

Does the Front Row Carry It All?

Concrete edge failure of multi-row anchor groups in EN 1992-4 — a critical re-examination by hand calculation, nonlinear finite-element analysis (Code_Aster) and the AAS equal-share method

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Abstract

EN 1992-4 verifies concrete edge failure of a multi-row anchor group loaded in shear towards a free edge by assuming that only the row closest to the edge is effective and that it carries the whole shear load. The consequence is paradoxical: the verified resistance does not grow when rows are added behind the front row, it falls when a row is added in front of existing anchors, and it rises when the holes of the front row are deliberately slotted so that those anchors carry nothing. For a 3×3 group of M12 wedge anchors ($s = 150$ mm, $h_{ef} = 70$ mm, C25/30, $h = 250$ mm) at $c_1 = 75, 100$ and 150 mm, the code gives 31, 39 and 56 kN (uncracked), while the same anchors with the front row slotted give 73, 78 and 87 kN. The paper examines this rule with three tools: the EN 1992-4 calculation, the equal-share method implemented in the AAS anchor-design software (every anchor carries the same share and the front row is verified with its share only), and a three-dimensional nonlinear finite-element analysis in Code_Aster with a regularised damage model for concrete, elastic steel shanks and a rigid fixture. The analyses show that, without hole clearance, every row is active up to failure: at peak load the front row carries practically its own stand-alone resistance while the two rows behind it carry 2.1 to 4.4 times as much, and the group resists 5.4, 3.9 and 2.9 times the load of the front row acting alone — where EN 1992-4 assumes 1.0. Removing the front row altogether does not increase the computed resistance, so adding anchors near the edge does not weaken the group, and with a hole clearance that lets the front row fail first the group still reaches 92 to 94 % of its resistance, at a larger displacement. Normalised to the characteristic level, the finite-element group resistance is about 147, 146 and 161 kN, i.e. 4.7, 3.8 and 2.9 times the EN value. The equal-share (AAS) method is close to this behaviour and conservative for $c_1 = 75$ and 100 mm, but at $c_1 = 150$ mm it reaches the resistance of the global break-out body that starts behind the back row, which the equal-share rule does not see. A verification rule is therefore proposed that keeps the equal share and adds that global check: the group resistance is the lesser of (n/n_1) times the front-row resistance and the resistance of the back row carrying the whole load. For the cases studied it is conservative by a nearly constant factor of 1.4 to 1.6 with respect to the normalised finite-element results, against 2.9 to 4.7 for the present rule. Evaluated with the ACI 318-19 break-out equations, the same two checks — which correspond to cases illustrated in the ACI commentary — give 94, 124 and 147 kN, and a worked design example in cracked concrete shows the practical consequence: a fixture that fails the EN check by a factor of two passes the proposed one with a utilisation of 0.8.

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Keywords: post-installed anchors; anchor groups; concrete edge failure; shear; load distribution; hole clearance; EN 1992-4; nonlinear finite-element analysis; Code_Aster; damage mechanics; crack band; ACI 318

1 Introduction

A fixture anchored near a free edge of a concrete member and pushed towards that edge can fail by concrete edge break-out: a half-cone of concrete, bounded by a crack that starts at the anchors and runs to the side face, is pushed out of the member. For a single anchor, the design model of EN 1992-4 [1] — the Concrete Capacity (CC) method developed at the University of Stuttgart and carried into fib Bulletin 58 [2] — is well supported by tests: the resistance grows with $c_1^{1.5}$, where c_1 is the edge distance, and the geometric effects of spacing, further edges and member thickness are taken into account by the ratio of projected areas $A_{c,v}/A_{c,v}^0$.

The difficulty begins with groups of several rows. EN 1992-4 §6.2.2.2 (1) b) and §7.2.2.5 (2) state that, for a fastening close to the edge ($c < \max\{10 h_{ef}, 60 d_{nom}\}$) loaded perpendicular to it, **only the fasteners closest to the edge are assumed to be effective** for the verification of concrete edge failure. The note to Figure 6.5 adds that the component perpendicular to the edge is taken up by those fasteners alone. In other words, the full shear load of the fixture is placed on the front row, and the rows behind it are ignored.

The author has taught anchor design to students and design offices for several years and designs anchorages daily with the AAS software of Adit Ltd (eng.adit.org.il in English, www.adit.org.il in Hebrew). The question that repeatedly comes up is simple: if a fixture is held by anchors far from the edge and a designer adds one more row closer to the edge, can the fixture really become weaker? Under the EN rule it does, and substantially. This paper, which continues the author's critical re-examinations of EN 1992-4 [7, 17], tests whether that consequence is physical. The thesis is that the rows of a group anchored to one stiff fixture act together, so that the load should be shared between all anchors and the verification should be governed by the weakest anchor — the one closest to the edge — carrying its own share, not the whole load.

To keep the discussion concrete, a single, common configuration is analysed throughout: a 3×3 group of M12 wedge anchors at 150 mm spacing, at 75, 100 and 150 mm from the edge. Three methods are compared: (i) EN 1992-4 as written; (ii) the equal-share method of the AAS software; and (iii) a nonlinear three-dimensional finite-element (FE) analysis carried out with the open-source code Code_Aster [14], which is used here in place of the tests that were not available for this exact configuration. The FE model is checked against the EN single-anchor and single-row values before it is used to judge the group.

2 The provisions under examination

2.1 EN 1992-4

Clause 6.2.2.2 (1) of EN 1992-4 distinguishes two situations. All fasteners are considered effective when the fastening is far from an edge, for the verification of steel and pry-out failure, and under torsion or shear parallel to the edge (case a). For concrete edge failure of a fastening close to an

edge and loaded perpendicular to it, only the fasteners closest to the edge are effective (case b). Clause 6.2.2.2 (2) adds that a fastener in a hole slotted in the direction of the shear is not considered to resist shear. Table 6.1 limits the hole clearance for an M12 fastener to a 14 mm hole (2 mm on the diameter), and Note 1 allows fastenings whose gap is filled with mortar (≥ 40 N/mm²), or whose bolts are welded or screwed to the fixture, to be treated as having no hole clearance.

It is worth reading the clause for what it does **not** say. Case b) is not restricted to fastenings with hole clearance: as written, it also applies to a group whose holes are grouted, welded or otherwise free of clearance, in which the fixture cannot move without moving every anchor with it. The boundary of the rule, $\max\{10 h_{ef}; 60 d_{nom}\}$, is 720 mm for the anchor considered here, so all three rows of the group — the back row is 375 to 450 mm from the edge — are "close to the edge" in the sense of the code.

The characteristic resistance is then given by Eq. (7.40) of the standard:

$$V_{Rk,c} = V_{Rk,c}^0 \cdot (A_{c,v} / A_{c,v}^0) \cdot \psi_{s,v} \cdot \psi_{h,v} \cdot \psi_{ec,v} \cdot \psi_{\alpha,v} \cdot \psi_{re,v} \quad (1)$$

$$V_{Rk,c}^0 = k_9 \cdot d_{nom}^\alpha \cdot l_f^\beta \cdot \sqrt{f_{ck}} \cdot c_1^{1.5}, \quad \alpha = 0.1 (l_f/c_1)^{0.5}, \quad \beta = 0.1 (d_{nom}/c_1)^{0.2} \quad (2)$$

with $k_9 = 1.7$ for cracked and 2.4 for uncracked concrete, $A_{c,v}^0 = 4.5 c_1^2$, and $A_{c,v}$ the area of the idealised break-out body limited by overlapping neighbours ($s \leq 3 c_1$), by edges parallel to the load ($c_2 \leq 1.5 c_1$) and by the member thickness ($h < 1.5 c_1$). All the quantities refer to the **front row**, which receives the whole design shear V_{Ed} .

2.2 Other codes and the research record

The EN rule is not the only possible reading of the mechanics. fib Bulletin 58 and ACI 318 allow the failure crack of a multi-row group to be assumed to originate from the back row [8]. The ACI 318-19 commentary (Fig. R17.7.2.1b) illustrates, for two anchors one behind the other, three distributions: half of the shear on the front anchor, checked with the front anchor's break-out area; the whole shear on the rear anchor, checked with the rear anchor's break-out area — the only case required when the anchors are welded to the attachment; and the whole shear on the front anchor when the row spacing is smaller than the front edge distance, a case that does not apply to welded anchors [3, 4]. The first of these is exactly the equal-share assumption examined in this paper, and for the present group ($s = 150$ mm $\geq c_1$) it is the case that the ACI commentary would allow.

Test programmes at the University of Stuttgart have shown why a single rule is difficult. Grosser [6] and Eligehausen et al. [5] describe how, with hole clearance, the anchors that bear first on the fixture take the load, and the front row can fail before the back row is engaged; the back anchors then carry the load after further displacement. Bokor, Sharma and Hofmann tested rectangular and non-rectangular groups [8, 9] and found that the behaviour of the group depends strongly on the load–displacement behaviour and stiffness of the individual anchors and on the hole-clearance pattern; tests on multiple-row anchorages with supplementary reinforcement have been reported by Sharma et al. [11]. Bokor et al. then proposed a displacement-based nonlinear spring model in which the load reaching each anchor follows from its stiffness and from the concrete volume it activates [10]. The common thread of this work is that the distribution of shear among rows is a question of **displacement compatibility**, not of geometry alone — which is precisely what a fixed rule such as "front row only" cannot represent.

2.3 The AAS equal-share method

The AAS software [18] offers the designer two methods side by side: the EN method as written, and the AAS method, which modifies the load distribution for concrete edge failure. In the AAS method, the shear is shared equally by all anchors of the group, and the verification of the front row is carried out with the share of the front row only — n_1/n of the shear, where n_1 is the number of anchors in the front row and n the total number. The resistance side is unchanged: the front row is still verified with its own break-out body by Eq. (1). For a group with n_r identical rows, the verified group resistance is therefore n_r times the EN front-row resistance:

$$V_{Rk,c,group}^{AAS} = (n / n_1) \cdot V_{Rk,c}(\text{front row}) \quad (3)$$

Because each anchor is verified with the resistance of the most exposed row, the method embodies the "weakest-anchor" principle: every anchor is assumed to carry the same load, and that load is limited by what the anchors closest to the edge can resist. The AAS values given below were obtained by running the AAS calculation engine itself (build 2026-09-24a) in a headless browser; they coincide with the hand calculation to the last digit.

3 The case study and the EN paradox

The group consists of nine MTP-X M12 $\times$ 110 wedge anchors ($d_{nom} = 12$ mm, $h_{ef} = l_f = 70$ mm), the torque-controlled expansion anchor of Adit Ltd, arranged 3×3 at $s = 150$ mm and fixed to one stiff steel plate (Figure 1). The concrete is C25/30, uncracked unless stated, and the member is $h = 250$ mm thick and wide enough that no side edge interferes ($\psi_{s,v} = 1$). The shear acts perpendicular to the edge, through the centre of the group ($\psi_{ec,v} = \psi_{\alpha,v} = 1$). Three front edge distances are studied, $c_1 = 75, 100$ and 150 mm, so that the back row lies at $375, 400$ and 450 mm.

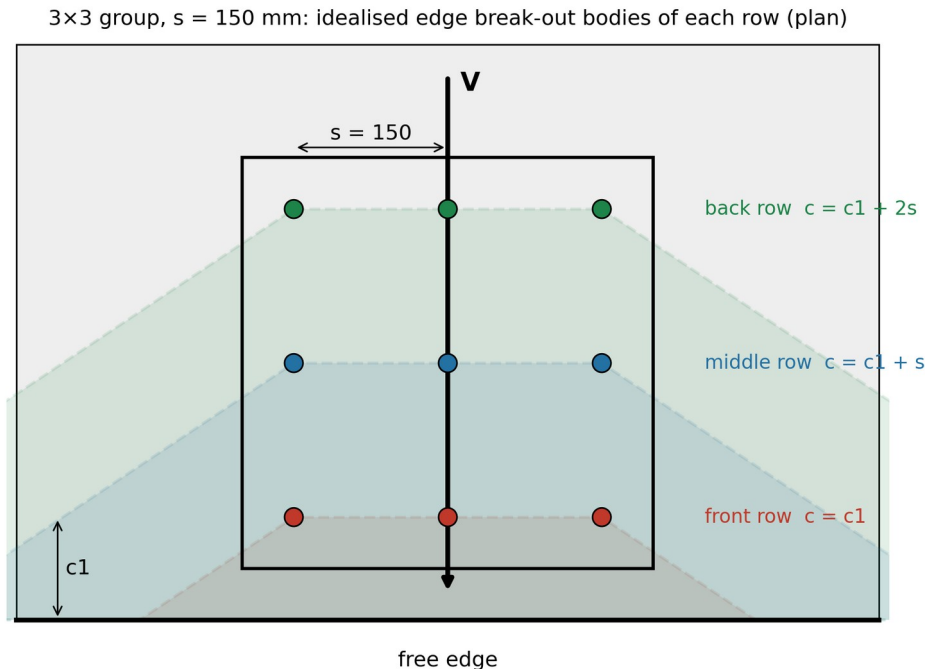


Figure 1. Plan of the 3×3 group ($c_1 = 100$ mm shown) with the idealised break-out bodies of EN 1992-4 for the front, middle and back rows.

Table 1 lists the EN 1992-4 resistances for four readings of the same fixture: the code as written (front row, whole load); the same plate with slotted holes in the front row, so that by §6.2.2.2 (2) the

middle row becomes the "closest" row; the plate with the two front rows slotted, so that only the back row is left; and the AAS equal-share value of Eq. (3). Detailed intermediate values are given in Appendix A.

Table 1. EN 1992-4 characteristic concrete edge resistance $V_{Rk,c}$ [kN] of the 3×3 group, uncracked concrete (cracked values: multiply by $1.7/2.4 = 0.708$).

c_1 [mm]	single anchor	EN: front row takes all	EN: front row slotted	EN: two front rows slotted	AAS: $3 \times$ front row
75	13.3	31.0	73.3	101.4	93.1
100	19.5	39.0	77.9	106.1	117.0
150	33.8	56.3	87.3	115.5	168.8

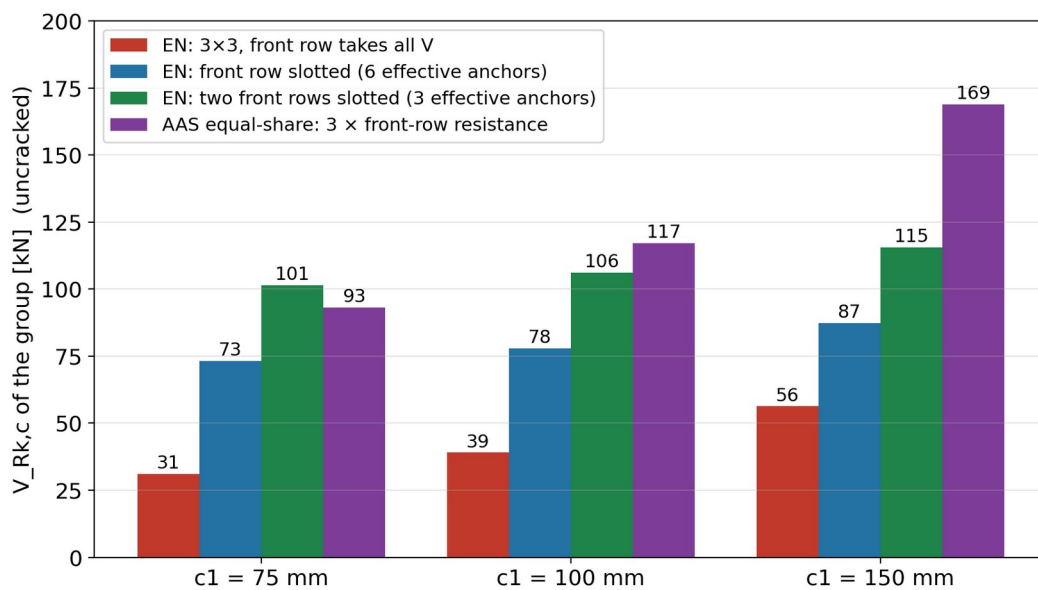


Figure 2. EN 1992-4 resistances of the same fixture under the four readings of Table 1 (uncracked). Slotting holes — i.e. switching anchors off — raises the verified resistance.

The table exposes the paradox in numbers. At $c_1 = 100$ mm the nine-anchor fixture is worth 39.0 kN. Slot the three front holes, so that those anchors can no longer transmit any shear towards the edge, and the fixture is worth 77.9 kN; slot the middle holes as well and it is worth 106.1 kN — with three working anchors instead of nine. Seen the other way round, a designer who holds a plate with the six anchors of the two back rows (77.9 kN) and adds a third row 100 mm from the edge loses 50 % of the verified resistance. At $c_1 = 75$ mm the loss is 58 %. The code thus rewards a fixture made worse on purpose, and penalises the addition of material.

Two remarks prevent a misreading. First, EN 1992-4 is internally consistent: once one accepts that the front row carries everything, the numbers follow. The question is whether the assumption itself is physical, and in particular whether it is physical for a fixture without hole clearance, to which the clause applies just the same. Second, the other failure modes do not rescue the rule: for this group the characteristic steel shear resistance is $9 \times 29.5 = 266$ kN and the pry-out resistance exceeds it, so the edge check governs by a wide margin in every case.

4 Nonlinear finite-element model (Code_Aster)

4.1 Geometry, mesh and boundary conditions

The analyses were run with Code_Aster version 18.1 [14], the finite-element code of Électricité de France, released under the GNU GPL; the meshes were generated with Gmsh [19] through its Python interface, and every run was scripted so that it can be repeated exactly (Data availability). Because the group and the loading are symmetric about the axis of the middle column, half of the slab is modelled, with a symmetry condition on the plane $y = 0$ (Figure 3). The model is a block of concrete 250 mm thick, 900 mm wide (half width, i.e. 1 800 mm in total) and $c_1 + 700$ mm deep perpendicular to the edge. The face opposite the free edge is restrained in the direction of the load, the soffit is supported vertically, and the free edge and the other faces are unrestrained.

Linear tetrahedra are used. In the zone where the break-out bodies can develop — from the free edge to 40 mm behind the back row, to $1.5 c_1 + 60$ mm beside the group and to $1.5 c_1 + 40$ mm (at most the full depth) below the surface — the target element size is 14 mm, refined to 3.5 mm on the surface of the anchor holes; elsewhere the mesh coarsens to about 50 mm. The models contain 14 000 to 30 000 tetrahedra for a single anchor and 40 000 to 85 000 for the 3×3 group.

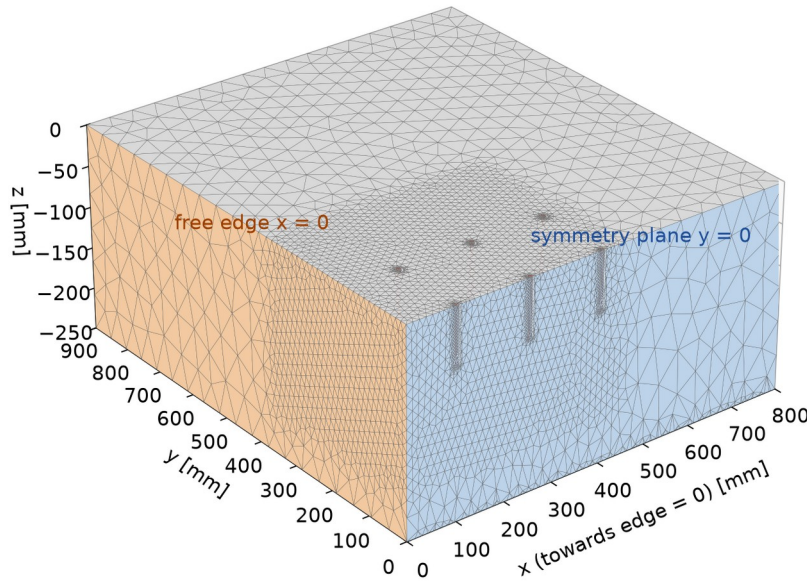


Figure 3. Finite-element model (half model, $c_1 = 100$ mm, 3×3 group): mesh of the concrete block with the refined zone and the nine anchor shanks; the free edge is on the left.

4.2 Concrete: isotropic damage with crack-band regularisation

Concrete is described by the ENDO_ISOT_BETON law of Code_Aster, an isotropic scalar-damage model of the Mazars family [15] in which damage is driven by the positive (tensile) strains, with a correction for confinement calibrated through the compressive strength, and follows a linear softening branch in uniaxial tension. The mean properties of C25/30 are taken from EN 1992-1-1 [16] and the fib Model Code 2010 [13]: $E = 31\,476$ N/mm², $\nu = 0.2$, $f_{cm} = 2.56$ N/mm², $f_{cm} = 33$ N/mm² and fracture energy $G_F = 73 f_{cm}^{0.18} = 0.137$ N/mm.

A local softening model depends on the element size unless the dissipated energy is tied to it. The crack-band approach of Bažant and Oh [12] is used: the softening modulus E_t of each element is chosen so that the energy dissipated in one element band equals G_F ,

$$G_F / h_e = (f_{ct}^2 / 2) \cdot (1/E + 1/|E_t|) \quad (4)$$

where h_e is the element size. The concrete elements were sorted after meshing into eight size classes (from 3.5 to about 55 mm, measured as the edge of the regular tetrahedron of equal volume) and each class received its own softening modulus. This makes the dissipated energy, and therefore the peak load, largely independent of the mesh (§4.5).

4.3 Anchors, fixture and hole clearance

Each anchor is represented by an elastic steel shank ($E = 210\,000\text{ N/mm}^2$, $\nu = 0.3$) of 12 mm diameter and 70 mm embedded length. The shank shares its nodes with the concrete over the half of the hole that faces the edge — the bearing side — while on the other half the hole is 2 mm larger, so that the shank can move away from the concrete behind it without pulling it. This reproduces the essential feature of a wedge anchor loaded in shear: it bears on the concrete in front of it and bends as a short pile, so that the bearing pressure concentrates in the upper part of the embedment.

The fixture is taken as rigid. It is modelled by prescribing the same horizontal displacement u towards the edge at the top section of every shank, which also clamps the shank heads against rotation as a thick plate does. The loading is displacement-controlled, which allows the load to redistribute among the rows as it would under a stiff plate, and allows the analysis to continue beyond the peak load. Without hole clearance, all nine anchors receive the same u . To represent hole clearance, the displacement of a row is delayed by its clearance: $u_{\text{row}} = \max(0; u - \Delta_{\text{row}})$.

The resultant shear of each row is obtained as the sum of the nodal reactions at the tops of its shanks. The steel is kept elastic on purpose: the study concerns the concrete mechanism, and steel failure (266 kN characteristic for the nine anchors) is a separate verification.

4.4 Solution procedure

The equilibrium equations are solved with the full Newton method (tangent matrix, direct MUMPS solver) in increments of 0.02 mm, with automatic subdivision of an increment in case of non-convergence and, if that fails, a second attempt with the elastic matrix. A concrete edge break-out is a brittle event: when the crack reaches the side face, the equilibrium path can turn back on itself (snap-back), and a displacement-controlled analysis then stops converging. Where this happened, the last converged load is reported as the peak and marked as a lower bound ($\geq$). In every such case the load–displacement curve was already flat when the analysis stopped.

4.5 Verification of the model

Mesh sensitivity was checked on the single anchor at $c_1 = 100\text{ mm}$ with target sizes of 7, 12 and 14 mm in the failure zone, using a preliminary version of the shank interface (0.5 mm gap behind the shank). The peak loads were 59.4, 60.3 and 57.9 kN respectively (the 7 mm run, which has five times as many elements, was stopped at 0.7 mm, after its peak), i.e. within $\pm 2.5\%$ of their mean: the crack-band regularisation works as intended, and the 14 mm size was retained for all analyses.

Table 2. Calibration: FE peak loads of a single anchor and of the front row acting alone, compared with the EN 1992-4 characteristic values (uncracked) [kN].

c_1 [mm]	EN single	FE single	ratio κ_S	EN row of 3	FE row of 3	ratio κ_F
75	13.3	40.0	3.01	31.0	71.0	2.29
100	19.5	55.9	2.87	39.0	102.8	2.64
150	33.8	93.8	2.78	56.3	153.8	2.73

Table 2 compares the FE resistance of a single anchor and of the front row acting alone (three anchors, the other rows absent) with Eq. (1). The model reproduces the relative effects reasonably well: the ratio of the row to the single anchor is 1.77, 1.84 and 1.64 in the FE analysis against 2.33, 2.00 and 1.67 in the code — close at 100 and 150 mm, and about 25 % low at 75 mm, where the three break-out bodies of the row overlap most. In absolute terms, however, the FE resistances are 2.3 to 3.0 times the characteristic values. Since the characteristic value is a 5 % fractile and the scatter of concrete break-out has a coefficient of variation of the order of 15 %, the mean resistance is roughly 1.3 times the characteristic one; the FE model therefore over-estimates the absolute resistance by a factor of about 1.7 to 2.3. Such an over-estimate is not unusual for continuum models with an idealised shank–concrete interface — a perfectly rigid plate, a bonded bearing half-shell and no local crushing of the hole wall — and it is the reason why the FE results are used below in **relative** form. Two normalisations are used: dividing the group resistance by the single-anchor ratio κ_S , or by the front-row ratio κ_F , of the same edge distance. Both give an "EN-equivalent characteristic" group resistance, and their spread is a measure of the uncertainty of the normalisation.

5 Results

5.1 Load sharing in the group without hole clearance

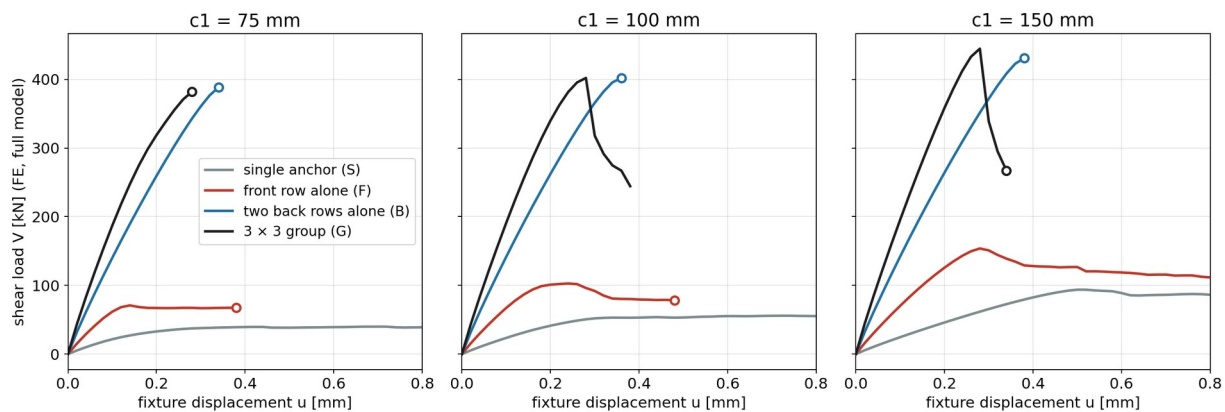


Figure 4. FE load–displacement curves: single anchor (S), front row alone (F), back two rows alone (B) and full 3×3 group (G), for $c_1 = 75, 100$ and 150 mm (full model, uncracked).

Markers at the end of a curve indicate that the analysis stopped at the peak.

Figure 4 shows the load–displacement curves and Figure 5 the resultant of each row of the 3×3 group. The picture is the same at the three edge distances. From the first increment all three rows are loaded, and up to a displacement of about 0.1 mm they carry almost equal shares. The front row then softens first — its break-out crack starts — while the rows behind it, which have more

concrete in front of them, keep their stiffness and attract the additional load. The front row reaches its peak first — at about the same displacement and practically the same load as the front row acting alone — and then stops gaining load, while the middle and back rows continue to take more until the group as a whole fails.

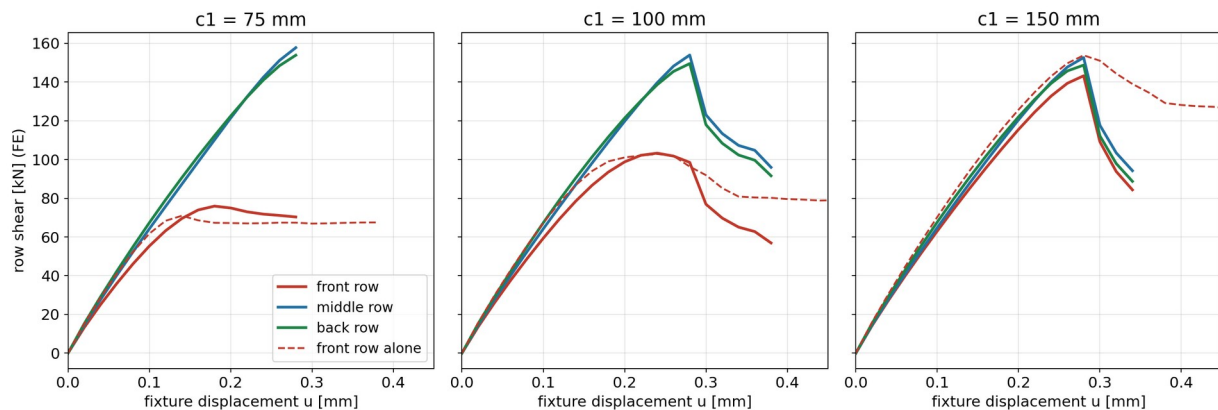


Figure 5. Resultant shear of each row of the 3×3 group as a function of the fixture displacement (FE, uncracked). The dashed line is the front row acting alone.

Table 3. FE peak load of the 3×3 group and its distribution among the rows at peak [kN] (full model, uncracked).

c_1 [mm]	group G	front row	middle row	back row	front row alone F	G / F
75	≥ 382	70 (18 %)	158 (41 %)	154 (40 %)	71	5.38
100	402	98 (25 %)	154 (38 %)	150 (37 %)	103	3.91
150	≥ 445	143 (32 %)	153 (34 %)	149 (33 %)	154	2.89

Table 3 quantifies this. At peak, the front row carries 70, 98 and 143 kN — 99 %, 96 % and 93 % of what it can carry on its own — and the two other rows carry the rest. The group resists 5.4, 3.9 and 2.9 times the load of the front row alone. EN 1992-4 assumes that this ratio is 1.0; the equal-share method assumes 3.0.

The distribution is not equal, and the difference is instructive. Once cracking starts, the front row carries less than one third of the load, because it softens first; the rows behind it carry more than one third. An equal-share verification of the front row is therefore on the safe side as far as the front row is concerned: in reality the front row is relieved by the stiffer rows behind it, not overloaded by them.

5.2 Failure mechanism

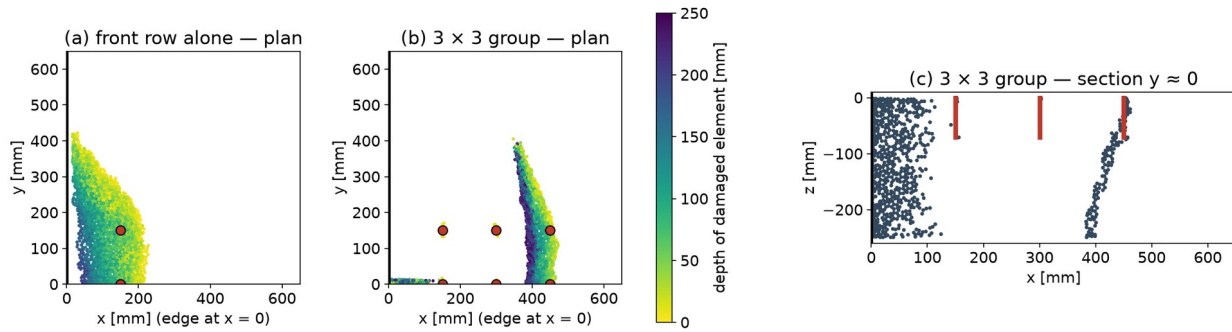


Figure 6. Damage ($D > 0.9$) at the end of the analyses, $c_1 = 150$ mm: (a) front row alone — edge break-out from the front row; (b) 3×3 group, plan, and (c) section on the axis of the middle column: the governing crack starts behind the back row and runs through the member thickness towards the edge.

Figure 6 compares the damage patterns. The front row acting alone fails by the classical edge break-out: a crack starts just behind the anchors and runs down and outwards to the side face. In the group, the front row develops the beginning of the same crack, but the failure that limits the group is different: the governing crack starts **behind the back row** and runs through the whole thickness of the member towards the edge, detaching a body of concrete that contains all nine anchors. A vertical splitting crack also forms on the axis of the group in front of the front row. The governing body is the break-out body that fib Bulletin 58 and ACI 318 associate with the back row [3, 8], and it explains why the group resistance changes only moderately with c_1 (382, 402 and 445 kN): it is governed by the distance of the back row, which varies from 375 to 450 mm, not by that of the front row.

5.3 Does a row close to the edge weaken the group?

Table 4. Effect of adding the front row to the two back rows: FE peak loads [kN] and the corresponding EN 1992-4 values.

c_1 [mm]	FE: 2 back rows (6 anchors)	FE: 3×3 (9 anchors)	FE ratio	EN: 6 anchors	EN: 9 anchors	EN ratio
75	≥ 388	≥ 382	0.98	73.3	31.0	0.42
100	≥ 402	402	1.00	77.9	39.0	0.50
150	≥ 431	≥ 445	1.03	87.3	56.3	0.64

Table 4 answers the question that motivated the study. Adding the front row to a plate already held by the two back rows changes the FE resistance by a factor of 0.98, 1.00 and 1.03 at $c_1 = 75$, 100 and 150 mm — within the resolution of the analyses, which in most of these cases stopped at the peak, no change at all. EN 1992-4 predicts a reduction to 0.42, 0.50 and 0.64 of the original value. The added row does fail locally before the others — at $c_1 = 75$ mm its own break-out starts at a fixture displacement of about 0.15 mm — but that local failure does not reduce what the rest of the group can carry, because the front-row break-out body lies in front of the front row, outside the body that finally fails.

5.4 Hole clearance

To examine hole clearance, the group at $c_1 = 100$ mm was analysed with the middle and back rows engaging only after the fixture had moved by $\Delta = 0.3$ mm and $\Delta = 0.4$ mm. These values place the engagement of the back rows after the peak of the front row (at 0.20 mm), i.e. in the least favourable sequence: the front row carries the load alone up to its peak of 96 kN — close to the 103 kN of the front row acting alone — breaks out, and only then do the other rows bear. Until they bear, their heads were held at zero displacement; the restraint forces this produced were below 4 kN per row. A clearance of the full 2 mm allowed by Table 6.1 could not be followed to the end: that analysis lost convergence in the softening branch of the front row at a displacement of 0.57 mm, before the back rows were reached, so the smaller clearances were analysed with 0.01 mm increments instead. Figure 7 shows the result. After its break-out the front row does not lose its load entirely: it softens to about 80 kN. The back rows then engage, and the group reaches 376 kN ($\Delta = 0.3$ mm) and 371 kN ($\Delta = 0.4$ mm) — 94 % and 92 % of its resistance without clearance — at a displacement of 0.57 and 0.67 mm. The front row, already cracked, still carries 76 and 72 kN at that moment. Hole clearance therefore changes the sequence of failure and the displacement at which the resistance is reached, but it hardly changes the resistance itself: a group whose front row fails first still ends with some 3.6 times the resistance of that row. This is the load transfer to the rear anchors after further displacement described in the Stuttgart tests [5, 6] — and it is precisely the stage that the EN rule leaves out.

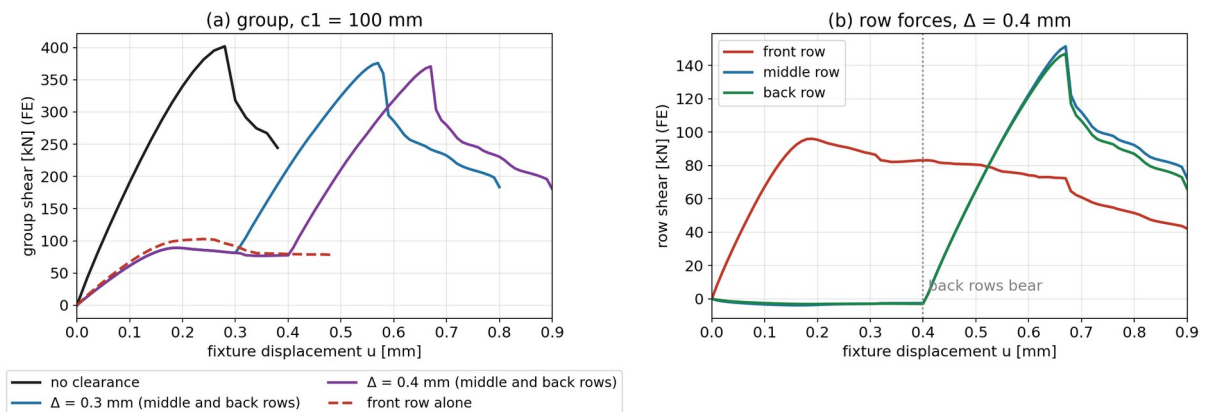


Figure 7. Hole clearance, $c_1 = 100$ mm: (a) group load–displacement curves without clearance and with the middle and back rows engaging after $\Delta = 0.3$ and 0.4 mm; (b) row forces for $\Delta = 0.4$ mm (FE, uncracked).

6 Which method describes the behaviour best?

Table 5 brings the methods together. The FE group resistance is given in the two normalised forms defined in §4.5, and the last columns give the ratio between the mean normalised FE value and each design method: a ratio above 1 means that the method is conservative, below 1 that it overestimates the resistance.

Table 5. Characteristic group resistance [kN] by method and ratio of the normalised FE value (mean of the two normalisations) to each method (uncracked).

c_1	FE norm.	EN	AAS	back row	Eq. (5)	FE/EN	FE/AAS	FE/back	FE/prop.
75	127 – 167	31.0	93.1	101.4	93.1	4.73	1.58	1.45	1.58

c_1	FE norm.	EN	AAS	back row	Eq. (5)	FE/EN	FE/AAS	FE/back	FE/prop.
100	140 – 152	39.0	117.0	106.1	106.1	3.75	1.25	1.38	1.38
150	160 – 163	56.3	168.8	115.5	115.5	2.87	0.96	1.40	1.40

EN: front row carries the whole load (§6.2.2.2 b). AAS: Eq. (3). Back row: EN Eq. (7.40) applied to the back row carrying the whole load (the fib/ACI reading). Proposed: Eq. (5), §8.

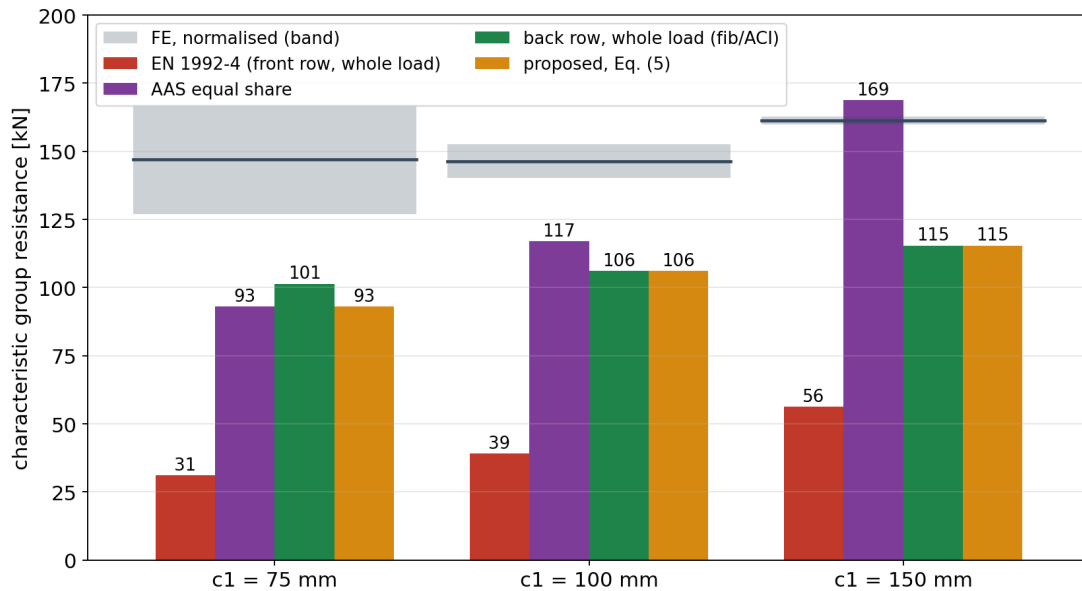


Figure 8. Characteristic group resistance by method compared with the band of the normalised FE resistance (uncracked).

The EN front-row rule is the least representative of the methods. It under-estimates the normalised FE resistance by a factor of 4.7, 3.8 and 2.9; the factor is largest where the front row is closest to the edge, i.e. exactly where the rule is most often decisive in practice. The error is not a matter of a safety margin that is simply generous: it has the wrong sign of dependence on the configuration, since it makes the resistance fall when anchors are added.

The AAS equal-share method follows the FE behaviour much more closely. It is conservative by 58 % and 25 % at $c_1 = 75$ and 100 mm and practically exact at 150 mm (ratio 0.96, i.e. 4 % above the normalised FE value). The last case shows the limit of the weakest-anchor principle: as the front row moves away from the edge its resistance grows with $c_1^{1.5}$, and n_r times that resistance eventually exceeds the resistance of the global body that fails behind the back row — a mechanism that a verification of the front row, however the load is shared, cannot detect.

Verifying the back row with the whole load — the fib/ACI reading — is conservative in all three cases, by a nearly constant factor of 1.45, 1.38 and 1.40. Its weakness is the opposite of that of the AAS method: it ignores the front rows entirely, and it would give the same resistance to a group whose front row was not there.

7 Discussion

7.1 Why the front-row rule is not rational for a fixture without clearance

A rigid plate imposes the same displacement on every anchor it holds. As long as the concrete in front of the front row has not broken out, the rows behind it cannot remain unloaded: to leave them unloaded the plate would have to move without them. The EN rule is therefore not an approximation of the mechanics for a fixture without clearance but a contradiction of it. The FE analyses confirm what compatibility requires: all rows are loaded from the start, and the stiffer rows behind carry more than the front row. When the front row reaches its capacity it keeps it, while the others continue — the group behaves as a ductile system of parallel elements even though each element is brittle, because the elements do not fail at the same displacement.

The rule has a plausible origin in fixtures with hole clearance, where the front anchors can bear first and fail before the others are engaged [5, 6]. But even there it describes only the first stage of the behaviour, and the standard applies it also to fastenings without clearance, where that first stage does not exist. A related inconsistency is visible in §6.2.2.2 itself: for pry-out and steel failure all fasteners are effective — the same anchors, in the same plate, under the same load, are assumed to share the load for one failure mode and not for the next.

7.2 The weakest-anchor principle and what it needs to be complete

The results support the central claim of this paper: all anchors of the group act together, and the verification can be based on the anchor closest to the edge carrying its share. They also show where the principle needs a complement. The equal share is conservative for the front row, because the stiffer back rows relieve it; but the group also has a global failure mode — the break-out of the whole body behind the back row — that the share of any single anchor cannot capture. For groups in which the front row is far enough from the edge that n_r times its resistance exceeds that of the global body, that body governs. In the present study this happens at $c_1 = 150$ mm.

7.3 Hole clearance, displacement and serviceability

Where the holes are not filled, the question is no longer only how much load the group can carry but at what displacement. The FE results show that the front row reaches its peak at a fixture displacement of about 0.2 mm, while a clearance of up to 2 mm may have to be taken up before the back rows bear. The front row then carries the load alone and may break out, and the fixture must move until the other rows are engaged; §5.4 shows that the group then still develops more than 90 % of its resistance, but at a larger displacement. Whether that is acceptable is a serviceability and robustness question — the cracked concrete in front of the fixture, the displacement of the attached element — rather than one of ultimate resistance. The simplest way to remove the question is the one the standard already offers in Note 1 to Table 6.1: fill the annular gap. For such a fixture the present analyses leave no reason to apply the front-row rule.

7.4 Other failure modes

The FE model keeps the steel elastic and does not model the pull-out of the expansion mechanism. At the peak of the group, the shank reactions in the FE analyses reach 50 kN per anchor in the rows behind the front row, well above the characteristic steel shear resistance of 29.5 kN — but this is a consequence of the absolute over-estimate discussed in §4.5; on the normalised scale they correspond to about 17 to 20 kN per anchor. In design, steel and pry-out failure must be verified as

usual, with all anchors effective as §6.2.2.2 already requires; the equal-share method is consistent with that, since it uses the same load per anchor for all failure modes.

7.5 The same checks with the ACI 318-19 equations

The ACI commentary quoted in §2.2 already contains, for two anchors one behind the other, the two checks that make up the verification proposed in §8: the front anchor with its share of the load, and the rear anchor with the whole load. To see whether the conclusions depend on the particular form of the EN equations, both checks were repeated with the concrete break-out equations of ACI 318-19 (SI version) [3]: $V_b = \min\{0.6 (l_e/d_a)^{0.2} \sqrt{d_a} \sqrt{f'_c} c_{a1}^{1.5}; 3.7 \sqrt{f'_c} c_{a1}^{1.5}\}$ with $l_e = h_{ef} = 70$ mm and $f'_c = 25$ N/mm², the same projected-area ratio A_{Vc}/A_{Vco} and thickness factor $\psi_{h,V}$ as in EN 1992-4, and $\psi_{c,V} = 1.4$ for uncracked concrete. Table 6 gives the result.

Table 6. Nominal concrete break-out resistance of the 3 × 3 group with the ACI 318-19 equations [kN] (uncracked, no supplementary reinforcement), compared with the normalised FE resistance.

c_1 [mm]	front row, whole load	front row, equal share (× 3)	back row, whole load	lesser of the two	FE norm.	FE / lesser
75	31.4	94.1	127.0	94.1	147	1.56
100	41.4	124.2	133.6	124.2	146	1.18
150	63.4	190.2	147.0	147.0	161	1.10

Three observations follow. First, the ACI and EN equations agree closely for the front row (31.4, 41.4 and 63.4 kN against 31.0, 39.0 and 56.3 kN), so the paradox of §3 is not a peculiarity of the EN formula but of the load assumption: an ACI designer who put the whole load on the front row would face the same loss of resistance when adding a row. Second, the ACI equations give a higher resistance for the back row (127 to 147 kN against 101 to 115 kN), because the ACI basic resistance keeps the factor $(l_e/d_a)^{0.2}$ constant while the EN exponents α and β decrease as c_1 grows. Third, the lesser of the two checks follows the FE results in the same way as with the EN equations — the equal share governs at 75 and 100 mm and the back row at 150 mm — but with a smaller margin, 1.56, 1.18 and 1.10. The ACI values are nominal strengths, used with a strength-reduction factor $\phi = 0.70$ to 0.75 , and are therefore not directly comparable with the EN characteristic values; what the comparison shows is that the structure of the proposal — equal share on the front row, whole load on the back row — is already present in one of the two major design codes, and that the governing check is the same under both sets of equations.

7.6 Limitations

- The FE model is not a substitute for tests. It reproduces the relative effects of spacing and edge distance well, but over-estimates the absolute resistance; the conclusions rest on the relative results and on the normalisation of §4.5.
- Only uncracked concrete, monotonic loading, one anchor type, one spacing and one member thickness were analysed. Cracked concrete lowers all break-out resistances roughly in proportion ($k_9 = 1.7$ instead of 2.4) and is not expected to change the distribution among rows, but this was not analysed.

- The fixture is rigid and clamps the anchor heads; a flexible plate would load the rows less uniformly.
- Where an analysis lost convergence at the peak, the reported value is a lower bound; in each case the curve was already flat.
- The anchor is modelled as a smooth elastic shank bearing on its front half; the expansion sleeve, the local crushing of the hole wall and friction under the plate are not modelled.

8 A proposed verification

The results suggest a verification that keeps the equal share of the AAS method and adds the global check that the equal share cannot see:

$$V_{Rk,c,group} = \min\{ (n / n_1) \cdot V_{Rk,c}(\text{front row}) ; V_{Rk,c}(\text{back row, whole load}) \} \quad (5)$$

where both terms are computed with Eq. (1) of EN 1992-4: the first with the break-out body of the front row (n_1 anchors at c_1), the second with the break-out body of the back row at its own edge distance $c_1 + (n_r - 1) s$, with the full member thickness and $\psi_{h,v}$ as in the code. The first term is the weakest-anchor verification of the front row with its share; the second is the fib/ACI verification of the global body. Neither term is ever smaller than the present EN value, so the proposal can never be less safe than the rule it replaces for the front row.

For the three cases studied, Eq. (5) gives 93.1, 106.1 and 115.5 kN (uncracked), which is conservative with respect to the normalised FE resistance by 1.58, 1.38 and 1.40 — a nearly constant margin, compared with 2.9 to 4.7 and a strong dependence on the configuration for the present rule. For fixtures with hole clearance that is not filled, the proposal applies at the ultimate limit state; the displacement needed to engage the back rows should then be considered at the serviceability limit state, or the gap filled.

8.1 Worked example

Consider the 3×3 group of this study at $c_1 = 100$ mm in **cracked** concrete C25/30, carrying a design shear $V_{Ed} = 40$ kN towards the edge through the centre of the group, with the annular gaps filled. With $\gamma_{Mc} = 1.5$ the design resistances are:

- EN 1992-4 as written (front row, whole load): $V_{Rd,c} = 27.6 / 1.5 = 18.4$ kN — utilisation 2.17, not verified.
- Proposed, first term (front row with its share, $n/n_1 = 3$): $3 \times 27.6 / 1.5 = 55.3$ kN.
- Proposed, second term (back row at $c = 400$ mm, whole load): $75.2 / 1.5 = 50.1$ kN.
- Proposed: $V_{Rd,c} = \min\{55.3; 50.1\} = 50.1$ kN — utilisation 0.80, verified.
- Steel failure without lever arm, all nine anchors effective as §6.2.2.2 (1) a) already requires: $V_{Rd,s} = 9 \times 29.5 / 1.25 = 212$ kN — utilisation 0.19. Pry-out is verified in the same way, with all anchors effective.

With the present rule the designer of this fixture must add anchors further from the edge, move the fixture, or add supplementary reinforcement, although the same rule would accept the fixture if the three front holes were slotted (36.8 kN, utilisation 1.09). Under the proposal the fixture is verified as it is, and the governing check — the back-row body — identifies the failure mechanism that the FE analyses actually show. Table 7 gives the cracked-concrete values of the two terms for the three edge distances.

Table 7. Characteristic concrete edge resistance of the 3×3 group in cracked concrete [kN]: present rule and the two terms of Eq. (5).

c_1 [mm]	EN: front row, whole load	term 1: $(n/n_1) \times$ front row	term 2: back row, whole load	Eq. (5)	Eq. (5) / EN
75	22.0	66.0	71.8	66.0	3.00
100	27.6	82.9	75.2	75.2	2.72
150	39.9	119.6	81.8	81.8	2.05

The proposal is offered as a basis for discussion and for testing, not as a finished rule. It should be checked against the test databases of the Stuttgart programmes [8–10] before it is used outside the range studied here.

9 Conclusions

1. EN 1992-4 verifies concrete edge failure of multi-row groups with the whole shear on the row closest to the edge. For a 3×3 group of M12 wedge anchors this makes the verified resistance fall by 36 to 58 % when a row is added in front of two existing rows, and rise when the holes of the front row are slotted — a result that has no physical basis for a fixture without hole clearance, to which the rule nevertheless applies.
2. Nonlinear FE analyses with Code_Aster show that all rows of a group on a rigid plate are loaded from the start. At peak the front row carries practically its own stand-alone resistance and the rows behind carry the rest; the group resists 5.4 to 2.9 times the load of the front row alone, where EN 1992-4 assumes 1.0.
3. Removing the front row does not increase the computed resistance: adding anchors close to the edge does not weaken the group. The front row fails locally first, but its break-out body lies outside the body that finally governs.
4. With hole clearance that lets the front row break out before the other rows bear, the load is transferred to the rear rows after further displacement and the group still reaches 92 to 94 % of its resistance without clearance ($c_1 = 100$ mm).
5. The failure that limits the group is the break-out of a body that starts behind the back row and contains all anchors. Its resistance depends on the position of the back row rather than of the front row.
6. The AAS equal-share method describes the behaviour far better than the EN rule: it is conservative by 58 % and 25 % at $c_1 = 75$ and 100 mm and within 4 % of the normalised FE resistance at 150 mm, where the global body starts to govern. The EN rule is conservative by a factor of 2.9 to 4.7 that depends strongly on the configuration.
7. A verification combining the equal-share check of the front row with a check of the back-row body carrying the whole load, Eq. (5), is conservative by a nearly constant factor of 1.4 to 1.6 in all cases studied and can never give less than the present rule. It is proposed for discussion and for validation against tests.
8. The two checks of Eq. (5) are those illustrated in the ACI 318-19 commentary for anchors one behind the other. Evaluated with the ACI equations they lead to the same governing

mechanism at each edge distance, which shows that the conclusions do not depend on the particular form of the EN break-out equation.

Data availability

The Gmsh and Code_Aster input generators, the command files, the load–displacement histories of every run and the EN/AAS calculation scripts are provided as a supplementary archive with this paper and are available from the author on request.

Changes from version 1

- New §7.5 and Table 6: the two checks of the proposed verification repeated with the ACI 318-19 break-out equations.
- New §8.1 and Table 7: a worked design example in cracked concrete and the cracked-concrete values of both terms of Eq. (5).
- Abstract, keywords and conclusions extended accordingly; ACI 318 added as a keyword.
- Links to both websites of Adit Ltd (English and Hebrew) added to the title block, the introduction and reference [18].
- Editorial corrections; the page footer is now set in 12 pt like the body text.

Conflicts of Interest

The author is Head of the Engineering Department of Adit Ltd, which supplies post-installed anchoring systems, including the wedge anchor analysed in this paper, and develops the AAS anchor design software whose equal-share method is examined here. A change to the provisions discussed could affect the design of fastenings with such products. All inputs, scripts and results are provided so that the analysis can be checked independently. The author declares no other conflicts of interest.

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Appendix A — EN 1992-4 calculation details (uncracked, C25/30, $h = 250$ mm)

Table 8. Intermediate values of Eq. (1)–(2) for each row verified with the whole load (A in 10^3 mm²).

c_1	row	c [mm]	α	β	$V_{Rk,c}^0$	$A_{c,v}$	$A_{c,v}^0$	$\psi_{h,v}$	$V_{Rk,c}$
75	front	75	0.0966	0.0693	13.3	59.1	25.3	1.000	31.0
75	middle	225	0.0558	0.0556	58.9	243.8	227.8	1.162	73.3
75	back	375	0.0432	0.0502	120.1	356.2	632.8	1.500	101.4
100	front	100	0.0837	0.0654	19.5	90.0	45.0	1.000	39.0
100	middle	250	0.0529	0.0545	68.2	262.5	281.2	1.225	77.9
100	back	400	0.0418	0.0496	131.5	375.0	720.0	1.549	106.1
150	front	150	0.0683	0.0603	33.8	168.8	101.2	1.000	56.3
150	middle	300	0.0483	0.0525	87.9	300.0	405.0	1.342	87.3

c_1	row	c [mm]	α	β	$V_{Rk,c}^0$	$A_{c,V}$	$A_{c,V}^0$	$\psi_{h,V}$	$V_{Rk,c}$
150	back	450	0.0394	0.0484	155.2	412.5	911.2	1.643	115.5

$d_{nom} = 12 \text{ mm}$, $l_f = h_{ef} = 70 \text{ mm}$, $f_{ck} = 25 \text{ N/mm}^2$, $k_9 = 2.4$; $\psi_{s,V} = \psi_{ec,V} = \psi_{\alpha,V} = \psi_{re,V} = 1$. $A_{c,V} = (2 \cdot 1.5 c + 2 s) \cdot \min(h; 1.5 c)$, with $s \leq 3 c$. The values coincide with those printed by the AAS engine in EN mode.

Remarks. An AI assistant was used for drafting support, for scripting the calculations and the finite-element pre- and post-processing, and for checking the numerical results. The research question, the analysis, the arguments and the conclusions are the author's, and all results were checked by the author.