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Resistance to fatigue and prediction of lifetime of wire tendons cast into concrete up to 10^8 cycles

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Discussion Paper

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Abstract

Usually for verification of compliance, the fatigue resistance of prestressing steel is determined from tests of naked specimens at 2 million cycles. However, for design the fatigue resistance of tendons cast into concrete, is substantially lower. To verify the resistance of existing older prestressed concrete bridges and for the design of new bridges, S-N curves of prestressing steel in curved steel ducts embedded into concrete are needed. In bridges, the load cycles due to heavy vehicles may rise up to about 10E8 cycles or even more. Previous tests with curved tendons in steel ducts primarily cover a range of up to about 20 million cycles. Thereby no real endurance strength has been estimated jet. Hence the S-N curves given in Eurocode 2 and Model Code 2010 are defined hypothetically for a range from 10^6 up to 10^8 and are not based on test results. The reason is that experimental investigations in a range up to 10^8 cycles are very expensive and also demand a very long duration.

Essential progress results from the development of an optimized test setup that allows a frequency of 10Hz for the applied load cycles. Therewith, the experimental investigations up to 10^8 cycles have been done by means of prestressed concrete beams with embedded curved tendons in steel ducts.

Furthermore, procedures to also forecast the lifetime in the case of very low stress ranges respectively the remaining lifetime of a running test had been developed in conjunction with an interdisciplinary research project. The procedures are based on refined statistical analysis of the extensively measured data including increase of crack width, strains, sound emission etc. Additionally the analysis of the latter leads to some interesting new perceptions.

Keywords: post-tensioning; concrete beams; fatigue behaviour; endurance strength; large-scale test; cyclical load; S-N curves;

1 Introduction

Today the maintenance of existing buildings is gaining more and more importance compared to the construction of new buildings. The current design codes have been developed over decades, always adapting new design approaches current at that time. Therefore, recalculations of older existing buildings often lead to deficiencies concerning durability, strength and performance capability. Moreover, the external impacts became more extensive, complex and intense.

In essence, two major influences substantially contribute the time dependent loss of load bearing. Firstly there is the influence of material corrosion, which can be caused, by e.g., chlorides or similar external impacts. Also, material fatigue may cause a slowly progressing material damage, due to frequently recurring cyclical loads. Cyclical loads may occur at, e.g., offshore-structures, bridges, noise protection walls beside high-speed rail links or at mechanical components, which are subjected to rotating machines.

During the last decades, a stead and strong ongoing increase of traffic has been recorded. Mainly the amount of heavy trucks is of great importance for the transport network and particularly for bridges. According to current forecasts, a further increase of road transport by 80 percent is expected in 2025 [1].

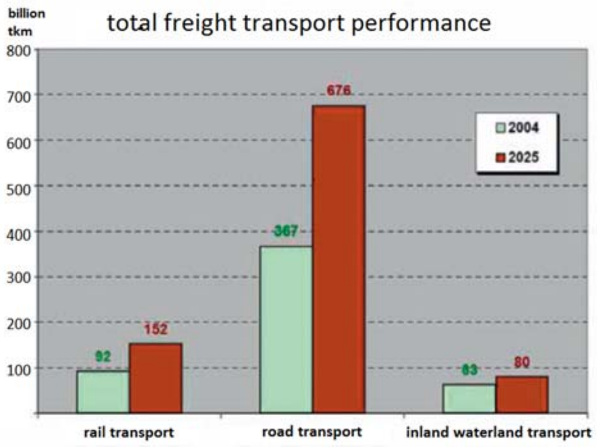

Figure 1. Forecast study of transport traffic [1]

The number of load cycles up to 10^8 might be reached over the scheduled useful life of a bridge. Recent traffic counts show that the motorway ring road at Cologne has a traffic volume of approximately 10,000 trucks per day. This means that after nearly 30 years, the amount of heavy-traffic, crossing a bridge deck for example, will have risen to about 10^8.

The fatigue behavior of a material or a component can only be determined by empirical experiments. Therefore, many experiments to analyze the fatigue behavior of prestressing steel, as an important component in concrete bridges, have been executed. A range of different documented tests is given in [2–5].

The execution of experiments using a large number of load cycles is very time-consuming and cost-intensive. Therefore, the runtime of all already documented and executed tests has been limited to load cycles up to $2x10^7$. Fatigue behavior of concrete structures with prestressed steel embedded in concrete needs to be tested on a large-scale. Thereby, the influence of interaction and friction between concrete, curved steel duct and the single strands are taken into account. As the test frequency for large-scale tests is very limited, the runtime for a single test could take several weeks or months to exceed the maximum number of $2x10^7$ load cycles.

As already described, there is only a limited amount of already conducted long running tests concerning the fatigue behavior of prestressing steel embedded in concrete. Therefore, further research is required, especially on long running test with very low stress range resp. a very high number of load cycles.

During the course of the research project for the state enterprise Landesbetrieb für Straßenbau NRW and an additional project SFB 823 "*Statistical modelling of nonlinear dynamic processes*" sponsored by the Deutsche Forschungs-gesellschaft (DFG, German Research Foundation) large scale test series in ranges up to 10^8 load cycles are carried out at the University TU Dortmund. For the first time, a prestressed concrete girder with internal post-tensioned tendons in a curved steel duct was tested for more than 10^8 load cycles. Consequently, for the first time, data could be determined in the endurance strength range.

Through the co-operation with the faculty of statistics at the TU Dortmund in the course of the research project SFB 823, the testing series has also been accompanied by statistical evaluation. Furthermore, forecasts have been made to estimate the testing runtime. Those forecasts for the expected runtime were an important issue in the optimization of the long-term planning of the whole test series. The development of the statistical methods was based on the measurement results during the running time and after the end of the test. Of particular importance here was the crack width and its rapid increase in case of a wire break. Based on the Bayesian analyses and data depth, the increasing crack widths and the corresponding load cycles were used to develop own statistical models. The aim was to find the most reliable method to simulate and predict the experiments.

2 Current state of research

The first test studies about fatigue behavior of prestressed concrete girders were probably carried out by *Magnel* [6] in 1950 and *Birkenmeier / Jacobsohn* [7] in1957. Until the 1980's mainly experiments on pretensioned concrete girders have been conducted. In particular the USA have experimented on this matter extensively. In [8] a summary of these experiments on pretensioned girders as well as results from further experiments in a time span between 1954 to 1982 is listed. Essentially, experiments on posttensioned concrete girders under cyclical load have been carried out first from the 1980's until today.

In figure 2, the experimental results of *Oertle et al.* [9], *Abel et al.* [4], *Bökamp* [5], *Voß et al*. [10] , *Müller* [11], *Eskola* [3] and *Hegger/Neuser* [12] are depicted contrasting the characteristic S-N curves according to recent valid standards DIN EN 1992-2/NA [13] and Model Code 2010 [14]. In evidence only two wire breaks lie below the S-N curves. Also, the first inclination according to Model Code 2010 [14] $k_1 = 5$ regarding the experimental results has been chosen slightly too flat.

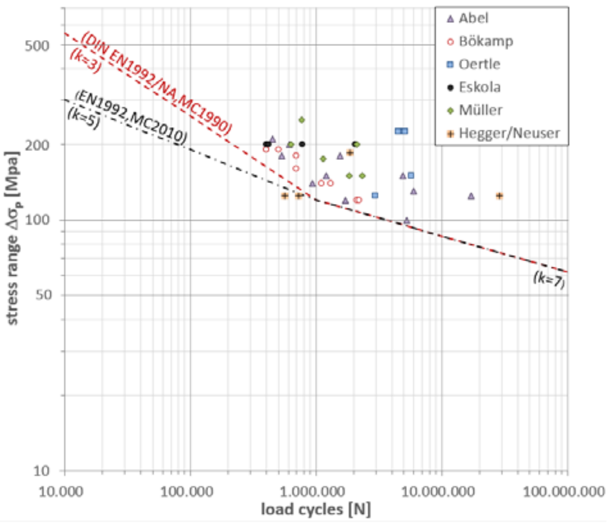

Figure 2. Test results and currently valid S-N curves

In the test series of *Abel et al.* [4] an experiment with a stress range of 100 MPa has been carried out. This experiment remains the only example with a load below the boundary of 120 MPa. Continuing experiments in the area of endurance strength are not known by the current state of science. Hence, the choice of inclination of the second branch appears to be merely hypothetical regarding the recent valid standards. The cause for the limited amount of experiments with low stress range in the area of endurance strength lies in the drawn-out time span necessary for the experiments. Therefore, in the area of the endurance strength persists an urgent need for research.

3 Experimental set-up

3.1 Extraction of the prestressing steel

In the course of a research project sponsored by Straßen NRW five large scale tests (*TR01 – TR05*) on prestressed steel embedded in concrete have been conducted in total [15]. Here, the opportunity arose to extract prestressing strands from a demolished building for the purpose of experimenting. The building in question was the bridge BW67, *Bhf. Westhofen* in Hagen, Germany.

The extraction of the prestressing steel happened directly during the demolition work. For this, an 11 m long tendon in a steel case was exposed by the demolition company, and subsequently, 35 strands were extracted. For the purpose of further use, those strands were examined on previous damage, cleaned and tested regarding actual tensile strength at the TU Dortmund. At this juncture, it was possible to determine the steel grade St1570/1770.

Figure 3. Opened tendon after demolition [15]

3.2 Girder of the test series *TR01 – TR05*

Finally, it was possible to select 5 intact strand bundles from the extracted prestressing steel, which could be used for the test girder. The prestressing steel had been strained at a length of 2m for the curved tendon with a minimum radius of r = 5m in a region of the test girder with pure bending without shear.

The tendons were stressed under the directive *DIN 4227 (1953)* [16]:

$$\sigma_p = 0.55 \cdot \beta_z \quad (\beta_z \triangleq f_{pk}) \qquad (1)$$

After having been prestressed, the tendon was subsequently grouted. The implementation of the test girder took place 28 days later at the earliest. Furthermore, the reinforcement steel B500B and a concrete strength class greater than, or equal, to C45/55 was used.

A recess in midspan of the girder in conjunction with a steel contact element ensures the unambiguous definition of the center of the compression zone in the upper cross-section part and from this, the exact inner lever arm and tension force in the tendon.

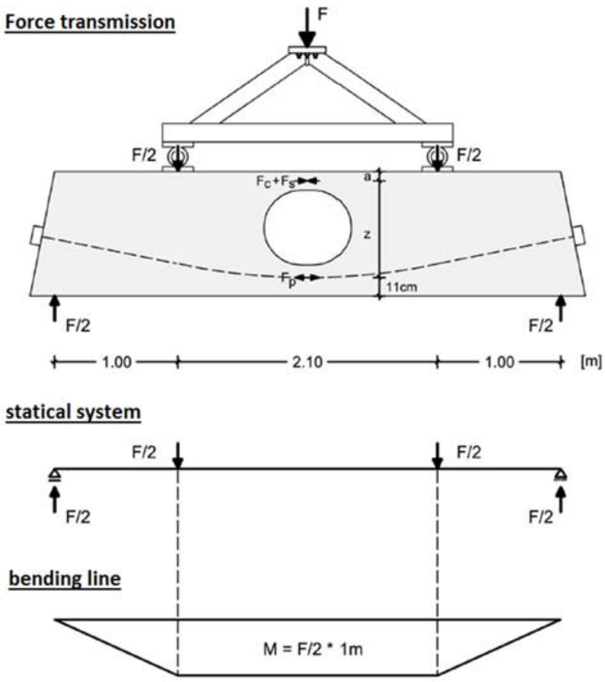

Figure 4. Statical system of the test series TR01 to TR05 [15]

In order that in this area, a bearing effect of the concrete in tension could be excluded, a wire mesh was implemented to force the predetermined breaking point.

The test series was planned as a 4-point bending test. The total dimensions of the test girder measured l / h / b = 4.50 / 1.00 / 0.30 m.

To determine the steel extension in the compression zone, strain gauges (DMS, Dehn-ungsmessstreifen) on the reinforcement bars (Ø30 mm) were applied. Inductive displacement transducers were used to measure the deflection of the beam, the crack width and concrete strains. Load cells at defined load transmission points and the supporting points were used to monitor the current load during the experiment. The testing machine can apply a cyclic load at maximum +/- 2500 kN.

Beside to the measurement of the crack width by the inductive displacement transducters, a microphone and an accelerometer were connected on the strain anchors. The microphone recorded the acoustic noise followed by the wire break in the tendon. In addition, the microphone measurement was complimented by the acceleration measurement of the impulse caused by the wire break. By combining both measurements, it was possible to exclude erroneous measurements, e.g. noise in the test hall. It was possible to identify a wire break unambiguously only if both, the microphone and acceleration measurement, were recorded in addition to an erratic increase of crack width.

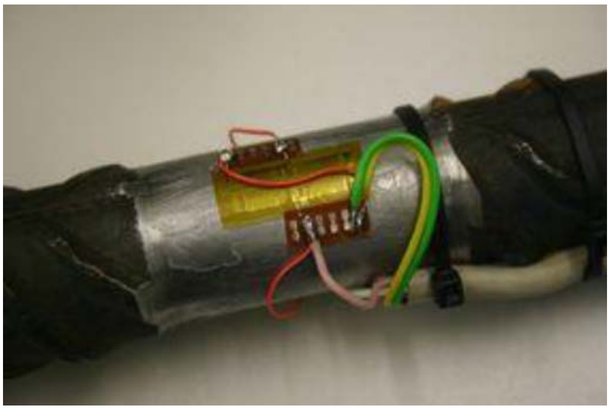

Figure 5. DMS on reinforcement bar [15]

3.3 Girder of the test series *SB01 – SB06*

Six additional large scale tests sponsored by the DFG have been carried out in the course of the research project SFB 823. It was not possible to conduct these experiments with the tendons from the demolished bridge, as it was for the test series "*TR*". However, comparable modern tendons have been obtained from a prestressing steel manufacture. To reduce potential variation during manufacturing, only tendons from the same production charge have been applied.

To ensure the highest possible comparability with the test series TR, the same test setup has been used as a basic principle. A few modifications, like a steel-link in the pressure area in the center of the girder, and the prestressing of the anchoring rods increase the stiffness of the whole test stand and the test frequency, whereby the duration has been reduced. The test frequency was set at 1.5 – 2 Hz for the first test series and was optimized up to 10 Hz for the second test series.

3.4 Test procedure of both test series

The test procedure of both test series followed the same basic principle. Firstly, the whole press force was applied to the concrete girder. The load was increased continuously until an initial crack in the tension zone appears and a bearing effect of the concrete in tension has been excluded. Initially the girder has been released in a way, so that the load can be increased up to the respective medium load range.

From the measurement of the crack width during the load increase, the decompression point and the actual preload force has been determined (see fig. 7).

After that, the fatigue strength of the embedded prestressing steel was tested under a constant cyclic loading until a critical number of wires had broken due to fatigue and the remaining section was not able to withstand the remaining load.

The tests *TR01* to *TR05* were carried out in a time span between April 2009 and March 2010. The stress ranges for the cyclical load lie between 455 MPa und 98 MPa.

Figure 6. Test rig during the test procedure [17]

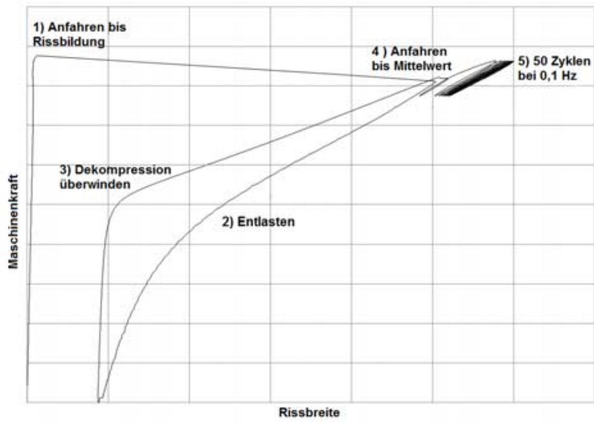

Figure 7. Exemplary presentation of the continuous measurement

The test running time greatly depends on the level of the stress range. The test girder *TR02* was tested with a stress range of 455 MPa and a test frequency of 1.5 Hz. The test ended after just a few hours and 21,100 load cycles. Because of the lower stress range of 98 MPa, it was possible to increase the test frequency of the girder *TR05* up to 2.0 Hz. Nevertheless, the total duration of the test still reached 13 weeks until it was prematurely stopped after 15,069,990 load cycles, as it was impossible to foresee the breakdown of the test girder.

The test series *SB* with the “modern” prestressing steel was carried out between June 2014 und February 2016. Compared to the first test series (*TR*), lower stress ranges between 200 MPa down to 50 MPa were tested. The test girder *SB06* was tested for 20 weeks on the stress range of 50 MPa and broke through the barrier of 10^8 load cycle. After a total of 108,257,340 load cycles the test was stopped as well, as there was no possibility to foresee the breakdown.

Following up to the test *SB06,* the test girder of *SB06* was tested once more under the new designation *SB06a* with a higher stress range of 120 MPa. The objective of this second test phase is to prove the already occurred damage during the first test phase. The first wires of test *SB06a* already broke after nearly 100,000 load cycle. This indicates already existing wire damage at the time of the forcible end of the previous test *SB06*. An almost infinite number of load cycles at the endurance strength with a stress range of 50 MPa can, therefore, be excluded.

By increasing the test frequency, the technical requirements of the testing machine elevates also. To control the actual load during the tests, a load cell was applied in the compression zone. It showed that even at a test frequency of 10 Hz, the matured load matched the calculated. Only the modification of the steel link in the compression zone for the second test series *SB* made it possible to control the actual load. Currently, the tensile force in the tendon can be confirmed.

4 Test results

The measurement values have already be evaluated during the test process. During the experiment runtime, the crack width in midspan of the girder was measured continuously. As soon as a wire has broken due to fatigue, the measurement of the crack width showed a sudden increase. The amount of increase depends on the total number of already broken wires. The more wires were broken, the greater the sudden increase was.

Taking into account the microphone and acceleration measurement, the number of load cycles related to a wire break can be assigned precisely. The maximum error range was about 10 load cycles.

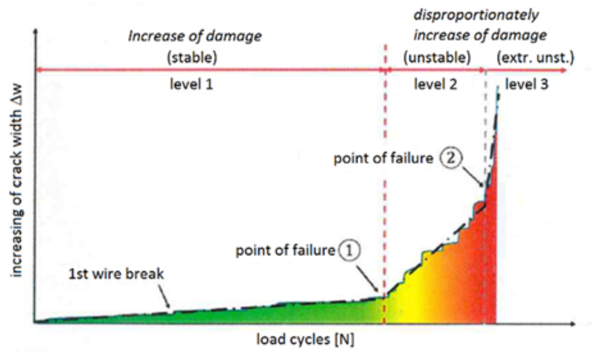

Figure 8. Visualization of a test process [17]

To evaluate the test results, the measurement values of the increasing crack width was plotted over the whole test duration. A single wire break was described by a sudden increase. Therefore, a steplike course was measured.

Furthermore, during each wire break, the pretensioning steel area was reduced, and the stress range of the remaining steel area was increased. The intensity of the increase of the crack width depended on the level of the stress range. The increase of the intensity is shown in figure 8. Accordingly, a test process is divided schematically into three levels.

During the first stable level, the crack width has grown constantly, even after the first few wire breaks. The course at the second level is characterized by a strong increase of the crack width and a closer interval between the wire breaks. The transition from level 1 to level 2 is defined as point of failure 1. The point of failure 2 describes the transition from level 2 to level 3, which indicates the failure of the component by a disproportionate increase of crack width.

A summary of the significant test results is listed in tables 1 and 2. The test *SB06* was stopped after only one wire break. Therefore, the points of failure 1 and 2 could not be determined.

Table 1. Summary of the conducted tests of both tests series TR and SB

test	low stress [Mpa]	high stress [MPa]	Stress range [MPa]	test frequency [Hz]	end of test	number of wire breaks at the end	total number of load cycles
TR01	577	777	200	1.5	failure	15	3,388,136
TR02	574	1029	455	1.5	failure	9	206,209
TR03	883	1083	200	1.5	failure	12	3,473,643
TR04	882	1032	150	1.5	stop	6	5,465,189
TR05	885	983	98	2.0	stop	3	15,069,990
SB01	900	1100	200	3.0	failure	17	5,657,301
SB02	900	1000	100	6.0	failure	18	16,193,259
SB03	900	960	60	10.0	failure	18	85,157,449
SB04	900	980	80	10.0	failure	20	21,625,421
SB05	900	980	80	10.0	failure	19	66,471,804
SB06	900	950	50	10.0	stop	1	108,273,608
SB06a	900	1020	120	10.0	failure	1+16	2,289,210

Table 2. Summary of the conducted tests of both tests series TR and SB

test	stress range [Mpa]	1st wire break	failure point 1		failure point 2	
		number of load cycles	number of load cycles	number of wire breaks	number of load cycles	number of wire breaks
TR01	200	1,027,503	2,617,902	5	3,323,244	13
TR02	455	107,843	159,282	3	203,051	8
TR03	200	906,628	2,664,148	6	3,314,813	10
TR04	150	2,441,109	4,964,423	4	-	-
TR05	98	4,222,271	>15,069,990*)	>4	-	-
SB01	200	931,621	4,684,395	5	5,552,588	13
SB02	100	3,075.,17	11,288,906	7	15,790,498	15
SB03	60	36,175,800	70,199,508	9	81,013,476	13
SB04	80	1,459,836	10,793,535	9	19,514,991	16
SB05	80	12,823,560	58,832,957	6	64,953,157	13
SB06	50	28,616,915	>108,273,608*)	>1	-	-
SB06a	120	143,8787	724,161	5	1,454,311	9

*) theoretical value, test was stopped prematurely

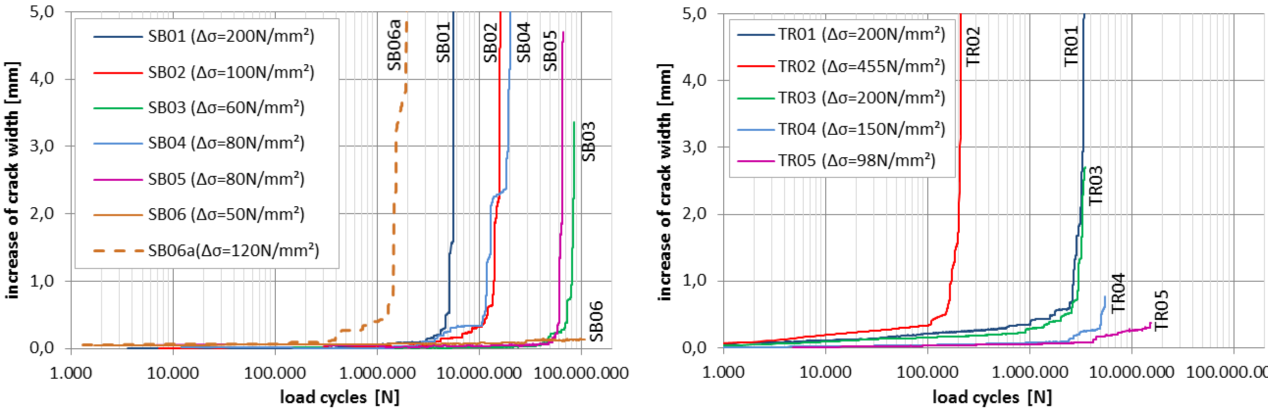

Figure 9. Increase of the crack width compared to both test series

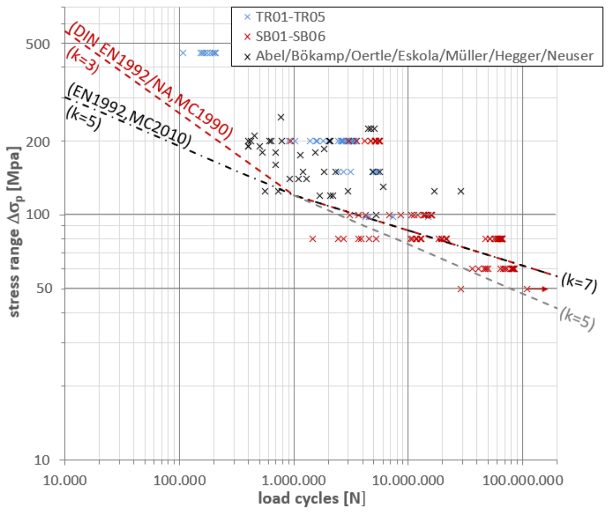

Figure 10. Test results of both test series taking into account the results of figure 2 and currently valid S-N curves with proposal $k_2 = 5$

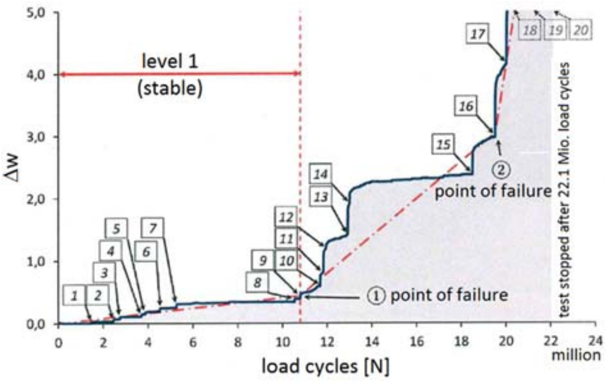

Figure 11. Overview of wire breaks and points of failure of test SB04 [17]

In figure 9, the test results are presented graphically. Every single wire break is plotted according to the stress range $\Delta\sigma_p$ and the corresponding load cycle N. Additionally included are S-N curves by *DIN EN 1992/NA* and *MC 2010.* It is obvious that many wire breaks according to a stress range at less than 100 MPa lie below the S-N curves and the assumption of the second branch ($k_2 = 7$) is too flat. Based on the results of the test series *TR01-TR05* and *SB01-SB06* in [17], a proposal for the second branch ($k_2 = 5$) is given. The inflexion point should remain unchanged at $\Delta\sigma_{Rsk} = 120$ MPa and 1 Mio. load cycles. The inflexion point $\Delta\sigma_{Rsk}$ usually describes the transition to the endurance strength. As this area has not been established by tests so far, an additional change should be avoided.

With the exception of the first 7 wire breaks of the test *SB04* and the first wire break of *SB06* all breaks lie above the S-N curve with the proposal $k_2 = 5$.

The exact reason for the premature wire breaks of the test *SB04* could not be clarified conclusively in [17]. As the affected number of wire breaks is 7, the presumption is that one whole strand of the tendon could possibly be pre-damaged by the production, transport, storage or installation in the test girder. The further test procedure after the 7 wire breaks was largely stable up to the failure point 1 with 10.8 Mio. load cycles. This fact gives rise to the presumption that the 7 wire breaks are caused by a single strand with premature damage.

4.1 Determination of the crack width

The course of the measurement value of the crack width is the primary indicator for the identification of a wire break. In [17], a formula was developed to calculate the increase of the crack width. The crack width depends on the number of already broken wires of the tendon.

$$w(t) = \frac{(1 - k_t(t)) \cdot (\Delta\sigma_{Pr}(t))^2 \cdot A_p(t)}{0.72 \cdot \pi \cdot f_{ctm} \cdot E_p \cdot \sqrt{A_p(t)}} \qquad (2)$$

$k_t(t)$: function for describing the solidity value at the time t

$\Delta\sigma_{pr}(t)$: difference of stress in the tendon depending on the remaining surface of prestressing steel at the time t (see fig. 13)

The formula described above can be used to recalculate the process of the increase of the crack width. As the number of the load cycle in case of a wire break, the surface of the prestressing steel A_p, the E-module E_p and the load were known, the recalculation of the crack width was possible (see. fig. 12). Only the tensile strength of the concrete was not known, and had to be calculated from the measured cube compression strength.

$$f_{ctm} = 0.3 \cdot (f_{ck,cube})^{2/3} \qquad (3)$$

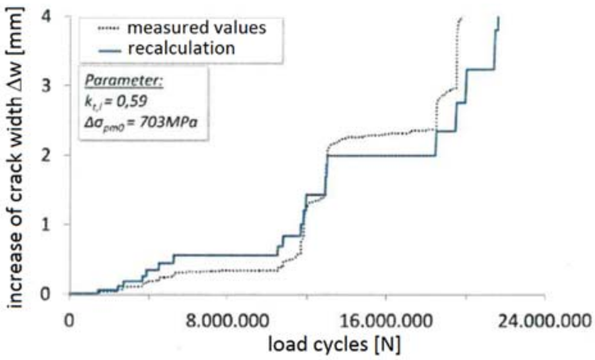

Figure 12. Recalculation and measured values (SB04, $\Delta\sigma_p$ = 80 MPa) [17]

The stress due to the prestressing σ_{Pm0} in the tendon could be determined from the decompression point (see fig. 7). She stress difference $\Delta\sigma_{Pr}$ complies with difference between maximum stress in prestressing steel as result of maximum load $\sigma_{P,max}$ and σ_{Pm0}.

The recalculation was in good agreement with the actual course of the measured values. The calculation assumed in accordance with the formula that any wire break happens in the separating crack in the middle of the girder. The surface of the prestressing steel would theoretically be reduced in the exact same location.

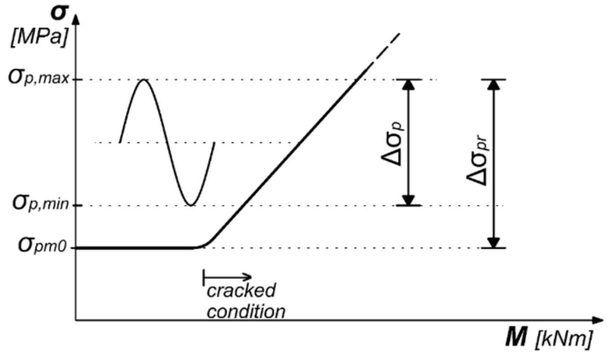

Figure 13. Definition of stress ranges [17]

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But the examination of the extended tendons showed that the wire breaks occurred slightly offset from each other. Wire breaks, which occurred not exactly in the separation crack had less effect to the increase of the crack width.

4.2 Determination of the difference load cycles

Since the actual numbers of load cycles N_i of each wire break were known, the difference between two consecutive wire breaks ΔN_i could be determined (see fig. 14).

The corresponding stress range $\Delta\sigma_p(i)$ depended on the number of already broken wires *i* and can be assigned to the according difference load cycle ΔN_i.

The total number of value pairs of both test series *TR* and *SB* is 129 (see fig. 15). For this purpose two fundamental assumptions were applied. The first assumption is that a possible damage prior (ΔN_{i-1}) to the considered period (ΔN_i) was ignored. Secondly, the value of a difference load cycles must be greater than 1.000 load cycles. Otherwise they were ignored.

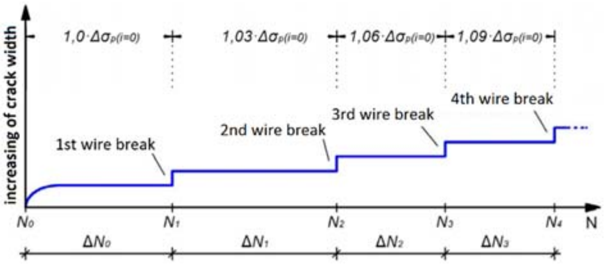

Figure 14. Definition of ΔN_i [17]

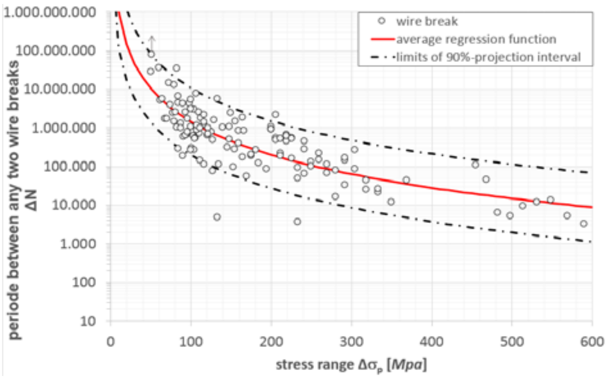

Figure 15. Presentation of the difference load cycles ΔN_i and corresponding stress ranges $\Delta\sigma_{p,i}$

The number of value pairs was the basis for the determination of a regression function as seen in figure 15. Additionally, the boundaries of the 90% projection interval is given. The regression function was determined by a power function:

$$\ln y = \ln a_0 + b_l \cdot \ln x \qquad (4)$$

Because of that, the curve runs asymptotically towards the zero line ($\Delta\sigma_p$ = 0 MPa). Figure 15 shows that the area above $\Delta\sigma_p \geq 100$ MPa can be very well described by the regression function. The value pairs in the area below 100 MPa indicate, that the actual course of the regression function should run asymptotically towards a value greater than zero. This value corresponds to the potential endurance strength. Unfortunately, a final determination of the endurance strength is still not possible, as the total amount of measured value in the area below 100 MPa is 33 out of 129 and below 100 MPa just 12 out of 129.

4.3 Simulation calculation to forecast and to assess test processes

The division of an experiment into single sequences between two wire breaks and the determination of the projection interval enable the further studies by simulation calculations. The “waiting period” between two consecutive wire breaks can be estimated by the regression function. It depends on the stress range.

By the use of a conforming program routine with a random number generator, several test processes can be simulated. In the subsequent sections, the typical procedure for a simulated calculation is described.

The key parameter for the simulation calculation is the stress range at the beginning of the test $\Delta\sigma_{p,i=0}$. Based on this stress range and the 90% projection interval, the corresponding amount of load cycles until the next simulated wire break (here, the first wire break ΔN_1) can be generated. Subsequently the increase of the crack width as the result of the wire break can be calculated by the already known formula.

After the first wire break, the stress range of the remaining surface of the prestressing steel increases as follows:

$$\Delta\sigma_{p,i=1} = 35/(35-1) \cdot \Delta\sigma_{p,i=0} \qquad (5)$$

Once again, the corresponding amount of load cycles until the next simulated wire break (ΔN_2) can be generated. This process can continually be repeated continually until too many wires have been broken and the calculated interval for the next wire break is too small.

The first simulation calculation with a stress range of $\Delta\sigma_p$ = 60 MPa is presented in figure 16 (a). In figure 16 (b), the following 499 simulation calculations have been supplemented. Also, the outer limits and course of the average value of all 500 simulation calculations are specified. For comparison purposes between the simulation calculation and an actual conducted experiment, the measured values of the test *SB03* with the similar stress range of $\Delta\sigma_p$ = 60 MPa have been presented as well.

The total failure of the test girder was observed after 15 wire breaks. According to the simulation calculations with 90% probability, the point of total failure should lie between 39.0 Mio. and 184.1 Mio load cycles. The average value was approx. 91.1 Mio. load cycles. The actual number of load cycle at the end of the test *SB03* was 85.2 Mio., and lies within the predicted range.

The range between 39.0 and 184.1 Mio load cycles is, admittedly, very inaccurate according to current findings. However, it could be used as a qualitative assessment of the test process. The course of the measurement of *SB03,* for example, is very close to the average value. This means that the course of the complete test process corresponds to the expected course of the simulation.

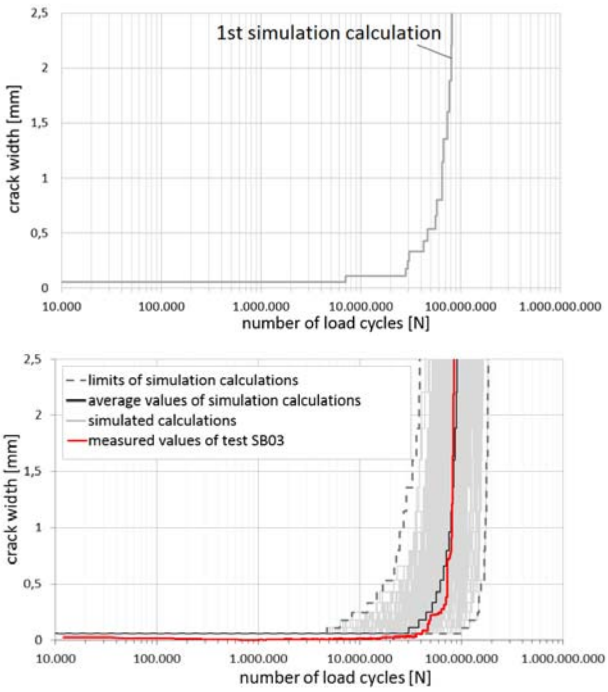

Figure 16. Presentation of 500 simulation calculations and actual results of the test girder SB03 with $\Delta\sigma_p$ *= 60* MPa

Furthermore, the boundary of the simulation calculations can be used as a base for estimations of any future experiments. Thus, for instance, based on the interval of the simulation calculations, the stress range for the test girder *SB06* was assessed to be 50 MPa instead of 40 MPa. The range of the estimated test end of an experiment with a stress range of 40 MPa was calculated between 123.8 and 584.4 Mio. load cycles. This corresponds to a test run time a minimum of 6 up to 24 months. With a stress range of 50 MPa, the estimated number of load cycles at the end of the experiment would lie between 73.4 and 316.4 Mio. and the run time would be divided in half.

4.4 Application of further statistical models to forecast wire breaks

In addition to the above described simulation, calculations of further statistical models have been carried out to forecast possible wire breaks of the course of a test process.

The research project of SFB 823 enabled a close cooperation with the faculty of statistics at TU Dortmund. Based on the test results of the test series *TR* and *SB,* several different statistical models have been developed.

A more detailed description of those models are not given here. For further details and informations corresponding texts can be found in the literature here [18–20].

Figure 17 shows a sample of the results of five different statistical models for a test girder with a stress range of $\Delta\sigma_p$ = 80 MPa. The objective of all models was the determination of the 90% projection interval for the 20th wire break, depending on the number of already broken (observed) wires. It is obvious that the forecast reliability increases with the amount of already observed wire breaks. The continuous horizontal line represents the actual number of load cycles of the experiment *SB04* with a stress range of $\Delta\sigma_p$ = 80 MPa.

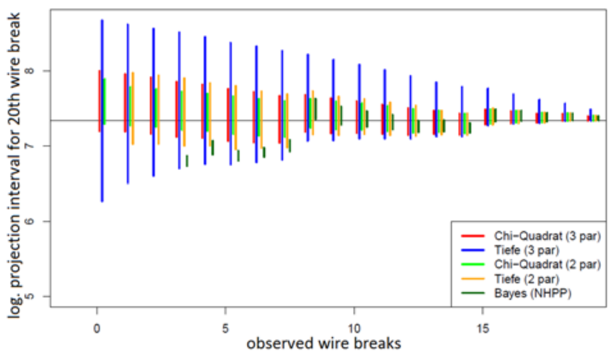

Figure 17. Several projection intervals for the 20th wire break of a test girder with a stress range of $\Delta\sigma_p$ *= 80* MPa

Whichever model is chosen, the start of the test the range of the projection interval is very large and could, therefore, be too imprecise. The implementation of informations, as wire breaks, ensures that the respective interval became more precise.

5 Summary and outlook

Overall 11, large scale tests on the fatigue behavior of prestressed concrete beam with embedded curved tendons in steel ducts have been carried out at the TU Dortmund. Here, for the first time, more than 10^8 load cycles have been applied to the concrete beam. The stress range of the tendon in the concrete beam amounted to $\Delta\sigma_p = 50$ MPa.

The test results regarding the fatigue strength shows, that the assumption of the second branch ($k_2 = 7$) according to current standards (*DIN EN 1992/NA* and *MC 2010)* is too flat. Based on the results of the test series *TR01-TR05* and *SB01-SB06* in [17], a new proposal for the slope of the second branch ($k_2 = 5$) is given.

Despite the results of the test series *TR* and *SB,* the amount of experiments with low stress range in the area of endurance strength is still limited. Therefore, there is an urgent need for research in the area of the endurance strength.

Furthermore, statistical models have been developed to simulate the cost- and time-intense large scale test. Through this, before and during the test process, forecasts of the expected runtime of the test can be given. To get an estimation of the ongoing test end was of vital importance for the test planning of further queuing experiments.

Another significant aspect of a reliable forecast model is the application on existing building structures.

If a building structures has deficiencies concerning resistance against fatigue, it is conceivable to prevent a sudden collapse by installing an appropriate monitoring of the crack width. By observing the increase of the crack width, a beginning failure of the tendon can be noticed and urgent immediately measures, e.g. traffic closures, can be initiated.

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