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Fatigue Tests on Couplings of Tendons under Service Conditions

Essais de fatigue des armatures précontraintes couplées, dans des conditions de service

Dauerschwellversuche an Koppelankern unter praxisähnlichen Bedingungen

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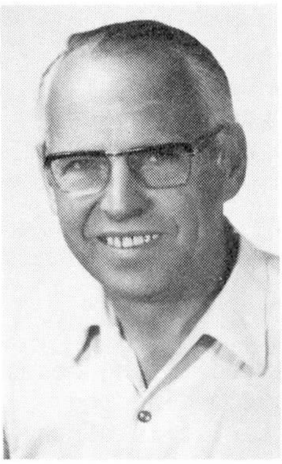

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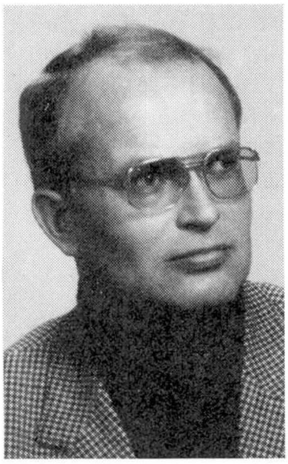

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SUMMARY
Cracks in the construction joints of post-tensioned roadway bridges initiated tests to find out, whether the fatigue strength of the couplings is reduced by being embedded in concrete. In fatigue tests on 14 post-tensioned beams the couplings at the construction joint in mid-span did not behave significantly worse compared with the fatigue loading in 2 million load cycles as applied on acceptance tests with straight bare tendons with couplings.

RESUME
Les fissures dans les joints de construction des ponts-routes précontraints ont conduit à examiner si la résistance à la fatigue des couplages noyés dans le béton était réduite. Des essais de fatigue furent réalisés sur 14 poutres précontraintes avec des couplages au joint de construction, au milieu de la portée. La résistance à la fatigue des couplages ne semblent pas être plus mauvaises que celle des armatures de précontrainte.

ZUSAMMENFASSUNG
Risse an Koppelfugen vorgespannter Strassenbrücken gaben Anlass zu untersuchen, ob die Ermüdungsfestigkeit von Koppelkonstruktionen durch den einbetonierten Zustand nachteilig beeinflusst wird. Dauerschwingversuche an 14 Spannbeton-Biegebalken mit Koppelankern an der Arbeitsfuge in Balkenmitte ergaben gegenüber den in Zulassungsversuchen an freiliegenden Koppelkonstruktionen nach 2 Millionen Lastwechseln nachgewiesenen Schwingbreiten kein signifikant schlechteres Verhalten.

1 INTRODUCTION

In the Federal Republic of Germany multi-span prestressed concrete roadway bridges are often built span by span and concreted in-situ. The construction joint is chosen near the point of zero moment for dead load. All tendons are often anchored at the construction joint for post-tensioning and then coupled for continuation into the next span. As the construction joints remain under service loads theoretically completely in compression there were often only light reinforcements crossing the construction joint. Many bridges are cracked along the joints. This happened especially in hollow-box beams and in the bottom slabs [1,2,3]. The uncracked state as assumed for design does no more exist.

As it is known, the stresses in the reinforcement are increasing with growing cracks. Thus the safety against withstanding the fatigue loading decreases considerably. With the acceptance test for the official authorization of the prestressing system, the admissible fatigue stress is only tested on unembedded tendons. It is well-known that the fatigue strength of embedded reinforcing steel may obviously be smaller than without embedding [4,5]. Regarding prestressing steel fretting corrosion may occur especially when using smooth bars or additional bending of the couplings may reduce the fatigue strength [6,7]. Sufficient results on the behaviour of embedded tendons were so far not available. Therefore it was necessary to realize fatigue tests on coupled tendons under service conditions.

2 TESTS ON EMBEDDED COUPLED TENDONS

2.1 Test Beams

Until now, tests on three typical prestressing systems are executed, using beams as shown by Fig.1 and reinforced as shown by Fig.3. The data concerning the tested prestressing systems are presented in Fig.2. The characteristics of the different couplings are:

- prestressing system DYWIDAG with a short coupling by means of a screwed sleeve,
- prestressing system POLENSKY + ZÖLLNER with a long coupling by means of a stiff coupling bar with thread and
- prestressing system PHILIPP HOLZMANN with a a so-called clamping package.

Tests on the prestresssing system LOSINGER/SUSPA, with a coupling by means of a so-called coupling box, have started.

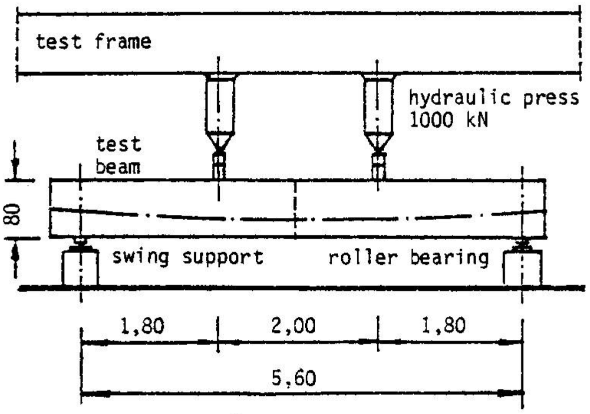

Fig.1 Test set-up

For all prestressing systems, a total prestressing force of 1000 kN was aspired. This led to two tendons for each test beam with the prestressing system DYWIDAG and PHILIPP HOLZMANN. The test beams were concreted in two steps. Diverging from normal construction, the prestressing of each half was executed for simplification from the ends of the beams and not from the internal anchorings at the construction joints. The first test serie which is reported here includes 14 beams. The only varied parameter was the range of the pulsating stress $\Delta\sigma$. The high $\Delta\sigma$-values, which made expect a failure before 1,000,000 cycles were chosen intentionally, because only 40 cycles/minute were executable. Detailed results are published in the report [8] and in [9].

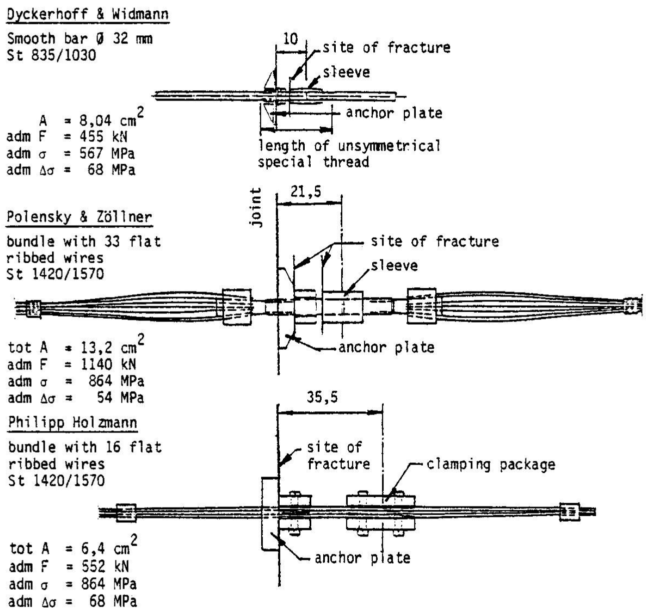

Fig.2 Characteristics of the tendons and their couplings

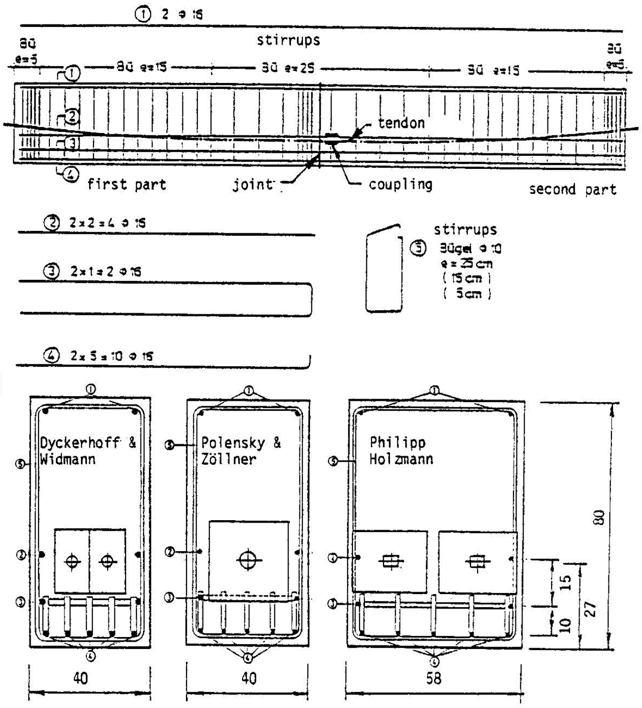

Fig.3 Reinforcement of the test beams and cross-sections

With further planned test series the behaviour under low $\Delta\sigma$-values will be investigated. Furtheron the prestressing in both parts of the beams will be chosen of different value, as this often occurs in reality. According to the regulations for edge distances, the widths of the beams were b = 40 cm with the prestressing system DYWIDAG and POLENSKY + ZÖLLNER and b = 48 cm with the system PHILIPP HOLZMANN. The height of the beams was uniformly chosen as h = 80 cm. The total length of the test beams was 6.30 m and the span 5.60 m. The loading acted approximately in the third points, except with beam no.11, where it acted in mid-span. The tendons were placed centrically at the end of the beams and with an eccentricity of h/6 in mid-span.

The beams were concreted on two successive days, the part with the coupling at last. A smooth casing was used for the joint, as it corresponds to usual execution. The concrete strength was B 45, corrresponding to a cylinder strength of ß = 38 MPa. The injection of the tendons was generally executed directly after the prestressing.

2.2 Measuring Device

Generally about 50 electrical-resistance strain gauges were arranged per tendon. Some of the wire strain gauges were installed directly on the coupling. For interpretation of these test data, the couplings equipped with strain gauges were first of all subjected to calibration tests. Besides of strain measurements on the prestressing steel, also concrete deformations in the compression zone were taken. The opening width of the joint was measured by means of electric-inductance gauges at the height of the tendon axis.

2.3 Test Procedure

The range of the pulsating stress $\Delta\sigma$ = max σ - min σ was chosen in such a way that the beam remained cracked with the minimum stress min σ and the maximum stress max σ was approximately 1.1 times the allowable prestress adm σ. The prestress had to be chosen accordingly. Detailed information is presented in Table 1. The change of stresses, which occurs during the execution of the test, was compensated after 10,000 cycles by altering the applied load accordingly.

3 TEST RESULTS

3.1 Formation of Cracks

Fig.4 shows a typical crack pattern. Exceeding the decompression load, the joint opened immediately because an appreciable tensile strength did not exist. When reaching the maximum load, the height of the crack corresponded to the height of the calculated tensile zone. Near the neutral axis horizontal cracks formed, as they are also typical for elements with unbonded tendons. Only in that part of the beam, where the coupling was installed, some diagonal cracks formed near the coupling. Flexural bending cracks starting from the bottom formed at some distance from the joint. During the fatigue loading, some cracks elongated and new cracks were initiated. The change of the crack width difference Δw in the joint is presented in Fig.5. It increases at first, then remains constant for a long period and increases once again shortly before failure.

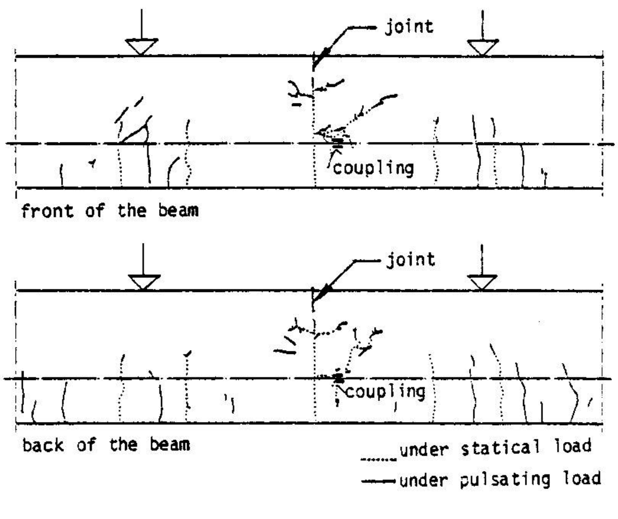

Fig.4 Typical cracks (beam no.10)

3.2 Stresses

The course of the means of the measured pulsating stresses $\Delta\sigma$ for several points of the tendons is presented in Fig.5. The corresponding steel value at the coupling is growing with increasing load cycles but often does not fully reach the calculated value. The stress difference $\Delta\sigma$ decreases sometimes considerably before the failure. The stress differences $\Delta\sigma$ of the prestressing steel near the couplings remain clearly below the value at the coupling. It is also remarkable, that the values for the second part of the beam, where the coupling is installed, remain always somewhat more below the calculated value. At the beginning of the fatigue loading the stress differences increase clearly, reach their maximum and decrease somtimes before the failure. The increase of the stress values at the beginning has to be attributed to gradual bond destruction. This contributes also to the contraction of the compression zone at the end of the fatigue loading. Thereby the distance of the inner forces increases and this explains the decrease of the stress difference before the failure.

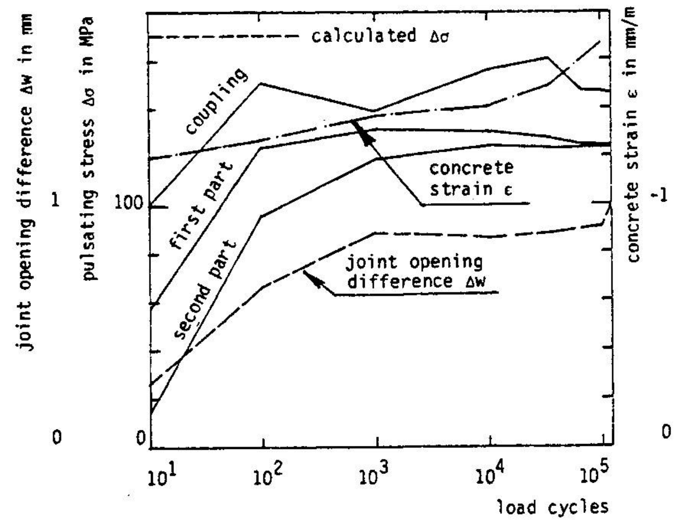

Fig.5 Typical variation of pulsating stress $\Delta\sigma$, joint opening difference Δw and concrete strain ε (beam no.5)

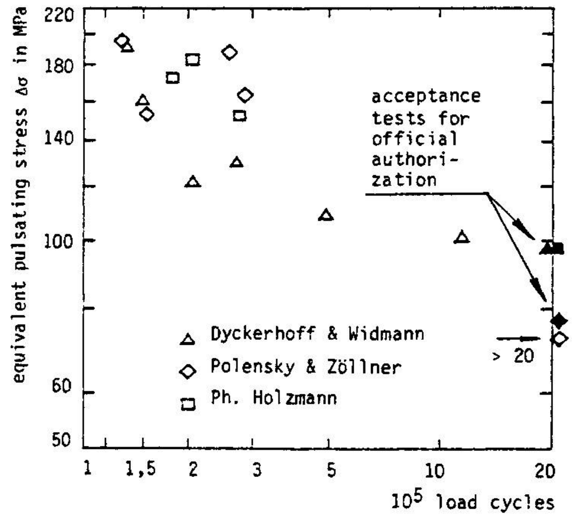

Fig.6 Withstanded load cycles as a function of the applied equivalent pulsating stress

prestress. system	DYCKERHOFF + WIDMANN						POLENSKY + ZÖLLNER					PH.HOLZMANN		
1 beam no.	1	2	3	10	11	12	4	5	6	13	14	7	8	9
2 prestress	373	390	405	393	426	348	644	654	595	570	365	623	644	651
3 max σ	665	650	675	650	642	666	905	905	930	962	814	955	878	955
4 mean σ	578	574	583	586	559	593	820	820	820	907	736	879	793	854
5 min σ	491	498	491	522	476	520	735	735	710	852	658	803	708	753
6 calc $\Delta\sigma$	174	152	184	128	166	146	170	170	220	110	156	152	170	202
7 meas $\Delta\sigma$ after 100,000 l.c.	155	134	191	109	116	109	197	150	180	-	170	194	188	175
8 equ $\Delta\sigma$	160	131	192	101	121	109	189	153	195	-	163	152	184	173
9 load cycles / 1000	147	270	133	1139	207	486	254	150	132	2000	287	274	202	178

Table 1 Stresses in MPa and wihstanded number of load cycles

Table 1 contains all stress values for the prestressing steel. The stresses given in line 2 were measured during the prestressing procedure and correspond to the stresses under permanent load.

The stresses given in the lines 3 to 6 were determined by computation. The measured stress differences $\Delta\sigma$ were transformed to an equivalent constant value equ $\Delta\sigma$, given in line 8, according to the hypothesis of Palmgren/Miner and Swanson.

3.3 Fatigue Strength and Fracture Reasons

The withstanded number of load cycles for different pulsating stresses $\Delta\sigma$ are presented in Fig.6 in double-logarithmic scale. The withstanded stresses for 2 million load cycles from the acceptance tests are also shown for comparison. Tests with embedded tendons do not seem to result in reduced fatigue strength compared with acceptance tests on unembedded tendons. As shown in Fig.2 fractures occured only within the thread or next to the ribs on the wires at the end of the clamp anchoring, just as this was observed with the acceptance tests.

The fractured planes were investigated metallurgically and fatigue fractures could unambiguously be identified. With the prestressing system PHILIPP HOLZMANN only some of all the 16 oval wires of each tendon showed no signs of fatigue failure. This pertained mainly to the upper wires, which failed due to forced rupture, when a certain part of the wires had already failed due to fatigue.

The reason for the observed relative insensibility of the couplings against additional influences under service conditions in the embedded state might be seen in the fact, that the fatigue strength of the couplings is primarily influenced by notch effects, so that influences due to additional bending or fretting corrosion are only less important.

REFERENCES

1. PFOHL, H., Risse an Koppelfugen von Spannbetonbrücken - Schadensbeobachtungen, mögliche Ursachen, vorläufige Folgerungen. Mitteilungen Institut für Bautechnik, Nr. 6, Dezember 1973, S. 161-165.

2. KORDINA, K. and G. IVANYI, Schäden an Spannbetonbrücken im Bereich von Koppelfugen. Technischer Beitrag zum 8. Internationalen Spannbeton-Kongress, FIP, 1978, S. 1-16.

3. KORDINA, K., Schäden an Koppelfugen. Beton- und Stahlbetonbau, Nr. 4, 1979, S. 95-100.

4. WASCHEIDT, H., Dauerschwingfestigkeit von Betonstählen im einbetonierten Zustand. Deutscher Ausschuß für Stahlbeton, Heft 200, Wilhelm Ernst und Sohn, Berlin, 1968.

5. HARRE, W. und U. NÜRNBERGER, Zum Schwingfestigkeitsverhalten von Betonstählen unter wirklichkeitsnahen Beanspruchungsbedingungen. Schriftenreihe des Otto-Graf-Instituts, Stuttgart, Heft 75, 1980.

6. XERXAVINS, Recherche de la valeur optimum de la tension des armatures de precontrainte. FIP Amsterdam 1955, Session Ib, Pap. 5.

7. WASCHEIDT, H., Ermüdungsfestigkeit von glatten und profilierten Spannstählen mit walzrauher Oberfläche. Beiträge der Friedrich Krupp Hüttenwerke AG, Duisburg-Rheinhausen, zum FIP-Symposium "Spannstähle", Madrid, 1968.

8. KORDINA, K., G. IVANYI und J. GÜNTHER, Dauerschwingversuche an Koppelankern unter praxisähnlichen Bedingungen (Koppelfuge im Zustand II). TU Braunschweig, Institut für Baustoffe, Massivbau und Brandschutz, Forschungsbericht, Dezember 1979.

9. KORDINA, K. and J. GÜNTHER, Dauerschwellversuche an Koppelankern unter praxisähnlichen Bedingungen. Bauingenieur, 1982, S. 103-108.