Determining load-carrying capacity and remaining lifetime of bridge in Vitanova, Slovakia

P Koteš, J Vičan, J Prokop, M Strieška

1 Department of Structures and Bridges, Civil Engineering Faculty, University of Žilina

Abstract. The paper is focused on diagnostic and calculation of load-carrying capacity of bridge object in village Vitanova in Slovakia. The bridge is on the road II/520 connecting town Tvrdošín and village Suchá Hora near border with Poland and bypass the Oravica River. The bridge object was built in 1957, so, it was 60 years old bridge in time of calculation (2017). It is reinforced concrete slab bridge of two single independent spans. In 2016, the Department of Structures and Bridges, the Faculty of Civil Engineering, University of Žilina, was asked to carry out the construction and technical survey and diagnostic of above mentioned bridge. The visual inspection, diagnostic, verifying real dimensions and material characteristics were requested. In 2017, the calculation of load-carrying capacity was done. For determining the load-carrying capacity, the standard approach given in Eurocodes was used. As an alternative, the modified (lower) reliability levels and their adequate partial safety factors according to Eurocodes were used. Moreover, the real traffic load was used for calculation of reliability index \( \beta \) and the failure probability \( P_f \).

1. Introduction
The bridge objects no. 520-060 is on a road II/520 in km 76.669 and bypass the communication over Oravica River in village Vitanova, north-west part in Slovakia. The road of II. class connects town Tvrdošín and village Suchá Hora near border with Poland, so the road has a big local significance for cross-border economy and cooperation. The bridge object was built in 1957. It is not known precisely when, but reconstruction was carried out by the concreting of the bottom parts of pier and abutments what replace the stone facing.

In 2016, the Department of Structures and Bridges, the Faculty of Civil Engineering, University of Žilina, was asked to carry out the construction and technical survey and diagnostic of above mentioned bridge. The visual inspection, diagnostic, verifying real dimensions and material characteristics and calculation of load-carrying capacity were requested. The calculation of load-carrying capacity was done in 2017 in a diploma work [1]. The administrator of the road including the bridge is Žilina self-governing region.

2. Description of bridge structure
The width of pavement on the bridge is 8.0 m of thickness about 100 mm with double sided inclines. On both sides of communication, there are paths for pedestrians of width only 500 mm on pavement cornices. The bridge handrails are on the edges of bridge.

The superstructure is made as reinforced concrete slab bridge with two simply supported spans, not as continuous slab (the slab is dilated over pier). Height of the slab is 550 mm and over slab is balancing.
concrete layer of thickness about 0-100 mm due to transverse slope of pavement. Width of the slab is 9.36 m. The crossing angle of the bridge with Oravica River is about 55°. The theoretical spans are 10.78 + 10.64 m. Total length of superstructure is equal to 22.93 m and of the whole bridge is 29.89 m. The free depth of the bridge over base of river is 3.13 m in span 1 and 3.28 m in span 2 (Fig. 1).

Figure 1. View of the bridge over the Oravica River in Vitanová

The substructure consists of two gravity abutments connected with straight abutment walls and one pier in the middle of bridge structure. The width of abutments is the same as slab. The width of pier is bigger due to connected ice barrier from inlet part.

In the frame of diagnostic survey, we mainly focused on verifying the actual state (behaviour) and developing a proposal to eliminate the defects and improve their technical condition in order to increase the load-carrying capacity and serviceability.

3. Current state of bridge and results of diagnostic

As a part of diagnostic survey, the real state of bridge structure was visually identified and all its basic geometric parameters and material characteristics were measured so that the documentation of existing state could be made. Hereby, non-destructive tests (NDT) of concrete, the depth of carbonization, content of chloride ions and identification of type of used reinforcement were made. The concrete reinforced slab is from concrete of grades C12/15 according to NDT using Schmidt hammer. The reinforcement Roxor \( \phi 28 \) (10 512) 8pc. per meter (8 \( \phi 28 / \text{m} \) - axis distance is 125 mm) at upper and bottom surfaces were identified as main longitudinal reinforcement at damaged parts of the slab. The reinforcement Roxor \( \phi 12 \) (10 512) 3pc. per meter (3 \( \phi 12 / \text{m} \) - axis distance is 350 mm) was identified as transverse reinforcement. The concrete cover is in range 25-30 mm.

The following basic shortcomings were found:

- Superstructure:
  - The superstructure was markedly leaked in and the chloride curtain were visible.
  - The slab was local and at edges defected and the concrete cover was dropped out.
  - The carbonization depth was 15 mm from bottom part and 25 mm from side part.
  - The transverse cracks and longitudinal cracks due to reinforcement corrosion of maximal width 0.1 mm were found.

- Substructure:
  - The concrete bridge caps of abutments and pier were weathered.
  - The straight abutment walls and outer parts of pier were leaked in and degraded and some parts were dropped out.

- Bridge equipment:
• The reinforced concrete cornices on both sides and also curbs were greatly degraded, the concrete cover was dropped out.
• The bridge handrail was in good state, just moderate damaged (screwed up).
• The asphalt pavement was also in good state without cracks – the pavement was a few years changed and repaired.

From the description of failures follows that there was not adequate maintenance on the bridge structure. There was just repaired the asphalt pavement due to reconstruction of road.

4. Calculation of load-carrying capacity

4.1. The first approach – standard approach

The static calculation was done in the diploma work [1]. In the first approach, the technical provision TP 02/2016 [2] was used. The calculation according that TP 02/2016 means to take into account a standard method of calculation with the partial safety factors recommended as for newly designed bridges.

According to Amendment A1 of Annex A2 [3] of Standard [4], two values for the partial safety factors for permanent loads of the structural and non-structural members of the bridges are recommended, depending on whether they are factory-made products (\( \gamma_G = 1.25 \)), or they are cast-in-place products (\( \gamma_G = 1.35 \)) due to possible different variations in the weights of the structural elements. The theoretical model in computer was created in program Scia Engineer, which is based on the finite element method (FEM). The superstructure structure was modeled as a slab of constant thickness 550 mm. The longitudinal slope was neglected due to the small values. The supports of bridge slab were modeled by the line supports, the hinge support was modeled on the one side of the slab and the other was longitudinally sliding (Fig. 2). For better results, the size of finite elements was limited to max. 300 mm.

![Figure 2. Numerical model of the bridge](image)

The actions of the bridge structure were considered according to codes [5, 6]. The dead load was calculated by program and other permanent loads were calculated using real dimensions, which were measured during diagnostic. The load models LM1, LM2, LM3-900/150, LM3-3000/240 and uniform load on pavement were considered as variable loads.

From the calculation, taking into account the moment and shear resistance according to codes [7, 8], follows that the load-carrying capacities of the bridge structure are equal to:

• The normal load-carrying capacity: \( W_n = 46.2 \text{ kN} \) (\( V_n = 4.62 \text{ tones} \)).
• The exclusive load-carrying capacity: \( W_r = 185.6 \text{ kN} \) (\( V_r = 18.56 \text{ tones} \)).
• The exceptional load-carrying capacity: \( W_e = 606.8 \text{ kN} \) (\( V_e = 60.68 \text{ tones} \)).
• The load on one axle: \( W_j = 196.8 \text{ kN} \) (\( V_j = 19.68 \text{ tones} \)).

As it can be seen, the load-carrying capacities are lower than required values. It is important to emphasize that the standard approach does not take into account the real age of the bridge object (in that case, it is equal to 60 years in time of verifying) and verifies the structure for whole design lifetime given in code (it is equal to \( T_d = 100 \text{ years} \) for bridge structures) starting at the time of verification. It means that the remaining lifetime of the bridge object (\( t_r = 40 \text{ years} \)) or shorter remaining lifetime (lower than 40 years) are not considered.
4.2. *The second approach – taking into account the modified reliability levels*

The actions of bridges according to the code [6] are defined by a representative (e.g. characteristic) value and the partial safety factors for loads. This standard also defines load combinations. However, these partial safety factors can be used only for the design of new bridge structure for the whole design lifetime $T_d = 100$ years, but not for the evaluation of existing bridge structure for their remaining lifetime $t$. The modified reliability levels given by the reliability index $\beta$ or by the probability of failure $P_f$ and the corresponding partial safety factors of material properties and load effects were determined in the frame of the research activities of the Department of Structures and Bridges of University of Žilina [9, 10, 11]. The recommended modified reliability levels for the planned remaining lifetime of the bridge structures in the simplified form are shown in Tab. 1 [12].

| Table 1. Modified values of reliability index $\beta$ and the probability of failure $P_f$ for remaining lifetime |
|----------------------------------------------------------|
| Planned remaining lifetime $t_r$ [years] | Inspection in time – age of the bridge structure/member |
| | 40 years | 60 years | 70 years |
| $\beta_h$ | $P_f$ | $\beta_h$ | $P_f$ | $\beta_h$ | $P_f$ |
| < 2 | 2.767 | 2.830·10^{-3} | 2.632 | 4.248·10^{-4} | 2.578 | 4.975·10^{-3} |
| 2 - 10 | 3.209 | 6.666·10^{-4} | 3.108 | 9.439·10^{-4} | 3.066 | 1.085·10^{-3} |
| 10 - 20 | 3.354 | 3.984·10^{-4} | 3.271 | 5.362·10^{-4} | 3.236 | 6.061·10^{-4} |
| > 20 | 3.424 | 3.093·10^{-4} | 3.352 | 4.012·10^{-4} | 3.322 | 4.477·10^{-4} |

Individual loads are considered as random variables and they are described by the coefficient of variation $v$ and the bias factor $\lambda$, which expresses the ratio between the mean value and the characteristic value [13, 14]. The modified reliability levels for the evaluation of existing bridge structures affect the modified values of the partial safety factors of the load effects and material properties. The modified partial safety factors were verified using calibration and recommended values are given in [12].

Taking into account these lower partial safety factors, the new load-carrying capacities were achieved depending on age of the bridge (equal to 60 years) and on length of planned remaining lifetime. The new modified load-carrying capacities are given in Tab. 2.

| Table 2. Load-carrying capacities for remaining lifetime taking into account the modified reliability levels |
|----------------------------------------------------------|
| Planned remaining lifetime $t_r$ [years] | Load-carrying capacity for age of the bridge equal to 60 years [kN] |
| | Normal Wn | Exclusive Wr | Exceptional Wc | Load on one axle Wj |
| < 2 | 92.2 | 340.8 | 1 128.6 | 319.2 |
| 2 - 10 | 78.6 | 293.6 | 971.1 | 279.6 |
| 10 - 20 | 52.3 | 204.3 | 669.4 | 212.6 |
| > 20 | 50.9 | 198.5 | 649.8 | 208.3 |

From the results follow that the load-carrying capacities are higher if the modified reliability levels are considered, which take into account planned shorter remaining lifetime. Moreover, it is possible to see that the load-carrying capacities are increasing mutually with shortening the remaining lifetime. The increase is from 5-10% (for planned remaining lifetime higher than 20 years) to 62-100% (for planned remaining lifetime lower than 2 years).

The required normal load-carrying capacity $W_n$ according to Technical provision TP 02/2016 [2] is equal to 320 kN. The minimal required value of the bridge administrator was equal to 260 kN. So, it is still lower values than requested.
4.3. The third approach – taking into account the real traffic loads
The real traffic loads are usually far away lower than loads given in codes. It is not standard approach, but take in account the real traffic load means to do calibration of the modified partial safety factors. The theory of calibration is shown in [12, 13, 14]. In this case, the Rackwitz-Fiessler iterative method was applied, which was also used to calibrate the partial safety factors for loads and material resistance for the American and Canadian standards [13, 14].

The calibration was performed on the basis of the actual traffic load, which was determined by the traffic count in 2010 by the Slovak Road Administration (SRA). The maximal positive moments in the middle of the span, respectively, at the point of maximum positive moments, if they are not in the middle of the span, were considered as the load effects for calibration.

Calculation of load effects from individual load models was performed in programme SCIA Engineer on slab model. The real traffic loads on the bridge was simulated by passage cars for 5 standard categories of vehicles N1, N2, N3, PN3 and NS. Brief descriptions of standard vehicles are:
- Category N1 - means vehicles designed and constructed for the carriage of goods with a maximum permissible weight of not more than 3 500 kg. As a model car, a vehicle with 15.8 kN and 17.2 kN axle was used.
- Category N2 - are vehicles designed and constructed for the carriage of goods with a maximum weight of over 3 500 kg but less than 12 000 kg. The sample vehicle used for the simulation had 28 kN and 56 kN axle forces.
- Category N3 - are vehicles designed and constructed for the carriage of goods with a maximum weight of over 12 000 kg. The sample vehicle had a propelling force of 45 kN and 90 kN.
- PN3 category - the trailers of heavy trucks. The model sample had 2 axle forces of 90 kN.
- Category NS - truck semi-trailers. The sample vehicle had 3 axles of 65 kN.

This approach should take into account the actual parameters of the bridge objects as well as the possible degradation of the materials [15-19], eventually the possible fatigue failure [20, 21].

Due to the character of the reinforced concrete, the cross-section resistance was considered as random variable with normal distribution. In the partial safety factor method, for the basic load combination with one variable short-term load, the equation for the element subjected to bending is given:

\[ \gamma_{G,1} M_{GL,k} + \gamma_{G,2} M_{G2,k} + \gamma_{F,s} \delta M_{L,k} \leq M_{k} / \gamma_{M,c} \]  

where
- \( \gamma_{G,1} \) is the partial safety factor for cast-in-place products - permanent load,
- \( \gamma_{G,2} \) is the partial safety factor for factory-made products - permanent load,
- \( \gamma_{F,s} \) is the partial safety factor for variable loads,
- \( \gamma_{M,c} \) is the partial safety factor for reinforced concrete material,
- \( M_{GL,k} \) are the characteristic values of bending moments due to permanent loads,
- \( M_{G2,k} \) are the characteristic values of bending moments due to variable loads,
- \( M_{L,k} \) is the characteristic values of bending resistance,
- \( \delta \) is the dynamic coefficient, in the case of road bridges is equal to 1.0.

The partial safety factor for material for reinforced concrete element subjected to bending \( \gamma_{M,c} \) as a whole cross-section includes the individual partial safety factors of the concrete for compression \( \gamma_{c,c} \) and the reinforcement \( \gamma_{s} \). For this reason, the partial safety factor was obtained from the ration between the characteristic moment of resistance of the reinforced concrete cross-section \( M_{k} \) and the design moment of resistance of the same cross-section \( M_{d} \) calculated according to the standard [7]:

\[ \gamma_{M,c} \leq M_{k} / M_{d} \]  

Since the reliability index has been calculated for several possible remaining lifetimes, it was also necessary to calculate the material reliability factor \( \gamma_{M,c} \) for each of them. The resulting material reliability factors for slab bridges can be found in Tab. 3. Because the age of the bridge is going to be over 60 years in short time, the calculation was done for both cases – age of the bridge lower and equal to 60 years, and age of the bridge higher than 60 years. In order to calculate the reliability indices, it was
also necessary to determine the statistical characteristics of the load effects for bridge object separately, namely the mean value $\mu_E$ and the standard deviation $\sigma_E$. Actual traffic data on selected bridges has been provided to us by the Slovak Road Administration (SRA). The actual number of traffic loads (standard vehicles crossings) on the bridge can be found in Tab. 4. The resulting reliability index values for the bridge are shown in Tab. 5.

When comparing calculated reliability indices with modified reliability indices according to [9, 10, 11, 12], it is possible to see that they are much higher. This means that the slab bridge is suitable for a remaining lifetime of at least 20 years – the real traffic loads are markedly lower than load given in codes.

| Table 3. Partial safety factors $\gamma_{M,e}$ of materials for the bridge |
|------------------------|--------------------------|--------------------------|
| Bridge structure / Planned remaining lifetime $t_r$ [years] | age of the bridge $\leq$ 60 years $\gamma_{M,e}$ | age of the bridge $> 60$ years $\gamma_{M,e}$ |
| $> 20$ | 1.188 | 1.138 |
| $10 < t_r \leq 20$ | 1.182 | 1.132 |
| $2 < t_r \leq 10$ | 1.132 | 1.125 |
| $\leq 2$ | 1.113 | 1.063 |

| Table 4. Average daily vehicle intensity on the slab bridge [pc.] |
|------------------------|------------------------|------------------------|
| Bridge structure       | Addition section N1   | N2   | N3   | PN3 | NS |
| n. 520-060 Vitanova    | 92460                  | 312  | 85   | 31  | 3  | 66 |

| Table 5. Values of reliability indices $\beta$ for remaining lifetime of slab bridge |
|------------------------|------------------------|------------------------|
| Bridge structure       | Planned remaining lifetime $t_r$ [years] | Age of the bridge $\leq$ 60 years | Planned remaining lifetime $t_r$ [years] | Age of the bridge $> 60$ years |
| $\leq 2$ | $2 < t_r \leq 10$ | $10 < t_r \leq 20$ | $> 20$ | $\leq 2$ | $2 < t_r \leq 10$ | $10 < t_r \leq 20$ | $> 20$ |
| n. 520-060 Vitanova    | 7.097                  | 7.353 | 7.657 | 7.675 | 7.636 | 7.236 | 7.260 | 7.281 |
| Recomm. value          | 2.632                  | 3.108 | 3.271 | 3.352 | 2.578 | 3.066 | 3.236 | 3.322 |

5. Results of diagnostic and calculation of load-carrying capacity

Based on the results of the visual diagnostic survey, the calculation and evaluation of the discovered faults as well as the evaluation of their impact on the resistance of the bridge superstructure and its substructure, it was possible to state:

- According to Technical provision TP [22], the building state of the object was evaluated by classification degree VI. – very bad condition.
- The load-carrying capacity is insufficient according to codes and technical provisions and therefore it would be necessary to design the strengthening or replacement of the superstructure.
- The poor quality of concrete of the bridge superstructure does not allow to increase the load-carrying capacity to 26.0 tones required by bridge administrator.
- It is possible to use the substructure of the bridge – if the new superstructure is designed in approximately the same dimensions (better quality) and does not increase the load.

In the frame of the thesis [1], there was proposed a possible reconstruction of the bridge object by replacing the superstructure with a new one. The bridge design was based on the current version of
Eurocodes. The bridge object was designed as two span continuous structure - since it adapted to the original object, the effective lengths are not exactly the same (11.35 + 11.20 m). The new superstructure should have two spans (continuous two span slab) embedded on elastomeric bearings. The cross-section was made of class of concrete C35/45 as reinforced concrete slab with a total length of 23.93 m and 11.06 m wide, the height is from 0.60 to 0.70 m (variable transverse inclination 2.5% from the center due to drainage of the cross-section) (Fig. 3). The bottom edge of the slab is straight. Therefore, the height of the concrete blocks under the bearings would be the same.

![Figure 3. Design of new structure – cross-section](image)

6. Conclusion
Diagnostics and calculation have shown that the bridge object is in a very poor state and its reconstruction is needed. Due to the relatively low quality of the concrete of the superstructure, it would be difficult to increase the load-carrying capacity of the object by common technologies and methods, so it was recommended to replace the superstructure with a new one for partial use / modification of the substructure.

Acknowledgements
This research is supported by the Slovak Research and Development Agency under contract No. APVV-14-0772, and by Research Project No. 1/0566/15 and 1/0413/18 of Slovak Grant Agency and also by the project SK-PL-2015-0004 and DS-2016-0039 in frame of bilateral cooperation.

References
[1] Prokop J.: Reconstruction of reinforced bridge object over river Oravica in village Vitanová, Diploma work, Edis Žilina, 2005 (in Slovak)
[2] Technical provision TP 02/2016: The load-carrying capacity of bridges on roads and foot-bridges, Technical provisions, MDPaT SR, 2016 (in Slovak)
[3] STN EN 1990/A1 Eurocode: Basis of structural design. Amendment A1. Annex A2: Use for bridges, Slovak office of Standards, Metrology and Testing, 2006 (in Slovak)
[4] STN EN 1990 Eurocode: Basis of structural design. Slovak office of Standards, Metrology and Testing, 2009
[5] STN EN 1991-1-1 Eurocode 1: Actions on structures. Part 1-1: General actions. Densities, self-
weight, imposed loads for buildings. *Slovak office of Standards, Metrology and Testing*. 2007

[6] STN EN 1991-2 Eurocode 1: Actions on structures. Part 2: Traffic loads on bridges. *Slovak office of Standards, Metrology and Testing*. 2007

[7] STN EN 1992-2 Eurocode 2: Design of concrete structures. Part 2: Concrete bridges – Design and detailing rules. *Slovak office of Standards, Metrology and Testing*. 2007

[8] STN EN 1992-1-1 Eurocode 2: Design of concrete structures. Part 1-1: General rules and rules for buildings. *Slovak office of Standards, Metrology and Testing*. 2007

[9] Koteš, P.: Reliability of existing bridge structures and possibilities of its increasing. *Habilitation thesis*, Žilina, 143 p. 2012 (in Slovak)

[10] Koteš, P., Vičan, J.: Reliability-based evaluation of existing concrete bridges in Slovakia according to Eurocodes. In: *The Fourth International fib Congress 2014*, Mumbai, „Improving Performance of Concrete Structures“, Proceedings, Mumbai, India, 2014, © IMC-FIB, pp. 227-229

[11] Koteš, P., Vičan, J.: Recommended reliability levels for the evaluation of existing bridges according to eurocodes. In: *Structural Engineering International – International Association for Bridge and Structural Engineering (IABSE)*, 23 (4), 2013, pp. 411-417

[12] Koteš, P., Prokop, J., Strieška, M., Vičan, J.: Calibration of partial safety factors according to Eurocodes. 26th R-S-P Seminar 2017 *Theoretical Foundation of Civil Engineering*, Warsawa, Poland, MATEC Web of Conferences, Volume 117, art. no 00088, 2017, DOI: 10.1051/matecconf/201711700088

[13] Nowak, A.S.: Calibration of LFRD Bridge Code. In: *Journal of Structural Engineering*, 1995, pp. 1245-1251

[14] Nowak, A.S., Grouni, H.N.: Calibration of the Ontario Highway Bridge Code 1991 edition. In: *Canadian Journal of Civil Engineering*, 21, 1994, pp. 25-35

[15] Macho, M., Ryjaček, P.: The impact of the severe corrosion on the structural behavior of steel bridge members. *Advances and Trends in Engineering Sciences and Technologies - Proceedings of the International Conference on Engineering Sciences and Technologies, ESaT* 2015, 2015, pp. 123-128

[16] Ryjaček, P., Macho, M., Stančík, V., Polák, M.: The Deterioration and assessment of steel bridges. *Maintenance, Monitoring, Safety, Risk and Resilience of Bridges and Bridge Networks - Proceedings of the 8th International Conference on Bridge Maintenance, Safety and Management*, IABMAS 2016, 2016, pp. 1188-1195

[17] Hollý, I., Bílčík, J., Gajdošová, K.: Numerical modeling of reinforcement corrosion on bond behaviour. *International Multidisciplinary Scientific GeoConference Surveying Geology and Mining Ecology Management, SGEM*, 249, 2016, pp. 191-196

[18] Paulík, P., Bačuvčík, M., Ševčík, P., Janotka, I., Gajdošová, K.: Experimental evaluation of properties of 120 years old concretes at two concrete bridges in Slovakia. *International Multidisciplinary Scientific GeoConference Surveying Geology and Mining Ecology Management, SGEM*, 249, 2016, pp. 227-234

[19] Kala, V., Valeš, J.: Stochastic analysis of the lateral beam buckling of beams with initial imperfections. *Safety and Reliability of Complex Engineered Systems - Proceedings of the 25th European Safety and Reliability Conference, ESREL* 2015, 2015, pp. 2547-2552

[20] Krejša, M., Koubova, L., Flodr, J., Protivinsky, J., Nguyen, Q.T.: Probabilistic prediction of fatigue damage based on linear fracture mechanics. *Frattura ed Integrita Strutturale*, 11, Issue 39, Volume 249, 2017, pp. 143-159

[21] Krejša, M.: Probabilistic reliability assessment of steel structures exposed to fatigue. *Safety, Reliability and Risk Analysis: Beyond the Horizon - Proceedings of the European Safety and Reliability Conference, ESREL* 2013, 2013, pp. 2671-2679

[22] Technical provision TP 9b/2005: Inspection, maintenance and repairs of road communications. Part: Bridges, Technical provisions, *MDPaT SR*, 2005 (in Slovak)