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# Assessment of the transportation route of oversize and excessive loads in relation to the load-bearing capacity of existing bridges

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**Abstract.** Transportation routes of oversize and excessive loads are currently planned in relation to ensure the transit of a vehicle through critical points on the road. Critical points are level-intersection of roads, bridges etc. This article presents a comprehensive procedure to determine a reliability and a load-bearing capacity level of the existing bridges on highways and roads using the advanced methods of reliability analysis based on simulation techniques of Monte Carlo type in combination with nonlinear finite element method analysis. The safety index is considered as a main criterion of the reliability level of the existing construction structures and the index is described in current structural design standards, e.g. ISO and Eurocode. An example of a single-span slab bridge made of precast prestressed concrete girders of the 60 year current time and its load bearing capacity is set for the ultimate limit state and serviceability limit state. The structure's design load capacity was estimated by the full probability nonlinear MKP analysis using a simulation technique Latin Hypercube Sampling (LHS). Load-bearing capacity values based on a fully probabilistic analysis are compared with the load-bearing capacity levels which were estimated by deterministic methods of a critical section of the most loaded girders.

## 1. General instructions

Transportation routes of oversize and excessive loads are currently planned in relation to ensure the transit of a vehicle through critical points on the road [1]. Critical points are level-intersection of roads, bridges etc. Bridge construction has to be designed in relation to the assumed reference period and the reliability level to resist all possible load combinations that might appear during its usage time.

In the Czech Republic load bearing capacity of new and existing road bridges is assessed according to the Czech technical standard ČSN 73 6222 [2]. Before the determination of load bearing capacity, a bridge inspection must be carried out and consequently, a real condition of the structure must be taken into account. If the load bearing capacity of concrete bridges is assessed with respect to the ultimate as well as the serviceability limit states, than the limit states of decompression (for prestressed concrete bridges) and the crack width (for prestressed and reinforced concrete bridges) should be verified.



There are three basic types of load bearing capacity which are defined by that norm. Normal load bearing capacity ( $V_n$ ) is defined by the load model LM1 according to the norm EN 1991-2 [3] and is represented by a continuous load and a two-axle vehicle. Reserved load bearing capacity ( $V_r$ ) is represented by a six-axle vehicle and exceptional load bearing capacity ( $V_e$ ) is defined by a nine-axle vehicle. Transportation of oversize and excessive loads is assessed individually for each vehicle and its corresponding load. Minimal recommended values of the load-bearing capacity of existing bridges after their reconstruction are listed in Table 1.

**Table 1.** Minimal recommended values of the load-bearing capacity of existing bridges after their reconstruction.

Group of roads according to ČSN EN 1991-2	Load bearing capacity type		
	Normal ( $V_n$ )	Reserved ( $V_r$ )	Exceptional ( $V_e$ )
1	32 t	80 t	<b>180 t</b>
2	22 t	40 t	-

Assessment of reliability of existing bridge structures can be performed by deterministic approach, which requires a design action effect  $E_d$  to be smaller than a design resistance  $R_d$  of the construction, or by stochastic approach, when calculated failure probability  $P_f$  is lower than the required failure probability  $P_{f,t}$  for the given reference period.

An equivalent term to the failure probability is the reliability index  $\beta$  that is a commonly used measure of reliability of existing bridge structures. The reliability index  $\beta$  is formally defined in terms of the probability of failure  $P_f$  as:

$$\beta = -\Phi^{-1}(P_f) \quad (1)$$

where  $-\Phi^{-1}(\cdot)$  is the inverse function of the standardized normal probability distribution.

Values of target reliability index  $\beta_t$  and corresponding probability of failure  $p_{f,t}$ , that are for selected limit states presented in Table 2, can be specified in more details depending on the estimated residual lifetime of a bridge, on the consequences of damage, or considering the economic, social and ecological consequences, e.g. Czech technical standard ČSN ISO 13822 [4] and ČSN EN 1990 [5].

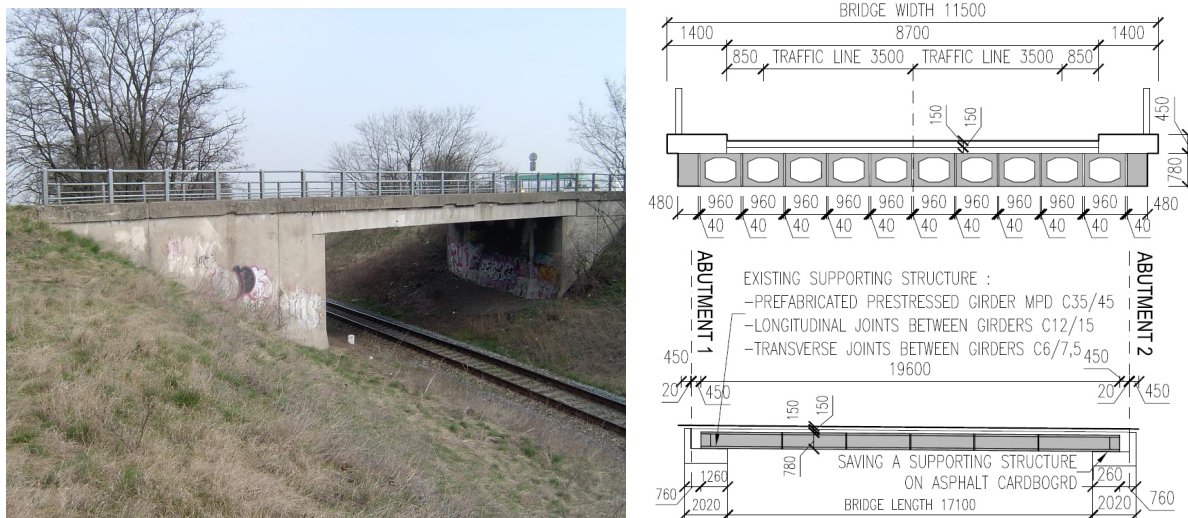
For common bridges on highways and roads class I. and II., the value of the reliability index of ultimate limit states (ULS) is  $\beta_t = 3.8$  and of the serviceability limit states (SLS) is  $\beta_t = 0.0$  (limit state of decompression and limit state of cracking).

**Table 2.** Target values of reliability index.

Limit state type	$\beta_t$	$P_{f,t}$
<i>Serviceability limit states:</i>		
- <b>reversible</b>	<b>0.0</b>	-
- irreversible - small consequence of damage	1.3	$9.6 \times 10^{-2}$
- irreversible - medium consequence of damage	1.5	$6.7 \times 10^{-2}$
- irreversible - high consequence of damage	2.3	$1.1 \times 10^{-2}$
<i>Ultimate limit states:</i>		
- very small consequence of damage	2.3	$1.1 \times 10^{-2}$
- small consequence of damage	3.1	$9.7 \times 10^{-4}$
- <b>medium consequence of damage</b>	<b>3.8</b>	<b><math>7.2 \times 10^{-5}</math></b>
- high consequence of damage	4.3	$8.5 \times 10^{-6}$

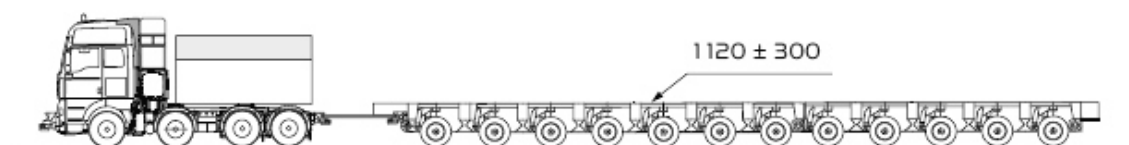
## 2. Load bearing capacity assessment of single-span post-tensioned composite bridge

A bridge with an ID 55-046 built in 1950 bridges a road class I over the railway. The bridge is considered as a single-slab span object with a prefabricated supporting structure and a monolithic substructure. Superstructure of the bridge is a simple beam consisting of 12 prefabricated and precast prestressed concrete girders MPD4 (10 intermediate) a MPD3 (2 outlying). Length of the superstructure is 20.50 m, length of the bridging is 17.50 m and width of the supporting structure is 11,0 m. Detailed diagnostic survey of the superstructure didn't show any damages of girders MPD4 a MPD3 as a result of exceeding neither the ultimate limit state nor the serviceability limit state. Corrosion of the reinforcement and prestressed tendons wasn't recorded either.



**Figure 1.** Side view of analyzed bridge, transversal section and longitudinal section of the bridge.

Bridge girders MPD3 and MPD4 were designed for continuous load of pavements of intensity  $0.6 \text{ t/m}^2$  in combination with a crawler vehicle having the weight of 60 tons according to the provisional directive for the design of bridges (year 1945). Load bearing capacity of superstructure was determined for the bridge before its reconstruction. Oversize vehicle was represented as twelve-axle THP type having the width of 3.0 m (see Figure 2).



**Figure 2.** Twelve-axle THP type vehicle.

### 2.1. Full-probabilistic analysis

Probabilistic analysis of resistance and action can be performed by numerical method of Monte Carlo-type of sampling, such as LHS sampling method. Results of this analysis provide random parameters of resistance and actions, such as mean, standard deviation, etc. and the type of distribution function for resistance. Load-bearing capacity function  $S$  is estimated according to the aggregate statistics of random simulations of structure response with a vector of random variables

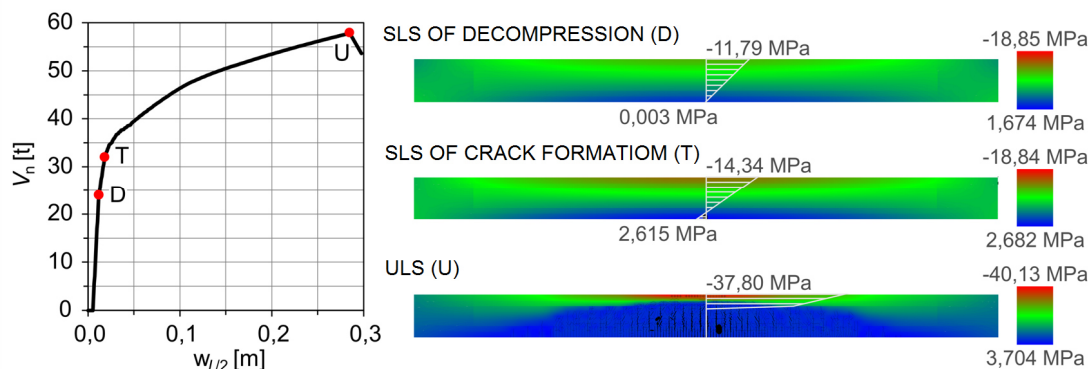
$$S = s(f_c, f_{ct}, E_c, f_{s,y}, f_{s,p}, \theta_R, \dots, g_0, g_1, V_i, \theta_E) \quad (2)$$

Material parameters, permanent loads  $g_0, g_l$  and model uncertainties of resistance  $\theta_R$  and uncertainties of model load effects  $\theta_E$  are considered as random variables. Random traffic load  $V_i$  is assumed to be deterministic. Design value of the load-bearing capacity  $V_{i,d}$  is estimated as

$$P(S \leq V_{i,d}) = \Phi(-\beta_t) \quad (3)$$

**2.1.1. Numerical model.** By assumption of forming ideally rigid slab from individual girders MPD3 a MPD4 thanks to their transverse prestressing, plain FEM model was created using the computer program ATENA 2D [6]. Concrete parts of each girder were modelled as a material model 3D Non Linear Cementitious 2, prestressed reinforcement was modelled as a discrete member and soft reinforcement of the girder individual segments was modelled as smeared. In both cases the reinforcement material was modelled by bi-linear working diagram with hardening. Simple beam placing on abutments was assumed. Structure model was loaded by its dead load, transverse prestressing effects and other permanent loads. Loading scheme for assessment of normal load-bearing capacity was represented by a continuous load and a two-axle vehicle. Loading schema for assessment of reserved load bearing capacity was represented by six-axle vehicle and in case of the exceptional load bearing capacity, the model of twelve-axle THP type vehicle was assumed.

Loading was simulated by increasing of deformation or by loading force until a limit state was observed which corresponds to the loading level at limit state of decompression (D), the limit state of crack formation (T) and the limit load-bearing capacity (U), see Fig. 2.



**Figure 3:** Diagram of normal load-bearing capacity – deflection; direct stress at the supporting structure.

**2.1.2. Stochastic modelling of basic variables.** Stochastic parameters of basic variables were defined using software FReET [7] according to Joint Committee on Structural Safety [8], including model uncertainties [9], and these were updated based on the material properties of concrete and reinforcement obtained from diagnostic survey.

32 random simulations were generated using stratified Latin Hypercube Sampling (LHS) method, which is capable to cover space of random variables in terms of relatively small number of sample [10]. As random variables material parameters of concrete as well as parameters of reinforcement, prestressed tendons and value of secondary dead load were chosen. The self-weight of the structure a prestressed force was also randomized using concrete mass density. Definitions of random input variables by their probability density functions (PDF), mean values and coefficients of variation are summarized in Table 3. The statistical correlation between material parameters of concrete and reinforcement was also considered and imposed with respect to formerly performed tests and JCSS recommendations using simulated annealing approach [11]. Finally, traffic load for determination of load bearing capacity was defined using deterministic value of load according to valid loading schemes introduced in current Standards.

**Table 3.** Definition of input parameters for assessment of load bearing capacity.

Value		Unit	PDF	Mean	$V_x$
Parameters of concrete girder MPD					
Young's modulus	$E_c$	[GPa]	Lognormal 2 par.	37.2	0.10
Tensile strength	$f_{ct}$	[MPa]	Lognormal 2 par.	3.30	0.15
Compressive strength	$f_c$	[MPa]	Lognormal 2 par.	43.4	0.08
Fracture energy	$G_f$	[N/m]	Lognormal 2 par.	$8.25 \times 10^{-5}$	0.15
Mass density	$\gamma_c$	[kN/m <sup>3</sup> ]	Normal	23.80	0.03
Parameters of concrete transverse joints between girders					
Young's modulus	$E_c$	[GPa]	Lognormal 2 par.	34.0	0.15
Tensile strength	$f_{ct}$	[MPa]	Lognormal 2 par.	2.81	0.35
Compressive strength	$f_c$	[MPa]	Triang.	19.1	<29.8;8.5>
Fracture energy	$G_f$	[N/m]	Lognormal 2 par.	$4.78 \times 10^{-5}$	0.25
Mass density	$\gamma_c$	[kN/m <sup>3</sup> ]	Normal	23.80	0.08
Parameters of prestressing tendons					
Young's modulus	$E_p$	[GPa]	Normal	190.0	0.03
Ultimate strain	$\varepsilon_{p,lim}$	[-]	Normal	0.05	0.07
Yield strength	$f_{p,y}$	[MPa]	Normal	1248.0	0.03
Ultimate strength	$f_{p,u}$	[MPa]	Normal	1716.0	0.03
Parameters of reinforcement					
Young's modulus	$E_s$	[GPa]	Normal	200.0	0.07
Ultimate strain	$\varepsilon_{s,lim}$	[-]	Normal	0.05	0.07
Yield strength	$f_{s,y}$	[MPa]	Lognormal 2 par.	462.1	0.07
Ultimate strength	$f_{s,u}$	[MPa]	Lognormal 2 par.	581.4	0.07
Load and prestressing force					
Sec. dead load	$g_l$	[kN/m <sup>2</sup> ]	Normal	6.600	0.05
Load model	$V_e$	[tuny]	Det.	180	-
Force in time t = 60 y.	$P_t$	[MN]	Normal	$P_{m,t}$	0.09
Model uncertainties					
resistance R	$\theta_{R,M}$	[-]	Lognormal 2 par.	1.00	0.10
load E	$\theta_{E,M}$	[-]	Lognormal 2 par.	1.00	0.10

**Table 4.** Final values of normal, reserved and exceptional load bearing capacity.

Limit state	Load bearing capacity by full-probabilistic method			Load bearing capacity by deterministic method			Load bearing capacity from bridge documentation (2012)		
	$V_n$ [tons]	$V_r$ [tons]	$V_e$ [tons]	$V_n$ [tons]	$V_r$ [tons]	$V_e$ [tons]	$V_n$ [tons]	$V_r$ [tons]	$V_e$ [tons]
decompression	25	80	172	21	71	163	25	48	-
crack formation	32	103	220	23	79	181			
ultimate	38	129	298	25	85	197			

2.1.3. *Probability analysis of load bearing capacity of superstructure.* Load-bearing capacity of the superstructure was estimated considering the reliability index  $\beta_t$  according to the Table 2. Final values of normal, reserved and exceptional load bearing capacity determined by the full-probabilistic analysis are summarized in the Table 4 and they are compared with values of load bearing capacity based on deterministic assessment of the critical section of the most stressed girder MPD3 and MPD4, as well as with load bearing capacity values listed in the bridge documentation from the year 2012.

Recommended values of maximum axle load for passage of the twelve-axle THP type vehicle was specified to 14 tons for the limit state of decompression and for the ultimate limit state, the pressure on a single axle was set to 25 tons.

### 3. Conclusions

In conclusion, it has been proved that probabilistic methods in combination with nonlinear FEM analysis represent an effective and practical tool in cases of evaluation of load bearing capacity and reliability of existing structures. It has been demonstrated that probabilistic approach is less conservative and leads to slightly higher capacities than the deterministic one, which is mostly applied using current Standards. However, for more detailed assessment of structures, more information is necessary to reduce model uncertainties and reach the most exact results therefore information from detailed diagnostic surveys should be used to improve the computational models.

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