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EXPERIMENTAL INVESTIGATIONS ON CONCRETE EDGE FAILURE OF HEADED STUDS GROUPS IN REINFORCED CONCRETE

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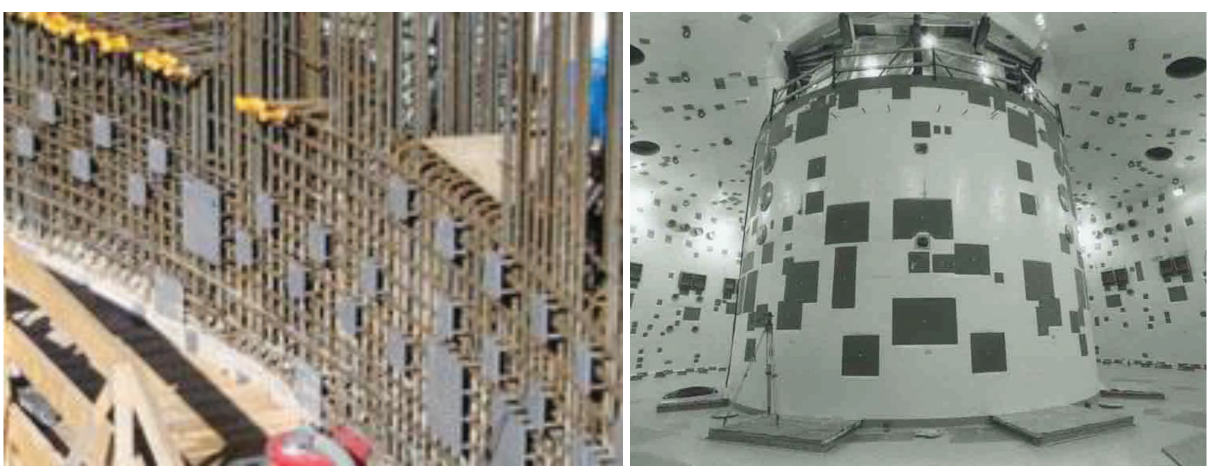
ABSTRACT

Due to their general robust behaviour and high reliability, embedded parts (EPs) as a group of headed studs welded to an anchor plate cast in concrete are heavily used in nuclear power plants. Frequently, the EPs consist of headed studs arranged in more than one anchor row. Typically, the concrete slabs in which the EPs are cast have significant amount of reinforcement. The behaviour of such anchor groups under shear, loaded perpendicular to an edge, in reinforced concrete is not well-researched to date. In this work, an optimum experimental program is carried out on anchor groups in unreinforced and reinforced concrete loaded in shear perpendicular to the edge. Anchor groups with up to four anchor rows perpendicular to the edge are tested in concrete with four different levels of shear reinforcement. A significant influence of shear reinforcement on failure loads of anchor groups was observed. The high conservatism in the approach according to draft EN 1992-4 to calculate the failure load of anchor groups under shear loading was clearly brought out. It was observed that in contrast to the current approach of assuming the failure crack at the front anchors, the real failure crack always starts from back row of anchors. Further, even a relatively small amount of reinforcement contributes to the failure load, however, beyond a certain point, adding further reinforcement does not increase the ultimate load. In these applications the peak load is limited by failure of compression strut. Both of these aspects are ignored in current model.

INTRODUCTION

Anchor plates in the form of a group of headed studs welded to the base plate form a reliable means of connection of structural or non-structural components to the parent concrete structure. In a typical nuclear power plant (NPP), hundreds to thousands of such anchor plates are often used (Figure 1).



Source: Köco Website — Eriksson and Gasch (2011)

Figure 1. Typical anchor plates installed during construction of NPP

The general behavior of such anchorages loaded in shear perpendicular to the edge is well-documented by Eligehausen et al. (2006). Under such loading conditions, the anchorages can fail by steel rupture, pryout or concrete edge failure. For concrete edge failure, the current approach to calculate the failure load of anchor groups under shear loading is deemed to be conservative when the anchors are installed in two or more rows perpendicular to the edge.

In case of anchors installed in concrete without reinforcement, the conservatism in the design approach according to EN1992-4 (2014) can be explained by the inbuilt assumption that states "the failure surface initiates from the first row of anchors and the total shear load is only transferred by this row of anchors". This assumption essentially implies that for the groups with same number of anchors parallel to the edge with same spacing and edge distance, the design failure load corresponding to concrete edge failure is independent of the number of anchor rows perpendicular to the edge. Figure 2 explains this assumption used for estimation of the failure load corresponding to concrete edge failure of a group of anchors in unreinforced concrete.

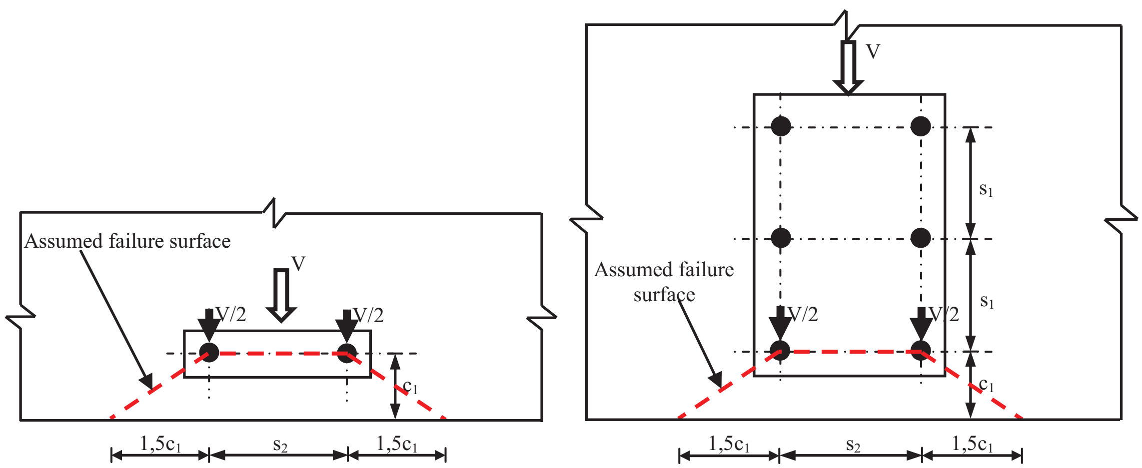


Figure 2. Assumption of failure surface for anchor groups loaded in shear perpendicular to the edge

Furthermore, the current models to evaluate failure of load of such anchor groups installed with supplementary reinforcement in concrete are not totally realistic. The presence of anchor (supplementary) reinforcement, in the form of edge reinforcement and stirrups, can have a significant influence on the load-carrying capacity of the anchorage subjected to shear loads perpendicular to the edge. In case of an anchor group in reinforced concrete, the possible failure modes can be listed as (i) Steel failure of anchor, (ii) Yielding of stirrups (following the concrete cracking), (iii) Strut (compression) failure, and (iv) Node (anchorage) failure. Currently, strut failure is neglected and a very conservative approach to consider the steel failure of anchor, stirrup yielding and node failure is given in EN1992-4 (2014).

The failure load corresponding to concrete edge failure followed by stirrup yielding is calculated assuming the failure crack originating from the front row of anchors (Figure 3). Due to this assumption, only the stirrups close to the anchor group are considered as active (stirrups A in Figure 3) while the stirrups farther away are ignored. Furthermore, a small anchorage length of the stirrups intercepted by the crack is calculated which results in a small capacity of the hook. Therefore, due to this assumption, the capacity evaluated by the current models is very conservative for groups with more than one anchor row. On the other hand, if the failure crack is assumed to initiate from the back row of anchors, more stirrups will be activated due to interception by the crack (stirrups A and B in Figure 3) and also a larger anchorage length is calculated for stirrups A resulting in a higher anchorage capacity of the stirrups. However, there is only limited research performed to investigate the anchorage capacity of anchor reinforcement and its influence on the failure load.

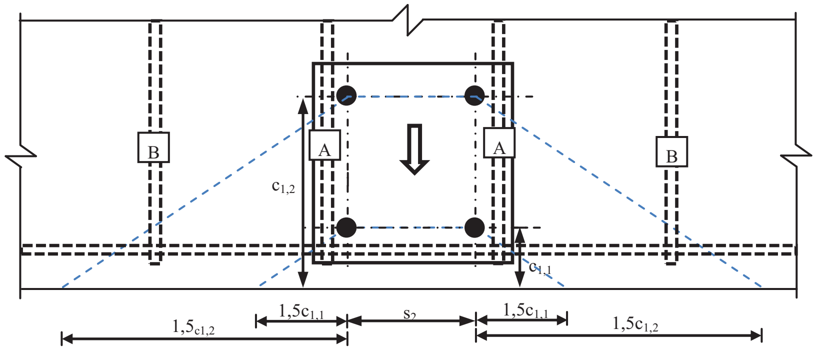


Figure 3. Influence of assumption of failure crack on stirrup contribution

Additionally, the enhancement in the capacity of the anchorages is considered only when the anchor reinforcement exceeds a certain minimum reinforcement. In reality however, even lesser amounts of reinforcement than the minimum value contribute towards enhanced load carrying capacity.

Although, the capacity of the anchorage under shear loading can be increased by providing anchor reinforcement, this increase is not unlimited. This is due to the fact that after a certain limit, the concrete strut becomes the weak link and starts governing the failure load. This is the absolute upper limit for the concrete edge resistance of an anchorage group. However, so far, there is no information on this limit due to strut failure and consequently EN1992-4 (2014) does not give any guidance to consider it.

In this work, experiments are performed on anchor groups with 2 to 8 headed studs cast in unreinforced and reinforced concrete, loaded in shear perpendicular to the edge to study their behavior in case of concrete edge failure. The evaluation of the test results clearly demonstrate that the current models are over-conservative in estimating the failure loads for unreinforced concrete as well as for low to medium percentage of reinforcement. However, for high percentages of reinforcement, the current models are prone to over-predict the failure load as they do not recognize strut failure as a possible failure mode.

TEST PROGRAM

Four different anchor group configurations were used for the experimental investigations. The groups are identified as (i) 1 x 2 group with one row of two headed studs; (ii) 2 x 2 group with two rows of two headed studs in each row; (iii) 4 x 2 group with four rows of two headed studs in each row; and (iv) 2 x 4 group with two rows of four headed studs in each row. The test program followed is given in Table 3.1. The concrete slabs were provided with edge reinforcement and stirrups of diameter, ds = 0 mm (unreinforced concrete), 12 mm, 16 mm and bundled reinforcement using 16 mm and 14 mm reinforcing bars. The tests were performed in the laboratory of Institute of Construction Materials, University of Stuttgart, Germany. The shear load was applied to the anchor plate through a shear loading plate. The load transfer between the shear loading plate and the anchor plate was facilitated through M24 bolts threaded into internally threaded holes, pre-drilled in the base plate. The uplift of the base plate was restrained during the tests. Figure 4 displays the typical test setup utilized to perform the experiments.

All the headed studs had a diameter of 22 mm. The minimum edge distance (distance from the free edge to the centre line of front row of anchors) was maintained at 85mm for all the cases, while the centre to centre distance between the anchors in either direction was kept as 150mm. The base plate had a thickness of 25 mm and the stud lengths were such that the effective embedment depth of the anchors was 190 mm. The average cubic concrete strength was obtained as 26.4 MPa, while the mean yield stress and mean ultimate stress for the reinforcing bars was obtained as 532 MPa and 573 MPa respectively. The mean ultimate strength of the headed studs was 518 MPa.

Table 1: Experimental program.

S. No.	Dia. of stirrups	Id.	n_{rows}	n_{cols}	$c_{1,1}$ (mm)	$c_{1,2}$ (mm)	$c_{1,3}$ (mm)	$c_{1,4}$ (mm)	s_1 (mm)	s_2 (mm)	No. of tests
1	0	1 x 2	1	2	85	--	--	--	--	150	3
2		2 x 2	2	2	85	235	--	--	150	150	3
3		4 x 2	4	2	85	235	385	535	150	150	3
4		2 x 4	2	4	85	235	--	--	150	150	3
5	12	1 x 2	1	2	85	--	--	--	--	150	3
6		2 x 2	2	2	85	235	--	--	150	150	3
7		4 x 2	4	2	85	235	385	535	150	150	3
8		2 x 4	2	4	85	235	--	--	150	150	3
9	16	1 x 2	1	2	85	--	--	--	--	150	3
10		2 x 2	2	2	85	235	--	--	150	150	4
11		4 x 2	4	2	85	235	385	535	150	150	3
12	16+14	1 x 2	1	2	85	--	--	--	--	150	3
13		2 x 2	2	2	85	235	--	--	150	150	3
14		4 x 2	4	2	85	235	385	535	150	150	3
15		2 x 4	2	4	85	235	--	--	150	150	3

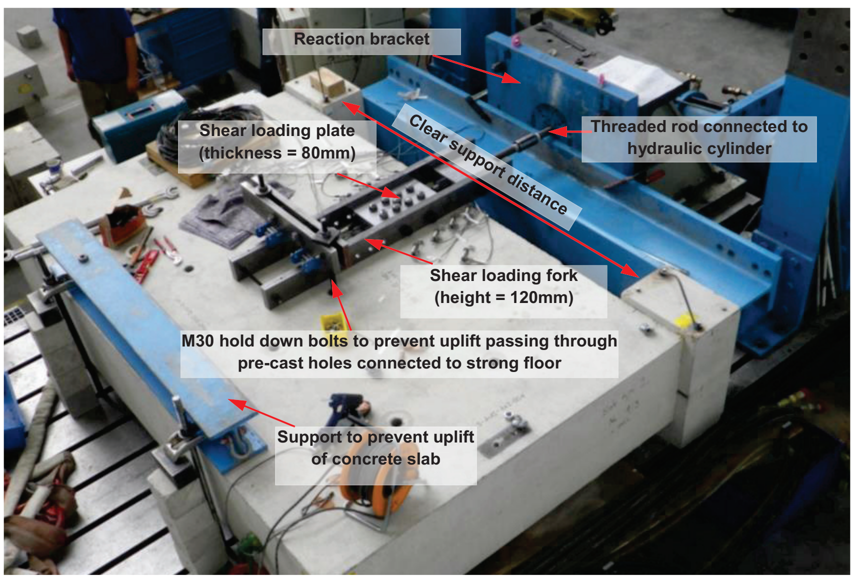
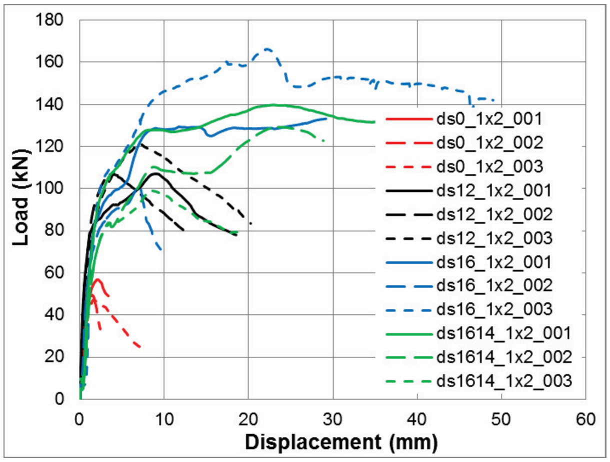


Figure 4. Test setup utilized for performing the experiments

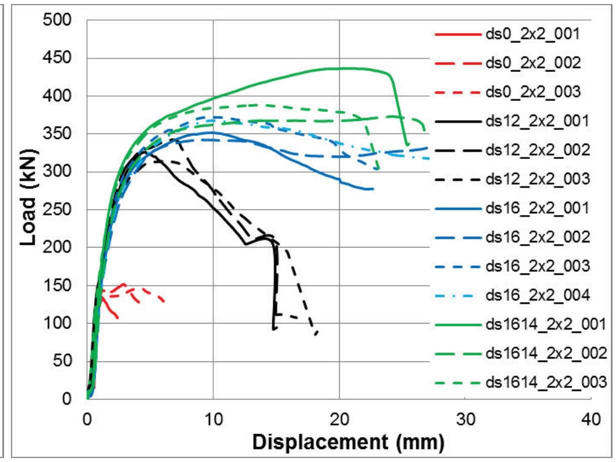
EXPERIMENTAL RESULTS

Figure 5 summarizes the load-displacement curves obtained from the tests performed in concrete slabs with different types of reinforcement for (a) Groups 1 x 2, (b) Groups 2 x 2, (c) Groups 4 x 2 and (d) Groups 2 x 4.

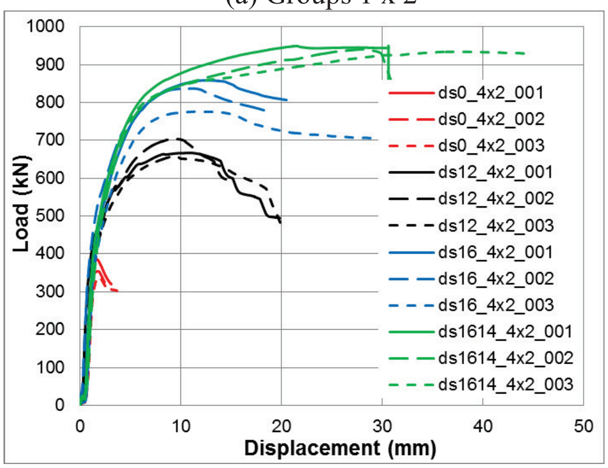
A very clear influence of the supplementary reinforcement is displayed by the load-displacement curves obtained from the tests. For group 1 x 2, due to small anchorage length of the stirrups, the contribution of reinforcement mainly came from the rope action offered by the longitudinal edge reinforcement. However, this contribution is unreliable as can be seen from Figure 5a, where the resistance in case of 16mm stirrups is higher than in case of 16+14mm bundled stirrups. The typical failure mode observed from the tests performed on groups 1 x 2 is shown in Figure 6a.



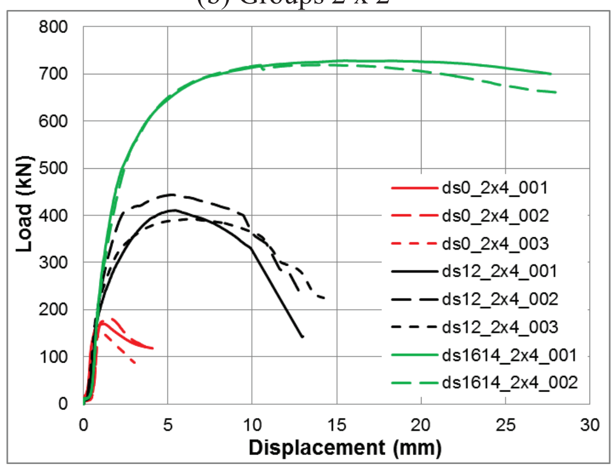
(a) Groups 1 x 2



(b) Groups 2 x 2



(c) Groups 4 x 2

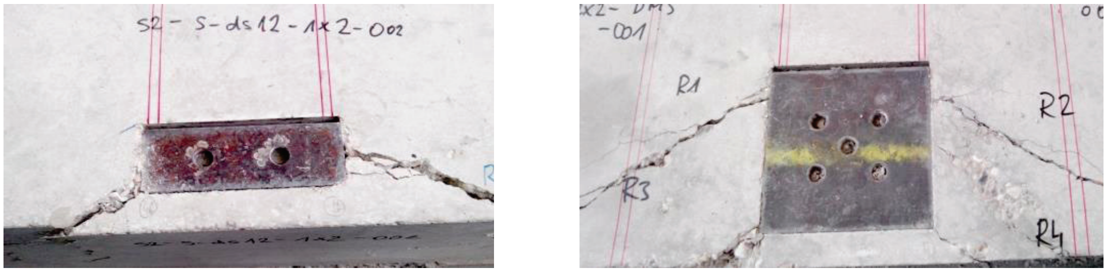


(d) Groups 2 x 4

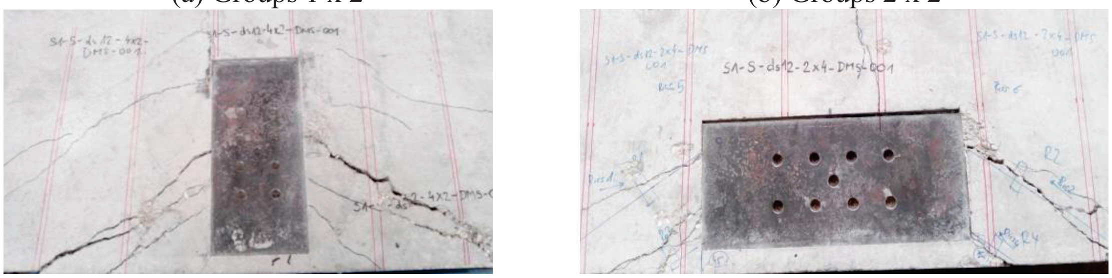
Figure 5. Load-displacement plots recorded from the tests performed on different anchor groups with different amounts of supplementary reinforcement

In case of groups 2 x 2, due to the crack initiating from the back anchor rows (Figure 6b), the stirrups have relatively large anchorage length and therefore a high contribution comes even at relatively low amount of reinforcement with the stirrups of 12 mm diameter (Figure 5b). On further increase in the reinforcement percentage (stirrups of 16mm diameter and bundled stirrups of 16+14mm diameter), the failure load further increases but the rise is not so strong and the failure load seems to get saturated. This points out to the fact that there is an upper limit to the beneficial effect of the supplementary reinforcement on the failure load. For this group, the upper limit of the failure load in reinforced concrete is of order of 2.8 times the failure load in unreinforced concrete.

In case of 4 x 2 anchor groups (Figure 5c) again an addition of a relatively small amount of reinforcement (12 mm stirrups) leads to a significant increase in the failure load compared to the tests in unreinforced concrete. However for this case, in contrast to the earlier two cases, further increasing the diameter of stirrups from 12mm to 16mm also led to a reasonable increase in failure loads suggesting that reinforcement failure continued to govern the failure mode. Increasing the reinforcement using 16+14mm bundled stirrups also led to a further increase in failure load but this increase was not proportional and the failure load seems to get saturated. For this group, the upper limit of the failure load in reinforced concrete is of order of 2.6 times the failure load in unreinforced concrete. The crack pattern (Figure 6c) shows the failure crack appearing from the back row of the anchors.



(a) Groups 1 x 2 (b) Groups 2 x 2



(c) Groups 4 x 2 (d) Groups 2 x 4

Figure 6. Typical failure modes obtained by the tests on anchor groups tested in concrete with stirrups of 12 mm diameter

In case of 2 x 4 anchor groups (Figure 5d), consisting of two anchor rows with 4 anchors in each row, the tests were performed only with 12 mm stirrups and 16+14mm bundled stirrups. Again an addition of a relatively small amount of reinforcement (12 mm stirrups) led to a significant increase in the failure load compared to the tests in unreinforced concrete. However for this case, in contrast to the earlier cases, increasing the reinforcement using 16+14mm bundled stirrups resulted in the failure load of the order of 4.3 times the failure load in unreinforced concrete. The crack pattern (Figure 6d) clearly shows the failure crack appearing from the back row of the anchors.

COMPARISON WITH CURRENT MODELS

According to EN1992-4 (2014), in case of anchorages with anchor reinforcement in form of stirrups and edge reinforcement, the load corresponding to failure of reinforcement in the concrete breakout body can be obtained on the basis of the strut-and-tie model as shown in Figure 7.

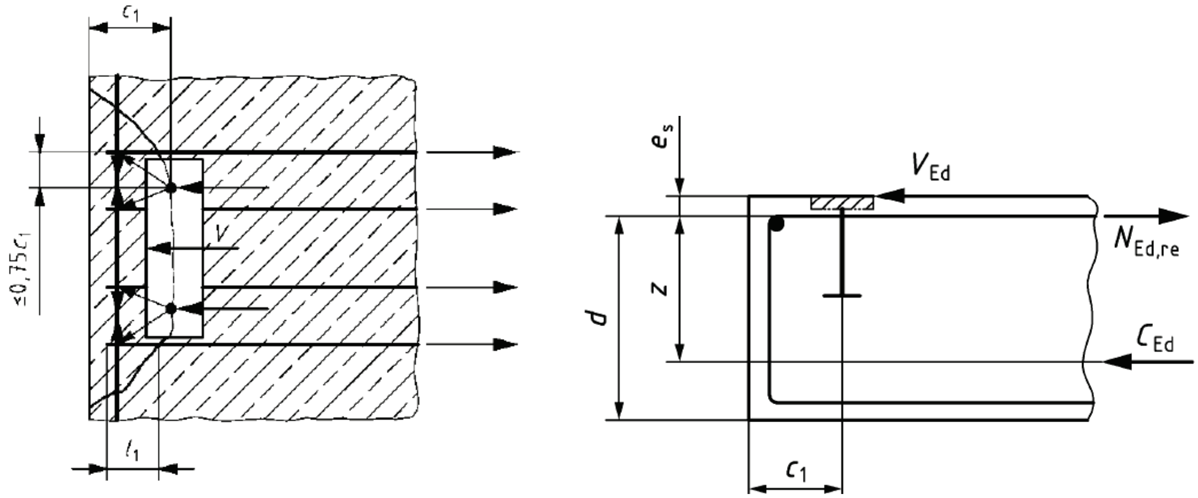


Figure 7. Simplified strut-and-tie model for anchor reinforcement by EN1992-4 (2014)

As per EN1992-4 (2014), if the shear force is taken up by anchor reinforcement according to Figure 7, the following additional requirements should be met:

i. Only bars with a distance $\leq 0.75c_1$ from the fastener should be assumed as effective.
ii. The anchorage length l_1 (Figure 7) in the concrete breakout body is at least equal to 10 times the rebar diameter (straight bars with or without welded transverse bars) or at least equal to 4 times the rebar diameter (bars with a hook, bend or loop).
iii. Reinforcement along the edge of the member should be provided and be designed for the forces according to an appropriate strut and tie model (Figure 7). As a simplification an angle of the struts of 45° may be assumed.

As per the strut-and-tie model, the characteristic resistance $V_{Rk,re}$ of the supplementary reinforcement of one fastener in case of an anchorage failure in the concrete edge break-out body is given by:

$$V_{Rk,re} = \sum_{n} V^{0}_{Rk,re} \tag{1}$$

With,

$$V^{0}_{Rk,re} = \frac{l_1 \cdot \pi d_s \cdot f_{bk} / \alpha}{x} \leq \frac{f_{yk} A_{s,re}}{x} \tag{2}$$

Where,
n = number of legs of the anchor reinforcement effective for one fastener
l_1 = anchorage length = distance from the intersection of theoretical crack and the rebar to the stirrup end
d_s = diameter of rebar
f_{bk} = characteristic bond strength = 1.5 f_{bd}
f_{bd} = design bond strength according to EN1992-1-1 (2004)
f_{yk} = characteristic yield strength of reinforcing bars
$A_{s,re}$ = Area of reinforcing bar used as stirrup
α = influencing factor that assumes a value of 0.7 for hooked rebar and 1.0 for straight rebar
x is the factor to consider for the lever arm between the reinforcement and the applied shear load (compare Figure 7)

$$x = \left(1 + \frac{e_s}{z}\right)$$

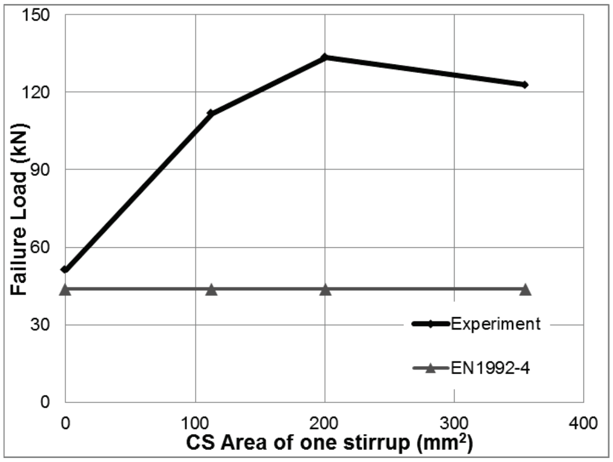
e_s = distance between reinforcement and shear force acting on a fixture
z = internal lever arm of the concrete member that is approx. equal to 0.85d
d = min(depth of concrete member, 2h_{ef}, 2c_1)

If the evaluated failure load corresponding to the concrete edge breakout in unreinforced concrete is $V_{Rk,c}$ then as per EN1992-4 (2014), the failure load corresponding to concrete edge failure for an anchorage in with supplementary reinforcement is given as

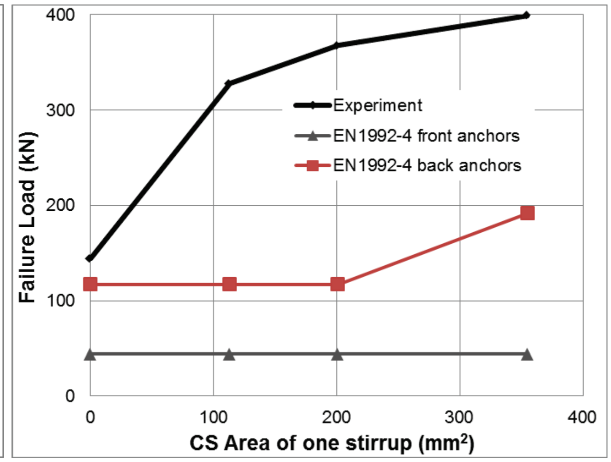
$$V_{Rk} = \max\left(V_{Rk,c}; V_{Rk,re}\right) \tag{3}$$

The failure loads predicted as per the EN1992-4 (2014) model, converted to the mean resistance values for different anchor groups are given in Figure 8. Although EN1992-4 (2014) recommends considering the failure crack initiating from the front anchor row, based on the tests where it was observed that the failure crack is always initiated from back anchor rows. Therefore, in this work, the experimentally obtained mean failure loads are compared with the analytical failure loads evaluated using EN1992-4 (2014) model considering crack once from front anchor row and once from back anchor row.

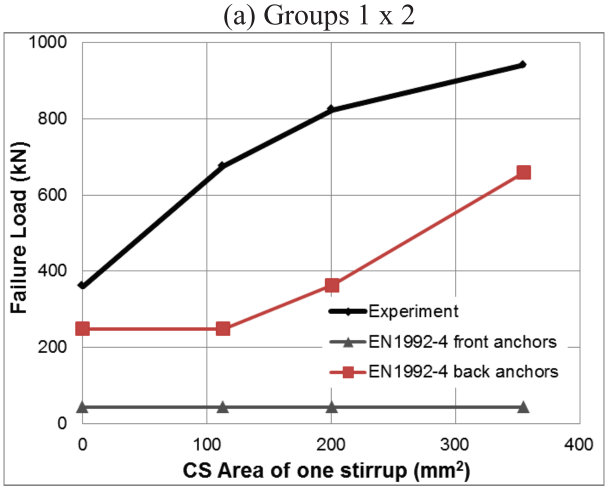
In figure 8, the mean failure loads for different groups are plotted as a function of the area (in mm^2) of a single stirrup used as supplementary reinforcement, namely 0 (unreinforced concrete), 113 (12 mm stirrups), 201 (16 mm stirrups) and 355 (16 mm + 14 mm bundled stirrups).



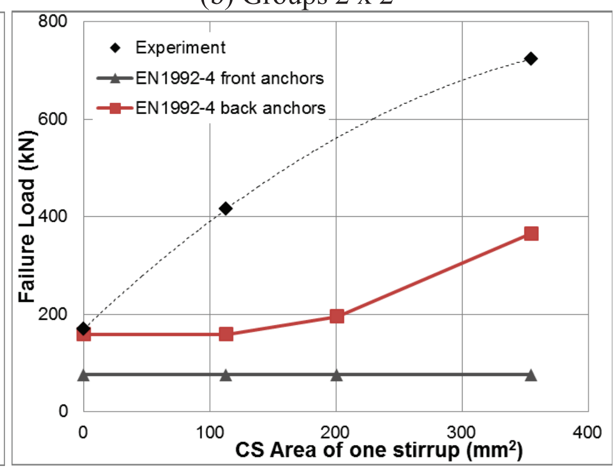
(a) Groups 1 x 2



(b) Groups 2 x 2



(c) Groups 4 x 2



(d) Groups 2 x 4

Figure 8. Load-displacement plots recorded from the tests performed on different anchor groups with different amounts of supplementary reinforcement

In case of groups 1 x 2 (Figure 8a), the mean test failure load for the groups tested in unreinforced concrete matches reasonably well with the evaluated mean failure load value. However, due to a short anchorage length, in no case any contribution from stirrups is considered by the EN1992-4 (2014) model. The analytically evaluated failure capacity for the anchor group 1 x 2 is independent of the diameter of stirrups and is equal to the capacity evaluated in unreinforced concrete. Although, the EN1992-4 (2014) model seems to under-predict the measured failure loads of the group, given the unreliability of the contribution of the rope action of edge reinforcement for this anchor group, the approach given in the model seems reasonable.

Figure 8b presents the comparison of experimentally obtained and analytically evaluated mean failure loads for the group 2 x 2, as a function of cross-sectional area of one stirrup, as calculated by EN1992-4 model considering crack once from front and once from back anchors. When the crack is assumed from front anchors, no stirrups are considered as effective. Consequently, the group capacity is always equal to the concrete capacity in unreinforced concrete evaluated from front anchors. When the crack is considered from back anchors, one stirrup on each side of the anchorage becomes effective. However, the stirrup contribution does not exceed that of concrete contribution until the case with ds16+14 bundled stirrups. Therefore, for the other three cases (unreinforced concrete, concrete reinforced with ds = 12mm stirrups and concrete reinforced with ds = 16mm stirrups), the capacity is the same as the concrete capacity in unreinforced concrete. It can be further observed that even in case of ds16+14 bundled stirrups the evaluated capacity is quite conservative compared to the experimentally obtained mean failure load.

Figure 8c gives the comparison of experimentally obtained and analytically evaluated mean failure loads for group 4 x 2, considering crack once from front and once from back anchors. When the crack is considered from front anchors, again no stirrups are considered effective and hence the failure load in reinforced concrete is the same as that in unreinforced concrete. Obviously this is unrealistic and overly conservative compared to the experimentally obtained failure loads. When the crack is considered from the back anchors, stirrups within a distance of 0.75 times the edge distance of back anchor row (= 535 mm) are considered as effective. Thus, two stirrups on either side of the anchorage at a distance of 25 mm and 225 mm from the headed studs are effective. For the case of 12 mm stirrups, the calculated failure load corresponding to reinforcement failure is less than the concrete edge failure load in unreinforced concrete and hence the failure no enhancement in the load-carrying capacity is obtained even when the crack is considered from back anchors. However, for the other two cases, the failure load increases due to the presence of reinforcement. The trend of calculated failure loads as a function of stirrup cross-section area shows that the failure load first remains constant and then rises sharply with the reinforcement. This trend is opposite to the actually obtained trend from experiments that display initially a very sharp rise in the failure load with reinforcement but later gets saturated.

Group 2 x 4 is different from all the other groups in the sense that the distance between the outermost anchors in this case is 450 mm instead of 150 mm and that two reinforcing bars lie in between the outermost anchors unlike any other anchor group that was tested in this program. The comparison of failure loads obtained for group 2 x 4 using EN1992-4 model considering crack from front and back anchors is given in Figure 8d. It may be noted that although, for this group, no tests were performed in concrete reinforced with 16mm stirrups, the analytical failure loads for such a case are included in Figure 8d. The dashed line shows a probable trend that the failure loads would have followed if tests were performed also in concrete reinforced with 16mm stirrups. For this anchor group, considering the crack from the front anchors, the calculated anchorage length, l_1, is less than the minimum required anchorage length of $4d_s$ for the case of 16mm and 16+14mm bundled stirrups. For d_s12mm stirrups, the calculated anchorage length is slightly larger than the minimum required anchorage length, which results in a small calculated value of anchorage resistance. However, since this resistance is smaller than the failure load corresponding to concrete edge failure, it does not lead to any increase in the failure load evaluated from the front anchors. Therefore, for this group, same as for previous cases, the reinforcement does not contribute to the failure load, as per the EN1992-4 model when assuming the crack from front anchors. While considering the crack from back anchors, two stirrups that lie in between the outermost stirrups and two stirrups at a distance of 25mm from outermost stirrups will be considered as effective. The comparison of experimental and analytical failure loads clearly indicate that the EN1992-4 approach for evaluating the capacity of anchor groups in reinforced concrete loaded in shear perpendicular to the edge is very conservative even when the crack is considered from back anchors.

CONCLUSION

In this work, an optimum experimental program is carried out on anchor groups in unreinforced and reinforced concrete loaded in shear perpendicular to the edge. Anchor groups with up to four anchor rows perpendicular to the edge are tested in concrete with four different levels of shear reinforcement. The major conclusions derived from the results of the tests and analytical evaluations are:

1. The reinforcement in the form of edge reinforcement and stirrups can significantly increase the load-carrying capacity of anchorages against shear loads applied perpendicular to the edge. This is valid also for a relatively small amount of anchor reinforcement.
2. The failure crack in case of supplementary reinforcement failure initiates from the back row of anchors resulting in a high anchorage length of the stirrups, which increases their capacity. Furthermore, more stirrups are engaged by the crack.
3. For the anchor group with only one anchor row (group 1 x 2) and with a small edge distance, the stirrups could not be activated. A rise in failure load due to a rope action of the edge reinforcement was observed. However, this increase is unreliable and should not be accounted for in design.
4. Although increasing the area of the anchorage reinforcement results in an increase of the load carrying capacity of the anchorage in general, this increase is not unlimited. It is capped by strut failure (compression failure of concrete) or steel failure of headed studs.
5. The current models to evaluate failure loads for anchorages with more than one anchor row perpendicular to the edge in reinforced concrete loaded in shear perpendicular to the edge, are in general, over conservative.
6. For the tested anchor groups, the EN1992-4 model, in its current form (assuming crack from front anchors), does not consider the contribution of reinforcement.
7. The EN1992-4 model is very conservative even when the crack is assumed from the back anchors.
8. Because no cap on the load carrying capacity for strut failure is assumed in the model, the analytical failure loads have a tendency to be unconservative for high amounts of shear reinforcement in the concrete slabs (higher than provided in the tests)
9. There is a need for developing a more rational and reliable method to analytically evaluate the shear failure loads of anchorages with more than one anchor row in reinforced concrete

Based on the test results reported in this paper, the authors are developing a new model that could realistically predict the failure loads for the anchorages in reinforced concrete loaded in shear perpendicular to the edge.

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REFERENCES

Eligehausen, R., Mallée, R. and Silva, J. (2006). *Anchorage in Concrete Construction*, Wilhelm Ernst & Sohn, Berlin, Germany, 391pp.

EN 1992-4. (2014). *Eurocode 2 — Design of concrete structures — Part 4: Design of fastenings for use in concrete*, final draft.

EN1992-1-1. (2014). *Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings*, European committee for standardization, Brussels.