SHEAR ANCHORING IN CONCRETE CLOSE TO THE EDGE

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Abstract
Since design rules for post-installed anchors are usually based on investigations in unreinforced concrete, the design resistance of fixings set close to the concrete edge and loaded in shear towards the edge is low. It is therefore necessary to consider the effective strength of the reinforced concrete edge. This paper first demonstrates that the current methods to take into account the strength of the concrete edge are insufficient in many cases and proposes different edge strengthening methods based on extensive laboratory testing. Finally a Eurocode 2 compatible design formula for cast-in hairpin reinforcement is derived.

1. Introduction

Limited space often requires the fixing of anchor plates like baseplates of steel columns, railings or lamp posts in the vicinity of the edge of the concrete foundation. Though in practice the concrete usually is reinforced, design rules for anchors are typically based on investigations in unreinforced concrete. The failure load of such fixings set close to the concrete edge and loaded in shear towards the edge is determined by a brittle concrete cone breaking out in front of the anchor. The shear resistance therefore reduces significantly with decreasing edge distance. If the edge distance is very small, splitting due to wedge forces must also be considered. Adhering to prescribed minimum edge distances will typically prevent this mode of failure.
Extensive testing has shown that standard slab lateral reinforcement is usually not sufficient to significantly increase the shear resistance of anchors loaded towards the edge. Higher shear resistance can be activated, for example, by using elongated holes in the anchorage base plate for the anchors situated closest to the edge or by externally reinforcing the edge. If no external reinforcement is to be applied, cast-in additional U-shaped reinforcement in the concrete has been shown to be a very effective method to strengthen the edge [1], [2].

For this strengthening method, a design formula based on the Eurocode safety concept as described in [3] has been derived from test results. This allows the design engineer to plan post-installed anchorages closer to the concrete edge, and with higher resistance than would be possible with standard anchor design concepts.

2. Unreinforced Concrete

The analytical determination of the ultimate load capacity is difficult as it depends on the behaviour of concrete under multiaxial stresses and has to consider the scatter in local concrete strength, size effects etc. Most equations for the prediction of failure loads have therefore been derived empirically, taking into account the observations from tests and are available only for the case without any retaining hanger reinforcement. Based on regression analyses of 147 tests Eligehausen and Fuchs propose the following equation for the calculation of the average ultimate failure load in unreinforced uncracked concrete [4, 5]:

\[ V_{\text{um,e}} = k \cdot d_{\text{nom}}^{0.5} \cdot f_{\text{cc}}^{0.5} \cdot \left( \frac{h_{\text{ef}}}{d_{\text{nom}}} \right)^{0.2} \cdot c_1^{1.5} \]  

(1)  

(units: [N, mm])

with:
- \( d_{\text{nom}} \) outside diameter of anchor
- \( h_{\text{ef}} \) anchorage depth [5]
- \( c_1 \) edge distance of anchor axis
- \( f_{\text{cc}} \) concrete cube strength
- \( k \) constant factor (\( k = 1.0 \))

3. Effect of Standard Reinforcement

A concrete plate with reinforcement bars parallel to the edge and with ties along the edge shall be considered as having standard reinforcement. Reinforced concrete is typically assumed to be cracked concrete. To account for the cracks, the load bearing capacity of uncracked concrete has to be multiplied by a global factor of 0.7 [5]. In the presence of minimum edge reinforcement, this reduction can be partially compensated for by a factor of 1.2 or 1.4, depending on the density of the edge reinforcement [6]. This is only a very rough approximation, and tests have shown that, for smaller anchor
diameters, it may even be unsafe. Small diameter anchors have a low bending resistance and introduce the load right at the top of the concrete surface. Thus, the failure cone is pushed over the edge reinforcement (Fig. 2) [7]. With greater anchor diameters the edge reinforcement may support the shear load more effectively because the anchor is able to introduce the load farther away from the concrete surface. Figures 3 and 4 show test results with edge reinforcement of diameters 6 – 12 mm and welded wire meshes. Especially the test results with M16 anchors are somewhat higher than those expected in unreinforced concrete, nevertheless the effect is rather negligible.

Fig. 2: Small anchors with edge reinforcement

Fig. 3 Tests with anchors HSL M12

Fig. 4 Tests with anchors HSL M16
4. Subsequent Constructive Measures

4.1 External Edge Support

A steel bar reinforces the concrete edge by anchoring the concrete edge behind the expected failure cone. The support steel bar is designed according to the rules of steel construction, assuming a uniformly distributed load. The anchorage is composed of bonded anchors reaching behind the expected failure cone (Fig. 5). The shear anchors (HSL) developed a very high resistance and typically failed by yielding of the steel rod in shear (Fig. 6).

4.2 Use of Elongated Holes in Anchorage Baseplate

If the anchorage baseplate has two or more rows of anchors, the holes for the anchors closest to the edge should be elongated, directed towards the edge. Thus, these anchors do not take any shear loads. The anchors of the row farther away can activate a bigger failure cone and, therefore, a higher load capacity.

5. Cast-In Hairpin Reinforcement

5.1 Research program

A comprehensive test program was carried out at the laboratories of Hilti AG [7] and the Institute for Concrete Construction at the University of Innsbruck [8] in order to quantify the effect of cast-in hairpin reinforcement and to develop corresponding design rules. The U-shaped hairpins were set with an inclination of 5°-10°.

The following parameters have been varied:

- hairpins: diameter d [mm]: 12, 16
- diameter of bend resp. distance e of hairpin legs: e = 88 - 134 mm
- concrete cover to front side (c_{sf} = 10 - 30 mm) and top surface (c_{st} = 7 - 33 mm)

- anchors: anchor type: expansion anchors HSL, bonded anchors HVU
- anchor diameter d [mm]: 12, 16, 20
The concrete cube strength $f_{cc}$ was between 25 and 30 N/mm² for all tests. The loading was displacement controlled. A total of 62 tests with cast-in hairpin reinforcement and 14 tests without any reinforcement in the anchorage area were carried out.

The load was introduced by a steel plate parallel to the concrete surface (Fig. 7). PTFE layers were put between steel plate and concrete in order to reduce friction.

5.2 Test results:

Failure usually occurs by formation of a failure cone followed by pullout or breakage of the anchor. Typically, the maximum load is reached, when the failure cone starts to break out. The cracks start from the anchor and run towards the edge. Their inclination to the concrete edge is 30° - 45°. Generally, the angle becomes smaller towards the edge (Fig. 8).

Some tests showed a second increase of the load after displacements of 10mm and more: One reason is the tensile capacity of the deformed, inclined anchor (kinking effect) and the second is that anchors set very close to the edge touched the hairpin reinforcement. Four tests with bonded anchors yielded in shear failure of the anchor without formation of a concrete cone (steel failure). These tests have not been considered in the development of the design recommendation for the resistance at the concrete edge.

The tests confirm that cast-in hairpin reinforcement can significantly increase the loadbearing capacity. Moreover, the post peak load behaviour becomes much more ductile. The most significant increase of up to 200% is observed if the edge distance is very small (Fig. 9). Due to the reinforcement, the first cracks appear at about 50% higher loads than in unreinforced concrete.
Fig. 9 Load-displacement curves for specimens without and with hairpin reinforcement

The parameter most strongly influencing the peak load is the distance of the hairpin reinforcement from the concrete surface ($c_{so}$), especially with small reinforcement diameters and with anchors set very close to the edge. While the peak load can be increased by a factor of 3 with a concrete cover of 10mm for 12mm diameter hairpins, the influence of the same reinforcement with a cover of 30mm is scarcely observable.

The tests also show that the peak load increases with the diameter of the cast-in reinforcement. However, this increase is not directly proportional to the increase of the steel area. The effect of the hairpin reinforcement is independent of the exact position of the anchor between its legs.

5.3 Theoretical considerations and prediction of failure load

Anchors subjected to shear loads experience bending, shear and with increasing lateral displacement also axial stresses. Small loads are directly introduced from the shaft into the surface concrete. The load-displacement curve is steep and shows a linear-elastic behaviour. The transmission of the shear force from the anchor bolt to the concrete takes place within a depth measured from the surface of 1 to 2 times the dowel diameter [9,10]. A hanger reinforcement makes sense only in this area, because the resulting compression strut then finds a support. Due to the locally high pressure in front of the bolt, the surface concrete plasticizes under increasing load and flexural stresses are generated in the anchor shaft. For anchors situated near the edge this finally leads to the formation of a concrete failure cone.

Some of the tests have also been analysed by means of a 3-dimensional finite element modelling using non-linear material laws and a smeared crack approach. The computer simulation confirmed that the failure is due to cracking and crushing of the concrete in front of the anchor. Provided that the anchor has sufficient embedment length, this leads to the development of a plastic hinge. The hinge is closer to the surface if the hanger reinforcement has less concrete cover and thus significantly reduces the lever arm of the anchor. Moreover the calculations demonstrated that the hairpins remain elastic until the peak load is reached.

The increase of the ultimate load is not proportional to the steel area because the
centerpoint of the support for the compression strut moves down with increasing hairpin diameter at constant concrete cover.

The results of the tests in unreinforced concrete correspond well to formula (1). The best agreement is reached with a factor $k = 1.0$, the coefficient of variation results in a rather low value of 14 %.

The cast-in reinforcement delays the formation of cracks starting from the anchor as well as their propagation. The additional energy required for crack growth corresponds to the possible increase of the load bearing capacity. Therefore, it is best represented by an additional term to equation (1). The effectiveness of the hanger reinforcement is about inversely proportional to the concrete cover and decreases with increasing distance between the anchor and the hairpin reinforcement bend. Moreover, the anchor stiffness, characterized by dowel diameter and embedment depth, has an influence on the loadbearing behaviour.

The combination of all relevant parameters with respect to their effects leads to the following approach for the prediction of the failure load:

$$V_{um} = V_{um,c} + \kappa \cdot A_{s,h} \cdot f_{y,h} \cdot \left(1 - f_1(l_{proj}, c_1, h_{ef})\right) \left(\frac{d_{nom}}{k_{1} \cdot c_{so} + d_s}\right)^{k_2} < V_{um,s} \tag{2}$$

with:

$\kappa$ .... effectiveness factor taking into account that the concrete will crush before the ultimate capacity of the hairpin is reached

$k_{1,2}$ ... constants

$f_1$....... empirical function

$d_s$ ..... nominal diameter of hairpin reinforcement

$h_{ef}$ .... anchorage length [5]

$d_{nom}$... outside diameter of anchor

$l_{proj}$ .... projective length: can be approximated in terms of the edge distance and the concrete cover ahead of the hairpin (Fig. 10):

$$l_{proj} \approx 1.7 \cdot (c_1 - c_{so}) \leq e$$

$A_{s,h}$.... total cross section of both hairpin legs

$f_{y,h}$..... yield strength of hairpin steel

Steel failure due to a combination of shear and bending of the anchor shaft represents an upper bound on the shear capacity: $V_{um,s} = \alpha \cdot A_s \cdot f_y$ with $\alpha \approx 0.6 - 0.7$ [5]. ($A_s$ = cross section and $f_y$ = yield strength of anchor)

The formation of the concrete cone is considered as the failure criterion. This generally corresponds to the first peak load or the first horizontal branch of the load displacement curve. The systematic variation of the different parameters and the subsequent statistical evaluation finally leads to the following form of equation (2):

$$V_{um} = V_{um,c} + 0.4 \cdot A_{s,h} \cdot f_{y,h} \cdot \left(1 - 0.5 \cdot \frac{l_{proj}^{0.5} \cdot c_1^{0.5} \cdot h_{ef}}{c_{so}}\right) \left(\frac{d_{nom}}{1.2 \cdot c_{so} + d_s}\right)^{1.5} \tag{3}$$
Fig. 11 shows the comparison of predicted and observed failure loads for undercut anchors type HSL in function of the edge distance $c_1$. Considering all specimens with introduced hairpin reinforcement except for the 4 tests with early shear failure of the dowels the average ratio of actual to predicted failure load is 1.05. With a coefficient of variation of 19% and a factor of correlation of 89%, the scatter zone is within acceptable limits (Fig. 12).

Fig. 11 Predicted (calculated) and observed failure loads in function of $c_1$

Fig. 12 Comparison of calculated and experimental results
5.4 Design formula and recommendations for execution

For practical design purposes the transition from mean ultimate loads to characteristic values (fractiles) is required. The characteristic resistance is derived as the 5%-fractile of the mean value of strength with a confidence level of 90%. Admitting a log-normal distribution, the statistical evaluation of the tests conducted in reinforced concrete yields a global reduction factor $\psi = 0.72$. This corresponds to the factor of $\psi = 0.7$ proposed by Eligehausen in [4] for unreinforced concrete.

The material properties are also considered by their lower fractile; this additional safety will not be used in the design formula.

The design resistance is derived in accordance with the safety concept of MC 90 [3] and Eurocode 2 by applying appropriate partial safety factors.

In unreinforced, cracked concrete, the design resistance is the mean ultimate load multiplied by $\psi = 0.7$ for the statistical evaluation of the 5%-fractile [4], multiplied again by 0.7 accounting for the effect of cracks [4, 5] and divided by the partial safety factor $\gamma_{Mc}$ [6]:

$$V_{Rd,c} = \frac{1}{\gamma_{Mc}} \cdot 0.5 \cdot \frac{d_{nom}^{0.5} \cdot f_{ck}^{0.5}}{\left(\frac{h_{ef}}{d_{nom}}\right)^{0.2} \cdot c_{1}^{1.5}} \quad (\text{units: [N, mm]}) \quad (4)$$

with:
- $f_{ck}$ ...... characteristic cylinder compressive strength of concrete
- $\gamma_{Mc} = \gamma_1 \cdot \gamma_2 = 1.8 \cdot 1.2 = 2.2$ [6].

According to equation (3) the shear load capacity with cast-in hairpin reinforcement becomes:

$$V_{Rd} = V_{Rd,c} + \frac{1}{\gamma_{Ms}} \cdot 0.3 \cdot A_{sh,b} \cdot f_{yk,h} \cdot \left(1 - 0.5 \cdot \left(\frac{c_{proj}}{c_{1}}\right)^{0.5} \cdot \left(\frac{c_{2}}{c_{1}}\right)^{0.5} \cdot \left(\frac{d_{nom}}{1.2 \cdot c_{so} + d_{k}}\right)^{1.5}ight) \quad (5)$$

(valid for $f_{yk,h} \leq 600$ N/mm², $f_{ck} \leq 40$ N/mm², $4 \cdot d_{nom} \leq h_{ef} \leq 8 \cdot d_{nom}$, $d_{nom} \leq 25$ mm)

with:
- $\gamma_{Ms} = \gamma_1 \cdot \gamma_2 \cdot \gamma_3 = 1.15 \cdot 1.5 \cdot 1.2 = 2.1$
- $\gamma_1$ .... partial safety factor for steel in tension
- $\gamma_2$ .... partial safety factor taking into account deviations in the height position of the reinforcement and uncertainties due to anchor installation
- $\gamma_3$ .... partial safety factor accounting for scatter of failure loads and model uncertainties

Eq. (5) is valid for single anchors and sufficient thickness of the concrete member ($h > 1.5 \cdot c_1$) as well as an edge distance $c_2$ perpendicular to the direction of the shear load measured from the axis of the anchor $c_2 > 1.5 \cdot c_1$.

For execution special attention should be paid to the following aspects:

The hairpin reinforcement should be anchored outside the assumed failure cone and consist of ribbed reinforcing bars with a diameter not larger than 16 mm. It should be
inclined with respect to the concrete cover. Thus the concrete cover of the hairpin legs is bigger than that of the bend, which improves the anchorage. Corrosion generally should not be relevant, since the baseplate of the fixed part is typically grouted over the bend of the hairpin reinforcement. If not, the reinforcement must be protected by galvanization, special coatings or the use of stainless steel. Sufficient bending radius will compensate for the positioning tolerances of the hairpin reinforcement and thus make sure that the anchors will be positioned within the hairpin.

6. Summary

After reviewing some alternative methods to most effectively utilize the concrete edge strength, the reader has been presented with a formula to design anchors under shear loads when hairpin reinforcement is present in the concrete. This formula is compatible with modern design concepts and has been adapted to Eurocode 2. Moreover it has been shown, that global design concepts do not always clearly represent the complex situation near a concrete edge. Therefore it is strongly recommended to use engineering judgement and well based design concepts in the planning of safety relevant fixings near a concrete edge.

7. References