THE FATIGUE BEHAVIOUR OF THE SHEAR CONNECTION IN THE HOGGING REGION OF STEEL AND CONCRETE COMPOSITE CONTINUOUS BEAMS UNDER REALISTIC LOADING

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Abstract
At Kaiserslautern University four large-scale tests on two span continuous composite beams have been performed [1,2]. The tests were aimed at investigating the fatigue behaviour of the shear connection in the hogging region under approximately realistic conditions. Therefore a new experimental simulation of the crossing of vehicles over bridges has been developed using two alternating hydraulic cylinders. It is shown that the studs’ flexibility and the cyclic cracking of the concrete directly adjacent to the stud influence the fatigue behaviour significantly. Both cannot be taken into account using the basis of design provided by the European standardization [3] or by the new national codes [4]. It turned out that some parts of the structure do not show elastic behaviour under service loads. Considerable shear force redistribution takes place along the connection and it seems probable that the fatigue phenomenon cannot be described in a sufficiently exact manner with the classical load-life fatigue approach.

1. Introduction

Headed studs are normally used as shear connectors in steel and concrete composite structures. In bridge constructions repeated non-static loading occurs originated by lorries or trains. Therefore special attention has to be paid to the fatigue phenomenon. Due to the harmonization of the European Codes a standardized proof of the fatigue strength for the shear connectors is required [3] and partly simplifying national codes, for e.g. [5], will be replaced. In [3] the classical load life approach is used, like it is the same as the high cycle fatigue concept for steel elements. The concept is based on nominal stresses and uses a fatigue strength curve, the so called Wöhler curve, to determine the expected number of load cycles for the connectors. In general, the load life approach is limited to the elastic behaviour of all structural components under non-static (service-)loads. In that case, the fatigue resistance mainly
depends on the amplitude of the load. Because of that in [3] the longitudinal shear per unit length has to be calculated by elastic theory, a complete shear connection has to be provided and the size and spacing of the studs should be such that the maximum shear force does not exceed 0.6*P_{RK}. Furthermore a full interaction between structural steel, reinforcement and concrete is assumed, so that the redistributions of shear forces along the connection can be neglected.

The characteristic value of the fatigue strength curve (eq. 1) [6] and the derived design value of the Wöhler curve (eq. 2) [3] are drawn up from a statistical re-evaluation of international data from stress controlled push-out tests.

\[
\log N = 25.340 - 9.2 \cdot \log \tau_{\text{TR}} \quad \text{"Characteristic fatigue strength curve"} \quad [6] \quad (1)
\]

\[
\log N = 22.123 - 8.0 \cdot \log \tau_{\text{TR}} \quad \text{“Design fatigue strength curve”} \quad [3] \quad (2)
\]

Because of the very low inclination of the fatigue strength curves the lifetime prediction is very sensible to inaccurately calculated shear forces.

In the hogging region of composite beams it is very likely that concrete cracking occurs. This can be taken into account according to [3] by considering the tension stiffening effects. Because of the concrete cracking the neutral axis descends into the web of the steel profile and causes considerable amplitudes of normal tensile stresses in the upper flange. Fatigue cracks in the flange may develop when shear connectors are welded over the centre support and the so-called fatigue failure mode “C” can occur, which causes a global failure of the beam. For this case a functional interaction relationship is proposed in [3] according to eq. (3).

\[
\gamma_{FF} \left( \frac{\Delta \sigma_{te}}{\sigma_{te}} + \frac{\Delta \tau_{c,e}}{\tau_{c,e}} \right) \leq 1.3
\]

In reality even for low load levels significant shear force redistributions along the connection take place, which are not constant over the course of time. Besides concrete cracking in tension this is mainly due to the following two effects:

1. In principle the shear connection with headed studs is not rigid. Even for very low values of the load, the load-slip relationship is non-linear. Plastic deformations occur which are strongly developed for the first load cycle [7].

2. The concrete directly adjacent to the feet of the headed studs is subjected to very high multiaxial stresses. Therefore a progressive cyclic crushing of concrete occurs and the slip at the steel-concrete interface steadily increases. The headed studs are more and more subjected to bending, which further amplifies the redistributions caused by the cyclic concrete crushing.

As a matter of fact, after each load cycle an increment of slip is accumulated due to the progressive damage both in the concrete adjacent to the stud and in the shank of the stud [8,9]. Because of that larger areas of the shear connection are subjected to alternating cyclic loads, which cannot be estimated in a sufficiently exact way using linear calculation methods. Besides that these alternating amplitudes cannot be judged regarding the fatigue damage caused because the fatigue strength curve is derived from push-out tests subjected to unidirectional cyclic loads.
2. Large scale tests on two span continuous composite beams

In the past, the most common method to perform beam tests where the fatigue behaviour of the shear connection in the hogging region has been investigated was to carry out tests on single span beams with an inverted test set-up. This means that the specimens were turned upside down and therefore the hogging region was isolated from the whole structure. Besides that the tests were usually carried out as single range tests.

In literature, for e.g. [10], other test set-ups can also be found but most of them have the single range loading in common. Fig. 1 shows the test set-up chosen at Kaiserslautern University.

![Figure 1 Test set-up](image)

Within the scope of two research projects at Kaiserslautern University [1,2] tests on two span continuous composite beams have been performed. The simulation of the crossing of vehicles was approximated using two alternating hydraulic cylinders which were synchronized. The crossing was approximated by a “run over” in five steps. It turned out that the fatigue behaviour under more realistic loading conditions differs significantly from single range (beam-) tests. Only under more realistic conditions the various different sections along the shear connection in real structures can be modelled. Fig. 2 shows the scheme of the simulation. One load cycle took one second, which results in a test frequency of 1 Hz.
In the Constructional Engineering Laboratory at Kaiserslautern University it is not possible to carry out large-scale tests on real bridge girders. Therefore girders on a reduced scale were tested. The first two (T7_1 and T7_2) of four specimens were performed to obtain an idea of the fatigue behaviour under realistic loading. The major aim was to realize the desired, very complicated synchronisation. The third (T7_3) and fourth (T7_4) tests, which will be mainly referred to in the discussion of the test results, can be distinguished from the first two tests by an improved arrangement of shear connectors, the width of the concrete slab and the ratio of reinforcement (table 1 and 2). Those parameters were determined by a linear predesign based on elastic analysis. The cracking of the concrete and the tension stiffening effects were taken into account according to [3]. The main difference to normal design methods was, that no design values were used, but characteristic, predicted material properties (table 3). When the longitudinal shear forces are determined the cracking of the concrete has to be considered. This results in three different values for the stud spacing according to fig. 3:

- \( e_1 \) within the length \( L_1 \)
- \( e_2 \) within the length \( (L_3-L_1) \) and
- \( e_3 \) within the length \( L_4 \).

The obtained values are given in the following tables. The studs directly over the centre support are subjected to alternating cyclic loads (Load step 2 + Load step 4) whereas all the other shear connectors are theoretically subjected to unidirectional amplitudes of shear forces (Load step 1 + Load step 3). The spacing of the shear connectors was chosen so that the required boundary conditions in [3] for non-static loads are satisfied and therefore the design concept mentioned should be theoretically applicable to determine the lifetime of the shear connectors as well as for the whole structure, by means of a safe lifetime prediction.

**Figure 2** Scheme of the simulation
Figure 3 Structural System of the predesign, linear shear forces (schematic) and designations

Table 1 Dimensions and results of the linear predesign

<table>
<thead>
<tr>
<th>Test</th>
<th>( l_{\text{TOTAL}} ) [m]</th>
<th>( l_0 ) [m]</th>
<th>( F_{\text{max}} ) [kN]</th>
<th>( F_{\text{min}} ) [kN]</th>
<th>( l_1 ) [m]</th>
<th>( l_2 ) [m]</th>
<th>( l_3 ) [m]</th>
<th>( e_1 ) [cm]</th>
<th>( e_2 ) [cm]</th>
<th>( e_3 ) [cm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>T7_3</td>
<td>8,20</td>
<td>4,10</td>
<td>400</td>
<td>20</td>
<td>2,51</td>
<td>1,02</td>
<td>0,57</td>
<td>31</td>
<td>12,5</td>
<td>25</td>
</tr>
<tr>
<td>T7_4</td>
<td>8,20</td>
<td>4,10</td>
<td>400</td>
<td>20</td>
<td>2,51</td>
<td>1,02</td>
<td>0,57</td>
<td>31</td>
<td>15</td>
<td>24</td>
</tr>
</tbody>
</table>

Table 2 Cross sections and ratios of reinforcement

<table>
<thead>
<tr>
<th>Test</th>
<th>Concrete width ( b_c ) [cm]</th>
<th>Concrete depth ( h_c ) [cm]</th>
<th>Steel-profile</th>
<th>( \mu_{11} ) [%]</th>
<th>( \mu_{12} ) [%]</th>
<th>( \mu_{13} ) [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>T7_3</td>
<td>80</td>
<td>12,5</td>
<td>HE300A</td>
<td>0,63</td>
<td>1,1</td>
<td>1,1</td>
</tr>
<tr>
<td>T7_4</td>
<td>80</td>
<td>12,5</td>
<td>HE300A</td>
<td>0,63</td>
<td>1,1</td>
<td>1,1</td>
</tr>
</tbody>
</table>
Table 3 Expected and real material properties

<table>
<thead>
<tr>
<th>Test</th>
<th>$f_y$ expected [N/mm$^2$]</th>
<th>$f_y$ test [N/mm$^2$]</th>
<th>$f_{ck,cube}$ expected [N/mm$^2$]</th>
<th>$f_{ck,cube}$ test [N/mm$^2$]</th>
<th>$f_k$ expected [N/mm$^2$]</th>
<th>$f_k$ test [N/mm$^2$]</th>
<th>KD [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>T7.3</td>
<td>300</td>
<td>362</td>
<td>40</td>
<td>42,2</td>
<td>500</td>
<td>540</td>
<td>22</td>
</tr>
<tr>
<td>T7.4</td>
<td>300</td>
<td>362</td>
<td>40</td>
<td>47,4</td>
<td>500</td>
<td>561</td>
<td>22</td>
</tr>
</tbody>
</table>

3. Test results [1,2,11]

3.1. Global beam failure
All tests including T7_1 and T7_2 finally failed according to failure mode „C“. The number of load cycles achieved in comparison to the number of cycles calculated using eq. (3) is displayed in table 4. It seems to be obvious that the design concept for the global beam failure underestimates the lifetime significantly. The functional relationship regarding the interactive effects in the hogging region seems to give very conservative results. The fact that the maximum values for $\Delta \tau$ and $\Delta \sigma$ do not coincide, which is the regular case in real bridge girders, obviously influences the fatigue behaviour positively.

Table 4 Comparison of the calculated number of load cycles for failure mode „C“ ($n_{calc}$) with the real number ($n_{test}$)

<table>
<thead>
<tr>
<th>Test</th>
<th>Failure mode</th>
<th>$n_{calc}$ for type „C“</th>
<th>$n_{test}$ for type „C“</th>
<th>$n_{test}/n_{calc}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>T7/3</td>
<td>Type „C“</td>
<td>153.695</td>
<td>797.000</td>
<td>5,19</td>
</tr>
<tr>
<td>T7/4</td>
<td>Type „C“</td>
<td>185.845</td>
<td>803.000</td>
<td>4,32</td>
</tr>
</tbody>
</table>

3.2. Shear stud failure and slip development
Regarding the pure shear stud failure and neglecting interaction effects, the four test specimens showed some similar results. The concrete slab was opened after the beams finally failed. It turned out that in all beam tests large areas of the shear connection had already failed due to fatigue effects.
In beam tests it is quite difficult to determine the moment of stud failure exactly. In literature [7,10] there are some different proposals how to measure the moment of failure or even the moment where a fatigue crack starts to develop. Most of these proposals are based on specific applications of strain gauges or ultrasound scans. These methods were also applied in the first two tests but this was more or less unsuccessful. Those methods are very sensible, some experience is required and sometimes “luck” is required. The estimated value of load cycles for the first stud failure by evaluating secondary test data of T7_1 and T7_2 was 200.000. Preparing the tests T7_3 and T7_4 new test methods were developed to determine the moment of shear stud failure exactly [11]. Fig. 4 and fig. 5 show the comparison of the number of load cycles reached for each row of headed...
studs with the number calculated using eq. (2). In both tests the shear studs over the centre support failed first. They are subjected to high amplitudes of alternating shear forces (fig. 6). After that a successive failure from the inside to the outside occurs. Table 5 shows the total number of failed shear studs and the comparison of the moments for the first real stud failure \(n_{\text{first,real}}\) with the calculated number for these studs \(n_{\text{first,calc}}\).

### Table 5 Shear stud failure

<table>
<thead>
<tr>
<th>Test</th>
<th>No. of studs between loaded points</th>
<th>No. of studs failed between loaded points</th>
<th>(n_{\text{first}}) (Real)</th>
<th>Position (Real)</th>
<th>(n_{\text{first,calc}}) (Calc.)</th>
<th>Position (Calc.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T7_3</td>
<td>42</td>
<td>26</td>
<td>348.000</td>
<td>Centre</td>
<td>456.000</td>
<td>Centre</td>
</tr>
<tr>
<td>T7_4</td>
<td>38</td>
<td>28</td>
<td>254.250</td>
<td>Centre</td>
<td>664.000</td>
<td>Centre</td>
</tr>
</tbody>
</table>

**Figure 4** Comparison of the calculated lifetime for pure shear stud failure with the lifetime obtained in test T7_3

By neglecting the tensile stresses in the upper flange of the steel profile (pure stud failure, no interaction effects) the extreme sensibility of the lifetime prediction for inaccuracies in the calculated shear forces becomes obvious. Regarding fig. 4 the curve representing the real stud failure always shows smaller values than the dashed curve which represents the results from the linear predesign. For the stud arrangement in T7_4 only the lifetime of the studs over the centre support is overestimated (fig. 5) although the differences in the stud arrangement were very small (table 1, table 5).
In Test T7_4 there is only one pair of shear studs less near the two loaded points within (L1-L3) (e2 = 15cm instead of 12.5cm) than in T7_4. This results in a total of 38 instead of 42. However, the fatigue behaviour of the shear connection was in fact significantly influenced. Regarding the final failure of the beam no significant difference occurred (797,000 load cycles instead of 803,000), but the stud failure started much earlier in the more flexible, “weaker” beam T7_4. Both tests have in common that the shear studs over the centre support—the studs with the largest amplitude of alternating shear forces—failed first and earlier than expected. After that a successive stud failure occurs which runs faster the weaker the whole connection is.

Regarding fig. 6 and fig. 7 some important notes have to be pointed out:

- The studs over the centre support are subjected to alternating cyclic loads (fig. 6).
- An initial slip occurs at the very beginning of the tests. The value is dependant on the friction on the steel-concrete interface, the peak of the shear force, the concrete compressive strength, the flexibility of the shear connectors and the stiffness of the whole shear connection (fig. 7).
- After that only small and slow changes in the system take place. A slow but almost linear slip growth occurs. This is mainly due to the cyclic cracking of the concrete adjacent to the stud. Towards the end of the test a strong and faster slip growth occurs but the shear studs have already failed (fig. 7).

**Figure 5** Comparison of the calculated lifetime for pure shear stud failure with the lifetime obtained in test T7_4.
The slip growth is stronger the less stiffness is provided in the whole shear connection from the very beginning (fig. 7).

The faster the slip develops the earlier the failure occurs. The value of slip determines the failure.

**Small changes in the stiffness of the shear connection affect the lifetime of several studs significantly.**

The studs fail due to fatigue for very small amplitudes of slip (fig. 6, fig. 7). These values are not comparable to the slip amplitudes obtained from stress controlled push-out tests. Nevertheless, the general course of the slip development (fig. 7) is quite similar, but in beam tests the moment of failure generally occurs before the major slip growth takes place.

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**Figure 6** Slip development with a load cycle over the centre support (Alternating cyclic load)

**Figure 7** Boundary curves for the slip over the centre support over the course of time

Regarding all shear studs between the loaded points it can be shown that the amplitudes of shear forces range from a pure alternating cyclic load to pure unidirectional cyclic loads. Fig. 8 shows the slip hysteresis and the boundary curves for a pair of shear studs between the loaded points and the centre (x=3.235m). For unidirectional cyclic loads the (one-sided) initial slip is usually greater than for alternating amplitudes (for comparable amplitudes), because it is strongly dependant on the peak of the cyclic load. The slip
growth throughout the first cycles is mostly larger, although this is heavily influenced by the stiffness of the whole shear connection.

**Figure 8** Slip hysteresis and boundary curves for a pair of shear studs arranged 80cm left of the centre support in test T7_3

**Summary**

This paper deals with results of testing done on two span continuous composite beams under cyclic loading. The special feature is the realistic loading situation, realized by two alternating, synchronized hydraulic cylinders. Therefore especially the hogging region could be investigated under more or less realistic conditions. It must be pointed out that the existing design concept for preventing the global failure of the beam (failure mode “C”) delivers very conservative results. It became obvious however that with the existing design models the real fatigue behaviour of the shear connection in the hogging region under realistic (service-) loading cannot be described in a sufficiently exact manner. Concrete cracking has to be considered, but it was shown that the flexibility of the shear connection, which is not taken into account in the European standardization, also influences the fatigue behaviour significantly, also under service loads. The headed studs over the centre support are subjected to large amplitudes of alternating shear forces and a considerable number of shear studs failed due to fatigue a long time before the whole beam failed according to failure mode “C”.

It is important to point out that the shear connectors fail for very small amplitudes of slip (some tenth of a millimetre) and for a greater part of the lifetime of the beam the shear connection is interrupted. By neglecting the interaction effects and regarding the pure stud failure the sensibility of the existing design format to inaccuracies in the calculated shear forces becomes obvious. This is due to the very flat inclination of the fatigue strength curve. On the other hand it turned out that the moment of failure for a single shear stud is almost unpredictable. The high cycle fatigue concept, transferred from pure steel constructions, which assumes a constant relationship between the applied (service-) loads and the resulting fatigue-stress-amplitudes seems to be hardly applicable. There are ongoing and significant shear force redistributions along the shear connection, which are mainly due to the studs’ flexibility and the cyclic crushing of the concrete adjacent to the studs. Both are neglected in the existing design concept if the constructive requirements
mentioned in the introduction are satisfied. It turned out that the slip at the steel-concrete interface, which is heavily dependent on the stiffness of the whole shear connection determines the moment of fatigue failure. Further research activities at Kaiserslautern University are dealing with the question if a strain-life approach which assumes a correlation between the slip and the resulting lifetime of the shear connectors may be more applicable.

4. References