Nonlinear Reliability Assessment of Post-tensioned Concrete Bridge Made of I-73 Girders

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Abstract. The paper describes a comprehensive computational procedure to determine global structural resistance of the existing bridge made of I-73 precast post-tensioned concrete girders using the advanced methods of statistical assessment in combination with nonlinear finite element method analysis. A fully probabilistic approach for the determination of the structural design resistance is compared to selected code-recommended semi-probabilistic methods, such as the ECoV method according to fib Model Code 2010, the method according to EN 1992-2 or the partial safety factor method. Load-bearing capacity is determined for the ultimate as well as selected serviceability limit states. The paper shows that applicability of nonlinear modelling considering uncertainties is feasible and can be applied in a routine way. Shortcomings and advantages of all utilized design/assessment methods are discussed.

Keywords: I-73 girder · Nonlinear analysis · Design resistance · Probabilistic analysis · ECoV method · EN1992-2 method

1 Introduction

The growing volume of traffic and aging transport infrastructure emphasize the need for advanced reliability and load-bearing capacity assessment of existing bridges. Crucial aspects of these analyses are the use of advanced material models, accurate consideration of uncertainties in the input data, and performing structural analysis using a global nonlinear approach. The safety formats and rules that are usually employed in the codes are tailored for classical assessment procedures based on beam models, linear analysis and local section checks. The nonlinear analysis is by its nature always a global type of assessment, in which all structural parts interact. Therefore, the safety format suitable for design of concrete structures using nonlinear analysis requires a global approach.

Current standards offer several ways to consider uncertainties when determining design resistance using global nonlinear structural analysis. Well-recognized method is a partial safety factor method which is not suitable approach for nonlinear problems, since it works consistently only for linear tasks. The most accurate but also the most time-consuming approach is the fully probabilistic method. Here the computational burden can be reduced by employing small-sample simulation technique. Such probabilistic

analysis takes into account all uncertainties due to random variation in material properties, dimensions, loading, and others, and provides quantitative information about safety level.

The trade-off between keeping the nonlinear calculation realistic and ensuring sufficient reliability can be the use of global safety factor methods. Current standards offer several of these methods. In this paper, the global safety factor method according to Eurocode 2 (EN 1992–2 2005) and Estimate of coefficient of variation (ECoV) method according to the *fib* model code 2010 (fib Bulletins 65 & 66 2012) are used to calculate the global design resistance of the existing precast post-tensioned concrete bridge in the Czech Republic. The obtained structural resistances are compared with the resistances obtained by the partial safety factor method and fully probabilistic analysis. The methods used are discussed in terms of differences in results for selected limit states and also in terms of the required computational effort.

2 Safety Formats for Global Design Resistance

2.1 EN 1992–2 Method

According to Eurocode 2 (EN 1992 2005) a design resistance R_d is calculated as

$$R_d = \frac{r\left(\tilde{f}_{cm}, \tilde{f}_{ym}, \dots\right)}{\gamma_R}, \tag{1}$$

where material parameters in nonlinear response function r are considered by their estimated mean values, i.e.

$$\tilde{f}_{ym} = 1.1 f_{yk} \tag{2}$$

for mean yield strength and

$$\tilde{f}_{cm} = 1.1 \frac{\gamma_s}{\gamma_c} f_{ck} = 0.843 f_{ck} \tag{3}$$

for mean compressive strength of concrete. Partial safety factors of steel and concrete are $\gamma_s = 1,15$ and $\gamma_c = 1,5$, respectively. The global safety factor of resistance should be considered as $\gamma_R = 1,27$ including model uncertainties. This value corresponds to safety index $\beta = 3.8$. The resistance function r in Eq. (1) is calculated using nonlinear analysis assuming above mentioned values of material properties.

2.2 ECoV Method

This method mentioned in *fib* Model Code 2010 (fib Bulletins 65 & 66 2012) is based on idea that the structural resistance is the random variable, and its coefficient of variation V_R can be estimated from its mean R_m and characteristic values R_k, see e.g. Červenka (2013). Let's consider that structural resistance is lognormally distributed, then

$$V_R = \frac{1}{1,645} \ln\left(\frac{R_m}{R_k}\right), \tag{4}$$

where

$$R_m = r(f_{cm}, f_{ym}, \ldots), \ \ R_k = r(f_{ck}, f_{yk}, \ldots) \tag{5}$$

are the mean and characteristic values of resistance which are obtained by performing two separate nonlinear analyses using mean and characteristic values of input material parameters, respectively. Global safety factor γ_R of resistance is then estimated as

$$\gamma_R = \exp(\alpha_R \beta V_R), \tag{6}$$

where α_R is the sensitivity (weight) factor for resistance which can be considered as $\alpha_R = 0.8$, and β is the reliability index which value depends on the analysed limit state, see the application section. The design value of resistance is calculated as

$$R_d = \frac{R_m}{\gamma_R \gamma_{Rd}}, \tag{7}$$

where $\gamma_{Rd} = 1.06$ is the safety factor related to model and geometrical uncertainties. The method is general and reliability level β and distribution type can be changed if required.

2.3 Probabilistic Approach

The most advanced technique is the fully probabilistic approach, which naturally reflects the uncertainties in the input data and follows the probabilistic nature of the analysis of random systems. Input parameters are considered as random variables with their prescribed correlation structure. Random realizations of input parameters are generated by a suitable sampling method, such as the Latin hypercube sampling method, which is extremely efficient for estimating statistical moments of response using a small number of samples (McKay et al. 1979; Novák et al. 2014). With the set of random realizations of input parameters, a nonlinear calculation is repeatedly performed, and a random global resistance of the structure is obtained. Design value of the resistance is defined such as the probability of having a more unfavourable value is as follows

$$P(R \le R_d) = \Phi(-\alpha_R \beta), \tag{8}$$

where Φ is the cumulative distribution function of the standardised Normal distribution. Probabilistic studies indicate that the random distribution of the reinforced concrete structures can be described by a two-parameter lognormal distribution with the lower bound at origin. Then the design value of resistance is calculated as.

$$R_d = R_m \cdot e^{-\alpha_R \beta V_R}, \tag{9}$$

where R_m and V_R are the mean value and coefficient of variation, respectively, obtained from a stochastic simulation of the nonlinear model, both of which include the model uncertainties of resistance θ_R. Random variable θ_R is described by two-parameter lognormal distribution with unit mean and coefficient of variation 0.1.

2.4 Partial Safety Factor Method

The partial safety factor method may be used for a safe estimate of the global design resistance in absence of a more refined solution. Nevertheless, in this case the structural analysis is based on extremely low material parameters in all locations, which does not correspond to the probabilistic concept of simulation. This may cause deviations in structural response, e.g. in failure mode.

When the partial safety factor method is applied, the design resistance is calculated using the design values of input parameters in the nonlinear analysis

$$R_d = r(f_{cd}, f_{yd}, \ldots), \tag{10}$$

where $r(\cdot)$ represents the nonlinear analysis model and design values are calculated as $f_{id} = f_{ik}/\gamma_{iM}$, f_{ik} are characteristic values and γ_{iM} are partial safety factors of materials, which in the case of the ultimate limit state are considered to be 1.5 and 1.15 for concrete and steel, respectively, and for the serviceability limit states 1.0 for both materials.

3 Post-tensioned Concrete Bridge
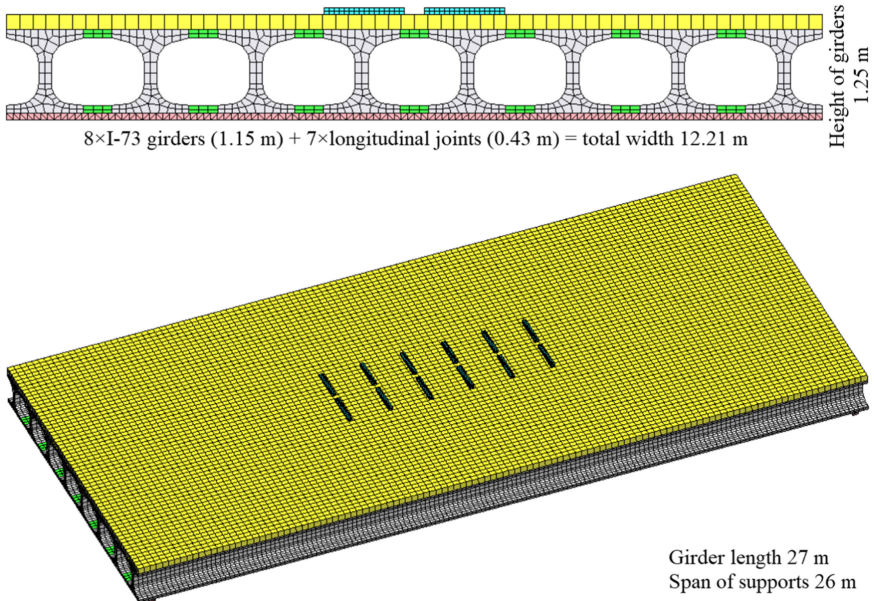
3.1 Basic Information and Computational Model

The analysed structure is the existing four field post-tensioned concrete bridge built in 1979 near Pasohlávky in the South Moravian region in the Czech Republic. The bridge length is 108 m, its width is 12.8 m. The superstructure in each of four fields is made of eight I-73 precast post-tensioned girders of lengths 27 m. Longitudinal joints between girders were made of cast concrete. Substructure system consists of concrete pillars, top transverse reinforced-concrete beams and rubber bearings (Fig. 1).

Fig. 1. Side view of the analyzed bridge.

Deterministic nonlinear computational finite element method model was developed in ATENA 3D software (Červenka et al. 2012). A single I–73 girder was created first, including 16 straight and parabolic prestressing tendons, shear reinforcement and top road layers. Created volumes are then copied eight-times and connected by the longitudinal joints made of reinforced concrete. Bridge was loaded by self-weight, prestress

forces, secondary dead load from road layers and reference 6-axle vehicle. Traffic load was increased in steps until ultimate load-bearing capacity of the bridge was reached. The nonlinear calculation was performed using a combination of Newton-Raphson and Arc-length iteration methods (Fig. 2).

8×I-73 girders (1.15 m) + 7×longitudinal joints (0.43 m) = total width 12.21 m

Height of girders 1.25 m

Girder length 27 m
Span of supports 26 m

Fig. 2. FEM computational model: transverse section (top) and spatial view with loading plates (bottom).

3.2 Material Parameters

I-73 girders and longitudinal joints were made of C50/60 and C12/15 concrete strength class, respectively; 3D Nonlinear Cementitous 2 material model was used. Each of 16 prestressing tendons was made of 20 wires of 4.5 mm diameter. Tendons and shear reinforcement were modelled using bilinear stress–strain law with and without hardening, respectively. Rubber bearings were modelled using linear elastic material. Basic material parameters and their statistics were obtained from production documentation, diagnostic survey, and JCSS Probabilistic Model Code (Joint Committee on Structural Safety 2002). Concrete compressive strength is considered to have two-parameter Lognormal distribution with coefficient of variation 0.1. Secondary concrete parameters were calculated from the compressive strength according to *fib* Model Code 2010. Pre-stress force is supposed to have coefficient of variation 0.06 and Normal distribution. Uncertainties of reinforcement parameters were neglected. Input parameters are summarized in Table 1.

Table 1. Mean values of input parameters.

Concrete		
Parameter	Concrete of girders	Concrete of joints
Compressive strength (MPa)	58.0	20.0
Tensile strength (MPa)	4.1	1.6
Fracture energy (J/m^2)	151.6	125.2
Modulus of elasticity (GPa)	37.3	27.1
Reinforcement		
Parameter	Tendons	Mild reinforcement
Modulus of elasticity (GPa)	190	200
Yield stress (MPa)	1377	477
Ultimate stress	1683	–
Prestress force (incl. 20% losses, kN)	326.9	–

3.3 Limit States

For each safety method described in Sect. 2, a global nonlinear analysis was performed with the corresponding values of the input variables. In accordance with the code recommendations, the bridge was assessed for the following three limit states:

1. Limit state of decompression (LSD) – consists of verifying that the prestressed concrete bridge is fully in compression state. In the case of LSD, a reliability index $\beta = 0$ was used.
2. Limit state of crack initiation (LSCI) – the cracks in concrete leads to a reduction in the service life of the structure due to the shortening of the initiation time of reinforcement corrosion. In the case of LSCI, a reliability index $\beta = 1.5$ was used.
3. Ultimate limit state (ULS) – it is represented by the maximum load-bearing capacity of the structure, which is accompanied by a rapid increase in deformations and a gradual collapse of the entire superstructure. In the case of ULS, a reliability index $\beta = 3.8$ was used according to EN 1990 (2002).

4 Results

From the obtained numerical responses of the structure, the design resistances for the three analysed limit states were determined by the fully probabilistic (FP), ECoV, EN1992-2, and partial safety factor (PSF) methods. Resulting values are depicted and graphically compared in Fig. 3. Note that in case of the FP method, 30 nonlinear FEM analyses were carried out to obtain a random structural response (see Fig. 3 bottom right), which was approximated by the two-parameter Lognormal distribution, Fig. 3.

Also note that EN1992–2 method is only designed for ULS with a fixed reliability index $\beta = 3.8$ and cannot be used for other two analysed serviceability limit states.

In the case of LSD, the highest value of design resistance was obtained by the FP method. The values obtained by other methods are slightly smaller, the PSF method is the most conservative. All methods led to relatively consistent results. The design resistance values obtained by the individual methods for LSCI show greater differences between the methods compared to LSD. The most conservative value was provided by the FP approach. ECoV and PSF methods somewhat overestimated the design resistance. This is due to the relatively small differences between the resistances obtained for the mean and the characteristic values, which affect the final results of both methods. In the case of ULS, the most conservative design resistance value was obtained by the EN 1992–2 and PSF methods followed by the FP method. The ECoV method resulted in significantly higher resistance compared to FP. The reason is the smaller variance obtained by ECoV compared to FP method, when the failure mode depends on the resistance of the tendons. In addition, the value of the prestressing is considered to be unchanged for all but FP method, in which the uncertainties of prestress losses are taken into account via random variable, see Sect. 3.2.
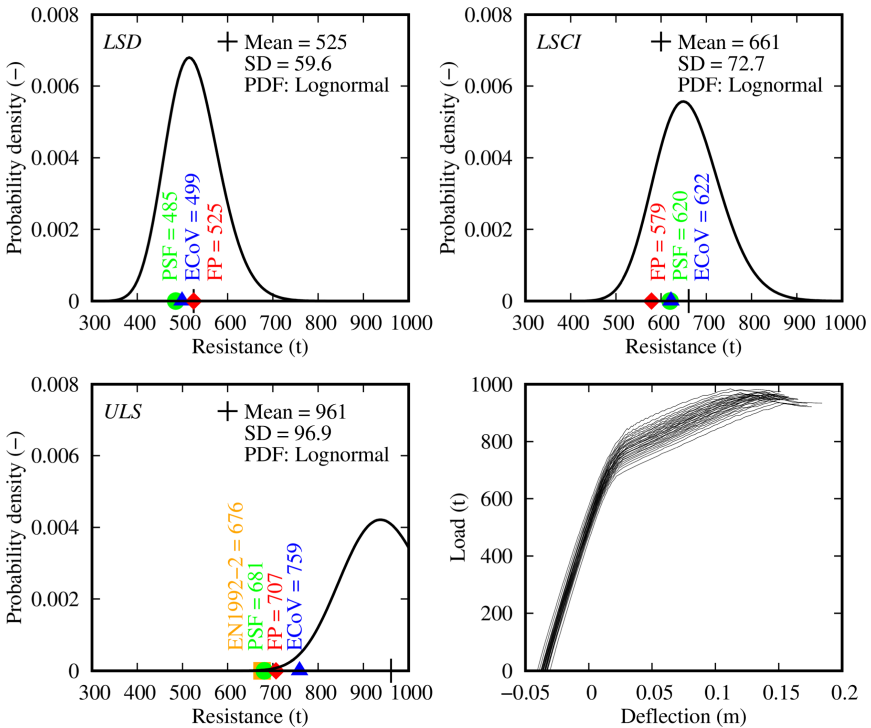
Fig. 3. Comparison of design resistance values for LSD (top left), LSCI (top right), ULS (bottom left), and 30 realizations of load–deflection diagrams from stochastic simulations (bottom right).

5   Conclusions

The paper compares results of several safety methods for assessing the design resistance of an existing post-tensioned concrete bridge. The determination of the load-bearing capacity of the structure was carried with the help of a global nonlinear analysis using finite element method. Load-bearing capacity was determined for the ultimate as well as two serviceability limit states. The paper shows that the stochastic nonlinear analysis can be routinely used to assess the design resistance of a reinforced concrete bridge.

The fully probabilistic approach can be considered the most accurate method for estimating design resistance. However, this method requires at least lower tens of simulations to obtain a good estimate of the resistance statistics. Other utilized safety methods are much more computationally efficient. ECoV methods requires 2 simulations, EN1992-2 and PSF methods only one simulation (later for each limit state category). But obtained results are not always consistent and are dependent on type of structure, critical components in terms of failure, and analysed limit state.

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