14th International Conference on Structural Safety and Reliability - ICOSSAR'25
1-6 June, 2025 - Los Angeles California, USA
R. Ghanem, E. Johnson, S. Masri and A. Olivier (Eds)

PROBABILISTIC ASSESSMENT OF A ROAD BRIDGE BASED ON INSPECTION DATA – A CASE STUDY EMPHASIZING CONCRETE STRENGTH

CHRISTIAN KAINZ[1], STEFAN KÜTTENBAUM[1], THOMAS BRAML[1], MATTHIAS HASLBECK[1] AND PETER KOTZ[2]

[1]University of the Bundeswehr Munich
Werner-Heisenberg-Weg 39, 85577 Neubiberg, Germany
christian.kainz@unibw.de
stefan.kuettenbaum@unibw.de
thomas.braml@unibw.de
matthias.haslbeck@unibw.de

[2]SSF Ingenieure AG
Domagkstraße 1a, 80807 München, Germany
pkotz@ssf-ing.de

Key words: Reliability analysis, Existing structures, Main tensile stress criteria, Decompression.

Abstract. This contribution deals with the reassessment and application of probabilistic methods for the reliability analysis on the example of a 130 m long prestressed concrete bridge. The single-cell box girder bridge from 1965 carries a highway over a river and has three spans. The focus of this study is on concrete stress limitations and on the shear load bearing capacity based on the main tensile stress criteria. Most of the current codes are based on the semi-probabilistic safety concept, while certain national regulations explicitly allow the use of a full-probabilistic verification format for the reassessment. The most significant developments and changes in the assessment framework are shown through a compilation of the standards over time. Based on the results from a finite element model and a semi-probabilistic assessment, the limit state for the probabilistic assessment of the shear capacity based on the main tensile stress criteria is derived. In this context, focus is put on the decompression proof. Relevant parameters are characterized and discussed. In view of the possibility to consider actual properties of existing structures through on-site data collection, parameter studies and sensitivity analyses are used to identify important measurement quantities. The concrete properties are derived from destructive and non-destructive material testing methods. In result, the in-situ concrete strength is higher than previously assumed and used to demonstrate the effects of including measured information in a probabilistic assessment. The normative and statistical background of the evaluation is explained. Finally, possibilities and limitations of the use of measurement data during the probabilistic assessment of infrastructure are highlighted and discussed.

1 INTRODUCTION

Many existing bridges are exposed to increasing traffic loads and deviations from evolving and current technical standards. Using data from on-site measurements can significantly enhance the accuracy of reliability evaluations and the load bearing capacity [1]. The need for regulation and the exchange of practical experiences was recognized, which led to the initiation of the national research project "ZfPStatik" in Germany, aimed at the development of a systematic approach for NDT-based reliability analysis of existing bridges. During the collaboration of several partners from academia, industry and a standardization body, a wide practical understanding and application is anticipated. The recommendation for action is aimed at key stakeholders involved in the assessment process. Excerpts of the project work can already be found in [2 - 9]. The present article gives further insight into a case study of a 130 m long prestressed concrete road bridge, which is a three-span hollow box girder built in 1965. At this specific bridge, the concrete strength turned out as a parameter of major interest and influence on the structural reliability.

2 HISTORAL EVOLUTION OF THE VERFICATION FORMAT AND STANDARDS

2.1 Shear force verification format

Initially, prestressed concrete bridges were verified using the national standard DIN 4227. The first version was published in 1953 [10] and established fundamental verification methods for the construction technique of prestressed concrete, that has previously been unregulated. However, no shear reinforcement had to be verified if a certain limit of the main tensile stresses was not exceeded. Only constructive reinforcement was required, the extent of which was left to the discretion of the design engineer. Compared to modern bridges, significantly less shear reinforcement in the webs has been mounted in most existing ones. It was assumed that no shear cracks would occur up to a certain limit stress. However, this approach contradicted the verification format of flexural failure, which is assuming a state of cracked concrete (state II). Primarily due to the reliance on the concrete tensile strength during the verification process, existing bridges designed with DIN 4227:1953-10 [10] tend to have significant deficits in shear capacity. Typically, prestressed concrete bridges do not have inclined shear cracks in the webs under everyday conditions. However, in unfavorable circumstances, e.g., under heavy transport crossings, the formation of shear cracks cannot be entirely ruled out. Therefore, these existing bridges require enhanced monitoring and inspections as specified in DIN 1076 [11].

The risk of was recognized soon and addressed through additional provisions [12] to DIN 4227 issued in 1966, where the verification of the shear reinforcement was generally required. Initially, the verification format was still based on main tensile stresses in the uncracked state (state I). However, a minimum amount of shear reinforcement was demanded.

With the introduction of the Eurocodes in the 2000s, the verification for new bridge constructions is carried out exclusively based on a truss model with a limited compression strut inclination. The inclination of the compression strut is determined according to equation (1):

$$1{,}0 \leq \cot\Theta \leq \frac{1{,}2 + 1{,}4 \cdot \sigma_{cp}/f_{cd}}{1 - V_{Rd,cc}\,/V_{Ed}} \leq 1{,}75 \tag{1}$$

The increased heavy load traffic and the deterioration of existing bridges raised public doubts about the stability and reliability of existing bridges in German society. As a result, the federal department for traffic introduced a national code called guideline for the reassessment of existing bridges [13] to achieve consistent and standardized recalculation methods in 2011. It quickly became evident, however, that many existing prestressed concrete bridges could no longer be verified under the conditions of the guideline. Therefore, a first amendment [14] was published in 2015 that included two additional methods for the shear verification with one verification format considering the shear reinforcement and another verification format without considering the shear reinforcement.

In the verification format considering the shear reinforcement, the compression strut angle may be reduced, following the approach of DIN 4227 to the limit value of tan θ = 0.4, see equation (2). Alternatively, the compression strut angle θ may be determined using the more precise calculated shear crack angle β_r, see equation (3).

$$\frac{4}{7} \leq \cot\Theta \leq \frac{1{,}2 - 1{,}4 \cdot \sigma_{cp}/f_{cd}}{1 - V_{Rd,c}\,/V_{Ed}} \leq 2{,}5 \tag{2}$$

$$\frac{4}{7} \leq \cot\Theta \leq \cot\beta_r + \frac{V_{Rd,c}}{(A_{sw}/s_w)\cdot z \cdot f_{yd}} \leq 2{,}5 \tag{3}$$

Currently, a new second amendment is drafted. Although this document has not yet been officially introduced, it can be used as a basis for verifications if instructed by clients. Here, the shear verification is divided into a concrete contribution and reinforcement (steel) contribution.

2.2 Decompression verification and stress limitation in the SLS

Generally, the verification format for bending or bending combined with normal forces shows significantly smaller differences between historic regulations and the current Eurocode.

For the bending verifications in the serviceability limit state, the stresses in the uncracked concrete (state I) are determined and compared with the permissible stresses specified in the DIN 4227 standard. Again, the utilization of concrete tensile stresses is an essential element here, as in the serviceability limit state permissible concrete tensile stresses are defined depending on the concrete strength class. Additionally, a “simplified decompression” under "half traffic loads" was already included in early versions of DIN 4227.

This simple verification concept was replaced by the decompression verification format with the introduction of the Eurocodes. The decompression verification must be performed using the combination of actions and tensile stresses on the cross-section edge closest to the tendon are prevented. The load combination contains permanent actions, post-tensioning forces multiplied by factors for unfavorable variation in the prestress $r_{inf} = 0.95$ or $r_{sup} = 1.05$, creep and shrinkage, the settlement of supports as well as combination factors ψ reducing temperature effects ($\psi_2 = 0.50$) and traffic loads ($\psi_2 = 0.20$). While the verification format is similar, load specifications have been periodically revised [15 - 19] to fit for current demands since the first German national standard DIN 1072 from 1925 [20]. Earlier versions of DIN 1072 did not include actions resulting from temperature differences between the top and bottom surfaces of the superstructure. They were only introduced with a revision in 1985.

Furthermore, the action due to settlement of supports were often neglected, based on the assumption that its influence would diminish over time due to creep and shrinkage. As restraint-induced internal forces were only partly or not considered at all in earlier standards, older structures sometimes show significant deficits during the assessment of stresses with current verification concepts. Restraint forces are given considerably greater importance in current standards. Points of zero moments between the span and support regions are particularly affected by this.

3 DETERMINATION AND MEASUREMENT OF THE CONCRETE STRENGTH

3.1 Concrete compression strength

For new construction projects, the compressive strength of concrete is determined according to DIN EN 206-1 [21] by testing ready-mixed concrete on cubes. The testing of hardened concrete is standardized in DIN EN 12390 [22]. The fundamental study on the variation in concrete strength was published by Rüsch et al. [23] more than 50 years ago in 1969, where the results of compressive strength tests on cast concrete specimens from 499 construction sites in 20 countries were evaluated. A key finding was that the mean of the standard deviation converges to 5 N/mm² regardless of the concrete strength, which is basis for current code regulations, e.g. EN 1992-2 [24] and fib model code 2020 [25]. More recent studies indicate a significantly smaller variation within concrete compression strength in the construction of new bridges than previously assumed [26, 27].

For existing structures, the determination of concrete strength must be carried out directly on the structure in most cases. In the simplest case, the classification can be based on documented information and a classification according to [14].

The testing of concrete in existing structures is standardized in DIN EN 12504 [28]. The evaluation of results is standardized in DIN EN 13791 [29] and based on measured mean values, minimum individual values, and corresponding standard deviations. For the testing of existing bridges in Germany, there are also regulations in the guideline for the reassessment of existing bridges [13, 14]. Supplementary indirect methods for estimating compressive strength, such as rebound hammer and ultrasonic testing, are described in Parts 2 and 4 of DIN EN 12504 [28].

When determining quantile values based on specimens, the sizes of the samples should be considered in the calculation. Annex D of EN 1990 [30] contains a general methodology for the statistical determination of a single property, which is widely used for practical purposes.

Table 1: Values of the factor k_n for the 5% characteristic value according to EN 1990 [30]

n	1	2	3	4	5	6	8	10	20	30	∞
V_x known	2,31	2,01	1,89	1,83	1,80	1,77	1,74	1,72	1,68	1,67	1,64
V_x unknown	-	-	3,37	2,63	2,33	2,18	2,00	1,92	1,76	1,73	1,64

The conversion of a characteristic value X_k from the mean value m_x and coefficient of variation V_x for a limited sample size n is given with equation (4).

$$X_k = m_x \cdot (1 - k_n \cdot V_x) = m_x - k_n \cdot \sigma_x \tag{4}$$

The factor k_n is given in table 1 and figure 1 and accounts for a limited sample size. The standard case for most practical applications is "V_x unknown". The p-quantile is a random variable, which is assumed to be normally distributed. A lower limit of the p-quantile needs to be established, which will not be exceeded with a probability of P = 1 – α (confidence level). The coefficient k_n is based on a Bayesian estimator considering vague priors.

$$k_n = \Phi_{0.05}^{-1} \cdot \sqrt{1 + \frac{1}{n}} \quad with \quad \Phi_{0.05}^{-1} \approx 1{,}645 \tag{6}$$

The equation for k_n in the case "V_x known" is given in [31] and shown with equation (6).

The approach of DIN EN 1990 [30] is referenced in DIN EN 13791 [29] and the characteristic compressive strength of the in-situ concrete on the existing structure, $f_{ck,is}$, is extended by a minimum value criterion according to equation (7):

$$f_{ck,is} = \min\begin{cases} f_{c,m(n)is} - k_n \cdot s \\ f_{c,is,minimum} + M \end{cases} \tag{7}$$

The given factors k_n for the mean value criterion are identical to table 1, case "V_x unknown". However, a minimum number of n = 8 drill cores is required. The factor M in the minimum criterion increases with bigger concrete strength and varies between 1 to 4. For a minimal value of $f_{c,is,minimum} \geq 20$ N/mm² the factor M is 4. With the characteristic compressive strength of the in-situ concrete on the existing structure $f_{ck,is}$ a classification to a concrete strength class according to DIN EN 206-1 [21] is possible with the tables in DIN EN 13791 [29].

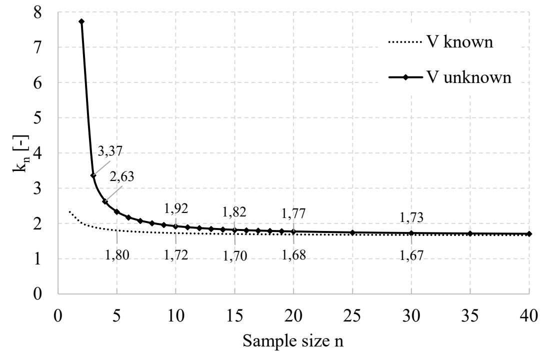

Figure 1: Factor k_n to account for the sample size (1-α = 0,50 and p = 0,05)

3.2 Concrete tensile strength

Concrete tensile strength is subject to increased variability compared to compressive strength, because of the greater effect of local defects, internal stresses due to shrinkage and the shape of aggregates. In addition, greater testing variability must be considered, as tensile tests involve more difficulties compared to compressive tests, e.g., load introduction issues.

A distinction is made between the flexural tensile strength $f_{ct,fl}$, the splitting tensile strength $f_{ct,sp}$ and the centric tensile strength f_{ct}. The tensile strength in codes refers to the centric tensile strength f_{ct}, unless otherwise specified. The mean tensile strength of concrete is determined by a conversion from the concrete compressive strength according to DIN EN 1992-2 [24], see equation (8):

$$\mathrm{f_{ctm}} = 0{,}30 \cdot \mathrm{f_{ck}}^{(2/3)} \tag{8}$$

An extensive overview for further stochastic modeling approaches of concrete tensile strength is included in [32]. The coefficient of variation is generally significantly larger than for the compressive strength, ranging between values of 0.05 and 0.40.

The JCSS Probabilistic Model Code [33] extends the equation from the Eurocode to spatial variability for a particular point i in a given structure j by the coefficient $Y_{2,i}$, see equation (9). $Y_{2,j}$ is a random variable for the consideration of additional variability contributions due to factors not accounted for by concrete compressive strength, e.g., gravel type and size, chemical composition of cement and other ingredients as well as climatical conditions. $Y_{2,j}$ is described as log. normal distributed with a mean μ = 1,0 and a COV = 0,30.

$$\mathrm{f_{ct,ij}} = 0{,}30 \cdot \mathrm{f_{c,ij}}^{(2/3)} \cdot Y_{2,j} \tag{9}$$

The testing of the flexural tensile strength and splitting tensile strength is carried out according to Parts 5 and 6 of DIN 12390 [22]. Environmental conditions (esp. moisture and temperature) during the testing as well as the properties of the hardened concrete and aggregate have a big influence on the results of the flexural tensile strength $f_{ct,fl}$ or the centric tensile strength f_{ct}. The measured splitting tensile strength $f_{ct,sp}$ shows smaller dependence on environmental conditions, so smaller variation can be assumed here. However, measured values for the concrete tensile strength still have greater variability than for compressive strength.

4 CASE STUDY OF A PRESTRESSED CONCRETE BRIDGE BUILT 1965

4.1 Description of the structure

The existing concrete bridge from 1965 carries a Federal Highway over a river in Germany. At both ends of the superstructure, box-shaped abutments with shallow foundations are placed. The superstructure is constructed as a vouted, three-span prestressed box girder with span lengths of 39.0 m – 55.0 m – 39.0 m, see Figure 2.

The superstructure consists of a single-cell box girder with an external width of 7.10 m, see Figure 2. The heights of the box-girder are 1.21 m at the ends of bridge, 1.40 m at the center of the middle span, and 2.25 m above the piers. The webs are 78 cm thick. Originally, concrete guard walls and railings were installed on the bridge. The cantilever arms feature a step in the cap area to accommodate horizontal forces acting in the transverse direction of the bridge.

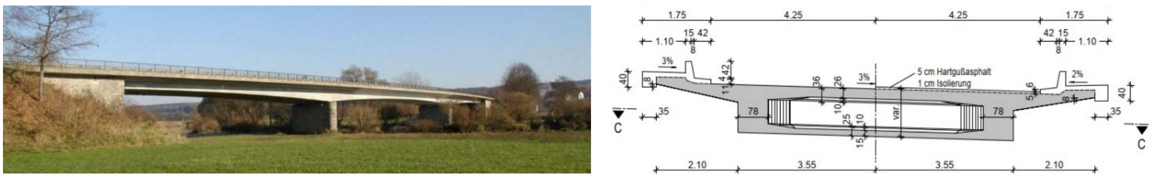

Figure 2: Side view and cross section of the case study bridge

The structure has longitudinal prestressing in the webs and transverse prestressing in the deck and bottom slab. The longitudinal prestressing was implemented using the *SUSPA IV* system with 32Ø7 tendons in St 150/170 steel. A total of 23 tendons were installed in each web. The structure is designed for a load model of bridge class 60 according to the former standard DIN 1072:1952-06 [15] and for military load class 100/50.

The concrete grade is B450, which is classified as a strength class of C30/37.

4.2 Assessment of the bridge with semi-probabilistic methods

The bridge is reassessed according to [13, 14] for the load model LM 1 of the DIN Technical Report 101 [18] for both the ultimate limit state and the serviceability limit state. In both limit states, the verification revealed areas with deficits. The most significant deficiencies are in the shear force verifications in the support areas, which will be discussed in detail in the following.

As equation (10) shows, the design value of the concrete compressive strength f_{cd} directly influences the shear crack angle β_r together with the reinforcement ratio ρ, the yield strength of the steel f_{yd} and the normal stress σ_c.

$$\cot\beta_r = 1{,}2 + \frac{f_{cd}}{70 \cdot \rho_w \cdot f_{yd}} - 1{,}4 \cdot \frac{\sigma_c}{f_{cd}} \leq 2{,}25 \tag{10}$$

The calculations according to stage 2 of the guideline produced more favorable results than those from stage 1. The verifications according to the second amendment reveal a higher numerical capacity compared to the results of the first amendment. However, structural safety could still not be proven in the critical areas around the bearings and piers as the utilization rates are still significantly greater than 1, see figure 3.

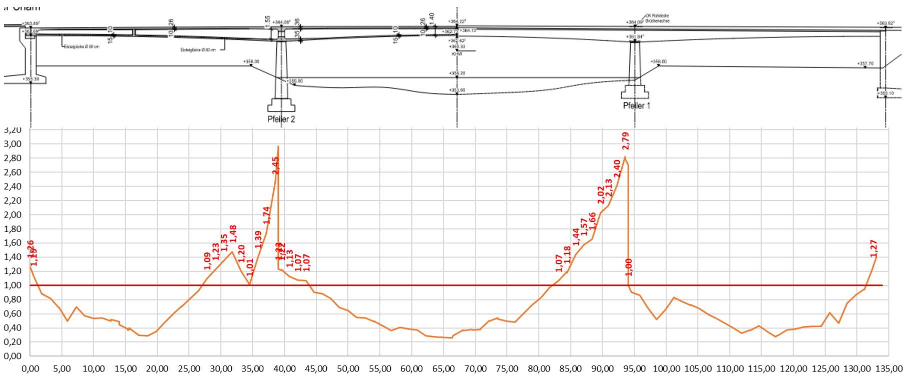

Figure 3: Utilization of the shear verification format considering the 2nd amendment of the guideline for the reassessment of existing bridges

4.3 Full-probabilistic calculation of the main tensile stress criterion

The shear load-bearing capacity was previously analyzed with full-probabilistic methods in [34, 35] with a limit state function for the load-bearing capacity of the tension strut based on the truss model of the Eurocode. The description of the variables as well as the models of the basic variables are given in [35]. In general, a small numerical shear force bearing capacity was identified consistently to the results of the semi-probabilistic assessment in section 4.2.

Therefore, further analysis based on the main tensile stress criterion is conducted. The basis for the formulation of the limit state function is the proposed semi-probabilistic verification format in [36] for the first amendment to the guideline for the reassessment of existing bridges [14]. In a semi-probabilistic assessment, acting normal stresses $\sigma_{l,Ed,i}$ must be smaller than the design value of the concrete tensile strength f_{ctd} reduced by a factor of k_l, see equation (11).

$$\sigma_{l,Ed,i} \leq k_l \cdot f_{ctd} \tag{11}$$

I_y [m^4]		A_c [m^2]	
4,8890		7,2634	
	z_i [m]	$b_{w,i}$ [m]	$S_{y,i}$ [m³]
i = 1	-0,65	0,780	2,56
i = 2	-0,29	0,780	2,68
i = 3	0,80	0,780	1,93
i = 4	0,45	1,480	1,27

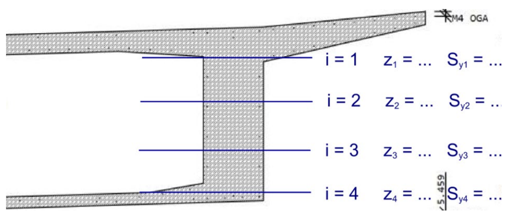

Figure 4: Verification sections at different heights “i” over the the cross-section

In this approach, the main tensile stresses are calculated at different verification sections at various heights "i" of the cross-section. In the example shown here, the verification is performed at four equidistant points along the web height, see Figure 4.

The general approach for the formulation of a limit state function is given in equation (12), with the resistance R and the action E. In the case of the assessment based on the main tensile stress criterion, the limit state function g(x) is given in equation (13) and depends on various random variables on the action side and the concrete tensile strength. Acting normal and shear stresses are based on statical principles and adapted from [36], see equation (14):

$$G = R - E \tag{12}$$

$$g(\boldsymbol{x}) = f_{ct} - \sigma_{I,i} \tag{13}$$

$$\sigma_{I,i} = 0{,}5 \cdot \sigma_{cx,i} + \sqrt{0{,}25 \cdot \sigma_{cx,i}^2 + \left(\tau_{v,i} + \tau_T\right)^2} \tag{14}$$

$$\sigma_{cx,Ed,i} = \frac{N}{A_c} + \frac{M_y}{I_y} \cdot z_i, \qquad \tau_{v,i} = \frac{V \cdot S_{y,i}}{I_y \cdot b_{w,i}}, \qquad \tau_T = \frac{T}{W_T}$$

In the present study, many acting bending moments, torsional moments, normal forces and shear forces are considered as random variables in the limit state function, each separated for dead load, UDL, Tandem-System (TS), temperature, creeping and shrinking. Additionally, material parameters and geometric dimensions are implemented. The statistical parameters for the most relevant parameters are given in table 2.

Table 2: Description of relevant parameters for the limit state function of the main tensile stress criterion

Symbol	Name		Distr.	Mean	COV
U_E	Model Uncertainty actions	[-]	LN	1.00	0.10
U_{RM}	Model Uncertainty resistance	[-]	LN	1.10	0.11
f_{ctm}	Concrete tensile strength	[MPa]	N	3.80	0.30
$N_{x,P}$	Normal force from prestress	[MN]	N	-43.21	0.10
$M_{y,1}$	Bending moment dead-weight	[MNm]	N	-31.39	0.10
$M_{y,P}$	Bending moment prestress	[MNm]	N	39.57	0.10
$V_{z,1}$	Shear force from dead-weigth	[MN]	N	3.64	0.10

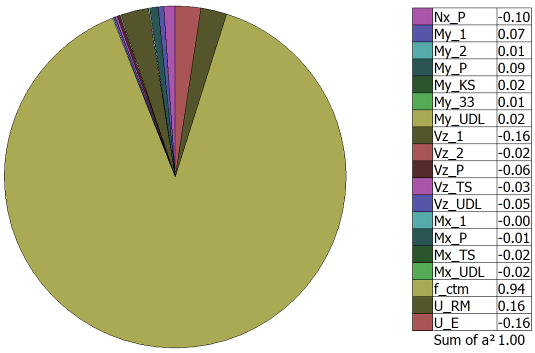

Figure 5: Sensitivity analysis based on alpha-values for the limit state of the main tensile stress criterion

The reliability analysis is performed with the commercial STRUREL (Version 14.00) software based on the First Order Reliability Method (FORM). The full-probabilistic calculation is primarily used for the determination of relevant parameters instead of the actual reliability assessment, because the main tensile stress criterion is subject to several boundary conditions by the code, see section 4.5, which cannot be implemented in the full-probabilistic calculation. In a sensitivity analysis, figure 5, the contribution of each parameter is examined and the relevance of their value and variation is shown. The influence of the concrete tensile strength f_{ctm} is predominant, which is expected in this calculation.

4.4 On-site testing and analysis of concrete strength

Various non-destructive and destructive on-site testing were conducted at the bridge. This contribution focuses on the destructive testing of concrete drill cores. For performed NDT methods, e.g. radar, it is referred to [37, 38, 39].

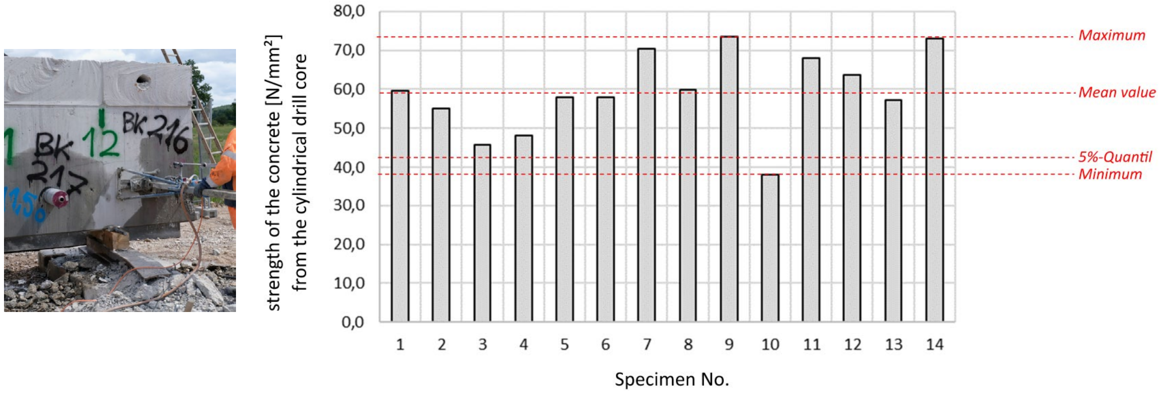

Figure 6: Extraction of the drill cores (left) and compressive strength values of the specimens (right)

More than 60 cores were taken at different locations of the superstructure, of which 14 are analyzed by descriptive statistics, see figure 6. The minimum value is 38.0 N/mm² and the maximum value is 73.4 N/mm². The mean value of the 14 samples is 59.1 N/mm² and the standard deviation is 10.3 N/mm², which is considered large with the respect to [23, 25, 26].

$$f_{ck,is} = \min\begin{cases} f_{c,m(n)is} - k_n \cdot s = 59.1 - 1.84 \cdot 10.3 = 40.1 \\ f_{c,is,minimum} + M = 38.0 + 4 = 42.0 \end{cases} = 40.1\,N/mm^2 \tag{15}$$

The characteristic compressive strength of the in-situ concrete $f_{ck,is}$ can be calculated using equation (15) according to the regulations of DIN EN 13791 [29] and section 3.1. The assignment to a strength class of C45/55 according to EN 206-1 is performed with a table provided in DIN EN 13791 [29]. In this context it should be highlighted that a three class higher concrete strength class is achieved based on the measurement on the structure. A mean concrete tensile strength of f_{ctm} = 3,80 N/mm² corresponds to the strength class C45/55.

Besides the standardized analysis of the drill cores, further Bayesian statistics and Markov Chain Monte Carlo (MCMC) method are performed and described in [40, 41].

4.5 Assessment of the bridge utilizing measurement and main tensile stress criterion

The shear verifications are performed with the measurement-based concrete strength class C45/55 and under neglection of the shear reinforcement. The main tensile stresses are determined at multiple sections distributed across the web height.

The mean value of the concrete tensile strength is f_{ctm} = 3,80 N/mm² and the characteristic value is $f_{ctk;0,05}$ = 2,7 N/mm². With consideration of the partial factor for concrete strength of γ_c = 1,50 the design value f_{cd} is calculated to $f_{ctk;0,05} / \gamma_c$ = 2,70 N/mm² / 1,50 = 1,80 N/mm².

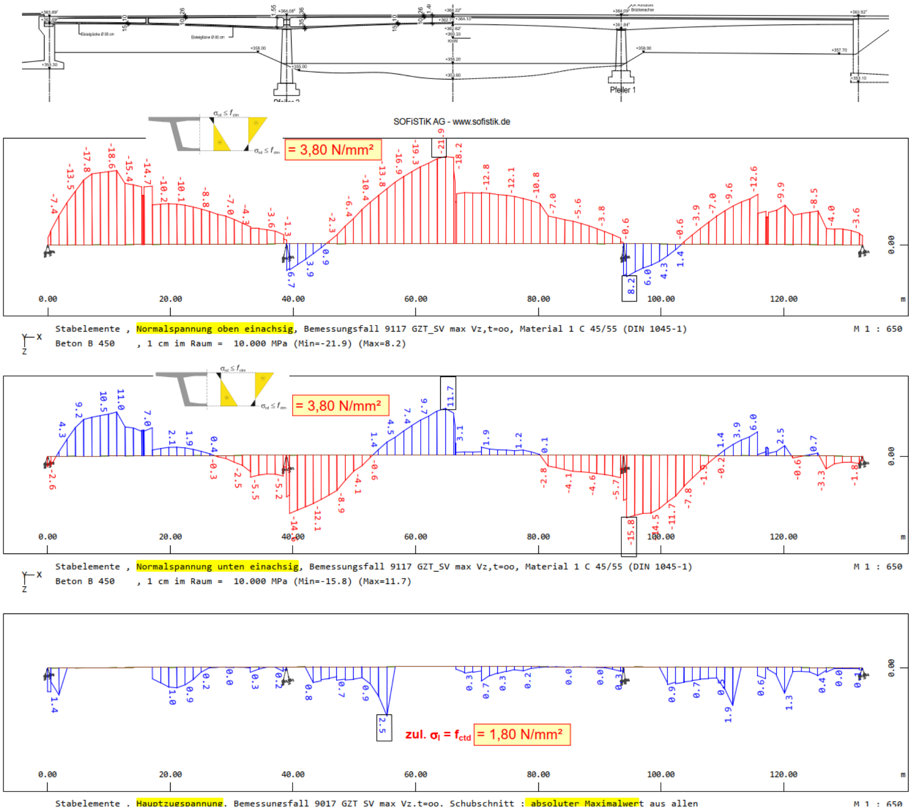

Figure 7: Resulting stresses in the superstructure, calculated by SSF Engineers using *SOFISTIK* Software. Top: Normal stress on top, Middle: Normal forces on the bottom, Bottom: Maximum main tensile stresses

Overall, the verifications are successfully completed with the main tensile stress criterion; a small number of exceeding values can be seen at the bottom of figure 7. Areas verified via the main tensile stress criterion must be inspected for crack-free condition at least every three years through a special inspection acc. to DIN 1076. Particular focus must be placed on the presence of inclined cracks or the development of new inclined cracks.

However, there are defined boundary conditions for the verification with the main tensile stress criterion required in the guideline [14] and permitted edge stresses in the ULS are a fundamental application criterion. The verification is only allowed in areas a) where only longitudinal compressive stresses occur in the ULS, b) where a flange is located within the tension zone or at the less compressed edge of the cross-section, provided that $\sigma_{cd} \leq f_{ctm}$ (see top and middle of figure 7) or c) without a flange located within the tension zone or where no flange is present at the less compressed edge of the cross-section, provided that $\sigma_{cd} \leq f_{ctd}$.

The results in the present case study do not meet the relevant requirement b), see figure 7 top and middle. The verifications therefore cannot be successfully completed at all locations. This is primarily due to the conservative boundary conditions defined in the standard.

5 CONCLUSIONS

In the present case study of an existing prestressed concrete bridge, the verification based on the main tensile stress criterion is performed. However, the boundary conditions of the code standards could not be fulfilled, and the verification therefore cannot be successfully completed at all locations due to the conservative boundary conditions defined in the standard.

The concrete strength was identified as a relevant parameter during the assessment. While the testing of concrete compression strength is a common procedure, the measurement of the concrete tensile strength depends on environmental conditions (esp. moisture and temperature). The tensile strength of concrete has significantly greater variability than compressive strength.

Data from on-site measurement can be implemented in the statical reassessment of existing structures, potentially allowing for the consideration of actual properties and more realistic verification results. There are, however, limitations in cases of significant exceedance due to changes in usage, such as a substantial increase in traffic loads. As the use of modern verification formats appears to be of limited effectiveness considering the historical developments in the design standards, further appropriate guidelines for the reassessment of existing structures are needed and a recommendation about the use of NDT results in the reliability analysis is currently drafted as part of the research project "ZfPStatik".

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support of the German Federal Ministry for Economic Affairs and Climate Action under grant number 03TN0042.

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