/2020

Date:

# Piece #83126A - L8" x 3" x 1/2" x 0'-6" Steel Angle Bracket (A36 min) w/ (10) 0.3125" dia Holes for 1/4" Hilti Kwik Flex #EAF-816 & (2) 0.5625" dia Hole for Hilti KB-TZ 3/8" x 3-3/4"

## Tension on Bracket Vertical Leg:

AISC 14th - Chapter D of Specification:

Bracket	Thickness:	th bracket =	0.500 in

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

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

Gross Area:  $A_g = (6.0in)*(0.5in) = 3.00 in2$ 

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

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

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

Net Area: 
$$A_n = (6.0in)^*(0.5in) - (5^*0.3125in)^*(0.5in) = 2.22 in2$$
 (Sec B4.3)

Effective Net Area: 
$$A_e = A_n * U = 2.22 \text{ in} 2$$
 (Eqn D3-1)

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

Tensile Rupture: 
$$T_{allow-rupture} = \phi_{t-r} *F_u *A_e = 96.52 \text{ kip}$$
 (Eqn D2-2)

$$T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-upture}) = 96.52 \text{ kip LRFD}$$
 (Vertical & Horizontal Component))

#### Shear on Bracket Vertical Leg:

Gross Area: 
$$A_{gv} = (6.0in)^*(0.5in) = 3.00 in2$$

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

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

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

Net Area: 
$$A_{nv} = (6.0in)^*(0.5in) - (5^*0.3125in)^*(0.5in) = 2.22 in2$$
 (Sec B4.3)

Shear Rupture: 
$$V_{allow-vupture} = \phi_{rv} *0.60 *F_{u} *A_{nv} = 77.21 \text{ kip}$$
 (Eqn J4-4)

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

# Bending Moment on Vertical Leg (Side A):

Plastic Modulus: 
$$Z=\frac{bd^2}{4}=\frac{6*0.5^2}{4}=~0.3750~\text{in}3$$
 
$$\phi_b=0.9~\text{LRFD}~\text{(Sec F1)}$$

$$M_{allow} = \phi * F_y * Z = 12.150 \text{ kip-in}$$

Allowable Load: 
$$P_{allow} = \frac{M_{allow}}{Moment_{orm}} = \ 1.944 \ \textit{kip LRFD} \quad \textit{(Vertical Component)}$$

# Bending Moment on Lower Leg (Side B):

Plastic Modulus: 
$$Z=\frac{bd^2}{4}=\frac{6*0.5^2}{4}=\begin{array}{c} \text{0.3750 in3} \\ \\ \phi_b = 0.9 \\ \text{LRFD} \end{array} \tag{Sec F1}$$

$$M_{allow} = \phi *F_y *Z = 12.150 \text{ kip-in}$$
 (Eqn F11-1)

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

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Piece #83126A - L8" x 3" x 1/2" x 0'-6" Steel Angle Bracket (A36 min) w/ (10) 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	$V_{\it allow}$	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

## 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:  $th_{bracket} = 0.500 \text{ in}$ 

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  $_{u}$ )] = 31.27 kip

 $2*Bolt_{allow-bolt} = 2*\phi *Bolt_{bearing}$  46.90 kip LRFD

#### 1/2" BRACKETS

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Job No.: 18-001

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# Piece #83126A - L8" x 3" x 1/2" x 0'-6" Steel Angle Bracket (A36 min) w/ (10) 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)

 $\phi N_n = 670 \, lbf$ 

 $\phi V_n = 1100 \, lbf$ 

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

 $\phi N_n = 1275 \, lbf$ 

 $\phi V_n = 2522 \, lbf$ 

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 #83126A - L8" x 3" x 1/2" x 0'-6" Steel Angle Bracket (A36 min) w/ (10) 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 = 2522 lbf	Shear	

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#### Design Scope:

Calculations to determine the Load and Resistance Factor Design of the seismic restraint as detailed by 9.0 SeismicCo., 1/2" Floor & Wall Brackets (included within this calculation package for reference).

Prying of Piece #83128A - L8" x 3" x 1/2" x 0'-8" Steel Angle Bracket (A36 min) w/ (10) 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:  $th_{bracket} = 0.500 \text{ in}$ 

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

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

s = 3.50Anchor 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 = 1.250 in

p = max 2b, Tributary length to each bolt: 2.50 in

but <u><</u> s

 $b' = \left[ b - \frac{d_b}{2} \right] = 1.00 \text{ in}$ Adjusted prying distance: (Eq. 9-21)

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

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

 $T = \frac{\Phi_{pr} * F_u * t h_{bracket}^2 * p}{2 * b} = 13.05 \text{ kip}$ Maximum bolt force for prying: (Eq. 9-22a)

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

> $t_c = \sqrt{\frac{4*B*b'}{\Phi_{LRFD}*p*F_u}} = \text{ 0.543 in}$ (Eq. 9-30a)

 $T_{avail} = BQ = 8.03 kip$ (Eq. 9-31)

Q = 0.632

 $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta * \alpha') = 0.847$   $0 \le a' \le 1, Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) = 1.481$ 

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:

$$\begin{split} \delta &= 1 - \frac{d^{\, +}}{p} = \quad 0.75 & a^{\, +} = a + \frac{d_{\, +}}{2} = \quad 1.75 \text{ in} \qquad \leq \left(1.25 * b * \frac{d_{\, b}}{2}\right) = \quad 1.81 \text{ in} \\ & \qquad \qquad Eq. \ (9 - 24) & \qquad \qquad Eq. \ (9 - 27) \\ \wp &= \frac{b^{\, +}}{a^{\, +}} = \quad 0.57 & \qquad \beta = \frac{1}{\rho} * \left(\frac{B}{T} - 1\right) = \quad 0.00 \text{ in} \\ & \qquad \qquad Eq. \ (9 - 26) & \qquad \qquad Eq. \ (9 - 25) \\ \alpha' &= if \left[\beta = > 1, 1, \min \left[1, \frac{1}{\delta} * \left(\frac{B}{1 - \beta}\right)\right]\right] = \quad 0.00 \end{split}$$

$$t_{\min} = \sqrt{\frac{4*T*b'}{\Phi_{pr}*p*F_{u}*(1+\delta*\alpha')}} = 0.50 \text{ in LRFD} \quad \text{ (Eq. 9-23a)}$$

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# Piece #83128A - L8" x 3" x 1/2" x 0'-8" Steel Angle Bracket (A36 min) w/ (10) 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:  $th_{bracket} = 0.500 \text{ in}$ 

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

Ultimate Strength: F., = 58 ksi

Gross Area:  $A_g = (8.0in)*(0.5in) = 4.00 in2$ 

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

Tensile Yielding:  $T_{allow-yielding} = \phi_{t-y} *F_y *A_g = 129.60 \text{ kip}$ (Eqn D2-1)

Shear Lag Factor: (Table D3.1)

 $A_n = (8.0in)^*(0.5in)^*(5^*0.3125in)^*(0.5in) = 3.22 in2$ Net Area: (Sec B4.3)

 $A_e = A_n *U = 3.22 in 2$ Effective Net Area: (Eqn D3-1)

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

Tensile Rupture:  $T_{allow-rupture} = \phi_{t-r} *F_u *A_e = 140.02 \text{ kip}$ (Eqn D2-2)

> $T_{allow-bracket} = min(T_{allow-yielding}, T_{allow-rupture}) = 129.60 \text{ kip LRFD}$ (Vertical & Horizontal Component))

#### Shear on Bracket Vertical Leg:

Gross Area:  $A_{qq} = (8.0in)*(0.5in) = 4.00 in2$ 

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

Shear Yielding:  $V_{allow-yielding} = \phi_{yv} *0.60 *F_y *A_{gv} = 86.40 \text{ kip}$ (Eqn J4-3)

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

 $A_{nv} = (8.0in)*(0.5in)-(5*0.3125in)*(0.5in) = 3.22 in2$ (Sec B4.3) Net Area:

 $V_{allow-rupture} = \phi_{rv} * 0.60 * F_{u} * A_{rv} = 112.01 kip$ Shear Rupture:

> $V_{allow} = min(V_{allow-yielding}, V_{allow-rupture}) = 86.40 \text{ kip LRFD}$ (Horizontal Component)

# Bending Moment on Vertical Leg (Side A):

Plastic Modulus: 
$$Z = \frac{bd^2}{4} = \frac{8*0.5^2}{4} = \text{ 0.5000 in 3}$$

$$\dot{s}_b = 0.9$$
 LRFD (Sec F1)

 $M_{allow} = \phi *F_y *Z = 16.200 \text{ kip-in}$ 

Moment Arm: Moment arm = 6.250 in (Eqn F11-1)

 $P_{allow} = \frac{M_{allow}}{Moment_{ann}} = 2.592 \ kip \ LRFD$  (Vertical Component) Allowable Load:

# Bending Moment on Lower Leg (Side B):

Plastic Modulus: 
$$Z = \frac{bd^2}{4} = \frac{8*0.5^2}{4} = 0.5000 \ \text{in3}$$
 
$$\phi_b = 0.9 \qquad \text{LRFD} \qquad \text{(Sec F1)}$$

$$M_{\text{allow}} = \phi^* F_y^* Z = 16.200 \text{ kip-in}$$
 (Eqn F11-1)

Moment <sub>arm</sub> = 1.250 in Moment Arm:

 $P_{allow} = \frac{M_{allow}}{Moment_{arm}} = \quad \mbox{12.960 kip LRFD} \qquad \mbox{(Horizontal Component)}$ Allowable Load:

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

Date: 5/7/2019

Job No.: 18-001 Sheet: 8 (3)

Piece #83128A - L8" x 3" x 1/2" x 0'-8" Steel Angle Bracket (A36 min) w/ (10) 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, IrfD 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) Irfd

Design Thickness	$V_{allow}$	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

## 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:  $th_{bracket} = 0.500 \text{ in}$ 

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  $_{u}$ )] = 31.27 kip

 $3*Bolt_{allow-bolt} = 3*\phi *Bolt_{bearing} 70.35 kip LRFD$ 

#### 1/2" BRACKETS

Date: 5/7/2010

Job No.: 18-001

Sheet: 8 (4)

# Piece #83128A - L8" x 3" x 1/2" x 0'-8" Steel Angle Bracket (A36 min) w/ (10) 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 #83128A - L8" x 3" x 1/2" x 0'-8" Steel Angle Bracket (A36 min) w/ (10) 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 allowable-total-on-concrete-ove-metal-deck = 1240 lbf	Shear	Vertical & Horizontal Allowable Load -
Load allowable-total-on-4" min-concrete-slab = 1400 lbf	Tension	(LRFD)
Load allowable-total-on-4" min-concrete-slab = 3150 lbf	Shear	