Fatigue Life of Coupling Joints of Prestressing Tendons

Comportement à la fatigue des ancrages de câbles

Ermüdungsfestigkeit von Spanngliedkopplungen

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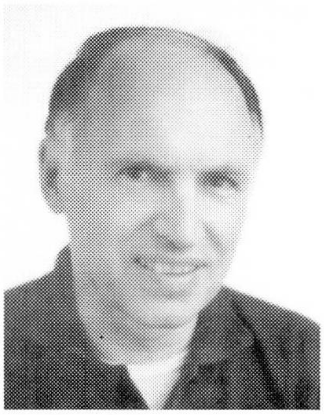

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SUMMARY
In the case of post-tensioned members, mainly the splice devices of the tendons are endangered by fatigue. The fatigue resistance of the coupler is considerably lower than that of the prestressing steel outside the coupling joint. Experimental work to study the fatigue behaviour of couplers embedded in concrete of two different post-tensioning systems has been carried out . This paper presents the experimental results, and a simple method to calculate the stresses in the tendons and in the recinforcing steel considering the realistic bond behaviour of the prestressing steel and the reinforcing steel.

RÉSUMÉ
Dans le cas de structures précontraintes, les ancrages des câbles sont les principaux points menacés par la fatigue. La résistance à la fatigue de l'ensemble est considérablement moindre que celle de l'acier prétendu. On a fait des études expérimentales pour déterminer le comportement de l'ancrage à la fatigue sur deux systèmes de précontrainte différents. Cet article présente les résultats expérimentaux ainsi qu'une méthode simple pour calculer les tensions des câbles et de l´armature renforcée, en considérant la relation entre le comportement de l'acier prétendu et celui de l'acier renforcé.

ZUSAMMENFASSUNG
In Spannbetontragwerken sind die Spanngliedkopplungen meist ermüdungsgefährdete Bauteile, da ihre Ermüdungsfestigkeit wesentlich niedriger ist als die des Spannstahls außerhalb des Kopplungsbereichs. Es wurden experimentelle Untersuchungen zur Erforschung des Ermüdungsverhaltens der Kopplungen eingeleitet. Dieser Artikel berichtet über die Ergebnisse der durchgeführten Versuche an Kopplungen zweier unterschiedlicher Spannsysteme und zeigt eine einfache Methode zur Berechnung von Spannungen des Betonstahls und des Spannstahls in Querschnitten mit gemischter Bewehrung.

1. INTRODUCTION

In addition to the simple method for the fatigue design, in which the highest calculated stress range should not exceed the nominal value of the endurance limit, more detailed design methods are presented in EC-2 and Model Code 90. These methods require realistic S-N curves of the materials used, as well as precise descriptions of the actual fatigue loading.

Investigations at the Otto-Graf Institute at Stuttgart [1] have shown that the fatigue failure is primarily caused by fretting corrosion between the coupling elements. These tests have been carried out with free straight tendons, but an additional fretting effect might occur, reducing the fatigue strength if a spliced tendon is embedded in concrete.

Different systems of spliced tendons embedded in concrete have been tested by Kordina and Günther [2]. As a result of these tests the fatigue limit for 2 million cycles is significantly lower than when tested in air. But these tests were performed without any additional ordinary reinforcement crossing the joint.
A safe fatigue design also requires a realistic calculation of steel stresses. Due to different bond behaviour of reinforcing and prestressing steel, the real stresses differ from the results of a common cracked stage calculation. The effect of this stress redistribution in the case of a coupling joint is not known until now.

In the following experimental work concerning the fatigue behaviour of embedded couplers under realistic conditions and the redistribution of the steel stresses is described, in addition a simple method to calculate the stresses in tendon and in reinforcing steel considering the different bond behaviour of tendons and reinforcing steel is presented.

2. EXPERIMENTAL PROGRAM

In total couplers of two different posttensioning systems will be tested.
In a first series the coupler K (Fig. 1) of the posttensioning system D & W (strands) was tested under constant amplitude loading. The stress ranges were chosen as follows:

$$\Delta\sigma_z = 140 \text{ MPa (5 tests)}$$
$$\Delta\sigma_z = 180 \text{ MPa (4 tests)}$$
$$\Delta\sigma_z = 220 \text{ MPa (5 tests)}$$

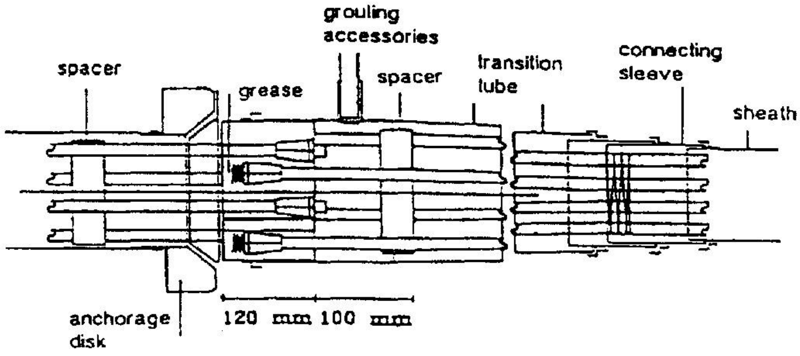

Fig. 1: Coupler K, posttensioning system D & W (strands)

The length of the test beams was 6.0 m and the depth 1.00 m. The beam width of 40 cm was determined by the edge distance for the anchorages given in the approval documents of the posttensioning system. Tendons consisting of 6 strands (type 6806 D & W) were used and the applied prestressing force was chosen to be 650 kN. This value is lower than the nominal value given by the German approval documents. Fig. 2 shows the test beams, its cross section, and the test setup in the first test series.

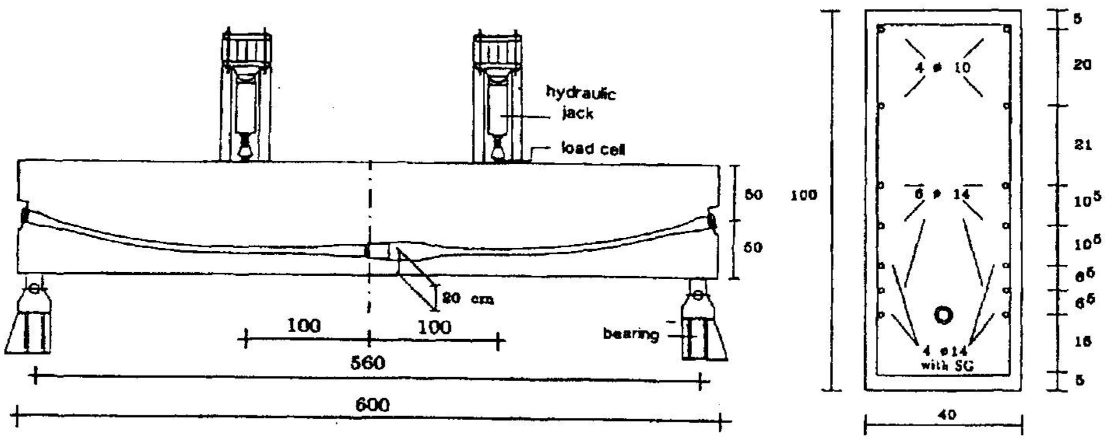

Fig. 2: Test beams, its cross section and test setup

In the second test series the splice device of the tendon of Company BBRV-SUSPA is been tested. Each of the two tendons at both sides of the coupler consist of 16 wires with diameter of 7mm made from cold drawn smooth prestressing steel. All wires of the tendon end in a common steel anchor head. The wires are fixed by cold pressed studs at their ends.

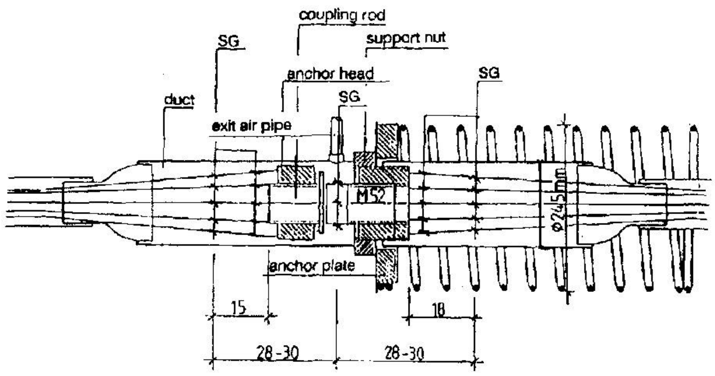

Fig. 3: Coupler B/Kl, posttensioning system BBRV-SUSPA II

The most important test parameters (σ_{zmax}, load arrangement, dimension of the test beams) were chosen in such a way that a comparison is possible between the new test results and those obtained by Kordina and Günther.

The experience from the first test series leads to a modification in the construction of the test beam. The type of beams used in the second test series is shown Fig. 4. It consists of clear defined compression and tension chord. The tendon runs in a straight line between the anchors at the ends of the beam in the center of the tension chord. The tendon coupler crosses the coupling joint which is located in the middle of the span length. Additionally four reinforcing bars of diameter 14mm cross the joint. The top compression steel chord is flexurally rigid and connected to the concrete part of the beam. During the test the compression force in the upper chord is measured by load cells. The measurement of the steel strains in the tension chord is performed by strain gauges (SG). Each of the two tendons is equipped with 24 measuring points: 16 in the vicinity of the anchor head and 8 more at a distance of 17 cm apart from it. The force in the coupling spindle is controlled by four strain gauges distributed over its circumference. Each reinforcing bar is equipped with 7 strain gauges. Their location corresponds to the position of SG on the prestressing wires. During cyclic loading the stress range in some selected wires is continuously controlled.

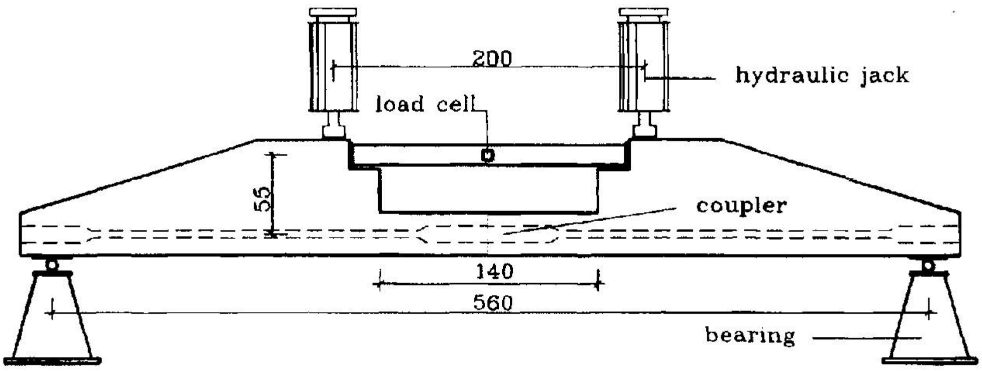

Fig. 4: Test beams for second test series

Up to now eight tests have been carried out. It was found, that the failure of the wires occurred always in the neighbourhood of the anchor head. The fracture developed from the fatigue notches which were located at the inside of the circle over which the wires are distributed. These are the regions of the highest lateral pressure along the wires in the coupling region. The stress redistribution was also observed as in former tests.

3. TEST RESULTS

3.1 Fatigue Behaviour

The statistical analysis of the numbers of cycles N was performed assuming a normal distribution for log N. The results for the first test series are shown in a S-N diagramm, Fig. 5. The straight lines connect the mean values. The dotted line represents the results of a linear regression analysis including all data:

$$\log N = 13.634 - 3.603 \log \Delta\sigma \quad (1)$$

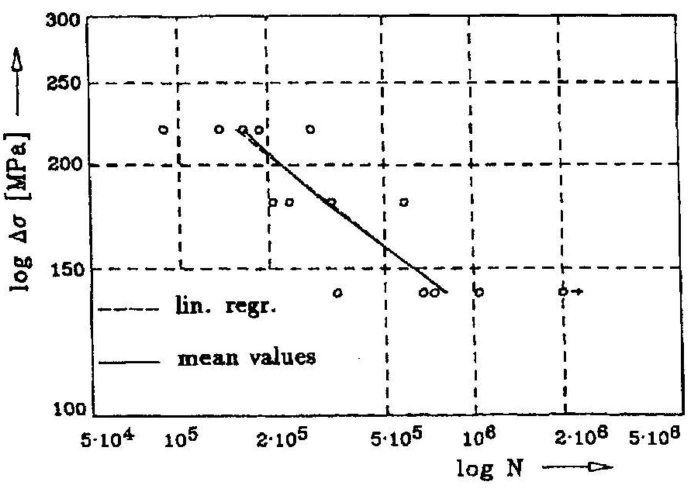

Fig. 5: Fatigue behaviour of the first test series

The first results in the second test series indicate the same tendency as in the first test series. The stress increase in the prestressing steel is significantly lower compared to that of the reinforcing steel.

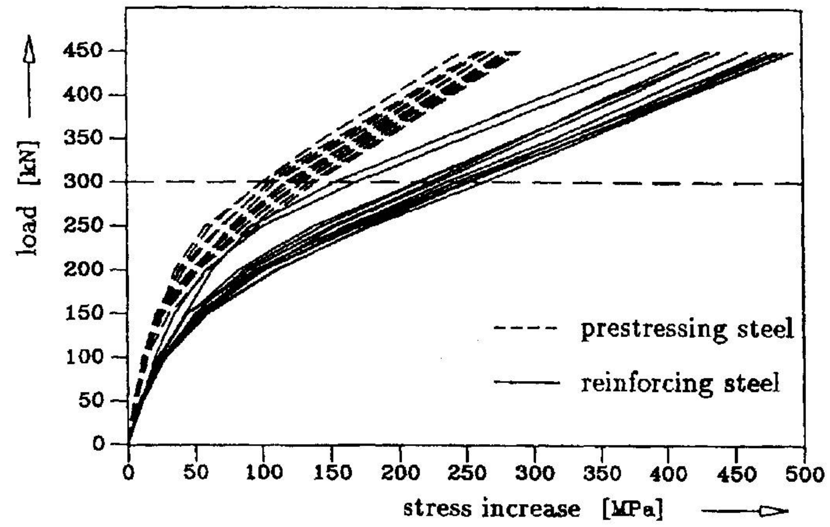

Fig. 7: Stress increases in the reinforcing bars and in the tendon

4. SIMPLE METHOD TO CALCULATE THE STEEL STRESSES

4.1 Background

Because of the different bond behaviour of tendons and reinforcing steel the increase of the stresses in both steel types after the appearance of cracks is different. Considering the compatibility and equilibrium in a cracked prestressed concrete member the following formulas can be stated with the external force F to calculate the actual stresses in tendons and in reinforcing steel.

For a single crack:

$$\sigma_{sR} = \frac{F}{A_s + \xi_1 \cdot A_z} \quad ; \quad \sigma_{zR} = \frac{\xi_1 \cdot F}{A_s + \xi_1 \cdot A_z} \qquad (4)$$

where

$$\xi_1 = \sqrt{\xi \cdot \frac{U_{zw} \cdot A_s}{U_s \cdot A_z}} \quad ; \quad \xi = \frac{\tau_{zm}}{\tau_{sm}}$$

For a stabilized crack pattern:

$$\sigma_{sR} = \frac{F - 0{,}6 \cdot T_{um}}{A_s + A_z} + \frac{0{,}6 \cdot T_{um}}{A_s + \xi_1 \cdot A_z} \qquad (5)$$

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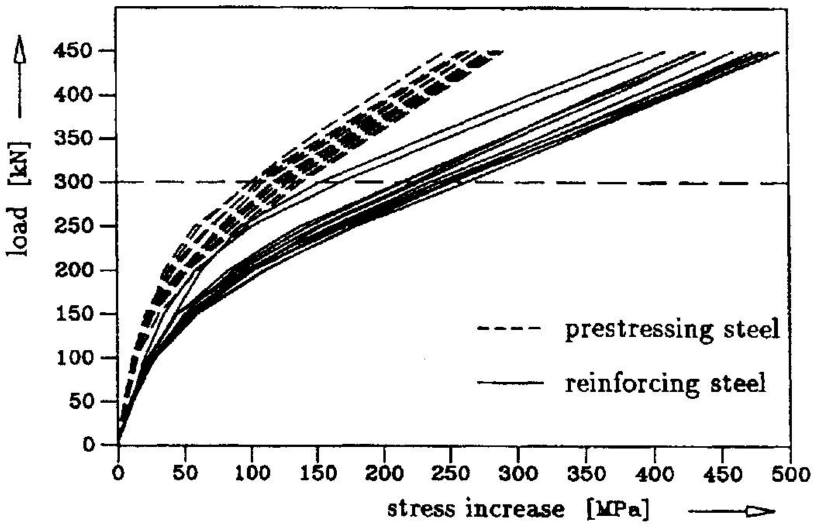

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These formulas show a stress calculation in prestressed concrete members close to reality, depending on the estimation of the bond force T_{um} and the relation between the mean bond stresses of tendons τ_{zm} and τ_{sm} of the reinforcing steel.

4.2. Calculation of the Mean Bond Stress τ_{zm} and τ_{sm}

The mean bond stress τ_{sm} and τ_{zm} can be calculated with the known crack spacing $s_r = 2\, l_{es}$ as follows:

$$\tau_{sm} = \frac{1}{l_{es}} \int_0^{l_{es}} \tau_s(x)dx \quad ; \quad \tau_{zm} = \frac{1}{l_{es}} \int_0^{l_{es}} \tau_z(x)dx \tag{6}$$

For the bond laws according to [6]:

$$\tau_s(x) = C_s \cdot s^{n_s}(x)dx \quad ; \quad \tau_z(x) = C_z \cdot s^{n_z}(x)dx$$

It can be written

$$\tau_{sm} = \frac{1}{l_{es}} \int_0^{l_{es}} C_s \cdot s^{n_s}(x)dx \; ; \; \tau_{zm} = \frac{1}{l_{es}} \int_0^{l_{es}} C_z \cdot s^{n_z}(x)dx \tag{7}$$

The slip distribution s(x) between two cracks must also be known in order to calculate the mean bond stress. In [5] it was shown that it is possible to describe the slip distribution with the following function

$$s(x) = \frac{w}{2} \cdot \left(\frac{x}{l_{es}} \right)^b \tag{8}$$

The exponent b depends on the load level. For reinforcing steel and deformed tendons b can be calculated as follows:

$$b_s = \begin{cases} 2.85 & \text{For } F \le F_R \\ 1.85 \cdot (F_R/F)^{1.5} + 1.0 & \text{For } F \ge F_R \end{cases} \tag{9}$$

$$n_s = 0.3; \qquad C_s = 0.29 \cdot f_{cc}$$
$$n_z = 0.3; \qquad C_z = 0.29 \cdot f_{cm}$$

for strands:

$$b_z = \begin{cases} 2.70 & \text{For } F \le F_R \\ 1.70 \cdot (F_R/F)^{1.5} + 1.0 & \text{For } F \ge F_R \end{cases} \tag{10}$$

$$n_z = 0.27; \qquad C_z = 0.15 \cdot f_{cm}$$

for smooth tendons:

$$b_z = \begin{cases} 2.40 & \text{For } F \le F_R \\ 1.40 \cdot (F_R/F)^{1.5} + 1.0 & \text{For } F \ge F_R \end{cases} \tag{11}$$

$$n_z = 0.17; \qquad C_z = 0.605 \cdot \sqrt{f_{cm}}$$

With the known slip distribution it is now possible to give the mean bond stress τ_{zm} and τ_{sm} directly:

$$\tau_{sm} = \frac{C_s}{1 + b_s \cdot n_s} \cdot \left(\frac{w}{2}\right)^{n_s}; \quad \tau_{zm} = \frac{C_z}{1 + b_z \cdot n_z} \cdot \left(\frac{w}{2}\right)^{n_z} \tag{12}$$

The relation between τ_{zm} and τ_{sm} is:

$$\xi = \frac{\tau_{zm}}{\tau_{sm}} = k_b \cdot k_s \cdot \frac{C_z}{C_s} \qquad (13)$$

where

$$k_s = \left(\frac{w}{2}\right)^{n_z - n_s} \qquad k_b = \frac{1 + b_s \cdot n_s}{1 + b_z \cdot n_z}$$

Considering the fact that the crack width in reality is between 0.1 and 0.3 mm, the factor k_s can be approximately replaced by the following constants:

for deformed tendons: $k_s = 1.00$
for strands: $k_s = 1.05$
for smooth tendon: $k_s = 1.34$

4.3. Calculation of the Bond Force T_{um}

The bond force is the part of the force which is transfered from the steel to concrete via the bond after the crack formation. This can be calculated generally as follows

$$T_u = \tau_{sm} \cdot U_s \cdot \frac{s_r}{2} \qquad (14)$$

The determination of the the bond force is simple in the case of single cracks because at the end of the transmission length the strain of concrete equals the strain of steel. With this boundary condition the bond force can be expressed with $\alpha = E_s/E_c$ and the reinforcement ratio ρ as follows:

$$T_u = \frac{F}{1 + \alpha_\rho} \qquad (15)$$

In the case of stabilized cracking, the bond force depends on different parameters. Its maximal value is the force which leads to the crack formation in concrete between two adjacent cracks. It can be written as:

$$T_{u,max} = A_{c,ef} \cdot f_{ct} = t_{sm,smax} \cdot U_s \cdot \frac{s_{rmax}}{2} \qquad (16)$$

where $A_{c,ef}$: effective area of concrete which contributes to the tension capacity of the section

Because the mean bond stress τ_{sm} is a function of the crack width the problem of determination of the bond force T_{um} is solved, if the ratio between the mean w_m and maximal crack width w_{max} as well as the relation between the mean and maximal crack spacing is known.

The assumption of a parabolic distribution curve of the steel stress between two cracks leads to the ratio:

$$\frac{w_m}{w_{max}} = r \cdot \frac{k_1 - 0.33 \cdot (3r - r^2)}{k_1 - 0.67} \qquad (17)$$

where r: ratio between the average and maximal crack spacing
$r = s_{rm}/s_{rmax}$
k_1: ratio between the actual load F and the first cracking load F_R; $k_1 = F/F_R$

The relation between maximal and mean crack spacing for the calculation of the bond force can be assumed als follows:

$$s_{rm} = s_{rmax} \left[1 - 0.5 \cdot \sqrt{1} \left(\frac{F_R}{F} \right)^t \right] \qquad (18)$$

This assumption is based on the following limitations:

- The properties of single cracks characterize the behaviour of the member in the first cracking load level.
  That means: $s_{rm} = 2l_{es} = s_{rmax}$.
- The mean crack spacing decreases with increasing loads F and tends towards the value of the single transmission length l_{es}.
  That means: $s_{rm} \Rightarrow l_{es}$
- The crack pattern changes most strongly around the load level when the first cracking occurs.

Having mathematically fitted the different test results, we can describe the exponent t in eq. (18) depending on the reinforcement ratio ρ as:

$$t = 1 + \frac{2.5}{\rho[\%]} \qquad (19)$$

By this the bond force above the first cracking load can be calculated depending on the external loading as follows:

$$T_{um} = A_{c,ef} \cdot f_{ct} \cdot \left[r \cdot \frac{k_1 - 0.33 \cdot (3r - r^2)}{k_1 - 0.67} \right]^{ns} \cdot r \qquad (20)$$

For the repeated loading the bond force can be calculated as follows [6]:

$$T_{um,lw} = T_{um} \cdot \left(\frac{F}{F_{max}} \right) \cdot \frac{1}{(1+r_n)^{0.07}} \qquad (21)$$

with r_n = $\log(10\,L_w) + L_w/20000$
L_w = number of cycles.

4.4 Comparison: Test - Calculation

The increase of the steel stresses measured in the first test series is averaged.
The comparison of these averaged values with the calculated stresses as explained above shows a good agreement (Fig. 8). For this calculation the bond stiffness of the tendons in the coupling region was reduced to 50%.

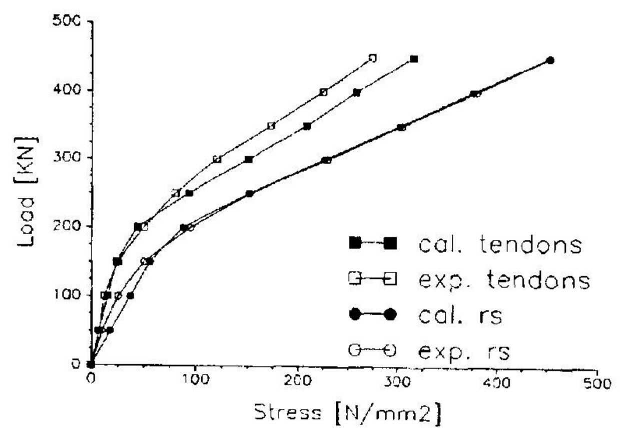

Fig. 8: Comparison Test-Calculation of the stress increase

5. CONCLUSION

The results of the first test series show that only a small reduction in the fatigue strength compared to that of wedge anchors tested in the air. Due to the better bond properties the stress increase in reinforcing steel is higher than that in the tendons. The consequence of this fact is a considerable support for the tendons by the ordinary reinforcement. Furthermore the discontinuity of the curvature within the joint region will be reduced. For the practical application of couplers, it is advisable to provide a minimum of ordinary reinforcement according to German DIN 4227.

The different increase of steel stress in tendons and reinforcing steel can be calculated with the presented simple method.

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