

A closed-Form Upper Bound on the Concrete Breakout Resistance of Anchor Groups in Rectangular Pedestals Under the EN 1992-4 Narrow-Member Rule

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Highlights

- Breakout resistance in a pedestal has a ceiling no bolt choice can exceed
- Ceiling for any symmetric group: $(\sqrt{6}/9)k_1\sqrt{f_{ck}} a\sqrt{b} \psi(r)/\gamma_{Mc}$, $a \leq b$
- Number of anchors does not raise the ceiling; the optimal gauge is $b/2$
- For equal plan area, a square pedestal gives the highest ceiling
- Ductile bolt failure needs pedestals far larger than those used in practice

Abstract

Column bases, tower stanchions and equipment supports often anchor tension through cast-in headed bolts into concrete pedestals of limited plan size. In such members the EN 1992-4 narrow-member rule fixes the effective embedment by the edge distances and bolt spacing, so the calculated concrete breakout resistance stops increasing with embedment. This paper derives the maximum of that resistance over all symmetric bolt layouts in a rectangular pedestal of sides $a \leq b$. The optimum places the outer anchors at a spacing of $b/2$ along the long side, and the resulting ceiling is $(\sqrt{6}/9)k_1\sqrt{f_{ck}} a\sqrt{b} \psi(r)/\gamma_{Mc}$, where $r = a/b$ and $\psi(r) = 1$ for $r \geq 0.5$ and $0.7 + 0.6r$ otherwise. For a single anchor the ceiling is $k_1\sqrt{f_{ck}} b^{1.5} r(0.7 + 0.3r)/(3^{1.5} \gamma_{Mc})$. The bound is independent of bolt diameter, grade and number of anchors, and for a given plan area it is largest for a square pedestal, falling as $r^{0.25}$. Numerical optimisation over the gauges of 1 by 1 to 4 by 4 groups reproduced the closed forms to within 0.03%, and symbolic algebra confirmed the derivation. The ACI 318-19 counterpart has the same form and is 1.18 times higher up to a pedestal side of about 1.7 m. Because the ceiling grows only with the 1.5 power of pedestal size, a four-bolt M24 grade 4.6 group requires a 1.6 m square C30/37 pedestal before its steel strength can be reached; in practical pedestals concrete

breakout governs and ductility requires anchor reinforcement. A design chart for the pedestal size needed to carry a given tension is provided.

Keywords: concrete breakout; anchor group; pedestal; narrow member; EN 1992-4; ACI 318-19; ductility

1. Introduction

Tension from a column, a tower stanchion or a piece of equipment is frequently anchored by a group of cast-in headed bolts in a concrete pedestal whose plan is only slightly larger than the base plate. Pedestals of 500 to 1000 mm are common, and their size is usually fixed by the base plate and by cover to the pedestal bars before the anchorage is checked. The concrete breakout resistance of such an anchorage is governed by the concrete capacity design (CCD) method [1,2] as implemented in EN 1992-4 [3] and ACI 318-19 [4].

In a pedestal, every anchor is close to all four faces. Both codes then replace the effective embedment h_{ef} by a reduced value h'_{ef} fixed by the largest edge distance and the largest spacing. This narrow-member rule prevents the resistance from being overestimated where the idealised failure body is cut by the member faces, but it also means that the calculated resistance reaches a plateau at a modest embedment and stays there however deep the bolts are placed. Designers therefore meet an unexpected limit: lengthening the bolts, increasing their diameter or their grade does not increase the breakout resistance at all.

The existence of this plateau is implicit in the code expressions, but its maximum over all possible bolt layouts has not, as far as the literature reviewed shows, been stated in closed form. Such a statement would tell a designer, from the pedestal size and concrete grade alone, the largest tension the pedestal can carry without anchor reinforcement. This paper derives that maximum for square and rectangular pedestals and for groups of any number of anchors, verifies it numerically and symbolically, compares it with ACI 318-19, and uses it to map the pedestal sizes at which steel failure, and therefore a ductile anchorage, becomes possible.

2. Background

2.1 The CCD method and its range

The CCD method gives the characteristic breakout resistance of a single anchor as $N_{Rk,c}^0 = k_1 \sqrt{f_{ck}} h_{ef}^{1.5}$ and scales it by the ratio of the projected area of the failure body to that of an unconstrained anchor, with factors for edge disturbance and shell spalling [1,3]. The 1.5 exponent follows from linear elastic fracture mechanics; ACI 318-19 replaces it by 5/3 for cast-in anchors with $280 \text{ mm} \leq h_{ef} \leq 635 \text{ mm}$ [4]. Karmokar et al. [5] reviewed the test database and noted that deep anchors lie outside the range on which the method was calibrated. Delhomme et al. [6] tested headed and bonded anchors at different embedments, Di Nunzio and Muciaccia [7] proposed a combined breakout and pull-out model, and Nilforoush and co-workers found that member thickness, head size and surface reinforcement raise the resistance above the CCD prediction [8–10].

2.2 Groups, edges and narrow members

For anchors with three or more edges closer than the characteristic edge distance, EN 1992-4 sets $h'_{ef} = \max(c_{max}/c_{cr,N}, s_{max}/s_{cr,N}) h_{ef}$, and ACI 318-19 sets $h_{ef} = \max(c_{a,max}/1.5, s/3)$ [3,4]. Stehle and Sharma [11] showed that this approach is conservative for fastenings in narrow members. Bokor et al. [12] tested rectangular and non-rectangular groups and proposed a spring model for the distribution of anchor forces [13]; Węglorz [14] analysed how the modification factors for groups change in narrow members; Mellios et al. [15] extended the projected-area concept to non-rectangular edges, and Lima et al. [16] found that flexural reinforcement kept the resistance of headed-bar pairs near edges close to code estimates. Qian et al. [17] reported that deep anchor groups in members of limited width can split before a cone forms. None of these studies states the largest resistance a pedestal of given size can provide.

2.3 Failure-mode hierarchy and ductility

Which mode governs depends on the anchor, its embedment and the concrete. Chen [18] reviewed non-breakout modes of mechanical anchors and noted that they dominate at deeper embedments with more ductile behaviour, and Y. Zhang et al. [19] observed a progression from breakout through pull-through to steel failure as embedment increased. Wu et al. [20] found the same transition for hooked bolts in plain concrete, and plate anchors were examined by Wu et al. [21]. Petersen et al. [22] showed that code-conforming anchor reinforcement prevented breakout but did not always ensure steel failure of the anchor, which matters when a ductile anchorage is sought. In a pedestal, the narrow-member rule stops the breakout resistance from rising with depth, so the progression towards steel failure observed in unconstrained members may never be reached.

3. Model

The EN 1992-4 design breakout resistance of a group in tension is

$$N_{Rd,c} = \frac{1}{\gamma_{Mc}} k_1 \sqrt{f_{ck}} h_{ef}'^{1.5} \frac{A_{c,N}}{A_{c,N}^0} \psi_{s,N} \psi_{re,N} \psi_{ec,N} \quad (1)$$

with $k_1 = 8.9$ for cast-in anchors in cracked concrete and 12.7 in uncracked concrete, $A_{c,N}^0 = s_{cr,N}^2 = 9h_{ef}'^2$, $A_{c,N}$ the projected area of the actual group truncated at the member faces, $\psi_{s,N} = 0.7 + 0.3c_{min}/c_{cr,N} \leq 1$, and $\gamma_{Mc} = 1.5$ [3]. The following conditions are adopted. The pedestal is rectangular with sides $a \leq b$ and its depth is sufficient for member thickness not to govern. The group is concentrically loaded ($\psi_{ec,N} = 1$) and doubly symmetric, with outer anchors spanning g_x along a and g_y along b . The embedment is deep enough ($h_{ef} \geq 100$ mm) that $\psi_{re,N} = 1$ and that the narrow-member rule governs, so that

$$h_{ef}' = \max\left(\frac{c_{max}}{1.5}, \frac{s_{max}}{3}\right) \quad (2)$$

104 where $c_{\max}$ is the larger of the outer edge distances $c_x = (a - g_x)/2$ and $c_y = (b - g_y)/2$, and $s_{\max}$
 105 is the larger of g_x and g_y , that is, the spacing of the outermost anchors. Section 6.4 examines
 106 the alternative reading of $s_{\max}$ as the spacing between adjacent anchors.

107

108 **4. Derivation**

109 **4.1 Square pedestal**

110 For a square pedestal of side a and a group with $g_x = g_y = g$, Eq. (2) gives $h'_{ef} = \max[(a - g)/3,$
 111 $g/3]$ and $c'_{cr,N} = 1.5h'_{ef}$. For $g \leq a/2$, $c'_{cr,N} = (a - g)/2 = c$, so the projected area extends exactly
 112 to the faces, $A_{c,N} = a^2$ and $\psi_{s,N} = 1$. Substituting in Eq. (1),

$$113 \quad N_{Rd,c} = \frac{k_1 \sqrt{f_{ck}} a^2}{9 \gamma_{Mc} \sqrt{h'_{ef}}} \quad (3)$$

114 Equation (3) holds whenever the projected area covers the pedestal, and it decreases with h'_{ef} .
 115 The resistance is therefore maximised by minimising h'_{ef} , which requires $(a - g)/3 = g/3$, that
 116 is $g = a/2$. For $g > a/2$, $h'_{ef} = g/3$ rises and $\psi_{s,N} = 0.7 + 0.3(a - g)/g$ falls, so the resistance
 117 decreases. The maximum is

$$118 \quad N_{Rd,c,max} = \frac{\sqrt{6}}{9} \frac{k_1 \sqrt{f_{ck}} a^{1.5}}{\gamma_{Mc}} = 0.272 \frac{k_1 \sqrt{f_{ck}} a^{1.5}}{\gamma_{Mc}} \quad (4)$$

119 attained with the outer anchors at the quarter points. For a single central anchor, $h'_{ef} = a/3$ and
 120 the coefficient is $3^{-1.5} = 0.192$. Two properties follow directly. The ceiling contains no bolt
 121 property, and, because Eqs. (2) and (3) depend only on the outer spread of the group, any
 122 symmetric group whose outer anchors lie at the quarter points, whether 2 by 2, 2 by 3, 3 by 3
 123 or larger, attains the same maximum.

124 **4.2 Rectangular pedestal**

125 For $a \leq b$ and $r = a/b$, Eq. (3) generalises to $N_{Rd,c} = k_1 \sqrt{f_{ck}} ab \psi_{s,N} / (9 \gamma_{Mc} \sqrt{h'_{ef}})$ while the
 126 projected area covers the pedestal. The smallest admissible h'_{ef} is set by the long direction:
 127 choosing $g_y = b/2$ gives $c_y = b/4$ and $s_{\max} = b/2$, so $h'_{ef} = b/6$, provided that $c_x \leq b/4$ and $g_x \leq$
 128 $b/2$. The remaining freedom in g_x is used to maximise $\psi_{s,N} = 0.7 + 0.3c_x/(b/4)$. If $r \geq 0.5$, the

anchors along a can be placed at $g_x = a - b/2$, making $c_x = b/4$ and $\psi_{s,N} = 1$; if $r < 0.5$, the best choice is $g_x = 0$ with $c_x = a/2$ and $\psi_{s,N} = 0.7 + 0.6r$. Hence

$$N_{Rd,c,max} = \frac{\sqrt{6}}{9} \frac{k_1 \sqrt{f_{ck}} a \sqrt{b}}{\gamma_{Mc}} \psi(r), \quad \psi(r) = \begin{cases} 1, & r \geq 0.5 \\ 0.7 + 0.6r, & r < 0.5 \end{cases} \quad (5)$$

which reduces to Eq. (4) for $a = b$. For a single central anchor, $h'_{ef} = b/3$, $A_{c,N} = ab$, $A^0_{c,N} = b^2$ and $\psi_{s,N} = 0.7 + 0.3r$, giving

$$N_{Rd,c,max}^{(1)} = \frac{k_1 \sqrt{f_{ck}} b^{1.5}}{3^{1.5} \gamma_{Mc}} r(0.7 + 0.3r) \quad (6)$$

Equation (5) shows that the ceiling scales with $a\sqrt{b}$, not with the plan area ab . For a fixed area $A = ab$, $a\sqrt{b} = A^{0.75} r^{0.25}$, so the square pedestal gives the highest ceiling for a given volume of concrete, and elongating the plan loses resistance slowly: a 600 by 900 mm pedestal has a ceiling of 159.2 kN in cracked C30/37, 9.6% less than the 176.2 kN of a square of the same area.

4.3 Required pedestal size

Inverting Eq. (4) gives the side of a square pedestal able to carry a design tension N_{Ed} by breakout alone,

$$a_{req} = \left(\frac{\gamma_{Mc} N_{Ed}}{\varphi_g k_1 \sqrt{f_{ck}}} \right)^{2/3}, \quad \varphi_g = \frac{\sqrt{6}}{9} \quad (7)$$

and, for a rectangular pedestal with $r \geq 0.5$, the long side follows from $b = (a_{req}^{1.5}/r)^{2/3}$.

4.4 ACI 318-19 counterpart

The three-edge rule of ACI 318-19 has the same structure, with $h_{ef} = \max(c_{a,max}/1.5, s/3)$, a projected area based on $1.5h_{ef}$, an edge factor $\psi_{ed,N} = 0.7 + 0.3c_{a,min}/(1.5h_{ef})$ and a strength reduction factor $\phi = 0.70$ for Condition B [4]. The derivation of Section 4.1 therefore applies unchanged, and while the limited embedment is below 280 mm, the ACI design ceiling differs from Eq. (4) only by the ratio of the basic coefficients:

$$\frac{\phi N_{cbg,max}}{N_{Rd,c,max}} = \frac{0.70 \times 10}{8.9/1.5} = 1.18 \quad (8)$$

For the optimal group, $h'_{ef} = a/6$, so this ratio holds up to $a \approx 1.7$ m; beyond it the ACI exponent of 5/3 applies, and the ratio rises.

5. Verification

5.1 Numerical optimisation

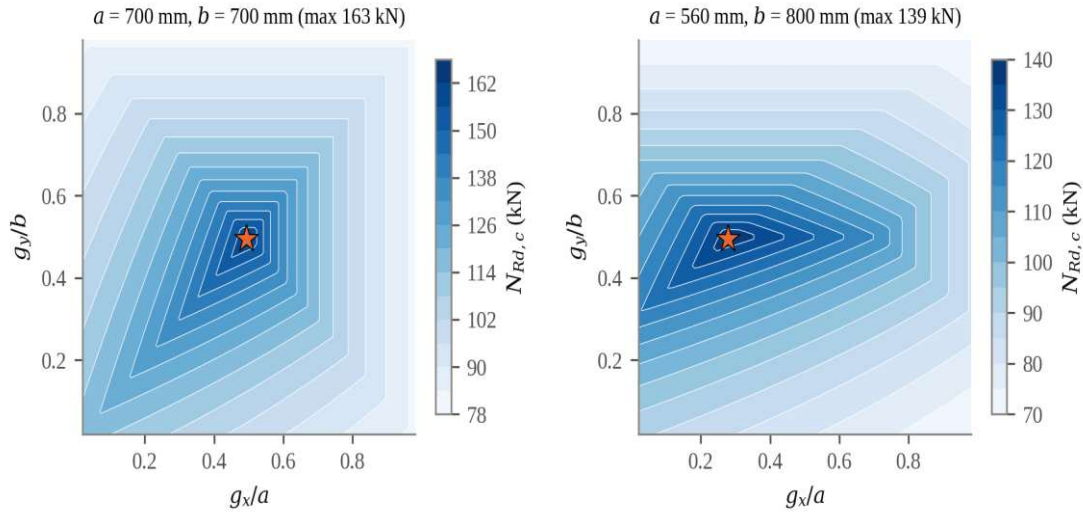
Equation (1) with the narrow-member rule was implemented for arbitrary groups in rectangular members and maximised over g_x and g_y by a 70 by 70 grid search followed by Nelder-Mead refinement, with $h_{ef} = 5000$ mm so that the rule governed throughout. The implementation had previously been checked by hand against both codes for a single anchor in a 700 mm pedestal of C30/37 concrete: EN 1992-4 gives $h'_{ef} = 233.3$ mm and $N_{Rd,c} = 173.7/1.5 = 115.8$ kN, and ACI 318-19 gives $\phi N_{cb} = 0.70 \times 195.2 = 136.7$ kN.

Table 1 lists the optimum for groups of one to sixteen anchors in a 700 mm square pedestal. All groups with two or more rows in each direction reached 163.8 kN at an outer spread of 350 mm, the value of Eq. (4); a single row of two anchors gained nothing over a single anchor. Across rectangular pedestals with $b = 600, 900$ and 1200 mm and r from 0.35 to 1.0, Eq. (5) matched the optimised group resistance to within 0.03%, and Eq. (6) matched the single-anchor resistance exactly. Fig. 1 shows the resistance surface over the two gauges for a square and a rectangular pedestal, and Fig. 2 compares the closed forms with the numerical optima.

Table 1. Optimal breakout resistance of symmetric groups in a 700 mm square pedestal (C30/37, cracked).

Layout	Anchors	Optimal outer spread (mm)	$N_{Rd,c,max}$ (kN)	ϕ_g
1 by 1	1	—	115.8	0.192
2 by 1	2	≈ 0	115.8	0.192
2 by 2	4	350×350	163.8	0.272
2 by 3	6	350×350	163.8	0.272
3 by 3	9	350×350	163.8	0.272
4 by 4	16	350×350	163.8	0.272

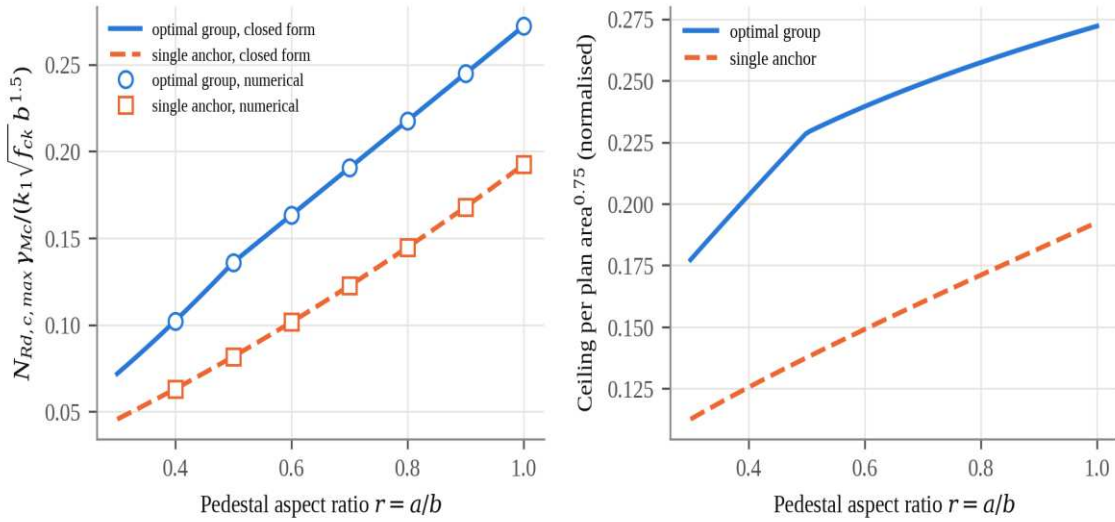
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174

175 Fig. 1. Design breakout resistance of a deeply embedded 2 by 2 group as a function of the outer
 176 gauges g_x/a and g_y/b (C30/37, cracked): (left) 700 by 700 mm pedestal; (right) 560 by 800
 177 mm pedestal. The star marks the optimum, $g_y = b/2$ and $g_x = a - b/2$.

178



179

180 Fig. 2. (left) Normalised ceiling against pedestal aspect ratio $r = a/b$, closed forms (lines, Eqs. 5 and 6)
 181 and numerical optima (markers); (right) ceiling per unit plan area to the power 0.75, showing that the
 182 square is the most efficient plan.

183 5.2 Symbolic check

184 The derivation was verified with a computer algebra system. The two branches of the square-
 185 pedestal resistance are equal at $g = a/2$; the derivative of the first branch, $\sqrt{3}k_1\sqrt{f_{ck}} a^2/[18(a -$
 186 $g)^{1.5}]$, is positive for $g < a/2$; the derivative of the second branch at $g = a/2$, $-11\sqrt{6}k_1\sqrt{f_{ck}}\sqrt{a}/45$,
 187 is negative; and the coefficient at the optimum simplifies to $\sqrt{6}/9$. For the single anchor in a

rectangular pedestal, the normalised resistance simplifies to $\sqrt{3r(3r + 7)}/90$, which equals $r(0.7 + 0.3r)/3^{1.5}$.

6. Implications

6.1 Concrete strength, cracking and code

Table 2 gives the ceiling of a 700 mm square pedestal for a range of concrete strengths. Because the ceiling scales with $\sqrt{f_{ck}}$, doubling the concrete strength from C25/30 to C50/60 raises it by 41%, whereas using uncracked concrete raises it by 43%. Fig. 3 compares EN 1992-4 and ACI 318-19 over pedestal sizes. For the optimal group the ratio is constant at 1.18 up to $a = 1.5$ m and rises to 1.22 at 2.1 m, where the limited embedment exceeds 280 mm; for a single anchor, whose limited embedment is twice as deep, the ACI ratio rises above 1.18 from $a \approx 0.84$ m, reaches 1.34 at 1.8 m and returns to 1.18 once h'_{ef} exceeds 635 mm.

Table 2. Ceiling of a 700 mm square pedestal with the optimal group (kN).

f_{ck} (MPa)	20	25	30	35	40	50
Cracked	133.8	149.5	163.8	176.9	189.2	211.5
Uncracked	190.9	213.4	233.8	252.5	269.9	301.8

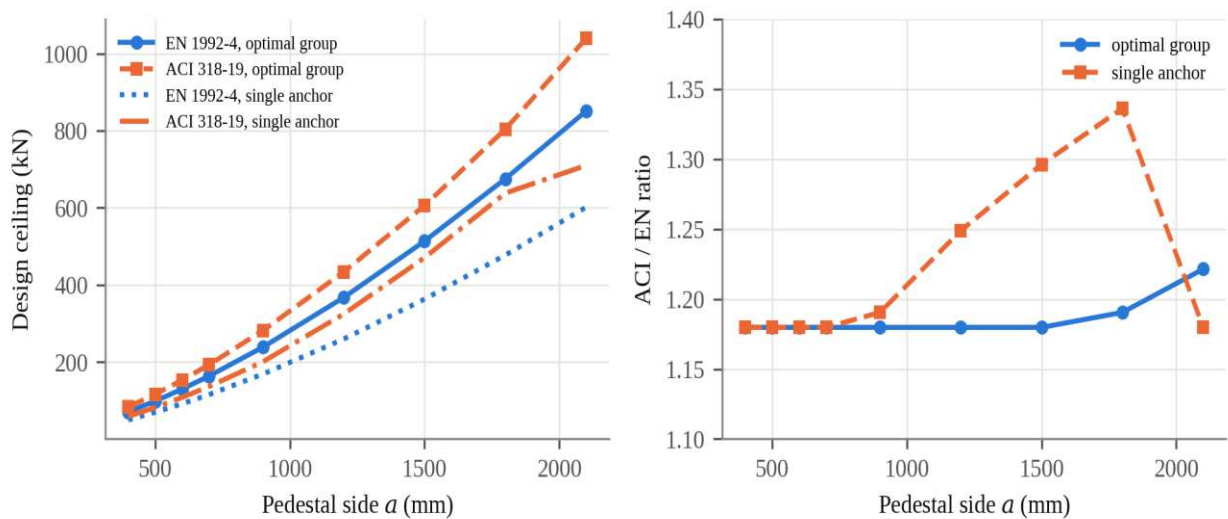


Fig. 3. EN 1992-4 and ACI 318-19 design ceilings (left) and their ratio (right) for single anchors and optimal groups in square pedestals, $f_{ck} = f'_c = 30$ MPa, cracked concrete.

6.2 Failure-mode hierarchy and the pedestal size for ductility

Because the ceiling contains no bolt property, the failure mode of a pedestal anchorage is decided by comparing the bolt steel resistance with the ceiling. Table 3 gives, for four-bolt

groups, the characteristic steel resistance $N_{Rk,s} = 4A_s f_{uk}$, the design steel resistance with $\gamma_{Ms} = 1.2f_{uk}/f_{yk} \geq 1.4$, and two pedestal sizes in C30/37 cracked concrete: the side at which the ceiling equals the design steel resistance, above which steel governs the design check, and the side at which the ceiling equals the characteristic steel resistance, above which the bolts can be expected to yield before the concrete breaks out. Fig. 4 plots the same results.

Table 3. Pedestal side (mm) at which steel failure governs and at which the ceiling reaches the characteristic steel resistance; four bolts, C30/37 cracked.

Bolts	Grade	$N_{Rk,s}$ (kN)	$N_{Rd,s}$ (kN)	a for $N_{Rd,c,max} = N_{Rd,s}$	a for $N_{Rd,c,max} = N_{Rk,s}$	Pull-out $N_{Rd,p}$ (kN)
4 M16	4.6	251	126	586	931	633
4 M20	4.6	392	196	789	1252	990
4 M24	4.6	565	282	1006	1598	1425
4 M24	8.8	1130	753	1935	2536	1425
4 M30	4.6	898	449	1371	2176	2227
4 M30	8.8	1795	1197	2636	3454	2227
4 M36	8.8	2614	1743	3386	4437	3206

Pull-out resistance was computed with $k_2 = 7.5$ (cracked) and an assumed bearing diameter of $2.5d$. Even the smallest group considered, four M16 grade 4.6 bolts, needs a 931 mm pedestal before its characteristic steel strength is below the concrete ceiling, and four M24 grade 4.6 bolts need 1598 mm. Higher-grade bolts move the threshold further out, because they raise the steel resistance without changing the ceiling. In pedestals of practical size, concrete breakout therefore governs every realistic bolt group, and a ductile hierarchy cannot be obtained by the choice of bolts. It requires anchor reinforcement sized to exceed the steel resistance of the bolts, with the caveat from Petersen et al. [22] that code-conforming reinforcement does not always guarantee steel failure.

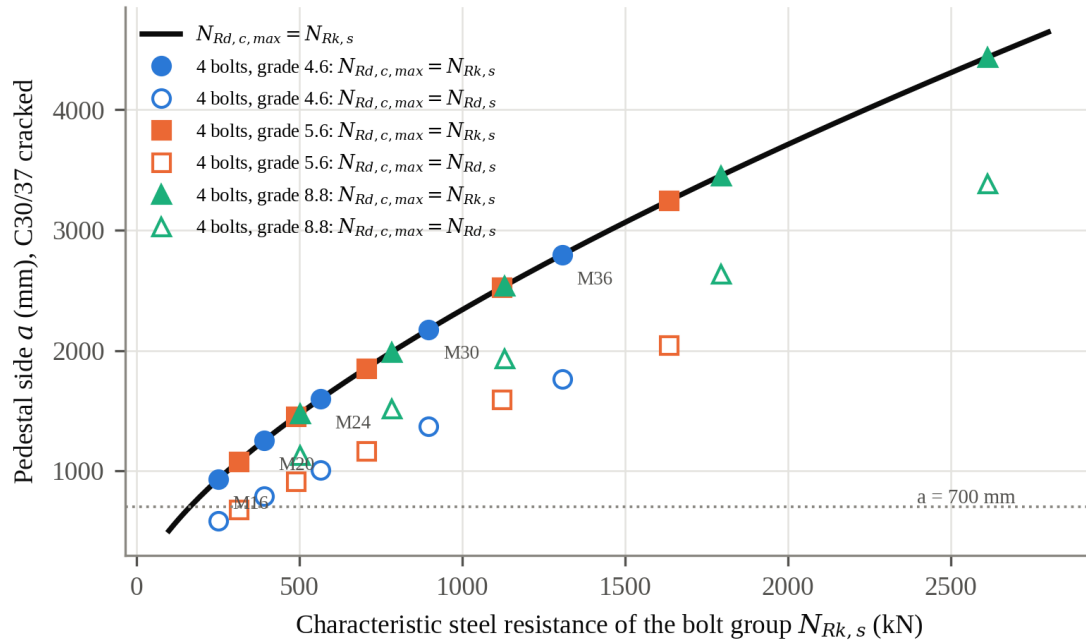


Fig. 4. Square pedestal side at which the breakout ceiling equals the characteristic (filled) and design (open) steel resistance of four-bolt groups, C30/37 cracked. The dotted line marks a typical 700 mm pedestal.

Side-face blow-out also needs attention at the embedment required for the anchor reinforcement. With the anchors at the quarter points, the edge distance is $a/4$, so the blow-out check applies once h_{ef} exceeds $a/2$. For a 700 mm pedestal, the basic design blow-out resistance of a single M30 anchor with a 75 mm bearing diameter is 339 kN, well above the ceiling share of 41 kN per anchor, so blow-out does not govern the concrete-only anchorage; it becomes relevant when anchor reinforcement carries loads several times the ceiling, and confining links near the heads are then advisable.

6.3 Design chart

Fig. 5 plots Eq. (7) for several concrete classes. It answers the scheme-stage question directly: a pedestal intended to carry 300 kN by concrete breakout in cracked C30/37 concrete needs a side of about 1.05 m, and 600 kN needs about 1.66 m. Where the pedestal is smaller than the chart indicates, anchor reinforcement is required regardless of the bolts chosen. As an illustration, a four-leg elevated tank tower 15 m high with a 100 m³ heavy tank, analysed at a basic wind velocity of 40 m/s, develops a design uplift of about 505 kN

per stanchion; Eq. (7) then requires a 1.48 m square pedestal in cracked C30/37 concrete, more than twice the 700 mm pedestal that such towers typically receive, so the decision to provide anchor reinforcement can be taken as soon as the uplift is estimated.

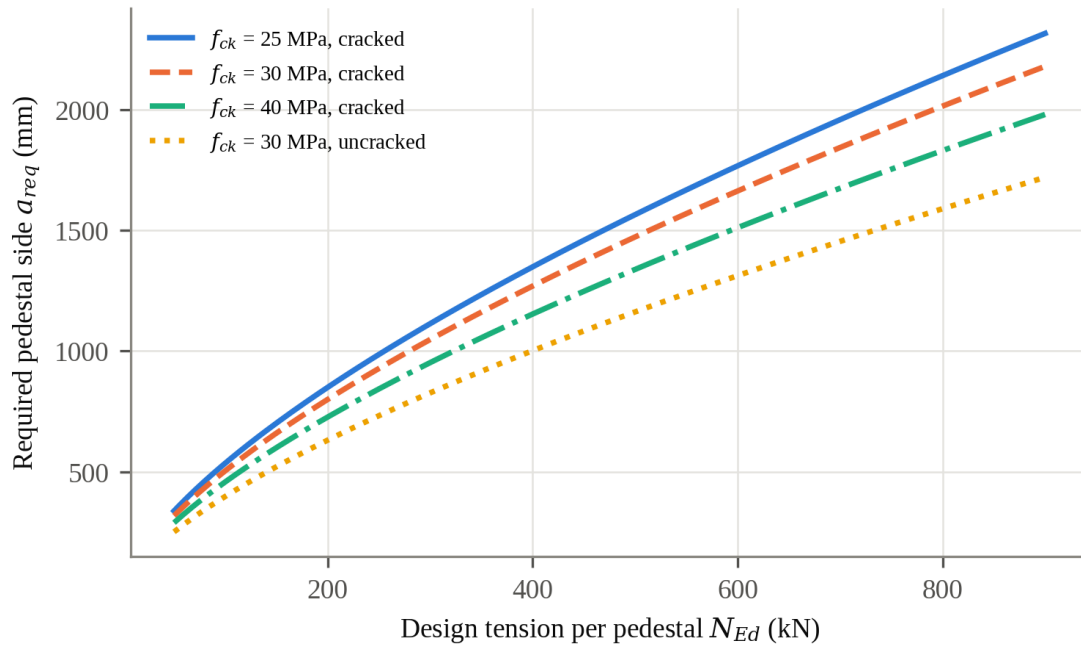


Fig. 5. Square pedestal side required to carry a design tension by concrete breakout alone, Eq. (7), for several concrete classes and cracking states.

6.4 Reading of s_{max}

Eq. (2) uses the spacing of the outermost anchors. If s_{max} were read as the spacing between adjacent anchors, groups with more than two rows would obtain a smaller h'_{ef} for the same outer spread, and the optimum of a 3 by 3 group in a 700 mm pedestal would rise from 163.8 kN to 200.3 kN (+22%) at an outer spread of 466 mm. The outer-spread reading is adopted here because it bounds the failure body of the whole group and gives the lower value; designers using software should check which reading is implemented.

6.5 Conservatism and limits

The ceiling is a property of the design model. Test evidence indicates that the true resistance continues to rise beyond the plateau and that the narrow-member rule is conservative [11], that member thickness and surface reinforcement add resistance [8,9], and that the CCD method can be accurate for column-base anchor groups [23]. The ceiling should therefore be

read as a lower bound on the physical resistance and as the upper bound the code allows. Its validity is limited by the conditions of Section 3: concentric tension, doubly symmetric groups, sufficient member depth, and cast-in headed anchors. Splitting of deep groups in narrow members [17] and the reinforcement of the pedestal, which is not credited unless detailed as anchor reinforcement, are outside the model. The evidence for conservatism comes mainly from members with two close edges; tests on anchor groups in pedestals with four close edges would establish how much of the reserve could be credited.

7. Conclusions

1. Under the EN 1992-4 narrow-member rule, the design breakout resistance of any doubly symmetric bolt group in a rectangular pedestal with sides $a \leq b$ cannot exceed $(\sqrt{6}/9)k_1\sqrt{f_{ck}}a\sqrt{b}\psi(r)/\gamma_{Mc}$, with $\psi = 1$ for $r = a/b \geq 0.5$ and $0.7 + 0.6r$ otherwise; for a single anchor the bound is $k_1\sqrt{f_{ck}}b^{1.5}r(0.7 + 0.3r)/(3^{1.5}\gamma_{Mc})$.
2. The bound is reached with the outer anchors spaced $b/2$ along the long side, is independent of bolt diameter, grade, length and number, and was reproduced by numerical optimisation to within 0.03%.
3. For a given plan area, the ceiling varies as $r^{0.25}$, so a square pedestal is the most efficient plan.
4. The ACI 318-19 ceiling has the same form and is 1.18 times the EN 1992-4 value, while the limited embedment is below 280 mm.
5. A ductile anchorage in which the bolts yield first requires pedestals far larger than those used in practice (about 1.6 m for four M24 grade 4.6 bolts in C30/37), so in practical pedestals concrete breakout governs and ductility must be provided by anchor reinforcement.

Declaration of generative AI and AI-assisted technologies in the writing process

During the preparation of this work, the author used Claude (Anthropic) to assist with drafting, restructuring the thesis material into article form and implementing the analysis scripts. After using this tool, the author reviewed and edited the content as needed and takes full responsibility for the publication's content.

CRedit authorship contribution statement

Conceptualization, methodology, software, validation, formal analysis, writing – original draft, review and editing

294 **Declaration of competing interests**

295 The authors declare that they have no known competing financial interests or personal
296 relationships that could have appeared to influence the work reported in this paper.

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300 **Data availability**

301 The analysis scripts and generated data (Python; seed 20260925 for the random sample) are
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303
304 **Ethical Approval**

305 Not applicable. This study did not involve human participants or animals.

306
307 **Consent to Participate**

308 Not applicable.

309
310 **Consent to Publish**

311 Not applicable.

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313 This article is based on typical project work in River State, Nigeria. The design case reactions
314 in Section 3.4 are published with the client's permission.

315
316 **Data Availability**

317 The datasets generated during and/or analyzed during the current study are available from the
318 corresponding author on reasonable request.

319
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