The Pry-Out Factor: A Critical Re-examination of the Link Between Tension and Pry-Out Resistance in EN 1992-4

Shear-induced pull-out, test evidence for the concrete term, anchor groups and the treatment of bonded fasteners

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ABSTRACT

EN 1992-4:2018 does not verify concrete pry-out on its own terms. It sets the pry-out resistance of a fastener at a multiple k_8 of its tension resistance: the concrete cone resistance for mechanical fasteners, and the smaller of the cone resistance and the combined pull-out resistance for bonded fasteners, with default values k_8 = 1 or 2. This paper examines that factor. For the bond term, clause 7.2.2.4(1) names the mechanism itself: pull-out under a tension force induced by the shear. With $k_8 = 2$ the code therefore assumes that half of the shear reaches the bond as tension. The mechanics of a laterally loaded anchor in concrete, with local crushing at the mouth of the hole, a pivot at $1.5d$–$2d$ and a tilted shank, give an induced tension of 14–27 % of the shear for plastic tilts of 8°–15°, so the factor between bond resistance and the shear it can resist is about 4 to 7. A credibility criterion shows that shear-induced pull-out can precede steel shear failure only below a bond-strength threshold that depends on h_{ef}/d alone; in 76 single-anchor configurations within the admitted embedment range the bond term governed 21 times under the code and never with a factor of 4. For the concrete term, the largest recent test database shows the code to be conservative, with mean test-to-prediction ratios of 1.27 for single anchors and 1.63 for groups, corresponding to an effective factor of about 2.5 and 3.3, and product assessments already grant k_8 up to 2.9. The group rules reduce the resistance with overlap factors derived for tension, although for anchors in line with the load the rear anchor compresses the concrete in which the front anchor's crater must form. The provisions are also inconsistent: the same Hilti HUS4 screw is checked without its pull-out resistance as a mechanical fastener and with it as a bonded screw, and the mortar is credited for embedment depth but not for diameter. A separate factor $k_p \geq 4$ on the bond term and a test-based

recalibration of k_8 for single anchors and groups are proposed, with a test programme to confirm them.

Keywords: post-installed anchors; bonded anchors; concrete pry-out; EN 1992-4; shear-induced tension; dowel action; catenary action; anchor groups; pry-out factor k_8

Notation

Symbol	Meaning
A, A_s	nominal and stressed cross-section of the rod
d, d_0	rod diameter, drill-hole diameter
e, z	eccentricity of the concrete bearing resultant; internal lever arm of the fixture
E_c, E_s, E_m	moduli of concrete, steel and mortar
f_{ck}, f_{uk}	characteristic concrete cylinder strength; steel ultimate strength
h_{ef}, l_1	effective embedment depth; depth of the plastic hinge
k_6, k_8	steel shear factor; pry-out factor (EN 1992-4)
k_p	factor on $N_{Rk,p}$ proposed in this paper
$M_{Rk,s}$	characteristic bending resistance of the rod
N, V	induced axial force; applied shear
$N_{Rk,c}$, $N_{Rk,p}$	concrete cone resistance; combined pull-out and concrete resistance
$V_{Rk,s}$, $V_{Rk,cp}$	steel shear resistance; pry-out resistance
$\eta = N/V$	ratio of induced tension to shear
θ, θ_u	tilt of the shank above the hinge; tilt at ultimate load
τ_{Rk}, ψ_{sus}	characteristic bond strength; sustained-load factor

1. Introduction

Concrete pry-out is the failure of the concrete *behind* a fastener under shear: the embedded part of a short, stiff anchor rotates, and its lower end levers a crater out of the concrete on the side opposite to the load direction. The mode is real and well documented for stocky headed studs and short post-installed anchors [5–7]. Since the first European design guidance its resistance has been expressed as a multiple of a tension resistance, because the concrete body mobilised at the rear of the anchor resembles, in size and in the way it fails in tension, the cone of a tension breakout.

EN 1992-4 [1] turns that analogy into a fixed ratio. The pry-out resistance is k_8 times the concrete cone resistance $N_{Rk,c}$ for mechanical and cast-in fasteners, and k_8 times the smaller of the cone resistance and the combined pull-out and concrete resistance $N_{Rk,p}$ for bonded fasteners, with default values of $k_8 = 1$ below 60 mm embedment and 2 above. Put simply, the code states that a fastener resists, by pry-out, one to two times what it resists in tension. This paper argues that the ratio is too low, clearly so for the bond term and measurably so for the concrete term, and that the way it is applied is not consistent.

The practical consequence is familiar to designers who work with injection systems. In cracked concrete, at moderate embedment and with a product of modest bond strength, $k_8 \cdot N_{Rk,p}$ can become the governing shear resistance of the whole fastening. It can fall below the steel shear resistance of the rod and below the pry-out resistance $k_8 \cdot N_{Rk,c}$ that the same anchor would be credited with if it were mechanical. Three research questions are addressed:

- **RQ1.** For the bond term: what ratio of induced tension to shear does Eq. (7.39c) imply, what ratio follows from the mechanics of anchor rotation in concrete, and when can shear-induced pull-out precede steel or concrete failure?
- **RQ2.** For the concrete term: what do test databases and product assessments indicate about the factor k_8, for single anchors and for groups?
- **RQ3.** Are the provisions consistent between mechanical and bonded fasteners, between single anchors and groups, and in the way the mortar of a bonded fastener is credited?

The method is analytical. A mechanical model is assembled from established elements, beam-on-elastic-foundation theory [13, 14], plastic dowel action [8, 12, 21] and equilibrium of the tilted shank, and used to derive a closed-form credibility criterion that is exercised in a parametric study. For the concrete term, published test data and European Technical Assessments are evaluated. No new tests were performed; Section 9 specifies the tests needed to confirm or refute the hypotheses. Section 2 reviews the code provisions. Sections 3 to 6 treat the bond term: the kinematics of a laterally loaded anchor (Section 3), a worked example (Section 4), the implied and mechanical tension ratios (Section 5) and a credibility criterion with a parametric study (Section 6). Section 7 treats the concrete term, including anchor groups and plates. Section 8 sets out the inconsistencies, with the Hilti HUS4 screw as the main example. Limitations, a proposed provision and a test programme follow in Section 9.

The calculations follow EN 1992-4:2018 as implemented in the ADIT Anchor Software (AAS), available through www.adit.org.il and eng.adit.org.il. The paper continues the author's critical reading of EN 1992-4 begun in [19].

2. The codified provisions and their origin

2.1 EN 1992-4:2018, clause 7.2.2.4

Clause 7.2.2.4(1) of EN 1992-4 [1] defines the mode and, unusually for a design standard, gives the reason for the bond term:

> *"Fastenings may fail due to a concrete pry-out failure at the side opposite to load direction. Pull-out failure may also occur due to a tension force introduced in the fasteners by the shear load. For reason of simplicity this effect is not verified explicitly, but implicitly accounted for in the verification for pry-out failure, where relevant."*

A Note adds that "the tension force is caused by the eccentricity between the applied shear force and the resultant of the resistance in the concrete". The resistances follow in 7.2.2.4(2) and (3):

$$V_{Rk,cp} = k_8 \cdot N_{Rk,c} \qquad \text{headed and mechanical post-installed fasteners} \qquad (7.39a)$$

$$V_{Rk,cp} = k_8 \cdot \min\{N_{Rk,c};\, N_{Rk,p}\} \qquad \text{bonded fasteners} \qquad (7.39c)$$

Both are multiplied by 0.75 where supplementary reinforcement is present, and k_8 "is a factor to be taken from the relevant European Technical Product Specification". $N_{Rk,p}$ is the combined pull-out and concrete resistance of 7.2.1.6, built from $N^0_{Rk,p} = \psi_{sus}\, \tau_{Rk}\, \pi\, d\, h_{ef}$ (Eq. 7.14).

Two features of this text matter for what follows. First, the bond term is not a second estimate of the concrete crater. It stands for a *different* mechanism, pull-out under shear-induced tension, placed inside the pry-out check "for reason of simplicity". Second, the same mechanism is not considered for mechanical fasteners (Eq. 7.39a), although torque-controlled expansion anchors and concrete screws also have a pull-out resistance $N_{Rk,p}$, which in cracked concrete is often their governing tension mode.

2.2 Where the factors 1 and 2 come from

EOTA TR 029 [2], the predecessor design method for bonded anchors, already contained both expressions: $V_{Rk,cp} = k \cdot N_{Rk,p}$ (5.7) and $V_{Rk,cp} = k \cdot N_{Rk,c}$ (5.7a), the lower being decisive. Its default values, still the values most frequently reproduced in European Technical Assessments, are introduced with a telling qualification:

> *"For anchors according to current experience failing under tension load by concrete cone failure the following values are on the safe side: $k = 1$ for $h_{ef} < 60$ mm, $k = 2$ for $h_{ef} \geq 60$ mm."*

The defaults were therefore calibrated for anchors whose tension capacity is governed by the *concrete cone*. They were carried over to the bond term without any stated calibration against shear tests in which bonded anchors failed by pull-out. The same structure appears in ACI 318-19 [4] (17.7.3), where $V_{cp} = k_{cp}N_{cp}$, N_{cp} is the lesser of N_a (bond) and N_{cb} (cone) for adhesive anchors, and $k_{cp} = 1.0$ or 2.0 with the break at $h_{ef} = 2.5$ in.

Table 1 – Treatment of shear-induced pull-out in the design methods reviewed

Document	Mechanical / cast-in	Bonded	Default factor
EOTA TR 029 (2007/2010) [2]	—	$k \cdot \min(N_{Rk,p}; N_{Rk,c})$	1 / 2
EN 1992-4:2018 [1]	$k_8 \cdot N_{Rk,c}$	$k_8 \cdot \min(N_{Rk,c}; N_{Rk,p})$	per ETA
ACI 318-19 [4]	$k_{cp} \cdot N_{cb}$	$k_{cp} \cdot \min(N_a; N_{cb})$	1.0 / 2.0

In all three documents the pry-out resistance is tied to a tension resistance of the same fastener by a factor of 1 or 2, and in two of them the bond resistance is included in that tension resistance. The rest of this paper asks whether the factor is right, first for the bond term (Sections 3 to 6) and then for the concrete term (Section 7), and whether it is applied consistently (Section 8).

3. Mechanics of a laterally loaded anchor in solid concrete

Sections 3 to 6 deal with the bond term, $k_8 \cdot N_{Rk,p}$, and with the mechanism the code gives for it, shear-induced tension.

3.1 Stiffness contrast and localized crushing

Even when an anchor completely fills its hole, as a bonded anchor, a concrete screw or an expanded sleeve does, the surrounding concrete is not a rigid support. Its modulus ($E_{cm} = 31$ GPa for C25/30 [17]) is about one-seventh of that of steel ($E_s = 210$ GPa). A shear load applied at the concrete surface is therefore resisted by a pressure distribution along the shank that is strongly concentrated at the mouth of the hole.

The elastic magnitude of that concentration can be estimated by treating the shank as a semi-infinite beam on an elastic foundation [13]. With Vesić's foundation modulus [14],

$$k = 0.65\left(\frac{E_c\, d^4}{E_s I}\right)^{1/12} \frac{E_c}{1-\nu_c^2}, \qquad \lambda = \left(\frac{k}{4E_s I}\right)^{1/4} \qquad (1)$$

a 12 mm shank ($I = 1018$ mm⁴) in C25/30 gives $k \approx 23.0$ kN/mm² and $\lambda = 0.072$ mm⁻¹ ($1/\lambda = 13.9$ mm $\approx 1.2d$). For a free head loaded by V at the surface, the peak line load is $q_0 = 2V\lambda$ and the peak bearing pressure over the diameter is

$$p_0 = \frac{2V\lambda}{d} \qquad (2)$$

At $V = 25$ kN this is about 300 N/mm², some twelve times f_{ck}. Confined bearing strength under a pin is several times f_{ck}. Even at four to six times f_{ck}, the elastic peak reaches it at a shear of only about 8–13 kN. Local crushing and micro-cracking at the front upper edge of the hole is therefore not an accident of poor installation. At serviceability-to-ultimate load levels it is unavoidable, and it creates the physical space the shank needs to tilt.

3.2 Migration of the pivot and the plastic hinge

In the elastic solution the deflection first changes sign at $3\pi/(4\lambda) \approx 33$ mm $\approx 2.7d$, and the maximum moment occurs at $\pi/(4\lambda) \approx 11$ mm $\approx 0.9d$. As the upper concrete yields, the bearing resultant moves downward. Test observations and plastic dowel models consistently place the point of maximum moment, and eventually the plastic hinge in the steel, at roughly $1.5d$ to $2d$ below the surface [8, 12, 21]. ACI 318 reflects the same fact from the other side: the load-bearing length used for concrete edge breakout is limited to $8d$, because only the upper part of the shank takes part in shear transfer [4]. The anchor thus behaves as a lever tilting about a submerged pivot, and at ultimate load it kinks at a hinge at depth $l_1 \approx 2d$.

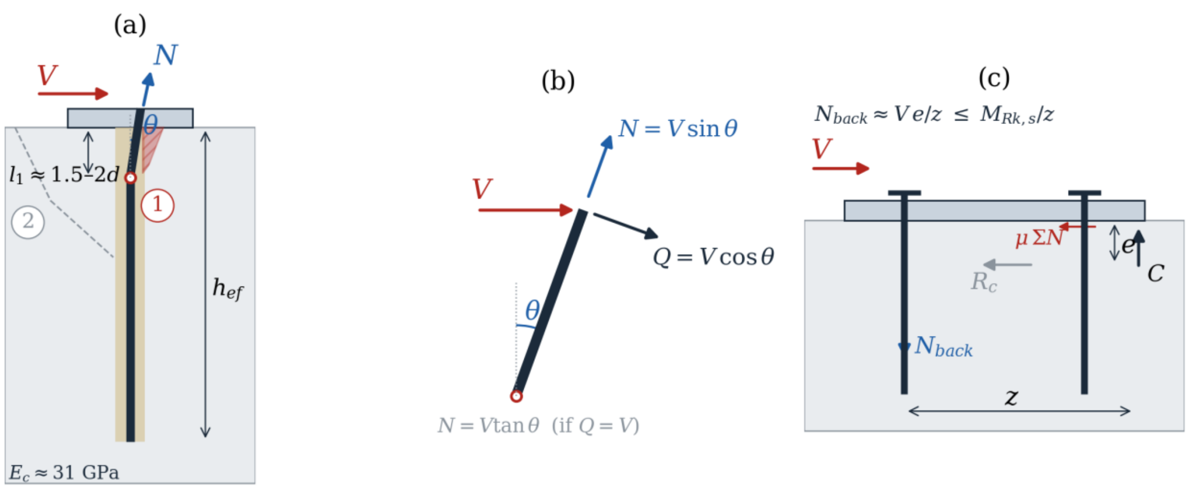

Figure 1 – (a) Laterally loaded bonded anchor: local crushing at the front upper edge (1), pivot/plastic hinge at $l_1 \approx 1.5$–$2d$, tilt θ; the rear pry-out crater (2) is shown dashed. (b) Equilibrium of the tilted segment. (c) Anchor plate: eccentricity

e between the applied shear and the concrete bearing resultant, internal lever arm z, and friction mobilised by the induced tension.

3.3 Two sources of axial force

(a) Kinematic (catenary) tension. Once the segment between the surface and the hinge has tilted by θ, its axis is no longer perpendicular to the shear. The mortar, or the thread interlock of a screw, prevents the shank from sliding freely along the hole, so part of the lateral displacement becomes axial action. For a fixture that is free to separate from the concrete, the equilibrium of the tilted segment (Figure 1b) gives

$$N = V\sin\theta, \qquad Q = V\cos\theta \qquad (3)$$

If the transverse component is instead taken equal to the full shear ($Q = V$), the familiar form $N = V\tan\theta$ follows. The two differ by less than 4 % up to 15°. The ratio

$$\eta = \frac{N}{V} \approx \sin\theta \qquad (4)$$

is the quantity the code implicitly fixes and the quantity this paper examines. The three-component model of Takase [8], who tested post-installed anchors under combined cyclic shear and constant tension, contains exactly this term. In that model the shear is carried by bending at the plastic hinge, by concrete bearing, and by the shear component of the catenary tension. In the same tests the joint separation remained below 1 mm at 6 mm slip when no external tension was applied, which shows that the fixture does separate and that the axial constraint is partly released.

That release matters. Pure geometric compatibility over the hinge length would demand an axial strain of $1/\cos\theta - 1$, which is 0.98 % at 8° and 3.5 % at 15° (0.24 mm and 0.85 mm over 24 mm). That is well beyond the yield strain of the rod. The demand is relieved by local slip in the crushed zone, by fixture uplift and by elastic extension over the bond development length. Where the fixture is prevented from lifting, the axial force can rise above Eq. (3). But the same force then clamps the fixture onto the concrete and mobilises friction, which EN 1992-4 does not credit and which reduces the shear the anchor must carry.

(b) Restraint-couple tension. The Note to 7.2.2.4(1) identifies a second source: the eccentricity e between the applied shear and the resultant of the concrete bearing. If the fixture restrains the head against rotation, the couple $V\,e$ must be balanced by anchor tension acting with an internal lever arm z against compression under the fixture (Figure 1c):

$$N_{back} \approx \frac{V\,e}{z} \leq \frac{M_{Rk,s}}{z} \qquad (5)$$

The upper bound holds because the moment at the head cannot exceed the bending resistance of the rod. For an M12 rod of grade 8.8, $M^0_{Rk,s} = 1.2\,W_{el}\,f_{uk} \approx 105$ N·m, so $N_{back} \leq 5.2$ kN for $z = 20$ mm (a washer), ≤ 2.6 kN for $z = 40$ mm and ≤ 1.0 kN for $z = 100$ mm. For any real base plate this component is a fraction of the kinematic term. It is also self-limiting: it cannot grow with V once the rod has yielded in bending.

4. Worked example: M12 bonded anchor, $h_{ef} = 150$ mm, $V = 25$ kN

Table 2 – Input data

Parameter	**Value**
Applied shear V	25 kN (≈ 2.5 t)
Rod diameter d / stressed area A_s	12 mm / 84.3 mm²
Embedment h_{ef} (h_{ef}/d)	150 mm (12.5)
Concrete	C25/30, E_{cm} = 31 GPa
Hinge depth l_1	≈ 2d = 24 mm

With $h_{ef}/d = 12.5$ the anchor is slender. Its behaviour at ultimate is governed by dowel bending and plastic kinking, not by rigid-body pry-out. The induced tension for the plausible range of plastic tilt is given in Table 3.

Table 3 – Induced axial force for V = 25 kN

θ	$\tan\theta$	$N = V\tan\theta$ **[kN]**	$\sin\theta$	$N = V\sin\theta$ **[kN]**	$1/\eta$
8°	0.1405	3.51	0.1392	3.48	7.1
10°	0.1763	4.41	0.1736	4.34	5.7
12°	0.2126	5.31	0.2079	5.20	4.7
14°	0.2493	6.23	0.2419	6.05	4.0
15°	0.2679	6.70	0.2588	6.47	3.7

A ratio of 15 % to 25 % of the shear, 3.75 kN to 6.25 kN (0.38 t to 0.64 t), corresponds to tilts of about 8.6° to 14.5°.

Steel. The example load is chosen to be realistic for this anchor. Under the Note to EN 1992-4 7.2.2.3.1, $V^0_{Rk,s} = k_6 A_s f_{uk}$, which gives 20.2 kN (grade 4.6), 25.3 kN (5.8), 29.5 kN (A4-70) and 33.7 kN (8.8) for M12. A shear of 25 kN is close to the design steel shear resistance of a grade 8.8 rod ($V_{Rd,s} = 33.7/1.25 = 27.0$ kN) and to the characteristic resistance of a grade 5.8 rod, so it represents the largest shear such an anchor would carry in practice. Combining shear and induced tension with the von Mises criterion on the stressed area,

$$\left(\frac{\sigma}{f_{uk}}\right)^2 + 3\left(\frac{\tau}{f_{uk}}\right)^2 \leq 1 \qquad (6)$$

gives $\tau = 297$ N/mm² and a left-hand side of 0.415 at 8° and 0.422 at 15° for grade 8.8. The shear term alone contributes 0.412, so the induced tension adds less than 3 % to the steel demand.

Concrete and bond. For the same anchor in cracked C25/30, $N_{Rk,c} = 7.7\sqrt{25}\,150^{1.5} = 70.7$ kN. With $\tau_{Rk} = 8$ N/mm², $N_{Rk,p} = 45.2$ kN, and Eq. (7.39c) with $k_8 = 2$ gives $V_{Rk,cp} = 90.5$ kN, 2.7 times the steel shear resistance. The induced tension of 3.5 kN to 6.7 kN uses only 8 % to 15 % of $N_{Rk,p}$. For this slender anchor, then, neither pry-out nor shear-induced pull-out is relevant: steel governs by a wide margin.

The same shear can be checked at the shallowest embedment the rules admit. EN 1992-4 (1.1(2)) requires $h_{ef} \geq 40$ mm, and TR 029 [2] adds that the minimum embedment of a bonded anchor should not be less than $4d$; for M12 this gives $h_{ef,min} = \max\{40;\ 48\} = 48$ mm (the ETA of a particular product may prescribe more). At 48 mm, $N_{Rk,c} = 12.8$ kN and, with $\tau_{Rk} = 8$ N/mm², $N_{Rk,p} = 14.5$ kN. The induced tension of at most 6.7 kN is 46 % of the bond resistance, so bond pull-out is still not reached at 25 kN, whereas $2N_{Rk,c} = 25.6$ kN shows that the concrete pry-out resistance, the documented mode, is exhausted. The bond term, not the concrete term, is the one without a mechanism behind it. The bond term becomes relevant only at smaller h_{ef}/d and lower bond strength, which is the domain examined in Section 6.

5. Implied versus mechanical tension ratio

If shear-induced pull-out is represented by $V = k \cdot N_{Rk,p}$, the ratio of induced tension to shear that the code assumes at failure is

$$\eta_{code} = \frac{1}{k_8} = \begin{cases} 1.0 & k_8 = 1 \\ 0.5 & k_8 = 2 \end{cases} \qquad (7)$$

The mechanics of Section 3 give $\eta \approx 0.14$ to 0.27 over $8° \leq \theta \leq 15°$ (Figure 2). The code value for $h_{ef} \geq 60$ mm is thus about two to three-and-a-half times the kinematic estimate, and the value for $h_{ef} < 60$ mm about four to seven times. The factor supported by the kinematics is

$$k_p = \frac{1}{\eta} \approx \frac{1}{\sin\theta_u} \qquad (8)$$

This gives 7.2 at 8°, 4.1 at 14° and 3.9 at 15°. A minimum factor of **4** corresponds to an ultimate tilt of about 14.5°. It is consistent with the upper part of the assumed range, but it is not a lower bound over the whole range: at a full 15° the kinematic factor is 3.9. The tilt itself can be bracketed. With a plastic zone about one diameter long and an outer-fibre strain ε_u, $\theta_u \approx 2\varepsilon_u$. For strains between 6 % and 12 %, the range between the uniform elongation and the minimum elongation after fracture of grade 8.8 [18], this gives 7° to 14°. That is an order-of-magnitude check, not a measured value.

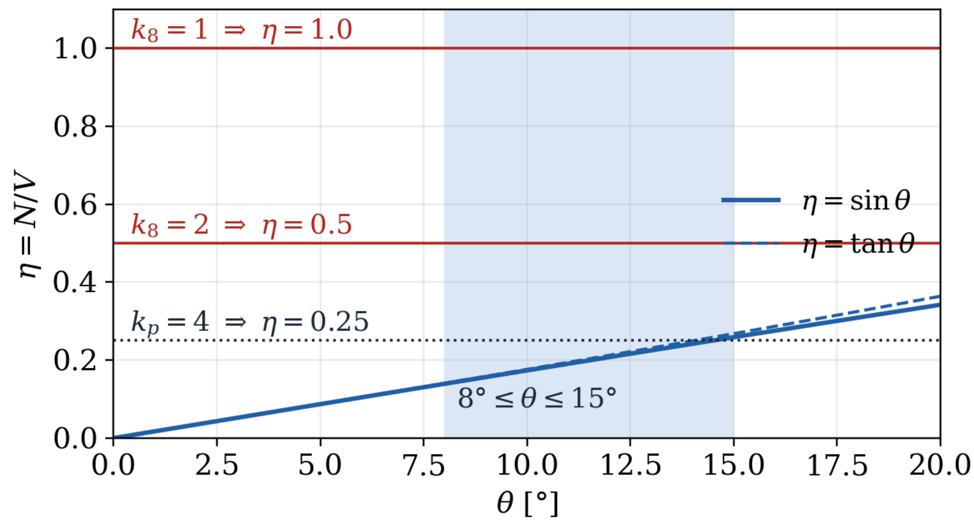

Figure 2 – Ratio of induced axial force to shear, $\eta = N/V$, as a function of tilt, compared with the ratios implied by $k_8 = 1$ and $k_8 = 2$ and by the proposed $k_p = 4$.

6. A credibility criterion for shear-induced pull-out

6.1 Derivation

The shear a single anchor can transmit is capped by its steel resistance. The induced tension is therefore capped at $\eta\, V_{Rk,s}$. Shear-induced pull-out can precede steel shear failure only if

$$N_{Rk,p} < \eta\, V_{Rk,s} \qquad (9)$$

Substituting $N_{Rk,p} = \psi_{sus}\tau_{Rk}\pi d h_{ef}$ and $V_{Rk,s} = k_6 A_s f_{uk}$, and writing $A = \pi d^2/4$:

$$\tau_{Rk}\,\psi_{sus} < \tau^* = \frac{\eta\, k_6\, f_{uk}\,(A_s/A)}{4\, h_{ef}/d} \qquad (10)$$

Equation (10) contains no diameter. The threshold depends only on the steel grade, the tilt-dependent ratio η and the embedment ratio h_{ef}/d. The intuition that small anchors fail in steel first and large anchors have so much tension capacity that the question disappears is therefore half right. What decides is the embedment ratio, not the diameter. With h_{ef} held constant, a larger diameter raises d/h_{ef} and makes the bond term *more* competitive with steel. In practice, however, large diameters at short embedment are governed by the concrete branch $k_8 \cdot N_{Rk,c}$ before either (Table 5).

Table 4 – Threshold bond strength τ^* [N/mm²], M12, grade 8.8 (A_s/A = 0.745)

h_{ef}/d	η = 0.5 (code, k_8 = 2)	η = 0.25 (k_p = 4)
4	9.3	4.7
6	6.2	3.1
8	4.7	2.3
12.5	3.0	1.5

For grade 5.8 the values are 25 % lower (Figure 3). Equation (10) is written in characteristic terms. Design values use $\gamma_{Ms} = 1.25$ for steel shear and $\gamma_{Mc} = 1.5$ for pry-out, which lowers the thresholds by a further factor of $1.25/1.5 = 0.83$. A designer can compare Table 4 directly with the $\tau_{Rk,cr} \cdot \psi^0_{sus}$ published in the product's ETA.

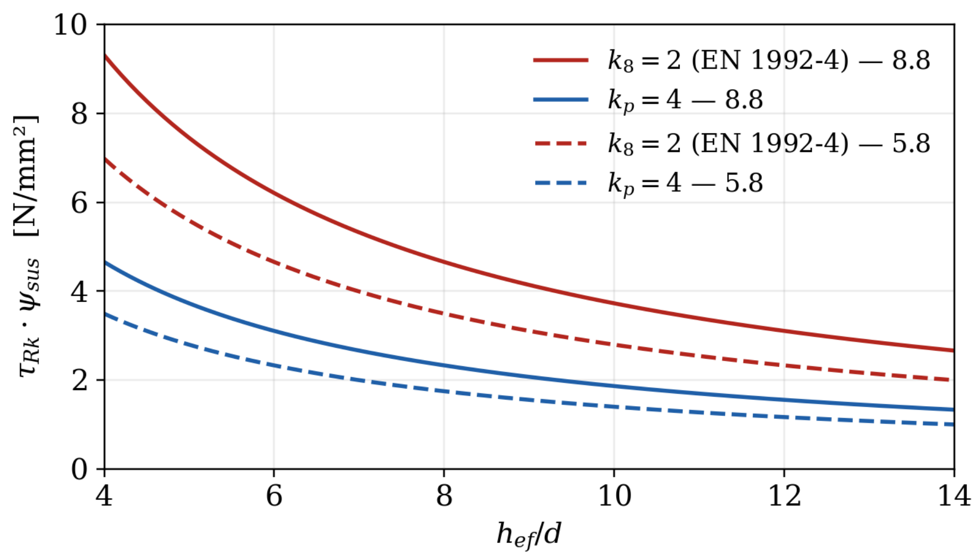

Figure 3 – Threshold bond strength below which the bond term can govern before steel shear failure, as a function of h_{ef}/d, for the code factor $k_8 = 2$ and for the proposed $k_p = 4$.

6.2 Parametric study

A single anchor without edge or spacing influence was analysed in cracked C25/30 with a grade 8.8 rod, d = 8, 12, 16, 20, 24 mm, h_{ef}/d = 4, 6, 8, 12 and τ_{Rk} = 4, 6, 8, 10 N/mm² (ψ_{sus} = 1). Embedment depths below the minimum admitted by the rules, $h_{ef,min}$ = max{40 mm; $4d$} (EN 1992-4 1.1(2); TR 029 [2]), were excluded; this removes M8 at $4d$ = 32 mm and leaves 76 configurations. The shear resistance was taken as the smallest of $V_{Rk,s}$, $2N_{Rk,c}$ and $k\,N_{Rk,p}$, with $k = 2$ (code) or $k = 4$ (proposal). Under the code the bond term governed in **21** configurations, the concrete term in 12 and steel in 43. With $k_p = 4$ the bond term governed in **none**: steel governed in 56 and concrete pry-out in 20. Representative cases are listed in Table 5.

Table 5 – Shear resistance [kN] of representative single anchors, cracked C25/30, grade 8.8, τ_{Rk} = 6 N/mm². V_{Rk} (code) = min($V_{Rk,s}$; $2N_{Rk,c}$; $2N_{Rk,p}$); V_{Rk} (k_p = 4) = min($V_{Rk,s}$; $2N_{Rk,c}$; $4N_{Rk,p}$). Governing mode: S = steel shear, C = concrete pry-out, B = bond term.

Anchor	h_{ef} [mm]	$V_{Rk,s}$	$2N_{Rk,c}$	$2N_{Rk,p}$	$4N_{Rk,p}$	V_{Rk} (code)	Mode	V_{Rk} (k_p = 4)	Mode
M12	48	33.7	25.6	21.7	43.4	**21.7**	B	**25.6**	C
M12	72	33.7	47.0	32.6	65.1	**32.6**	B	**33.7**	S
M12	96	33.7	72.4	43.4	86.9	**33.7**	S	**33.7**	S
M20	80	98.0	55.1	60.3	120.6	**55.1**	C	**55.1**	C
M20	120	98.0	101.2	90.5	181.0	**90.5**	B	**98.0**	S
M20	160	98.0	155.8	120.6	241.3	**98.0**	S	**98.0**	S

The columns $2N_{Rk,p}$ and $4N_{Rk,p}$ give the bond resistance under each factor; the two V_{Rk} columns give the resulting governing resistance. Wherever the bond term governs under the code, raising its factor to 4 lifts it above at least one documented mode (for M12, h_{ef} = 72 mm: 65.1 kN against 33.7 kN for steel) and hands the verdict to a mode that is physically documented, either steel shear or concrete pry-out. The resulting gain in shear resistance is bounded by those modes, typically 3 % to 18 % in Table 5.

7. The concrete term: what tests and assessments say about k_8

Sections 3 to 6 concern the bond term. The concrete term, $k_8 \cdot N_{Rk,c}$, rests on a different footing: the rear crater is a concrete body related to the tension cone, and for short, stiff anchors pry-out is a documented failure mode. The question for this term is therefore not whether it belongs in the verification, but whether a factor of 1 to 2 is right. In practice few anchors work alone, so the answer has to cover groups under a common base plate as well as single anchors.

7.1 Test databases

Jebara, Sharma and Ožbolt [6] evaluated 214 monotonic shear tests (66 single anchors, 54 multiple-row headed-stud groups, 94 multiple-row post-installed groups). Against EN 1992-4 the mean ratio of test to prediction was 1.27 (COV 26.7 %) for single anchors and 1.63 (COV 33.5 %) for groups. Since those predictions were made with the default $k_8 = 2$, a simple scaling gives an effective factor between tension resistance and pry-out resistance of about 2.5 for single anchors and 3.3 for groups. The tests concern stiff anchors, which is where pry-out actually governs, and the ratios are mean values with considerable scatter. Their proposed model, $V_{R,m,cp} = k_{cp,m}\, d^{0.5}\, f_{cc}^{0.5}\, h_{ef}^{1.5}$, applies to stiff anchors with $h_{ef}/d < 4.5$. It shows that pry-out resistance depends on anchor diameter, which the k_8 formulation ignores. Their description of the mechanism is relevant to Section 3.3: an internal overturning moment of the anchor plate induces bearing under the front of the plate and tension in the anchors.

7.2 Values in product assessments

Where products are tested for pry-out, the assessments grant more than the default. The author has previously noted that the assessment of a torque-controlled expansion anchor, the Hilti HSA (ETA-11/0374), gives $k_8 = 2.9$ at M16, which places the pry-out resistance at 1.47 to 2.84 times the anchor's own steel shear resistance [19]. The Hilti HUS4 screw discussed in Section 8.1 carries $k_8 = 1.0$ at the shortest embedment of sizes 8 and 10 and 2.0 otherwise [22]. A product-specific k_8 above 2 is therefore neither unusual nor, in the view of the assessment bodies, unsafe; what is missing is a general recalibration of the default.

7.3 Anchor groups: a second reduction from tension-derived factors

For a group, 7.2.2.4 evaluates $N_{Rk,c}$ and $N_{Rk,p}$ "for all fasteners in a group loaded in shear". The resistances therefore carry the projected-area ratios $A_{c,N}/A_{c,N}^0$ and $A_{p,N}/A_{p,N}^0$, which reduce the resistance of closely spaced fasteners because their tension cones and bond stress fields overlap. For bonded fasteners the bond term also carries the group factor $\psi_{g,Np}$ of Eq. (7.17). All of these factors were derived for fasteners loaded *simultaneously in tension*, each at its full tension resistance.

Under shear that premise does not hold. To begin with, the induced tension in each anchor is only ηV, so the bond stress fields that are supposed to overlap carry a fraction of the load the reduction assumes. More important, for anchors in line with the load the two anchors do not stress the concrete between them in the same way (Figure 4a). Each anchor bears forward on the concrete in front of it over the upper $1.5d$ to $2d$ (zone 1), and each anchor's pry-out crater forms behind it (zone 2). Between two anchors A and B, the crater of the front anchor B must form in the very concrete that the rear anchor A is compressing (zone 3). The compression from A acts against the tensile stresses that the crater of B requires. The code nonetheless treats the zone as an overlap of two tension bodies and deducts it from both (Figure 4b). Because the minimum spacing s_{min} keeps the anchors apart, the bearing zone of the rear anchor lies between the two anchors in every permitted in-line arrangement; and whenever $s < s_{cr,N}$ it lies inside the region that the code counts as overlap.

To put a number on it, take two M12 bonded anchors in line with the shear, $h_{ef} = 72$ mm, $s = 100$ mm, cracked C25/30, $\tau_{Rk,cr} = 6$ N/mm² and $\tau_{Rk,ucr} = 10$ N/mm², without edge influence. Then $s_{cr,N} = 3h_{ef} = 216$ mm, $s_{cr,Np} = \min\left(7.3d\sqrt{\tau_{Rk,ucr}};\, 3h_{ef}\right) = 216$ mm, and both area ratios equal $(216 + 100)/216 = 1.463$. With $\tau_{Rk,c} = 8.67$ N/mm² (Eq. 7.19), $\psi^0_{g,Np} = 1.176$ and $\psi_{g,Np} = 1.056$.

Table 6 – Two bonded anchors M12 in line with the shear: group rules against two single anchors [kN]

Quantity	Two single anchors	Group per EN 1992-4	Ratio
$N_{Rk,c}$	2 × 23.5 = 47.0	34.4	0.73
$N_{Rk,p}$	2 × 16.3 = 32.6	25.2	0.77
$V_{Rk,cp} = 2\min\{N_{Rk,c}; N_{Rk,p}\}$	65.1	50.3	0.77
$V_{Rk,s}$ (grade 8.8)	67.4	67.4	1.00
V_{Rk} with $k_p = 4$	67.4 (steel)	67.4 (steel)	1.00

The group rules lower the bond-governed pry-out resistance by 23 %, to 50.3 kN, and make it govern over the steel resistance of 67.4 kN. The reduction is applied to a mechanism whose induced tension is a fraction of the shear, and it is applied to concrete that the neighbouring anchor is compressing. With $k_p = 4$ the group bond term becomes 100.7 kN and the group is governed by steel.

Test data lean the same way. As Section 7.1 shows, the group rules are measurably more conservative than the single-anchor rules, with a mean test-to-prediction ratio of 1.63 against 1.27. That finding concerns the concrete term and does not by itself isolate the interaction described here, but it is consistent with it. For anchors placed side by side, perpendicular to the load, the bearing zones do not reach the neighbour's crater, and the overlap reduction retains its physical meaning.

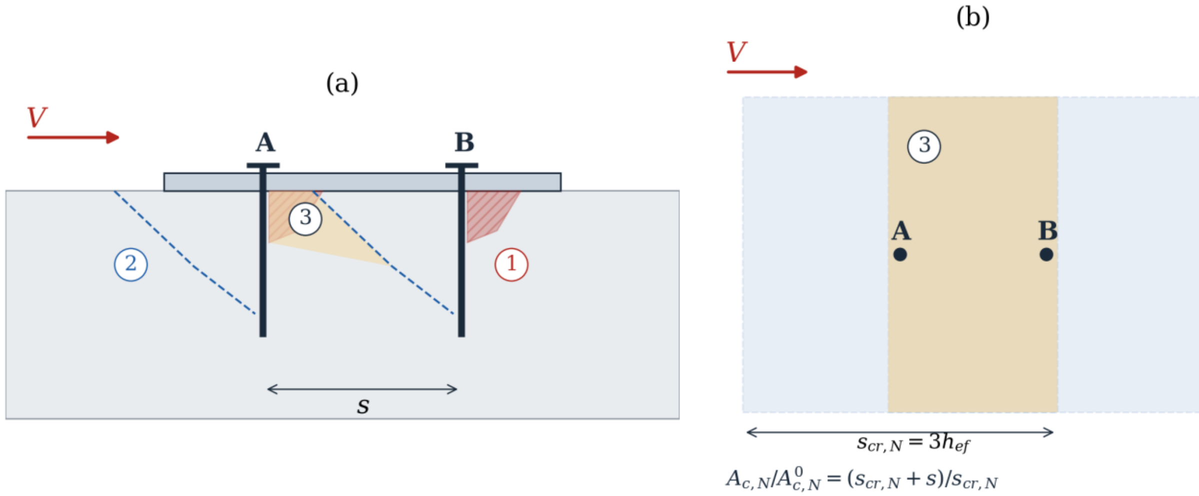

Figure 4 – Two anchors in line with the shear. (a) Section: forward bearing zones (1), rear pry-out craters (2), and the zone between the anchors (3) where the crater of B would have to form in concrete compressed by A. (b) Plan: the same zone is deducted from both anchors as an overlap of tension projected areas.

7.4 Plates: kinematics and documented failures

A base plate that is stiff compared with the rods does not let the heads rotate. The heads can only translate, so each rod bends in double curvature between the plate and the hinge below. Some axial elongation remains, but a second hinge forms under the plate before the free-head tilt of Section 3 is reached.

Whatever tension does develop pulls the plate down onto the concrete. The contact pressure, equal to ΣN plus the restraint couple, mobilises friction $\mu\,\Sigma N$ at the interface, and that friction takes part of the shear off the anchors. EN 1992-4 ignores it. The mechanism that is supposed to pull the anchors out therefore reduces the very shear that drives it.

The overturning couple $V\,e$ also looks different under a plate. It is resisted over the lever arm of the plate, typically 100 mm to 300 mm, not over the 20 mm to 40 mm of a washer. By Eq. (5) the back-row tension is then a small fraction of the shear, and in any case cannot exceed $M_{Rk,s}/z$ per rod.

In the literature reviewed for this paper, including the 214-test pry-out database of [6], the author has found no report of an anchor plate under shear failing by bond pull-out of its anchors. The failures reported are concrete pry-out, concrete edge failure and steel failure. The absence of a report is not proof that the mechanism cannot occur. It does mean that the bond branch of Eq. (7.39c), when it governs a group, governs by a mechanism for which the author has found no documented instance. Within the analysis of Section 6, steel shear or concrete pry-out precedes the bond term in every configuration once the induced tension is taken at its kinematic value, and under a plate friction and restraint reduce that tension further.

7.5 What factor for the concrete term?

The evidence supports three statements. First, a factor of 2 is conservative for the concrete term: the database mean corresponds to about 2.5 for single anchors. Second, the effective factor for groups is higher, about 3.3, and the neighbour-compression argument of Section 7.3 gives a mechanical reason why groups in line with the load should do better still. Third, a factor of 4 for the concrete term has not been demonstrated by the tests available to the author. For the bond term, a factor of 4 follows from the mechanics of Section 5; for the concrete term it remains a hypothesis. The paper therefore proposes a test-based recalibration of k_8, separately for single anchors and for groups, rather than a fixed value.

8. Inconsistencies in the treatment of fasteners and of the mortar

Beyond the value of the factor, the provisions apply it inconsistently. Two cases are examined: the same screw assessed in two categories, and the mortar of a bonded fastener credited in depth but not in width.

8.1 One screw, two treatments: the Hilti HUS4

The asymmetry noted in 2.1 is easiest to see with a real product, and it is here that the provision is hardest to defend. The Hilti HUS4 concrete screw is assessed as a mechanical fastener under EAD 330232 (ETA-20/0867 [22]). Exactly the same steel element, set into a hole pre-filled with a HUS4-MAX foil capsule, is assessed as a *bonded screw fastener* under EAD 332795 (ETA-18/1160 [23]) and designed to EN 1992-4 together with EOTA TR 075 [24]. That the screw is the same is visible in the two assessments themselves: at the same nominal embedment the characteristic steel shear resistance and bending resistance are identical ($V^0_{Rk,s}$ = 32.0 kN and $M^0_{Rk,s}$ = 64 N·m for size 10; 73.1 kN and 240 N·m for size 16), and both give $k_8 = 2.0$.

As a mechanical fastener the screw is checked for pry-out with Eq. (7.39a), $k_8 \cdot N_{Rk,c}$, whatever its tested pull-out resistance. As a bonded screw it fell, under TR 075:2020, within “the design provisions for bonded fasteners given in EN 1992-4”, and therefore under Eq. (7.39c), where the combined pull-out resistance enters the pry-out check. TR 075 builds that resistance from two parts, the concrete screw contribution $N^0_{Rk,p,CS}$ and the bond material contribution $N^0_{Rk,p,B}$ (Eqs. 1 and 12 of [24]). Adding mortar can only add to the screw’s own interlock, and the mortar fills the annulus, removes the clearance and adds a continuous axial constraint. None of this can make shear-induced withdrawal *easier*. Yet the classification alone decides whether the pull-out term is looked at.

Table 7 sets the two assessments side by side for a single fastener in cracked C20/25 without edge influence.

Table 7 – The same HUS4 screw assessed as a mechanical fastener and as a bonded screw (single fastener, cracked C20/25, values in kN)

Quantity	HUS4 10	HUS4-MAX 10	HUS4 16	HUS4-MAX 16
h_{nom} / h_{ef} for cone [mm]	85 / 68.0	85 / 85	130 / 104.9	130 / 130
$N_{Rk,p,cr}$ (screw + bond)	$\geq N_{Rk,c}$	19.3 + 4.5 = 23.8	32.0	32.0 + 23.0 = 55.0
$N_{Rk,c}$	19.3	27.0	37.0	51.0
$V_{Rk,s}^{0}$	32.0	32.0	73.1	73.1
$V_{Rk,cp}$ by Eq. (7.39a), $k_8 N_{Rk,c}$	38.6	54.0	74.0	102.1
$V_{Rk,cp}$ by Eq. (7.39c), $k_8 \min\{N_{Rk,c}; N_{Rk,p}\}$	—	47.6	—	102.1

For the mechanical HUS4 16 the tested pull-out resistance, 32.0 kN, is lower than the cone resistance, 37.0 kN, and governs its tension design; it is nonetheless left out of the pry-out check. For the bonded HUS4-MAX 10 the screw contribution was taken as $N_{Rk,c}^{0}$ with h_{ef} = 68.0 mm from the mechanical assessment. ETA-18/1160 defines it as "$\geq N_{Rk,c}^{0}$" with $h_{ef} = 0.85(h_{nom} - 0.5h_t)$, which is somewhat larger; but even with $h_t = 0$ that depth cannot exceed 72.3 mm, so $N_{Rk,p}$ cannot exceed 25.7 kN and stays below $N_{Rk,c}$ = 27.0 kN. The pull-out term therefore governs Eq. (7.39c) in any case, and it lowers the pry-out resistance of the bonded screw by 5 % to 12 % (to between 51.3 kN and 47.6 kN) compared with what the same bonded screw would receive under the mechanical rule, 54.0 kN. Put differently, under Eq. (7.39c) the bonded screw is credited with the pry-out resistance of the plain screw plus k_8 times the bond contribution, $2 \times (19.3 + 4.5)$ kN: the deeper effective embedment that its own assessment grants it for the concrete cone (h_{ef} = 85 mm instead of 68.0 mm) is never used. The bonded screw is still not worse than the plain screw (47.6 kN against 38.6 kN), and in this product the difference does not change the design, because steel governs every column. What changes is the logic: the same mechanism is counted or ignored according to the label on the assessment.

EOTA appears to have recognised this. The December 2024 amendment of TR 075 [24] added a clause on pry-out that did not exist in the 2020 edition: "The characteristic pry-out resistance $V_{Rk,cp}$ shall be calculated with the equation according to EN 1992-4, 7.2.2.4 (3) for mechanical post-installed fasteners." The wording is internally inconsistent. In EN 1992-4:2018, paragraph 7.2.2.4(3) is the bonded-fastener rule, Eq. (7.39c), while the rule for mechanical post-installed fasteners is paragraph (2), Eq. (7.39a). Read by its words, the amendment moves bonded screws to the mechanical rule and so removes the pull-out term from their pry-out check, which is exactly the change this paper argues for. Read by its paragraph number, nothing changed. Either way, the drafters of the design method for mortar-bedded screws saw a reason to address the question, and the reason they would have had applies with equal force to a threaded rod bedded in the same kind of mortar.

So the rule follows the product *category* rather than the mechanism it claims to represent. If shear-induced pull-out were a credible failure under the loads EN 1992-4 assigns to it, it would have to be verified for every fastener with a pull-out resistance, mechanical ones included. If it is not credible, and Sections 3 to 6 argue that at the implied ratio of 50 % it is not, it should not govern bonded fasteners either.

One may go further and ask why the check was attached to bonded fasteners in the first place, rather than to mechanical ones. A bond that runs continuously along the whole embedment restrains axial slip at least as well as the discrete interlock of an expansion sleeve or a screw thread, so if either category deserved a check against shear-induced withdrawal it would, if anything, be the mechanical one. Neither EN 1992-4 nor TR 029 states the reason. Two can be imagined. The first is historical: in TR 029 the bond resistance was defined as a *combined pull-out and concrete cone* failure, a mode in which a shallow cone forms together with bond failure below it, so it may have looked natural to treat it as a second concrete resistance next to $N_{Rk,c}$. EN 1992-4, however, justifies the term by shear-induced tension, not by the shape of the failure body, and it is that justification this paper examines. The second is reliability: bond strength is more sensitive than mechanical interlock to hole cleaning, moisture, temperature, sustained load and cracks running along the hole, which is why bonded fasteners carry ψ_{sus} and product-specific installation factors. Those sensitivities are real, but they are already built into $N_{Rk,p}$ where it governs in tension; they do not create a tension that the shear does not induce. Whatever the reason, the exposure to shear-induced withdrawal depends on the ratio of $N_{Rk,p}$ to the shear, not on the category. By that measure the plain HUS4 16 of Table 7, whose tested pull-out resistance is below its cone resistance, would be the more exposed fastener, and it is the one that is exempt.

8.2 Can the rod and a stronger mortar be treated as one unit?

With injection systems one naturally asks: if the mortar bonds to the threads better than the concrete bonds to the mortar, and is stronger in compression than the concrete, can the rod and the mortar annulus be treated as a single composite dowel of diameter d_0, and its rotation used to compute the induced tension? The answer is **partly yes**, and it matters which part.

Table 8 – Representative material properties

Material	Compressive strength [N/mm²]	Modulus [N/mm²]	Source
Concrete C25/30	25 (f_{ck})	31 000	[17]
Epoxy mortar A (7-day, ASTM D695)	82.7	2 600	[15]
Epoxy mortar B (7-day, ASTM D695)	89.5–100	3 300–4 100	[16]
Steel rod	f_{uk} = 800 (8.8)	210 000	—

Where the composite assumption holds. Kinematically, rod and mortar do act as one. There is no relative slip between them at working or ultimate load, so the axial constraint of Section 3.3, on which the catenary term rests, is real. The lateral pressure also reaches the concrete over d_0 rather than d; for M12 in a 14 mm hole the contact width grows by 17 %. And because rod and mortar work together, the induced tension is resisted along the whole embedment at the mortar–concrete interface, which is the same interface, and the same resistance $N_{Rk,p}$, that governs in tension. In tests failure can occur at either interface [10, 11]; τ_{Rk} is referred to d only by convention.

Where it does not. Stiffness is a different story. The mortar is about three times stronger than the concrete but roughly **ten times less stiff**, and the annulus adds only 1.1 % to the bending stiffness of an M12 rod. Laterally, it behaves as a soft layer between steel and concrete. Treating a 1 mm annulus as a confined layer ($M \approx 4.2$ GPa for $\nu = 0.35$) in series with the Vesić foundation lowers the foundation modulus from 23.0 to 16.5 kN/mm², by 28 %. At $V = 25$ kN the elastic head displacement rises from 0.16 mm to 0.20 mm (+28 %) and the elastic head rotation by 18 %. Strength at the pivot does not improve either: over d_0 the elastic peak pressure at 25 kN is still about 240 N/mm², far above what mortar or concrete can take. A stronger mortar cannot prevent crushing, because the concrete behind it still crushes at its own strength. Finally, the rotation capacity is set by the plastic hinge in the steel, not by the mortar.

Depth yes, diameter no? The assessments already treat the mortar as part of the fastener in one respect. For a bonded screw the concrete cone is calculated with $h_{ef} = h_{nom}$ (Section 8.1), because the mortar carries the load to the bottom of the hole, whereas the plain screw is credited only with $h_{ef} = 0.85(h_{nom} - 0.5h_t - h_s)$. If the mortar is accepted as part of the load path in depth, it is fair to ask why it is not accepted in width, where the outside of the mortar-filled hole, d_0, is the surface that actually bears on the concrete.

In most checks the answer makes no numerical difference. The cone resistance, Eq. (7.2), contains no diameter at all. The bond resistance $\psi_{sus}\,\tau_{Rk}\,\pi\,d\,h_{ef}$ does, but τ_{Rk} is back-calculated from tests with the rod diameter, so replacing d by d_0 would simply lower τ_{Rk}

in proportion and leave $N_{Rk,p}$ unchanged; for bonded screws TR 075 gives the bond contribution directly in kN and does not use τ_{Rk} at all. The diameter does enter, with physical meaning, in concrete edge failure. There $V^0_{Rk,c} = k_9\, d^{\alpha}_{nom}\, l^{\beta}_f \sqrt{f_{ck}}\, c_1^{1.5}$ with $\alpha = 0.1\left(l_f/c_1\right)^{0.5}$ and $\beta = 0.1(d_{nom}/c_1)^{0.2}$ (Eqs. 7.41–7.43), $l_f = h_{ef} \leq 12 d_{nom}$, and EN 1992-4 defines d_{nom} as the outside diameter of the fastener. For a bonded anchor the concrete receives the bearing pressure through the mortar annulus, so the outside diameter in contact with the concrete is d_0.

Table 9 – Concrete edge resistance $V^0_{Rk,c}$ [kN] of a bonded M12 anchor (d_0 = 14 mm) with $d_{nom} = d$ or $d_{nom} = d_0$ (cracked C25/30)

c_1 [mm]	h_{ef} [mm]	d_{nom} = 12 mm	d_{nom} = 14 mm	Increase
60	100	7.60	7.84	3.1 %
100	100	14.73	15.10	2.5 %
100	150	15.85	16.47	3.9 %
150	150	26.89	27.77	3.3 %

The effect is small, 2 % to 5 %, because the diameter appears with an exponent of about 0.1, and for M16 in an 18 mm hole it is about 2 %. There is also a physical reason for caution: bearing pressure follows stiffness, and a mortar ten times softer than concrete does not spread the pressure over d_0 as a rigid body would. The point is therefore one of consistency rather than of magnitude. The mortar is recognised as part of the fastener when that lengthens the cone and ignored when it would widen the bearing. It would matter more if a diameter-dependent pry-out model were adopted: in the model of Jebara et al. [6] the pry-out resistance grows with $d^{0.5}$, and M12 in a 14 mm hole would gain 8 % from using d_0.

Consequence for the calculation. The composite concept is the right model for the axial constraint and for the bond demand. The tilt, however, must be computed with the steel section's bending stiffness on a two-layer foundation, concrete and compliant mortar, and its ultimate value is limited by steel ductility. The compliant layer slightly increases the tilt and so slightly increases η, but within the range of Section 5. The composite view therefore does not undermine the argument. Because the induced tension is resisted by the whole bonded length, it confirms that the correct comparison is between ηV and the full $N_{Rk,p}$.

9. Discussion: limitations, proposed provision and test programme

The argument has limits, and they should be stated as plainly as the conclusions.

The most important limitation is that the tilt at failure is modelled, not measured. The range 8°–15° comes from plastic dowel kinematics and an order-of-magnitude ductility estimate; direct measurement of θ and N in bonded anchors at failure is the missing evidence. The model also does not apply to short, stiff anchors. For $h_{ef}/d \lesssim 5$ the anchor rotates as a rigid body about a pivot near mid-depth, and the concrete term $k_8 \cdot N_{Rk,c}$ remains the appropriate check.

Restraint works in both directions. Fully restrained fixtures, torqued nuts and filled annular gaps (6.2.2 of [1]) raise the axial force above Eq. (3), but they also generate friction that is not credited. Bond degradation needs separate attention: cracks passing through the hole, wet or poorly cleaned holes, elevated temperature and sustained load all reduce τ_{Rk}. The factor ψ_{sus} in $N_{Rk,p}$ was derived for sustained *tension*; inside a shear verification it should arguably apply only to the sustained part of ηV, not to the whole bond resistance.

Seismic and fatigue loading, fire, stand-off with lever arm (Eq. 7.37), clearance holes and near-edge fastenings were not examined. The parametric study of Section 6.2 covers single anchors with default factors only; eccentricity factors and product-specific k_8 values were not included, and the group interaction of Section 7.3 is argued qualitatively and illustrated by one example rather than modelled.

Proposed provision (hypothesis for validation, not for design use):

$$V_{Rk,cp} = \min\{k_8 \cdot N_{Rk,c};\; k_p \cdot N_{Rk,p}\}, \qquad k_p \geq 4 \qquad (11)$$

The bond term receives its own factor, justified by the induced-tension ratio instead of borrowed from concrete-cone calibration. An equivalent practical form is to omit the bond term wherever Eq. (10) with $\eta = 0.25$ shows that steel shear precedes it; for mechanical fasteners, which are already exempt, nothing changes. For the concrete term the default k_8 should be recalibrated on the available test databases, on the same reliability basis as the other concrete failure modes and separately for single anchors and for groups. The mean ratios of Section 7.1 show room for an increase; the characteristic values must allow for the considerable scatter (COV 27 % to 34 %), and the tests below should establish whether groups in line with the load justify a factor approaching 4.

Proposed tests. Single anchors and 2×2 groups of M12 and M16 bonded rods in cracked and uncracked concrete, with h_{ef}/d = 5, 8 and 12. Each configuration should be tested with a sound installation and with a deliberately reduced bond strength (a low-τ_{Rk} product or a defined cleaning deficiency), so that the bond term governs under the current rules. Each rod should carry axial strain gauges above the hinge zone. Horizontal slip, fixture uplift and head rotation should be recorded, and the failure surface documented. Groups of two anchors in line with the load and side by side should be compared at equal spacing, to isolate the interaction of Section 7.3, and mechanical fasteners at the same

h_{ef}/d should be included for comparison. For the concrete term, a separate series of stiff single anchors and groups ($h_{ef}/d \leq 5$), mechanical and bonded, loaded in line with and across the group, would supply the data for recalibrating k_8. The primary outputs are the measured $\eta = N/V$ at peak load and the observed failure mode. If the premise of this paper is correct, bond pull-out should not be observed wherever Eq. (10) with the measured η predicts steel or concrete failure first.

10. Conclusions

1. EN 1992-4 sets the pry-out resistance at k_8 = 1 to 2 times a tension resistance of the same fastener: the concrete cone resistance for mechanical fasteners, and the smaller of cone and bond resistance for bonded fasteners. For the bond term the code names the mechanism itself, pull-out under shear-induced tension, placed inside the pry-out check "for reason of simplicity". With $k_8 = 2$ this implies an induced tension of 50 % of the shear, and with $k_8 = 1$, 100 %.
2. Because concrete is roughly seven times less stiff than steel, local crushing at the mouth of the hole is unavoidable. The pivot moves to $1.5d$–$2d$ and the shank tilts. The resulting kinematic tension is $N = V\sin\theta \approx V\tan\theta$, which is 14–27 % of the shear for 8°–15°. The restraint-couple tension named in the code's Note is bounded by $M_{Rk,s}/z$ and is small for real base plates.
3. The supported factor between the bond resistance and the corresponding shear resistance is therefore $k_p = 1/\eta \approx$ 4 to 7, against the code's 1 to 2. A value of 4 matches an ultimate tilt of about 14.5°.
4. Shear-induced pull-out can precede steel shear failure only below the threshold bond strength of Eq. (10). The threshold is independent of diameter and falls with h_{ef}/d: 4.7 N/mm² at $4d$ and 2.3 N/mm² at $8d$ for grade 8.8 with $\eta = 0.25$. In 76 single-anchor configurations within the admitted embedment range the bond term governed 21 times under the code and never with $k_p = 4$. In every case the verdict passed to a documented mode, steel shear or concrete pry-out.
5. For the concrete term the largest recent test database gives mean test-to-prediction ratios of 1.27 for single anchors and 1.63 for groups, an effective factor of about 2.5 and 3.3, and product assessments already grant k_8 up to 2.9. A factor of 2 is conservative. A factor of 4 for the concrete term has not been shown by tests and remains a hypothesis.
6. For groups the resistance is reduced a second time by tension-derived area and group factors, although for anchors in line with the load the rear anchor compresses the concrete in which the front anchor's crater forms. In a two-anchor example this lowers the pry-out resistance by 23 %. The author has found no

documented case of an anchor plate failing in shear by bond pull-out, and friction mobilised by the induced tension further opposes the mechanism.

7. The bond term is attached to the product category, not to the mechanism. The same Hilti HUS4 screw is checked for pry-out without its pull-out resistance as a mechanical fastener and, under TR 075:2020, with it as a bonded screw, although the mortar can only add to the axial constraint. The 2024 amendment of TR 075 refers bonded screws to the mechanical pry-out rule, with a paragraph reference that contradicts its own wording; the same reasoning would remove the pull-out term for bonded anchors.

8. The mortar is credited as part of the fastener for embedment depth but not for diameter. The numerical effect on concrete edge resistance is small, 2–5 %, but the inconsistency is one of principle. The rod–mortar composite model is valid for axial constraint, bearing width and bond demand, and invalid for stiffness: the mortar is stronger than concrete but about ten times more compliant.

9. A separate factor $k_p \geq 4$ on $N_{Rk,p}$ and a test-based recalibration of k_8 for single anchors and groups are proposed as hypotheses, to be confirmed by the instrumented test programme of Section 9 before they are used in design.

Declarations

Competing interests. The author is an engineer at Adit Ltd, which supplies post-installed anchoring systems, including bonded anchors and concrete screws, and develops design software. A change to the provision discussed here could affect the design resistance of such products. The argument is presented with its full calculation basis so that it can be checked independently.

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Data and code availability. All results in Sections 3–8 follow from the closed-form expressions given in the text. The calculation scripts used for Tables 3–7 and 9 and Figures 2–3 are available from the author on request.

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