Shear resistance of bridge decks without shear reinforcement

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ABSTRACT: This paper presents experimental and numerical investigations of a research project, carried out to examine whether bridge deck slabs under concentrated wheel loads exhibit reserves of shear capacity, which have been neglected in the current Eurocode design provisions so far. Tests conducted on large-scale cantilever slabs under concentrated loads demonstrated significant load redistributions after the formation of bending and diagonal shear cracks. The evaluation of the test results showed, that the current design formula leads to rather conservative values of shear capacity for bridge deck slabs when combined with a shear force distribution according to linear elastic slab analysis. A beneficial effect of a tapered slab bottom on the shear capacity, as implied by most codes, could not be verified with these tests.

1 INTRODUCTION

The design of concrete bridge deck slabs is of major concern since the introduction of the Eurocodes and the German DIN Fachbericht 102 (DIN-Fb 102). The calculated shear capacity of slabs without stirrups and staggered reinforcement according to the new design standards, as illustrated in Figure 1, is often considerably smaller compared to the former regulations i.e. DIN 1045:88. Thus, more massive constructions or shear reinforcements are now required in bridge deck slabs, whereas this was formerly not the case. This raises the question whether there is a lack of safety for existing deck slabs which have been built mainly without shear reinforcement, or whether deck slabs under concentrated wheel loads exhibit reserves of shear capacity which have been neglected in the current design provisions.

Determining the shear capacity of reinforced concrete structures without shear reinforcement is a classical problem of slabs. Despite this the majority of research and experiments has so far concentrated on simply supported beams or one-way spanning slab strips \( b/d < 4 \) with loads applied over the full width of the specimens. Over the years an extensive database containing three or four point bending tests from various researchers was established. The design equations of the EC2 and DIN-Fb 102 are based on an...
slabs, the ratio of bending moment to shear ratio for a beam and a slab with concentrated load.

Figure 2. Comparison between the ratio \( a/d \) and the bending moment to shear ratio for a beam and a slab with concentrated load.

An empirical evaluation of such a database. A similar database containing 374 well-documented tests was published by Reineck & Kuchma et al. (2003). The significance of many of those beam tests is, however, limited for bridge deck slabs. Unrealistic high longitudinal reinforcement ratios were chosen for many of the beams in the database to avoid bending failures prior to the formation of critical shear cracks. In contrast to this, the reinforcement ratio for bridge decks slabs with \( d > 250 \text{ mm} \) and a reasonable maximum amount of reinforcement of \( a_d < 25 \text{ cm}^2/\text{m} \) will typically be below \( \rho_l = 1\% \). From the 374 beam tests only 58 fulfill this constraint. Only 27 relevant test remain, if the focus is laid on slender beams \( (a/d \geq 2.9) \) with parameters typical for bridge deck slabs \( (d < 550 \text{ mm}) \) and \( f_{ck} < 50 \text{ MPa} \).

Furthermore, the ratio of bending moment to shear force \( m/v \) for beams is connected to the distance \( a \) between load and support and is directly proportional to the geometrical ratio \( a/d \). This condition does not apply for bridge deck slabs under concentrated loads, where \( m/v \) depends on the distribution of shear and bending moments in the support region. Due to the larger distribution of bending moments compared to shear forces in slabs, the ratio \( m_{\text{max}}/v_{\text{max}} \) of a slab is always smaller than that of a beam as pointed out in Figure 2.

A critical assessment of the published experimental data in literature, thus, leads to the conclusion, that the conducted experimental studies so far did not contain the most relevant parameters for the design of bridge deck slabs under concentrated wheel loads. Therefore, experimental and theoretical investigations of the shear capacity of reinforced concrete bridge deck slabs without shear reinforcement under concentrated 'wheel' loads financed by the German Federal Highway Research Institute (BASt) have been carried out at the Institute of Concrete Structures at the Hamburg University of Technology.


data available in Laute (2003).

The experimental program consisted of 12 load tests on 4 large-scale test specimens of reinforced concrete bridge deck slabs measuring 2.40 m in width and ranging from 5.68 m to 6.58 m in length with an overall slab thickness from 200 mm to 300 mm (Fig. 4). Each test specimen comprised two cantilever slabs of 1.65 m and a centre slab supported on two web beams. This offered the possibility to conduct two cantilever slab tests (V1 and V2) and one centre slab test (V3) with each specimen. While the centre slabs of tests V1 and the cantilever slabs of tests V3 did not contain any shear reinforcement the cantilever slabs of tests V2 were equipped with stirrups. This paper will focuses only on the cantilever slab tests. Further information is available in Laute (2008).

The cantilever slabs of specimen VK1 were designed to represent a full-scale cantilever of a concrete box girder bridge with 2.60 m span using traffic loads as prescribed by Eurocode 1. Because the design shear capacity according to Eurocode 2 is independent of the rate of bending the governing load case is found by applying the wheel loads in a distance of 2.5·d towards the support. To account for uniformly distributed traffic loads and a reduction of the cantilever slab length to feasible dimensions for the test apparatus a constant line load \( f_q \) was applied over the full width at the tip of the cantilevers. The specimens were tested in a 1 MN test frame as shown in Figure 3. The line load \( f_q \) was applied with a secondary hydraulic jack and kept constant during the whole duration of each test. After applying the line load the slab was loaded by a concentrated load \( F_0 \) applied on a square loading plate \( 400 \times 400 \times 100 \text{ mm} \) until failure.

To study the influence of a tapered slab bottom on the shear capacity two test specimen VK2 and VK3 with identical cross-sections at the web beams were tested. While VK2 had a constant slab thickness of 250 mm, VK3 was laid out with a slope of 1:15 reducing the slab thickness to 140 mm at the outer edge. All other parameters were kept constant for the two tests. Figure 4 shows the geometry, dimensions and reinforcement layout of each test specimen.

The load arrangements and concrete properties at the time of testing are listed in Table 1. The reinforcing steel was of grade BSt 500 S with yield strength of 550 MPa for the Ø16 longitudinal reinforcement in the top layer. A normal ready-mix concrete of class C 30/37 with a maximum aggregate size of \( d_{ag} = 16 \text{ mm} \) was used. The concrete cover was chosen to 45 mm as typical for bridges for VK1 and 25 mm for VK2 to VK4 respectively.

2 TESTS OF REINFORCED CONCRETE DECK SLABS UNDER CONCENTRATED LOADS

2.1 Test specimens and load arrangement

The experimental program consisted of 12 load tests on 4 large-scale test specimens of reinforced concrete bridge deck slabs measuring 2.40 m in width and ranging from 5.68 m to 6.58 m in length with an overall slab thickness from 200 mm to 300 mm (Fig. 4). Each test specimen comprised two cantilever slabs of 1.65 m and a centre slab supported on two web beams. This offered the possibility to conduct two cantilever slab tests (V1 and V2) and one centre slab test (V3) with each specimen. While the centre slabs of tests V1 and the cantilever slabs of tests V3 did not contain any shear reinforcement the cantilever slabs of tests V2 were equipped with stirrups. This paper will focuses only on the cantilever slab tests. Further information is available in Laute (2008).

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Figure 3. Test setup and load arrangement of the cantilever test for test specimen VK1.

Figure 4. Dimensions and reinforcement layout of the test specimen VK1 to VK4.
2.2 Test results

All slab tests V1 of cantilever without shear reinforcement showed a brittle shear failure with no yielding of the longitudinal reinforcement. Only small deformations and fine cracks were visible at the top surfaces before shear failure. In contrast to this the stirrup reinforcement of the cantilever slabs V2 ensured a ductile bending failure with large deformations and plastic strains in the longitudinal reinforcement accompanied by wide crack openings over the full width of the slab. The load deflection curves for all cantilever tests are shown in Figure 5 a & b. The measured failure loads are summarized in Table 2. The behaviour of the test specimens will be explained for VK1 only, as all other specimens exhibited a similar failure mechanism.

The crack pattern of specimen VK1 after the two cantilever slab tests is shown in Figure 6. The cracks on the side faces of the slabs propagated almost vertically. The characteristic diagonal shear cracks on the outer slab faces of test V1 were only visible after a pronounced drop of load \( F_Q \). The inclined crack is joined by cracks propagating along the reinforcement layers resulting from dowel action. The bending shear crack propagated along the reinforcement in the bottom layer towards the supporting web beam and cut through the compression zone with a straight crack over the whole 2.40 m width of the slab.

Table 1. Concrete strength at the time of testing and load arrangement.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( \rho )</th>
<th>( f_{c,ed} )</th>
<th>( f_{c,op} )</th>
<th>( f_q )</th>
<th>( e )</th>
<th>( a** )</th>
<th>( \frac{a}{d} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>VK1</td>
<td>0.81</td>
<td>35.0</td>
<td>2.85</td>
<td>32.1</td>
<td>1.5</td>
<td>0.71</td>
<td>2.88</td>
</tr>
<tr>
<td>VK2</td>
<td>1.16</td>
<td>46.0</td>
<td>3.42</td>
<td>22.5</td>
<td>1.5</td>
<td>0.71</td>
<td>3.27</td>
</tr>
<tr>
<td>VK3</td>
<td>1.16</td>
<td>46.5</td>
<td>3.34</td>
<td>22.5</td>
<td>1.5</td>
<td>0.71</td>
<td>3.27</td>
</tr>
<tr>
<td>VK4</td>
<td>1.20</td>
<td>42.5</td>
<td>3.23</td>
<td>–</td>
<td>–</td>
<td>0.71</td>
<td>4.25</td>
</tr>
</tbody>
</table>

*Tests on cylinders Ø 150 mm \( h = 300 \) mm.
** Distance between load center and edge of the web beam.

Table 2. Measured and calculated failure loads.

<table>
<thead>
<tr>
<th>Test</th>
<th>Shear cracking ( F_{Q,cr} ) kN</th>
<th>Yielding ( F_{Q,y} ) kN</th>
<th>Failure ( F_{Q,u} ) kN</th>
<th>Linear elastic FE model</th>
<th>Bending</th>
<th>Yield line with effective width of 2.40 m</th>
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</thead>
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<tr>
<td>VK1V1</td>
<td>350</td>
<td>–</td>
<td>690</td>
<td>290</td>
<td>2.38</td>
<td>537</td>
</tr>
<tr>
<td>VK1V2</td>
<td>–</td>
<td>671</td>
<td>758</td>
<td>–</td>
<td>–</td>
<td>537</td>
</tr>
<tr>
<td>VK2V1</td>
<td>360</td>
<td>–</td>
<td>678</td>
<td>392</td>
<td>1.73</td>
<td>691</td>
</tr>
<tr>
<td>VK2V2</td>
<td>–</td>
<td>808</td>
<td>877</td>
<td>–</td>
<td>–</td>
<td>691</td>
</tr>
<tr>
<td>VK3V1</td>
<td>400</td>
<td>–</td>
<td>672</td>
<td>359</td>
<td>1.87</td>
<td>645</td>
</tr>
<tr>
<td>VK3V2</td>
<td>–</td>
<td>808</td>
<td>870</td>
<td>–</td>
<td>–</td>
<td>645</td>
</tr>
<tr>
<td>VK3V1</td>
<td>260</td>
<td>–</td>
<td>487</td>
<td>308</td>
<td>1.58</td>
<td>469</td>
</tr>
<tr>
<td>VK3V2</td>
<td>–</td>
<td>548</td>
<td>590</td>
<td>–</td>
<td>–</td>
<td>469</td>
</tr>
</tbody>
</table>

* Deduced from the calculated vertical strains of the measured slab thickness variation
** Beginning of yielding in the longitudinal reinforcement

Figure 5. Load–deflection curves for the measured deformation of the cantilever tips.
Measuring the deviations of the slab thickness at various points throughout the tests made it possible to detect the appearance of inclined cracks inside the slabs. Figure 7a shows the load-strain relation calculated from the measured increase of slab thickness. From these vertical strains it can be followed that the first inclined bending shear crack inside the slab emerged in the middle of the slab and crossed the locations of the measuring points W25 and W24 at load levels of $F_Q \approx 350\, \text{kN}$ and $F_Q \approx 440\, \text{kN}$ respectively. As can be observed from Figure 7b the formation of the first inclined shear bending cracks did not cause an immediate increase of the concrete strains in the compression zone. A local increase of concrete strains in the centre of the slab could not be noticed until loads above $F_Q \approx 560\, \text{kN}$. At this load level the diagonal shear crack had grown towards the compression zone in the support region and was also detected in the measuring points W27 and W28. The fully developed crack in the centre of the slab did, though, not lead to a direct loss of bearing capacity. The sudden and brittle shear failure at a load of $F_Q = 690\, \text{kN}$ did only occur after the diagonal crack propagated over a larger width of the slab.

In contrast to the test without shear reinforcement the stirrups provided in cantilever slab V2 reduced the crack width of the diagonal shear crack considerably and lead to a ductile bending failure by yielding of all reinforcements in the top layer with large crack openings and deformations. The test was stopped at a load of $F_Q \approx 785\, \text{kN}$ after considerable crushing of the concrete compression zone in the bottom was observed.

3 DISCUSSION OF TEST RESULTS

A three dimensional FE model using shell elements with linear elastic material properties taking into account the exact boundary conditions was used to calculate the sectional forces. The bending moment capacity calculated with the elastic model resulted in the load value $F_{y,\text{cal}}$ for which the onset of yielding is expected. Assuming a straight yield line over the full
width of the slab resulted in the calculated ultimate bearing capacity $F_{mu, cal}$.

The calculation of shear capacity was performed with the sectional shear forces of the linear elastic model according to the common design approach for a vertical section directly at the intersection of the slab with the web beam. Mean values of shear capacity were required to compare the calculated values to test results. To calculate mean values of the shear capacity the code equation had to be multiplied by a transformation factor which was verified by an evaluation of the shear database published by Reineck & Kuchma et al. (2003). The mean value of the shear capacity can be calculated according to Equation 1.

$$v_{Rm,ct} = 1.5 \cdot v_{Rd,ct} = 0.18 \cdot \left(100 \cdot d \cdot f_c \right)^{1/3} \cdot d$$ (1)

The shear bearing capacity predicted with the load distribution according to the linear elastic model is denoted $F_{ct, cal}$ in Table 2. Since all tests exhibited a failure over the full width of the specimens the load capacity $F_{vu, cal}$ was calculated with an effective shear force distribution over the full 2.40 m slab width. Note that for the calculation of the load capacities in Table 2 a reduction of the shear force due to an inclined compression chord was omitted for the tapered slabs of specimen VK1 and VK3.

Comparing the calculated values with the test results it can be observed that the ultimate bending capacity of the slabs $F_{mu, cal}$ could be predicted quite well. The shear capacity according to the code with elastic shear force distribution leads to rather conservative values. The calculated value $F_{ct, cal}$ captures the onset of diagonal cracking in the slab structure at loads $F_{Q,ct}$, but the crack initiation does not immediately lead to failure of the slabs. Considering load redistributions in the slab and utilising the shear capacity over the full width for the calculation of $F_{vu, cal}$ does far better capture the measured maximum load $F_{Qu}$.

It should be noted that, although the slab bottom of VK3 was tapered 1:15 under otherwise identical conditions, the cantilever slab tests V1 of VK2 and VK3 failed at the same load level with a difference of only 1%. Many design codes allow for a possible reduction of the design shear force due to an inclined compression cord $V_{cd}$ according to Equation 2 and 3. If the reduction would be taken into account the load bearing capacity of test VK3 with tapered slab bottom would be predicted to be higher than that of VK2 (see Table 3). Furthermore, taking in to account an influence of the inclined compression zone and a load distribution of shear forces over the full 2.40 m width would lead to an unsafe estimation of the bearing capacity changing the predicted failure mode to bending failure.

$$V_{Ed} = V_{Ed0} \cdot V_{ccd} \leq V_{Rd,ct}$$ (2)

$$V_{ccd} = \frac{M_{Ed} \cdot \tan \alpha}{z} \leq \frac{M_{yd}}{z} \cdot \tan \alpha$$ (3)

### 4 CONCLUSIONS

The failure loads of tests conducted on reinforced concrete slabs without shear reinforcement under concentrated loads were considerably higher than calculated according to Eurocode 2 in combination with an effective width based on an elastic FE solution. The onset of diagonal shear cracking in the slabs could, though, be predicted with this design approach. The increase of ultimate bearing capacity could mainly be related to redistributions and diffusion of concentrated loads in concrete slabs.

Furthermore, it is concluded that for an efficient design against shear failure of concrete bridge deck slabs under wheel loads the provisions should take into account the influence of bending strains on the shear capacity. Since the ratio of bending moment to shear force for slabs under concentrated loads considerably differs from that of beams, the real boundary conditions of bridge deck slabs are not modelled properly by normal beam tests. Therefore, it will be necessary to perform more tests and analyses of concrete slabs under concentrated loads.

The experiments and theoretical studies additionally indicate the need to further investigate the influence of an inclined compression chord on the shear capacity of reinforced concrete bridge deck slabs with tapered bottom edges.

### ACKNOWLEDGEMENTS

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Table 3. Comparison of failure load VK2V1 to VK3V1 calculated with shear reduction due to $V_{cd}$.

<table>
<thead>
<tr>
<th>Test</th>
<th>Linear elastic FE model</th>
<th>2.40 m effective width</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$m_{ct,cal}$ kNm/m</td>
<td>$v_{cd}$ kN/m</td>
</tr>
<tr>
<td>VK2V1</td>
<td>176</td>
<td>–</td>
</tr>
<tr>
<td>VK3V1</td>
<td>210</td>
<td>72</td>
</tr>
</tbody>
</table>
The authors would like to express their gratitude to the German Federal Highway Research Institute (BASt) for their support and financing of this research.

REFERENCES


