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ESR-2691

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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:

FISCHERWERKE GMBH & CO. KG

WEINHALDE 14-18
72178 WALDACHTAL
GERMANY

EVALUATION SUBJECT:

**FISCHER FH II CARBON STEEL HEAVY DUTY SLEEVE ANCHOR FOR ANCHORING IN
CRACKED AND UNCRACKED CONCRETE**



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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:
**fischer FH II CARBON STEEL HEAVY DUTY SLEEVE
ANCHOR FOR ANCHORING IN 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)
- 2013 *Abu Dhabi International Building Code* (ADIBC)[†]

[†]The ADIBC is based on the 2009 IBC. 2009 IBC code sections referenced in this report are the same sections in the ADIBC.

Property evaluated:

Structural

2.0 USES

The fischer FH II Carbon Steel Heavy Duty Sleeve Anchor is used to resist static, wind and seismic tension and shear loads in cracked and uncracked normal-weight and lightweight concrete having a specified compressive strength, f'_c of 2,500 psi to 8,500 psi (17.2 MPa to 58.6 MPa) [minimum of 24 MPa is required under ADIBC Appendix L, Section 5.1.1].

The fischer FH II anchors comply as anchors installed in hardened concrete in accordance with 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 2009 and 2006 IRC.

3.0 DESCRIPTION

3.1 FH II Carbon Steel Heavy Duty Sleeve Anchor:

3.1.1 General: The fischer FH II Carbon Steel Heavy Duty Sleeve Anchor, designated as the FH II, is a torque-controlled, sleeve-type mechanical expansion anchor. The components, dimensions and installation criteria are set forth in Tables 1, 2 and 4, and Figures 1, 2 and 3 of this report. Four head configurations are available: Version S, Version B, Version H and Version SK. The head configurations are illustrated in Figure 3. All carbon steel parts have a minimum 0.0002 inch (5 μ m) thick galvanized zinc coating according to DIN EN ISO 4042.

Application of torque at the head of the anchor causes the cone to be drawn into the expansion sleeve. This cone movement in turn causes the expansion sleeve to expand against the wall of the drilled hole. The ribs on the collapsible sleeve prevent rotation of the expansion sleeve and cone during application of torque. Application of the specified installation torque induces a tension force in the concrete acting through the component being fastened. Telescopic deformation of the collapsible sleeve prevents buildup of pre-compression in the expansion sleeve in cases where the steel sleeve is in contact with the washer, and permits the closure of gaps between the work surface and the component being fastened. Application of tension loads that exceed the pre-compression force in the anchor will cause the cone to displace further into the expansion sleeve (follow-up expansion), generating additional expansion force.

3.1.2 FH IIS (Hexagonal Screw Version): The anchor consists of an ISO 898-1, Class 8.8 hexagonal screw, steel washer, steel sleeve, collapsible plastic sleeve, steel expansion sleeve and steel cone. This anchor is available in carbon steel only. The material specifications are as follows:

- Hexagonal screw: Carbon steel complying with ISO 898-1, Class 8.8
- Washer: Carbon steel complying with EN 10139.
- Expansion cone: Carbon steel complying with EN 10277.
- Expansion sleeve: Carbon steel complying with EN 10305.
- Steel sleeve: Carbon steel complying with EN 10305-1.
- Collapsible sleeve: Plastic blend of polymer complying with ISO 527-1 and ISO 527-2.

3.1.3 FH II B (Bolt Version with Threaded Rod): The FH II B has the same components and material specifications as the FH II S (hexagonal screw) except that the hexagonal screw is replaced by a carbon steel threaded rod complying with ISO 898-1 Class 8.8 and a carbon steel hexagonal nut complying with ISO 898-2.

3.1.4 FH II H (Hexagonal Cap Nut Version): The FH II H has the same components and material specifications as the FH II B (bolt) except that a carbon steel hexagonal cap nut complying with ISO 898-2 replaces the nut.

3.1.5 FH II SK (Countersunk Screw Version): The FH II SK has the same components and material specifications as the FH II S (hexagonal screw) except that the hexagonal screw head is configured for countersunk applications, is configured to accept a hexagonal Allen wrench, and is provided with a conical washer. The screw is formed from carbon steel complying with ISO 898-1 and the conical washer is formed from carbon steel complying with EN 10277.

3.2 Concrete:

Normal-weight and lightweight concrete must conform to Sections 1903 and 1905 of the IBC, as applicable.

4.0 DESIGN AND INSTALLATION

4.1 Strength Design:

4.1.1 General: Design strength of anchors in accordance with the 2006 IBC and 2006 IRC must be in accordance with ACI 318-05 Appendix D and this report.

Design strength of anchors in accordance with the 2009 IBC as well as Section 301.1.3 of the 2009 IRC must be determined in accordance with ACI 318-08 Appendix D and this report. Design examples according to the 2006 IBC are given in Figures 4 through 7 of this report. Design parameters are based on the 2009 IBC (ACI 318-08) unless noted otherwise in Section 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, ϕ , 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, ϕ , 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. Strength reduction factors, ϕ , corresponding to ductile steel elements may be used for the FH II.

4.1.2 Requirements for Static Steel Strength in Tension, N_{sa} : The nominal steel strength of a single anchor in tension in accordance with ACI 318 D.5.1.2, N_{sa} is given in Table 3 of this report. Strength reduction factors, ϕ , corresponding to ductile steel elements may be used for the FH II.

4.1.3 Requirements for Concrete Breakout Strength in Tension, N_{cb} and N_{cbg} : The nominal concrete breakout strength of a single anchor or group of anchors in tension, N_{cb} and N_{cbg} , respectively must be calculated in accordance with ACI 318 D.5.2, with modifications as described in this section. The basic concrete breakout strength of a single anchor in tension, N_b , must be calculated according to ACI 318 D.5.2, using the values of k_{cr} and h_{ef} and as given in Table 3 of this report. The value of f'_c is limited to 8,000 psi (55.2 MPa), maximum, 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 shall be calculated with $\psi_{cN} = 1.0$ and using the value of k_{uncr} as given in Table 3 of this report.

4.1.4 Requirements for Critical Edge Distance, Splitting: 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 $\psi_{cp,N}$ according to ACI 318 D.5.2.7 (Eq-1 of this report). In lieu of ACI 318 D.8.6, values of c_{ac} provided in Table 4 of this report must be used.

$$\psi_{cp,N} = \frac{c}{c_{ac}} \quad (\text{Eq-1})$$

whereby the factor $\psi_{cp,N}$ need not to be taken as less than $\frac{1.5h_{ef}}{c_{ac}}$

For all other cases $\psi_{cp,N} = 1.0$.

4.1.5 Requirements for pullout strength in tension, N_{pn} : The nominal pullout strength of a single anchor in tension in accordance with ACI 318 D.5.3 in cracked and uncracked concrete, $N_{p,cr}$ and $N_{p,uncr}$, respectively, do not need to be evaluated.

4.1.6 Requirements for Static Steel Strength in Shear, V_{sa} : In lieu of the value of the nominal steel strength values of V_{sa} as given in ACI 318, D.6.1.2 in shear, the values of V_{sa} for single anchor given in Table 3 of this report shall be used and not derived by calculation. Strength reduction factors, ϕ , corresponding to ductile elements may be used for the FH II.

4.1.7 Requirements for Static Concrete Breakout Strength in Shear V_{cb} or V_{cbg} : The nominal concrete breakout strength of a single anchor or group of anchors in shear, V_{cb} or V_{cbg} , respectively must be calculated in accordance with ACI 318 D.6.2, with modifications as described in this section. The basic concrete breakout strength of a single anchor in shear, V_b , must be calculated in accordance with ACI 318 D.6.2.2 using the value of l_e and d_a given in Table 3 of this report.

4.1.8 Requirements for Static Concrete Pryout Strength in Shear V_{cp} or V_{cpg} : The nominal concrete pryout strength of a single anchor or group of anchors in shear, V_{cp} or V_{cpg} , must be calculated in accordance with ACI 318 D.6.3, modified by using the value of k_{cp} provided in Table 3 of this report and the value of N_{cb} or N_{cbg} as calculated in accordance with Section 4.1.3 of this report.

4.1.9 Requirements for Minimum Member Thickness, Minimum Anchor Spacing and Minimum Edge Distance: In lieu of ACI 318 D.8.5, minimum member thickness, $h_{a,min}$, must comply with Table 4 of this report. In lieu of ACI 318 D.8.3, minimum edge distance and minimum spacing, $c_{a,min}$ and $s_{a,min}$ must comply with Table 4 of this report. Intermediate values between s_{min} and c_{min} can be calculated by linear interpolation. Figures 4 through 7 of this report provide more detail.

4.1.10 Requirements for Seismic Design:

4.1.10.1 General: For load combinations including seismic, the design must be performed according to ACI 318 D.3.3, as modified by Section 1908.1.9 of the 2009 IBC or Section 1908.1.16 of the 2006 IBC.

The nominal steel strength and the nominal concrete breakout strength for anchors in tension, and the nominal concrete breakout strength and pryout strength for anchors

in shear, must be calculated according to ACI 318 Sections D.5 and D.6, respectively, taking into account the corresponding values in Table 3 and 4 of this report. The anchors comply with ACI 318 Section D.1 as ductile steel elements and must be designed in accordance with ACI 318-05 Section D.3.3.4 or D.3.3.5 or ACI 318-08 Section D.3.3.4, D.3.3.5 or D.3.3.6.

4.1.10.2 Seismic Tension: The nominal steel strength and nominal concrete breakout strength for anchors in tension must be calculated according to ACI 318 D.5.1 and D.5.2, as described in Sections 4.1.2 and 4.1.3 of this report and in accordance with ACI 318 Section D.5.3.2. The value for pullout strength in tension for seismic loads, N_{eq} , does not need to be evaluated.

4.1.10.3 Seismic Shear: The nominal concrete breakout strength and pryout strength for anchors in shear must be calculated according to ACI 318 Section D.6.2 and D.6.3, as described in Sections 4.1.7 and 4.1.8 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_{eq} , described in Table 3 of this report must be used in lieu of V_{sa} . Strength reduction factors, Φ , corresponding to ductile elements must be used for the FH II.

4.1.11 Requirements for Interaction of Tensile and Shear Forces: For loadings that include combined tension and shear, the design must be performed in accordance with ACI 318 D.7.

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

For ACI 318-08 (2009 IBC), λ shall be determined in accordance with ACI 318-08.

For ACI 318-05 (2006 IBC), λ 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.

4.2 Allowable Stress Design (ASD):

4.2.1 General: Design values for use with allowable stress design load combinations calculated in accordance with Section 1605.3 of the IBC shall be established using Eq-2 and Eq-3:

$$T_{allowable, ASD} = \frac{\phi N_n}{\alpha} \quad (\text{Eq-2})$$

and

$$V_{allowable, ASD} = \frac{\phi V_n}{\alpha} \quad (\text{Eq-3})$$

where:

$T_{allowable, ASD}$ = Allowable tension load [lbf or kN]

$V_{allowable, ASD}$ = Allowable shear load [lbf or kN]

ϕN_n = Lowest design strength of an anchor or anchor group in tension as determined in accordance with ACI 318 Appendix D and 2009 IBC Section 1908.1.9, or 2006 IBC Section 1908.1.16, as applicable (lbf or kN).

ϕV_n = Lowest design strength of an anchor or anchor group in shear as determined in accordance with ACI 318 Appendix D and 2009 IBC Section 1908.1.9, or 2006 IBC Section 1908.1.16, as applicable (lbf or kN).

α = Conversion factor calculated as a weighted average of the load factors for the controlling load combination. In addition, α shall include all applicable factors to account for nonductile failure modes and required over-strength.

An example of allowable stress design values for illustrative purposes is shown in Table 5 of this report.

4.2.2 Interaction of Tensile and Shear Forces: The interaction must be calculated in accordance and consistent with ACI 318 Section D.7 as follows:

For shear loads $V \leq 0.2 V_{allowable, ASD}$, the full allowable load in tension $T_{allowable, ASD}$ must be permitted.

For tension loads $T \leq 0.2 T_{allowable, ASD}$, the full allowable load in shear $V_{allowable, ASD}$ must be permitted.

For all other cases Eq-4 applies:

$$\frac{T}{T_{allowable, ASD}} + \frac{V}{V_{allowable, ASD}} \leq 1.2 \quad (\text{Eq-4})$$

4.3 Installation:

Installation parameters are provided in Tables 2 and 4 and in Figure 2 of this report. Anchor locations must be in accordance with this report and the plans and specifications approved by the code official. The FH II anchors must be installed according to the manufacturer's published instructions and this report. Anchors must be installed in holes drilled into the concrete using carbide-tipped masonry drill bits complying with the requirements of Table 2 of this report. The minimum drilled hole depth is given in Table 2. The predrilled hole must be cleaned free of dust and debris using a hand pump, compressed air or vacuum. The anchor must be hammered into the predrilled hole until the proper nominal embedment depth is achieved. The anchor must be tightened against the washer until the torque values T_{inst} specified in Table 2 of this report are achieved.

4.4 Special Inspection:

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, hole cleaning procedures, anchor spacing(s), edge distance(s), slab thickness, anchor embedment depth, tightening torque and adherence to 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 and 1706 must be observed, where applicable.

5.0 CONDITIONS OF USE

The FH II 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 minimum embedment depths are as set forth in this report.
- 5.2** The anchors must be installed in accordance with the manufacturer's published installation instructions and this report. In case of a conflict, this report governs.
- 5.3** 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) [minimum of 24 MPa is required under ADIBC Appendix L, Section 5.1.1].

- 5.4 The values of f'_c used for calculation purposes shall 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(s) and edge distance(s) as well as minimum member thickness must comply with Table 4 of this report.
- 5.8 Prior to installation, calculations and details demonstrating compliance with this report must be submitted to the code official. 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, subject to the conditions of this report.
- 5.12 Where not otherwise prohibited in the code, FH II 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 inspection must be provided in accordance with Section 4.4 of this report.

5.15 Anchors are manufactured by fischerwerke, under an approved quality control program with inspections by ICC-ES.

6.0 EVIDENCE SUBMITTED

Data in accordance with the ICC-ES Acceptance Criteria for Mechanical Anchors in Concrete Elements (AC193), dated October 2015, for use in cracked and uncracked concrete and quality control documentation.

7.0 IDENTIFICATION

The anchors can be identified on the packaging label with the manufacturer's name (fischer) and address, anchor name, anchor size and evaluation report number (ICC-ES ESR-2691). The "fish" symbol, the letters FH II, the anchor diameter and the maximum fixing thickness are stamped on each anchor.

Example: ("fish") FH II 24/25 S, means 24 mm outer anchor diameter, 25 mm maximum thickness of fixture, S stands for Hexagonal Screw Version.

TABLE 1—ANCHOR DIMENSIONAL CHARACTERISTICS¹

Anchor Version ²	Nominal Bolt Diameter (mm)	d_{bit} (mm)	d_s (mm)	L_1 (mm)	t_{fix} ³ (mm)		l_1 (mm)	l_2 (mm)	l_3 (mm)	l_4 (mm)	t_{washer} (mm)
					Min.	Max.					
FH II S	M8	12	12.0	74	0	250	11	30	10	22	≥1.8
	M10	15	14.8	89	0	300	13	35.5	10.5	26.5	≥2.3
	M12	18	17.8	99	0	350	16	41	15	26.5	≥2.7
	M16	24	23.7	124	0	400	19	46	20	38	≥2.7
	M20	28	27.5	148	0	600	22.4	50	17.5	61.5	≥3.4
	M24	32	31.5	168	0	600	22.4	55	25	74	≥4.0
FH II B	M8	12	12.0	74	0	250	11	30	10	22	≥1.8
	M10	15	14.8	89	0	300	13	35.5	10.5	26.5	≥2.3
	M12	18	17.8	99	0	350	16	41	15	26.5	≥2.7
	M16	24	23.7	124	0	400	19	46	20	38	≥2.7
	M20	28	27.5	148	0	600	22.4	50	17.5	61.5	≥3.4
	M24	32	31.5	168	0	600	22.4	55	25	74	≥4.0
FH II H	M10	15	14.8	89	0	300	13	35.5	10.5	26.5	≥2.3
	M12	18	17.8	99	0	350	16	41	15	26.5	≥2.7
	M16	24	23.7	124	0	400	19	46	20	38	≥2.7
FH II SK	M10	15	14.8	89	6	300	13	35.5	10.5	26.5	0
	M12	18	17.8	99	6	350	16	41	15	26.5	0

For pound-inch units: 1 mm = 0.03937 inches

¹ Figure 1 describes location of dimensions. All dimensions are nominal excluding manufacturing tolerances.

² Figure 3 illustrates the available anchor versions.

³ Thickness applies to the attached fixture.

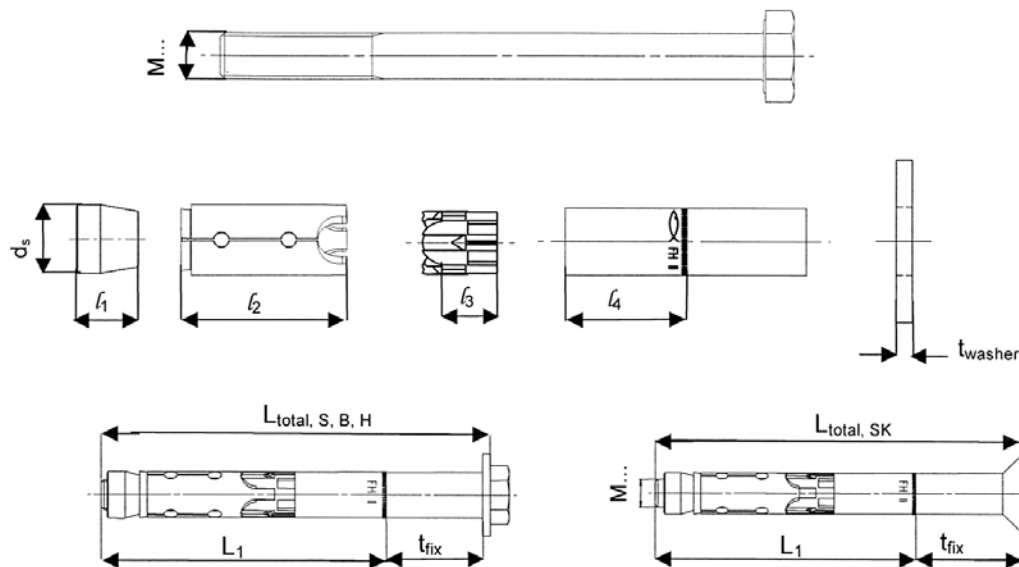


FIGURE 1—ANCHOR DIMENSIONAL CHARACTERISTICS

TABLE 2—INSTALLATION INFORMATION¹

Characteristic	Symbol	Units	Anchor Designation and Size						
			FH II 12 M8	FH II 15 M10	FH II 18 M12	FH II 24 M16	FH II 28 M20	FH II 32 M24	
Nominal drill hole diameter ²	d_{bit}	mm	12	15	18	24	28	32	
Cutting diameter of drill bit	$d_{bit,min}$	mm	12.10	15.10	18.10	24.10	28.10	32.15	
	$d_{bit,max}$	mm	12.50	15.50	18.50	24.55	28.55	32.70	
Minimum drill hole depth	$h_{1,min} \geq$	mm	80	90	105	125	155	180	
		in.	3.15	3.54	4.13	4.92	6.10	7.09	
Diameter of clearance hole in the fixture ⁴	$d_{f,S,H,B} \leq$	mm	14	17	20	26	31	35	
		in.	0.55	0.67	0.79	1.02	1.22	1.38	
Diameter of countersunk hole in the fixture	$d_{f,SK}$	mm	-	24.5	31.6	-	-	-	
		in.	-	0.97	1.25	-	-	-	
Required installation torque	Type S	T_{inst}	Nm	22.5	40	80	160	180	200
	Type SK			-	40	80	-	-	-
	Type H			-	40	80	90	-	-
	Type B			17.5	38	80	120	180	200
	Type S	ft-lbf	16.6	29.5	59.0	118.0	132.8	147.5	
	Type SK		-	29.5	59.0	-	-	-	
	Type H		-	29.5	59.0	66.4	-	-	
	Type B		12.9	28.0	59.0	88.5	132.8	147.5	
Wrench size ³	Type S	-	mm	13	17	19	24	30	36
	Type SK			-	6	8	-	-	-
	Type H			-	17	19	24	-	-
	Type B			13	17	19	24	30	36
Washer Diameter	d_w	mm	≥ 20	≥ 24	≥ 28	≥ 36	≥ 40	≥ 46	
		in.	≥ 0.79	≥ 0.95	≥ 1.10	≥ 1.42	≥ 1.57	≥ 1.81	

For pound-inch units: 1 mm = 0.03937 inches, 1 Nm = 0.7376 ft-lbf.

¹All specifications excluding manufacturing tolerances.

²Only metric drill bits must be used.

³For Type SK Allen wrench size.

⁴Tolerances complying with ISO 273

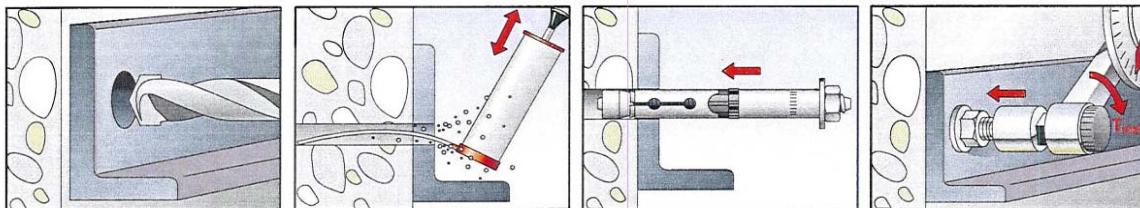
Installation:

Step 1: Using the correct metric bit diameter, d_{bit} (Table 2), drill hole to minimum required hole depth $h_{1,min}$ (Table 2) or deeper.

Step 2: Remove drilling debris with a blowout bulb or with compressed air.

Step 3: Using a hammer, tap the anchor through the part being fastened into the drilled hole until the washer is in contact with the fastened part. Do not expand anchor prior to installation.

Step 4: Using a torque wrench, apply the specified installation torque T_{inst} (Table 2).



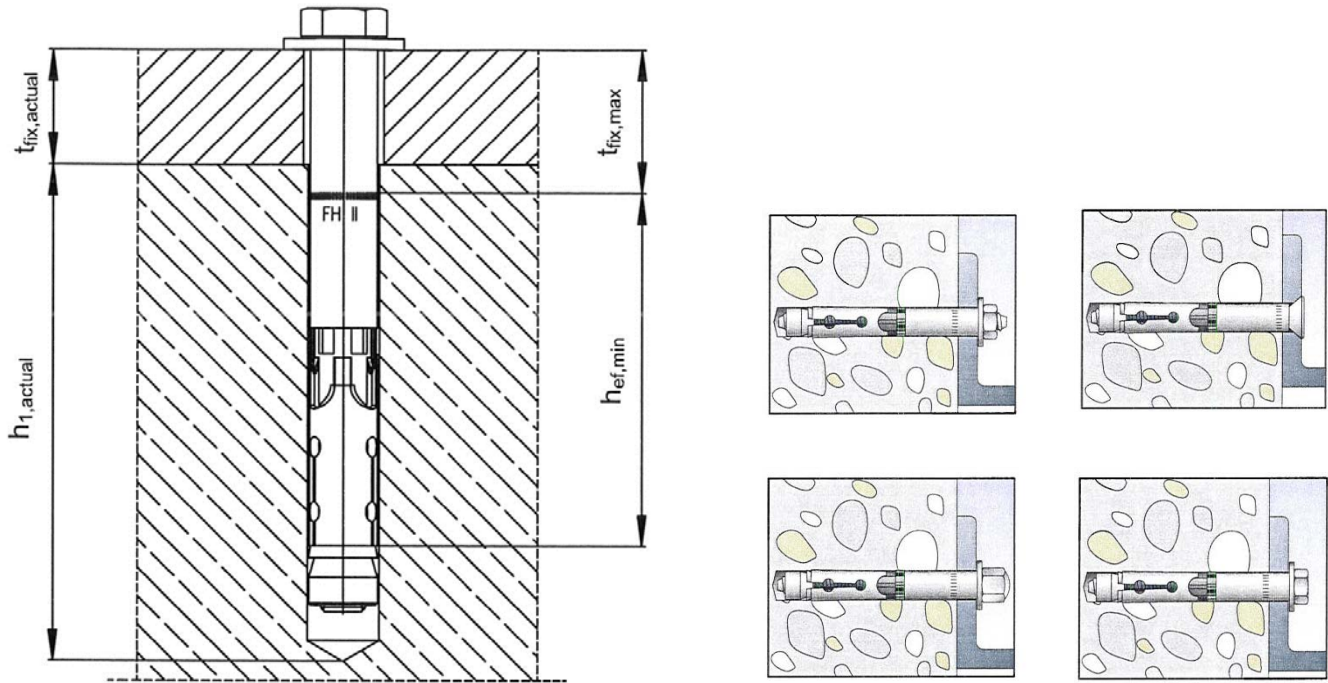


FIGURE 2—CORRECT INSTALLED ANCHOR

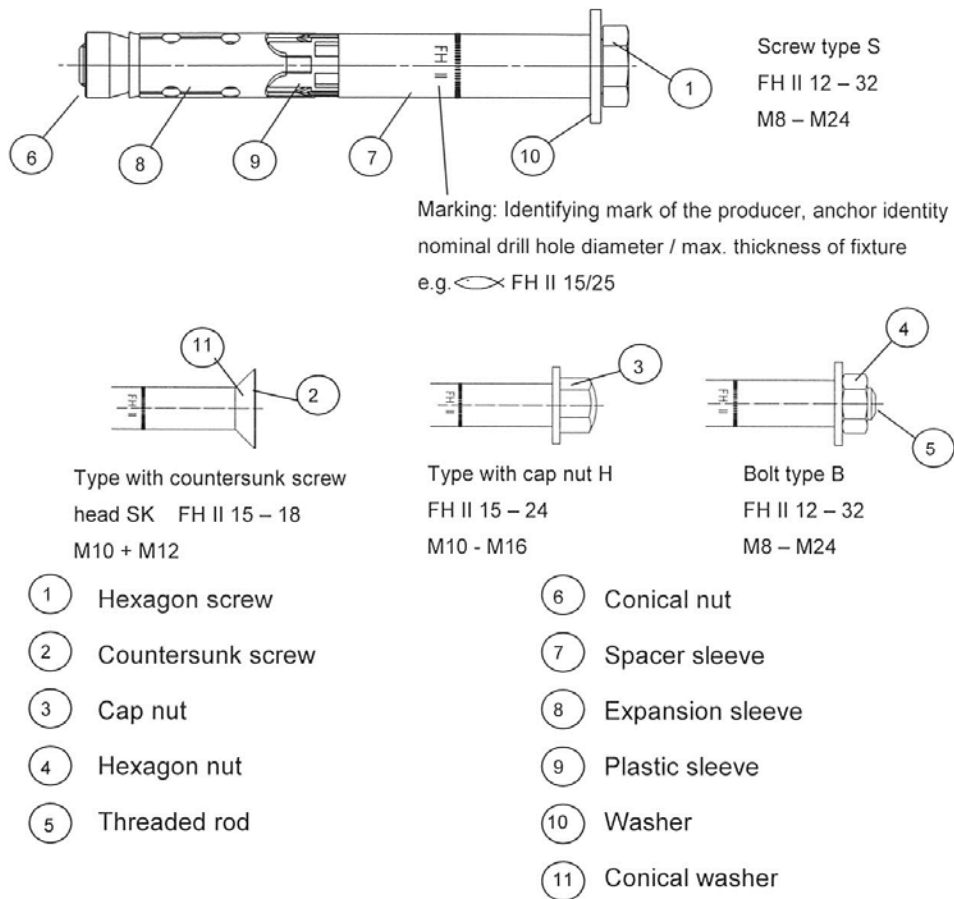


FIGURE 3—FH II HEAD CONFIGURATIONS

TABLE 3—DESIGN INFORMATION

Design parameter	Symbol	Units	Nominal anchor diameter FH II						
			M8	M10	M12	M16	M20	M24	
Outside diameter of anchor	$d_a (d_c)^{8,9}$	mm	12	15	18	24	28	32	
		in.	0.47	0.59	0.71	0.94	1.10	1.26	
Effective min. embedment depth ¹	$h_{ef,min}$	mm	60	70	80	100	125	150	
		in.	2.36	2.76	3.15	3.94	4.92	5.91	
Anchor category ²	1,2 or 3	-	1	1	1	1	1	1	
Strength reduction factor for tension, steel failure modes ³	ϕ	-	0.75						
Strength reduction factor for shear, steel failure modes ³	ϕ	-	0.65						
Strength reduction factor for tension, concrete failure modes ^{4,5}	ϕ	Cond.A	0.75						
		Cond.B	0.65						
Strength reduction factor for shear, concrete failure modes ^{4,5}	ϕ	Cond.A	0.75						
		Cond.B	0.70						
Yield strength of anchor steel	f_{ya}	N/mm ²	640						
		lbf/in. ²	92,824						
Ultimate strength of anchor steel	f_{uta}	N/mm ²	800						
		lbf/in. ²	116,030						
Tensile stress area	A_{se}	mm ²	36.6	58.0	84.3	156.7	244.8	352.5	
		in. ²	0.057	0.090	0.131	0.243	0.379	0.546	
Steel strength in tension	$N_{seN} (N_{sa})^9$	kN	29	46	67	125	196	282	
		lbf	6,516	10,337	15,056	28,090	44,045	63,370	
Effectiveness factor cracked concrete	k_{cr}	SI	7.1	8.8	8.8	10	10	10	
		Imp	17	21	21	24	24	24	
Effectiveness factor uncracked concrete	k_{un-cr}	SI	11.3	11.3	11.3	12.6	12.6	12.6	
		Imp	27	27	27	30	30	30	
Additional effectiveness factor for uncracked concrete ⁶	$\psi_{c,N}$	-	1.0	1.0	1.0	1.0	1.0	1.0	
Pullout strength uncracked concrete ⁷	$N_{p,un-cr}$		not decisive						
Pullout strength cracked concrete ⁷	$N_{p,cr}$		not decisive						
Tension pullout strength seismic ⁷	N_{eq}		not decisive						
Steel strength in shear S, SK	V_{sa}	kN	33	59	76	146	174	217	
		lbf	7,419	13,264	17,085	32,822	39,117	48,784	
Steel strength in shear B, H	V_{sa}	kN	27	41	62	119	146	169	
		lbf	6,070	9,217	13,938	26,752	32,822	37,993	
Steel strength in shear, seismic S,SK	V_{eq}	kN	25	38	57	110	143	174	
		lb	5,620	8,543	12,814	24,729	32,148	39,117	
Steel strength in shear, seismic B,H	V_{eq}	kN	17	31	46	98	119	169	
		lb	3,822	6,969	10,341	22,031	26,752	37,993	
Coefficient for pryout strength	k_{cp}	[-]	1	2					
Effective length of anchor in shear loading	l_e	mm	24	30	36	48	56	64	
		in.	0.94	1.18	1.42	1.88	2.20	1.56	
Axial stiffness in service load range	cracked concrete	β_m	kN/mm	21	24	27	33	39	45
			10 ³ lbf/in	120	137	154	188	223	257
	uncracked concrete	β_m	kN/mm	65	91	118	171	224	277
			10 ³ lbf/in	371	520	674	976	1,279	1,582

¹Figure 2 illustrates the location of $h_{ef,min}$.

²Sections ACI 318 Section D.4.4 and D.4.5 set forth the permitted strength reduction factor ϕ based on anchor category.

³The FH II anchors are considered as ductile steel element as defined by ACI 318 D.1.

⁴Condition A requires supplemental reinforcement, while Condition B applies where supplemental reinforcement is not provided or where pullout or pryout governs, as set forth in ACI 318 D.4.4(c). The tabulated value of ϕ applies when the load combinations of Section 1605.2.1 of the IBC or ACI 318 9.2 are used. If the load combinations of ACI 318 Appendix C are used, the appropriate value of ϕ must be determined in accordance with ACI 318 D.4.5(c).

⁵Anchors are permitted to be used in lightweight concrete provided that the provisions in accordance with Section 4.1.12 of this report are taken into account.

⁶The value $\psi_{c,N} = 1.0$ for all design cases.

⁷As described in Section 4.1.5 of this report, pullout resistance is not decisive.

⁸All dimensions are nominal excluding manufacturing tolerances.

⁹The notation in brackets is for the 2006 IBC.

TABLE 4—EDGE DISTANCE, SPACING AND MEMBER THICKNESS REQUIREMENTS^{1,2}

Dimension	Symbol	Units	FH II 12 M8	FH II 15 M10	FH II 18 M12	FH II 24 M16	FH II 28 M20	FH II 32 M24
Minimum thickness of concrete member	$h_{a,min}$	[mm]	120	140	160	200	250	300
		[in.]	4.75	5.50	6.25	7.88	9.88	11.88
Minimum spacing ^{1,3}	$s_{a,min}$	[mm]	60	70	80	100	120	160
	for $c_a \geq$	[mm]	100	120	160	220	240	380
Minimum spacing ^{1,3}	$s_{a,min}$	[in.]	2.36	2.76	3.15	3.94	4.72	6.30
	for $c_a \geq$	[in.]	3.94	4.72	6.30	8.66	9.45	14.96
Minimum edge distance ^{1,3}	$c_{a,min}$	[mm]	60	70	80	100	120	180
	for $s_a \geq$	[mm]	120	140	200	240	260	400
Minimum edge distance ^{1,3}	$c_{a,min}$	[in.]	2.36	2.76	3.15	3.94	4.72	7.09
	for $s_a \geq$	[in.]	4.72	5.51	7.87	9.45	10.24	15.75
Critical edge distance ²	c_{ac}	[mm]	155	165	170	225	265	285
		[in.]	6.10	6.50	6.69	8.86	10.43	11.22

¹According to section 4.1.9 of this report.

²According to section 4.1.4 of this report.

³Intermediate values by linear interpolation.

TABLE 5—EXAMPLE ALLOWABLE STRESS DESIGN VALUES FOR ILLUSTRATIVE PURPOSES

Anchor type FH II S, SK, B, H	Effective Embedment depth h_{ef}	Allowable Tension Load
FH II 12 M8	2.36 in	2,150 lbf
	60 mm	9.57 kN
FH II 15 M10	2.76 in	2,719 lbf
	70 mm	12.05 kN
FH II 18 M12	3.15 in	3,315 lbf
	80 mm	14.73 kN
FH II 24 M16	3.94 in	5,152 lbf
	100 mm	22.77 kN
FH II 28 M20	4.92 in	7,189 lbf
	125 mm	31.82 kN
FH II 32 M24	5.91 in	9,465 lbf
	150 mm	41.83 kN

Design Assumptions:

¹Single anchor with static tension load only.

²Concrete determined to remain uncracked for the life of the anchorage.

³Load combinations from ACI 318 Section 9.2 (no seismic loading).

⁴30% dead load and 70% live load, controlling load combination 1.2 D + 1.6 L.

⁵Calculation of weighted average for $\alpha = 0.3 \times 1.2 + 0.7 \times 1.6 = 1.48$.

⁶ $f'_c = 2,500$ psi (17.2 MPa) (normal weight concrete).

⁷ $c_{a1} = c_{a2} \geq c_{ac}$

⁸ $h \geq h_{min}$.

⁹Condition B according to ACI 318 Appendix D, Section 4.4(c), no supplementary reinforcement is present.

Given:
 2 x FH II 12 S (M8)
 Slab on grade 5,076 psi
 No supplementary reinforcement =>
 Condition B per ACI 318 D.4.4 c)
 Assume cracked concrete
 $h_a = 4.72$ in.
 $h_{ef} = 2.36$ in.
 $s_a = 3.94$ in.
 $c_a = 3.15$ in.

Calculate the allowable static tension load for this configuration.

Calculation according to ACI 318-05 Appendix D and this report	ACI 318-05 Ref.	Report Ref.
<p>Step 1. Verify minimum member thickness, spacing and edge distance: Table 4: $h_a = 4.72$ in. $\geq h_{a,min} = 4.72$ in. => ok slope: $\frac{4.72 - 2.36}{2.36 - 3.94} = -1.5$ For $c_a = 3.15$ in. $s_{min,calc} = [(3.15 - 3.94) \times (-1.5)] + 2.36 = 3.54$ in. $\leq s_a = 3.94$ => ok</p>	D.8	Table 4
<p>Step 2. Calculate steel strength of anchor in tension $N_{sa} = n N_s = 13,032$ lbf Calculate steel capacity $\phi N_{sa} = 0.75 \times 13,032 = 9,774$ lbf</p>	D.5.1.2 D.4.4 b)	Table 3
<p>Step 3. Calculate concrete breakout strength of anchor in tension $N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \times \psi_{ec,N} \times \psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N} \times N_b$</p>	D.5.2.1 b)	§ 4.1.3 § 4.1.5
<p>Step 3a. Check $1.5 h_{ef} = 1.5 \times 2.36 = 3.54$ in. $> c_a = 3.0$ $h_{ef} = 3.0 \times 2.36 = 7.08$ in. $> s_a$</p>	D.5.2.1 b)	Table 4
<p>Step 3b. Calculate A_{Nco} and A_{Nc} $A_{Nco} = 9 \times h_{ef}^2 = 9 \times (2.36)^2 = 50.13$ in.² $A_{Nc} = (1.5 h_{ef} + c_a) \times (3 h_{ef} + s_a) = [1.5 \times 2.36 + 3.15] \times [3 \times (2.36) + 3.94] = 73.72$ in.² $< 2 \times A_{Nco}$</p>	D.5.2.1 b)	Table 3
<p>Step 3c. Calculate $\psi_{ec,N} : e_N = 0 \Rightarrow \psi_{ec,N} = 1$</p>	D.5.2.4	-
<p>Step 3d. Calculate $N_b = k_{cr} \sqrt{f'_c} h_{ef}^{1.5} = 17 \times \sqrt{5,076} \times 2.36^{1.5} = 4,391$ lbf</p>	D.5.2.2	Table 3
<p>Step 3e. Calculate modification factor for edge distance: $\psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5 \times h_{ef}} = 0.7 + 0.3 \times \frac{3.15}{1.5 \times 2.36} = 0.97 < 1$</p>	D.5.2.5	Table 4
<p>Step 3f. $\psi_{c,N} = 1.0$ (cracked concrete)</p>	D.5.2.6	Table 3
<p>Step 3g. $\psi_{cp,N} = 1.0$</p>	D.5.2.7	Table 3
<p>Step 3h. Calculate $N_{cbg} = (73.72/50.13) \times 1 \times 0.97 \times 1 \times 4,391 = 6,264$ lbf</p>	D.5.2.1 b)	§4.1.1 Table 3
<p>Step 4. Controlling strength: $\phi N_{cbg} = 0.65 \times 6,264 = 4,071$ lbf $< \phi N_s = 9,774$ lbf</p>	D.4.4.c)	Table 3
<p>Step 5. Calculate allowable stress design (30% dead load, 70% live load) $\alpha = 1.2 \times 0.3 + 1.6 \times 0.7 = 1.48 \rightarrow T_{allowable, ASD} = 4,071 / 1.48 = 2,751$ lbf</p>	ACI 318 Section 9.2	§4.2

FIGURE 4—EXAMPLE CALCULATION FOR TENSION CAPACITY (IMPERIAL UNITS)

Given:
 2 x FH II 12 S (M8)
 Slab on grade 35 MPa
 No supplementary reinforcement. =>
 Condition B per ACI 318 D.4.4 c)
 Assume cracked concrete
 $h_a = 120$ mm
 $h_{ef} = 60$ mm
 $s_a = 100$ mm
 $c_a = 80$ mm

Calculate the allowable static and seismic tension load for this configuration.

Calculation according to ACI 318-05 Appendix D and this report	ACI 318-05 Ref.	Report Ref.
<p>Step 1. Verify minimum spacing and edge distance: Table 4: $h_a = 120$ mm $\geq h_{a,min} = 120$ mm => ok</p> <p>slope: $\frac{120 - 60}{60 - 100} = -1.5$ For $c_a = 80$ mm</p> <p>$s_{min,calc} = [(80-100) \times (-1.5)] + 60 = 90$ mm $\leq s_a$ => ok</p>	D.8	Table 4
<p>Step 2. Calculate steel strength of anchor in tension $N_{sa} = n N_s = 58$ kN Calculate steel capacity $\phi N_{sa} = 0.75 * 58 = 43.5$ kN</p>	D.5.1.2 D.4.4 b)	Table 3
<p>Step 3. Calculate concrete breakout strength of anchor in tension</p> $N_{cbg} = \frac{A_{Nc}}{A_{Nco}} \times \psi_{ec,N} \times \psi_{ed,N} \times \psi_{c,N} \times \psi_{cp,N} \times N_b$	D.5.2.1 b)	§ 4.1.1 § 4.1.2
<p>Step 3a. Check $1.5 h_{ef} = 1.5 \times 60 = 90$ mm $> c_a$ $3.0 h_{ef} = 3.0 \times 60 = 180$ mm $> s_a$</p>	D.5.2.1 b)	Table 4
<p>Step 3b. Calculate A_{Nco} and A_{Nc}. $A_{Nco} = 9 \times h_{ef}^2 = 9 \times (60)^2 = 32,400$ mm² $A_{Nc} = (1.5 h_{ef} + c) \times (3 h_{ef} + s) = [1.5 \times 60 + 80] \times [3 \times (60) + 100] = 47,600$ mm² $< 2 \times A_{Nco}$</p>	D.5.2.1 b)	Table 3
<p>Step 3c. Calculate $\psi_{ec,N} : e_N = 0 \Rightarrow \psi_{ec,N} = 1$</p>	D.5.2.4	-
<p>Step 3d. Calculate $N_b = k_c \sqrt{f'_c} h_{ef}^{1.5} = 7 \times \sqrt{35} \times 60^{1.5} = 19,246$ N = 19.25 kN</p>	D.5.2.2	Table 3
<p>Step 3e. Calculate modification factor for edge distance:</p> $\psi_{ed,N} = 0.7 + 0.3 \frac{c_{a,min}}{1.5 \times h_{ef}} = 0.7 + 0.3 \times \frac{80}{1.5 \times 60} = 0.97 < 1$	D.5.2.5	Table 4
<p>Step 3f. $\psi_{c,N} = 1.0$ (cracked concrete)</p>	D.5.2.6	Table 3
<p>Step 3g. $\psi_{cp,N} = 1.0$</p>	D.5.2.7	Table 3
<p>Step 3h. Calculate $N_{cbg} = (47,600/32,400) \times 1 \times 0.97 \times 1 \times 19,246 = 27.33$ kN</p>	D.5.2.1 b)	§4.1.1 Table 3
<p>Step 4. Controlling strength: $\phi N_{cbg} = 0.65 \times 27.33 = 17.77$ kN $< \phi N_s = 43.5$ kN</p>	D.4.4.c)	Table 3
<p>Step 5. Calculate allowable stress design (30% dead load, 70% live load) $\alpha = 1.2 \times 0.3 + 1.6 \times 0.7 = 1.48 \rightarrow T_{allowable, ASD} = 17.77 / 1.48 = 12,01$ kN</p>	ACI 318 Section 9.2	§4.2

FIGURE 5—EXAMPLE CALCULATION FOR TENSION CAPACITY (SI UNITS)

<p>Given: 2 x FH II 12 S (M8) Slab on grade = 5,076 psi No supplementary reinforcement. => Condition B per ACI 318 D.4.4 c) Assume cracked concrete $h_a = 4.72$ in. $h_{ef} = 2.36$ in. $s_a = 3.94$ in. $c_{a1} = 3.15$ in. $c_{a2} = 7.87$ in.</p> <p>Calculate the allowable static shear load for this configuration.</p>		
<p>Calculation according to ACI 318-05 Appendix D and this report</p>	<p>ACI 318-05 Ref.</p>	<p>Report Ref.</p>
<p>Step 1. Verify minimum spacing and edge distance according to Figure 4 – Tension</p>	<p>D.8</p>	<p>Figure 4</p>
<p>Step 2. Calculate steel strength of anchor in shear $V_{sg} = n V_{sa} = 14,838$ lbf Calculate steel capacity $\phi V_{sg} = 0.65 * 14,832 = 9,645$ lbf</p>	<p>D.6.1 D.4.4 b)</p>	<p>Table 3</p>
<p>Step 3. Calculate concrete breakout strength of anchor in shear $V_{cbg} = \frac{A_{vc}}{A_{vc0}} \times \psi_{ec,V} \times \psi_{ed,V} \times \psi_{c,V} \times V_b$</p>	<p>D.6.2.1 b)</p>	<p>§ 4.1.4 § 4.1.6</p>
<p>Step 3a. Check $3 c_{a1} = 3 \times 3.15 = 9.45$ in. > $s = 3.94$ in. s_a controls $1.5 c_{a1} = 1.5 \times 3.15 = 4.72$ in. < h_a c_{a1} controls $1.5 c_{a1} < c_{a2}$ c_{a1} controls</p>	<p>D.6.2.2</p>	<p>-</p>
<p>Step 3b. Calculate A_{vc0} and A_{vc} $A_{vc} = 4.5 \times c_1^2 = 4.5 \times (3.15)^2 = 44.65$ in.² $A_{vc0} = (1.5 c_{a1}) \times (3 c_{a1} + s) = [1.5 \times 3.15] \times [3 \times (3.15) + 3.94] = 63.27$ in.² < $2 \times A_{vc0}$</p>	<p>D.6.2.2</p>	<p>-</p>
<p>Step 3c. Calculate $\psi_{ec,V}$: $e_v = 0 \Rightarrow \psi_{ec,V} = 1$</p>	<p>D.6.2.5</p>	<p>-</p>
<p>Step 3d. Calculate $V_b = 7 \sqrt{f'_c} \times \sqrt{d_0} \times c_{a1}^{1.5} \times \left(\frac{l_e}{d_0}\right)^{0.2} = 7 \sqrt{5,076} \times \sqrt{0.47} \times 3.15^{1.5} \times \left(\frac{0.94}{0.47}\right)^{0.2} = 2,196$ lbf</p>	<p>D.6.2.2</p>	<p>-</p>
<p>Step 3e. Calculate modification factor for edge distance: $c_{a2} = 7.87 \geq 1.5 \times c_{a1} = 7.2$ $\psi_{ed,V} = 1$</p>	<p>D.6.2.6</p>	<p>-</p>
<p>Step 3f. $\psi_{c,V} = 1.0$ (cracked concrete)</p>	<p>D.6.2.7</p>	<p>-</p>
<p>Step 3g. Calculate $V_{cbg} = (63.27/44.65) \times 1 \times 1 \times 2,196 = 3,112$ lbf</p>	<p>D.6.2.1 b)</p>	<p>-</p>
<p>Step 3h. Calculate $\phi V_{cbg} = 3,112$ lbf $\times 0.7 = 2,178$ lbf</p>	<p>D.4.4 c)</p>	<p>Table 3</p>
<p>Step 4. Calculate Pryout: $\phi V_{cp} = k_{cp} \times \phi N_{cbg} = 1 \times 0.7 \times 6,264 = 4,385$ lbf N_{cbg} According to Figure 4 – Tension ($c_{a2} > 1.5 h_{ef}$)</p>	<p>D.6.3.1 b)</p>	<p>Table 3</p>
<p>Step 5. Controlling strength: $\phi V_n = \min \{ \phi V_{cp}, \phi V_{cbg}, \phi V_{sg} \} = 2,178$ lbf (static)</p>	<p>D.4.1.2</p>	<p>-</p>
<p>Step 6. Calculate allowable stress design (30% dead load, 70% live load) $\alpha = 1.2 \times 0.3 + 1.6 \times 0.7 = 1.48 \rightarrow V_{allowable, ASD} = 2,178 / 1.48 = 1,471$ lbf</p>	<p>ACI 318 Section 9.2</p>	<p>§4.2</p>

FIGURE 6—EXAMPLE CALCULATION FOR SHEAR CAPACITY (IMPERIAL UNITS)

Given:
 2 x FH II 12 S (M8)
 Slab on grade 35 Mpa

No supplementary reinforcement. =>
 Condition B per ACI 318 D.4.4 c)
 Assume cracked concrete
 $h_a = 120$ mm
 $h_{ef} = 60$ mm
 $s_a = 100$ mm
 $c_{a1} = 80$ mm
 $c_{a2} = 200$ mm.

Calculate the allowable static shear load for this configuration.

Calculation according to ACI 318-05 Appendix D and this report	ACI 318-05	Report Ref.
Step 1. Verify minimum spacing and edge distance according to Figure 5 – Tension	D.8	Figure 5
Step 2. Calculate steel strength of anchor in shear $V_{sg} = n V_{sa} = 66$ kN Calculate steel capacity $\phi V_{sg} = 0.65 * 68 = 42.9$ kN	D.6.1 D.4.4 b)	Table 3
Step 3. Calculate concrete breakout strength of anchor in shear $V_{cbg} = \frac{A_{Vc}}{A_{Vc0}} \times \psi_{ec,V} \times \psi_{ed,V} \times \psi_{c,V} \times V_b$	D.6.2.1 b)	§ 4.1.4 § 4.1.6
Step 3a. Check $3 c_{a1} = 3 \times 80 = 240$ mm > $s = 100$ mm s controls $1.5 c_{a1} = 1.5 \times 80 = 120$ mm < h_a c_{a1} controls $1.5 c_{a1} < c_{a2}$ c_{a1} controls	D.6.2.2	-
Step 3b. Calculate A_{Vc0} and A_{Vc} $A_{Vc} = 4.5 \times c_1^2 = 4.5 \times (80)^2 = 28,800$ mm ² $A_{Vc0} = (1.5 c_{a1}) \times (3 c_{a1} + s) = [1.5 \times 80] \times [3 \times (80) + 100] = 40,800$ mm ² < $2 \times A_{Vc0}$	D.6.2.2	-
Step 3c. Calculate $\psi_{ec,V}$: $e_V = 0 \Rightarrow \psi_{ec,V} = 1$	D.6.2.5	-
Step 3d. Calculate $V_b = 0.6 \sqrt{f'_c} \times \sqrt{d_0} \times c_{a1}^{1.5} \times \left(\frac{l_e}{d_0}\right)^{0.2} = 0.6 \times \sqrt{35} \times \sqrt{12} \times 80^{1.5} \times \left(\frac{24}{12}\right)^{0.2} = 10.11$ kN	D.6.2.2	-
Step 3e. Calculate modification factor for edge distance: $c_{a2} = 200 \geq 1.5 \times c_{a1} = 120$ $\psi_{ed,V} = 1$	D.6.2.6	-
Step 3f. $\psi_{c,V} = 1.0$ (cracked concrete)	D.6.2.7	-
Step 3g. Calculate $V_{cbg} = (40,800/28,800) \times 1 \times 1 \times 10.11 = 14.32$ kN	D.6.2.1 b)	-
Step 3h. Calculate $\phi V_{cbg} = 14.32$ kN $\times 0.7 = 10.02$ kN	D.4.4 c)	Table 3
Step 4. Calculate Pryout: $\phi V_{pg} = k_{cp} \times \phi N_{cbg} = 1 \times 0.7 \times 27.33 = 19.13$ kN N_{cbg} According to Figure 5 – Tension ($c_{a2} > 1.5 h_{ef}$)	D.6.3.1 b)	Table 3
Step 5. Controlling strength: $\phi V_n = \min \{ \phi V_{pg}, \phi V_{cbg}, \phi V_{sg} \} = 10.02$ kN (static)	D.4.1.2	-
Step 6. Calculate allowable stress design (30% dead load, 70% live load) $\alpha = 1.2 \times 0.3 + 1.6 \times 0.7 = 1.48 \rightarrow V_{allowable, ASD} = 10.02 / 1.48 = 6.77$ kN	ACI 318 Section 9.2	§4.2

FIGURE 7—EXAMPLE CALCULATION SHEAR CAPACITY (SI UNITS)