Practical-Orientated Full-Scale Tests In Comparison With Bond Checks Of Different Guidelines

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ABSTRACT

Various types of concrete structures have been strengthened by using externally bonded CFRP strips. For the design of such strengthening projects numerous design standards and guidelines are available. The design approaches of some of these guidelines and standards are mainly based on full-scale tests. These full-scale tests are often carried out on three- or four-point bending tests with continuous reinforcement. By contrast in actual building practice, continuous loads, cut-off reinforcement and continuous girders dominate. It can be theoretically shown that the loading difference and the changing of the reinforcement ratio both have a big influence on the bond force transfer of externally bonded CFRP strips.

In this paper full-scale tests on strengthened reinforced concrete single-span slabs with cut-off reinforcement and full-scale tests on two-span slabs strengthened in the hogging region are presented. Some of these tests were also loaded using a setup which simulates a continuous load approximately. In the tests, either shear failure or bond failure occurred. After the description of the tests the results are compared to the predicted load-carrying capacity of several guidelines. These comparisons show that some guidelines and approaches, which are mainly based on mechanical principles, perform well with the results. By contrast some of the more empirically based guidelines do not predict the failures quite as well.

KEYWORD

CFRP strips, bond performance, full-scale tests, code evaluation
1. INTRODUCTION

The strengthening of existing concrete structures with externally bonded FRP materials is a widely used construction method. In most cases in Germany, prefabricated CFRP strips are used for this purpose.

One of the main issues with the design of such a strengthened structural element is the verification of the bond, because the moderate bond between the CFRP strips and the concrete often fails in full-scale tests. For this reason a lot of research has been carried out on the bond behavior of externally bonded FRP materials which has resulted in different verification concepts for the bond force transfer.

To estimate which concepts describe the reality accurately some full-scale tests have been carried out. In most of these tests in the literature symmetric 3-point and 4-point bending tests were carried out. But these tests do not represent the actual building practice very well as in reality uniform loads instead of single loads prevail and many structural elements are continuous girders instead of simply-supported beams.

For this reasons the presented full-scale tests herein focus on these more practice-oriented problems.

2. TEST PROGRAMS

2.1 Materials

The materials used in the tests presented are described in the following. Besides the material properties also the testing standards are mentioned because in many cases the results of material testing depend on the testing method or standard.

(1) Concrete

In the tests, concrete grades C20/25 and C40/50 according to EN 206-1 [1] with a maximum aggregate size of 8 mm were used. The concrete compressive strength \( f_{cm,cube} \) was tested on the same day as the test on 150 mm cubes according to EN 12390-3 [2] as well the surface tensile strength \( f_{ctm,surf} \) in pull-off tests according to EN 1542 [3]. The other concrete properties were tested on each casting. The Young's modulus \( E_c \), the flexural tensile strength \( f_{ct,fl} \) and the splitting tensile strength \( f_{ct,sp} \) were tested on 150/300 cylinders according to DIN 1048-5 [4], the flexural tensile strength \( f_{ct} \) and the splitting tensile strength \( f_{ct,sp} \) were tested on 150 mm cubes according to EN 12390-6 [6]. The resulting mechanical properties of the concrete for the different specimens, which are described in section 2.2 are listed in Table 1.

<table>
<thead>
<tr>
<th>Name</th>
<th>( f_{cm,cube} ) mm</th>
<th>( f_{cm,surf} ) mm</th>
<th>( E_c ) N/mm²</th>
<th>( f_{ct,fl} ) N/mm²</th>
<th>( f_{ct,sp} ) N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>VVP2</td>
<td>55.3</td>
<td>1.6</td>
<td>29.4</td>
<td>-</td>
<td>3.9</td>
</tr>
<tr>
<td>VVP4</td>
<td>32.0</td>
<td>2.4</td>
<td>22.7</td>
<td>3.7</td>
<td>2.8</td>
</tr>
<tr>
<td>VVP5</td>
<td>31.7</td>
<td>2.3</td>
<td>22.7</td>
<td>3.7</td>
<td>2.8</td>
</tr>
<tr>
<td>VVP6</td>
<td>32.1</td>
<td>2.4</td>
<td>22.7</td>
<td>3.7</td>
<td>2.8</td>
</tr>
<tr>
<td>QVPA3</td>
<td>44.9</td>
<td>2.4</td>
<td>26.5</td>
<td>4.7</td>
<td>2.7</td>
</tr>
</tbody>
</table>

(2) Reinforcement

For the internal steel reinforcement ribbed bars according to EN 10080 [7] and plain bars according to EN 10025-2 [8] were used. The rebars were tested according to EN 10002-1 [9]. The mechanical properties (Young's modulus \( E_s \), yield strength \( f_{sy} \) and tensile strength \( f_{st} \)) of the reinforcement are listed in Table 2 for each nominal diameter \( \phi_s \) used.

<table>
<thead>
<tr>
<th>( \phi_s )</th>
<th>( E_s ) (N/mm²)</th>
<th>( f_{sy} ) (N/mm²)</th>
<th>( f_{st} ) (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 (ribbed)</td>
<td>205.8</td>
<td>566.7</td>
<td>615.4</td>
</tr>
<tr>
<td>10 (ribbed)</td>
<td>196.6</td>
<td>445.6</td>
<td>572.6</td>
</tr>
<tr>
<td>20 (ribbed)</td>
<td>204.0</td>
<td>513.3</td>
<td>616.2</td>
</tr>
<tr>
<td>16 (plain)</td>
<td>204.3</td>
<td>304.5</td>
<td>433.9</td>
</tr>
</tbody>
</table>

(3) CFRP strips

S&P 150/2000 CFRP strips [10] with a size of \( b_f \times t_f = 100 \text{ mm} \times 1.4 \text{ mm} \) and 
50 mm x1.4 mm were used in the test. The tensile strength \( f_{fu} \) and the Young's modulus \( E_f \), tested according to EN 2561 [11] were 3121 N/mm² and 168.4 kN/mm² for the 100 mm x1.4 mm strip and 3003 N/mm² and 174.6 kN/mm² for the 50 mm x1.4 mm strip.

(4) Adhesive

For the bonding of the CFRP strips an epoxy adhesive of type Sikadur 30 in accordance with EN 1504-4 [12] was used.

(5) Strengthening

Before the CFRP strip was bonded to the beam, respective surface was sand-blasted while the surrounding areas were covered with tape to obtain a distinctly defined bond area.

(6) Bond

Besides the full-scale tests, some bond tests were
carried out on the internal reinforcement according RILEM RC6 [13] and on the externally bonded reinforcement according to Zilch and Niedermeier [14], as the bond performance of each concrete mixture and reinforcement differs. These tests are documented and evaluated in [15].

2.2 Test specimens

Five structural elements with geometric properties listed in Table 3 were tested. (Where $h_c$ is the height, $b_c$ is the width and $c$ is the concrete cover of the specimens)

Table 3 Geometric properties of the specimens

<table>
<thead>
<tr>
<th>Name</th>
<th>$b_c$ mm</th>
<th>$h_c$ mm</th>
<th>$c$ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>VVP2</td>
<td>500</td>
<td>200</td>
<td>25</td>
</tr>
<tr>
<td>VVP4</td>
<td>500</td>
<td>160</td>
<td>15</td>
</tr>
<tr>
<td>VVP5</td>
<td>500</td>
<td>160</td>
<td>15</td>
</tr>
<tr>
<td>VVP6</td>
<td>500</td>
<td>160</td>
<td>15</td>
</tr>
<tr>
<td>QVPA3</td>
<td>500</td>
<td>200</td>
<td>25</td>
</tr>
</tbody>
</table>

The internal reinforcement is listed in Table 4, where $A_{s1}$ is the reinforcement in the sagging region and $A_{s2}$ is the reinforcement in the hogging region. In Table 4 the strengthening is also listed, where $A_{f1}$ is the strengthening in the sagging region and $A_{f2}$ is the strengthening in the hogging region.

Table 4 Reinforcement and strengthening of the specimens

<table>
<thead>
<tr>
<th>Name</th>
<th>$A_{s1}$ mm</th>
<th>$A_{s2}$ mm</th>
<th>$A_{f1}$ mm</th>
<th>$A_{f2}$ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>VVP2</td>
<td>4Ø20</td>
<td>4Ø10</td>
<td>2x100x1.4</td>
<td></td>
</tr>
<tr>
<td>VVP4</td>
<td>7Ø6</td>
<td>2x50x1.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>VVP5</td>
<td>7Ø6</td>
<td>1x50x1.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>VVP6</td>
<td>7Ø6</td>
<td>2x50x1.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>QVPA3</td>
<td>6 Ø16</td>
<td>3x100x1.4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In specimen VVP4 and VVP6 three of the seven bars ended 200 mm before the support, to simulate cut-off reinforcement as commonly practiced. The CFRP strips ended 100 mm before the support in the specimen VVP4 and VVP5 and 370 mm in the specimen VVP6. In the specimens strengthened in the hogging region the CFRP strip ended 100 mm before the loading points on specimen VVP2 and 300 mm on QVPA3.

2.3 Test setup

Three different types of test setups were used as shown in Figure 1.

The 2-span beams with two equal spans of $l_0$ were loaded with two equal forces at a distance $a$ from the middle support. These tests were carried out because it is an important issue in current practice to strengthen continuous beams in the hogging region where maximum shear and maximum moment coincide and thus cause high stresses in the bond.

The eccentric 4-point bending tests had two loads at a distance $a$ from the support. The distance was chosen so that moment and shear were similar to a uniformly loaded beam.

The 6-point bending tests were loaded with four symmetric loads to simulate a uniformly distributed load.

Fig.1 Different test setups

Load spacing and the span lengths for each specimen are listed in Table 5.

Table 5 Characteristics of reinforcements

<table>
<thead>
<tr>
<th>Name</th>
<th>Test</th>
<th>$a$ mm</th>
<th>$l_0$ mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>VVP2</td>
<td>2-span</td>
<td>1050</td>
<td>2000</td>
</tr>
<tr>
<td>VVP4</td>
<td>Eccentric 4 point bending</td>
<td>800</td>
<td>4000</td>
</tr>
<tr>
<td>VVP5</td>
<td>Eccentric 4 point bending</td>
<td>800</td>
<td>4000</td>
</tr>
<tr>
<td>VVP6</td>
<td>6 point bending</td>
<td>800</td>
<td>4000</td>
</tr>
<tr>
<td>QVPA3</td>
<td>2-span</td>
<td>1500</td>
<td>3000</td>
</tr>
</tbody>
</table>

2.4 Test procedure and measurement

During the testing, the girders were loaded with a linearly increasing force at each point of application. The forces were measured with load cells at each of those points and the strain in the CFRP strip was measured using numerous strain gauges. Additionally, the displacements were measured with six LVDTs and an optical
measurement system. The latter was also used to measure the crack pattern and crack widths.

2.5 Test results
In Table 6 the maximum moment \( M_{\text{max}} \), the maximum force at each concentrated load \( F_{\text{max}} \), the measured maximum strain in the CFRP strip \( \varepsilon_{f,\text{max}} \) and the failure mode are listed for the different specimens.

The moment was calculated linear-elastically based on the forces at the points of application. However, for the two-span beams there was some moment redistribution caused by the difference in stiffness of the cracked sections at the support and within the span (cf. [16]).

<table>
<thead>
<tr>
<th>Name</th>
<th>( M_{\text{max}} ) kNm</th>
<th>( F_{\text{max}} ) kN</th>
<th>( \varepsilon_{f,\text{max}} ) mm/m</th>
<th>Failure Mode (location)</th>
</tr>
</thead>
<tbody>
<tr>
<td>VVP2</td>
<td>-61.0</td>
<td>175.2</td>
<td>2.5</td>
<td>IC-debonding (mid-support)</td>
</tr>
<tr>
<td>VVP4</td>
<td>52.6</td>
<td>36.5</td>
<td>8.3</td>
<td>shear failure (end-support)</td>
</tr>
<tr>
<td>VVP5</td>
<td>36.6</td>
<td>25.4</td>
<td>8.3</td>
<td>IC-debonding (between loads)</td>
</tr>
<tr>
<td>VVP6</td>
<td>36.8</td>
<td>15.3</td>
<td>4.3</td>
<td>IC-debonding (cut off)</td>
</tr>
<tr>
<td>QVPA3</td>
<td>-44.0</td>
<td>78.0</td>
<td>2.0</td>
<td>IC-debonding (mid-support)</td>
</tr>
</tbody>
</table>

For the tests on single-span beams, the load deflection curves are shown in Figure 2.

![Fig.2 Maximum moment over maximum deflection](image2)

For the tests on two-span beams, the linear-elastic calculated moment without moment redistribution over the middle support is shown over the deflection in Figure 3.

![Fig.3 Maximum hogging moment over mid-span deflection](image3)

2.6 Interpretation
The results show that the specimen with the cut-off rebar does not act very favorably. It can also be seen that strengthening in the hogging region is more critical for debonding, since and only low strains in the CFRP strips where reached at the ultimate load.

The strains of the CFRP strip at the ultimate moment were back calculated by cross section analysis using the parabola-rectangle diagram for concrete under compression according to EN 1992-1-1 [17]. In the first step the strains \( \varepsilon_{f,\text{LE}} \) were calculated using the maximum moment of the linear elastic analysis. Because the two-span beams show a little moment redistribution as do the tests in [16] the moments were calculated in a second step by using the different stiffness of the cracked cross-section in the hogging and sagging regions \( M_{\text{max},\text{Re}} \) which leads to the strain \( \varepsilon_{f,\text{MRe}} \). In a third step the bond caused interaction between the internal and external reinforcement according to [18] was considered which leads to the strain \( \varepsilon_{f,\text{Bint}} \).

The strains of this analysis are listed in Table 7.

<table>
<thead>
<tr>
<th>Name</th>
<th>( \varepsilon_{f,\text{LE}} ) mm/m</th>
<th>( M_{\text{max},\text{Re}} ) kNm</th>
<th>( \varepsilon_{f,\text{MRe}} ) mm/m</th>
<th>( \varepsilon_{f,\text{Bint}} ) mm/m</th>
<th>( \varepsilon_{f,\text{calc}}/\varepsilon_{f,\text{mes}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>VVP2</td>
<td>4.1</td>
<td>49.0</td>
<td>2.8</td>
<td>2.7</td>
<td>2.7/2.5=1.08</td>
</tr>
<tr>
<td>VVP4</td>
<td>10.7</td>
<td>-</td>
<td>-</td>
<td>8.9</td>
<td>8.9/8.3=1.07</td>
</tr>
<tr>
<td>VVP5</td>
<td>11.6</td>
<td>-</td>
<td>-</td>
<td>9.4</td>
<td>9.4/8.3=1.13</td>
</tr>
<tr>
<td>VVP6</td>
<td>5.7</td>
<td>-</td>
<td>-</td>
<td>4.8</td>
<td>4.3/4.8=0.90</td>
</tr>
<tr>
<td>QVPA3</td>
<td>2.9</td>
<td>29.9</td>
<td>2.1</td>
<td>-</td>
<td>2.1/2.0=1.05</td>
</tr>
</tbody>
</table>

The comparison between the calculated strains in Table 7 to the measured strains in Table 6 shows a good agreement. The implication is that there is both moment retribution as well as a bond caused by interaction between the different reinforcements.
3. COMPARISON WITH SOME GUIDELINES

3.1 Description
In this paper the bond force transfer of externally bonded CFRP strips will be discussed. The test VVP4 is not part of the following comparison, because it failed in shear. For this test the empirical equations for the shear resistance of members without shear reinforcement of the EN 1992-1-1 [17] describe the ultimate load quite well.

In the following only the result of the verifications of the intermediate crack debonding are compared with the test results, because in every test except the test VVP4 such intermediate crack debonding occurred. Mainly there are three approaches for analysis of the IC debonding:

- Strain/stress limitations of the FRP (fixed or depending on material parameters)
- Limitation of the tensile stress increase of the CFRP
- Approach based on the envelope line of tensile stress

In the following four different guidelines which each include one of the mentioned concepts are summarized briefly.

(1) FIB Bulletin 14 Approach 1:
The fib Bulletin [19] contains three approaches. The first approach consists of an end anchorage verification and a fixed strain limitation of $\varepsilon_f, \text{max} = 6.5 \text{ mm/m}$ for design values and $8.5 \text{ mm/m}$ for mean values. This strain limitation should avoid IC debonding altogether.

(2) CNR-DT 200
The Italian Guideline CNR-DT 200 [20] also uses end anchorage verification and strain limitation to avoid IC debonding. The strain limitation is calculated using equation (1):

$$\varepsilon_f, \text{max} = k_{cr} \cdot \frac{2 \cdot G_F}{E_f \cdot \tau_f}$$

where,

- $k_{cr}$ empirical calibration coefficient, 4.3 for mean values and 3.1 for design values
- $G_F$ bond fracture energy
- $E_f$ modulus of elasticity of FRP
- $\tau_f$ failure shear stress of FRP

Bond fracture energy $G_F$ in equation (1) can be calculated by equation (2)

$$G_F = k_G \cdot k_b \cdot \sqrt{f_{ck} \cdot f_{clm}}$$

where,

- $k_G$ experimental bond coefficient, 0.064 for mean values and 0.026 for design values
- $f_{ck}$ char. concrete compression strength

The geometrical factor $k_b$ can be calculated as follows:

$$k_b = \sqrt{\frac{2 - h_f}{h_f}} \geq 1.0$$

(4) SIA 166
In the Swiss Guideline SIA 166 [21] there are three bond checks -- an end anchorage check, a strain limitation of 8 mm/m and a limitation of the tensile force/stress increase of the CFRP. In this paper only the limitation of the force increase is considered.

The force increase can be calculated using equation (4)

$$\left( \frac{\Delta F_f}{\Delta x} \right)_R = F_f \left( x + \Delta x \right) - F_f \left( x \right)$$

where,

- $x$ coordinate on the element,
- $\Delta x$ length of the considered element (arbitrary)
- $F_f$ tensile force in the FRP

This force increase should be smaller than the resistance, which can be calculated using equation (5):

$$\left( \frac{\Delta F_f}{\Delta x} \right)_R = \tau_{f, \text{lim}} \cdot b_f = 0.75 \cdot \sqrt{f_{ck} \cdot b_f}$$

(5) DAfStb Guideline
The German Guideline [22] has a progressive verification procedure for IC debonding. The most accurate verification is an approach based on the envelope line of the tensile stresses. This is explained more precisely in this paper as the guideline is not currently available in English.

In this approach which is based on the authors' previous work [23] a check of elements between the flexural cracks has to be performed. Therefore, it must be verified whether the change of the tensile force of the FRP $\Delta F_{fR}$ at each intermediate crack element is higher than the resistance at each intermediate crack element $\Delta F_{fR}$:

$$\Delta F_{fR} \leq \Delta F_{fR}$$

The exposure at the intermediate crack elements can be calculated from the difference of the tensile stresses at the cracks:

$$\Delta F_{fR} = F_{fR} \left( x + s_i \right) - F_{fR} \left( x \right)$$

For this equation the crack distance $s_i$ is needed, which can be simplified on the safe side by using one and half times the transmission length of the reinforcing steel $l_{e0}$ according to equation (8). In reality, the crack distances will be smaller, since
both the CFRP strips change the crack pattern and crack distances depend on the position of the load.

\[ s_i = 1.5 \cdot l_{i,0} \]  

(8)

The transmission length of the reinforcing steel can be calculated using equation (9) by dividing the cracking moment of the RC \( M_{cr} \) through the lever arm and the bond strength per length of rebar \( F_{b,m} \). The cracking moment of the RC can be calculated using the simplified equation (10) by multiplying the section moment \( W_{c,0} \) with the flexural tensile strength which is determined by multiplying the surface tension with a coefficient depending on the structural height.

\[ l_{c,0} = \frac{M_{cr}}{z_s \cdot F_{b,m}} \]  

(9)

\[ M_{cr} = \kappa_{fl} \cdot f_{c,msurt} \cdot W_{c,0} \]  

(10)

where,

\[ \kappa_{fl} = (1.6 - h_c/1000) \geq 1.0 \]

\[ h_c \] height of the structural element in mm

The bond force per length can be calculated using equation (11) by taking the circumference of the rebars and their average bond stress. The latter can be calculated depending on the bond performance of the existing ribbed rebar using equation (12). In this equation \( k_{vb1} = 1 \) should be used for good bond conditions and \( k_{vb1} = 0.7 \) for moderate bond conditions.

\[ F_{b,m} = \sum_{i=1}^{n} n_{si} \cdot \phi_i \cdot \pi \cdot f_{b,m} \]  

(11)

\[ f_{b,m} = \kappa_{vb1} \cdot 0.43 \cdot f_{cm}^{2/3} \]  

(12)

where,

\( n_{si} \) number of the rebars;

\( \phi_i \) diameter of the rebars

The resistance at each intermediate crack element (ICE) can be calculated using the following equation which considers the three different effects determined by equations (14), (19) and (21):

\[ \Delta F_{IR} = \Delta F_{E,Bl} + \Delta F_{E,BF} + \Delta F_{E,KF} \]  

(13)

\( \Delta F_{F,IR} \) of the existing ribbed rebar using equation (12). In

The share of the bond friction \( \Delta F_{E,BL} \) can be determined by using the values of the bilinear bond approach (the maximum slip \( s_{0f} \) and the maximum bond stress \( \tau_{fl} \), which are listed in Table 8, and equation (14), where \( F_{Fm} \) is the FRP stress at the lower stressed crack of the ICE. This two-tiered equation depends on the FRP force \( F_{Fm,0} \), which can be calculated using the equation (15), and the resistance \( \Delta F_{E,Bl} \), which can be calculated using the second part of equation (14) with FRP Force \( F_{Fm,0} \)

\[ \Delta F_{E,Bl} = \begin{cases} F_{Fm,0} - \Delta F_{E,Bl} & \text{if } F_{Fm} \leq F_{Fm,0} \\ \sqrt{b_l^2 \tau_{fl} s_0} + \frac{f_{cm}^2}{b_l} - F_{Fm} & \text{if } F_{Fm} > F_{Fm,0} \end{cases} \]  

(14)

\[ F_{Fm} = s_0 \cdot h_l \cdot b_l \cdot f_{FL} \]  

(15)

The still missing resistance force \( \Delta F_{E,BF} \) can be calculated using equations (16) to (18)

\[ \Delta F_{E,BF} = \begin{cases} \frac{F_{Fm,0} \cdot 1 - s_{0f}}{b_l \cdot \tau_{fl}} & \text{if } s_{0f} < b_l \cdot \tau_{fl} \\ \frac{F_{Fm,0} \cdot 1 - s_{0f}}{b_l \cdot \tau_{fl}} & \text{if } s_{0f} \geq b_l \cdot \tau_{fl} \end{cases} \]  

(16)

\[ F_{Fm,0} = \sqrt{b_l^2 \tau_{fl} s_0} \]  

(17)

\[ l_{b,eff} = \frac{2}{1.128} \sqrt{\frac{E_t \cdot \phi_l \cdot \tau_{fl}}{\tau_{fl}}} \]  

(18)

The share of the bond friction \( \Delta F_{E,BF} \) can be determined by multiplying the bond friction strength listed in Table 8 with the already debonded part which corresponds to the crack spacing \( s_i \) minus the effective length of the intermediate crack element \( s_{eff} \). If the effective length of the intermediate crack element is bigger than the crack spacing the share of the bond friction is zero.

\[ \Delta F_{E,BF} = \tau_{fl} \cdot b_l \cdot (s_i - s_{eff}) \geq 0 \]  

(19)

The equation for the share of the bond friction is dependent on the effective length of the intermediate crack element \( s_{eff} \) which can be calculated using the parameters of the bilinear approach and the following equation:

\[ s_{eff} = \sqrt{\frac{E_t \cdot s_0^2 + \frac{F_{Fm,0}^2}{b_l^2 \cdot \tau_{fl}^2}}{b_l^2 \cdot \tau_{fl}}} \]  

(20)

\[ F_{Fm,0} = \frac{F_{Fm,0}}{b_l^2 \cdot \tau_{fl}} \]  

Table 8 Values of the extended bilinear bond approach [22]

<table>
<thead>
<tr>
<th>Name</th>
<th>( \tau_{fl} ) N/mm²</th>
<th>( s_{0f} ) mm</th>
<th>( \tau_{fl} ) N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>0.527 . ( \sqrt{f_{cm} \cdot f_{c,msurt}} )</td>
<td>0.212</td>
<td>10.8 . ( f_{cm}^{-0.89} )</td>
</tr>
<tr>
<td>Design</td>
<td>0.366 . ( \sqrt{f_{cm} \cdot f_{c,msurt}} )</td>
<td>0.202</td>
<td>17 . ( f_{cm}^{-0.89} )</td>
</tr>
</tbody>
</table>

The contribution \( \Delta F_{F,KL} \) of the curvature can be calculated using the following equation where \( \varepsilon_l \) is the strain at the lower stressed crack, \( \varepsilon_c \) is the concrete strain at the lower stressed crack, \( h_c \) is the structural height and \( \kappa_3 \) is a calibration factor which can be set to \( \kappa_3 = 3.33 \times 10^3 \) N/mm for mean values and \( \kappa_3 = 24.3 \times 10^3 \) N/mm for design values.

\[ \Delta F_{F,KL} = s_i \cdot \kappa_3 \cdot \frac{\varepsilon_c}{h_c} \cdot \varepsilon_l \cdot b_L \]  

(21)
The aforementioned approach for IC debonding by verifying each intermediate crack element is quite complex in comparison to the other approaches. But it is possible to make simplifications on the safe side for this mechanically based approach. In addition to the complex approach, the guideline also contains two simplified versions. These simplified approaches result in a more conservative ultimate load.

3.2 Comparison
This comparison is less about the guideline itself and more about the concepts discussed above. In the above summary of guidelines and their comparison the safety factor was set to 1.0 as the safety factor should not be used for mapping model uncertainties but should ensure that the national safety level corresponds to the failure probability. (cf. EN 1990 [24])

In the following the calculated ultimate load according to the IC debonding design approaches of different guidelines is compared with the ultimate load of the test. Figure 4 shows the experimental ultimate load divided by the calculated ultimate load for each approach. If this value is equal or bigger than 1.0 the calculation is on the safe side, otherwise it is unsafe.

The comparison shows that the approaches with the strain limitation (CNR-DT-200 and fib approach 1) have a high standard deviation and lead either to an unsafe or uneconomic calculation. The approach with the limitation of the force increase (SIA 166) leads to an unsafe analysis in most cases. Only the analysis based on the envelope of tensile stresses (DAfStb-Guideline) leads to a safe calculation for all specimens with the lowest standard deviation. This approach also indicates the location where debonding occurs in the structural element. In Zilch et al. [25] [26] there is also a comparison of the German Guideline and other International guidelines, with a test database of more than 400 full scale test, which almost shows the same result.

4. CONCLUSIONS
In this paper five full-scale tests simulating practically oriented cases were presented. The tests show that the strengthening in hogging regions and the cut-off points are critical places for intermediate crack debonding of the CFRP strip. The comparison of the test with some design approaches for intermediate crack debonding presented in various guidelines showed that a reliable prediction of the failure load and position is only possible with a continuous verification concept using a tensile stress envelope. Using this method and by verifying intermediate crack elements, a safe and economic design of FRP-strengthened structural elements is possible.

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REFERENCES


