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# ICC-ES Evaluation Report

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## ESR-2461

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**DIVISION: 03 00 00—CONCRETE**

**SECTION: 03 16 00—CONCRETE ANCHORS**

**DIVISION: 05 00 00—METALS**

**SECTION: 05 05 19—POST-INSTALLED CONCRETE ANCHORS**

**REPORT HOLDER:**

**MKT METALL-KUNSTSTOFF-TECHNIK GMBH & CO. KG**

**EVALUATION SUBJECT:**

**MKT SRS TZ ANCHORS FOR CRACKED AND UNCRACKED CONCRETE**



*“2014 Recipient of Prestigious Western States Seismic Policy Council (WSSPC) Award in Excellence”*



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**DIVISION: 03 00 00—CONCRETE**  
**Section: 03 16 00—Concrete Anchors**

**DIVISION: 05 00 00—METALS**  
**Section: 05 05 19—Post-Installed Concrete Anchors**

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**EVALUATION SUBJECT:**

**MKT SRS TZ ANCHORS FOR CRACKED AND  
UNCRACKED CONCRETE**

**1.0 EVALUATION SCOPE**

**Compliance with the following codes:**

- 2009 and 2006 *International Building Code*® (IBC)
- 2009 and 2006 *International Residential Code*® (IRC)

**Property evaluated:**

Structural

**2.0 USES**

The MKT SRS TZ carbon and stainless steel expansion anchors are used to resist static, wind and seismic (Seismic Design Categories A through F) tension and shear loads in cracked and uncracked normal-weight or lightweight concrete having a specified compressive strength,  $f'_c$ , ranging from 2,500 psi to 8,500 psi (17.2 MPa to 58.6 MPa). The MKT SRS TZ carbon steel expansion anchors are also used to resist static, wind and seismic tension and shear loads in cracked and uncracked normal-weight or sand-lightweight concrete over metal deck having a minimum specified compressive strength,  $f'_c$ , of 3,000 psi (20.7 MPa).

The MKT SRS TZ anchoring system complies with anchors as described in Section 1912 of the 2009 and 2006 IBC. The anchor system is an alternative to cast-in-place anchors described in Section 1911 of the 2009 and 2006 IBC. The anchors may also be used where an engineered design is submitted in accordance with Section R301.1.3 of the IRC.

**3.0 DESCRIPTION**

**3.1 MKT SRS TZ Anchors:**

**3.1.1 General:** SRS TZ anchors are torque-controlled, mechanical expansion anchors consisting of a stud anchor body, expansion clip, nut, and washer. The anchor is illustrated in Figure 1.

The stud anchor body has a tapered mandrel formed on the installed end of the anchor, and a threaded section at the opposite end. The taper of the mandrel increases in diameter toward the installed end of the anchor. The three-segment expansion clip wraps around the tapered mandrel. Before installation, this expansion clip is free to rotate about the mandrel. The anchor is installed in a predrilled hole. When the anchor is set using an applied torque to the hex nut, the mandrel is drawn into the expansion clip, which engages the drilled hole and transfers the load to the base material. Pertinent dimensions are as set forth in Table 1.

**3.1.2 SRS TZ Carbon Steel:** The stud elements of carbon steel anchors are manufactured from carbon steel complying with DIN EN 10263-3 and have a minimum 5  $\mu$ m (0.0002 inch) zinc plating. The expansion clip elements of the carbon steel anchors are fabricated from stainless steel complying with AISI/SAE 304 or 316. The carbon steel hex nut complies with ASTM A563, Grade A. The carbon steel washer complies with ANSI/ASME B18.22.1 or EN ISO 7089.

**3.1.3 SRS TZ 304 Stainless Steel:** The stud elements of 304 stainless steel anchors are manufactured from stainless steel complying with AISI/SAE 304 or 316. The expansion clip elements are fabricated from stainless steel complying with AISI/SAE 304 or 316. The stainless steel hex nuts comply with the chemical and dimensional requirements of ASTM F594, 304 alloy group. The stainless steel washer complies with ANSI/ASME B18.22.1 or EN ISO 7089, 304 alloy group.

**3.1.4 SRS TZ 316 Stainless Steel:** The stud elements of 316 stainless steel anchors are manufactured from stainless steel complying with AISI/SAE 316. The expansion clip elements are fabricated from stainless steel complying with AISI/SAE 316. The stainless steel hex nuts comply with the chemical and dimensional requirements of ASTM F594, 316 alloy group. The stainless steel washer complies with ANSI/ASME B18.22.1 or EN ISO 7089, 316 alloy group.

**3.2 Concrete:**

Normal-weight and lightweight concrete must comply with Sections 1903 and 1905 of the IBC.

**3.3 Steel Deck Panels:**

Steel deck panels must comply with the configuration in Figure 4 and have a minimum base-metal thickness of 0.035 inch (0.889 mm) [20 gage]. Steel deck must comply with the requirements of ASTM A653/A653M SS Grade 33, and have a minimum yield strength of 33 ksi (228 MPa).

## 4.0 DESIGN AND INSTALLATION

### 4.1 Strength Design:

**4.1.1 General:** Design strength of anchors complying with the 2009 IBC as well as Section R301.1.3 of the 2009 IRC, must be determined in accordance with ACI 318-08 Appendix D and this report.

Design strength of anchors complying with the 2006 IBC and Section R301.1.3 of 2006 IRC must be in accordance with ACI 318-05 Appendix D and this report.

Design examples according to the 2009 IBC are given in Figure 5 of this report. Design parameters are based on the 2009 IBC (ACI 318-08) unless noted otherwise in Sections 4.1.1 through 4.1.12 of this report. The strength design of anchors must comply with ACI 318 D.4.1, except as required in ACI 318 D.3.3.

Strength reduction factors,  $\phi$ , as given in ACI 318 D.4.4 must be used for load combinations calculated in accordance with Section 1605.2.1 of the IBC and Section 9.2 of ACI 318. Strength reduction factors,  $\phi$ , as given in ACI 318 D.4.5 must be used for load combinations calculated in accordance with ACI 318 Appendix C.

The value of  $f'_c$  used in the calculations must be limited to 8,000 psi (55.2 MPa), maximum, in accordance with ACI 318 D.3.5.

Design parameters are provided in Tables 3 and 4 of this report. Strength reduction factors,  $\phi$ , corresponding to ductile steel elements may be used.

**4.1.2 Requirements for Static Steel Strength in Tension:** The nominal steel strength in tension,  $N_{sa}$ , in accordance with ACI 318 D.5.1.2, is given in Table 3 of this report. Strength reduction factors,  $\phi_{sa}$ , corresponding to ductile steel elements as described in Table 3 must be used.

**4.1.3 Requirements for Static Concrete Breakout Strength in Tension:** The nominal concrete breakout strength of a single anchor or group of anchors in tension,  $N_{cb}$  and  $N_{cbg}$ , respectively, must be calculated according to ACI 318 D.5.2.1, modified as described in this section. The basic concrete breakout strength in tension,  $N_b$ , must be calculated according to ACI 318 D.5.2.2 using the values of  $h_{ef}$  and  $k_{cr}$  as given in Table 3. The value of  $f'_c$  used in the calculations must be limited to a maximum of 8,000 psi (55.2 MPa), in accordance with ACI 318 D.3.5. The nominal concrete breakout strength in tension in regions where analysis indicates no cracking at service loads in accordance with ACI 318 D.5.2.6 must be calculated with  $\psi_{c,N} = 1.0$  and  $k_{uncr}$  as given in Table 3.

For SRS TZ carbon steel anchors installed in the soffit of sand-lightweight or normal-weight concrete over profile steel deck floor and roof assemblies, as shown in Figure 4, determination of the concrete breakout strength in accordance with ACI 318 D.5.2 is not required.

**4.1.4 Requirements for Static Pullout Strength in Tension:** The nominal pullout strength of a single anchor in tension in accordance with ACI D.5.3 in cracked and uncracked concrete,  $N_{p,cr}$  and  $N_{p,uncr}$ , respectively, is given in Table 3 of this report. For all design cases,  $\psi_{c,P} = 1.0$ .

The nominal pullout strength in cracked concrete may be adjusted by calculations according to Eq-1:

$$N_{p,f't_c} = N_{p,cr} \left( \frac{f't_c}{2,500} \right)^{0.5} \quad (\text{lb, psi}) \quad (\text{Eq.-1})$$

$$N_{p,f't_c} = N_{p,cr} \left( \frac{f't_c}{17.2} \right)^{0.5} \quad (\text{N, MPa})$$

where  $f'_c$  is the specified concrete compressive strength.

In regions of a concrete member where analysis indicates no cracking in accordance with ACI 318 D.5.3.6, the nominal pullout strength in tension can be adjusted by calculations according to Eq-2.

$$N_{p,f't_c} = N_{p,uncr} \left( \frac{f't_c}{2,500} \right)^{0.5} \quad (\text{lb, psi}) \quad (\text{Eq.-2})$$

$$N_{p,f't_c} = N_{p,uncr} \left( \frac{f't_c}{17.2} \right)^{0.5} \quad (\text{N, MPa})$$

Where values for nominal pullout strengths,  $N_{p,cr}$  or  $N_{p,uncr}$ , are not provided for the  $5/8$ -inch-diameter (15.9 mm) anchors in Table 3, the pullout strength does not need to be considered.

The pullout strength in cracked concrete of the carbon steel anchors installed in the soffit of sand-lightweight or normal-weight concrete over profile steel deck floor and roof assemblies, as shown in Figure 4, is provided in Table 5. The nominal pullout strength in cracked concrete must be calculated according to Eq-1, whereby the value  $N_{p,deck,cr}$  must be substituted for  $N_{p,cr}$  and the value of 3,000 psi (20.7 MPa) must be substituted for the value of 2,500 psi (17.2 MPa) in the denominator. In regions of a concrete member where analysis indicates no cracking in accordance with ACI 318 D.5.3.6, the nominal pullout strength in uncracked concrete must be calculated according to Eq-2, whereby the value of  $N_{p,deck,uncr}$  must be substituted for  $N_{p,uncr}$  and the value of 3,000 psi (20.7 MPa) must be substituted for the value of 2,500 psi (17.2 MPa) in the denominator. The use of SRS TZ stainless steel anchors installed in the soffit of concrete on steel deck assemblies is beyond the scope of this report. For all design cases  $\psi_{c,p} = 1.0$ .

**4.1.5 Requirements for Static Steel Strength in Shear:** The nominal steel strength in shear,  $V_{sa}$ , in accordance with ACI 318 D.6.1.2, is given in Table 4 of this report and must be used in lieu of the values derived by calculations from ACI 318, Eq. D-20. Strength reduction factors,  $\phi_{sa}$ , corresponding to ductile steel elements as described in Table 4 must be used.

The shear strength,  $V_{sa,deck}$ , of anchors installed in the soffit of sand-lightweight or normal-weight concrete over profile steel deck floor and roof assemblies, as shown in Figure 4, is given in Table 5.

**4.1.6 Requirements for Static Concrete Breakout Strength of Anchor in Shear:** The nominal concrete breakout strength of an anchor or group of anchors in shear,  $V_{cb}$  and  $V_{cbg}$ , respectively, must be calculated in accordance with ACI 318 D.6.2, modified as described in this section. The basic concrete breakout strength in shear,  $V_b$ , must be calculated in accordance with ACI 318 D.6.2.2 using the values of  $l_e$  and  $d_a$  described in Table 4 of this report. The value of  $f'_c$  used for calculation purposes must not exceed 8,000 psi (55.2 MPa) in accordance with ACI 318 D.3.5.

For anchors installed in the soffit of sand-lightweight or normal-weight concrete over profile steel deck floor and roof assemblies, as shown in Figure 4, calculation of the concrete breakout strength in accordance with ACI 318 D.6.2 is not required.

**4.1.7 Requirements for Static Concrete Pryout Strength of Anchor in Shear:** Static nominal concrete pryout shear strengths of an anchor or group of anchors,  $V_{cp}$  and  $V_{cpg}$ , respectively, must be calculated in accordance with ACI 318 D.6.3, modified by using the value of  $k_{cp}$  described in Table 4 of this report and the value of  $N_{cb}$  or  $N_{cbg}$  as calculated in accordance with Section 4.1.3 of this report.

For anchors installed in the soffit of sand-lightweight or normal-weight concrete over profile steel deck floor and roof assemblies, as shown in Figure 4, calculation of the concrete pryout strength in accordance with ACI 318 D.6.3 is not required.

**4.1.8 Requirements for Seismic Design:** For load combinations including seismic, the design must be performed in accordance with ACI 318 D.3.3 as modified by Section 1908.1.9 of the 2009 IBC, Section 1908.1.16 of the 2006 IBC.

The anchors comply with ACI 318 D.1 as ductile steel elements and must be designed in accordance with ACI 318-08 D.3.3.4, D.3.3.5 and D.3.3.6 or ACI 318-05 D.3.3.4 or D.3.3.5.

**4.1.8.1 Seismic Tension:** The nominal steel strength and the nominal concrete breakout strength for anchors in tension must be calculated in accordance with ACI 318 D.5.1 and D.5.2 as described in Sections 4.1.2 and 4.1.3 of this report and taking into account the corresponding values given in Tables 1, 3, 4 and 5.

In accordance with ACI 318 D.5.3.2, the appropriate value for nominal pullout strength in tension for seismic loads,  $N_{p,eq}$  or  $N_{p,deck,cr}$ , described in Tables 3 and 5, must be used in lieu of  $N_{pn}$ . The values of  $N_{p,eq}$  can be adjusted by calculations according to Eq-3:

$$N_{p,eq,f'c} = N_{p,eq} \left( \frac{f'c}{2,500} \right)^{0.5} \quad (\text{lb, psi}) \quad (\text{Eq.-3})$$

$$N_{p,eq,f'c} = N_{p,eq} \left( \frac{f'c}{17.2} \right)^{0.5} \quad (\text{N, MPa})$$

In addition, for anchors installed in the soffit of sand-lightweight or normal-weight concrete over profile steel deck floor and roof assemblies, the nominal pullout strength in tension for seismic loads must be calculated according to Eq-3, whereby the value  $N_{p,deck,cr}$  must be substituted for  $N_{p,eq}$  and the value of 3,000 psi (20.7 MPa) must be substituted for the value of 2,500 psi (17.2 MPa) in the denominator.

A value for nominal pullout strengths,  $N_{p,eq}$ , is not provided for the  $\frac{5}{8}$ -inch-diameter (15.9 mm) anchors in Table 3, hence the pullout strength in tension for seismic loads does not need to be considered.

**4.1.8.2 Seismic Shear:** The nominal concrete breakout strength and pryout strength for anchors in shear must be calculated in accordance with ACI 318 D.6.2 and D.6.3, as described in Sections 4.1.6 and 4.1.7 of this report. In accordance with ACI 318 D.6.1.2, the appropriate value for nominal steel strength in shear for seismic loads,  $V_{sa,eq}$  or  $V_{sa,deck,eq}$ , described in Tables 4 and 5, must be used in lieu of  $V_{sa}$ .

**4.1.9 Interaction of Tensile and Shear Forces:** The interaction of tensile and shear forces must be calculated in accordance with ACI 318 Section D.7.

**4.1.10 Requirements for Critical Edge Distance:** In applications where  $c < c_{ac}$  and supplemental reinforcement to control splitting of the concrete is not present, the concrete breakout strength in tension for uncracked concrete, calculated according to ACI 318 D.5.2, must be further multiplied by the factor of  $\Psi_{cp,N}$  given by the following equation:

$$\Psi_{cp,N} = \frac{c}{c_{ac}} \quad (\text{Eq.-4})$$

whereby the factor  $\Psi_{cp,N}$  need not be taken as less than  $\frac{1.5h_{ef}}{c_{ac}}$ . For all other cases  $\Psi_{cp,N}=1.0$ . In lieu of ACI 318 D.8.6, the critical edge distance,  $c_{ac}$ , required to develop the basic concrete breakout strength of a post-installed

anchor in uncracked concrete without supplementary reinforcement, given in Table 3, must be used.

**4.1.11 Requirements for Minimum Member Thickness, Minimum Anchor Spacing, and Minimum Edge Distance:** In lieu of ACI 318 D.8.1 and D.8.3, values of  $c_{min}$  and  $s_{min}$  as given in Table 1 of this report must be used. In lieu of ACI 318 D.8.5, minimum member thicknesses,  $h_{min}$ , as given in Table 1 of this report must be used. Additional combinations for minimum edge distance,  $c_{min}$  and spacing,  $s_{min}$  may be derived by linear interpolation between the given boundary values. The interpolation is shown in Figure 3. For anchors installed through the soffit of steel deck assemblies, the minimum anchor spacing along the flute must be the greater of  $3.0h_{ef}$  or 1.5 times the flute width in accordance with Figure 4.

**4.1.12 Lightweight Concrete:** For the use of anchors in lightweight concrete, the modification factor  $\lambda_a$  equal to  $0.8\lambda$  is applied to all values of  $\sqrt{f'c}$  affecting  $N_n$  and  $V_n$ .

For ACI 318-08 (2009 IBC),  $\lambda$  shall be determined in accordance with the corresponding version of ACI 318.

For ACI 318-05 (2006 IBC),  $\lambda$  shall be taken as 0.75 for all lightweight concrete and 0.85 for sand-lightweight concrete. Linear interpolation shall be permitted if partial sand replacement is used. In addition, the pullout strengths  $N_{p,cr}$ ,  $N_{p,uncr}$ , and  $N_{eq}$  shall be multiplied by the modification factor,  $\lambda_a$ , as applicable.

For anchors installed in the soffit of sand-lightweight concrete-filled steel deck and floor and roof assemblies, further reduction of the pullout values provided in this report is not required.

**4.2 Allowable Stress Design (ASD):**

**4.2.1 General:** Design strengths for use with allowable stress design (ASD) load combinations calculated in accordance with Section 1605.3 of the IBC must be established using the following relationships:

$$T_{allowable, ASD} = \frac{\phi N_n}{\alpha} \quad (\text{Eq.-5})$$

and

$$V_{allowable, ASD} = \frac{\phi V_n}{\alpha} \quad (\text{Eq.-6})$$

where:

$T_{allowable, ASD}$  = Allowable tension load (lbf or N).

$V_{allowable, ASD}$  = Allowable shear load (lbf or N).

$\phi N_n$  = The lowest design strength of an anchor or anchor group in tension as determined in accordance with ACI 318 Appendix D, 2009 IBC Section 1908.1.9 or 2006 IBC Section 1908.1.16, and Section 4.1 of this report (lbf or N).

$\phi V_n$  = The lowest design strength of an anchor or anchor group in shear as determined in accordance with ACI 318 Appendix D, 2009 IBC Section 1908.1.9 or 2006 IBC Sections 1908.1.16, and Section 4.1 of this report (lbf or N).

$\alpha$  = A conversion factor calculated as a weighted average of the load factors for the controlling load combination. In addition,  $\alpha$  must include all applicable factors to account for non-ductile failure modes and required over-strength.

The requirements for member thickness, edge distance and spacing, as described in this report, must apply. Table 6 shows an example of calculated ASD values for each anchor diameter and embedment.

#### 4.2.2 Interaction of Tensile and Shear Forces:

Interaction of tensile and shear loads must be calculated as follows:

If  $T_{applied} \leq 0.2T_{allowable,ASD}$ , then the full allowable strength in shear,  $V_{allowable,ASD}$ , shall be permitted.

If  $V_{applied} \leq 0.2V_{allowable,ASD}$ , then the full allowable strength in tension,  $T_{allowable,ASD}$ , shall be permitted.

$$\text{For all other cases: } \frac{T_{applied}}{T_{allowable,ASD}} + \frac{V_{applied}}{V_{allowable,ASD}} \leq 1.2 \quad (\text{Eq.-7})$$

#### 4.3 Installation:

Installation parameters are provided in Table 1, Figure 2 and Figure 4. Anchor locations must comply with this report and the plans and specifications approved by the code official. MKT SRS TZ anchors must be installed in accordance with manufacturer's published instructions and this report. Embedment, spacing, edge distance, and concrete thickness must comply with Table 3, 4 or 5, as applicable. Holes must be predrilled in concrete with a compressive strength from 2,500 to 8,500 psi (17.2 MPa to 58.6 MPa), using carbide-tipped masonry drill bits manufactured within the range of the maximum and minimum drill tip dimensions of ANSI B212.15-1994. The nominal diameter of the carbide-tipped drill bit must equal the nominal anchor diameter. Holes must be created by drilling to the required hole depth,  $h_o$ , as described in Table 1. The holes must be cleaned with a hand pump, blow-out bulb, or compressed air to remove loose debris. The nut and washer must be assembled on the end of the anchor, leaving the nut slightly below the end of the anchor. The anchors are then driven with a hammer through the fixture to the required embedment depth in concrete. The nut and washer must be tightened against the base material or material to be fastened until the required installation torque value,  $T_{inst}$ , as specified in Table 1 is achieved.

For installation in the soffit of normal-weight or sand-lightweight concrete over profile steel deck floor and roof assemblies, the diameter of the hole in the steel deck must not exceed the diameter of the hole in the concrete by more than  $\frac{1}{8}$  inch (3.2 mm). Installation torque values for anchors installed in the soffit of sand-lightweight or normal-weight concrete over profile metal deck floor and roof assemblies, must comply with values in Table 5.

#### 4.4 Special Inspection:

Periodic special inspection is required in accordance with Section 1704.15 of the 2009 IBC or Section 1704.13 of the 2006 IBC. The special inspector must make periodic inspections during anchor installation to verify anchor type, anchor dimensions, concrete type, concrete compressive strength, hole dimensions, anchor spacing, edge distances, concrete member thickness, anchor embedment, tightening torque, and adherence to the manufacturer's published installation instructions. The special inspector must be present as often as required in accordance with the "statement of special inspection".

Under the IBC, additional requirements as set forth in Sections 1705, 1706 and 1707 must be observed, where applicable.

#### 5.0 CONDITIONS OF USE

The MKT SRS TZ Steel Anchors described in this report comply with, or are suitable alternatives to what is specified in, those codes listed in Section 1.0 of this report, subject to the following conditions:

- 5.1 Anchor sizes, dimensions, and installation parameters are as set forth in this report.
- 5.2 The anchors must be installed in accordance with the

manufacturer's published instructions and this report. In case of conflicts, this report governs.

- 5.3 The anchors must be installed in cracked and uncracked, normal-weight or lightweight concrete having a specified compressive strength,  $f'_c$ , of 2,500 psi to 8,500 psi (17.2 MPa to 58.6 MPa); and in cracked and uncracked sand-lightweight or normal-weight concrete over profile metal deck having a minimum specified compressive strength,  $f'_c$ , of 3,000 psi (20.7 MPa). The use of SRS TZ stainless steel anchors installed in the soffit of concrete on steel deck assemblies is beyond the scope of this report.
- 5.4 The values of  $f'_c$  used for calculation purposes must not exceed 8,000 psi (55.2 MPa).
- 5.5 Strength design values must be established in accordance with Section 4.1 of this report.
- 5.6 Allowable stress design values must be established in accordance with Section 4.2 of this report.
- 5.7 Anchor spacing, edge distance, and minimum member thickness must comply with Tables 1 and 3 and Figures 2, 3 and 4 of this report.
- 5.8 Prior to installation, calculations and details demonstrating compliance with this report must be submitted to the code official for approval. The calculations and details must be prepared by a registered design professional where required by the statutes of the jurisdiction in which the project is to be constructed.
- 5.9 Since an ICC-ES acceptance criteria for evaluating data to determine the performance of expansion anchors subjected to fatigue or shock loading is unavailable at this time, the use of these anchors under such conditions is beyond the scope of this report.
- 5.10 Anchors may be installed in regions of concrete where cracking has occurred or where analysis indicates cracking may occur ( $f_t > f_r$ ), subject to the conditions of this report.
- 5.11 Anchors may be used to resist short-term loading due to wind or seismic forces in locations designated as Seismic Design Categories A through F under the IBC subject to the conditions of this report.
- 5.12 Where not otherwise prohibited in the applicable code, MKT SRS TZ anchors are permitted for use with fire-resistance-rated construction provided that at least one of the following conditions is fulfilled:
  - Anchors are used to resist wind or seismic forces only.
  - Anchors that support a fire-resistance-rated envelope or a fire-resistance-rated membrane are protected by approved fire-resistance-rated materials, or have been evaluated for resistance to fire exposure in accordance with recognized standards.
  - Anchors are used to support nonstructural elements.
- 5.13 Use of zinc-coated carbon-steel anchors is limited to dry, interior locations.
- 5.14 Special inspections must be provided in accordance with Section 4.4 of this report.
- 5.15 Anchors are manufactured in Weilerbach, Germany, under an approved quality-control program with inspections by ICC-ES.

**6.0 EVIDENCE SUBMITTED**

Data complying with the ICC-ES Acceptance Criteria for Mechanical Anchors in Concrete and Masonry Elements (AC193), dated October 2015.

**7.0 IDENTIFICATION**

7.1 Anchors are identified by packaging labeled with the anchor name and size, the manufacturer’s name (MKT) and contact information, the evaluation report number (ESR-2461). The MKT SRS TZ anchors have the letters SRS-TZ embossed on the threaded end of the stud anchor body. Length identification is required to be visible after installation (see Table 2).

7.2 The report holder’s contact information is the following:

**MKT METALL-KUNSTSTOFF-TECHNIK GmbH & Co. KG**  
**AUF DEM IMMEL 2**  
**WEILERBACH, 67685**  
**GERMANY**  
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**TABLE 1—SRS TZ INSTALLATION INFORMATION<sup>1</sup>**

SETTING INFORMATION	SYMBOL	UNITS	NOMINAL ANCHOR DIAMETER	
			<sup>1</sup> / <sub>2</sub> inch	<sup>5</sup> / <sub>8</sub> inch
Nominal Diameter <sup>5</sup>	$d_a$	in.	<sup>1</sup> / <sub>2</sub>	<sup>5</sup> / <sub>8</sub>
Drill Bit Diameter	$d_{bit}$	in.	<sup>1</sup> / <sub>2</sub>	<sup>5</sup> / <sub>8</sub>
Minimum hole depth	$h_o$	in.	3 <sup>1</sup> / <sub>4</sub>	4 <sup>1</sup> / <sub>8</sub>
Minimum Baseplate Clearance Hole Diameter <sup>2</sup>	$d_c$	in.	<sup>9</sup> / <sub>16</sub>	<sup>11</sup> / <sub>16</sub>
Installation Torque (Carbon Steel)	$T_{inst}$	ft-lbf	35	65
Installation Torque (304 Stainless Steel)	$T_{inst}$	ft-lbf	60	110
Installation Torque (316 Stainless Steel)	$T_{inst}$	ft-lbf	60	96
Nominal Embedment depth	$h_{nom}$	in.	2 <sup>7</sup> / <sub>8</sub>	3 <sup>3</sup> / <sub>4</sub>
Effective Embedment Depth	$h_{ef}$	in.	2 <sup>1</sup> / <sub>2</sub>	3 <sup>1</sup> / <sub>4</sub>
Minimum Edge Distance	$c_{min1}$	in.	5	6
Minimum Spacing <sup>3</sup>	$s_{min1}$	in.	2 <sup>1</sup> / <sub>2</sub>	3
Minimum Edge Distance	$c_{min2}$	in.	3	3 <sup>1</sup> / <sub>2</sub>
Minimum Spacing <sup>4</sup>	$s_{min2}$	in.	6	9 <sup>1</sup> / <sub>2</sub>
Minimum Concrete Thickness	$h_{min}$	in.	5	6 <sup>1</sup> / <sub>2</sub>

For **SI**: 1 inch = 25.4 mm, 1 ft-lbf = 1.356 N-m, 1 psi = 6.89 Pa, 1 in<sup>2</sup> = 645 mm<sup>2</sup>, 1 lb/in = 0.175 N/mm.

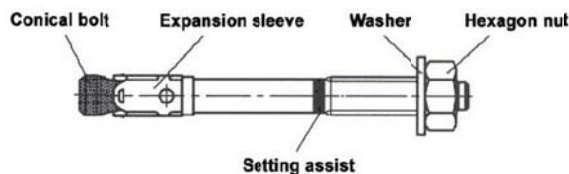
<sup>1</sup>The information presented in this table must be used in conjunction with the design requirements of ACI 318 Appendix D.

<sup>2</sup>The clearance must comply with applicable code requirements for the connected element.

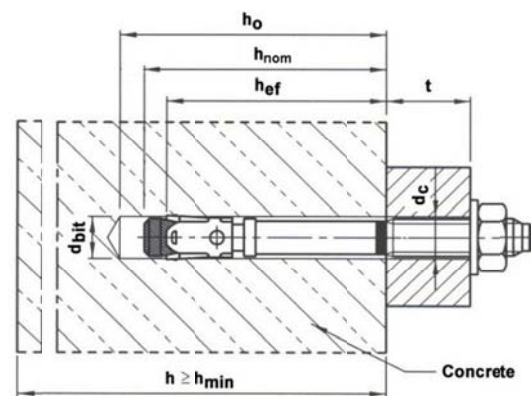
<sup>3</sup> $s_{min1}$  applies when  $c_{min1}$  is provided.

<sup>4</sup> $s_{min2}$  applies when  $c_{min2}$  is provided.

<sup>5</sup> $d_a = d_o$  (2006 IBC)



**FIGURE 1—MKT SRS TZ ANCHOR**



t=Thickness of base plate

**FIGURE 2—MKT SRS TZ ANCHOR (INSTALLED)**

**TABLE 2—LENGTH IDENTIFICATION SYSTEM (CARBON STEEL AND STAINLESS STEEL ANCHORS)**

Length ID marking on stud	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W
Length of anchor min ≥ (in.)	2 <sup>1</sup> / <sub>2</sub>	3	3 <sup>1</sup> / <sub>2</sub>	4	4 <sup>1</sup> / <sub>2</sub>	5	5 <sup>1</sup> / <sub>2</sub>	6	6 <sup>1</sup> / <sub>2</sub>	7	7 <sup>1</sup> / <sub>2</sub>	8	8 <sup>1</sup> / <sub>2</sub>	9	9 <sup>1</sup> / <sub>2</sub>	10	11	12	13	14	15
Length of anchor max < (in.)	3	3 <sup>1</sup> / <sub>2</sub>	4	4 <sup>1</sup> / <sub>2</sub>	5	5 <sup>1</sup> / <sub>2</sub>	6	6 <sup>1</sup> / <sub>2</sub>	7	7 <sup>1</sup> / <sub>2</sub>	8	8 <sup>1</sup> / <sub>2</sub>	9	9 <sup>1</sup> / <sub>2</sub>	10	11	12	13	14	15	16

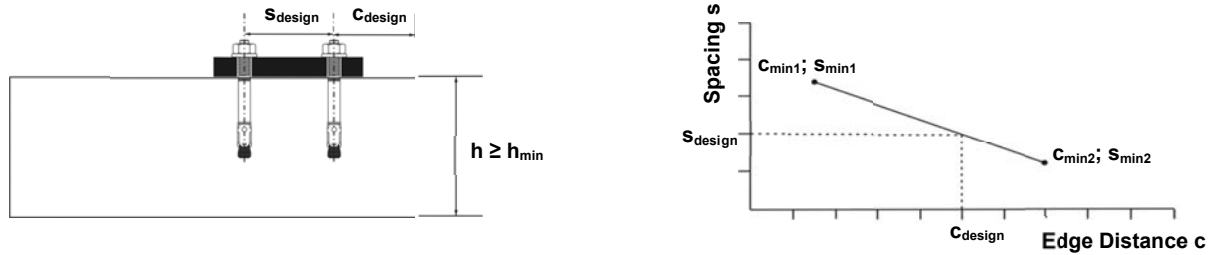


FIGURE 3—INTERPOLATION OF MINIMUM EDGE DISTANCE AND ANCHOR SPACING

TABLE 3—SRS TZ CHARACTERISTIC TENSION STRENGTH DESIGN INFORMATION<sup>1</sup>

CHARACTERISTIC	SYMBOL	UNITS	NOMINAL ANCHOR DIAMETER	
			1/2 inch	5/8 inch
Anchor Category	1,2 or 3	-	1	1
Nominal Embedment Depth	$h_{nom}$	in. (mm)	2 7/8 (73)	3 3/4 (95)
<b>Steel Strength in Tension (ACI 318 D.5.1)</b>				
Specified Yield Strength (Carbon Steel)	$f_{ya}$	psi (N/mm <sup>2</sup> )	75,420 (520)	75,420 (520)
Specified Yield Strength (Stainless Steel)	$f_{ya}$	psi (N/mm <sup>2</sup> )	81,280 (560)	81,280 (560)
Specified Tensile Strength (Carbon Steel)	$f_{uta}$	psi (N/mm <sup>2</sup> )	94,275 (650)	94,275 (650)
Specified Tensile Strength (Stainless Steel)	$f_{uta}$	psi (N/mm <sup>2</sup> )	101,600 (700)	101,600 (700)
Effective Tensile Stress Area <sup>8</sup>	$A_{se}$	in <sup>2</sup> (mm <sup>2</sup> )	0.0964 (62)	0.1419 (92)
Tension Resistance of Steel (SRS TZ Carbon Steel) <sup>8</sup>	$N_{sa}$	lbf (kN)	9,080 (40.4)	13,375 (59.5)
Tension Resistance of Steel (SRS TZ Stainless Steel) <sup>8</sup>	$N_{sa}$	lbf (kN)	9,775 (43.5)	14,405 (64.1)
Strength Reduction Factor-Steel Failure <sup>2</sup>	$\phi_{sa}$	-	0.75	0.75
<b>Concrete Breakout Strength in Tension (ACI 318 D.5.2)</b>				
Effective Embedment Depth	$h_{ef}$	in. (mm)	2 1/2 (64)	3 1/4 (83)
Critical Edge Distance	$c_{ac}$	in. (mm)	8 (203)	9.75 (248)
Effectiveness Factor-Uncracked Concrete	$k_{unscr}$	-	24 (10)	24 (10)
Effectiveness Factor-Cracked Concrete	$k_{cr}$	-	17 (7)	17 (7)
Modification Factor	$\Psi_{c,N}$	-	1.0	1.0
Strength Reduction Factor-Concrete Breakout Failure <sup>3</sup>	$\phi_{cb}$	-	0.65	0.65
<b>Pull-Out Strength in Tension (ACI 318 D.5.3)</b>				
Pull-Out Resistance Cracked Concrete ( $f'_c = 2,500$ psi) <sup>5,7</sup>	$N_{p,cr}$	lbf (kN)	2,810 (12.5)	NA <sup>4</sup>
Pull-Out Resistance Uncracked Concrete ( $f'_c = 2,500$ psi) <sup>5,7</sup>	$N_{p,unscr}$	lbf (kN)	4,495 (20)	NA <sup>4</sup>
Strength Reduction Factor-Pullout Failure <sup>6</sup>	$\phi_p$	-	0.65	0.65
<b>Tension Strength for Seismic Applications (ACI 318 D.3.3.3)</b>				
Tension Resistance of Single Anchor for Seismic Loads ( $f'_c = 2,500$ psi) <sup>5,7</sup>	$N_{p,eq}$	lbf (kN)	2,810 (12.5)	NA <sup>4</sup>
Strength Reduction Factor-Pullout Failure <sup>6</sup>	$\phi_{eq}$	-	0.65	0.65
Axial Stiffness in Service Load Range	$\beta$	lb/in. (kN/mm)	342,620 (60)	342,620 (60)

For SI: 1 inch = 25.4 mm, 1 ft-lbf = 1.356 N-m, 1 psi = 6.89 Pa, 1 in<sup>2</sup> = 645 mm<sup>2</sup>, 1 lb/in = 0.175 N/mm.

<sup>1</sup>The information presented in this table must be used in conjunction with the design requirements of ACI 318 Appendix D.  
<sup>2</sup>The tabulated value of  $\phi_{sa}$  applies when the load combinations of Section 1605.2.1 of the IBC or ACI 318 Section 9.2 are used. If the load combinations of ACI 318 Appendix C are used, the appropriate value of  $\phi_{sa}$  must be determined in accordance with ACI 318 D.4.5. The 1/2-inch- and 5/8-inch-diameter anchors are ductile steel elements as defined in ACI 318 D.1.  
<sup>3</sup>The tabulated value of  $\phi_{cb}$  applies when both the load combinations of Section 1605.2.1 of the IBC or ACI 318 Section 9.2 are used and the requirements of ACI 318 D.4.4(c) for Condition B are satisfied. For installations where complying supplementary reinforcement can be verified, the  $\phi_{cb}$  factors described in ACI 318 D.4.4 for Condition A are allowed. If the load combinations of Section 1605.2.1 of the IBC or ACI 318 Section 9.2 are used and the requirements of ACI 318 D.4.4 for Condition A are satisfied, the appropriate value of  $\phi_{cb}$  must be determined in accordance with ACI 318 D.4.4(c). If the load combinations of ACI 318 Appendix C are used, the appropriate value of  $\phi_{cb}$  must be determined in accordance with ACI 318 D.4.5.  
<sup>4</sup>As described in Section 4.1.4 and 4.1.8 of this report, N/A (Not Applicable) denotes that pullout resistance is not critical and does not need to be considered.  
<sup>5</sup>The characteristic pull-out resistance for greater concrete compressive strengths may be increased by multiplying the tabular value by  $(f'_c / 2,500)^{0.5}$ . See Section 4.1.4 and 4.1.8 of this report.  
<sup>6</sup>The tabulated value of  $\phi_p$  or  $\phi_{eq}$  applies when both the load combinations of Section 1605.2.1 of the IBC or ACI 318 Section 9.2 are used and the requirements of ACI 318 D.4.4(c) for Condition B are satisfied. Condition B applies where supplementary reinforcement is not provided or where pullout strength governs. For installations where complying supplementary reinforcement can be verified, the  $\phi$  factors described in ACI 318 D.4.4 for Condition A are allowed. If the load combinations of ACI 318 Appendix C are used, appropriate value of  $\phi$  must be determined in accordance with ACI 318 D.4.5.  
<sup>7</sup>For all design cases,  $\Psi_{cp} = 1.0$   
<sup>8</sup>For 2006 IBC,  $N_{sa} = N_s$ ,  $A_{se} = A_s$

**TABLE 4—SRS TZ CHARACTERISTIC SHEAR STRENGTH DESIGN INFORMATION<sup>1</sup>**

CHARACTERISTIC	SYMBOL	UNITS	NOMINAL ANCHOR DIAMETER	
			<sup>1</sup> / <sub>2</sub> inch	<sup>5</sup> / <sub>8</sub> inch
Anchor Category	1,2 or 3	-	1	1
Nominal Embedment Depth	$h_{nom}$	in.	2 <sup>7</sup> / <sub>8</sub> (73)	3 <sup>3</sup> / <sub>4</sub> (95)
<b>Steel Strength in Shear (ACI 318 D.6.1)</b>				
Specified Yield Strength for Shear (Carbon Steel)	$f_{ya}$	psi (N/mm <sup>2</sup> )	69,620 (480)	69,620 (480)
Specified Yield Strength for Shear (Stainless Steel)	$f_{ya}$	psi (N/mm <sup>2</sup> )	60,960 (420)	75,470 (520)
Specified Tensile Strength for Shear (Carbon Steel)	$f_{uta}$	psi (N/mm <sup>2</sup> )	87,025 (600)	87,025 (600)
Specified Tensile Strength for Shear (Stainless Steel)	$f_{uta}$	psi (N/mm <sup>2</sup> )	75,470 (520)	94,340 (650)
Effective Shear Stress Area <sup>5</sup>	$A_{se,v}$	in <sup>2</sup> (mm <sup>2</sup> )	0.142 (92)	0.226 (146)
Shear Resistance of Steel (SRS TZ Carbon Steel)	$V_{sa}$	lbf (kN)	7,420 (33)	11,015 (49)
Shear Resistance of Steel (SRS TZ Stainless Steel)	$V_{sa}$	lbf (kN)	6,430 (28.6)	12,790 (56.9)
Strength Reduction Factor-Steel Failure <sup>2</sup>	$\phi_{sa}$	-	0.65	0.65
<b>Concrete Breakout Strength in Shear (ACI 318 D.6.2)</b>				
Nominal Diameter <sup>5</sup>	$d_a$	in.(mm)	<sup>1</sup> / <sub>2</sub> (12.7)	<sup>5</sup> / <sub>8</sub> (15.9)
Load Bearing Length of Anchor in Shear	$l_e$	in. (mm)	2 <sup>1</sup> / <sub>2</sub> (64)	3 <sup>1</sup> / <sub>4</sub> (83)
Strength Reduction Factor-Concrete Breakout Failure <sup>3</sup>	$\phi_{cb}$	-	0.7	0.7
<b>Concrete Pryout Strength in Shear (ACI 318 D.6.3)</b>				
Coefficient for Pryout Strength	$k_{cp}$	-	2	2
Strength Reduction Factor-Concrete Pryout Failure <sup>4</sup>	$\phi_{cp}$	-	0.7	0.7
<b>Shear Strength for Seismic Applications (ACI 318 D.3.3.3)</b>				
Shear Resistance of Single Anchor for Seismic Loads ( $f'_c = 2,500$ psi) (SRS TZ Carbon Steel)	$V_{sa,eq}$	lbf (kN)	5,170 (23)	7,645 (34)
Shear Resistance of Single Anchor for Seismic Loads ( $f'_c = 2,500$ psi) (SRS TZ Stainless Steel)	$V_{sa,eq}$	lbf (kN)	4,500 (20)	8,950 (39.8)
Strength Reduction Factor-Steel Failure	$\phi_{eq}$	-	0.65	0.65

For **SI**: 1 inch = 25.4 mm, 1 ft-lbf = 1.356 N-m, 1 psi = 6.89 Pa, 1 in<sup>2</sup> = 645 mm<sup>2</sup>, 1 lb/in = 0.175 N/mm.

<sup>1</sup>The information presented in this table must be used in conjunction with the design criteria of ACI 318 Appendix D.

<sup>2</sup>The tabulated value of  $\phi_{sa}$  applies when the load combinations of Section 1605.2.1 of the IBC or ACI 318 Section 9.2 are used and the requirements of ACI 318 D.4.4(c) for Condition B are satisfied. If the load combinations of ACI 318 Appendix C are used, the appropriate value of  $\phi_{sa}$  must be determined in accordance with ACI 318 D.4.5. The <sup>1</sup>/<sub>2</sub> inch and <sup>5</sup>/<sub>8</sub> inch diameter anchors are ductile steel elements as defined in ACI 318 D.1.1.

<sup>3</sup>The tabulated value of  $\phi_{cb}$  applies when both the load combinations of Section 1605.2.1 of the IBC or ACI 318 Section 9.2 are used and the requirements of ACI 318 D.4.4(c) for Condition B are satisfied. Condition B applies where supplementary reinforcement is not provided or where pryout strength governs. For installations where complying supplementary reinforcement can be verified, the  $\phi$  factors described in ACI 318 D.4.4 for Condition A are allowed. If the load combinations of Section 1605.2.1 of the IBC or ACI 318 9.2 are used and the requirements of ACI 318 D.4.4 for Condition A are satisfied, the appropriate value of  $\phi$  must be determined in accordance with ACI 318 D.4.4(c). If the load combinations of ACI 318 Appendix C are used, the appropriate value of  $\phi$  must be determined in accordance with ACI 318 D.4.5.

<sup>4</sup>The tabulated value of  $\phi_{cp}$  applies when both the load combinations of Section 1605.2.1 of the IBC or ACI 318 Section 9.2 are used and the requirements of ACI 318 D.4.4(c) for Condition B are satisfied. Condition B applies where supplementary reinforcement is not provided or where pryout strength governs. For installations where complying supplementary reinforcement can be verified, the  $\phi$  factors described in ACI 318 D.4.4 for Condition A are allowed. If the load combinations of ACI 318 Appendix C are used, the appropriate value of  $\phi$  must be determined in accordance with ACI 318 D.4.5.

<sup>5</sup>For 2006 IBC,  $A_{se} = A_s$ ,  $d_a = d_o$

**TABLE 5—SRS TZ CARBON STEEL CHARACTERISTIC TENSION AND SHEAR STRENGTH DESIGN INFORMATION FOR THE SOFFIT OF CONCRETE-FILLED PROFILE STEEL DECK ASSEMBLIES<sup>1,2,6</sup>**

CHARACTERISTIC	SYMBOL	UNITS	NOMINAL ANCHOR DIAMETER	
			<sup>1</sup> / <sub>2</sub> inch	<sup>5</sup> / <sub>8</sub> inch
Nominal Embedment Depth	$h_{nom}$	in.	2 <sup>7</sup> / <sub>8</sub>	3 <sup>3</sup> / <sub>4</sub>
Effective Embedment Depth	$h_{ef}$	in.	2 <sup>1</sup> / <sub>2</sub>	3 <sup>1</sup> / <sub>4</sub>
Installation Torque	$T_{inst}$	ft-lbf	35	65
Pull-Out Resistance Cracked Concrete <sup>3</sup>	$N_{p,deck,cr}$	lbf	1,460	2,730
Pull-Out Resistance Uncracked Concrete <sup>4</sup>	$N_{p,deck,uncr}$	lbf	2,335	3,855
Steel Strength in Shear <sup>5</sup>	$V_{sa,deck}$	lbf	2,785	4,410
Steel Strength in Shear, Seismic <sup>5</sup>	$V_{sa,deck,eq}$	lbf	1,940	3,060
Strength Reduction factor	$\phi$		0.65	0.65

For **SI**: 1 inch = 25.4mm, 1 ft-lbf = 1.356 N-m, 1 lbf = 4.45 N.

<sup>1</sup>Installation must comply with Section 4.3 and Figure 4. Except as specifically noted, all installations must be into the lower flute of the deck.

<sup>2</sup>Profile steel deck must comply with Figure 4 and have a minimum base steel thickness of 0.034 inch (0.9 mm). Steel must comply with ASTM A653 / A635M, with minimum yield strength of 33 ksi (228 MPa).

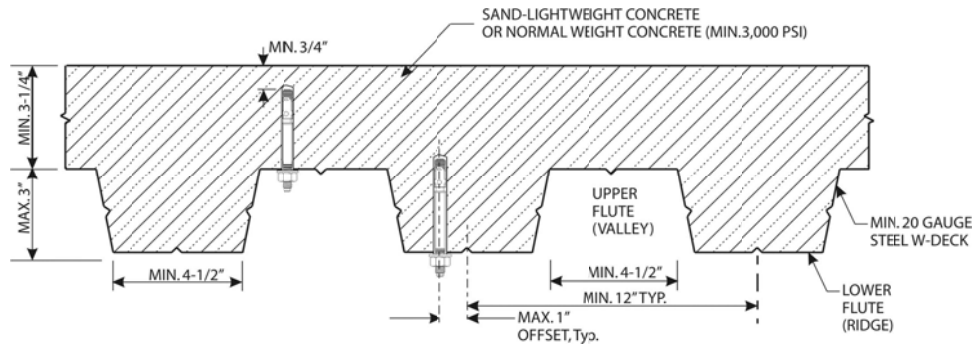
<sup>3</sup>The values must be used in accordance with Section 4.1.4 and 4.1.8.1 of this report.

<sup>4</sup>The values must be used in accordance with Section 4.1.4 of this report.

<sup>5</sup>The values must be used in accordance with Section 4.1.5 and 4.1.8.2 of this report.

<sup>6</sup>The minimum anchor spacing along the flute must be the greater of 3.0 $h_{ef}$  or 1.5 times the flute width.





**FIGURE 4—INSTALLATION IN THE SOFFIT OF CONCRETE OVER PROFILE STEEL DECK FLOOR AND ROOF ASSEMBLIES<sup>1</sup>**

<sup>1</sup> Anchors may be placed in the upper flute or lower flute of the steel deck profiles in accordance with Figure 3 provided the minimum hole clearance is satisfied. Anchors in the lower flute of Figure 4 profiles may be installed with a maximum 1-inch offset in either direction from the center of the flute. The offset distance may be increased proportionally for profiles with lower flute widths greater than those shown provided the minimum lower flute edge distance is also satisfied.

**TABLE 6—EXAMPLE ALLOWABLE STRESS DESIGN VALUES FOR ILLUSTRATIVE PURPOSES<sup>1,2,3,4,5,6,7,8</sup>**

Nominal Anchor Diameter, $d_a$ (in.)	Embedment Depth, $h_{nom}$ (in.)	Effective Embedment Depth, $h_{ef}$ (in.)	Allowable Tension Load, $\phi N_n / \alpha$ (pounds)
<sup>1</sup> / <sub>2</sub>	<sup>2</sup> / <sub>8</sub>	<sup>2</sup> / <sub>2</sub>	1,975
<sup>5</sup> / <sub>8</sub>	<sup>3</sup> / <sub>4</sub>	<sup>3</sup> / <sub>4</sub>	3,085

For SI: 1 inch = 25.4 mm, ft-lbf = 1.356 N-m, 1 lbf = 4.45 N.

<sup>1</sup> Single anchor with static tension load only.

<sup>2</sup> Concrete determined to remain uncracked for the life of the anchorage.

<sup>3</sup> Load combination 9-2 from ACI 318 Section 9.2 (no seismic loading) with  $\phi_{sa} = 0.75$ ,  $\phi_{cb} = 0.65$ , and  $\phi_p = 0.65$ .

<sup>4</sup> 30% dead load and 70 % live load. Controlling load combination is 1.2D + 1.6L. Calculation of  $\alpha$  based on weighted average:  $\alpha = 0.3 \cdot 1.2 + 0.7 \cdot 1.6 = 1.48$ .

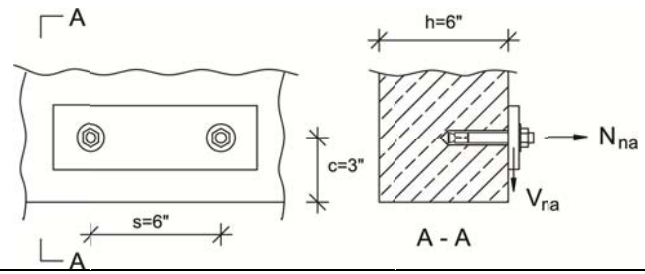
<sup>5</sup>  $f'_c = 2,500$  psi (normal weight concrete).

<sup>6</sup>  $C_{a1} = C_{a2} \geq C_{ac}$

<sup>7</sup>  $h \geq h_{min}$

<sup>8</sup> Values are for Condition B where supplementary reinforcement in accordance with ACI 318 D.4.4 is not provided.

Determine if two  $1/2$  inch diameter SRS TZ carbon steel anchors with an effective embedment depth  $h_{ef} = 2 1/2$  inches installed 6 inches from center to center and 3 inches from the edge of a 6 inch deep slab is adequate for a service tension load of 2,000 lb. for wind and a reversible service shear load of 400 lb. for wind (no seismic loads). The anchor group will be in the tension zone, away from other anchors in  $f'_c = 3,000$  psi normal – weight (cracked) concrete.



	ACI 318-08 Code Ref.	Report Ref.
<u>1. Verify minimum Member Thickness, Spacing and Edge Distance:</u>		
$h = 6 \text{ in.} \geq h_{\min} = 5 \text{ in.}$ o.k.		Table 1
$s = 6 \text{ in.} \geq s_{\min} = 6 \text{ in.}$ o.k.		Table 1
$c_{a, \min} = 3 \text{ in.} \geq c_{\min} = 3 \text{ in.}$ o.k.		Table 1
<u>2. Determine the Factored Tension and Shear Design Loads:</u>	9.2.1	
$N_{ua} = 1.6 W = 1.6 \times 2,000 = 3,200 \text{ lb.}$		
$V_{ua} = 1.6 W = 1.6 \times 400 = 640 \text{ lb.}$		
<u>3. Steel Capacity under Tension Loading:</u>	D.5.1	
$N_{sa} = 9,080$		Table 3
$\phi = 0.75$		Table 3
$n = 2$ (double anchor group)		
Calculating for $\Phi N_{sa}$ :		
$\phi N_{sa} = 0.75 \times 2 \times 9,080 = 13,620 \text{ lb.}$		
<u>4. Concrete Breakout Capacity under Tension Loading</u>	D.5.2	
$N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$	Eq.(D-5)	
where:		
$N_b = k_c \lambda \sqrt{f'_c} h_{ef}^{1.5}$	Eq.(D-7)	
with $k_c = k_{cr} = 17$	Table 3	
$\Psi_{ec,N} = 1.0$ since eccentricity $e_N = 0$	Eq.(D-9)	
$\Psi_{ed,N} = 0.7 + 0.3 \frac{C_{a,\min}}{1.5h_{ef}}$ when $C_{a,\min} \leq 1.5h_{ef}$	Eq.(D-11)	
by observation $C_{a,\min} = 3 < 1.5h_{ef} = 3.75 \text{ in.}$		
$\Psi_{ed,N} = 0.7 + 0.3 \frac{(3)}{1.5(2.5)} = 0.94$		
$\Psi_{c,N} = 1.0$ assuming cracking at service loads ( $f_t > f_r$ )	D.5.2.6	
$\Psi_{cp,N} = 1.0$ designed for cracked concrete	D.5.2.7	

	ACI 318-08 Code Ref.	Report Ref.
calculating for $\frac{A_{Nc}}{A_{Nco}}$		
$A_{Nco} = 9h_{ef}^2 = 9(2.5)^2 = 56.25 \text{ in.}^2$	Eq. (D-6)	
$A_{Nc} = (c_{a1} + 1.5 h_{ef}) (2 \times 1.5 h_{ef} + s_1)$ $= (3 + 1.5 \times 2.5) (2 \times 1.5 \times 2.5 + 6)$ $= 91.1 \text{ in.}^2$	Fig. RD.5.2.1 b	
$\frac{A_{Nc}}{A_{Nco}} = \frac{91.1}{56.25} = 1.62$		
Calculating for $N_b$ and $N_{cbg}$ :		
$N_b = 17 \times 1.0 \times \sqrt{3,000} \times (2.5)^{1.5} = 3,681 \text{ lb.}$		
$\lambda = 1.0$ for normal weight concrete		
$N_{cbg} = 1.62 \times 1.0 \times 0.94 \times 1.0 \times 1.0 \times 3,681 = 5,605 \text{ lb.}$		
$\phi = 0.65$ for Condition B (no supplementary reinforcement provided)		Table 3
$\Phi N_{cb} = 0.65 \times 5,605 = 3,643 \text{ lb.}$		
<u>5. Pullout Capacity</u>	D.5.3	
$N_{pn, cr} = 2 \times 1.0 \times 2,810 \times \left( \frac{3,000}{2,500} \right)^{0.5} = 6,156 \text{ lb.}$		Table 3
$\Psi_{c,p} = 1.0$ for cracked and uncracked concrete		
$\phi = 0.65$		Table 3
$\phi N_{pn, cr} = 0.65 \times 6,156 = 4,002 \text{ lb.}$		
<u>6. Check all Failure Modes under Tension Loading:</u>	D.4.1.2	
Summary:		
Steel Capacity	= 13,620 lb	
Concrete Breakout Capacity	= 3,643 lb. ← <b>Controls</b>	
Pullout Capacity	= 4,002 lb.	
<b><math>\Phi N_n = 3,643 \text{ lb.}</math> as Concrete Breakout Capacity controls &gt; <math>N_{ua} = 3,200 \text{ lb.}</math> - OK</b>		
<u>7. Steel Capacity under Shear Loading:</u>	D.6.1	
Calculating for $\Phi V_{sa}$ :		
$V_{sa} = 2 \times 7,420 = 14,840 \text{ lb.}$		
$\phi = 0.65$		Table 4
$\phi V_{sa} = 0.65 \times 14,840 = 9,646 \text{ lb.}$		Table 4

FIGURE 5—SRS TZ EXAMPLE CALCULATION

	ACI 318-08 Code Ref.	Report Ref.		ACI 318-08 Code Ref.	Report Ref.
8. <u>Concrete Breakout Capacity under Shear Loading:</u>	D.6.2			D.4.1.2	
$V_{cbg} = \frac{A_{vc}}{A_{vc0}} \Psi_{ec,v} \Psi_{ed,v} \Psi_{c,v} V_b$	Eq. (D-22)				
where:					
$V_b = 7 \left( \frac{l_e}{d_a} \right)^{0.2} \sqrt{d_a} \lambda \sqrt{f'_c} c_{a1}^{1.5}$	Eq. (D-24)				
$\Psi_{ec,v} = 1.0$ since eccentricity $e'_v = 0$	Eq. (D-26)				
$\Psi_{ed,v} = 1.0$ since $c_{a2} > 1.5c_{a1}$	Eq. (D-27)				
$\Psi_{c,v} = 1.0$ assuming cracking at service loads ( $f_t > f_r$ )	D.6.2.7				
calculating for $\frac{A_{vc}}{A_{vc0}}$					
$h = 6 > 1.5 c_{a1} = 1.5 \times 3 = 4.5$ in.					
$A_{vc} = (2(1.5c_{a1}) + s_1) 1.5c_{a1}$ $= (2 \times 1.5 \times 3 + 6) \times 1.5 \times 3$ $= 67.5$ in. <sup>2</sup>	Fig. RD.6.2.1b				
$A_{vc0} = 4.5 (c_{a1})^2 = 4.5 \times 3^2 = 40.5$ in. <sup>2</sup>	Eq. (D-23)				
$\frac{A_{vc}}{A_{vc0}} = \frac{67.5}{40.5} = 1.67$	D.6.2.1				
calculating for $V_b$ and $\Phi V_{cbg}$					
$d_a = 0.5$ in.		Table 4			
$l_e = h_{ef} = 2.5$ in.	D.6.2.2				
$c_{a1} = 3$ in.					
$\lambda = 1.0$ for normal weight concrete					
$\phi = 0.70$ for Condition B (no supplementary reinforcement provided)		Table 4			
$V_b = 7 \times \left( \frac{2.5}{0.5} \right)^{0.2} \times \sqrt{0.5} \times 1.0 \times \sqrt{3,000} \times (3)^{1.5} = 1,944$ lb.					
$\Phi V_{cbg} = 0.70 \times 1.67 \times 1.0 \times 1.0 \times 1.0 \times 1,944 = 2,272$ lb.					
9. <u>Concrete Pryout Strength:</u>	D.6.3				
$V_{cpg} = k_{cp} N_{cbg}$	Eq.(D-30)				
where:					
$\phi = 0.70$		Table 4			
$k_{cp} = 2.0$	D.6.3.1	Table 4			
$V_{cpg} = k_{cp} N_{cbg} = 2.0 \times 5,605 = 11,210$ lb.	Eq. (D-30)				
$\phi V_{cpg} = 0.70 \times 11,210 = 7,847$ lb.					
10. <u>Check all Failure Modes under Shear Loading:</u>					
Summary:					
Steel Capacity = 9,645 lb.					
Concrete Breakout Capacity = 2,272 lb. ← <b>Controls</b>					
Pryout Capacity = 7,847 lb					
$\phi V_n = 2,272$ lb. as <b>Concrete Breakout Capacity controls</b> <b>&gt; <math>V_{ua} = 640</math> lb. – OK</b>					
11. Check Interaction of Tension and Shear Forces					
D.7					
If $0.2 \phi V_n \geq V_{ua}$ then the full tension design strength is permitted.			D.7.1		
By observation, this is not the case.					
If $0.2 \phi N_n \geq N_{ua}$ then the full shear design strength is permitted.			D.7.2		
By observation, this is not the case.					
Therefore:					
$\frac{N_{ua}}{\Phi N_n} + \frac{V_{ua}}{\Phi V_n} \leq 1.2$					
$\frac{3,200}{3,643} + \frac{640}{2,272} = 0.88 + 0.28 = 1.16 < 1.2$ – OK					
12. <u>Summary</u>					
<b>Two 1/2 in. diameter SRS TZ at 2 1/2 in. effective embedment depth are adequate to resist the applied service tension and shear loads of 2,000 lb. and 400 lb., respectively.</b>					

FIGURE 5—SRS TZ EXAMPLE CALCULATION (Continued)