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Shear Design of Concrete Members without Shear Reinforcement - A Solved Problem?

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Abstract

The design of concrete members for shear without stirrups has become a major issue worldwide especially for bridge decks as the shear capacity according to the ‘new’ regulations like the Eurocode often gives significant smaller values than the one predicted by former codes. Therefore nowadays stirrups are required in bridge decks. In addition the safety of existing structures mainly built without shear reinforcement has been brought into focus. The lack of the available design models will be demonstrated by comparing the results of various codes. The inaccuracy of the EC2 approach is checked by means of a shear database. Experimental as well as numerical studies revealed that the shear capacity of haunched beams is different from members with constant height. It is questionable whether this behavior is caused by the vertical component of the inclined compression chord V_{cc} .

Keywords: shear design, concrete bridge decks, shear capacity of haunched beams

1. Introduction

Even though structural engineers and researchers have dealt with the question of shear behaviour of reinforced concrete members without shear reinforcement for more than 100 years, there is still no obvious and consistent mechanical model in use. Nearly all design regulations and codes are based on empirical based equations.

With the introduction of the Eurocodes and the German DIN 1045-1, a serious problem came up. In the past deck slabs of RC hollow box or T-beam bridges could be designed without shear reinforcement. Nowadays, however, stirrups near the webs or a significant increase of the slab thickness are required

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(Rombach 2008). Hence the issue of the practicability as well as the issue of cost effectiveness (high effort for placing the rebars) exists. Furthermore there is the problem with already existent bridges that cannot be verified by the new codes.

This paper contains four parts. First the various shear-load transfer mechanisms are briefly illustrated. Then the differences in calculating the shear capacity of RC elements as a result of the different weighting of the explained mechanisms are demonstrated. In part three the deficits of the EC2 approach are presented by means of a shear database. The fourth part deals with the shear design of haunched beams without web reinforcement. The results of an extensive experimental program and nonlinear finite element analysis are presented.

2. Shear-Load Bearing Mechanisms

While the hypothesis that plane sections remain plane (Bernoulli Hypothesis) is the international accepted base for the flexural analysis, there are different models for the description of the shear load bearing behaviour of RC elements without shear reinforcement. For example: Modified Compression Field Theory (Vecchio & Collins 1986), Critical Shear Crack Theory (Muttoni 2008) or Tooth Models (Kani 1964, Reineck 1991). Although there is agreement on the mechanisms that participate in carrying shear loads over the cross section (fig. 1), their significance on the ultimate shear failure load V_u after exceeding the tensile strength of the concrete is treated differently in the various models.

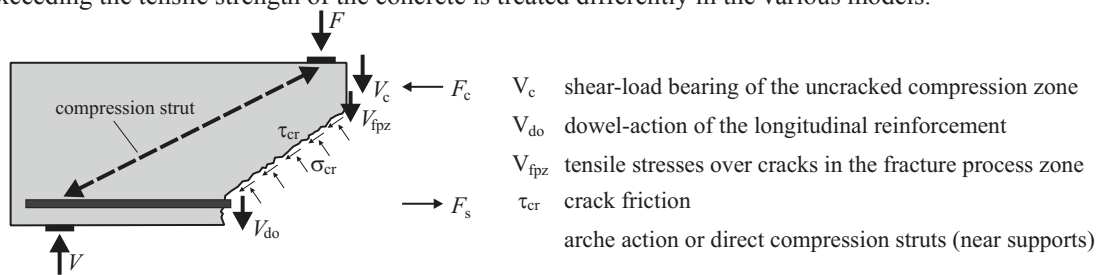


Figure 1: Shear-load transfer mechanisms

3. Comparison OF the shear- capacity acc. to different codes

As written above, there is no internationally accepted mechanical model for RC elements without web reinforcement. Therefore the various national design codes contain different approaches in considering the load transfer mechanisms. This results in significant different design loads for identical members. It should be noted that all equations are not dimensionless. This will be demonstrated by means of a simple RC beam. The design shear capacity V_{Rd} of a rectangular cross-section with a concrete compressive strength of $f_c' \approx f_{ck} = 25$ MPa and 50 MPa resp. without web reinforcement is listed in Table 1 according to ACI, BS 8110-1 and EC2.

The shear capacity according to BS 8110-1 is 24 % and 11 % respectively higher than ACI 318. The American code is conservative compared to EC2 and BS 8110-1 for a beam with that depth.

The beam was tested up to failure as a reference for the haunched members, shown in chapter 5. The maximum load was 151 kN and 158 kN respectively, which is more than 2 times the design value ($f_{ck} \approx 50$ MPa). This demonstrates the high inaccuracies of the available design models.

Table 1: Shear capacity of a beam without web reinforcement acc. to ACI 318, BS8110-1 and EC2

ACI 318 $\phi_c = 0,75$	$V_{Rd} = 0.17 \cdot \phi_c \cdot \sqrt{f'_c} \cdot b_w d$	$V_{Rd} = 38 / 54$ kN	
BS 8110-1 $\gamma_m = 1.25$	$V_{Rd} = \frac{0.79}{\gamma_m} (100 \cdot \rho_1)^{1/3} \left(\frac{400}{d} \right)^{1/4} \left(\frac{f_{ck}}{25} \right)^{1/4} b_w d$	$V_{Rd} = 47 / 60$ kN	
EC2 $\gamma_c = 1.5, k = 1.8$	$V_{Rd} = \frac{0.18}{\gamma_c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{1/3} \cdot b_w d$	$V_{Rd} = 45 / 56$ kN	

4. design Models Based on Statistical Approaches - EUROCODE 2

Due to a lack of a consistent mechanical model the shear capacity of a non-prestressed RC element without web reinforcement is based on empirical or semi-empirical deduced test data (Rombach 2009, Latte 2010). It is obvious, that leaving the range of the database constrains the validity of the derived formula. This problem will be demonstrated by means of a shear database published by Reineck & Kuchma et al. (2003). First the transferability to real structures like bridge decks seems questionable as a lot of the tests were conducted with an unusable high reinforcement ratio $\rho_1 > 1\%$ to avoid flexural failure. Bridge deck slabs, however, normally have reinforcement ratios $\rho_1 < 1\%$. Furthermore the database includes a lot of tests with $a/d < 2.9$, where strut-and-tie models are applicable, as well as concrete strength $f_{ck} > 50$ MPa or $d > 550$ mm which is untypical for slabs.

Figure 2 shows that there is a huge difference between the calculated ultimate shear capacity $1.5 \cdot V_{EC2}$ (mean value) acc. to EC2 and the one taken from the database by Reineck & Kuchma V_{test} . This applies especially for a relevant range of reinforcement ratios $\rho_1 \leq 1\%$ and $a/d < 4$. The great scatter demonstrates that the design equation acc. to EC2 does not include a lot of relevant effects. The same issue is mentioned by Latte (2010) for other design models.

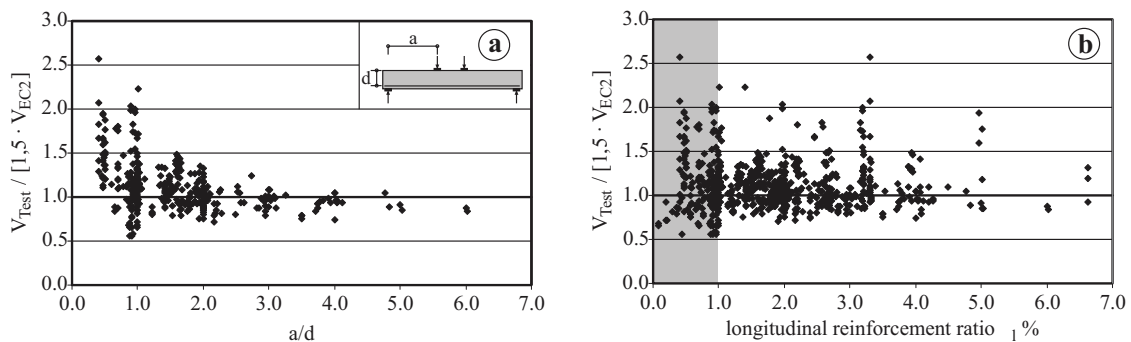


Figure 2: Comparison of the shear capacity between 374 tests out of the database by Reineck et al. (2003) and calculated values. (a) depending on a/d (b) depending on ρ_1

5. Shear design of haunched beams

The thickness of concrete bridge decks in transverse direction is usually greater near the webs than at the tips. Most codes do not offer any instructions for designing these structures with inclined compression

zones except the German DIN and the ACI code. Only a very limited number of investigations were conducted worldwide regarding the shear capacity of haunched beams.

In section 11.1.1.2 of ACI 318–05, the term “effects of inclined flexural compression” is used to explain the different stress distribution of haunched beams compared with that of constant depth beams. This stress distribution results in a shear resistance force V_{cc} as a vertical component of the inclined flexural stresses. On the other hand, the German code DIN 1045-01 explains the shear resistance mechanism of haunched beams in details (Fig. 3). The shear design formula is as follows:

$$V_{Ed} = V_{Ed0} - V_{ccd} - V_{td} - V_{pd} \leq V_{Rd} \quad (1)$$

Where: V_{Ed} : Design value of shear force.
 V_{Ed0} : Design value of shear force due to dead loads and live loads.
 V_{ccd} : Design value of shear resistance component of compression zone.
 V_{td} : Design value of shear resistance of the force in the inclined tension reinforcements.
 V_{pd} : Design value of shear resistance component of prestressed force.
 V_{Rd} : Design value of shear resistance

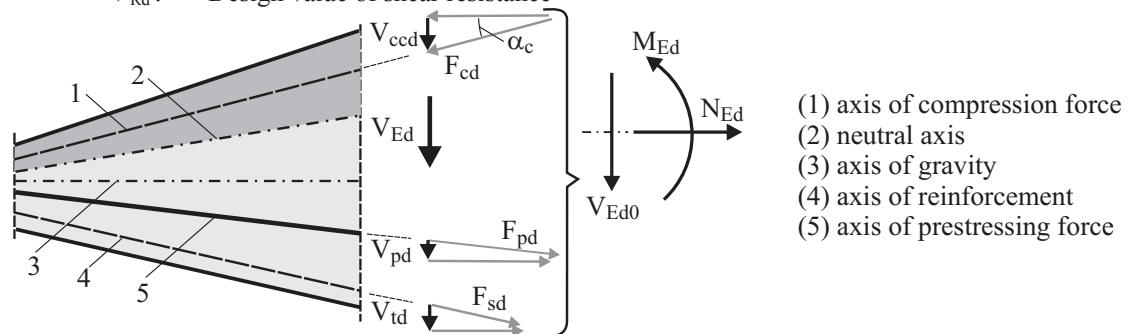


Figure 3: Shear components for variable-depth concrete members

Please note that the shear force V_{Ed} is not perpendicular to the axis of gravity. In case that there is no prestressing or normal force and the longitudinal tension reinforcement is not inclined ($V_{pd} = V_{td} = 0$), the shear design formula becomes:

$$V_{Ed} = V_{Ed0} - V_{ccd} \leq V_{Rd} \quad \text{with:} \quad V_{ccd} = \frac{M_{Ed}}{z} \tan \alpha \approx \frac{M_{Ed}}{0,9d} \tan \alpha \quad (2, 3)$$

It is questionable whether the model is correct and whether V_{ccd} reduces the design shear force V_{Ed} or not. An extensive experimental program of 18 concrete test beams without stirrups having different inclinations of α between 0° - 10° (Fig. 4) was conducted to investigate the behaviour of haunched RC members. Two identical beams were always tested for statistical reasons. The main results are shown in Table 2 and Figure 8. Further details are given in Rombach et al. (2009).

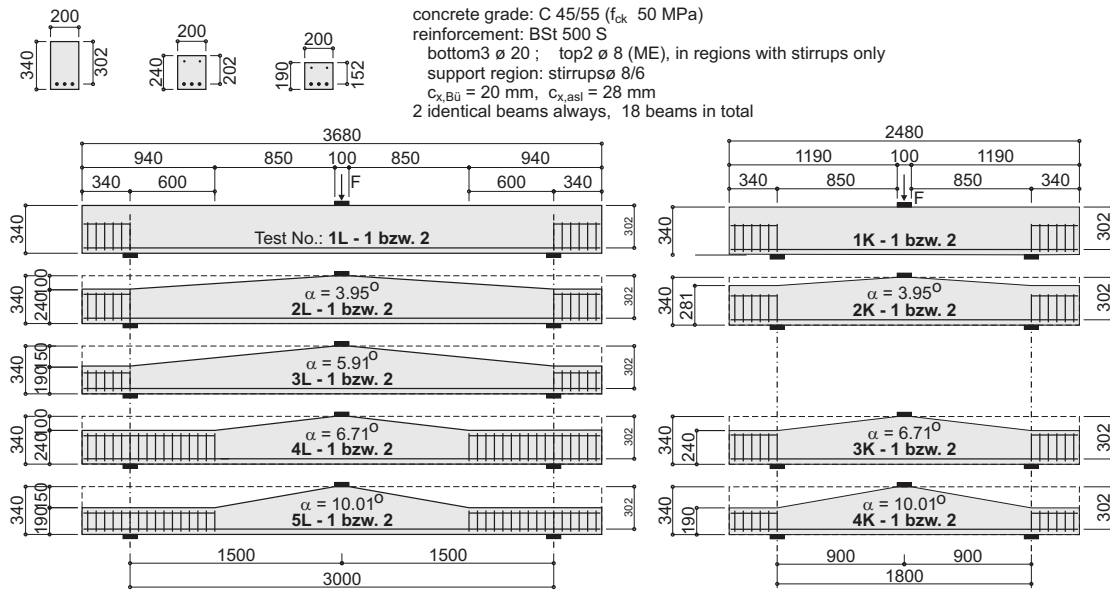


Figure 4: Test beams with $a/d = 5$ and $a/d = 3$

Beam	F [kN] without V_{ccd}	F [kN] with V_{ccd}	F_{Test} [kN]	Failure
1L-1	166	166	151	Shear
1L-2	167	167	158	Shear
2L-1	143	158	150	Shear
2L-2	143	158	149	Shear
3L-1	117	137	133	Shear
3L-2	117	138	139	Shear
4L-1	Flexural failure		207	Moment
4L-2	Flexural failure		207	Moment
5L-1	Flexural failure		206	Moment
5L-2	Flexural failure		207	Moment
1K-1	173	173	151	Shear
1K-2	173	173	139	Shear
2K-1	163	180	167	Shear
2K-2	163	180	170	Shear
3K-1	155	184	159	Shear
3K-2	155	184	160	Shear
4K-1	134	178	170	Shear
4K-2	134	178	168	Shear

Table 2: Shear capacity of the test beams

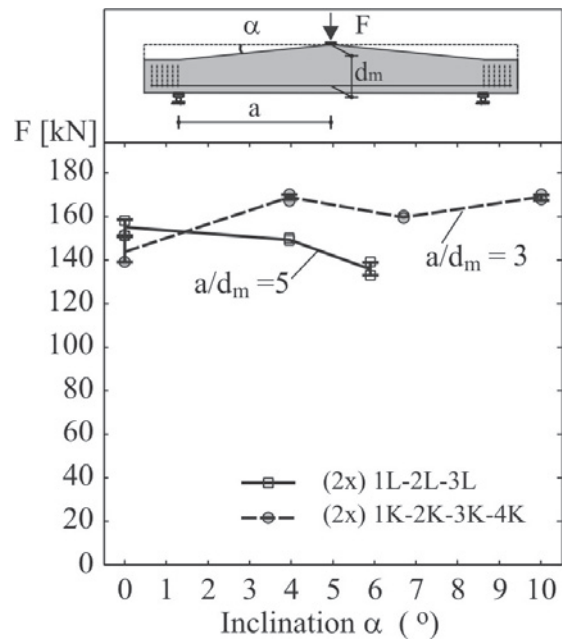


Figure 5: Shear capacity versus haunch slope α

Fig. 5 shows that the shear capacity of beams with $a/d_m \approx 5$ decreases with increasing the inclination angle α which is contrary to eq. 2. The failure load V_u is nearly constant for members with $a/d_m \approx 3$. It is questionable, whether V_{ccd} can cover this behaviour. Please note that the design loads are not shown as the required safety coefficients are still under discussion.

All 18 test beams were modelled in ABAQUS 6.9 Explicit to get a better understanding of the crack propagation and the failure mechanism. 8 noded brick elements and a damaged plasticity model were used to model the concrete. The reinforcement was simulated by bar elements which were rigid fixed to the nodes of the concrete elements. From world-wide experience with nonlinear Finite Element Analysis it could not be expected, that the main failure cracks are identical with the tests. But over all the numerical and test results show good agreements (see Fig. 6 - 9).

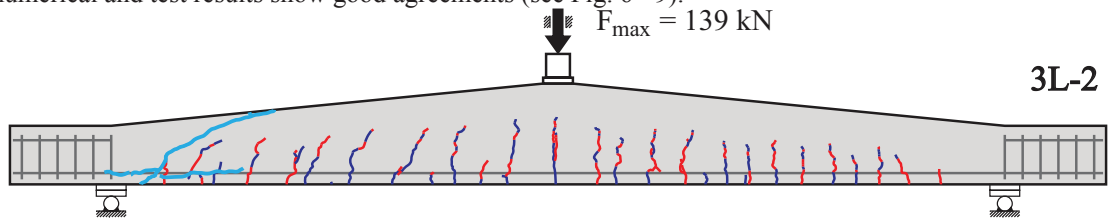


Figure 6: The propagation of cracks from test at 139 KN

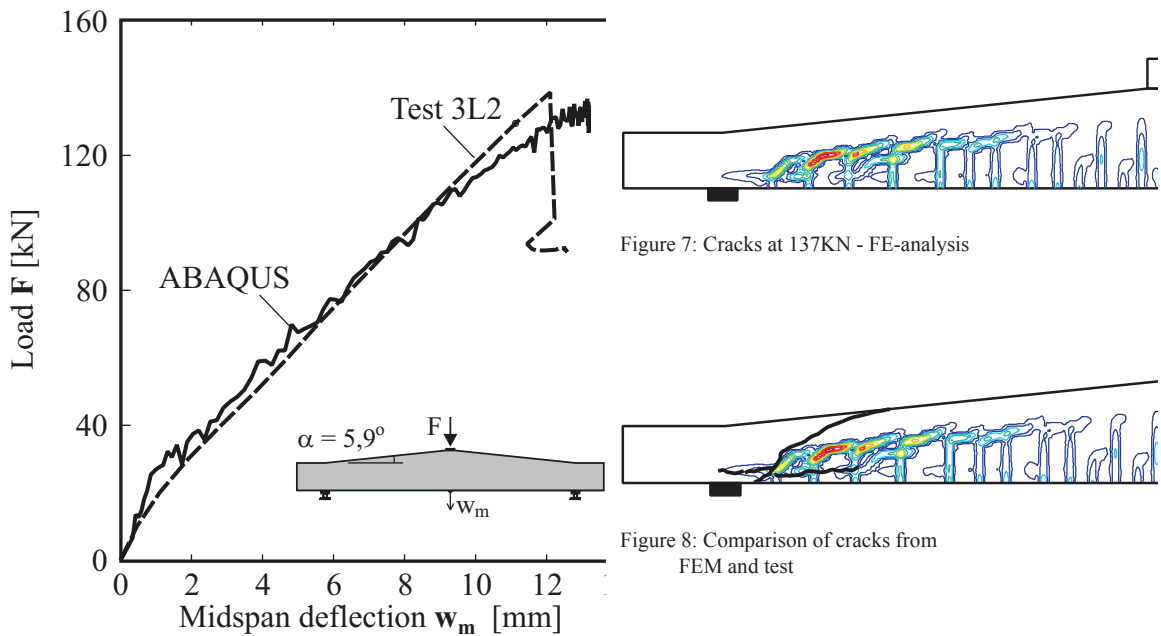


Figure 9: Load-displacement curve from tests and FE-analysis

The nonlinear FEM is strongly expected to give more understanding on the shear failure mechanism and further to develop a more reasonable shear design model of the concrete structures without stirrups. The research is ongoing.

6. CONCLUSIONS

There still exists no consistent mechanical model for design of RC members without shear reinforcement. The evaluation of the shear database shows a great difference between test results and analytical models. This indicates that the available approaches for shear design of concrete members without stirrups have still a great uncertainty. Tests and FE-analysis revealed that haunched beams show a different crack pattern than RC members with constant depth. From a mechanical point of view it is doubtful whether this different behaviour can be modeled by the vertical component of an inclined compression strut V_{cc} .

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