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CONCRETE AND RELIABILITY OF EXISTING PRESTRESSED BRIDGE STRUCTURES

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Abstract. A large number of post-tensioned concrete bridges were built in the second half of the last century. They have often been insufficiently maintained during their lifetime (usually around 50 years). Nowadays, these structures exhibit significant deterioration, mainly due to leakages and also due to various other deficiencies such as a small concrete cover. Their load-bearing capacity needs to be verified. This paper focuses on estimating the load-bearing capacity calculation of existing post-tensioned concrete bridges. In the engineering practice, this is carried out using the partial factor method according to the currently valid standards (Czech standards ČSN and the Eurocodes), which often impose more stringent requirements than the original standards. The partial factor method then often leads to low load-bearing capacities. This study deals with the bridge for which a very low load-bearing capacity has been determined. For this reason, a comparative probabilistic analysis was performed, allowing for a better description of the uncertainties in the resistance and load effect variables. The probabilistic approach appears to be less conservative and yields a higher load-bearing capacity.

Keywords: Concrete, bridge, partial factor method, reliability, load-bearing capacity.

1. Introduction

In the second half of the last century, the post-tensioned bridges in Czechoslovakia developed intensively, largely utilising standardized structural members and systems. This was a response to the growing needs of transport and infrastructure, where the main goal was to connect regions as efficiently as possible and to provide for the efficient transport of persons and goods. The choice of this structural system was motivated by the ability to build quickly and use materials efficiently, therefore several systems of post-tensioned precast beam bridges were developed and standardised [1].

This study specifically focuses on the KA type post-tensioned precast beams. These beams were manufactured off-site under controlled conditions, which guaranteed the high quality of the concrete and prestressing elements. The method was well standardised, which enabled many bridges to be built quickly and economically. The beams were commonly manufactured in several segments to facilitate transportation.

In the present territory of the Czech Republic, a large number of bridges were built from the KA beams, and subsequently, during the operation, the deficiencies of this technology gradually became apparent:

- imperfect grouting of cable ducts and poor protection of prestressing reinforcement;
- low concrete cover of the reinforcement;
- closed hollows that are difficult to survey and can collect water (often the drainage holes of the hollows were not drilled and surveys revealed that in some cases the hollows were full of water);
- the beams were assembled on site in several segments and thus contain joints without passing-through reinforcement, thereby creating weak sections of the structure;
- in the transverse direction, joints between the individual beams allow for rotation between the individual beams and this behaviour often results in damage to waterproofing and leaking into the structure, even early after commissioning [2].

It should be pointed out that imperfect grouting is particularly dangerous as it does not protect the prestressing reinforcement against corrosion and, furthermore, creates conditions for moisture condensation that can lead to reinforcement corrosion [3].

2. Bridge under investigation

The bridge under investigation was built in 1964 and carries the 2nd class road over a small water stream. The superstructure is made of nine post-tensioned precast beams KA-61/18 (Figure 1) and acts as a simply supported beam with span of 19 m. Width of the bridge is 9.18 m. At both sides of the cross-section there are 0.75 m-wide precast reinforced concrete cornices with railings. The bridge deck surfacing is made of asphalt and width between the safety barriers is 7.68 m. The one-sided transversal slope of 3 % was created by a variable thickness of concrete deck. Reliability analysis of the critical edge beam (Figure 2 and Figure 3), which exhibits the most severe corrosion weakening, is presented in detail. According to long-term experience gained by surveys of these bridges,

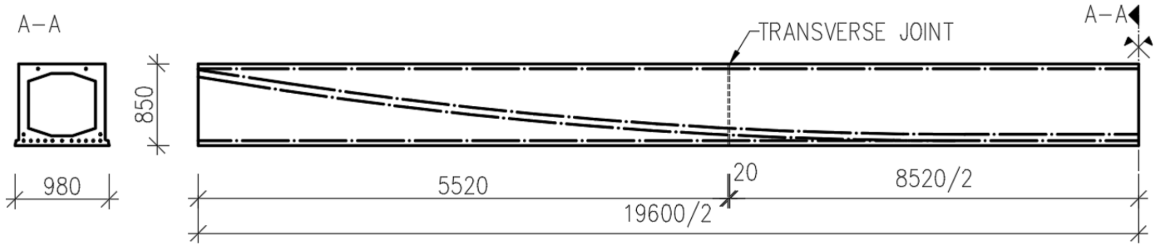

FIGURE 1. Cross-section at mid-span and longitudinal section of half of beam KA 61/18 with prestressing tendons [1].

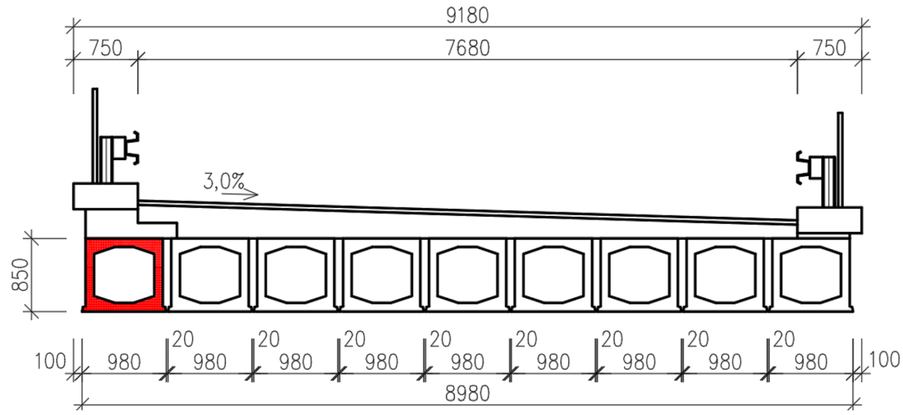

FIGURE 2. Cross-section of bridge (critical beam marked in red).

FIGURE 3. View of bridge.

edge beams generally prove to be the most exposed to corrosion weakening of prestressing tendons [4, 5].

The material characteristics of the concrete were determined by a structural survey carried out by the Klokner Institute of CTU in Prague. The characteristic value of the concrete compressive strength was determined by tests on samples as $f_{ck} = 40$ MPa and the mean value of the concrete compressive strength was $f_{cm} = 57.4$ MPa. For the application of the partial factor method, concrete strength class C 35/45 is considered, which well corresponds to the original documentation [1] (concrete class B500 according to the past classification). The prestressing tendons consist of patented wires of 4.5 mm diameter with a guaranteed ultimate strength $f_{pk} = 1\,650$ MPa and yield strength $\sigma_{0.2} = 1\,200$ MPa. According to [1], yield strength was increased by applying a prestress equal to the yield stress for two minutes. In this way yield strength was increased to $\sigma_{0.2} = 1\,350$ MPa. In reliability analysis, yield strength $f_{p0.1k} = 0.935\sigma_{0.2} = 1\,262$ MPa according to [6] was then considered. The wires and tendons approximately correspond to relaxation class 1 according to EN 1992-1-1 [7]. The prestress losses in the time of assessment – after 59 years – were determined by TDA analysis in SCIA Engineer v. 21.1 [8] with a value of 25.1 % at mid-span.

3. Partial factor method

Reliability analysis of existing structures is commonly based on the principles of ISO 1382 [9] and ČSN 73 0038 [10]. The partial factor method is widely applied when calculating the load-bearing capacity of existing bridges. ČSN 73 6222 [11] provides supplementary guidance in accordance with the Eurocodes. In the partial factor method, the basic reliability condition for the determination of the load-bearing capacity can be written using the load combination rule (6.10a, b) according to EN 1990 [12] as follows:

$$\gamma_G(G_{k0}+G_k)+\gamma_Q\psi_0\delta_{nom}V_{n,PFM} \leq R(f_{pk}/\gamma_P, f_{ck}/\gamma_C, fsk/\gamma_S, b, ...) \quad (1)$$

$$\gamma_G\xi_j(G_{k0}+G_k)+\gamma_Q\delta_{nom}V_{n,PFM} \leq R(f_{pk}/\gamma_P, f_{ck}/\gamma_C, fsk/\gamma_S, b, ...) \quad (2)$$

Maximum of the load effects in Equation 1 and Equation 2 is considered.

Based on a survey and inspection of the bridge, the actual condition of the structure is taken into account. Due to incomplete information and large uncertainties, weakening of the prestressing reinforcement area up to 25 % of the cross-sectional area is conservatively considered, tensile strengthening of the prestressing reinforcement is ignored, and weakening of the shear reinforcement up to 5 % of the cross-sectional area is taken into account.

The permanent loads are based on the nominal dimensions of the structure and densities of the materials according to EN 1991-1-1 [13]. Traffic loads for the determination of the load-bearing capacity are considered according to ČSN 73 6222 [11]. The load cases are considered so as the variable loads are always placed in the most unfavourable position as determined from influence line graphs.

The capacity in the critical cross-section is estimated considering the ultimate limit state (ULS) – bending and shear. Considering that the prestressed structure was designed using the theory of allowable stresses, it was also necessary to assess it for the serviceability limit states (SLS) – stress and crack width limitations. In the former, the normal stresses are verified for the characteristic, frequent and quasi-permanent load combinations and crack widths are verified for the characteristic load combination. Table 1 provides load-bearing capacities for shear and bending and the individual load combinations in the ultimate and serviceability limit states. Note that in this study, only normal load-bearing capacity is discussed, i.e. the vehicle on the bridge is not restricted in terms of speed and position.

Load combination	$V_{n,PFM}$ **[t]**
ULS – shear (6.10a)	41.3
ULS – shear (6.10b)	36.8
SLS – shear (characteristic combination – verification of crack width)	76.4
ULS – bending (6.10a)	**10.8**
ULS – bending (6.10b)	16.0
SLS – bending (verification of stress limitation)	11.7

TABLE 1. Load-bearing capacity for considered load combinations.

It follows from Table 1 that the critical failure mode is mid-span bending with a very low load-bearing capacity $V_{n,PEM} = 10.8$ t. For example, an unloaded Tatra 815 weighs approximately 15 t. This is why a probabilistic approach is also applied to better describe the uncertainties in the resistance of the critical section and the load effects of permanent and traffic loads.

4. Probabilistic method

The probabilistic approach relies on the determination of the reliability index β or, equivalently, probability of failure p_f. For existing structures, according to ISO 13822, tab. F.1 [9] the target reliability level is $\beta = 3.8$. The limit state function reads:

$$g(x) = \theta_R R(f_p, f_c, f_s, b, \dots) - \theta_E(G_0 + G + \delta V_{n,PM}) \quad (3)$$

The symbols and probabilistic models of the basic variables are given in Table 2.

Probability of failure was estimated by the Monte Carlo method. Note that the mean value of the bending resistance of the prestressed section is $R_m = 1\,960$ kNm and the characteristic value is $R_k = 1\,744$ kNm ($R_k/R_m = 0.89$). The coefficient of variation $V_R = 0.071$ corresponds approximately to the coefficient of variation of the prestressing force taking into account the prestressing losses.

The load-bearing capacity $V_{Vn,PM} = 22$ t is estimated iteratively to achieve a reliability index of $\beta = 3.8$. The resulting load-bearing capacity is, therefore, approximately doubled in comparison to the value determined by the partial factor method.

5. Discussion

It is assumed that the difference in load-bearing capacities can be attributed to:

- conservative value of the partial factor γ_G – for an existing bridge it is possible to measure the dimensions of structural members and thicknesses of pavement layers, specify material densities, and therefore to significantly reduce the uncertainties and subsequently update (reduce) the partial factor according to the procedure in ČSN 730038 [10],
- a conservative value of the partial factor γ_Q – in the determination of the load-bearing capacity, the uncertainties associated with the effects of vehicle loading on the bridge are lower than for the traffic flow considered in the design; therefore, the value of the partial factor 1.35 could be reduced.

6. Conclusion

Post-tensioned prestressed concrete bridges built in the second half of the last century have often been poorly maintained; nowadays their age reaches mostly cca 50 years. These structures exhibit significant degradation (mainly due to adverse long-term effects of leakages and various other deficiencies such as a

Symbol	Variable	Characteristic value X_k	X_k/μ_X	Distribution	Coefficient of variation V_X	Source
b	width of beam	0.98 m	1	-	-	[1]
h	height of beam	0.85 m	1	-	-	[1]
f_c	concrete compressive strength	43 MPa	0.75	LN0	0.115	-
f_p	prestressed force	1 327 kN	0.88	N	0.075	-
L	span length	19 m	1	-	-	[1]
γ_C	partial factor for concrete	1.50	-	-	-	[12]
γ_G	partial factor for permanent actions	1.35	-	-	-	[12]
γ_Q	partial factor for traffic load	1.35	-	-	-	[12]
γ_S	partial factor for reinforcement	1.15	-	-	-	[12]
δ	dynamic factor	1.20	1	LN0	0.05	-
G_0	self-weight load	493 kNm	1	LN0	0.04	-
G	other permanent loads	421 kNm	1	LN0	0.10	-
$V_{n,PM}$	V_n (normal load-bearing capacity)	330 kNm	1	N	0.05	[14]
θ_E	model uncertainty for action effects	1	1	LN0	0.10	[15]
θ_R	model uncertainty for resistance	1	0.98	LN0	0.06	[16]

TABLE 2. Probabilistic models of basic variables.

low concrete cover) and their load-bearing capacity needs to be verified. Focusing on an example of the representative bridge, the verification by the partial factor method reveals that the mid-span bending failure (with a load-bearing capacity of 10.8 t) is the decisive ULS.

Since this is a very low value, a probabilistic method was also applied to better describe the uncertainties in resistance and load effects. The load-bearing capacity was determined iteratively to achieve the target reliability index $\beta = 3.8$. The load-bearing capacity, 22 t, is approximately doubled in comparison to that obtained by the partial factor method. The difference between the load-bearing capacities is attributed to the conservative values of the partial factors for the permanent and variable loads. The probabilistic analysis further shows that the variability of the resistance of the critical section is mainly determined by the variability of the prestressing force including the effect of prestress losses. However, it is important to note that the determination of the corrosion weakening of prestressing reinforcement is a largely uncertain parameter, since surveys reveal conditions of prestressing steel only in a limited number of locations. The obtained results should therefore be considered as indicative only; further research will specifically focus on better characterisation of corrosion effects.

Acknowledgements

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