Full-Scale Testing of PA-12 Series

CONNECT-EZ®

SIMPLE. SAFE. SECURE.™

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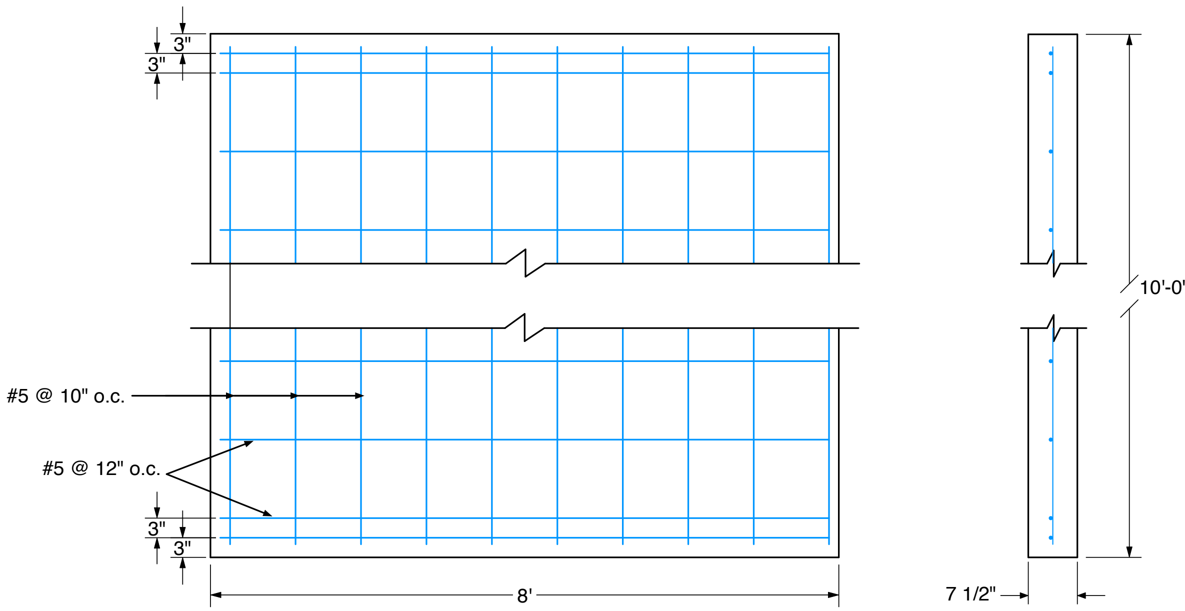
1. Introduction

Three 7.25" thick x 8'-0" wide x 10'-0" tall, reinforced concrete panels were cast in order to test a number of Connect-EZ devices under various loading configurations. The panels were reinforced with one curtain of No. 5 Gr. 60 reinforcing bars spaced vertically at 10" o.c. and at 12" o.c. horizontally. The reinforcement was placed at mid-thickness. The average 28-day concrete strength was 4,610 psi. Near the ends, where the devices were to be installed, the spacing between the horizontal bars was reduced to 3" based on common practice. The reinforcement layout is illustrated in Figure 1a. The photograph in Figure 1b shows the panel reinforcement and formwork. The panels were designed to allow multiple tests on each panel or three interconnected panels.



(a) Panel reinforcement details

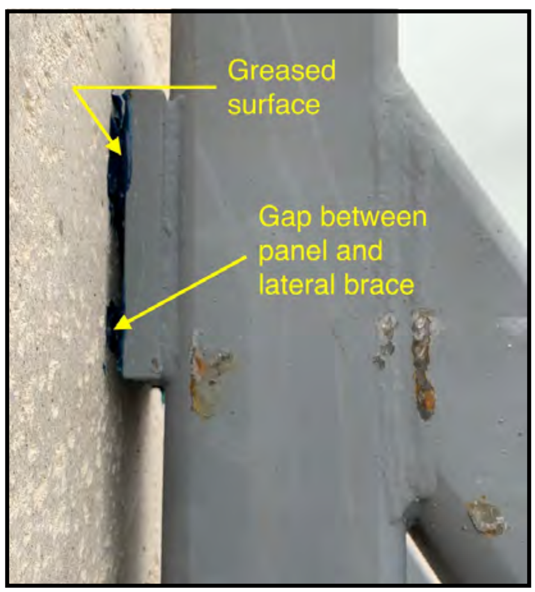
(b) Panels before concrete placement

Figure 1. Panel details

After adequate curing, each panel was tilted upright (Figure 2a) and connected to a 24" wide by 30" deep foundation reinforced with No. 5 Gr. 60 reinforcing bars, and braced as shown in Figure 2b. Approximately 1/8" gap was kept between the panel and lateral braces. The panel at the contact point to the brace was ground and greased (Figure 2c). The combination of having a gap and the greased surfaces eliminated any potential influence of friction between the panel and lateral bracing system on the test results. It should be noted that the tests were conducted without any grout between the panel and foundation.

(a) Panels before bracing

(b) Panels after installation of lateral braces



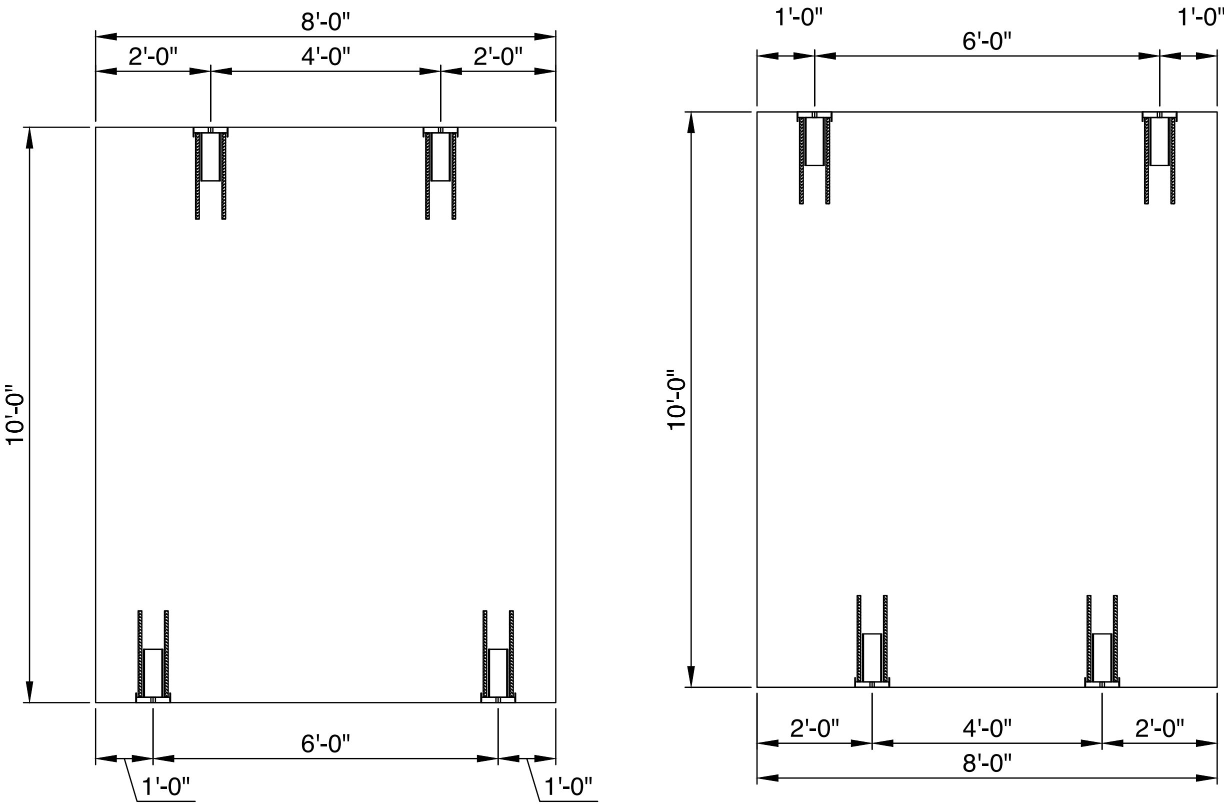
(c) Gap and greased surface

Figure 2. Overview of test panels and lateral bracing

The focus of panel 2 was on PA-12 series. At one end of the panel two PA-12 series were placed at 1'-0" from each edge (Figure 3a) while two PA-12 series were placed with 2'-0" edge distance at the other end (Figure 3b). The panel was tested according to the loading protocol summarized in Table 1, in which "x" indicates a test was conducted.

Table 1. Loading protocol

Configuration and edge distance	Tension	In-plane shear	Out-of-plane shear	Combined in-plane and out-of-plane shear
a 1'-0" (Figure 3a)	x	---	---	---
b 2'-0" (Figure 3b)	---	x	x	x



Reinforcement is not shown for clarity.
(a)

Reinforcement is not shown for clarity.
(b)

Figure 3. Locations of PA-12 devices for test configuration a and b

2. Test setup

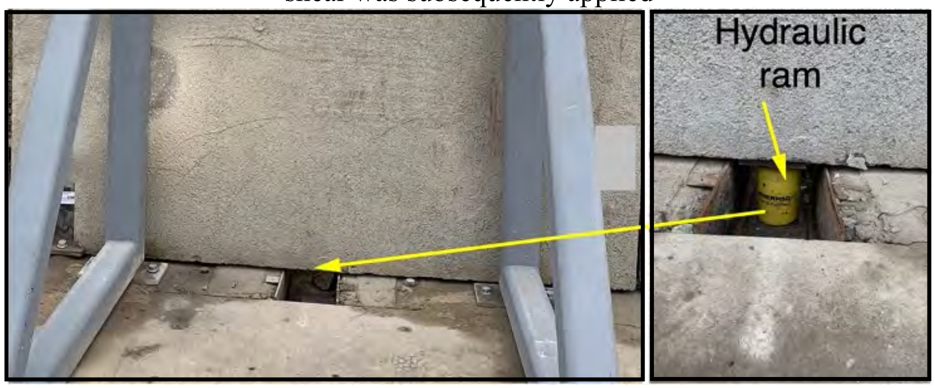
Pockets had been cast in the foundation to accommodate hydraulic rams. The locations of these pockets were selected to avoid interaction with the devices that were being tested. The following procedures were followed to load the devices under different loading conditions:

(a) **Tension test:** A60-kip hydraulic ram was placed vertically in a pocket in the middle of the panel to apply a vertical uplift force (Figure 4a). The load was transferred to the panel through a bearing plate centered on the panel thickness.

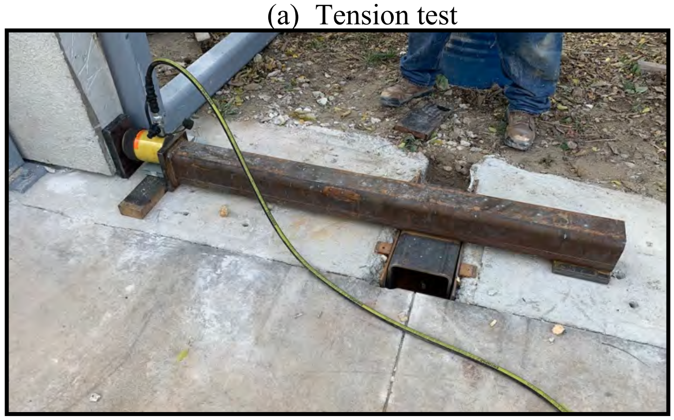
(b) **In-plane shear test:** The loading device consisted of a longitudinal HSS in the plane of the panel and a transverse HSS that reacted against a pocket cast in the foundation – see Figure 4b. The load was applied by a 60-kip hydraulic ram. The load point, defined as the center of hydraulic ram, was 9.5" (vertically) from the bottom of panel, resulting in tensile/compressive force equal to 9.5"/48" $V = 0.198V$ (V is the applied in-plane shear force) in the devices for test configuration b in addition to the applied in-plane shear.

(c) **Out-of-plane shear test:** The panel was loaded in the out-of-plane direction through a reaction frame that was anchored to the slab on grade in front of the test panel (Figure 4c). The load, applied by a 60-kip ram, was transferred to the test panel through two 7"x7" plates placed at 9-1/4" from the panel centerline. The loading apparatus did not bear against the devices as evident from the gap shown in Figure 4c.

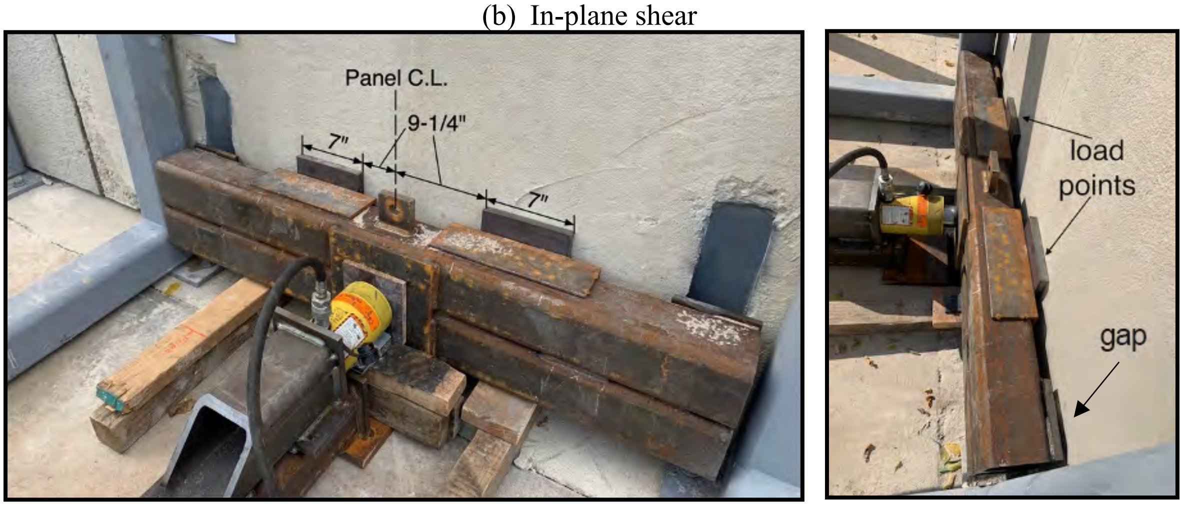
(d) **Combined in-plane and out-of-plane test:** The panel was loaded in the out-of-plane direction first through the apparatus discussed in (c). After reaching the target shear, the out-of-plane load was maintained by closing a needle valve between the pump and hydraulic ram. The in-plane shear was subsequently applied



(a) Tension test



(b) In-plane shear



(c) Out-of-plane shear

Figure 4. Test setup

3. Results and discussion

The maximum load applied for each test is summarized in Table 2. The panel self-weight was taken into account in the reported tension test results.

Tension loading for configuration “a” (1’-0” edge distance instead of the typical 2’-0”) was stopped after reaching 24.4 kips in each device. At this point, the concrete around one device had cracked and the two parts of the device had separated, as shown in Figure 5a. The concrete around the other device remained uncracked after resisting 24.4 kips of tensile force. Additional load could not be applied beyond 24.4 kips. The maximum applied load corresponds to 2.44 times the nominal design strength of 10 kips.

Each device in configuration “b” (2’-0” edge distance to the device) was subjected to (1) 10.1 kips in-plane shear (2) 10.1 kips out-of-plane shear, and (3) 14.3 kips combined in-plane and out-of-plane shear. Each device was loaded to slightly above the shear resistance of each anchor bolt alone (“resistance of steel”, $\phi_{sa} V_{sa}$), which is 9.89 kips. The concrete around the devices did not crack during any of the shear tests; however, the anchor bolts deformed significantly at the conclusion of combined in-plane and out-of-plane shear test, as evident from Figure 6. It is important to note that the device closer to the application of in-plane shear was subjected to a tensile force equal to $0.198V = 0.198*10.1$ kips = 2 kips (where 10.1 kips is the applied in-plane shear) in addition to the combined shear of 14.3 kips. However, the concrete around this device did not crack. (The forces in the far end device consisted of a combined shear of 14.3 kips and a compressive force equal to 2 kips.)

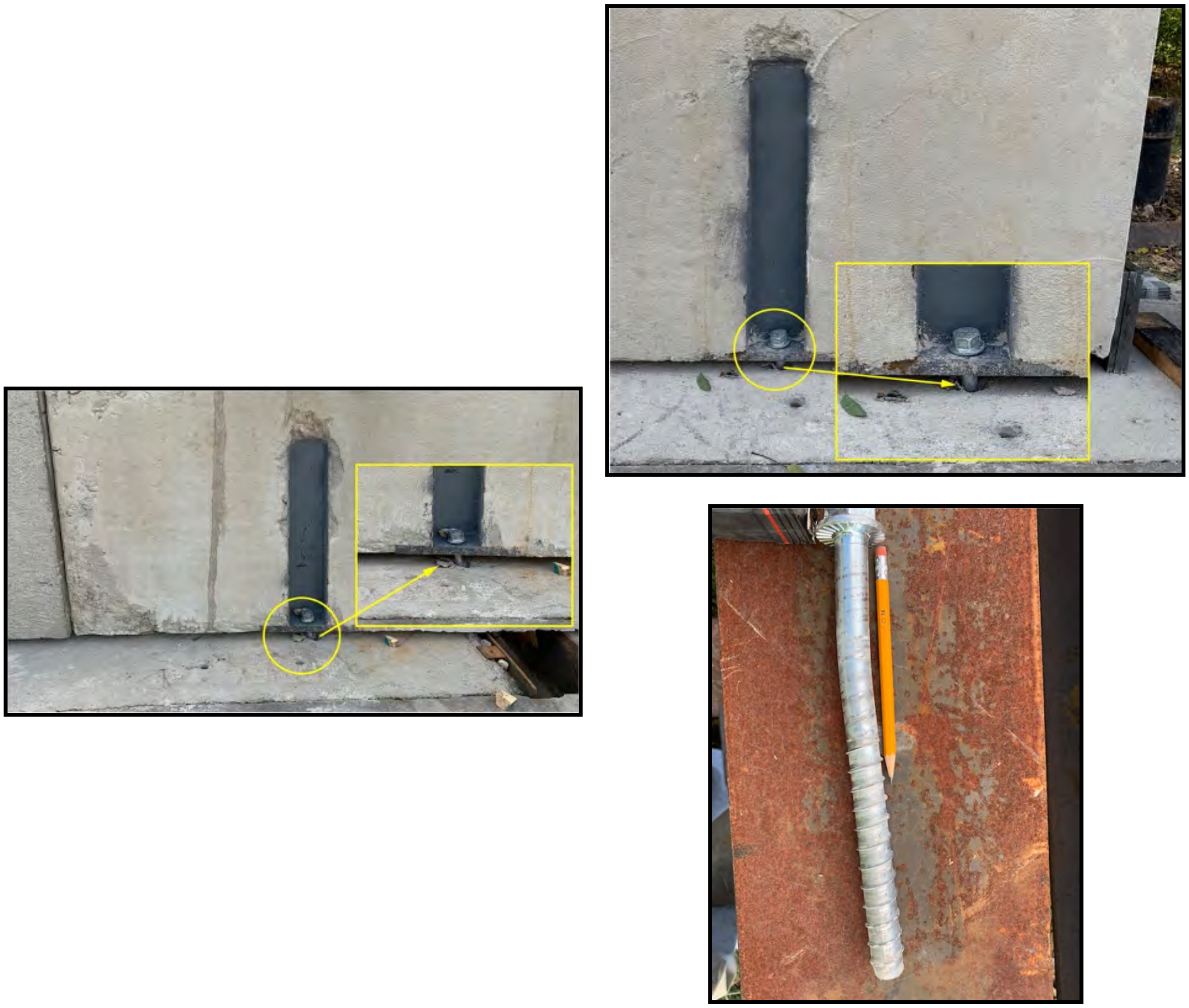
Table 2. Applied loads (kips) in each device

Configuration and edge distance	Tension	In-plane shear	Out-of-plane shear	Combined in-plane and out-of-plane shear
a 1’-0”	24.4	---	---	---
b 2’-0”	---	10.1	10.1	14.3

(a) Device 1

(b) Device 2

Figure 5. Crack pattern after applying 24.4 kips tension in each device



Permanent deformation of anchor bolt

(a) PA-12 away from the application of in-plane shear

(b) PA-12 close from the application of in-plane shear

Figure 6. Permanent deformation in anchor bolts applying 14.3 kips combined in-plane and out-of-plane shear in each device

4. Summary and observations

Two PA-12 devices were subjected to (1) tension, (2) in-plane shear, (3) out-of-plane shear, and (4) combined in-plane and out-of-plane shear. The devices did not experience any brittle failure during any of the tests. Clearly, such gravity loads would enhance the performance of PA-12 series.

Each PA-12 could resist 2.44 times the nominal design strength in tension even though the edge distance was one half of what is commonly used in practice (1’-0” vs. 2’-0”). The reported load carried by each device accounted for the self-weight of the panel. Additional superimposed dead load and live loads were, however, not simulated in the tests. Clearly, such gravity loads would enhance the performance of PA-12 device.

Each device could resist at least the shear capacity of the anchor bolt alone, so called “shear resistance of steel”, for all the three types of shear tests (in-plane, out-of-plane, and combined in-plane and out-of-plane). The anchor bolts experienced appreciable amounts of permanent deformations, but the concrete around the devices did not crack and devices did not fail.

The opening at the interface between the two parts of PA-12 (Figure 5a) is consistent with the eccentricity between the load point and reinforcing bar of PA-12. This eccentricity is shown as *e* in Figure 7.

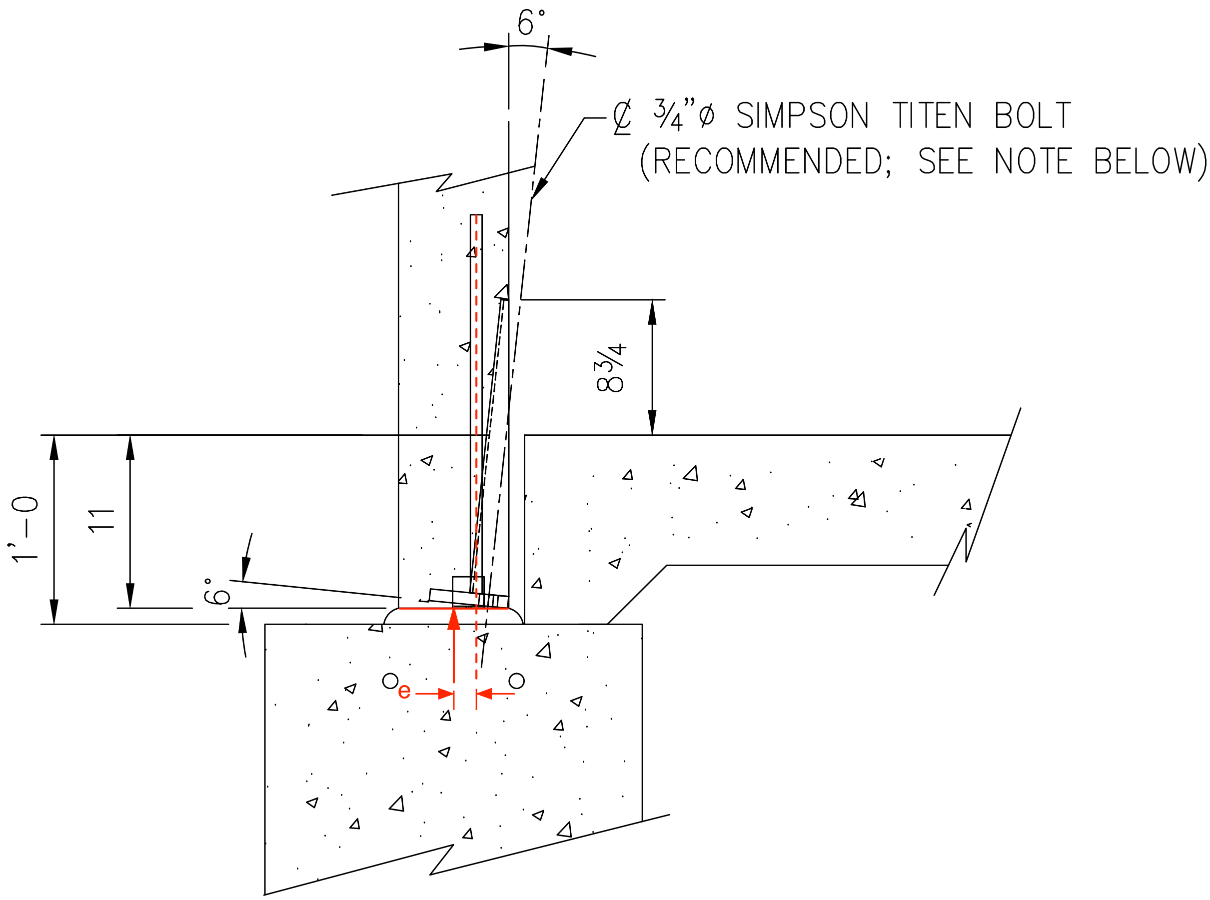


Figure 7. Eccentricity between load point and deformed vertical bar

e = 2 ½"

Titen HD® Design Information — Concrete

Titen HD Tension Strength Design Data[1]

Characteristic	Symbol	Units	Nominal Anchor Diameter, d_a (in.)									
			¼		⅜		½		⅝		¾	
Nominal Embedment Depth	h_{nom}	in.	1⅝	2½	2½	3¼	3¼	4	4	5½	5½	6¼
Steel Strength in Tension												
Tension Resistance of Steel	N_{sa}	lb.	5,195		10,890		20,130		30,360		45,540	
Strength Reduction Factor — Steel Failure	ϕ_{sa}	—	0.65[2]									
Concrete Breakout Strength in Tension[6,8]												
Effective Embedment Depth	h_{ef}	in.	1.19	1.94	1.77	2.40	2.35	2.99	2.97	4.24	4.22	4.86
Critical Edge Distance[6]	c_{ac}	in.	3	6	2¹¹⁄₁₆	3⅝	3⁹⁄₁₆	4½	4½	6 ⅜	6⅜	7⁵⁄₁₆
Effectiveness Factor — Uncracked Concrete	k_{uncr}	—	30	24								
Effectiveness Factor — Cracked Concrete	k_{cr}	—	17									
Modification Factor	$\psi_{c,N}$	—	1.0									
Strength Reduction Factor — Concrete Breakout Failure	ϕ_{cb}	—	0.65[7]									
Pullout Strength in Tension[8]												
Pullout Resistance, Uncracked Concrete (f'_c = 2,500 psi)	$N_{p,uncr}$	lb.	—[3]	—[3]	2,700[4]	—[3]	—[3]	—[3]	—[3]	9,810[4]	—[3]	—[3]
Pullout Resistance, Cracked Concrete (f'_c = 2,500 psi)	$N_{p,cr}$	lb.	—[3]	1,905[4]	1,235[4]	2,700[4]	—[3]	—[3]	3,040[4]	5,570[4]	6,070[4]	7,195[4]
Strength Reduction Factor — Concrete Pullout Failure	ϕ_p	—	0.65[5]									
Breakout or Pullout Strength in Tension for Seismic Applications[8]												
Nominal Pullout Strength for Seismic Loads (f'_c = 2,500 psi)	$N_{p,eq}$	lb.	—[3]	1,905[4]	1,235[4]	2,700[4]	—[3]	—[3]	3,040[4]	5,570[4]	6,070[4]	7,195[4]
Strength Reduction Factor — Breakout or Pullout Failure	ϕ_{eq}	—	0.65[5]									

1. The information presented in this table is to be used in conjunction with the design criteria of ACI 318-14 Chapter 17 and ACI 318-11 Appendix D, except as modified below.
2. The tabulated value of ϕ_{sa} applies when the load combinations of Section 1605.2.1 of the IBC, ACI 318-14 Section 5.3 or ACI 318-11 Section 9.2 are used. If the load combinations of ACI 318-11 Appendix C are used, the appropriate value of ϕ_{sa} must be determined in accordance with ACI 318-11 D.4.4. Anchors are considered brittle steel elements.
3. Pullout strength is not reported since concrete breakout controls.
4. Adjust the characteristic pullout resistance for other concrete compressive strengths by multiplying the tabular value by $(f'_{c,specified} / 2{,}500)^{0.5}$.
5. The tabulated value of ϕ_p or ϕ_{eq} applies when the load combinations of Section 1605.2.1 of the IBC, ACI 318-14 Section 5.3 or ACI 318-11 Section 9.2 are used and the requirements of ACI 318-14 17.3.3.(c) or ACI 318-11 D.4.3(c) for Condition B are met. If the load combinations of ACI 318-11 Appendix C are used, appropriate value of ϕ must be determined in accordance with ACI 318-11 Section D.4.4(c).
6. The modification factor $\psi_{cp,N}$ = 1.0 for cracked concrete. Otherwise, the modification factor for uncracked concrete without supplementary reinforcement to control splitting is either:
   (1) $\psi_{cp,N} = 1.0$ if $c_{a,min} \geq c_{ac}$ or (2) $\psi_{cp,N} = \frac{c_{a,min}}{c_{ac}} \geq \frac{1.5h_{ef}}{c_{ac}}$ if $c_{a,min} < c_{ac}$
   The modification factor, $\psi_{cp,N}$ is applied to the nominal concrete breakout strength, N_{cb} or N_{cbg}.
7. The tabulated value of ϕ_{cb} applies when both the load combinations of Section 1605.2.1 of the IBC, ACI 318-14 Section 5.3 or ACI 318-11 Section 9.2 are used and the requirements of ACI 318-14 17.3.3(c) or ACI 318-11 D.4.3(c) for Condition B are met. Condition B applies where supplementary reinforcement is not provided. For installations where complying supplementary reinforcement can be verified, the ϕ_{cb} factors described in ACI 318-14 17.3.3(c) or ACI 318-11 D.4.3(c) for Condition A are allowed. If the load combinations of ACI 318-11 Appendix C are used, the appropriate value of ϕ_{cb} must be determined in accordance with ACI 318-11 D.4.4(c).

Titen HD® Design Information — Concrete

Titen HD Shear Strength Design Data[1]

Characteristic	Symbol	Units	Nominal Anchor Diameter, d_a (in.)									
			¼[5]		⅜		½		⅝[5]		¾	
Nominal Embedment Depth	h_{nom}	in.	1⅝	2½	2½	3¼	3¼	4	4	5½	5½	6¼
Steel Strength in Shear												
Shear Resistance of Steel	V_{sa}	lb.	2,020		4,460		7,455		10,000		16,840	
Strength Reduction Factor — Steel Failure	ϕ_{sa}	—	0.60[2]									
Concrete Breakout Strength in Shear[6]												
Outside Diameter	d_a	in.	0.25		0.375		0.500		0.625		0.750	
Load Bearing Length of Anchor in Shear	ℓ_e	in.	1.19	1.94	1.77	2.40	2.35	2.99	2.97	4.24	4.22	4.86
Strength Reduction Factor — Concrete Breakout Failure	ϕ_{cb}	—	0.70[4]									
Concrete Pryout Strength in Shear												
Coefficient for Pryout Strength	k_{cp}	lb.	1.0					2.0				
Strength Reduction Factor — Concrete Pryout Failure	ϕ_{cp}	—	0.70[4]									
Steel Strength in Shear for Seismic Applications												
Shear Resistance for Seismic Loads	V_{eq}	lb.	1,695		2,855		4,790		8,000		9,350	
Strength Reduction Factor — Steel Failure	ϕ_{eq}	—	0.60[2]									

1. The information presented in this table is to be used in conjunction with the design criteria of ACI 318-14 Chapter 17 and ACI 318-11 Appendix D, except as modified below.
2. The tabulated value of ϕ_{sa} applies when the load combinations of Section 1605.2.1 of the IBC, ACI 318-14 Section 5.3 or ACI 318-11 Section 9.2 are used. If the load combinations of ACI 318-11 Appendix C are used, the appropriate value of ϕ_{sa} must be determined in accordance with ACI 318 D.4.4.
3. The tabulated value of ϕ_{cb} applies when both the load combinations of Section 1605.2.1 of the IBC, ACI 318-14 Section 5.3 or ACI 318-11 Section 9.2 are used and the requirements of ACI 318-14 17.3.3(c) or ACI 318-11 D.4.3(c) for Condition B are met. Condition B applies where supplementary reinforcement is not provided. For installations where complying supplementary reinforcement can be verified, the ϕ_{cb} factors described in ACI 318-14 17.3.3(c) or ACI 318-11 D.4.3(c) for Condition A are allowed. If the load combinations of ACI 318-11 Appendix C are used, the appropriate value of ϕ_{cb} must be determined in accordance with ACI 318-11 D.4.4(c).
4. The tabulated value of ϕ_{cp} applies when both the load combinations of IBC Section 1605.2, ACI 318-14 5.3 or ACI 318-11 Section 9.2 are used and the requirements of ACI 318-14 17.3.3(c) or ACI 318-11 D.4.3(c) for Condition B are met. If the load combinations of ACI 318-11 Appendix C are used, appropriate value of ϕ_{cp} must be determined in accordance with ACI 318-11 Section D.4.4(c).

Titen HD Tension and Shear Strength Design Data for the Soffit of Normal-Weight or Sand-Lightweight Concrete over Metal Deck[1,6,8]

Characteristic	Symbol	Units	Nominal Anchor Diameter, d_a (in.)									
			Lower Flute						Upper Flute			
			Figure 2		Figure 1				Figure 2		Figure 1	
			¼[8]		⅜		½		¼[8]		⅜	½
Nominal Embedment Depth	h_{nom}	in.	1⅝	2½	1⅞	2½	2	3½	1⅝	2½	1⅞	2
Effective Embedment Depth	h_{ef}	in.	1.19	1.94	1.23	1.77	1.29	2.56	1.19	1.94	1.23	1.29
Pullout Resistance, concrete on metal deck (cracked)[2,3,4]	$N_{p,deck,cr}$	lb.	420	535	375	870	905	2,040	655	1,195	500	1,700
Pullout Resistance, concrete on metal deck (uncracked)[2,3,4]	$N_{p,deck,uncr}$	lb.	995	1,275	825	1,905	1,295	2,910	1,555	2,850	1,095	2,430
Steel Strength in Shear, concrete on metal deck[5]	$V_{sa,\,deck}$	lb.	1,335	1,745	2,240	2,395	2,435	4,430	2,010	2,420	4,180	7,145
Steel Strength in Shear, Seismic	$V_{sa,\,deck,eq}$	lb.	870	1,135	1,434	1,533	1,565	2,846	1,305	1,575	2,676	4,591

1. The information presented in this table is to be used in conjunction with the design criteria of ACI 318-14 Chapter 17 and ACI 318-11 Appendix D, except as modified below.
2. Concrete compressive strength shall be 3,000 psi minimum. The characteristic pullout resistance for greater compressive strengths shall be increased by multiplying the tabular value by $(f'_{c,specified}/3{,}000)^{0.5}$.
3. For anchors installed in the soffit of sand-lightweight or normal-weight concrete over metal deck floor and roof assemblies, as shown in Figure 1 and Figure 2, calculation of the concrete breakout strength may be omitted.
4. In accordance with ACI 318-14 Section 17.4.3.2 or ACI 318-11 Section D.5.3.2, the nominal pullout strength in cracked concrete for anchors installed in the soffit of sand-lightweight or normal-weight concrete over metal deck floor and roof assemblies $N_{p,deck,cr}$ shall be substituted for $N_{p,cr}$. Where analysis indicates no cracking at service loads, the normal pullout strength in uncracked concrete $N_{p,deck,uncr}$ shall be substituted for $N_{p,uncr}$.
5. In accordance with ACI 318-14 Section 17.5.1.2(C) or ACI 318-11 Section D.6.1.2(c), the shear strength for anchors installed in the soffit of sand-lightweight or normal-weight concrete over metal deck floor and roof assemblies $V_{sa,deck}$ and $V_{sa,deck,eq}$ shall be substituted for V_{sa}.
6. Minimum edge distance to edge of panel is $2h_{ef}$.
7. The minimum anchor spacing along the flute must be the greater of $3h_{ef}$, or 1.5 times the flute width.