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Design:	K-10 Shear Lug	Date:	8/7/2023
Fastening point:			

Specifier's comments:

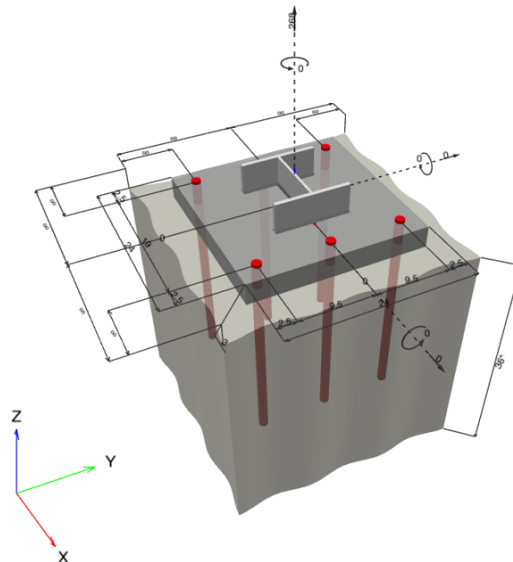
1 Anchor Design

1.1 Input data

Anchor type and diameter:	Heavy Square Head ASTM F 1554 GR. 36 1 1/4	
Item number:	not available	
Effective embedment depth:	$h_{ef} = 25.000$ in.	
Material:	ASTM F 1554	
Evaluation Service Report:	Hilti Technical Data	
Issued Valid:	- -	
Proof:	Design Method ACI 318-14 / CIP	
Stand-off installation:	$e_b = 0.000$ in. (no stand-off); $t = 3.000$ in.	
Anchor plate ^{CBFEM} :	$l_x \times l_y \times t = 24.000$ in. \times 24.000 in. \times 3.000 in.;	
Profile:	W shape (AISC), W10X49; (L x W x T x FT) = 9.980 in. \times 10.000 in. \times 0.340 in. \times 0.560 in.	
Base material:	cracked concrete, 4000, $f'_c = 4,000$ psi; $h = 36.000$ in.	
Reinforcement:	tension: condition A, shear: condition A; edge reinforcement: none or $<$ No. 4 bar	
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (a)) Shear load: yes (17.2.3.5.3 (a))	

CBFEM - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [kip, ft.kip]



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1.1.1 Load combination and design results

Case	Description	Forces [kip] / Moments [ft.kip]	Seismic	Max. Util. Anchor [%]
1	Pu max	N = -409.000; V _x = 0.000; V _y = 0.000; M _x = 0.00000; M _y = 0.00000; M _z = 0.00000;	yes	1
<u>2</u>	<u>Pu min</u>	<u>N = 268.000; V_x = 0.000; V_y = 0.000;</u> <u>M_x = 0.00000; M_y = 0.00000; M_z = 0.00000;</u>	<u>yes</u>	<u>159</u>

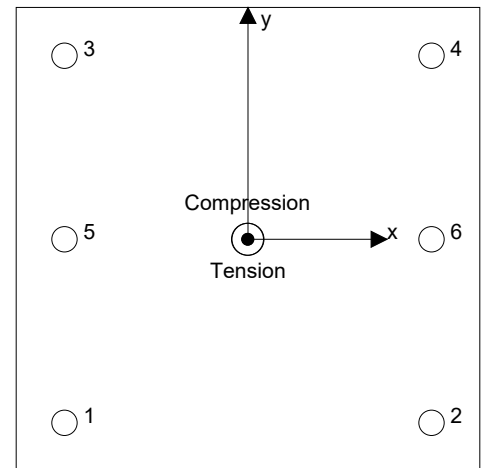
1.2 Load case/Resulting anchor forces

Controlling load case: 2 Pu min

Anchor reactions [kip]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	33.783	0.067	0.050	0.045
2	33.805	0.067	-0.050	0.045
3	33.785	0.067	0.050	-0.045
4	33.804	0.067	-0.050	-0.045
5	66.720	0.075	0.075	-0.000
6	66.677	0.075	-0.075	-0.000



resulting tension force in (x/y)=(-0.000/0.000): 268.575 [kip]
 resulting compression force in (x/y)=(-0.026/0.001): 0.981 [kip]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N _{ua} [kip]	Capacity ϕN_n [kip]	Utilization $\beta_N = N_{ua} / \phi N_n$	Status
Steel Strength*	66.720	42.151	159	not recommended
Pullout Strength*	66.720	46.586	144	not recommended
Concrete Breakout Failure**	268.575	191.119	141	not recommended
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)



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1.3.1 Steel Strength

$$N_{sa} = A_{se,N} f_{uta} \quad \text{ACI 318-14 Eq. (17.4.1.2)}$$

$$\phi N_{sa} \geq N_{ua} \quad \text{ACI 318-14 Table 17.3.1.1}$$

Variables

$A_{se,N} [\text{in.}^2]$	$f_{uta} [\text{psi}]$
0.97	58,000

Calculations

$N_{sa} [\text{kip}]$
56.202

Results

$N_{sa} [\text{kip}]$	ϕ_{steel}	$\phi N_{sa} [\text{kip}]$	$N_{ua} [\text{kip}]$
56.202	0.750	42.151	66.720

1.3.2 Pullout Strength

$$N_{pN} = \psi_{c,p} N_p \quad \text{ACI 318-14 Eq. (17.4.3.1)}$$

$$N_p = 8 A_{brg} f'_c \quad \text{ACI 318-14 Eq. (17.4.3.4)}$$

$$\phi N_{pN} \geq N_{ua} \quad \text{ACI 318-14 Table 17.3.1.1}$$

Variables

$\psi_{c,p}$	$A_{brg} [\text{in.}^2]$	λ_a	$f'_c [\text{psi}]$
1.000	2.77	1.000	4,000

Calculations

$N_p [\text{kip}]$
88.736

Results

$N_{pn} [\text{kip}]$	ϕ_{concrete}	ϕ_{seismic}	$\phi N_{pn} [\text{kip}]$	$N_{ua} [\text{kip}]$
88.736	0.700	0.750	46.586	66.720



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1.3.3 Concrete Breakout Failure

$$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{ACI 318-14 Eq. (17.4.2.1b)}$$

$$\phi N_{cbg} \geq N_{ua} \quad \text{ACI 318-14 Table 17.3.1.1}$$

$$A_{Nc} \text{ see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)}$$

$$A_{Nc0} = 9 h_{ef}^2 \quad \text{ACI 318-14 Eq. (17.4.2.1c)}$$

$$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.4)}$$

$$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.5b)}$$

$$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.7b)}$$

$$N_b = 16 \lambda_a \sqrt{f'_c} h_{ef}^{5/3} \quad \text{ACI 318-14 Eq. (17.4.2.2b)}$$

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
25.000	0.000	0.000	∞	1.000
c_{ac} [in.]	k_c	λ_a	f'_c [psij]	
-	16	1.000	4,000	

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [kip]
8,836.00	5,625.00	1.000	1.000	1.000	1.000	216.297

Results

N_{cbg} [kip]	$\phi_{concrete}$	$\phi_{seismic}$	ϕN_{cbg} [kip]	N_{ua} [kip]
339.768	0.750	0.750	191.119	268.575



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1.4 Shear load

	Load V_{ua} [kip]	Capacity ϕV_n [kip]	Utilization $\beta_v = V_{ua} / \phi V_n$	Status
Steel Strength*	0.075	21.919	1	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	0.075	71.267	1	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

1.4.1 Steel Strength

$$V_{sa} = 0.6 A_{se,V} f_{uta} \quad \text{ACI 318-14 Eq. (17.5.1.2b)}$$

$$\phi V_{steel} \geq V_{ua} \quad \text{ACI 318-14 Table 17.3.1.1}$$

Variables

$A_{se,V}$ [in. ²]	f_{uta} [psi]
0.97	58,000

Calculations

V_{sa} [kip]
33.721

Results

V_{sa} [kip]	ϕ_{steel}	$\phi V_{sa,eq}$ [kip]	V_{ua} [kip]
33.721	0.650	21.919	0.075



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1.4.2 Pryout Strength

$$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \right] \quad \text{ACI 318-14 Eq. (17.5.3.1a)}$$

$$\phi V_{cp} \geq V_{ua} \quad \text{ACI 318-14 Table 17.3.1.1}$$

$$A_{Nc} \text{ see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)}$$

$$A_{Nc0} = 9 h_{ef}^2 \quad \text{ACI 318-14 Eq. (17.4.2.1c)}$$

$$\Psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e_N}{3 h_{ef}}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.4)}$$

$$\Psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.5b)}$$

$$\Psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.7b)}$$

$$N_b = 16 \lambda_a \sqrt{f_c} h_{ef}^{5/3} \quad \text{ACI 318-14 Eq. (17.4.2.2b)}$$

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	25.000	0.000	0.000	∞
$\Psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c [psi]
1.000	∞	16	1.000	4,000

Calculations

A_{Nc} [in. ²]	A_{Nc0} [in. ²]	$\Psi_{ec1,N}$	$\Psi_{ec2,N}$	$\Psi_{ed,N}$	$\Psi_{cp,N}$	N_b [kip]
1,323.83	5,625.00	1.000	1.000	1.000	1.000	216.297

Results

V_{cp} [kip]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	ϕV_{cp} [kip]	V_{ua} [kip]
101.810	0.700	1.000	1.000	71.267	0.075

1.5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
1.583	0.003	1.000	133	not recommended

$$\beta_{NV} = (\beta_N + \beta_V) / 1.2 \leq 1$$



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1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates as per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to <https://submittals.us.hilti.com/PROFISAnchorDesignGuide/>
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0 .
- The anchor design methods in PROFIS Engineering require rigid anchor plates, as per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means that the anchor plate should be sufficiently rigid to prevent load re-distribution to the anchors due to elastic/plastic displacements. The user accepts that the anchor plate is considered close to rigid by engineering judgment."

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1.7 Installation data

Profile: W shape (AISC), W10X49; (L x W x T x FT) = 9.980 in. x 10.000 in. x 0.340 in. x 0.560 in.

Hole diameter in the fixture: $d_f = 1.312$ in.

Plate thickness (input): 3.000 in.

Anchor type and diameter: Heavy Square Head ASTM F 1554 GR. 36 1 1/4

Item number: not available

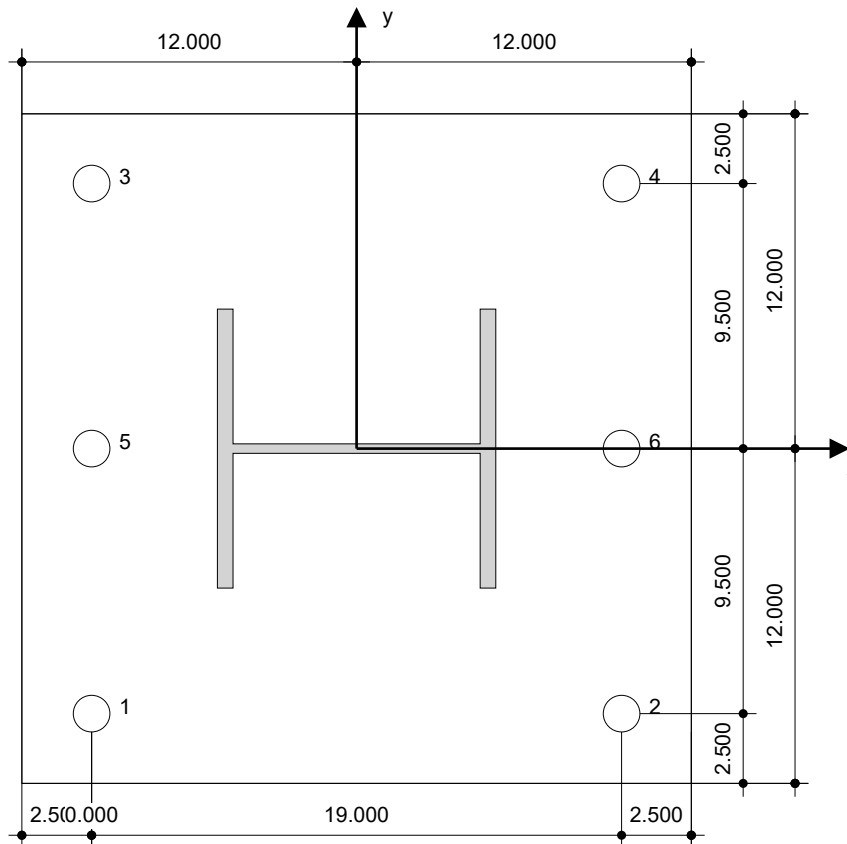
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 25.000 in.

Minimum thickness of the base material: 26.344 in.

Hilti Heavy Square Head headed stud anchor with 25 in embedment, 1 1/4, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c _{-x}	c _{+x}	c _{-y}	c _{+y}	Anchor	x	y	c _{-x}	c _{+x}	c _{-y}	c _{+y}
1	-9.500	-9.500	-	-	-	-	4	9.500	9.500	-	-	-	-
2	9.500	-9.500	-	-	-	-	5	-9.500	0.000	-	-	-	-
3	-9.500	9.500	-	-	-	-	6	9.500	0.000	-	-	-	-

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2 Anchor plate design

2.1 Input data

Anchor plate: Shape: Rectangular
 $l_x \times l_y \times t = 24.000 \text{ in} \times 24.000 \text{ in} \times 3.000 \text{ in}$
Calculation: CBFEM
Material: ASTM A36; $F_y = 36,000 \text{ psi}$; $\epsilon_{lim} = 5.00\%$

Anchor type and size: Heavy Square Head ASTM F 1554 GR. 36 1 1/4, $h_{ef} = 25.000 \text{ in}$

Anchor stiffness: The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Design method: AISC and LRFD-based design using component-based FEM

Seismic loads (cat. C, D, E or F): Tension load: Yes (17.2.3.4.3 (a))
Shear load: Yes (17.2.3.5.3 (a))

Stand-off installation: $e_b = 0.000 \text{ in}$ (No stand-off); $t = 3.000 \text{ in}$

Profile: W10X49; (L x W x T x FT) = 9.980 in x 10.000 in x 0.340 in x 0.560 in
Material: ASTM A500 Gr.B Rect; $F_y = 46,000 \text{ psi}$; $\epsilon_{lim} = 5.00\%$
Eccentricity x: 0.000 in
Eccentricity y: 0.000 in

Base material: Cracked concrete; 4000; $f_{c,cyl} = 4,000 \text{ psi}$; $h = 36.000 \text{ in}$

Welds (profile to anchor plate): Type of redistribution: Plastic
Material: E70xx

Mesh size: Number of elements on edge: 8
Min. size of element: 0.394 in
Max. size of element: 1.969 in

2.2 Summary

Description	Profile	Anchor plate			Welds [%]	Concrete [%]
		σ_{Ed} [psi]	ϵ_{Pl} [%]	Hole bearing [%]		
1 Pu max		36,634	0.00	14,406	89	17
2 Pu min		32,230	0.00	18,274	79	2

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Pu min

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	44.662 kip	33.783 kip
Anchor 2	44.662 kip	33.805 kip
Anchor 3	44.662 kip	33.785 kip
Anchor 4	44.662 kip	33.804 kip
Anchor 5	44.662 kip	66.720 kip
Anchor 6	44.662 kip	66.677 kip

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

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2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

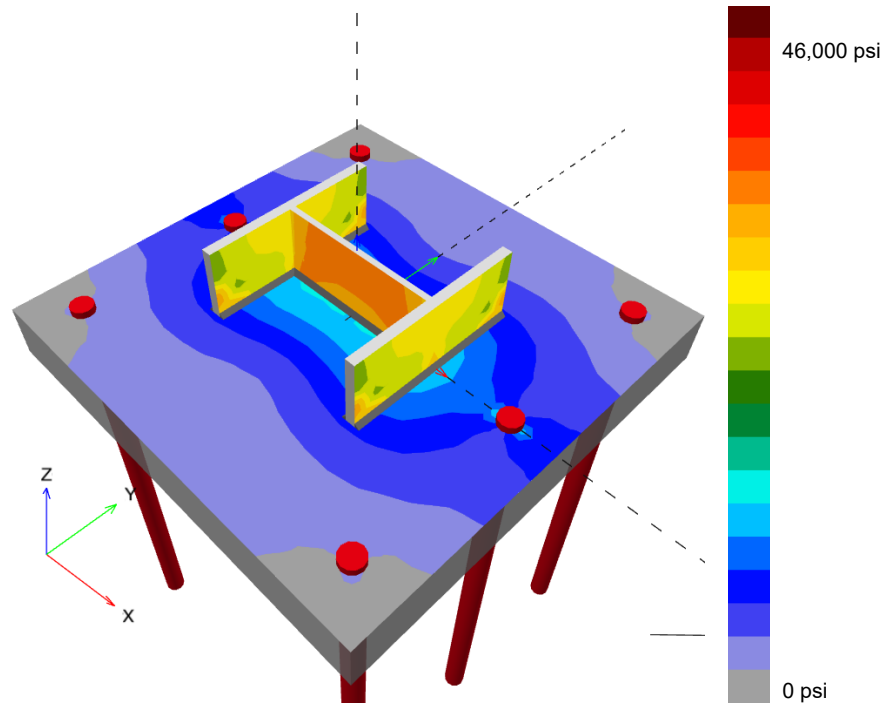
PROFIS Engineering does not verify the ductility requirements of the fixture as required by ACI 318-14. It is the responsibility of the user to determine if the loads that are input create ductile yielding in the attachment.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ϵ_{lim} [%]	σ_{Ed} [psi]	ϵ_{Pl} [%]	Status
Plate	Pu min	ASTM A36	36,000	5.00	18,274	0.00	OK
Profile	Pu min	ASTM A500 Gr.B Rect	46,000	5.00	31,723	0.00	OK
Profile	Pu min	ASTM A500 Gr.B Rect	46,000	5.00	31,733	0.00	OK
Profile	Pu min	ASTM A500 Gr.B Rect	46,000	5.00	36,634	0.00	OK

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Pu max



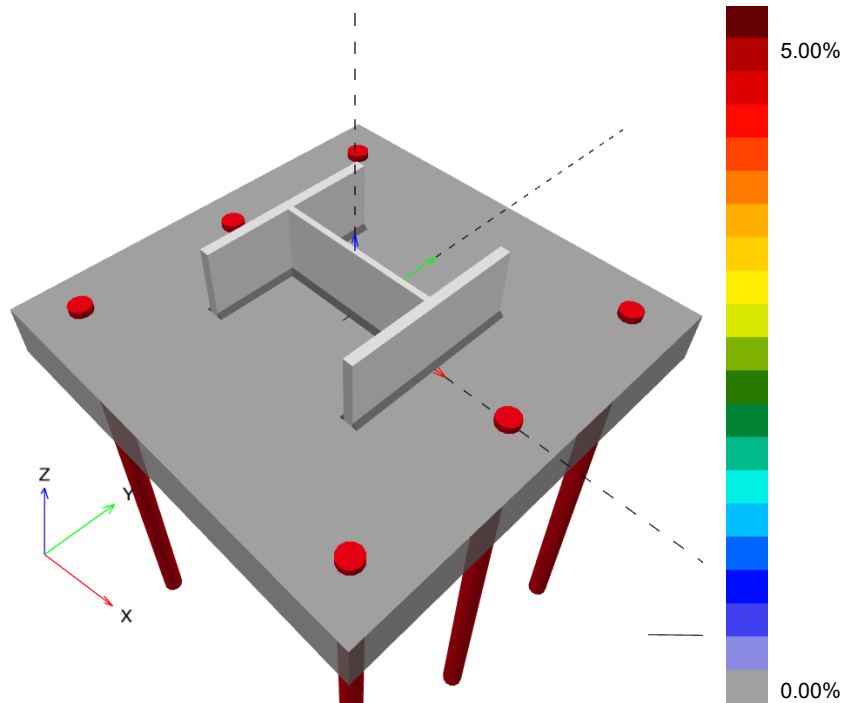
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2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Pu max



2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 2 - Pu min

Equations

$$R_n = \min(1.2 l_c t F_u, 2.4 d t F_u) \quad (\text{AISC 360-16 J3-6a, c})$$

$$\Phi R_n = 0.75 R_n$$

$$V \leq \Phi R_n$$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [kip]
Anchor 1	2.705	3.000	58,000	1.250	522.000
Anchor 2	2.703	3.000	58,000	1.250	522.000
Anchor 3	2.705	3.000	58,000	1.250	522.000
Anchor 4	2.703	3.000	58,000	1.250	522.000
Anchor 5	1.844	3.000	58,000	1.250	384.975
Anchor 6	1.844	3.000	58,000	1.250	384.975

Results

	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 1	0.067	391.500	1	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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	V [kip]	ΦR_n [kip]	Utilization [%]	Status
Anchor 2	0.067	391.500	1	OK
Anchor 3	0.067	391.500	1	OK
Anchor 4	0.067	391.500	1	OK
Anchor 5	0.075	288.731	1	OK
Anchor 6	0.075	288.731	1	OK

2.5 Welds

Profiles are modeled without taking the corner radius into account. Special rules for welding (e.g. for cold-formed profiles ...) are not taken into account by the software.

2.5.1 Anchor plate to profile

Decisive load combination: 1 - Pu max

Equations

$$F_{nw} = 0.6 F_{EXX} (1.0 + 0.5 \sin^{1.5} \Theta)$$

$$\Phi R_n = \Phi F_{nw} A_w$$

$$\text{Utilization} = \frac{F_n}{\Phi R_n}$$

Variables

Edge	X_u	T_h [in]	L_s [in]	L [in]	L_c [in]	F_{EXX} [psi]	Θ [°]	A_w [in ²]
Member 1-bfl 1	E70xx	▲0.180▲	0.255	9.984	1.248	70,000	76.4	0.22
Member 1-bfl	E70xx	▲0.180▲	0.255	9.984	1.248	70,000	77.6	0.22
Member 1-tfl 1	E70xx	▲0.180▲	0.255	9.984	1.248	70,000	77.6	0.22
Member 1-tfl	E70xx	▲0.180▲	0.255	9.984	1.248	70,000	76.4	0.22
Member 1-w 1	E70xx	▲0.180▲	0.255	9.404	1.176	70,000	83.5	0.21
Member 1-w	E70xx	▲0.180▲	0.255	9.404	1.176	70,000	83.5	0.21

Results

Edge	F_n [kip]	ΦR_n [kip]	Utilization [%]	Status
Member 1-bfl 1	9.174	10.466	88	OK
Member 1-bfl	9.324	10.492	89	OK
Member 1-tfl 1	9.323	10.491	89	OK
Member 1-tfl	9.174	10.466	88	OK
Member 1-w 1	8.006	9.966	81	OK
Member 1-w	8.006	9.966	81	OK

2.6 Concrete

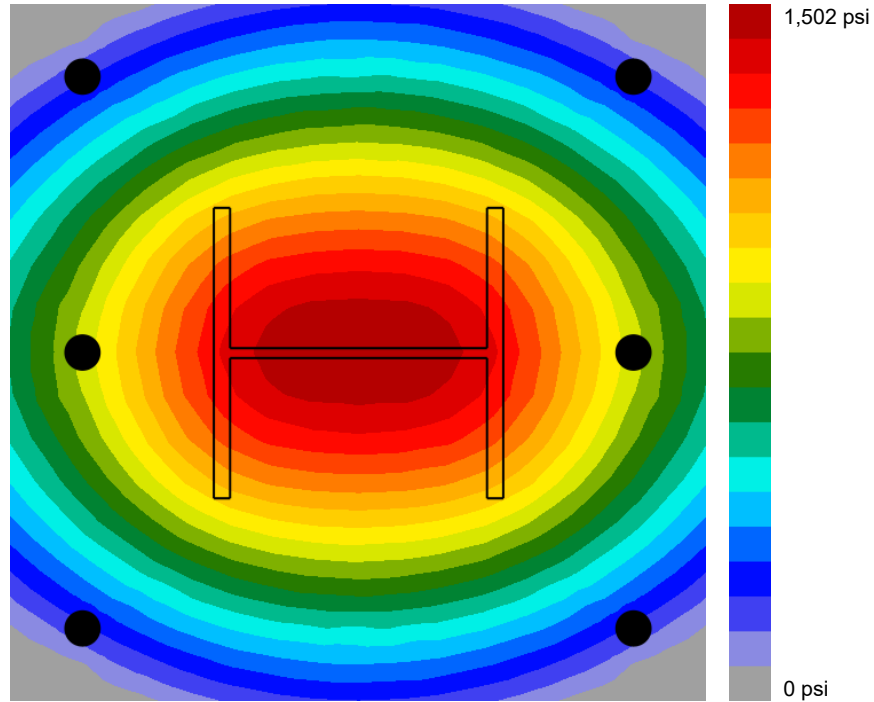
Decisive load combination: 1 - Pu max

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2.6.1 Compression in concrete under the anchor plate



2.6.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$$F_p = \Phi f_{p,max}$$

$$f_{p,max} = 0.85 f'_c \sqrt{\left(\frac{A_2}{A_1}\right)} \leq 1.7 f'_c; \sqrt{\left(\frac{A_2}{A_1}\right)} \leq 2$$

$$\sigma = \frac{N}{A_1}$$

$$\text{Utilization} = \frac{\sigma}{F_p}$$

Variables

N [kip]	f' _c [psi]	Φ	A ₁ [in ²]	A ₂ [in ²]
409.423	4,000	0.65	567.64	24,639.94

Results

Load combination	F _p [psi]	σ [psi]	Utilization [%]	Status
Pu max	4,420	721	17	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2.7 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
A_w	Effective area of weld critical element
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{PI}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f'_c	Concrete compressive strength
F_{EXX}	Electrode classification number, i.e. minimum specified tensile strength
F_u	Specified minimum tensile strength of the connected material
F_n	Force in weld critical element
F_{nw}	Nominal stress of the weld material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
L	Length of weld
L_c	Length of weld critical element
L_s	Leg size of weld
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
Φ	Resistance factor
ΦR_n	Factored resistance
R_n	Resistance
t	Thickness of the anchor plate
Θ	Angle of loading measured from the weld longitudinal axis
T_h	Throat thickness of weld
V	Resultant of shear forces V_y, V_z in bolt.
X_u	Filler metal tensile strength

2.8 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.



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3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Pu min	159%	NOT OK
Anchor plate	Pu min	51%	OK
Welds	Pu max	89%	OK
Concrete	Pu max	17%	OK
Profile	Pu max	80%	OK

Fastening does not meet the design criteria!



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4 Remarks; Your Cooperation Duties

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