Full-Scale Testing of SC Series

CONNECT-EZ®

SIMPLE. SAFE. SECURE.™

Bahram M. Shahrooz, PE, FACI, FASCE, FSEI
Director of University of Cincinnati Large Scale Test Facility

Table of Contents

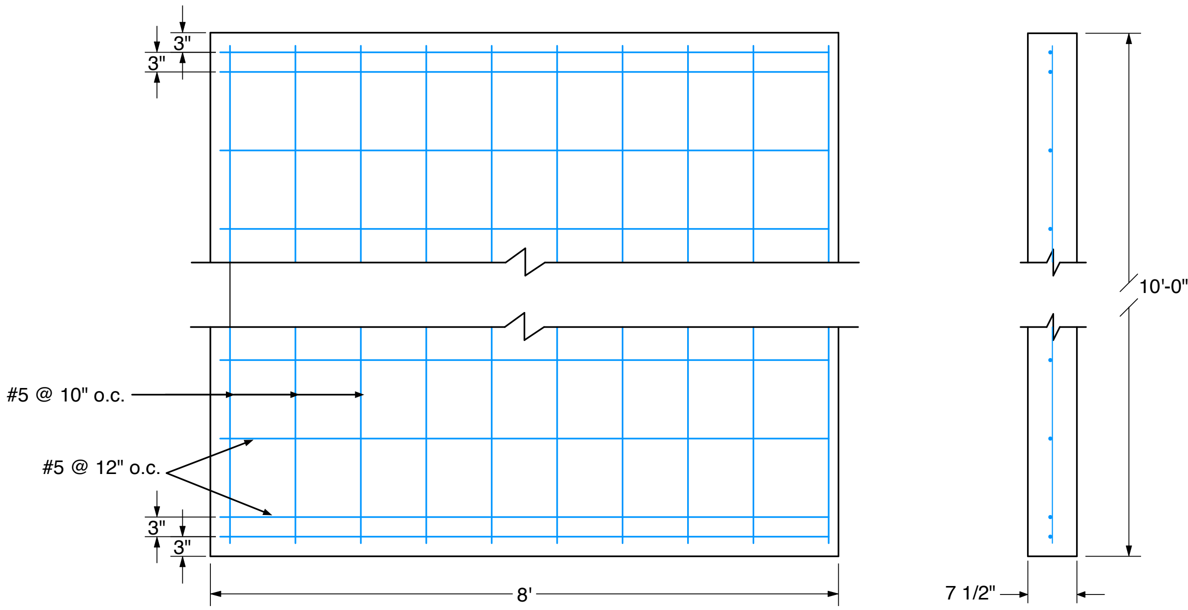
List of Tables

List of Figures

Addendum

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

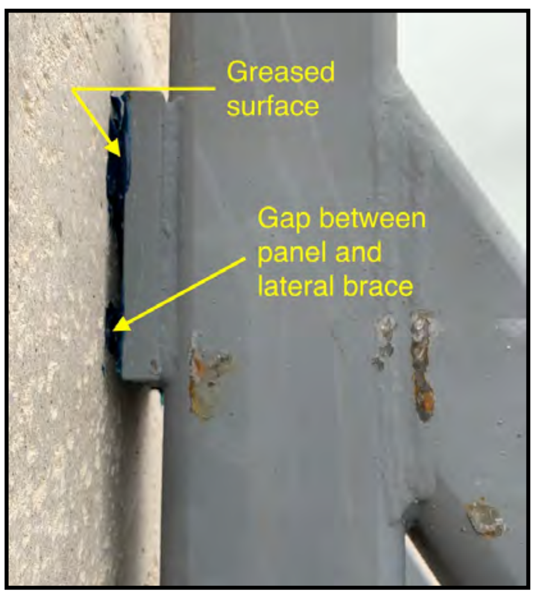
(b) Panels before concrete placement

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 this series of tests was on SC-a and SC-b. The plate dimensions and bolt gages are different between these two devices as shown in Table 1, but both devices use 3/8” thick plates. In the first series, Panel 2 was connected to Panels 1 and 3 with SC-a, as shown in Figure 3. SC-b was used to connect the panels for the second series of tests. The devices for the two tests were 4’-0” apart so testing of one device will not impact the result of the other device. Each device had a 3’-0” edge distance.

Table 1. Dimensions of connection plates

Device	Plate dimensions	Bolt gages
SC-a	13" wide x 6" deep	Horizontal gage = 10" Vertical gage = 3"
SC-b	15" wide x 11" deep	Horizontal gage = 12" Vertical gage = 8"

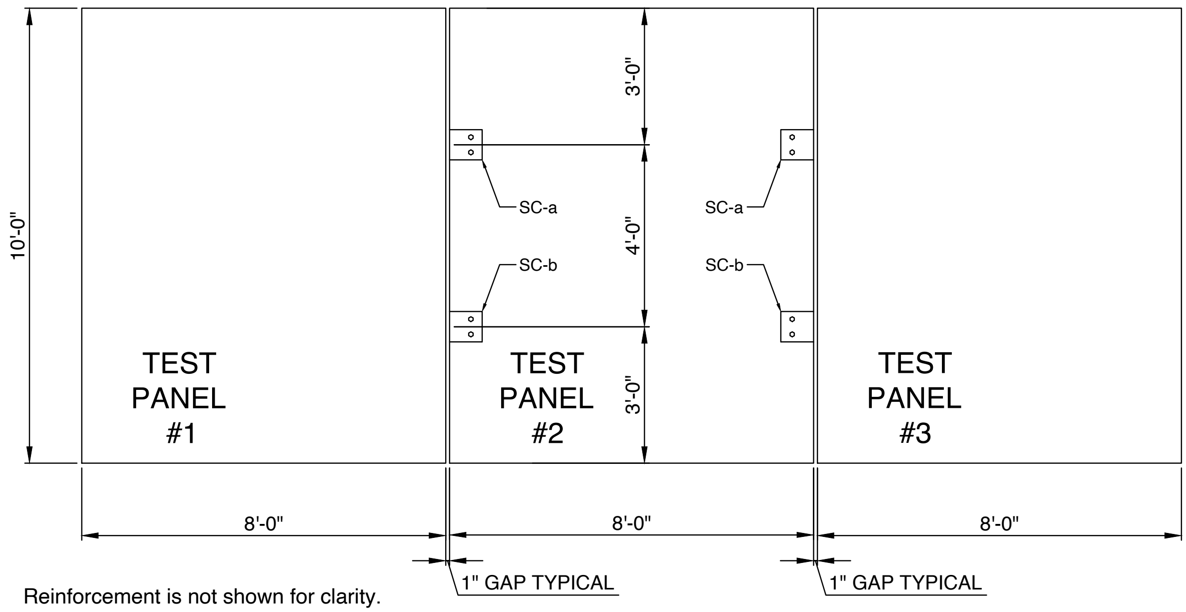


Figure 3. Locations of SC-a and SC-b

2. Test setup

The test setup is shown in Figure 4. Pockets had been cast in the foundation to accommodate hydraulic rams. A 60-kip hydraulic ram was placed vertically in a pocket in the middle of the Panel 2 to apply a vertical uplift force. The load was transferred to the panel through a bearing plate centered on the panel thickness. Each stationary panel (Panel 1 and Panel 3) was held in place by four 5/8" A193-B7 threaded rods that were threaded into anchors cast in the foundation and restrained at the top by two strong tie backs consisting of a pair of back-to-back channels.

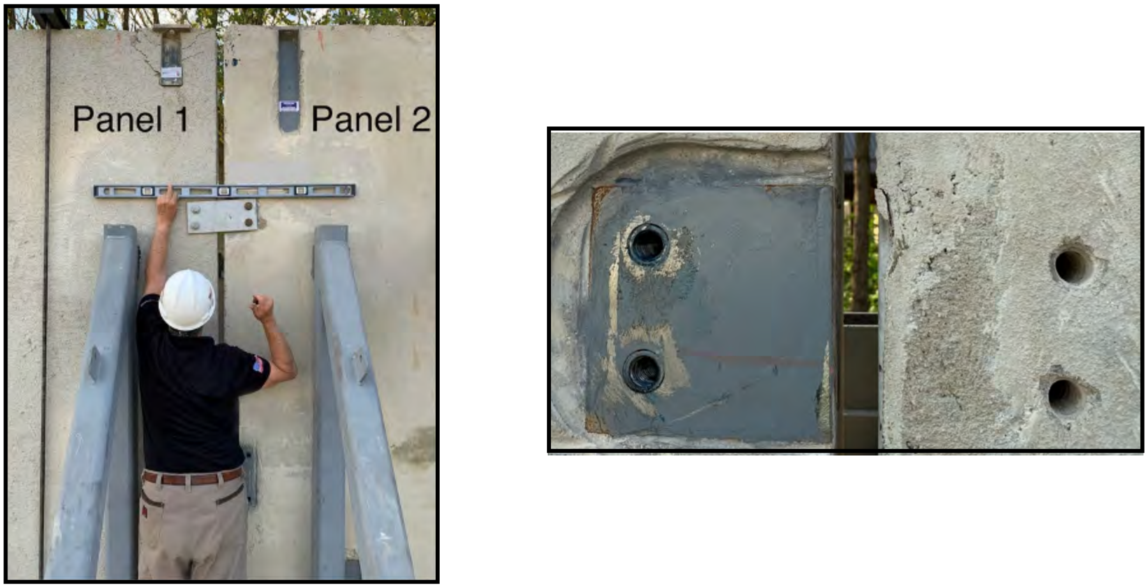
Figure 4. Test setup

3. Results and discussion

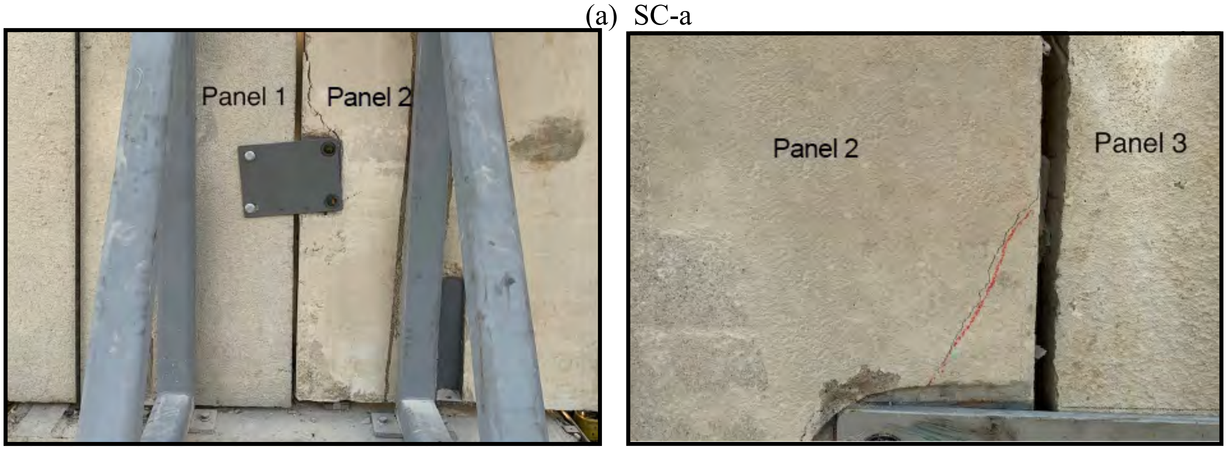
The maximum load resisted by each device is tabulated in Table 2. As evident from Figure 5, the connection plates in both devices rotated. This rotation is expected considering that the holes for the post-installed anchors (i.e., those in Panels 1 and 3) are slotted. Furthermore, the structural bolts (i.e., those in Panel 2) are not meant to be slip critical. No cracks were observed around SC-a, but diagonal cracks radiated from the top structural bolt which was subjected to tension as the plates rotated.

Table 2. Peak loads

Device	Maximum load (kips)
SC-a	10
SC-b	10



(a) SC-a



(b) SC-b

Figure 5. Condition of devices after resisting 10 kips

4. Summary and observations

Both SC-a and SC-b resisted 10 kips suggesting the larger bolt gages in SC-b did not affect the capacity. The larger connection plates and bolt gages in SC-b did not impact the capacity.

SIMPSON Strong-Tie®

Titen HD® Design Information — Concrete

Titen HD Tension Strength Design Data[1]

IBC *

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

SIMPSON Strong-Tie®

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.