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Concrete Capacity Design (CCD) Approach for Fastening to Concrete. Paper by Werner Fuchs, Rolf Eligehausen, and John E. Breen

Discussion by **LEROY A. LUTZ**
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This is a very informative article on anchorage behavior and on explanation of an analytical procedure to predict tensile and shear capacity for anchors. I had previously examined the proposed ACI Code provisions developed earlier based on using the CCD method. I found that several of my questions developed from that examination of the CCD method have been answered in this article.

My initial concern related to the fact that the original equation for tensile capacity of a section of limited size indicated that a lesser capacity was calculated for a deep anchor as compared to a shallow anchor. This occurred because the nominal stress decreased in proportion to $1/\sqrt{h_{ef}}$, while the area of the failure cone was constant when it encompassed the entire concrete cross section. This concern was addressed by the expansion of the definition of h_{ef} used in Eq. (10) so that the value of h_{ef} used in the equation was limited by the maximum edge distance. To be more correct, the reference for use of the newly defined h_{ef} should include Eq. (9), as well as Eq. (10b) and Eq. (10a) to assure that the same h_{ef} is used in evaluating N_{no} in Eq. (9).

A similar concern with the calculated shear capacity of anchors with large edge distances in the direction of the shear force was addressed by the expansion of the definition for c_1. However, this definition is complex. It took some effort to understand this definition when it was first encountered.

I believe that a more straightforward approach to handle this situation would be to create a new variable c that differs from c_1. c_1 would be the actual edge distance; c would be defined as $max(c_{2max},h) \leq 1.5c_1$, the governing perpendicular edge distance.

Then

$$\psi_5 = 0.7 + 0.3c_{2min}/c;\ c_2 \text{ is actually } c_{2min}$$

$$\psi_4 = 1/(1 + e_v'/c)$$

and

$$V_{no} = 7.08(\ell/d_o)^{0.2}\ \sqrt{d_o}\sqrt{f_c'} \cdot c^{1.5}$$

since the defined c was indicated as being applied to all components of Eq. (13a). Since variable c represents the dimension perpendicular to the anchor, the ψ_4 and ψ_5 expressions become simpler without a 1.5 factor. As noted in the definition, the shear failure load may be independent of the actual edge distance c_1.

This approach of defining the governing effective distance with a different variable name could also be used for the anchor in tension. For example, the actual anchor depth could be identified as h_a, while h_{ef} could be retained as the effective anchorage depth.

Even with this change, the use of Eq. (10) and (13) would be easier to apply and visualize conceptionally if the expressions were rewritten simply in terms of a stress times an area. This is done in ACI 318-89, Chapter 11 in the evaluation of the various shear force limits and in the evaluation of anchors using ACI 349. If the user is performing a hand calculation using Eq. (10) and (13), less computation is necessary if N_{no}/A_{no} are combined into a simple axial stress term, and V_{no}/A_{vo} are combined into a simple shear stress term. The present format is of little benefit to the designer.

On another matter, I was glad to see that a specific provision for shear capacity parallel to an edge was presented. This allows the designer to take advantage of the extra capacity as compared with load toward an edge. It appears that one must simply calculate $V_{n\perp}$ and then multiply by 2 to obtain $V_{n||}$. However, one should be aware that one can calculate V_n using Eq. (13a) for a single anchor near a corner for shear toward each of the two faces. By doing this and varying the relative distance to the two faces, one will find that the relative capacity in the two perpendicular directions continues to increase as the ratio of the two edge distances increases. These results indicate that the factor of 2 previously noted cannot be substantiated by the V_n calculations. Maybe a limit needs to be applied to the shear calculation for the corner condition, perhaps by using a further redefinition of c_1.

One item of importance to designers that was not addressed by the authors is a recommendation on evaluating the shear capacity of anchor groups where all anchors do not have the same edge distance. The commentary to the proposed ACI Code provisions developed earlier suggested that all anchors share the total load equally. This was identified as the "elastic distribution," where the capacity of the weakest anchor controls the group's capacity.

This approach does not necessarily give the "strength" of the group, but may be just the load where initial failure occurs and load redistribution begins. The current procedure for anchor groups in the *PCI Design Handbook*[5] or in Section 1925.3.3 of the UBC Code,[33] where the capacity of the anchors away from the edge is considered, is a definite improvement for group capacity evaluation.

Admittedly, one anchor's failure may influence the capacity of an adjacent anchor, but the designer needs a procedure that will give a reasonable strength value for all situations. A very common situation, that of four anchors in a 2 x 2 pattern in the top of a rectangular pier, should be addressed properly, not in an overly conservative manner.

REFERENCES

33. *Uniform Building Code*, V. 2, International Conference of Building Officials, Whittier, CA, 1994.

RESPONSE TO LEROY A. LUTZ

The authors appreciate the positive and helpful discussion by Mr. Lutz.

As indicated in the original paper, for fastenings with three or four edges and $c_{max} \leq 1.5\ h_{ef}$ [c_{max} = largest edge distance; for examples, see Fig. J(a) and (b)], the embedment depth should be limited to $h_{ef}' = c_{max} / 1.5$. This gives a constant failure load for deep embedments. Mr. Lutz is correct that for these cases, the newly defined h_{ef}' should be used in Eq. (9), (10d), and (11b) and for the calculation of A_N and A_{No} (see Fig. 11). Similarly, for fastenings in a narrow, thin member with $c_{2,max} \leq 1.5\ c_1$ (for example, see Fig. K), the edge distance should be limited to $c_1 = max\ (c_{2,max}/1.5;\ h/1.5)$. This gives a constant failure load independent of the edge distance c_1. The newly defined c_1 should be used in Eq. (12), (13b), and (13c) and for the calculation of A_V and A_{Vo} (see Fig. 17). The use of this proposal is demonstrated in Fig. J and K. Apparently, these more precise definitions are needed to clarify the application raised by Mr. Lutz. The corrected equations should read

$$V_m = \left(\frac{\ell}{d_o}\right)^{0.2} \sqrt{d_o}\sqrt{f_{ci}'}\ c_i'^{\,1.5},\ \text{lb} \tag{14a}$$

$$\psi_4 = \frac{1}{1 + (2_v'/3c_i')} \tag{14b}$$

$$\psi_5 = 0.7 + 0.3\frac{\min c_2}{1.5c_i'} \tag{14c}$$

Mr. Lutz also suggests the combination of the quotient N_{no}/A_{no} to a shear stress τ_o. In principal, this could be done. However, the authors do not propose to do so because the stresses σ_o and τ_o are not constant. Instead, they decrease with increasing embedment depth or increasing edge distance, respectively. They believe this would be an annoyance to the designers.

Mr. Lutz discusses the case of a single anchor near the corner. In this case, the capacity $V_{u\perp}$ and $V_{u||}$ must be calculated for both edge distances, and the smaller value governs the design (see Fig. L). The ratio $V_{u\perp}/V_{u||}$ varies with varying ratio c_1/c_2.

It is $\alpha = 2.0$ for $c_1 = c_2$, $\alpha > 2$ for $c_1 < c_2$ and $\alpha < 2$ for $c_1 > c_2$. This tendency seems correct. However, it should be noted that up to the present time, only the case of a single anchor in a corner with equal edge distances in the two directions has been tested.

Lastly, Mr. Lutz asks for a procedure to calculate the shear resistance of anchor groups where all anchors do not have the same edge distance (e.g., double-fastening perpendicular to the edge or quadruple fastenings) situated at the edge or in a corner and loaded by a shear force with an arbitrary angle between shear force and edge. This application has not been covered in the paper because of reasons of space. Engineering models based on the CCD method for calculating the shear resistance in this case have been proposed in References 8 and 42. This proposal has also been incorporated in the draft CEB Design Guide.[43] However, it should be pointed out that only limited test data is available to check the accuracy of the proposed equations. Hopefully, it will be discussed in ACI Committee 318 whether this proposal should be incorporated in the proposed Fastening to Concrete Chapter of ACI 318.

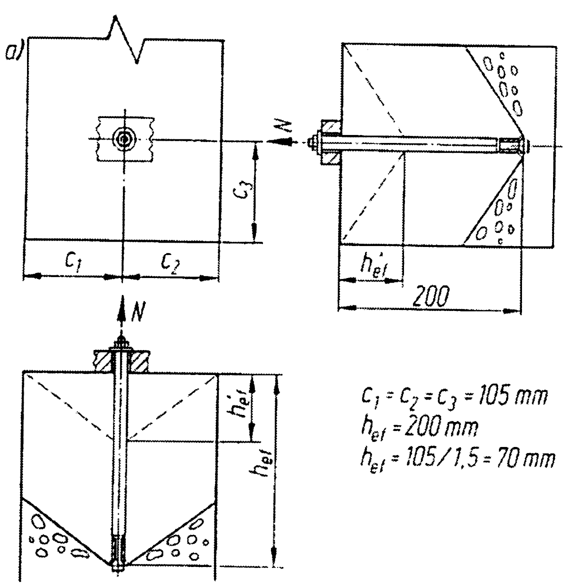


Fig. J(a)

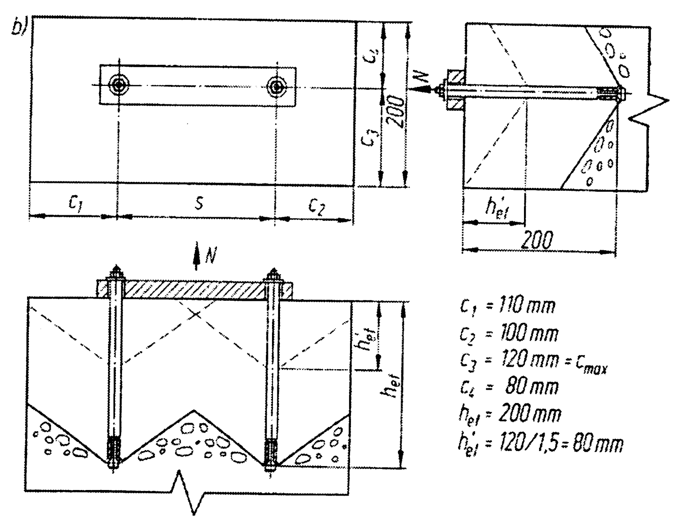
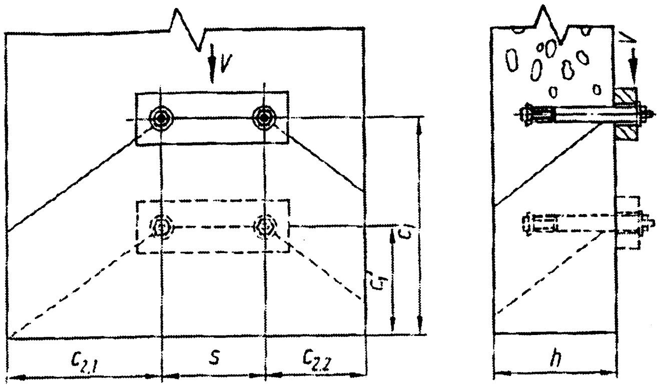


Fig. J(b)



$h = 120\,mm,\ s = 100\,mm,\ c_1 = 200\,mm < 1{,}5 \cdot 200\,mm,$

$c_{2,1} = 150\,mm < 1{,}5 \cdot 200\,mm,\ c_{2,2} = 100\,mm < 1{,}5 \cdot 200\,mm,\ c_1' = 150/1{,}5 = 100\,mm$

Fig. K

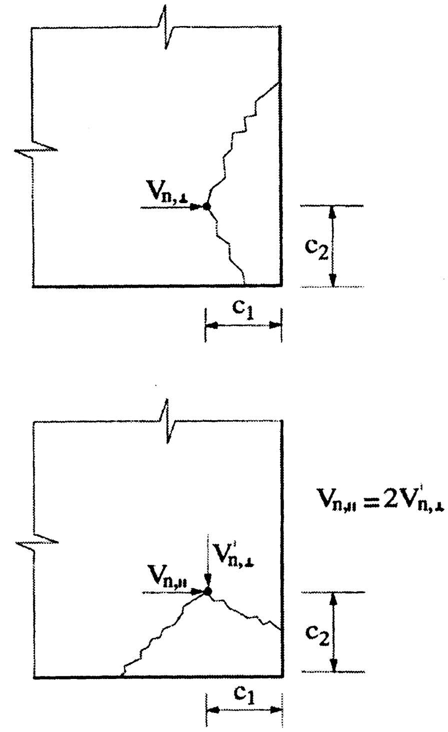


Fig. L